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Tailings and Mine Waste ’17

Edited by

G. Ward Wilson
David C. Sego
Nicholas A. Beier
University of Alberta Geotechnical Center &
Oil Sands Tailing Research Facility (OSTRF)

University of Alberta Geotechnical Center Edmonton, Alberta, Canada

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Forward

It is a pleasure and a privilege to host **Tailings and Mine Waste 2017** among the majestic Rocky Mountains in Banff, Alberta, and to mark the 21st conference of the series. Next year, the conference returns to Colorado, where it all began in 1978, originally as the "Uranium Mill Tailings Management Conference", thanks to visionaries like Dr. John Nelson of Colorado State University. Nearly 40 years after the inaugural conference, Tailings and Mine Waste continues to provide a premium forum and meeting place for members of the mining community, researchers, engineers and scientists, regulatory groups and other interest groups concerned with environmental issues related to tailings and mine waste management. Presentations will cover an array of topics related to the engineering and management of tailings and mine waste, including case histories; the design, operation, and disposal for mine waste management; geotechnical considerations; mine waste/tailings modeling; liners, covers and barriers for waste control; acid mine drainage; reclamation and remediation of mine impacted sites; oil sands tailings; surface water and groundwater management and geochemistry; and policies, procedures and public safety.

The conference features 22 technical sessions over three days. Special to this year, the Mining Association of Canada (MAC) will hold a workshop on "Updates to the Tailings Management Component of the *Towards Sustainable Mining*® (TSM) Program". Dr. Michael Davies (Senior Advisor, Tailings & Mine Waste, Teck Resources, and Chair of MAC's Tailings Working Group) and Charles Dumaresq (Vice President, Science and Environment Management, Mining Association of Canada) will provide an overview of MAC's TSM program detailing revisions to the TSM *Tailings Management Protocol*, which describes performance measurement indicators for tailings management, and the third edition of *A Guide to the Management of Tailings Facilities*.

We want to personally thank the University of Alberta Geotechnical Centre and Dr. Norbert Morgenstern for their encouragement and support for the conference. The conference would not have been possible without the dedication of Annette Busenius, Vivian Giang, Elena Zabolotnii and especially Sally Petaske who provided so much assistance and leadership throughout the planning and execution of the conference.

A successful conference is only possible due to the presentations and the quality of the research presented in the manuscripts contained in the proceedings. The manuscripts become a lasting archival record and a snapshot of the state of knowledge in 2017. We want to thank our professional colleagues who willingly shared their experiences and insight with us. To all the authors, thank you for contributing your technical knowledge and for your efforts in submitting your manuscripts, especially in these extremely busy days when time is our most precious commodity. The proceedings contain information representing hundreds of years of collective experience. We know you will find insight and answers that will assist you in a better understanding of tailings and mine waste management.

David C. Sego, Nicholas A. Beier & G. Ward Wilson
Co-Chairs, Tailings and Mine Waste 2017 Organizing Committee

ORGANIZATION

The Tailings and Mine Waste 2017 Conference was organized by the Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Alberta, Canada in conjunction with Colorado State University, Fort Collins, Colorado, and the University of British Columbia, Vancouver, British Columbia.

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Catastrophic Tailings Dam Failures – Path Forward

Henry Brehaut
Global Sustainability Services Inc, Toronto, ON, Canada

ABSTRACT: Recent dam failures have led to the identification of incremental improvements in the design, management and regulation of tailings dams that should be adopted on a global basis to reduce the likelihood and consequences of future catastrophic failures. The primary purpose of this paper is to identify further improvements that must also be implemented in order to achieve a significant reduction in the frequency of dam failures in general and the elimination of catastrophic failures in particular. While the initial the focus of this paper will be on what mining companies need to know and do, it will also examine the roles and responsibilities of other key players, including consultants, regulators, industry associations and standard setting bodies. All players need to up their game and the latest round of incremental improvements should be looked at as only the first step of a committed process leading to the highest standards of tailings dam design and management.

1 INTRODUCTION

The future success of the mining industry is dependent on its ability to gain the trust of a wide range of stakeholders in order to obtain its social license to develop and operate mines. While tailings management is only one of many major issues that must be addressed, recent history has shown that the mining industry has yet to achieve the level of performance required to eliminate catastrophic dam failures. However, there are companies, consultants and regulators around the world that have a high level of commitment to tailings dam safety and have developed practices that have been instrumental in ensuring the safe operation of tailings dams by many companies in many countries. The challenge is to use this experience as the starting point in the development of standards, practices and guidelines that must be used by all companies to establish higher levels of performance, by governments for effective oversight of the industry and to provide the basis for a high level of public trust.

While the initial focus of this paper will be on what mining companies need to know and do to achieve a significantly higher level of performance, it will also examine the roles and responsibilities of other key players, including consultants, regulators, industry associations and standard setting bodies. All players need to up their game and the latest round of incremental improvements should be looked at as only the first step of a committed process leading to the highest level of performance possible.

2 PAPER OUTLINE

This paper will focus primarily on the design and operating stages of the mining cycle. It is at the design stage when the highest consequences of failure need to be defined in relation to the proposed deposition method and site location. It is during the operating stage where mining companies must strive for the lowest likelihood of failure. Closure plans and risks will only be discussed in sections of this paper pertaining to deposition method and site location alternatives.

The author will offer his perspectives based on experience gained working at the executive and board level of mining companies. This paper will not address matters pertaining to the geotechnical aspects of tailings dam design but will offer comments regarding the important roles of geotechnical engineers in support of corporate governance systems and government oversight.

Sections 3&4 will comment on some broader issues relating to the discussions to follow, including the definition of a framework to guide the examination of the interrelated nature of the activities that companies, governments and consultants must each perform in meeting their respective responsibilities. Sections 5-7 will examine the roles and responsibilities of each group with the objective of defining leading practices in each area. This will be followed in Section 8 by an overview of the contribution of other associations to the improved performance of their members. Section 9 will address two major issues, acceptable risk and corporate commitment, followed by Section 10 that will present ideas as to the path forward.

3 INITIAL OBSERVATIONS

The topics in this section of the paper are presented to provide a broader context for certain subjects discussed in this paper. The reader may or may not agree with what is stated but will, at least, better understand the basis of related statements or comments.

3.1 *Dam Safety*

For companies, engineers, and regulators, to portray, describe, or assess a tailings dam in terms of it being safe is misleading. All believe their tailings designs and facilities are safe. However, all dams are not created equal and some have failed. As stated by Freeze (2000);

> "The owners and operators of large engineering facilities want the public to hear about the great benefits to be bestowed upon it by their facility, not about how likely it is to fall down, or what the probability is that it will pollute the environment".

All communications, research, design and operating controls must be clearly focused on the likelihood and consequences of failure and the risk management strategies that are being or should be used to protect its employees, the public and the environment. The word safety will not be used in this paper other than when referring to the safety of employees, the public or the environment

3.2 *Prime responsibilities*

A complex relationship exists between mining companies, regulators and consultants with regard to ensuring that the highest standards of tailings dam management are identified and implemented. For the highest level of risk reduction to be achieved, the primary role of each group must be clearly defined primarily to avoid confusion as to what each group must do, but also to identify gaps that must be filled.

3.2.1 *Mining companies*

Mining companies must accept full responsibility for the location, design, construction, operation, decommissioning and closure of tailings facilities in a manner that ensures its risk management strategies provide an acceptable level of protection for the safety, health, and welfare of the public and the environment. They should discharge this responsibility and express their commitment to the highest standards of risk mitigation through the adoption of strong policy statements, the establishment of a comprehensive governance framework and the implementation of comprehensive assurance activities.

3.2.2 *Governments*

Governments must be responsible for the protection of employees, the public and the environment from undue impacts and risks arising out of or in connection with mining operations. They must discharge this responsibility primarily through the development of laws, regulations and guidances, by granting permits on the basis of a strong regulatory framework and public consultation and being responsible for an effective compliance and enforcement regime.

3.2.3 *Professional geotechnical engineers*

Professional geotechnical engineers should be responsible for the design of tailings facilities in accordance with the highest state of practice and applicable regulations, statutes, guidelines, codes and standards while fulfilling their professional obligations that, "…hold paramount the safety, health, and welfare of the public…"(APEGBC 2014)

3.3 *Risk Exposure*

All tailings dams have a likelihood of failure. Hazards related to natural causes are of primary concern through all phases of the design and mine life cycle. Risks specifically related to the tailings deposition method, dam design and site selection are equally important and must be identified as part of comprehensive approval process that is informed by a dam break analysis and inundation study. Mitigating strategies should be identified for inclusion in the design and management system and to provide the basis for strong management control and regulatory oversight. However, it should be noted that such approval rests on assumptions related to the interpretation of guidances supporting the design process, the effectiveness of a company's governance system and the effectiveness of regulatory oversight.

While the design process will address recognized natural hazards, it must be acknowledged that additional design risks could exist because of gaps in the knowledge base supporting current technical standards leading to a hazard not being recognized. Variations in the degree of professional experience, judgement and conduct may also be factors.

Management risks can arise because of low corporate commitment, economic feasibility pressures, and insufficient resources provided to support dam design and the implementation of management systems. Regulatory risks may be introduced as part of the permit approval process and through inadequate compliance and enforcement activities.

Another way of looking at how a tailings facility can be exposed to further risks is to consider the dynamics within each major participant. The term regulatory capture has been used by the Auditor General of British Columbia to describe the situation where the regulator, created to act in the public interest, may, in certain situations, serve instead the interests of a company or the industry (BC AG 2016*)*. Using the same perspective for mining companies, economic capture could be described as the situation where short or long term economic factors are given precedence over sustainability commitments and permit obligations. For the geotechnical community, client capture could be used to describe situations where the engineer may be influenced by client pressures in the performance of their work.

3.4 *Commitment*

To meet the above responsibilities, a high degree of commitment to risk reduction and control is required by a company and all its employees, by government officials and by geotechnical consultants. In the case of a mining company, the existence of a tailings management policy means nothing in itself. Policies will vary in terms of commitment and actions taken will vary in terms of effectiveness. Recognizing the fact that some companies are more committed than others leads to two key observations. The first is that well documented governance, design and operating practices have been developed by some highly committed companies that go beyond what is generally called best practices. The second is that some companies may not have a high degree of commitment and it is these that need strong industry leadership, strong regulatory oversight and a high standard of professional ethics from the geotechnical profession.

3.5 *Quality designations*

The quality designation for an industry practice rests solely in the eye of the beholder which is most likely to be the person or organization that has developed it. Common terms are good, best and, less often, leading practice. Even the term emerging practice has recently been used to describe a practice that has been employed by some committed companies for over 20 years.

The concern with most of such designations stems from how they were developed. At the lowest level, it is a self-designation by an individual, company or an organization. Moving up the scale, a committee of experts, usually under the umbrella of a mining association, government or a professional body, is mandated to develop a set of practices or guidelines based on their collective experience. Such efforts require a consensus at the committee level and approval by the supporting organization. Because of varying degrees of commitment at both the committee and approval level, this inevitably leads to a less than aggressive pushing of the boundaries.

The next level of practice development occurs when other stakeholders are involved as it has been shown that the resulting practices lead to the inclusion of more demanding and detailed expectations such as more specific guidances and performance thresholds. The term best practice may be applicable to such situations.

Statements regarding leading practice often refer to instances where a practice exhibits a higher degree of commitment as compared to other companies, other associations or other political jurisdictions. Such definitions are essentially backward looking. While undoubtedly some aspects may represent a high degree of commitment for specific practices, many still fall short of what constitutes real leadership. Leading practice should be forward looking and defined as a practice that goes well beyond the norm, providing the highest degree of commitment and/or a significant improvement in risk reduction. In this paper, practices that the author has judged to be leading will be presented in a text box or be otherwise identified.

While an understanding as to how industry practices are developed and agreed upon is important, the real test is how they are defined. Prescriptions can range from a list of things to think about to detailed descriptions as to what is required. Without a detailed description of what each practice element requires, little guidance is provided for the both the practitioner and those that need to judge the level of commitment being applied. Again experience has shown that stakeholder involvement has helped identify the need for more detail.

The bottom line is that existing self-identified good and best practices have failed to prevent catastrophic dam failures and more progress is required. Furthermore, the more general the description of a practice, the easier it is for the less committed to claim they have adopted it.

4 STRATEGIC FOCUS

The continued occurrence of catastrophic dam failures suggests that the system attempting to prevent such occurrences is not yet fully developed. What some describe as a broken system is best described as a patchwork system with more patches to be added, gaps to be filled and overlaps to be removed.

Rather than identifying further steps that could be taken to fill existing shortcomings, the following sections of this paper will be based on a strategic approach to the improvement of tailings dam design and operation. Strategic planning is based on the premise that if you do not know where you are going, you will never get there. A strategic plan will provide the context for the definition of stretch goals and the establishment of a continuous improvement program. Waiting for the next dam failure to make further improvements is not good enough.

4.1 *Strategic Vision*

The ultimate goal should be that a future mining project should not be approved by a company's board of directors or by a government unless the proponent can demonstrate to itself, the government and the public, beyond reasonable doubt, that the proposed tailing dams can be managed in a manner that meets each party's definition of acceptable risk.

For a company, its strategic vision should be to gain the confidence of the government and the public for its tailings management plans though the demonstration of a commitment to strong policies and practices that are capable of earning their trust and meeting their definition of acceptable risk.

For a government, its strategic vision should be to contribute to meet the economic, social and environmental goals of their jurisdiction through a balanced approval process and a strong compliance program while providing for the protection of employees, the public and the environment from undue impacts and risks arising out of or in connection with mining operations.

4.2 *Tailings Responsibility Framework*

The design, construction, operation and closure of any tailings dam is carried out within a complex system requiring high levels of expertise, commitment and diligence. A mining company, which must accept ultimate responsibility, may retain professional consultants to assist them in meeting their responsibilities and may rely on the regulatory system to add rigor through government permitting and compliance responsibilities. In situations where the regulatory system lacks substance, a company must internalize these aspects within their own system.

Larger multinational companies have realized the need to reduce their reliance on external services and standards and have taken on more responsibility internally. Smaller companies, which have fewer resources, are more dependent on outside to help in the development and implementation of high standards.

Whatever the situation, a complex relationship exists between mining companies, regulators and consultants with regard to ensuring that the highest standards of tailings dam design and management are identified and implemented. Many parts have to come together within a framework that examines the interrelated nature of the activities that each must perform in meeting their respective responsibilities. For the purposes of this paper it will be called the Tailings Responsibility Framework (TRF).

In sections 5-7 to follow, the roles and responsibilities of mining companies, governments and geotechnical consultants will be examined within the context of the TRF in order to identify those leading practices that are required to significantly reduce the consequences and likelihood of catastrophic dam failures. The prime focus will be of a strategic nature. That is to develop an outline of what is needed and to provide examples of the leading practices required to get there.

5 RESPONSIBILITY FRAMEWORK – MINING COMPANIES

5.1 *Corporate governance*

Mining companies must accept full responsibility for the location, design, construction, operation, and closure of tailings facilities in a manner that ensures its risk management strategies will provide an acceptable level of protection for the safety, health, and welfare of the public and the environment. It may be a company's objective to ensure dams are designed and operated to the highest standards but it is impossible for a company to ensure that its facilities will be risk free. They should discharge this responsibility and express their commitment to the highest standards of risk mitigation through the adoption of strong policy statements and require the establishment of a comprehensive governance system supported by the implementation of comprehensive assurance activities.

Board leadership is an absolute necessity for a company to achieve high levels of performance. A company's tailings management programs must be driven from the top. Without such support, those responsible for designing and operating a tailings facility will have more difficulty in gaining acceptance and receiving adequate resources for what needs to be studied and designed and then operated to the highest standards.

5.1.1 *Materiality*

From an internal perspective materiality for a corporation is primarily defined in financial terms. For a government, materiality is largely defined in non-economic terms considering possible loss of life, environmental damage and economic loss. For the public, materiality is primarily defined in terms of personal impact with their personal safety being paramount.

The potential economic losses to a company for even a partial failure of a tailings dam include loss of profits and costs related to dam reconstruction, environmental rehabilitation, lawsuits and government fines. Economic factors alone will usually dictate that tailings dams be considered a material risk for a corporation. In addition, a company must also consider potential impacts such as the loss of human life, environmental damage and public economic loss in its materiality ranking. When all potential impacts are considered, including a company's loss of public credibility, it is hard to visualize a company not considering the design and operation of any one of their tailing dams not to be a material risk issue. A large company may be able to rationalize that the economic risk of a single dam failure may not be material but, in situations where a tailings dam could put the local population at immediate risk, it would be cavalier not to recognize its materiality to the corporation.

5.1.2 *Board governance – Sustainability committees*

Corporate law requires directors to use their skill and experience to provide oversight of the business of a company. Directors have a duty to act honestly and in good faith with a view to the best interests of the company and to exercise the care, diligence and skill that a reasonably prudent person would in comparable circumstances. Duty of care responsibilities now requires the company's directors to provide oversight of the material risks of a corporation. Oversight of tailings dam risks is typically assigned to the board committee mandated to oversee sustainability issues.

The advantages that result from having a board committee oversee material risks are, firstly, that the company accepts overall responsibility at the highest level for ensuring that strong standards and management processes are established and that they continue to be effective on an ongoing basis. The second is that the directors will bring a higher level of review to the determination of acceptable risk. As stated by KingIII 2009, in making such assessments "…the board should be expected to take account of the legitimate interests and expectations of the company's stakeholders in making decisions in the best interests of the company." A third advantage is that by having the directors review and assess the risk management strategies for new projects the chances are better that any conflicts between economic and sustainability objectives within the company are identified and addressed.

5.1.2.1 Board governance leading practice would require that the terms of reference for the sustainability committee should, at a minimum, include responsibilities to:

- Make recommendations to the board regarding the approval of corporate policies and standards for the management of material risks;
- Oversee the policies and management processes used to manage the material sustainability risks of the corporation;
- Review and assess assurance reports to verify that corporate policies and standards have, in fact, been implemented;
- Review and assess management's risk management strategies for new projects and report to the board as to their adequacy in the light of possible consequences;
- Assess the application of adequate corporate resources; and
- Review and assess verification reports pertaining to the management of material risks.

5.1.3 *Tailings Governance Policy*

Leading practice requires that a company develop a tailings governance policy to be approved by its board of directors. General practice is for the Chief Executive Officer (CEO) of a company to develop the policy and submit it to the sustainability committee for review and assessment. Leading practice would then dictate that the policy, once found to be acceptable by the sustainability committee, be referred to the board for approval. The primary purpose of a tailings management policy should be to demonstrate corporate commitment to a meaningful set of objectives and actions that would serve as the basis for the design and management of tailings facilities and the development of risk management strategies.

5.1.3.1 Corporate tailings policy leading practice would require that the corporate tailings governance policy include commitments to:

- Locate, design, construct, operate, and close tailings facilities in a manner that provides an acceptable level of protection for the safety, health, and welfare of the public and the environment;

- Implement a tailings governance framework management system based the ISO 14001 environmental management standard or equivalent;
- Utilize robust risk management systems and processes to identify and mitigate material risks;
- Implement comprehensive change management and emergency preparedness and response plans;
- Conduct an integrated tailings disposal method and site location selection process for new tailings dams based on a thorough understanding of the costs and consequences of failure of alternate methods and site locations;
- Ensure the public is adequately informed of the nature of the risks relating to both proposed and existing tailings facilities and can effectively influence, in a collaborative manner, decisions that may interest or affect them;
- Establish a comprehensive review and assurance program to verify that the commitments stated in the corporate governance policy are been met on a continuing basis and to provide the foundation for continual improvement; and
- Make the assurance protocols and reports available to the public.

5.2 *Tailings Governance Framework*

The Chief Executive Officer is appointed by a company's board of directors and is responsible for the execution of a company's strategy and policies within the limits of the CEO's delegated authority. With regard to tailings management, the CEO will be guided by corporate policies approved by the board. Corporate risk and sustainability policies will provide general guidance, but for material risks such as tailing management specific policies and standards will also be required.

To implement the requirements of the policies as they apply to tailings management, a CEO will assign responsibility for their development and implementation to key members of the company's corporate office. In a large company this may include the chief operating officer, the risk management officer, the sustainability officer and the officer assigned responsibility for the internal audit function. The assignment of responsibilities on specific aspects may be a combination of individual and team efforts. However, what will make it all work will be the demonstrated commitment of the CEO and the executive team to high standards of performance at all levels of the organization and for all activities, not just tailings management.

A common corporate practice is for a company to organize its policy requirements within a tailings governance framework with the development of a management system as the core element. Other important aspects include the application of corporate risk management policies, the establishment of organizational, design and operating standards and requirements for assurance and reporting activities.

> **Tailings Governance Framework:** Leading practice would require that the executive office of a company respond to its TSF policy requirements through the establishment of a framework that will define requirements for a management system, operating manuals, corporate standards, risk assessments and assurance activities.

5.2.1 *Tailings Management System (TMS)*

A management system provides the framework within which a company can organize the activities required by its policies and standards. An environmental management system, as defined by ISO 14001, helps organizations identify, manage, monitor and control their environmental issues in a holistic manner (ISO 14001 2015).

The Mining Association of Canada (MAC) in The Guide to the Management of Tailings Facilities (MAC Guide) describes a comprehensive tailings management system as "one that integrates technical and managerial aspects, and one that individual companies may adapt and implement under often widely ranging conditions." MAC Guide 2011 also states that:

> "The *Guide* is not a technical manual; technical guidance may be found in other publications. Nor does the *Guide* replace professional expertise or regulatory requirements. Mining companies should obtain professional and/or expert advice to be sure that each company's specific needs are

> addressed. Mining companies and tailings facility owners and operators are encouraged to adapt and extend the principles contained in this *Guide* to meet their own site, operational and community requirements, incorporating appropriate site-specific performance measures."

To a great extent, the quality of a TMS will depend on the degree of input provided by a dam's Engineer of Record (EOR) and other experts on specific issues. A mine's management system only provides the framework for the documentation of the procedures required by the experts to do the job properly. It is important to recognize that a TMS is only a system. Its value is that it identifies and forces attention on all activities required to effectively respond to a company's policy commitments. Issues and activities have to be defined; prime responsibilities have to be identified; objectives have to be agreed upon; measures have to be identified; procedures have to be established, monitoring and inspection programs have to be documented and reporting requirements established. A management system does not provide the answers. It only requires that they be answered by those fully qualified to do so.

With regard to the above, a leading practice for companies committed to high environmental standards is to require the adoption of ISO 14001 or its equivalent for all their environmental issues. This helps establish a high standard of care for such issues and creates a common performance culture within an organization. However, ISO 14001, because of its generic nature, is not mining specific and the application of the MAC Guide within an ISO 14001 system will add to the strength of a TMS. From another perspective the MAC Guide, which is based on ISO 14001, will have greater acceptance and speed of adoption if introduced within the strong performance culture establish by ISO 14001

TMS: Leading practice with regard to management systems would require the use of ISO 14001 or equivalent for all environmental issues with guidance provided by the MAC Guide for tailings management systems.

5.2.2 *Risk Management Framework*

To develop robust risk management systems and processes to identify and mitigate material risks, some mining companies have adopted ISO 31000: 2009 Risk Management - Principles and Guidelines (IS0 31000) to provide a framework for the management of all corporate risks. The introduction to the standard states:

> "…the adoption of consistent processes within a comprehensive framework can help to ensure that risk is managed effectively, efficiently and coherently across an organization. The generic approach described in this International Standard provides the principles and guidelines for managing any form of risk in a systematic, transparent and credible manner and within any scope and context." (ISO 31000 2009)

The main value of this standard is to provide a structured basis for the integration of risk management processes and the establishment of a risk management culture. With regard to ISO 31000's use by mining companies, it should be noted that this international standard is generic in nature and, for its effective application, expert advice will be required for its adaption and use.

Companies will need to establish a corporate standard for the identification, evaluation and management of TSF risks at each of their mine sites. In addition to the usual requirements of a corporate risk standard, the standard should also address the qualifications of the assessment team and require the implementation of critical control procedures.

In the position statement "Preventing catastrophic failure of tailings storage facilities" issued by the International Council on Mining & Metals (ICMM) it is stated that enhanced efforts are required to ensure that "Suitably qualified and experienced experts are involved in TSF risk identification and analysis, as well as in the development and review of effectiveness of the associated controls."(ICMM 2016) Regarding qualifications, the Rio Tinto management system standard states "Qualitative and quantitative risk analysis must be facilitated by competent personnel and include personnel with adequate knowledge and experience for the risk being evaluated." (Rio Tinto 2014)

One of the most important factors is that the lead assessor be fully qualified in the conduct such assessments. Expert knowledge in the design and operation of tailings storage facilities is not required as the lead assessor is primarily required to lead the process, not be a factor in influencing the outcomes. With regard to the assessment team, it is important that a range of perspectives and experience be represented and includes the participation of the EOR. What should not be allowed by the lead assessor is for persons to pull rank or to dominate the discussions.

ICMM also stated in its position statement that performance criteria should be "…established for risk controls and their associated monitoring, internal reporting and verification activities." ICMM further suggests that "Critical control management has been identified as an approach to managing low probability, high impact events such as catastrophic failures of tailings storage facilities." The identification of those issues that will require the highest level of attention is a necessary outcome of any risk assessment.

Risk Management Framework: Leading practice would require that a company, working within the framework of ISO 31000, establish a corporate risk management standard that would include statements regarding the qualifications of audit assessment teams and require the identification of critical risks and their controls.

5.2.3 *Operating Manual*

One way of looking at a tailings management system is that it requires the documentation of the best way to do something, to write it down and to make sure it happens all the time. In response to this need, MAC has published the Developing an Operation, Maintenance and Surveillance Manual (OMS Manual) for Tailings and Water Management Facilities (MAC OMS Manual) that describes the rationale, organization and contents for an OMS Manual. (MAC Manual 2011).

Operating Manual: Leading practice would require:

- The preparation of a site specific operating manual based on the framework and detail provided by the MAC OMS Manual;
- The completion of an operating manual prior to the commissioning of a new mine.
- Audit protocols to verify the adequacy of the manual in general and that professional and/or expert advice has been obtained in the development of critical procedures.

As noted above, issues and activities have to be defined; prime responsibilities have to be identified, objectives have to be agreed upon, objectives and measures have to be determined, procedures, monitoring and inspection programs have to be documented and reporting requirements established. The operating manual should clearly identify the high consequence risks and ensure that the highest level attention is paid to the development and documentation of critical control measures. For more on the subject of critical controls, refer to ICMM's Health and Safety Critical Control Management – Good Practice Guide (ICMM 2015), which is being used by its members in applying the same principles to TSF management.

Operating Manual: Leading practice would require:

- The identification of high consequence risks with the highest level of attention being given to the development of appropriate critical control measures and procedures; and
- The involvement, including final sign-off, of the Engineer of Record and other experts in the identification and preparation of critical control measures and procedures.

Since the primary focus of an operating manual will be on dam stability, the Engineer of Record should be an active participant in its preparation and be requested to sign-off on the identification of issues requiring critical controls and the procedures for their management. Experts in other areas such as emergency planning and response should also be retained to sign-off on related plans and procedures. As stated in the MAC OMS Manual "This guide does not replace professional expertise. Professional advice should be obtained in order to ensure that site and operational requirements are addressed and all regulatory requirements are met."

5.2.4 *Independent Tailings Board (ITB)*

Whether a mine is part of a large company or a small one, a high level of technical and operating expertize is required to support each company's Tailings Governance Framework. For large mining companies, this expertize may be internalized in varying degrees with the creation of a dedicated head office team that is independent from the mine operations and the corporate project development team. Smaller companies will probably not have the resources to internalize all the expertize required and will need to seek external advice on a wide range of matters for each of their mine sites.

One approach would be to establish an independent tailings support mechanism for each TSF. Such practices have been adopted by some companies more than 20 years ago that started with a focus on the technical aspects of tailings dam designs. Practices have evolved over the years requiring, in BC for example, where government has mandated (BC Guide 2016) that a mine manager create an Independent Tailings Review Board (ITRB) of subject matter experts that, amongst other duties:

- "Provides an independent assessment to senior mine management and regulators whether the tailings storage facility is designed, constructed and operated appropriately, safely and effectively;
- Provides the site team with practical guidance, perspective, experiences and standard/best practices from other operations; and
- Reviews and comments on the planning and design process, monitoring programs, data analysis methodology and work performed by site team and/or contract consultants."

The importance of having an independent review of this nature has been reinforced by MAC's Tailings Review Task Force (MAC TRTF 2015) that stated:

> "Independent review provides an important layer of due diligence on both the Engineer-of-Record and the owner of the facility. As such, it should be viewed as being in addition to, rather than a replacement for, external audits or assessments, and the role of the Engineer-of-Record."

With the proper terms of reference and composition, an ITB would add most value as the upper layer of an assurance program for a company. Its scope should be as broad as possible covering all aspect of the tailings governance framework. Its composition should include, as a minimum, two geotechnical experts with expertise related to the dam design being assessed and one expert with broad experience in the management of dams at a senior corporate level. Expertize in other subjects such as risk assessment and emergency preparedness could be considered but it is likely that the core members of an ITB would have the experience to identify concerns in such areas and recommend that they be addressed by management.

The ITB will have to review documents, studies, operating manuals, audits, reviews, assessments and other reports but not in a formal manner and its comments should be in the form of advice and suggestions. The ITB should not be expected to formally validate or commit to any statements regarding the "appropriate, safe and effective management" of a tailings dam. The ITB should only advise the company on improvements it should consider and its ability to make such statements. The term review should not be included in the name of the ITB.

Independent Tailings Board: Leading practice requires that an ITB be formed for each TSF comprised of recognized industry geotechnical and management experts in the design and operation of TSFs for the purpose of providing a company with an annual assessment of the effectiveness of its Tailings Governance Framework and to offer its advice and comments on key matters such as the integrity of the dam structure, the identification and management of high consequence risks, the comprehensiveness of the assurance programs and the scope, depth and team qualifications for individual assurance activities.

The value of the ITB will rest primarily on its ability to advise a company on those matters the company should undertake to improve the strength of its tailings governance framework and to advise the company on matters pertaining to their assurance program. The terms of reference for the ITB should also include a role for the board members to provide advice when needed on important matters such as the selection of geotechnical consultants, critical control measures and procedures and the terms of reference for assurance activities.

5.2.5 *Engineer of Record*

Common practice requires the appointment of an Engineer of Record (EOR) by a company for each TSF. It is also common practice for government to require the retention of an EOR by a company and to define certain roles and responsibilities expected of this position through the life-cycle of the facility.

From a corporate perspective, the EOR should have professional responsibility for the design of the TSF in a manner that is in compliance with applicable laws and regulations, is in accord with the highest standards of international practice and is capable of the meeting the risk tolerances of the client and the government. On an ongoing basis, through the life-cycle of the TSF, the EOR should be retained to provide professional design services for all modifications or changes to the original dam design. The EOR should also conduct annual validations of the integrity of the dam design and key operating parameters that go beyond the scope of the annual inspections that may be required by governments. In addition the EOR should, as appropriate, participate in or be expected to contribute to all risk assessments, critical control determinations, assurance activities and the development of the operating manual.

As an important part of a corporate assurance program, the annual validations by the EOR should focus on an evaluation the adequacy of design performance and operating procedures for the overall facility during the past year, the identification of deficiencies or opportunities for improvement and providing assurance that the current design and operating manual will continue to provide an acceptable level of protection in the coming year.

Engineer of Record: Leading practice requires the appointment of an EOR to have professional responsibility for the design of and changes to each TSF, to conduct annual validations of performance and to participate, as appropriate, in activities related to risk assessments, critical control measures and procedures, assurance activities.

5.2.6 *Assurance activities*

Assurance activities are a key component of a comprehensive tailings governance framework and are essential in ensuring that the designs and procedures adopted by a company enable it to meet the objectives of its policies and all legislative requirements. To be able to satisfy or assure a board of directors of a company, the government and the public that its tailings governance policy has been effectively implemented, a comprehensive and integrated program of audits, assessments and reviews is required.

A survey of corporate sustainability reports, corporate websites and government requirements has shown very little commonality regarding the use of such terms. The use of the term audit is generally clear although some audits rely on the use of judgement to a qualified extent while other instances an activity described as an audit is more of a review. The terms assessments and reviews, and sometimes evaluations, are used interchangeably and sometimes together. Some companies refer to formal reviews or periodic reviews, which sounds good but gives no evidence of the substance of the activity suggested.

Audits are typically described as the independent, formal, systematic and documented examination of an organization's or facility's performance with explicit, agreed, prescribed criteria. To be effective audits need detailed protocols that provide specific questions to which factual answers can be provided as proof of conformance with practice requirements.

Assessments and reviews differ from audits primarily to the extent that judgement, based on relevant levels of experience and professional qualifications, is used to evaluate the effectiveness of designs or practices in achieving desired outcomes. For the purposes of this paper, a review is defined as a formal examination of something with the objective of verifying attainment of a required performance level and, if not so, identifying the need for improvement. In this context, assessment is defined as the process of making a judgment about something. Assessment is primarily a tool to judge progress or lack thereof against an objective although possibilities for improvement can also be a valuable outcome.

While the terminology used to describe a company's assurance activities needs better clarity, the effectiveness of any one activity depends on the establishment of a clear understanding of what is expected, what the main issues are and what qualifications are needed from the audit, review or assessment team. The judgement and experience of a team must be matched with the objectives of the assurance activity. Judgement and experience levels will increase in inverse proportion to the availability of detailed protocols. Judgment and experience levels will be especially important when examining critical control procedures and practices related to high consequence risks. The quality of any given verification activity will always depend on the experience and judgement of the verifier and the quality of the verification protocol.

Assurance Activities: Leading practices requires that each assurance activity have a well-defined terms of reference that describes the scope, depth of evaluation, the appropriate judgement and experience levels required in the assessment team and the protocols or appropriate professional standards of practice to guide their work

Mining companies vary considerably in their approach to assurance activities. Larger companies have started to internalize some of their requirements for assurance through the establishment of corporate internal audit functions that are described as being independent. Such companies may also internalize technical and operating expertize for the development of corporate standards, to provide support for individual operations and to provide support for and be involved in internal assurance programs.

Smaller companies, that do not have the resources to establish a corporate internal audit function or to provide technical and operating guidance, must rely on external providers to meet their policy requirements. For example, assurances related to an ISO 14001 management system are available from professional auditors and assurances related to technical design can be provided by geotechnical engineers. MAC members benefit from the assurance protocols and providers as part of their Towards Sustainable Mining (MAC TSM 2017) program.

Technical dam reviews on a periodic basis are generally required by governments and are also essential parts of a comprehensive corporate assurance program. One feature of the Legislated Dam Safety Reviews guideline published in British Columbia (BC) (APEGBC 2014) is its recognition that the level of assurance should depend on a number of site specific circumstances such as consequence rating, dam type and use. It also requires an assessment of "the operations, maintenance and surveillance practices at the *dam* including the assessment of the overall *dam* safety management system and identification of any non-conformances;" without providing any guidance relating the conduct this part of the dam review. This just one example of the need to examine all assurance activities test for overlap and gaps as well as to the suitability of the supporting protocols and guidances.

The main challenge for all companies in the establishment of a comprehensive and effective assurance program is to ensure a high level of technical and operating expertize is available, internally or externally, to assist in the development of the scope, terms of reference and the selection of suitably qualified professionals for each assurance activity. The second challenge is to ensure that adequate assurance protocols and guidances are available to meet the objectives of the assurance activity. The third challenge is to ensure that their assurance activities also assess the quality of site-specific operating, monitoring, surveillance, maintenance and reporting procedures as described in the operating manual and audit their implementation and conformance in practice.

Assurance scope: Leading practice would require that a site's operating manual be assessed by qualified experts as to the quality of its procedures and its application be audited regarding site conformance with its requirements.

5.2.7 *Meaningful Communication and Engagement*

"Trust us" no longer works. The social licence to receive a permit to construct and operate a tailings facility now depends on a company's willingness to engage in meaningful communication with the public with the objective of gaining their trust. This view is supported by the Australian Government (AG TMH 2016), which states that;

> "A key challenge for mining companies is to earn the trust of the communities in which they operate and to gain the support and approval of stakeholders to carry out the business of mining. A 'social licence to operate' can only be earned and preserved if mining projects are planned, imple-

> mented and operated by incorporating meaningful consultation with stakeholders, in particular with the host communities. The decision-making process, including where possible the technical design process, should involve relevant interest groups, from the initial stages of project conceptualisation right through the mine's life and beyond."

Stakeholder consultation, information sharing and dialogue should occur throughout the TSF design, operation and closure phases, so viewpoints, concerns and expectations can be identified and considered. Regular, meaningful engagement between the company and affected communities is particularly important for developing trust and preventing conflict.

It should be noted that the term consultation is only one aspect of a meaningful communicating program by a company. According to The International Association for Public Participation (IAP2), community engagement consists of a spectrum of approaches described as follows;

- inform (provide information),
- consult (obtain feedback),
- involve (act on what we hear),
- collaborate (public participates in decision-making process but company makes the final decision)
- empower (public decides)

The fourth level, "collaboration", closely parallels one of MAC's "leadership" level requirements as part of its Towards Sustainable Mining initiative. In its Aboriginal and Community Outreach Protocol (MAC Outreach 2015) one of the requirements at their leadership level is that formal mechanisms are in place to ensure that the public "…can effectively participate in issues and influence decisions that may interest or affect them."

Meaningful Engagement: Leading Practice requires that the public is adequately informed of the nature of the risks relating to proposed and existing tailings facilities and can effectively participate, in a collaborative manner, in decisions that may interest or affect them.

Meaningful communication also requires that a company demonstrate its commitment by making its assessment protocols and results publically available. In addition to helping to drive internal improvement, this practice will go a long way towards earning public trust by showing the comprehensive nature of the standards of practice being used and the efforts being made to ensure that they provide ongoing protection for the public and the environment.

Meaningful Communication: Leading practice requires that a company make its assessment protocols and reports available to the public.

5.2.8 *Deposition Method and Site Selection*

In response to government and public expectations there is an increasing requirement for the assessment of alternate deposition methods for the purpose of reducing site specific risks and impacts As stated in the Australian Government Tailings Management (AG TMH 2016) publication,

> "Regulators nowadays expect all TSF design submissions to demonstrate beyond reasonable doubt that sustainable outcomes will be achieved during operations and after closure by the application of leading practice risk-based design that:
>
> - Fully assesses the risks associated with tailings storage at a particular site;
> - Compares the suitability of all available tailings storage methods, in particular those that involve tailings dewatering and/or eliminate the requirement for the damming of surplus water within the TSF;
> - Demonstrates that the tailings storage method selected will manage all risks to within acceptable levels and as low as reasonably practicable (ICOLD 2013)."

To demonstrate to government and the public beyond reasonable doubt that the proposed site selection and deposition method provides an acceptable level of risk protection, a company must fully disclose the nature of the risks and convince the government and the public that its risk management strategies and its commitment to a strong governance framework will adequately address their concerns.

This will require that the results of a dam breach and inundation study be disclosed and its risk mitigation measures be described. It will require that information be provided that supports the selection of the proposed alternative based on operating and closure requirements. Furthermore, as recommended in BC Guide 2106, "Selection indicators for large projects should be conducted in consultation with local communities, First Nations, and stakeholders in order to maintain a transparent, defensible evaluation."

There are two main benefits of a meaningful communication process. The first is that by listening to and collaborating with the public regarding their concerns, a company will have a better appreciation of the risk mitigation measures it should adopt. The second is that a company will gain the opportunity to demonstrate its commitment to high governance and risk management standards in a constructive manner and, if done right, set the basis for it to earn the social license to operate.

Deposition Method and Site Selection: Leading practice requires that an integrated tailings disposal method and site location selection process be conducted for new tailings dams that:

- Is based on a thorough understanding of the costs and consequences of alternate deposition and storage methods and their consequence ratings;
- Considers alternatives that reduce or eliminate water stored within the containment facility;
- Considers closure requirements and its associated risks;
- That demonstrates beyond reasonable doubt that risks will be managed within acceptable limits; and
- That enables the public to participate in a collaborative manner in the examination of alternate deposition methods and their related risk management strategies.

For new alternatives to be credible they must be supported by a high level of design, operating and closure expertize similar to that currently available for slurry deposition. They must also receive the highest level of corporate governance as the new technologies will present their own challenges and require greater attention to design assumptions and operating controls.

6 RESPONSIBILITY FRAMEWORK - GOVERNMENTS

Governments must be responsible for the protection of employees, the public and the environment from undue impacts and risks arising out of or in connection with mining operations. They must discharge this responsibility primarily through the development of laws, regulations and guidances, by granting permits on the basis of a strong regulatory framework and public consultation and by being responsible for an effective compliance and enforcement system. To meet their responsibilities, some governments are responding with a higher level of oversight through a more rigorous permit approval process, expanded tailings management oversight, improved compliance and enforcement activities and increased transparency.

The leading practices put forward in this section of the paper are important for two reasons. The first is that most of them are necessary to deal with mining companies that are not fully committed to the highest standards of tailings dam design and management. The second is that these practices will help to define the principles and practices that a company will have to internalize for mines in countries without a high level of capacity or willingness to regulate the industry.

6.1 *Permit Approval Process*

Governments are now putting the onus on companies to put forward alternate disposal methods as part of the permitting process, with a particular emphasis on those that reduce or eliminate water within the TSF. A supporting stipulation, as described in BC Guide 2016, requires that "a dam breach and inundation study or a run-out analysis conformant to CDA guidelines be conducted" and that a dam consequence rating be assigned to each alternative. This will provide government with the information it needs to make a balanced decision regarding the approval of a mine project and will help force companies to pay greater attention to risk reduction in their selection of the tailings disposal method and the location of the disposal area.

The alternatives submitted for the approval of a TSF should be based on very specific operating parameters and risk mitigating measures. These will provide the basis for the consequence classifications which in turn will provide the basis for regulatory review and public comment. Public engagement is a necessary part of the approval process so that it can become adequately informed and be able to express its views as to the acceptability of any given proposal or alternatives. For a permit to be granted, a government's decision to approve a particular design will depend on its determination that the dam design, with its risk mitigating strategies, will provide an acceptable level of protection for the safety, health, and welfare of the public and the environment.

Permit approval process: Leading government practice requires that a company engage in a meaningful manner with the public regarding its TSF alternatives with the objective of providing government with an understanding of the issues it must consider in approving a particular proposal or alternative.

When and if a permit is approved it must be granted with the requirement that critical design operating parameters and risk mitigating strategies are strictly adhered to. One example is the requirement, as stated in the BC Guide, to include measureable monitoring parameters that are identified and required to be maintained within predetermined limits for a tailings storage facility. This subject was also addressed in a report by the Auditor General of BC (BC AG 2016) that stated that permits should be written with enforceable language.

Permit conditions: Leading practice requires that permits and permit amendments be granted on the basis of strict conditions related to critical operating parameters and risk mitigating strategies and that they be measurable and enforceable.

6.2 *Permit Amendments*

Companies should be required to provide notification of any proposed changes to the permitted deposition method, dam design or operating conditions for government review and approval. Such notifications and supporting material should be accompanied by the results of a risk assessment that clearly identifies any consequence or likelihood changes. If there is any doubt as to the acceptability of the revised risk profile, governments should require a public review as part of its approval process. Governments, at any time, should also establish the right to compel companies to provide an independent opinion on any proposed changes based on terms of reference approved by government.

Permit amendments: Leading government practice requires that companies be directed to submit any proposed changes to permit conditions for approval and that government institute a public review process if there is any doubt as to the acceptability of the revised risk profile.

6.3 *Government Oversight*

Government oversight in general should include requirements for regulatory inspections, annual company performance reports, third party annual inspections and periodic integrity reviews. Further requirements must also be considered, either in a permit or as a general requirement of all companies, particularly if a government has concerns about the industry's commitment to or its understanding of what is needed to adequately manage its TSFs. Examples are ITBs and operating manuals. When such actions are taken it is important that specific direction be given as to what is expected and to insist that assurance be provided to verify that, as a minimum, that industry standards of practice have in fact been achieved.

The BC Guide notes that:

> "Several bodies provide guidance of how to develop a tailings management system, including:
> - The Mining Association of Canada (MAC). A Guide to the Management of Tailings Facilities, Second Edition, 2011.
> - The International Organization for Standardization (ISO 14000).
> - Governments of Australia and New Zealand."

Permit approval process: Leading government practice would require companies to put forward alternate tailings disposal methods considering the reduction or elimination of water within the disposal area and that a dam breach and inundation study be conducted in support of each alternative.

The guidance document goes on to state that tailings management system should complement a mine's environmental management system, which "…is expected to have been developed in conformance with ISO14001."

In the case of operating manuals, the BC Guide requires that "…mines develop and implement operational procedures, maintenance procedures and a surveillance and monitoring program…and be formally documented in an Operations Maintenance and Surveillance (OMS) manual." The Dam Safety Guidelines of the Canadian Dam Association and the MAC OMS Manual are referenced as providing guidance as to industry standards of practice.

The need for government to mandate the use of high management standards is not because they do not exist. It is that they have not been fully adopted by all companies. Just as governments require and rely on independent reviews and assessments for technical matters, the same holds true for managements systems. Using the operating manual as an example, once a government has mandated its development and use, it should require third party verification that it meets prescribed standards. As a further precaution it should also require the completion of the operating manual prior to commissioning of a TSF.

Government oversight: Leading practice requires that governments dictate the adoption of specified management standards and that assurance be provided that they have been implemented in keeping with prescribed standards.

6.4 *Compliance & enforcement*

As stated by the Auditor General of British Columbia in the report, An Audit of Compliance and Enforcement of the Mining Sector:

> "Enforcement is the backbone to any compliance program. It is the final line of defence against environmental degradation. According to good practice, strategies involving education, assistance, incentives, monitoring and inspections are effective only if backed by a credible threat of enforcement sanctions. To be effective, enforcement programs must involve: swift and predictable responses to violations and responses that include appropriate sanctions."

6.4.1 *Compliance & enforcement effectiveness*

Regulatory effectiveness: Leading practice would require that governments establish an integrated and coordinated regulatory approach with the objective of ensuring the effectiveness of its compliance and enforcement activities.

In response to the published BC AG 2016 report, the BC Government "…committed to establish a Mining Compliance and Enforcement Board…to oversee an integrated and coordinated regulatory approach to mining in the province of B.C." The multistakeholder Board was given the mandate to oversee the development of "…strategic improvements that enhance compliance and enforcement effectiveness through integration and coordination of planning, training, policies, procedures, tools, evaluation and public reporting for mines in British Columbia." One of its deliverables is to identify "…the necessary capacity, tools, training and expertise required to achieve goals and objectives." (BC C&E 2016)

6.4.2 *Administrative Monetary Penalties*

In 2016, British Columbia amended its Mines Act to establish key components for administrative monetary penalties (AMP), such as the authority to make findings of contravention or non-compliance and to impose AMPs. The BC Code states that a person who commits an offence is liable to a fine of not more than $1,000,000 or to imprisonment for not more than 3 years or both. It goes on to state that "If a corporation commits an offense, a director or officer of the corporation who authorized, permitted or acquiesced in the offence…" is also liable to the penalty limits stated above.

The recognition that a corporation, not just the mine manager, may be at fault is important as it relieves the mine manager from being ultimately accountable for all aspects of a TSF's operation. It recognizes the fact that the original design would have been developed under the guidance of the corporate office, that the corporation is responsible for the provision of adequate resources and that the corporate office may be complicit in any decisions that result in an increase in risk levels.

As stated in the discussion paper BC AMP 2016, "An AMP is a financial penalty that can be imposed on individuals or companies who fail to comply with a particular provision of a statute, regulation, an order or a requirement, or the terms and conditions of a permit." AMPs provide an effective enforcement mechanism for a wide range of contraventions and will allow governments to match the penalty with the severity of the non-compliance. By reserving its authority to shut a mine down for only the most severe cases of non-compliance, governments will have a more effective system for forcing compliance.

The ability and willingness of government regulators to apply AMPs can be powerful tool in ensuring compliance with the conditions of a permit. With removal of the shut-down alternative as the only choice, there is no excuse not to apply a penalty for non-compliant situations that result in increased risk.

Regulatory Enforcement: Leading practice would require that a full range of administrative tools, including an AMP program, be developed to support the enforcement of permit conditions.

6.4.3 *Deferred Action*

Other effective strategies to promote compliance include education, assistance, incentives, monitoring and inspections. Another practice that is used at times is to defer enforcement action in order to provide a company time to rectify non-compliant situations based on the rationale that such action would compel the shut-down of the mine. This approach is not unreasonable as long as the risk level of the TSF has not been raised significantly in the interim. When action is deferred on any TSF permit condition, each instance should be documented by the regulatory authority supported by a clear determination that the non-compliant condition will not significantly change the risk profile of the TSF.

Regulatory Enforcement: Leading practice would require the documentation and publication of deferred regulatory action and provide justification for such action including a statement regarding any change in risk level.

6.5 *Transparency - website*

One of the newer ideas to appear in response to recent catastrophic dam failures has been the establishment of the BC Mine Information web platform (BC Info) by the Ministry of Energy and Mines in British Columbia to make information on permitted mines more accessible to interested parties. The information posted to date relating to the 15 active mining operations in BC and, to date, includes:

- Authorizations – permits and amendments;
- Compliance Oversight – inspections; and
- Other Documents – 2014 & 2015 Annual Dam Safety Inspection reports.

The stated plan is to continue developing the use of this web platform with particular reference the Other Documents section.

The value of such a website is that it will add to the transparency and accountability of both government and industry which in turn will help to establish a higher degree of trust through the demonstration of the commitment of both government and industry to protect the public interest. Further benefits can be achieved by posting, for each mining site, the Annual Manager's Report, Periodic Safety Reviews and enforcement activities, particularly those instances pertaining to deferred enforcement action.

Transparency: Leading practice would require that governments create and maintain a website for the posting of permit authorizations, compliance and enforcement reports and other documents relating to operating and closed TSFs in their jurisdiction.

7 RESPONSIBILITY FRAMEWORK - GEOTECHNICAL CONSULTANTS

Professional geotechnical engineers should be responsible for the design of tailings facilities in accordance with the highest state of practice and applicable regulations, statutes, guidelines, codes and standards while fulfilling their professional obligations that "…hold paramount the safety, health, and welfare of the public…". However, it is understood that lawyers will argue that generally accepted professional standards should be defined as work performed in keeping with the prevailing level of care, skill and diligence ordinarily exercised by others who perform similar services under comparable circumstances. This is not a leading practice.

7.1 *Design team selection*

The selection of the qualified professional engineer (QPE) or engineering firm for the design of a TSF should be based on their qualifications, availability and local knowledge. Basic qualifications relate to having the appropriate level of education, training and experience. A more detailed listing requires ensuring that the QPE is knowledgeable in alternate deposition methods and key technical areas related to a particular site. An added qualification is the need for good judgement when dealing with the many uncertainties encountered in the design of a TSF. Furthermore, because of the complexity of tailings dams, a company needs to assess not only the lead engineer but also the composition and members of the design team. Companies should use their ITB to assist in the selection of the design consultant. In cases when the ITB may not be fully formed, the early appointment of its chair person would add significant value to the selection process.

On the other hand, it is the responsibility of the lead professional engineer or professional geoscientist to determine whether he/she is qualified by training and/or experience to undertake and accept responsibility The professional engineer should only take responsibility for design and field review activities related to the design and construction of a dam that are consistent with his/her training and experience.

Geotechnical services: Leading company practice requires that a company carefully select their design team based a knowledgeable assessment of the design team's qualifications utilizing the experience of their ITB or other experts.

7.2 *Client assessment*

Not only should a mining company assess geotechnical qualifications in the selection of a design team, but geotechnical consultants also need to assess the company with regard to its commitment to high design standards and the implementation of a comprehensive tailings governance framework. Why would any self-respecting geotechnical consultant work for a company that may not provide adequate resources for site characterization, may not be willing to implement a strong management system and may not be willing to extend design services to include a role in the development of an operating manual?

Client assessment: Leading consulting practice requires that services are only provided to clients that demonstrate a high level of commitment to the design and operation of TSFs

8 OTHER ORGANIZATIONS

8.1 *Professional engineering associations*

The role of professional engineering associations in providing the technical guidelines and professional standards required for the design of TSFs have been and will continue to be very important in the design of tailings dams that, in conjunction with a strong management commitment and government oversight, can provide an acceptable level of protection for the safety, health, and welfare of the public and the environment.

8.1.1 *Technical guidelines*

Guidelines provided by ICOLD, CDA and ANCOLD are of a high standard and are widely used. ANCOLD, in their website have stated that their Guidelines on Tailings Dams – Planning, Design, Construction, Operation and Closure (ANCOLD 2012) provide:

> "…engineering detail that can be accepted by all relevant government authorities, and national and international companies involved in tailings dam development, allowing them to undertake design and construction consistent with leading industry practice. ANCOLD guidelines are not a design, construction or operation code and practitioners must apply their own considerations, judgements and professional skills when designing, operating and managing dams. As time goes on there will be improvement in contemporary dam practice and it is intended that ANCOLD guidelines will be updated as circumstances dictate."

While recognizing the value of these guidelines, it is important not be complacent about their value. Statements that the technical guidance exists to prevent catastrophic dam failures are largely self-serving. Any body of science that relies on safety factors as a main design parameter has room for improvement in their technical understanding of underlying conditions and the application of new technologies. Furthermore, the application of the current set of technical guidelines requires judgement and interpretation, indicating the need for better guidance for critical areas of uncertainty. This has been illustrated by the recognition by the Association of Professional Engineers and Geoscientists of BC (APEGBC) that, in response to the Mount Polley tailings dam failure, developed "guidelines that would lead to improved site characterization for tailings dams with respect to the geological, geomorphological, hydrogeological and possibly seismotectonic characteristics." (APEGBC 2016)

Whenever there is uncertainty in any design element or the need for judgement or interpretation, there is need for improvement in the underlying science or technology. Having the protection of a safety factor is no excuse not to have an aggressive initiative by the geotechnical profession and the mining industry to identify and then support work on standards of practice for those subject areas that would benefit from improved guidance.

It must be noted, however, that existing guidelines for mining dams have been largely created as an extension or add-on to guidelines for water reservoir dams. Consideration must now be given for their further extension to cover alternate deposition and storage methods. The mining industry, governments and the geotechnical profession must jointly support efforts that, for each potential alternative, will bring together the body of knowledge developed to date, identify and support the need for further study and support the development of the technical guidances and leading practices equal to those for slurry deposition methods.

8.1.2 *Professional practice guidelines*

Two professional practice guidelines, Site Characterization for Dam Foundations (SCDF) in BC (APEGBC 2016) and Legislated Dam Safety Reviews (LDSR) in BC (APEGBC 2014), have been issued by the Association of Professional Engineers and Geoscientists in British Columbia (APEGBC) and both are available on its website.

With regard to SCDF, important features from a management as well as an assurance perspective are:

- The provision of an assurance statement whereby the design engineer verifies that defined activities have been conducted according to the guideline and that the work of supporting professionals has reviewed and accepted; and
- The identification of uncertainties in the site characterization program so that they can be dealt with the design, construction, and operation of the dam through additional investigations, instrumentation, and contingency plans.

An important aspect of the LSDR guideline is the recognition that the terms of reference for any review be appropriate for its intended purpose. The guideline recognizes that:

> "The types of dam safety review can be broadly considered to cover a spectrum ranging from an audit-type review to a comprehensive and detailed design and performance review. The qualified professional engineer should recommend an approach to the dam safety review that will cause the result of the dam safety review to be appropriate for its intended purpose."

It is important for a company to fully understand what level of review is needed for the consequence classification of each dam and to understand how its scope fits with other corporate assurance activities. In the case of the BC dam safety reviews, it is noted that a LDSR will also review operating manuals, confirm proper functioning of management and environmental control systems and identify the magnitude of deficiencies in the dam management system. Of concern in this regard would be the availability of adequate standards of practice and/or protocols and the level of experience and judgement needed to adequately assess the functioning of the tailings governance system at a mine.

8.2 *Mining Association of Canada*

MAC's tailings management program is one of the six key focus areas of their Towards Sustainable Mining program (MAC TSM 2017). MAC describes TSM "… an award-winning performance system that helps mining companies evaluate and manage their environmental and social responsibilities. It is a set of tools and indicators to drive performance and ensure that key mining risks are managed responsibly at participating mining and metallurgical facilities." Members of the Quebec and British Colombia provincial mining associations, the Finnish Mining Association, The Argentinean Chamber of Mining Entrepreneurs and the Botswana Chamber of Mines have also adopted TSM for their members.

Commitment to the TSM program is mandatory for all MAC members' Canadian-based operations requiring self-assessment of performance annually and external verification of self-assessed results every three years. Performance rankings are based on a five point scale (C, B, A, AA, and AAA), with distinct criteria needing to be met at each level before a facility can move to the next one. MAC defines Level A as "good practice" and attaining at least a Level A is a goal for every MAC member site. Level AAA represents "Excellence and Leadership". Each operation's performance ranking for each indicator is reported annually on MAC's website. The supporting auditing and assessment protocols are also made available on their website.

The key elements of MAC's tailings management program are its publications (1) A Guide to the Management of Tailings Facilities (MAC Guide 2011), (2) Developing an Operation, Maintenance and Surveillance Manual for Tailings and Water Management Facilities (MAC Manual 2013) and (3) A Guide to Audit and Assessment of Tailings Facility Management (MAC Audit 2011). These publications have, together, been very effective in improving the overall quality of tailings management, particularly with regard to management systems and operating manuals. MAC is careful to point out that these guides are not technical in nature, do not replace professional expertise and that "…professional advice should be obtained in order to be sure that site and operational requirements are addressed and all regulatory requirements are met."

In 2015, MAC formed an independent TSM Tailings Review Task Force to perform an external review of the guides and tailings protocol to provide advice on potential improvements. The task force submitted 29 recommendations, all of which were accepted by MAC's Board of Directors. The most important of the recommendations were related to:

- Policy endorsement at the governance or board level;
- Improving timelines for the achievement of the A performance level;

- Requiring an independent review mechanism (ITB) to provide additional oversight and advice, including guidance as to its appropriate scope and mandate;
- Providing guidance on the assessment and selection of best available deposition technologies and practices for TSFs.;
- Providing greater guidance for the development of emergency preparedness and response plans;
- Providing more specific technical guidance related to site selection and design; and
- Posting good practice examples of actual OMS manuals on the MAC website.

The implementation of these recommendations will signal a shift from the MAC Guide being primarily a management system guide to one that also includes more specific technical guidance. This will be consistent with MAC's stated desire to be seen as demonstrating leadership worldwide and these improvements should be viewed as a step in the evolution of TSM towards a more comprehensive framework for the management of TSFs.

8.3 *International Council on Mining & Metals (ICMM)*

The ICMM website describes itself as follows:

- "ICMM is an international organisation dedicated to a safe, fair and sustainable mining industry.
- Bringing together 23 mining and metals companies and over 30 regional and commodities associations we strengthen environmental and social performance.
- We serve as a catalyst for change; enhancing mining's contribution to society."

As a catalyst for change it has mounted comprehensive initiatives on issues such biodiversity, water, climate change, community development and employee safety. It has shared its position papers, good practice guides, practical guides and toolkits through its website so that the whole mining industry could benefit from its leadership.

With regard to tailings management, it has only published to date a position statement (ICMM PS) that its members are using to review their tailings governance frameworks. No indication has been given yet as to its intentions to prepare a good practice guide or toolkit that could be made available for the benefit of the mining industry. The position paper describes member commitments to enhanced focus on six briefly described key elements as follows.

8.3.1 *Accountability, responsibility and competency*

This is primarily an outline of a good management system with added emphasis on critical control management. Critical control management is a leading practice developed by ICMM and used by its members to drive progress on safety in the workplace. The adaption this good practice guide for environmental issues, including TSF management, would be a useful contribution to overall industry performance.

8.3.2 *Planning and resourcing*

This commitment addresses the need for ensuring that adequate human and financial resources are available to support a company's tailings governance framework.

8.3.3 *Risk management*

In addition to committing to a comprehensive risk assessment program, members have committed to ensuring that "Suitably qualified and experienced experts are involved in TSF risk identification and analysis, as well as in the development and review of effectiveness of the associated controls" and that "Performance criteria are established for risk controls and their associated monitoring, internal reporting and verification activities".

8.3.4 *Change management*

Members have committed to the practice that "Risks associated with potential changes are assessed, controlled and communicated to avoid inadvertently compromising TSF integrity"

8.3.5 *Emergency preparedness and response*

This commitment includes the requirement that "Processes are in place to recognize and respond to impending failure of TSFs and mitigate the potential impacts arising from a potentially catastrophic failure." In 2005 ICMM and UNEP jointly published a "good practice in emergency preparedness and response" document (ICMM 2005) that provides excellent guidance on this subject.

8.3.6 *Review and assurance*

This commitment requires that "Internal and external review and assurance processes are in place so that controls for TSF risks can be comprehensively assessed and continually improved". Specific requirements are:

- "Internal performance monitoring and inspections and internal and external reviews and assurance are conducted commensurate with consequences of TSF failure to evaluate and to continually improve the effectiveness of risk controls;
- Outcomes and actions arising from TSF review and assurance processes are recorded, reviewed, closed-out and communicated; and
- Performance of risk management programs for TSFs is reported to executive management on a regular basis."

8.4 *Australia*

The Australia Government has prepared a series of handbooks as part of their Leading Practice Sustainable Development Program for the Mining Industry. The handbooks are designed "to share Australia's world-leading experience and expertise in mine management and planning". The handbooks provide practical guidance on environmental, economic and social aspects through all phases of mineral extraction, from exploration to mine construction, operation and closure.

The primary audience for their Tailings Management Handbook (TMH) is stated to be onsite mine management, the primary level for implementing practices at mining operations. The TMH covers all phases of the mining cycle with particular attention to the selection of a suitable disposal method. In this regard, the handbook (AG TMH 2016) states:

> "Regulators now expect all TSF design submissions to demonstrate beyond reasonable doubt that sustainable outcomes will be achieved by the application of leading practice risk-based design that:
> - fully assesses the risks associated with tailings storage at the particular site;
> - compares the suitability of all available storage methods, in particular those that dewater tailings before disposal and/or eliminate the requirement for the damming of surplus water within the TSF; and
> - demonstrates that the selected tailings storage method will manage all risks to within acceptable levels and as low as reasonably practicable."

To assist in the selection process, the handbook has a good description of alternate disposal and storage methods including a description of the advantages and disadvantages of each. The handbooks also support the adoption of AS/NZS ISO 31000 Risk management standard and the critical control management approach as developed by ICMM.

8.5 *Cyanide Code*

The Cyanide Code is introduced here for two reasons. The first is that its existence is primarily due to the efforts of The Gold Institute, an association of gold producers in the United States, that provided the lead in putting forward the idea for a management code following the Baia Mare tailings dam failure that occurred Jan 30, 2000 and then raised the funds and provided the leadership to make it happen.

The "International Gold Cyanide Management Code For the Manufacture, Transport, and Use of Cyanide In the Production of Gold" (Cyanide Code 2017) was developed under the guidance of a multi-stakeholder Steering Committee formed under the umbrella of the United Nations Environmental Program (UNEP) and the then International Council on Metals and the Environment (ICME). The steering committee members represented 8 gold mining companies, 5 governments, 3 NGOs and 2 cyanide producers.

When the code was introduced in 2005 by The International Cyanide Management Institute as an independent organization, nine gold mining, two cyanide producers and three transport companies were the original signatories. At the end of 2016, the Cyanide Code had 46 signatory mining companies covering 102 mining operations, 28 cyanide producers and 139 transporters. Signatory operations are located in 51 countries on 6 continents.

The second reason for its presentation in this paper is that its success may offer guidance in meeting the challenges of gaining public trust in the design and operation of TSFs. The risk associated with the manufacture, transportation and use of cyanide can be equally high in terms of consequence and has required a committed effort to maintain its social licence as an acceptable reagent.

The Cyanide Code commits signatories to manage cyanide in a responsible manner and provides the standards of practice and third party audits to make it happen. The Cyanide Code covers nine key areas: cyanide production, transportation of cyanide to the mine site, handling and storage of reagent cyanide, on-site use and management of cyanide at mining operations, decommissioning of facilities, worker safety, emergency response, training, and communications with the public. The main distinguishing feature of the Cyanide Code is its focus on the adequacy of an operation's plans, procedures and systems and verifying the actual adherence to those requirements in the workplace.

Detailed verification protocols are provided for use by qualified auditors at three year intervals at each mine site. Summary audit reports, usually 30 to 40 pages long, are posted to the Cyanide Code website with the basis for the audit finding for each standard of practice stated in the report.

The verification protocols and the posting of audit results and action plans are considered to be success factors in achieving high performance standards and in earning public trust. The manner in which certification is granted and maintained has also added to the credibility of the Cyanide Code. A company can join the program by agreeing to bring their designated gold mining operations into compliance with the Cyanide Code within three years. Certified operations found in substantial but not full compliance with the Cyanide Code are conditionally certified and must develop and implement a corrective action plan to achieve full compliance, which is also posted on their website. Those operations that fail to substantially meet the requirements of the Cyanide Code have their certification withdrawn.

As a confirmation as to the success of the Cyanide Code, it has been identified by the Australian Government, as part of its Leading Practice program, in its Cyanide Management Handbook as a leading practice. As stated in the handbook (AG CMH 2016):

> "Managing cyanide to minimise risks to human and environmental health represents one of the key challenges that continues to face the mining industry. In order to assist the global mining industry to improve its management of cyanide, the Code was developed by a multi stakeholder steering committee and is today managed by the International Cyanide Management Institute (ICMI 2006) to provide a risk-based management process by which the mining industry is able to implement and demonstrate that it can meet leading practice for cyanide management."

9 ACCEPTABLE RISK & CORPORATE COMMITMENT

The tailings responsibility framework, as described in this paper, has two major themes that will be discussed in more detail in this section. In particular, the concept of acceptable risk as it relates to the deposition method and site selection approval process is very important. The need for demonstrated commitment is an equally important part of the approval process and is linked to the effectiveness of the assurance activities at each operating mine.

9.1 *Acceptable risk*

What constitutes acceptable risk depends on the perspectives of the organizations or persons involved. A family or community living in the dam breach inundation zone will have different perspectives than the mining companies whose executives have been told that their designs and risk management practices have been based on best practices. The level of risk deemed acceptable to a corporation is not necessarily what may be considered acceptable to government or the public even if it is based on a collaborative engagement process. The acceptability of risk also has to considered closure implications as well the broad benefits to society that flow from economic development.

9.1.1 *Corporate perspective*

In addition to the potential financial impacts of dam failure a company must also consider potential external impacts such as the loss of human life, environmental damage and public economic loss in what it believes to be an acceptable level of risk protection for its proposed deposition method. The challenge for a company will be to separately consider the costs, consequences and likelihood of failure for each alternative and then make a balanced decision based on a meaningful public engagement process and its own risk tolerances.

From a corporate perspective, the financial consequences of a slurry dam failure or the possible malfunctioning of thickened, filtered or paste alternatives all carry significant financial risks. The problem is that the possible alternatives to slurry deposition have not yet established the same body of knowledge that could support development of professional guidances and professional protocols of a quality equal to that for slurry deposition.

While the public consequences resulting from dam failure may be lower, the financial consequences related to any form of storage facility malfunction will still be material. With the added complexity presented by closure considerations, a company's definition of acceptable risk may lead it to rule out certain alternative storage methods.

9.1.2 *Public perspective*

From a public perspective, what defines acceptable risk is not found in the results of a dam safety review report. As an example, the guideline Legislated Dam Safety Reviews (APEGBC 2014) states:

> "The determination of what is the acceptable level of risk or safety for the various elements which are identified as being at risk is not the role of the qualified professional engineer and is outside the scope of the dam safety analysis. The acceptable level of risk must be established and adopted by the regulatory authority in consultation with the dam owner. However, an assessment of the various elements at risk, through the dam failure consequences classification established by the relevant regulatory authority will guide the qualified professional engineer's dam safety analysis."

What this basically states is that a government's or regulatory authority's approval of a design, based on an acceptance of its identified risks, defines what constitutes an acceptable level of risk. The assurance statement required as part of the Dam Safety Review Report verifying that "the dam is reasonably safe" means nothing more than the dam's level of risk is no worse than that level of risk previously approved by the government. Clearly the government is the final arbiter as to the determination of acceptable risk on a case by case situation.

The ideal outcome is to have all parties agree, based on informed opinions, that the risks, and their mitigating measures, for a proposed mine plan are acceptable. Informed opinions are only possible when all parties, particularly the public, have been provided with:

- The opportunity to participate in a meaningful communication and engagement process;
- The consequence rating of the proposed dam, including the results of a dam breach and inundation study;
- Information that supports the selection of the deposition method and site location;
- Information that demonstrates beyond reasonable doubt that the owner is committed to the management of the dam and its critical risks through the establishment of a comprehensive management framework and assurance program; and
- Information that demonstrates that the government has established and will be committed to an effective compliance and enforcement regime.

9.1.3 *Government perspective*

If all parties agree, government has the mandate to proceed with the approval of a TSF plan as proposed. If a government has continuing concerns related to any of the above factors, it has the responsibility to request additional information, specify certain conditions or, if not fully satisfied, refuse to approve the proposed TSF plan. In making a decision to approve or not approve a TSF plan, government also must consider its responsibility for the protection of employees, the public and the environment from undue impacts and risks arising out of or in connection with mining operations. Obtaining and considering public input is an integral aspect of the decision making process that a government must adopt when considering the approval of a TSF plan. The value of economic activity to society should not be an over-riding factor in its decisions.

Once a decision has been made by a government to approve or not approve a TSF plan, it should make its decision public in a manner that addresses any outstanding public concerns and explains why the acknowledged risks have been judged to be acceptable in the specific circumstances of the mining operation and its TSF plan.

9.1.4 *Approval Process*

It is believed that, by having a process that requires a high degree of engagement and transparency, trust in the actions of both mining companies and governments will be increased and the risks related to TSFs will be reduced. The consequences of failure will be more apparent, forcing companies to address the potential risks through the development of improved deposition methods and the use of more committed management frameworks. Using the consequences of failure to drive risk reduction will be more effective than dictating the use of best available technologies. Perhaps in this context, the main reference should be to acceptable consequences. The use of the term risk only serves to confuse or mask the issues.

In situations where governments lack the capacity or the will to properly address the issue of acceptable risk, company directors will have to be extra diligent in their review and approval of new projects. In this regard, full consideration must be given to the potential consequences of failure in order to be sensitive to the independent perspectives that would normally be provided by strong governments and informed public opinion. One idea would be to have the CEO justify in writing the acceptability of a TSF proposal having given full consideration to the consequences of failure.

9.2 *Demonstrated Corporate Commitment*

A high degree of corporate commitment serves two purposes. First of all, it addresses a material corporate risk. Secondly, it provides the basis for earning the trust and acceptance of the government and its public stakeholders.

9.2.1 *Commitment*

Leading practice requires that corporate directors of a company recognize that the management of its TSFs is a material risk and show commitment through a strong governance and oversight program that requires its approval of a corporate tailings management policy and, through its sustainable development committee, requiring assurance that the policy commitments are implemented and maintained on an ongoing basis.

With committed leadership being provided by its board of directors, the chief executive officer becomes accountable for of the implementation of the company's tailings management policy. This will require the establishment of a tailings governance framework that will include the development of a TSF management system, operating manuals, corporate standards, risk assessments and assurance activities. All these requirements will have to be well documented for them to be effective and to provide the basis for assuring both the directors and the stakeholders of a company that it is capable of managing its TSFs within the limits of acceptable risk.

Corporate commitment is easy to state but more difficult to instill in an organization. Board and CEO leadership is a significant contributor to the establishment of a committed culture. However, the real test is what happens at different mines and at the working level. With regard to commitment, employee surveys have proven to be a useful tool in the assessment of commitment. One survey the author is aware of measured employee perceptions of safety leadership at two different mines in the same country. One mine clearly was recognized as having a higher level of leadership but still ranked just above median on the "ensure rule compliance" sub-measure. In another case, a corporate employee survey showed differences in perception as to commitment to safety and environmental management between mines and regions. From another perspective, an analysis of fatalities, major accidents and near-misses at a mine over a five year period showed the main contributing factors to be no or inadequate procedures and the lack of enforcement of existing procedures. This supports the belief that the real tests of commitment during the mine operating stage are to be found in the adequacy of and actual adherence to documented operating procedures at the working level. It then follows that it is the implementation level that should be a major focus of a company's assurance program.

9.2.2 *Demonstrated Commitment*

Demonstrated commitment is required at the stage of project approval and on a continuing basis by:

- Company's directors who need to be assured that their policies have been implemented;
- Employees who need to believe in the importance of what they are asked to do;
- Regulatory authorities who need to have confidence in what they are asked to approve; and
- The public that is expected to accept corporate commitments on the basis of trust.

All seek assurance that a company has or will establish high performance standards and that they have been fully implemented. Such assurance is now being provided in many ways. Governments carry out inspections and require annual manager reports, annual EOR reports and periodic dam safety reviews. BC has also required the establishment of independent tailings review boards to provide advice and assurance to fill in some of the gaps that exist in current assurance coverage, a practice that some companies adopted at least 20 years ago. Major companies have established assurance capabilities within their internal audit functions. Member companies of MAC are required to undergo an audit of tailings management systems and qualitative assessments of certain elements every three years.

Whether or not these assurance activities are sufficient to ensure a high level of performance across all aspects of a tailings governance framework is an open question. Gaps and half-measures still exist. Geotechnical experts are asked to review management systems without the aid of detailed protocols. Requirements that operating manuals be prepared prior to commencement of operations are not supported by audit protocols to verify their quality. Management system audits do not address the qualitative aspects of procedures, particularly those of a critical nature.

Further assurance is required to prove that appropriate professional and/or expert advice has been obtained and that site-specific procedures and performance measures are of the highest standard. One way this could be provided is by extending management system audits to verify that adequate professional and/or expert advice has in fact been obtained, included in the operating manual and carried out in practice. Another approach would be to formally require that the EOR's annual reports include not just a review of critical control measures but also the verification that their requirements have been adhered to over the full year under review.

The final and most important step in demonstrating commitment is to prove that all the words contained in policies, standards, management systems and operating manuals actually result in meaningful action in practice. Assurance is required to ensure that site-specific operating, monitoring, surveillance, maintenance and reporting procedures have been fully implemented and are being adhered to on a continuing basis. This requires an approach similar to that adopted for the Cyanide Code. That is, stated principles, standards of practice, detailed verification protocols, independent third-party audits, public posting of audits and strict certification standards.

9.3 *Public Trust*

Acceptable risk and corporate commitment are inextricably linked. A company that is able to demonstrate its commitment to the public through the application of a strong tailings governance framework will stand a better chance of having its proposed tailings plans accepted by the public. A company that is able to demonstrate its commitment to its own employees will more likely develop a lower risk proposal. A company that is able to demonstrate its commitment to the regulatory authorities and the public will find the permitting process much easier to navigate. A company that has done all these things will also stand an excellent chance of not having a catastrophic dam failure.

10 PATH FORWARD

The strategic intent for the mining industry must be to eliminate tailings dam failures and incidents. The leading practices identified above, within the context of a comprehensive responsibility framework, will, if adopted, significantly contribute to a reduction in the frequency of dam failures in general, a reduction of the consequences of failure and the reduction of catastrophic failures in particular. However, any framework based on safety factors, unpredictable natural events, judgement, human errors and varying levels of commitment will always be less than perfect requiring a continuing focus on the reduction of the consequences of failure.

For new tailings dams, a rigorous deposition method and site selection process must be used to ensure that the consequence rating of the approved method does not exceed an acceptable level. Existing operations must seek ways to reduce risks related to the original design. More importantly, companies, governments and geotechnical professionals must look to their own commitment and embrace the leading practices suggested in this paper with the objective of providing the highest standard of risk management for their tailings dams.

In the short term, action should be taken to reduce the risks posed by using tailings impoundments to store water. For climates where water storage is inevitable, design requirements regarding freeboards, beach lengths and phreatic lines should be clearly identified and strictly enforced by both corporate management and regulatory authorities. The use of tailings impoundments as polishing ponds to reduce contaminant levels and to handle run-off and excess pit water must be stopped as such practices only add to the risk level of a dam. Regulatory approvals for new tailings storage facilities must require the consideration of alternatives based on the minimization or elimination of water storage within the impoundment.

To support the consideration and adoption of lower consequence alternatives, a process must be initiated for the purpose of:

- Bringing together the body of knowledge developed globally for each alternative deposition method;
- Identifying and funding areas requiring further research or study; and
- Developing technical guidances and leading practices for their design and operation.

For the longer term, it is difficult to identify what organization or what group of organizations that will accept the challenge of moving aggressively towards the development of a comprehensive and leading edge Tailings Responsibility Framework. Unfortunately, this may have to wait for the next catastrophic failure. To prepare for that eventuality, useful progress could be made on focused priorities, which should be to:

- Define the principles and standards of practice to be expected of a company's board of directors and provide appropriate protocols to guide and measure their implementation;
- Prepare a guidance document to support the application of critical control methodology to the identification and management of critical tailings dam risks;
- Define the principles, standards of practice and protocols required to guide an annual validation of the integrity of a tailings dam design, the adherence of the company to its regulatory and internal requirements and the implementation and maintenance of its critical control procedures; and
- Develop a model of a comprehensive and integrated assurance and reporting program that supports the needs of companies, governments and public.

Work must now start on the next round of incremental changes. It is hoped that this paper will have dispelled any complacencies that the current situation is satisfactory. Waiting for the next catastrophic dam failure is not good enough. It is also hoped that many of you will work towards the further development and implementation of some of the ideas within your own organization and the associations to which your organization belongs.

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Density and strength requirements for capping and reclaiming soft tailings deposits to meet land-use goals

Gord McKenna, Brent Mooder, Bill Burton, & Andy Jamieson
BGC Engineering Inc., Vancouver, BC, Canada

ABSTRACT: Six capping techniques for soft tailings are commonly employed: water capping, sand raining, low-density floating covers, hydraulic sand beaching, soft-ground techniques, and standard earthworks. The choice of techniques depends on the tailings properties, proposed land uses, and the availability of suitable capping materials. Capping fluid and soft tailings is challenging and expensive due their low density and low shear strength. Design and construction of a safe and stable cap often involves the interplay of fluid and soil mechanics.

Considerable international experience has been gained in capping soft tailings in metal, coal, and oil sands mines. There have been comparable achievements in capping dredge spoils; natural harbour, lake, and river sediments; sewage lagoons; and low-strength industrial wastes. The paper presents the results from a literature review of notable mining and heavy-civil capping projects.

Post-reclamation settlement is usually a major design consideration. Underconsolidated tailings settles due to the combination of self weight and the weight of the cap and can result in tens of metres of post-reclamation settlement over decades or centuries. Even fully consolidated tailings can settle several metres due to the added weight of the cap. The magnitude and timing of the settlement can be difficult to predict. Periodic or permanent flooding of settled areas can have adverse impact on dam safety, land use, and the water balance.

Mines are testing commercially available tailings processes that create dense, firm to stiff tailings deposits that provide lower geotechnical risks, are much easier to cap and reclaim, and can allow mines to more reliably meet their environmental and land-use goals. The paper discusses the pros and cons of these new technologies and provides a framework for selecting and assessing tailings strengths and densities for each of the six capping options.

The State of Mining Geotechnics

Richard R. Davidson
AECOM, Denver, CO, USA

ABSTRACT: A string of recent failures in the mining industry has severely eroded the trust by which we in the geotechnical profession must practice. These failures have included large tailings dams, open pit slopes, and crushed leach pads located at some of the premier operations around the globe. Forensic studies have revealed significant erosion in the three R's of resilience, robustness and reliability, not only of the designs and construction but in the management of the facilities. Major changes are now being implemented in the regulatory review process, as well as the way we design and manage these facilities. How do we win back that trust?

1 A CRISIS OF TRUST

We geotechnical engineers serving the mining industry in 2017 find ourselves in a crisis of trust. We have lost the trust of the public and our regulators. We have lost the trust of our shareholders and managers. We have lost the trust of our geotechnical colleagues…

In the last few years we have suffered previously unimagined catastrophes with our open pits, i.e. Manefay failure (Figure 1), waste dumps, crushed leach operations and tailings dams (Figure 2). Have these structures ceased to obey the laws of physics and geotechnical engineering, or are there other more systemic issues at play?

2 EROSION OF THE THREE R'S

I believe that our recent string of unfortunate mining industry failures reflect an erosion of resilience, robustness and reliability, the 3R's, resulting in incremental increases in risk which were not recognized or not acted upon. Failures typically result from a sequence of events that may not only be geotechnical but are often aggravated by management decisions.

We must assure that we provide the 3R's in our mining structure designs:

- Resilience
- Robustness and
- Reliability

Resilience is like Rocky Balboa who keeps taking hit after hit but continues to stand. Resilience is defined as the capacity to recover quickly from difficulties or as toughness.

Figure 1. Manefay Open Pit Failure of April 2013 (KSL TV News).

It is the ability to absorb or avoid damage without suffering complete failure. The USACE says resiliency is “the ability to avoid, minimize, withstand, and recover from the effects of adversity, whether natural or manmade, under all circumstances.”

Robustness in design usually refers to the number of lines of defense. Robustness contributes to resilience. The US Army Corps of Engineers (USACE) says robustness is “the ability of a system to continue to operate correctly across a wide range of operational conditions (the wider the range of conditions, the more robust the system), with minimal damage, alteration or loss of functionality, and to fail gracefully outside that range.”

Reliability in design refers to whether you can count on those lines of defense during an emergency. Will they work as intended? The USACE adds that redundancy is “The duplication of critical components of a system with the intention of increasing reliability of the system, usually in the case of a backup or failsafe.”

These principles are directly relevant to all of our mining industry geotechnical projects, not just our tailings dams.

3 MANAGEMENT SYSTEMS

The 3R’s are also relevant to our management systems, operations and construction, and downstream consequences. “3R” Management has not been well understood in the mining industry. The application of “3R” management requires a structure and clear definition of the responsibility, authority, capability, resources and liability of each position and role. How can management contribute to resilience, robustness and reliability?

Figure 2. Samarco Fundao TSF Failure of November 2015 (Orpo de Bombeiros-MG 06/11/2015)

The International Council on Mining and Metals (ICMM) has tackled this issue with a far reaching "Position statement on preventing catastrophic failure of tailings storage facilities" issued in December 2016. They have identified the key elements of TSF management:

- Accountability, responsibility and competence
- Planning and resourcing
- Risk management
- Change management
- Emergency preparedness response
- Review and assurance

Each of these elements deserves a paper by itself.

The importance of management has become clear in the aftermath of the catastrophic TSF failure at Mt Polley in 2014 (Figure 3), which had significant and unintended erosion of resilience, robustness and reliability resulting in part from alleged management decisions. An example is change of the Engineer of Record (EOR). Whether a new Engineer of Record is assigned internally or externally, there is a risk that the historical knowledge of the facility and its design basis would be lost in the transition and the new EOR does not fully own the design. The undocumented understanding of the level of design resilience and vulnerabilities may not be obvious to the new team and could inadvertently be eroded. Managers may be forced into difficult compromises with the typical cycles of financial pressure and market forces that erode resilience, robustness and reliability. Without clear understanding of the risks and implications of this compromise, critical mistakes can be made with the best of intentions.

4 OPERATIONAL AND CONSTRUCTION RESILIENCE

Operations and construction resilience, robustness and reliability can be eroded by gradual and seemingly insignificant compromises in construction, and becoming complacent with marginal situations with an imperfect knowledge of just how close you are to failure. Because our mining structures are under a constant state of construction, unknown changes can occur that have devastating consequences. This is the reason we preach the application of the observational method for all of these structures. The observational method, if properly implemented, provides extensive instrumentation coverage and thresholds for potentially dangerous conditions with enough time allowed to respond and correct the situation. However, rapid loading of materials that behave in a brittle, collapsible manner can cause strength loss quickly without much warning. Therefore it is essential to avoid putting structural materials that you must rely on into this vulnerable state. Furthermore, if monitoring lapses or it is not focused on the right failure mode or location, these problems can grow into a failure. Mt Polley operated with a very steep temporary downstream rockfill slope without understanding of the impact of the soft lacustrine clay layer in the foundation. Flattening the slope would have prevented the failure. Another common operational challenge is raising the structure too quickly without understanding the geotechnical impact under undrained conditions. It is amazing how many geotechnical mining practitioners still fail to grasp the importance of undrained stability.

5 CONSEQUENCE RESILIENCE

Consequence resilience refers to the ability of the area downstream to absorb failure consequences. This is clearly demonstrated in the differences between Mt Polley and Samarco (Figure 3). In the case of Mt Polley, the geotechnical breach of the rockfill dam had a limited geometric extent and the tailings released were primarily driven by the excessive amount of water stored in the impoundment (another erosion of management resilience). The consequences were limited to tailings flow into a nearby deep lake, and current indications are that the lake is recovering well from this unfortunate tailings incursion.

However, the Fundao failure released retained tailings into the Fundao drainage with enough energy to flow through the town of Bento Rodriguez (Figure 3) and then some 640 km to the ocean. There were nineteen fatalities. Hence this structure had low propagative resilience and low consequential downstream resilience.

Fortunately, the Manefay failure shown in Figure 1 was fully contained within the open pit, and radar monitoring provided sufficient warning to withdraw the mine operating staff from “harm’s way”. In spite of losing the main haul road access as well as covering substantial ore reserves with slide debris, this failure had relatively high consequence resilience, but the propagative resilience was over-estimated.

Figure 3. Contrast of Mt Polley tailings release in August 2014 and Impact to Bento Rodrigues from the Fundao breach in November 2015 (Cariboo Regional District, Ricardo Moraes/Reuters)

6 "3R" DESIGN

One protection against these devastating consequences is having a design that meets the 3R's. For example, upstream method tailings dams may have relatively little design robustness. To overcome this we must add operational reliability with critical operational practices that maintain the decant pond as far as possible from the crest, and we must create a wide structural shell of drained sand to maintain stability, in effect adding resilience and robustness. Centerline and downstream method dams can be more resilient with robust lines of defense and high reliability if they are constructed correctly. How can the 3R's be eroded? Compromising on slopes, beach width and freeboard, not conducting sufficient geotechnical investigations to identify flaws in the foundation, not providing important quality control and quality assurance that what is being built satisfies the design intent; these can all erode the resilience, robustness and reliability of the design. The reader is referred to excellent post failure reports for Mt Polley and Samarco developed by the forensic panels chaired by Dr. Norbert Morgenstern.

7 PIT SLOPES

With our aging pit slopes, robustness erodes as we go deeper and deeper to scavenge that last pocket of valuable ore. There may be pressure to compromise slope stability in the declining years of mine operation. Pit designers used to say that the perfect pit design is one that fails as the last ton of ore is removed. Manefay proved that we must move away from this design paradigm. Mine plans and slope designs are constantly evolving over the mine life, but must maintain a degree of resilience and robustness to handle the upsets and inevitable unknown geologic conditions. In some cases we may be ignorant of the gradual erosion of slope resilience relying on past performance precedent which is no longer applicable; or with an incomplete appreciation of the complex structural geology of the site and the changing geomechanical properties of the rockmass and discontinuities. We must ask ourselves if the rock mass has degraded with time? Do we have sufficient slope investigations? Are more comprehensive 3-D models required to simulate actual pit slope behavior?

Another design difficulty is our inability to correctly simulate propagation resilience in debris flow and tailings runout after a failure. New tools are being developed, but the profession must be cautious about using any one method without checking other methods and review by subject matter experts. A large uncertainty band is essential.

Younger pits are often be-deviled by an incomplete understanding of the critical geologic structures within the rockmass. You can never drill enough core holes to understand the 3-dimensional slope stability throughout the mine life. Hence, the caution and guidance provided by the Large Open Pit (LOP) manual is a must for every pit geologist and geotechnical engineer. Fortunately our ability to monitor pit slopes has improved dramatically with the introduction of powerful radar monitoring systems, SAA inclinometers and TDR devices, and drone reconnaissance / surveys of previously inaccessible areas. Mining marginally stable areas has been facilitated with remote controlled autonomous equipment. Ross (2017) provides a gripping story of Rio Tinto Kennecott Copper's heroic return to operation after the 149 M m^3 Manefay pit slope failure. It is amazing what can be accomplished when the entire focus of a mine is on returning to operation safely with the 3R's addressed.

8 HEAP AND CRUSHED LEACH

Heap leach and crushed leach operations are challenged by a material that alters its engineering properties through the passage of acid through the ore (Figure 4). The profession needs to continue to evolve our understanding of these unique materials. High fills can be influenced by crushing of weak materials under their extreme heights, as well as foundation undrained performance under rapid dump advance loading. The position of the phreatic surface, percentage of full hydrostatic pressure and the degree of saturation are critical to maintaining stability as the facility rises with additional lifts. Compaction or fines dominated layers can develop at lift boundaries that exhibit relatively low permeability and it can impede downward flow of the solution to the drain system. The layers can also introduce zones of weakness that are vulnerable to undrained shearing. The CPTu has proven to be an effective means to characterize these complex in situ conditions. Providing a resilient, robust and reliable design of these facilities is a clear and present challenge to the mining industry.

Figure 4. Crushed Leach

9 PREVENTING EROSION OF THE 3R'S

How do we prevent erosion of the 3R's? First and foremost, the mining company must retain a competent engineer of record with sufficient resources to develop and maintain a safe design. The designer must consider various alternative technologies and mining approaches to achieve the project objectives by avoiding potentially dangerous conditions. Secondly, a design review board can be utilized to provide independent opinion on each stage of the design and construction process. This group of experienced subject matter experts must report directly to senior mine management and provide unbiased assessment of all aspects of the project and the performance of implementation team. They must be free to ask the uncomfortable and difficult questions that may challenge the design and operational "sacred cows". Mine management, including the corporate board of directors must have a clear understanding of the risks they bear. Thirdly, an independent safety reviewer may be helpful to conduct periodic inspections which he reports to the mine engineer. Finally, sufficient geotechnical resources must be committed by the mine to the safe operation of the facility. Each of these roles must be integrated but remain independent, and all are essential.

Not surprisingly, these tragedies have lost the trust of the public and galvanized opponents to mining. It is time for us to earn back this trust by demonstrating that we can indeed provide resilient, robust and reliable structures that can protect the public as well as shareholder investment. There has been significant progress as evidenced by the ICMM commitment, as well as widespread use of design review boards, alternative tailings technologies, and state of the art monitoring systems.


Figure 5. Golden Cross Mine, New Zealand

10 RESPONSE OF THE INDUSTRY

Firstly, mining companies are embracing one of the most advanced safety cultures worldwide of any industry. Secondly, more corporate tailings stewardship programs are being implemented based on the model originated some twenty years ago with Freeport McMoRan. Finally, regulators are becoming better informed and more integrated into the process of assuring public safety and environmental stewardship. New regulatory guidance has been adopted by the Province of British Columbia and the State of Montana mandating the use of independent subject matter expert review.

Secondly, the public watches closely how the industry responds to these failures. Is it business as usual or is there fundamental change in the way we do business. Catastrophies suffered by the Bureau of Reclamation at Teton Dam in 1976 and the USACE in New Orleans from Hurricane Katrina led to major changes in those organizations such as routine use of independent review boards and quantitative risk assessment. How is the mining industry responding?

One of the best examples of successfully rebuilding public trust is the Golden Cross Mine in New Zealand shown in Figure 5 and discussed in Davidson (2007). In this case the trust of the public and the regulator was lost when it was announced that the tailings dam had been unknowingly situated on a large landslide that had been reactivated. We were called in to try to identify the cause of the landslide and to develop a stabilization solution before the dam breached releasing its cyanide tailings to the Watekauri Stream. The mine went through one of the most gruesome public meetings with local neighbors and NGO's clamoring for the head of the ones who were responsible. Fortunately, we were able to geotechnically stabilize the landslide and win back that trust. Afterwards, attending a picnic sponsored by the mine for those same neighbors and their families to celebrate the completion of the project, provided a rare opportunity to hear words of thanks and praise for how the mine handled this difficult situation.

What we do in mining geotechnics is difficult and risky. Surprisingly, the frequency of failures is not diminishing with time and technological advances. Is it because our sites are worse and our structures continue to grow? In South Africa, upstream paddock tailings dams were not allowed to exceed 30 m in height because of a history of failures at that height there. Hopefully we learn from our failures.

Perhaps a renewed focus on the 3R's by our engineers, operators and managers could make a difference. Regardless, we must earn back trust to demonstrate to all that we can safely manage our mine facilities and deliver value to our stakeholders.

ACKNOWLEDGMENTS

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Filtered Tailings Disposal Case History: Operation and Design Considerations Part I

B. Erickson, D. Butikofer & A. Marsh
Hecla Greens Creek Mining Company, Juneau, AK, United States

R. Friedel, L. Murray & MJ. Piggott
Klohn Crippen Berger, Vancouver, BC, Canada

ABSTRACT: The Hecla Greens Creek polymetallic mine in Alaska has the longest serving surface filtered tailings disposal facility in the mining industry. Filtered tailings disposal is seen as one of the best available solutions for tailings stewardship for certain operations especially: low to mid-production mine; footprint constrained or mines requiring water recovery; and reduced long-term risk. However, there are a limited number of operational examples of this technology. This paper, which is Part I of a two-part case history, describes the key lessons learned over 25 years of construction of the filtered tailings stack. Challenges that have been faced include the wet cold climate, variations in ore body and process affecting filtered tailings characteristics, and the constraints of a tight footprint. This operations-focused case history will provide valuable insights into operational challenges that have been successfully dealt with at Greens Creek over a prolonged period. The lessons learned at this site will be of value to others considering or planning similar facilities in any climate.

1 BACKGROUND

1.1 *Overview*

The Hecla Mining Company (Hecla) owns and operates the Greens Creek polymetallic mine located on northern Admiralty Island, about 29 kilometers southwest of Juneau, Alaska. A portion of the mine facilities are located within Admiralty Island National Monument.

Admiralty Island is known for having one of the largest populations of brown bears in the world and is home to many species of salmon, birds, whales, and deer, which makes the island a popular destination for tourists. Western Hemlock and Sitka Spruce dominate the prolific rainforest vegetation. This temperate coastal rainforest ecosystem is characterized by cool temperatures and high annual precipitation.

One of the key elements in obtaining a permit to operate in such a sensitive area was to find and implement a tailings disposal method that would be safe and effective in this harsh environment while preserving the world class tourism attractions in the area. The solution was the world's first large filter-pressed tailings disposal facility (TDF). The key advantages of this approach were: the ability to withstand static and seismic forces; more than 50% smaller footprint when compared to conventional tailings storage and no pond on the tailings deposit; allows wildlife to roam freely without risk; and allows for progressive reclamation of the land.

Construction of the TDF commenced in 1988, and tailings placement began in early 1989. In 1993 mine operations were suspended due to low metal market prices. The mine re-opened in 1996 and the TDF has been expanded numerous times to support the extended mine life.

During the initial design phase for the TDF, there was no precedence for filter-pressed tailings disposal in the mining industry. Issues that had to be considered during design were: interaction with the environment; closure; static and seismic stability; wet and cold weather placement; protection of ground water; and control of surface water. While the Community, mining industry and Regulators are now convinced that the fine tailings material is stable under the high rainfall conditions experienced at the site, there continue to be operational challenges.

Of the 2300 tons of tailings produced daily, roughly half is sent through a batch plant where 5% to 8% cement is added prior to haulage underground for use as structural backfill. Tailings not sent underground are trucked by 38-tonne, 18-wheel off-highway tractor-trailer trucks approximately 12 kilometers to the TDF.

1.2 *Regulatory Framework*

The Greens Creek TDF is regulated under the Federal and State governments. Federally, the U.S. Forest Service, through the National Environmental Policy Act (NEPA) process, evaluates the environmental impacts for the existing facility and any proposed expansions. The State, primarily through the Alaska Department of Environment Conservation (ADEC) and Alaska Department of Natural Resources (ADNR), permits/authorizes waste management, including approving pollutant discharge levels, dam certification, water authorizations, and to some extent, air quality.

Hecla has used both the Federal and State requirements to develop a General Plan of Operations (GPO) that documents the various permit requirements as well as mitigation measures to minimize the impact of the mine on the environment. Prior to implementation, the GPO was submitted to both the Federal and State government agencies for approval. Amendments/updates to the GPO are completed through a re-approval process, which occurs every five years, and in conjunction with permit renewals. Smaller amendments to the GPO can be made on an as-needed basis.

1.3 *Hecla's Tailings Framework*

Hecla is committed to the safe and environmentally responsible design, construction, operation and closure of its TDF's. Key aspects of this commitment include: development of site specific management plans; engaging qualified consultants and contractors; maintaining water balances and integrating them into tailings management decisions; periodic reviews, audits and risk assessments; leveraging employee and contractor knowledge; and consultation with the community.

To this end, in October 2016 Hecla formalized a tailings management standard as part of its Environmental Management System for the secure life cycle tailings management at all Hecla facilities. To effectively implement this policy, Greens Creek Surface Operations (Surface-Ops) technical services staff work closely with the tailings operators to ensure tailings are placed in an efficient and stable manner. The Surface-Ops technical staff readily recognizes that the "boots on the ground" are an invaluable resource for developing, defining, and implementing best practices. Significant effort by both parties is made to build and maintain positive and effective relationships between the operators and technical staff, and creativity is encouraged. Experience has shown that the success of a filter-pressed tailings facility is predicated on the skill and dedication of the operators.

2 OPERATIONS

The management of any TDF comes with a unique set of site-specific challenges. For the Greens Creek mine, these include:

- changes in regulations;
- general management of the facility;
- environmental conditions and performance;
- material quantity/quality; and
- people.

2.1 *Changes in Regulations*

Planning and permitting of the Greens Creek TDF began in the late 1970's. Since that time, there have been numerous changes in regulations and personnel on both Hecla's and the agencies' side.

While changes in regulations do impact operations, these changes rarely occur quickly and are rolled out over a period. Generally speaking, changes in environmental regulations tighten the acceptable limits for design and discharges. The roll-out period gives Hecla time to assess the potential effects of the change and work with the relevant agencies to minimize the negative impacts to the operation.

With respect to personnel changes, this can result in a different interpretation of previous agreements made between Hecla and the authorities in relation to how regulations are interpreted and how they should be applied. The "why" of these discussions/decisions is buried in the regulatory codes and permits as well as numerous supporting documents (e.g. reports, letters, e-mails, meeting minutes, etc.). The "how", for complying with these decisions and the regulations, is documented in various GPO's.

While there are systems in place for the transfer of institutional knowledge on both Hecla's and the agencies' side, there are significant challenges for new personnel. Duplication of work can often occur as a result of not having the history of both the "why" and the "how", and can slow the permitting process, and be detrimental to the management of the facility.

2.2 *Facility Management*

2.2.1 *Access Roads*

Providing safe, efficient, and effective access to placement areas within the TDF has been an ongoing challenge since the mine began operations. Historically, facility access roads were constructed of road rock (typically 6-inch minus jaw-run with sufficient hardness to minimize pulverization) and raised as necessary with additional lifts of rock to accommodate increasing pile height. Though these kinds of roads provided a stable platform for max-haul and other traffic, they created other problems when they are abandoned and covered because of changes in the pile geometry. The abandoned roads collect water and become "perched aquifers" within the pile. Tailings placed around these perched aquifers tend to wick water from the perched aquifer into active placement areas, inhibiting the operators' ability to place and compact additional lifts of tailings. Water from these perched aquifers has also been observed seeping from outer pile slopes.

Because of these challenges, Hecla has abandoned this practice and now constructs tailings access roads on a tailings base. When a firmer foundation is required, a separation fabric is placed beneath road rock to minimize mixing of the tailings and road rock. As pile geometry or placement needs change, the road rock is removed prior to placing additional lifts of tailings.

2.2.2 *Lining*

Prior to 1996, the TDF was developed on a low permeability clay foundation. The years from 1996 to 2005 brought an increased understanding of the TDF (e.g., the effect of different handling and placement techniques on the tailings behavior) and changing regulations. As a result, the design of the facility progressed to incorporate geomembrane-lined cells; slurry cutoff walls; upgrades to the water management system; and construction of infrastructure to support mine operations.

When the mine was first opened, the TDF was constructed directly on the ground surface without an engineered liner. The justification for this approach was that there are artesian pressures beneath the pile limiting the downward migration of contaminants from the TDF. However, additional expansions were constructed over areas of exposed bedrock, and geomembrane liners have been included to minimize the potential of contaminants infiltrating through cracks in the bedrock and contaminating the groundwater.

2.2.3 *Water Management*

Because the Greens Creek Mine is located in a rainforest, particular care must be given to manage water flowing into, around, and through the TDF. Managing “water controls” can constitute a significant portion of the tailings operators’ day.

The water management system for the TDF is comprised of “clean water” diversion ditches around the perimeter of the facility, and “dirty” ditches within the facility that collect water that has come in contact with the tailings. Above- and below-liner French drains are also part of the design in the geomembrane-lined portions of the facility. In the most recent expansion, segregated the outfalls from the various water sources were included in the design so that water quality monitoring of the discrete sources can occur. This will assist in understanding the in-pile geochemistry and water pressure.

Surface runoff from the TDF is collected in off-pile settling ponds, which are designed to accommodate storm flows, and routed to a water treatment plant for treatment prior to discharge to the environment. When long slopes are left unarmored, ponds can quickly fill up with sediment creating an unnecessary burden on the water treatment facility.

Managing runoff has become more challenging as the tailings pile has grown in height and footprint (40 meters) and area (27 ha). The silt-sized tailings, where un-compacted and not “seal-rolled”, can erode easily and gullies form rapidly on exposed slopes, even on grades less than 2 to 3 percent (Figure 1). Sealed and compacted tailings are relatively erosion resistant except in heavy rain and/or concentrated flows. Waste rock (argillite), sediments from road ditch maintenance, peat, and other readily available and suitable materials are used to protect slopes from erosion and to help limit fugitive dust generation. Gullies that form are either cut and recompacted or filled with compacted tailings or rock. Tailings washed from the gullies are problematic because they are too wet to place conventionally. They are typically removed from the placement area, spread to allow excess water to drain out or evaporate, and then mixed and placed with fresh tailings during periods.

Figure 1 Erosion gullies

2.3 *Environmental*

Hecla is committed to being a "…. responsible environmental steward and strive[s] to minimize environmental effects and risks today and for future generations". For Greens Creek, this is evident in the level of effort that has been applied to the design of water management and monitoring systems along with maintaining a small disturbance footprint. The impact of these measures is evident in the overall positive environmental performance of the site and continued renewals of the operating permits. Despite this commitment, there continue to be challenges in relation to dust and water management.

2.3.1 *Tails Tracking*

Every effort is made to contain the tailings within the TDF to minimize the impact to the environment. Equipment transiting through the TDF is exposed to the tailings and has the potential to transport tailings outside of the facility on the tires and undercarriage of the equipment. Numerous methods are employed to minimize tracking.

Truck wash: Every vehicle that enters the facility must exit through a wash system. The water from the wash is routed through degrit basins to the water treatment plant. During rare instances when the tailings are frozen and tracking is minimal, trucks are allowed to exit the facility without being washed.

Dump pads: elevated dump pads are utilized to minimize the amount of tailings that can reach the truck undercarriage while dumping. Operators closely monitor the buildup of tailings on the truck from dumping and from driving on the roads inside the facility. If buildup is excessive, operators will use the truck wash and hand wash the trucks further to minimize contaminant transport on the clean roads outside of the facility.

Tails roads: Roads within the facility are engineered with a running surface of coarse rock. Once the roads are compacted and have been subjected to truck traffic, they are durable and shed water well. Roads are regularly maintained with a rotary broom mounted on a front-end loader to reduce tracking of tailings on equipment tires.

2.3.2 *Dust*

The mine is located within the southeastern portion of the Alaska Coastal Maritimes Zone. Cloudy skies, abundant precipitation, and moderate temperatures characterize the climate and are comparable to those recorded at Juneau (see Figure 2). These conditions result in a low evaporation rate.

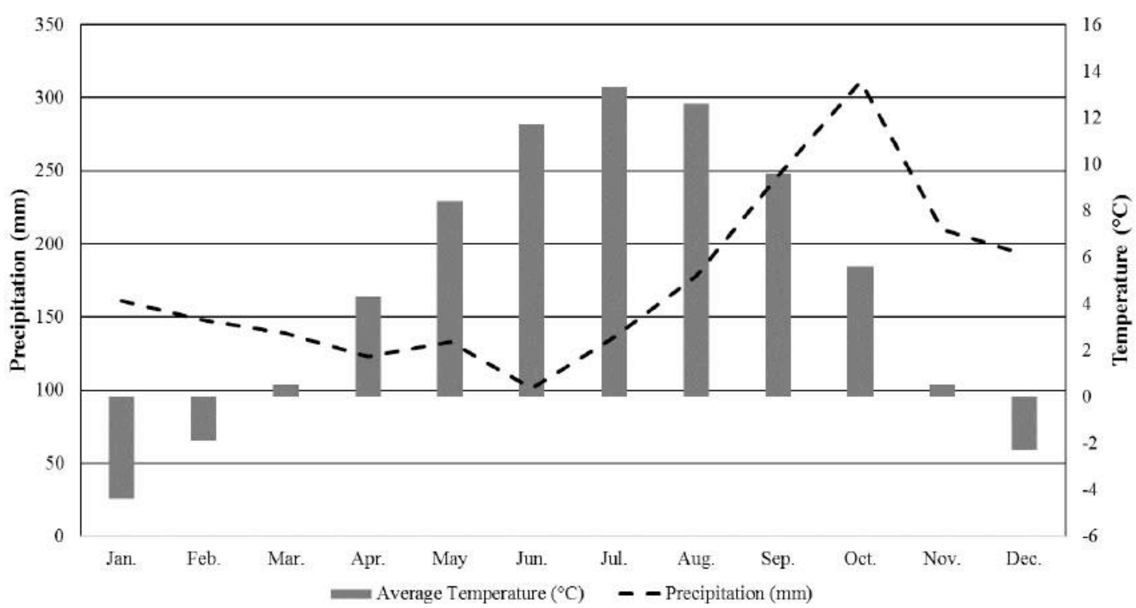

Figure 2 Juneau Average Climate

While approximately 60% of the TDF has been temporarily covered, the challenges associated with the management of dust have increased over time. This is likely a result of the increased height and surface area of the pile, combined with an overlap between cold dry weather and windy conditions (predominately from the north), which occur between mid-November and late March.

Greens Creek completed a dust study over a five-year period during which snow samples were collected from various locations within and around the TDF just prior to the loss of snow cover in the spring of 2007, 2008, 2009 and 2011. The objective was to quantify the location and quantity of tailings dust that had accumulated on the snow pack during the period when conditions for dust loss were considered to be the greatest (December through February). The study found that the predominant wind direction, as well as the locations where tailing were placed in the TDF during the dust-producing weather events, impacted the quantity of tailings material that could potentially contribute to the dust load.

At the conclusion of the study Hecla implemented several mitigation measures, which have since been expanded upon, intended to minimize the potential for fugitive dust generation. These included:

- Installation of snow fencing and concrete block wind breaks on the crest of the TDF.
- Installation of wind fencing on the northern boundary of the TDF and at the upper elevation on the southern end of the TDF.
- Limit snow removal to active placement areas only.
- Cover temporary slopes with rock.
- Apply polymers to temporary outer slopes.
- Hydroseed outer slopes, where appropriate.
- Application of water to select areas of the TDF during freezing temperatures to create an ice layer.

Results (visual observations, snow sample assays, etc.) suggest that these mitigation measures have helped reduce the dispersion of dust at the TDF; however, minimizing the generation of fugitive dust remains an ongoing challenge.

2.3.3 *Water Management*

The mine is located adjacent to Hawk Inlet which discharges into Chatham Straight, an environmentally sensitive marine fjord area Discharges from the mine are regulated under Greens Creek's Alaska Pollutant Discharge Elimination System (APDES) permit, which requires compliance with the Alaska Water Quality Standards (AWQS). To meet this standard, surface runoff and groundwater flows from the TDF, Hawk Inlet Port Facility, Waste Rock Site 23 and the 920 facilities (the mill) are directed to a lined centralized collection system (Pond 7) and treated prior to discharge. The discharge rate from the treatment plant varies seasonally but averages at around 65 l/s annually.

Regular monitoring of water quality occurs across the site, with particular attention given to the TDF. The objective of this monitoring is to collect data that can be used to provide a continuing perspective of the in-pile geochemical processes and enable a comparison between the predicted and actual water quality.

2.4 *Materials*

The location and nature of the TDF poses many challenges that need to be taken into consideration with regards to the "what", "how" and "when" material is placed. These include:

- the need to deliver and place fine grained (silty) material 365 days a year in a wet climate;
- changes in tailings production rate and characteristics;
- containing deleterious materials within the TDF; and
- progressive reclamation.

2.4.1 *Tailings*

Over the past 20 years, the mill throughput has increased from 1,300 tons per day to 2,300 tons per day, approximately 1,700 of which are converted to tailings (see Figure 3).

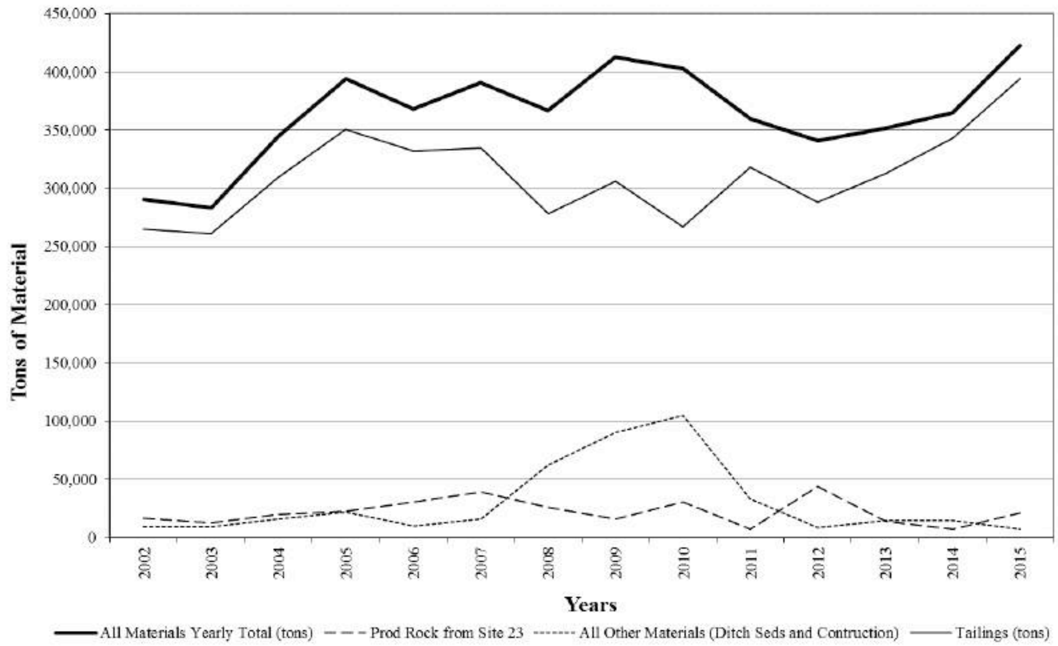

Figure 3 Tailings Production Rate

Tailings are delivered from the mill to the TDF 365 days a year, regardless of weather, tailings characteristics or TDF capacity. The characteristics of the tailings delivered to the TDF vary depending on the ore body encountered; changes in the mill operating procedures, and mill throughput rate (i.e. filter-press residence time). Typically, the tailings are well-graded and consist of approximately 80 percent non-plastic fines. Standard Proctor tests show that the maximum dry density varies from about 2095 to 2390 kilograms per cubic meter, and the optimum moisture content varies from approximately 11 to 14 percent.

When the tailings arrive at the TDF their moisture content is generally one or two percentage points above optimum. From the mill operators' perspective producing tailings with a moisture content at or below optimum is irrelevant – in fact, mill operators add water to the filter-pressed tailings that are mixed with Portland cement and used as paste backfill in the mine. From the tailings operators' perspective, the moisture content of the tailings can and does have a significant impact on how and where the tailings can be placed, how the tailings will behave over the long term, and how long the operators can continue to work in a given area.

Under optimal weather conditions the tailings are dumped at or near the placement area, preferably at the top of a tailings "bench", are spread with a bulldozer downward from the top of a 3:1 slope (see Figure 4), and are track-walked in for compaction (see Figure 5). The target density is at least 90 percent of Standard Proctor density; however, densities much higher (96 percent or more) are typical and preferred. The total placement height may be 5 meters or more, but the material is spread and compacted on the face in lifts that are 0.3 to 0.5 meters thick. To minimize the potential for erosion the upper surface of the bench is typically sloped less than 1 to 2 percent to reduce runoff velocities and associated erosion. At the end of the shift the surface of the tailings is roller-sealed to minimize the infiltration of surface water. Unless dictated by weather conditions, only the last lift of the shift is compacted with the roller.

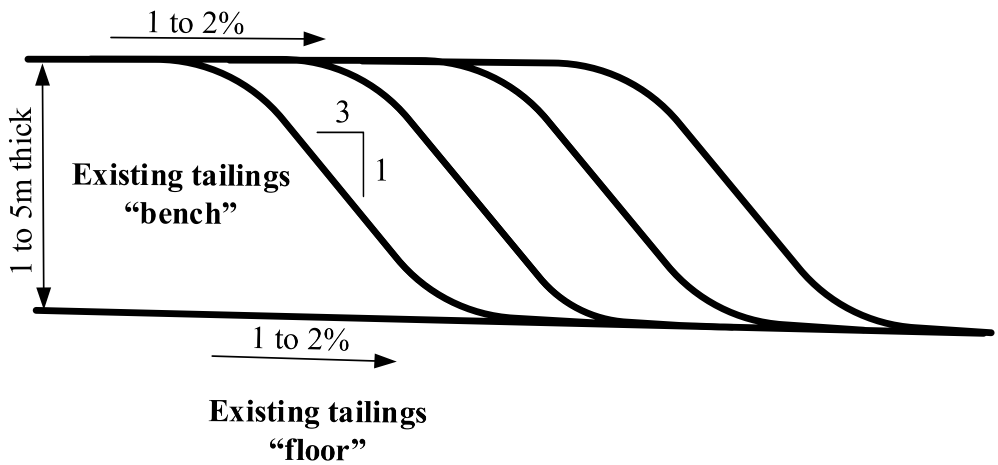

Figure 4 Lift Progression Schematic

Figure 5 Tailings Placement

In wet weather conditions, which often occur in Southeast Alaska, the tailings can quickly gain moisture from precipitation as they are spread and compacted. Tailings placed with excessively high moisture contents will readily pump and rut as they are worked, do not consolidate quickly, can create long-term "soft spots" inhibiting future tailings placement, and can crack as they dry (see Figure 6). To minimize the adverse effects of placing tailings in inclement weather, the tailings operators will often stockpile the tailings in mounds, or "bread loaves," immediately adjacent to the dump pad and seal with the roller as soon as practicable to encourage runoff and minimize precipitation infiltration. Once weather conditions improve (i.e., the rain stops), the stockpile can be cut with the bulldozer and pushed to the final placement area and compacted as described above. Excess moisture in the tailings can usually be worked out by re-cutting with the bulldozer, track walking and rolling again with the vibratory roller, though this requires additional time and effort.

Greens Creek tailings operators have found that placing tailings on slopes allows construction-generated pore pressures to dissipate, allows excess moisture to more readily bleed out, and reduces the likelihood of having large areas of tailings not meet target densities. That being said, if sufficient placement areas are available within the facility, the above described problems are not overly significant. Past experience, including formal research, has shown that given adequate time, even tailings placed in wet conditions with poor drainage and with poor moisture-density control will consolidate and gain strength.

Figure 6 Cracking because of placement of wet material

2.4.2 *Waste Rock*

The majority of the rock utilized within, or for construction of, the TDF is imported from other quarries in Southeast Alaska. This off-island rock is assessed for hardness and geochemical properties prior to being brought to the site. Storage and use is closely monitored to minimize the effect on the environment.

In addition, over the years Hecla has made numerous attempts to identify effective and efficient methods for disposing mine waste rock in the TDF. Hecla's reasons for doing so generally stem from the need to remove all the waste rock used for constructing building pads, roads, and other infrastructure foundations throughout the mine site at reclamation. The waste rock generally consists of argillites and phyllites, both of which have the potential to negatively impact water quality, and neither of which are hard enough to be used as access roads within the TDF, or elsewhere in the facility (e.g., as underground haul roads). The phyllites contain pyrite and other heavy metals which can produce acid runoff if exposed to water and air. The argillite has excess carbonate, which provides buffering capacity to the iron-containing tailings, but can also negatively impact the water chemistry of the surrounding areas. Argillite is often used as interim cover material on outside slopes both to minimize the potential for acid runoff, and to protect the tailings from erosion.

Waste rock can be blended and "co-disposed" with the tailings to minimize risks to water quality. Blending the tailings and waste rock is a time- and labor-intensive process, but there can be several advantages to placing co-disposed tailings and waste rock. First, while the tailings do pose a long-term risk to water quality, the acid buffering capacity provided by the carbonate in the tailings prevents short-term generation of acid runoff. Second, if sufficient quantities of waste rock are mixed in the tailings, the coarser waste rock can increase the shear strength of the material. While not immediately intuitive, the combined waste rock and tailings also have a lower permeability and higher air entry value relative to either pure tailings or pure waste rock.

However, these benefits do come at a cost. Most significantly, the blasting, handling, and weathering processes produces fine-grained material in the waste rock matrix that exhibits plastic behavior. When mixed with the non-plastic tailings, the fine portion of the waste rock slows down the process by which excess water can be pulled from the tailings through mechanical compaction, or consolidation. If the waste rock and tailings are dry relative to the optimum moisture content, and if the weather is warm and dry, these effects can be minimized; however, given the typical wet conditions associated with Southeast Alaska, attempting to place too much co-disposed material can quickly result in heavily rutted placement areas, and poor surface drainage, and can seriously inhibit efforts to effectively place additional material. HGCMC is actively working to develop other cost effective methods for disposing of waste rock in the TDF.

2.4.3 *Peat*

Peat is an extremely prolific material in the area surrounding the TDF and is a waste product generated as the TDF is expanded. Because of the highly compressible and variable nature of the peat, past expansions of the facility have included the removal of all underlying peat. As such, there have been numerous challenges associated with identify appropriate locations for peat disposal. Over the past couple of years, through additional assessments, it has determined that the peat can be used as an interim cover on the TDF that may ultimately have to be removed and disposed of in the TDF prior to a final cover being placed. Placement of peat as a temporary cover mitigates two challenges associated with the TDF: identifying a place to put waste peat, and providing a temporary cover to minimize dust generation from the TDF. Additional studies intending to understand the suitability of the peat as a component of the final pile cover at closure are ongoing and planned for future TDF expansions.

2.4.4 *Ongoing Research*

The TDF is a complex landfill. The Surface-Ops technical staff is always seeking to understand how this facility functions over time and what can be doing to improve its function. The way the facility is operated and maintained has long term consequences. Many things must be understood, and items of interest include the following:

- Erosion prevention techniques
- Cover studies on outside of the pile for reclamation
- Handling of peat wasted from TDF expansion construction
- Densification of tailings at depth over time
- Tailings properties relative to ore body properties
- Effect of aging on tailing properties in regard to cementation
- Tailing properties relative to meteorological conditions
- Peat degradation through a “composting” process over time
- TDF drain system function relation to phreatic surface in base of pile

The issues described above are challenging, but they provide numerous opportunities to push the envelope of the Surface-Ops technical staff to understand what does and doesn’t work in the TDF. Other opportunities abound for identifying cost-effective methods for reclaiming the TDF at mine closure. Because the tailings are not placed as a slurry and stored below water, Greens Creek and their consultants can go back into any area of the TDF and access the tailings material to assess how they are performing from both a geotechnical and geochemical perspective over time. The configuration of the pile allows TDF expansions to be incorporated without significant impacts on operations.

2.5 *People*

As with any organizations, Greens Creek’s people are its biggest asset and its biggest challenge. The mine is spread-out with the mine/mill complex being 12 km away from the TDF and 15 km away from the camp, port and offices. This can result in the personnel at the various locations around the mine not interacting with each other on a regular basis, which can in turn result in a “silo” effect. The operators at the mill and the TDF have extensive knowledge associated with their own roles and responsibilities and the requirements for handling materials at their location. What they don’t necessarily have is a clear understanding of the requirements of others and why these requirements are important. One example is the difference in viewpoint associated with the moisture content of the tailings. As previously mentioned, achieving optimum is a key criterion for the tails operators but for the mill operators, not so much. To overcome this, the mill regularly collects pressed tailings for moisture analysis and reporting. Surface personnel also collect tailings for moisture analysis once the material has reached the TDF. The results are plotted and reported. TDF engineers have also met with mill personnel to discuss operational challenges and share knowledge.

With a focus on the personnel associated with the TDF, there has been surprisingly little turnover in tails operators with only seven operators in the past 20-years and reasonable overlap during several of those operators' tenure. The advantage this has for Greens Creek is the wealth of knowledge that is kept on-site and transferred to new employees. In addition to this, past Greens Creek employees, particularly technical resources, remain available to current employees to offer their knowledge of previous operations/decisions. This is a testament to Hecla's commitment to their staff.

3 CONCLUSIONS

Disposal of tailings in a dry-stack provides several advantages over traditional slurry tailings facility including minimizing the required footprint, minimizing water consumption, creating a more environmentally stable TDF during operations and at closure, and improving access during routine operations.

Challenges associated with operating this type of facility include balancing the need of the tailings operators to have tailings that are close to an optimum moisture content with the need for the mill operators to maximize mill throughput; minimizing the amount of material rehanding due to inclement weather; increased dust management requirements; complex water management systems; potentially poor water quality, and the potential requirement to treat water in perpetuity.

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Filtered Tailings Disposal Case History: Operation and Design Considerations Part II

D. Butikofer, B. Erickson & A. Marsh
Hecla Greens Creek Mining Company, Juneau, AK, USA

R. Friedel, L. Murray & MJ. Piggott
Klohn Crippen Berger, Vancouver, BC, Canada

ABSTRACT: This paper looks at how the design of the Hecla Greens Creek Mine was influenced by and responded to the constraints imposed by the development, placement and operational challenges discussed in Part I of this case history. Design issues such as static and dynamic stability, surface water and groundwater management need to be sufficiently robust to deal with the practical constraints described in Part I. The basis for successfully overcoming these challenges was characterizing the physical behavior of the tailings which differs from a natural soil of similar gradation. Key issues to be dealt with include climate (rain and snow), tight space restrictions leading to limited placement zones, over-wet tailings, minimizing the use of expensive imported materials, seismic response and dealing with heterogeneous glacial, marine and lacustrine foundation conditions. An interesting feature of the Greens Creek tailings is the apparent time related strength gain in the material which greatly improves the dynamic performance of the material.

1 BACKGROUND

1.1 *Overview*

The Hecla Mining Company (Hecla) own and operate the Greens Creek polymetallic mine located on northern Admiralty Island, about 29 kilometers southeast of Juneau, Alaska. A portion of the mine facilities are located within Admiralty Island National Monument.

The physiography of Admiralty Island is characterized by mountains that rise steeply from Hawk Inlet to El. 1,430 masl. The tailings disposal facility (TDF) is located on a relatively flat terrace at about natural ground level El. 40 m to 80 m and is bounded to the east by steep, rugged mountain slopes and to the west by gently-sloped peat wetland that discharges to Hawk Inlet. Muskeg areas and creeks, underlain by marine and glacial sediment, are generally found within the low elevation terrace areas.

1.2 *Geology*

1.2.1 *Regional*

Admiralty Island is located in the Admiralty Subterrane of the larger Alexander Geotectonic Terrane. Admiralty Subterrane bedrock consists of Triassic through Ordovician sediments, meta-sediments and volcanics from 190 to 500 million years old (Ma). Bedrock in the vicinity of Greens Creek Mine generally consists of foliated Triassic (about 220 Ma) marine sediments and meta-volcanics (e.g., greywacke, argillite, phyllite, mafic tuffs, gneiss and schist). These rocks have been deformed by complex northwest-southeast striking folding and are cut by high-angle strike-slip faults and low-angle thrust faults (Apel, 1995) (Terrasat 1991).

1.2.2 *Surficial*

Pleistocene glaciers deposited layers of basal till, ground moraine and outwash sediments in the valley bottoms, that are in-turn overlain by glaciolacustrine and infrequent glaciomarine sediments, colluvium and peat. Glacioestuarine or glaciomarine silt and clay have been identified at the base of eroded terrace remnants below El. 150 m to 180 m in the Greens Creek valley (Terrasat, 1991), indicating that a comparable amount of isostatic rebound and/or tectonic uplift has occurred. Raised beaches have also been identified at 15 m, 30 m, 40 m and 50 m elevations above sea level (Terrasat, 1991). The Greens Creek valley was occupied by at least one glacial lake with associated non-marine silt and clay deposits.

Sub-surface site investigations have been carried out on and around the TDF since the early 1980's and the generalized stratigraphy in the footprint of the TDF, from bottom to top, is summarized as follows (Figure 1):

- bedrock (argillite and graphitic or sericite/chlorite phyllite) at depths from surface to more than 40 m;
- dense marine sandy-clay up to about 20 m thick;
- firm to very-soft lacustrine and/or marine clay up to about 15 m thick;
- dense fluvial or shallow marine sand up to about 7 m thick;
- loose sand or sand and gravel immediately below the peat up to about 4 m thick; and
- amorphous to fibrous peat and organic matter to more than 6 m thick.

Most soil layers lense in and out, vary erratically in thickness, interlayer with each other, and are not always present. Bedrock is exposed directly beneath the peat over about a third of the TDF footprint.

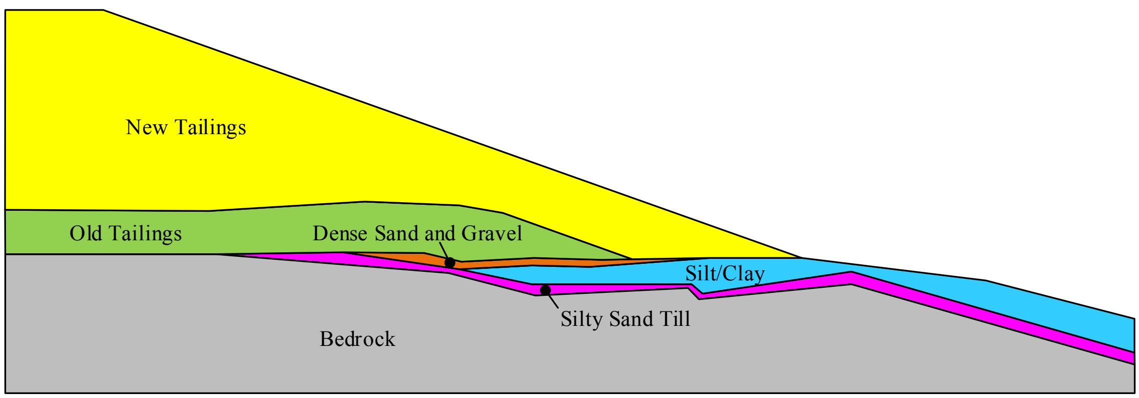

Figure 1 Generalized Stratigraphy

1.3 *Hydrology and Hydrogeology*

In general, surface and groundwater flow(s) in the TDF area are influenced by the local terrain and geology. The facility straddles a hydrological divide, with surface water flow draining northward to Cannery Creek, southward to Tributary Creek, and westward to Hawk Inlet.

In general, the groundwater recharge area for the site is the mountain slope to the east. Groundwater in the upper, unconfined peat and sand aquifer follows a similar drainage pattern to surface water (i.e., drains to Cannery Creek and Tributary Creek) while groundwater in the lower till and bedrock aquifer(s) flows predominantly westward under the TDF toward Hawk Inlet. The upper and lower aquifers are separated in some areas by a relatively low hydraulic-conductivity, discontinuous silty-clay layer. There is no known regional aquifer system in the tailings facility area.

1.4 *Tectonic Setting*

Interaction between the Pacific and North American plates is primarily responsible for the seismicity in Alaska. The northwestward motion of the Pacific plate relative to the North American plate is accommodated by right lateral strike-slip faulting in southeast Alaska on the Fairweather-Queen Charlotte fault system, and by underthrusting and subduction of the Pacific plate along the Aleutian trench, further to the west.

Southeast Alaska contains several regional-scale faults, most of which have been caused by recent and past strike-slip movement. Major faults and lineaments that are potentially significant at the Greens Creek mine site (**Figure 2**) are:

- Fairweather-Queen Charlotte fault system (110 km west);
- Chatham Strait fault (10 km west); and
- Coast Range megalineament (30 km east).

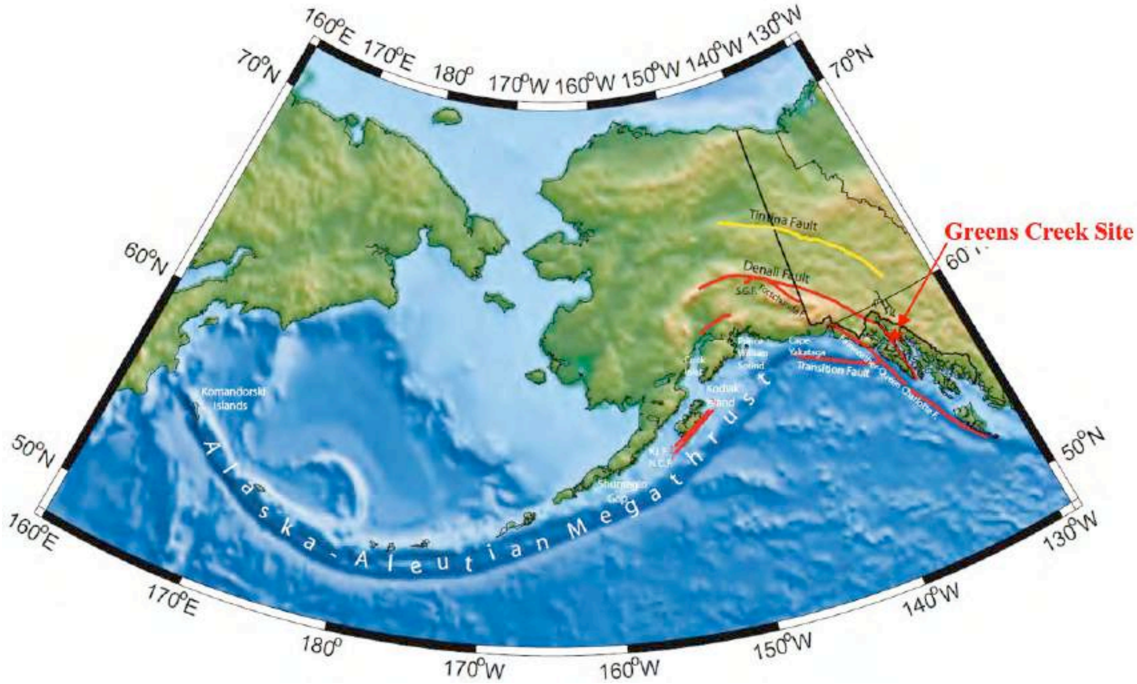

Figure 2 Crustal Faults and Alaska-Aleutian Mega Thrust

1.5 *Development History*

Construction of the TDF commenced in 1988, and tailings placement began in early 1989. In 1993 mine operations were suspended due to low metal market prices however; the mine re-opened in 1996 and the TDF was expanded.

Development of the TDF commenced with construction of various sediment control dams, a surface water collection system, and french drains. Upon start-up in 1989, tailings placement commenced at the northwest corner of the Original Tailings Development and progressed south and east until 1993. Placement was by dumping and spreading in thick lifts with minimal compaction to achieve the permit required 90% standard Proctor density. As the footprint extended south the sediment ponds were buried leaving Pond 6 as the main sedimentation pond at shut down in 1993.

During the shut-down period between 1993 and 1996, lightly compacted tailings became saturated in the prevailing wet climate.

Since the re-start in 1996, tailings in the TDF have generally been placed in thin lifts and compacted to over 96% standard Proctor density (often over 100% standard Proctor density) and has been progressively developed with no less than seven expansions (Figure 3) including:

- Northeast expansion
- West Buttress Expansion
- Southeast 1 Expansion
- Southeast 2 Expansion
- Northwest / Pit 5 Expansion
- East Ridge Expansion

– Stage 3 Phase 1

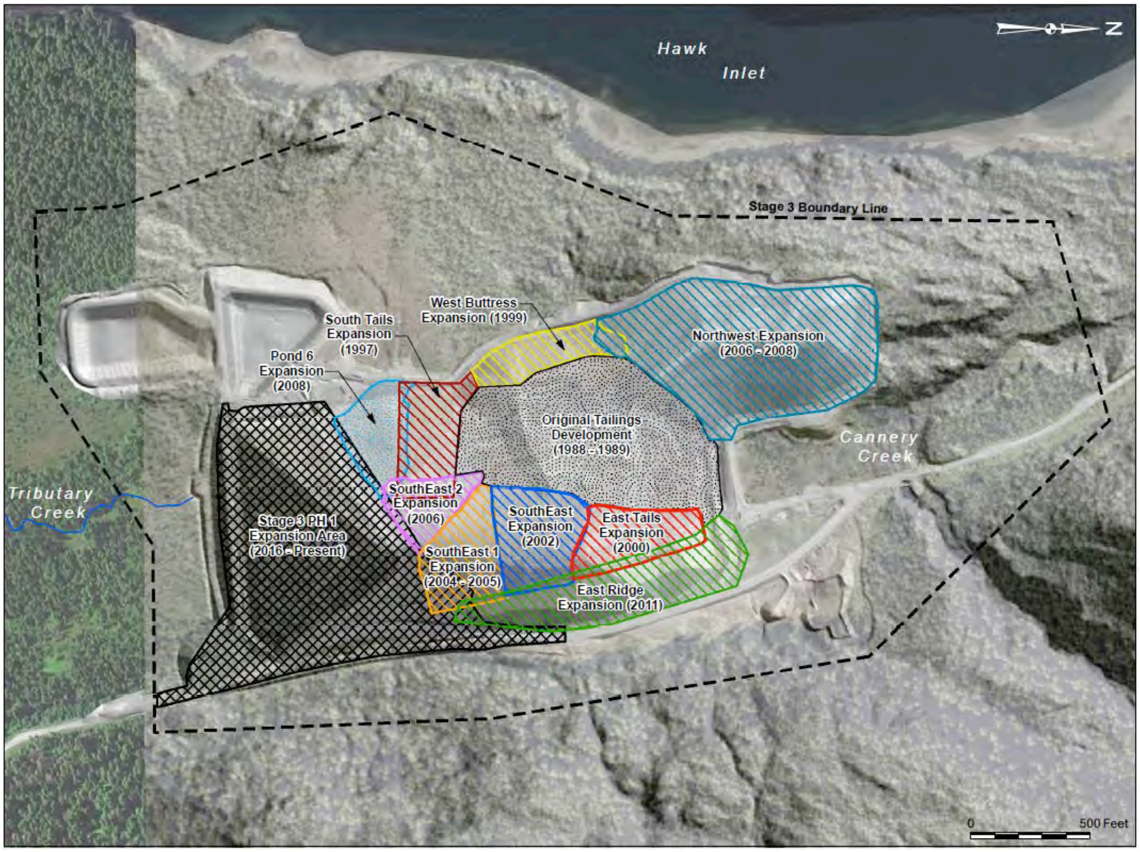

Figure 3 Development Phases

2 DESIGN

The design of any TDF comes with a unique set of site specific challenges. For the Greens Creek mine, these include:
- long-term stability;
- environmental performance; and
- the availability and suitability of various materials.

2.1 *Long-term Stability*

The foundation conditions and seismic behavior of the subsurface materials and tailings is important in determining the long-term stability of the TDF. For the Greens Creek mine, the methodology that has been generally adopted for assessing long-term stability is:
- characterize the ground conditions
- characterize the cyclic loading using input data from the seismic hazard review
- complete deformation analyses (static and dynamic)

2.1.1 *Complex Surficial Geology*

All sites have unique geologic challenges. For Greens Creek, the complexities arise from the variation in thickness and extent of the units, as well as how they interlay with each other. From a design perspective, this variability has been the driver for numerous site investigations over the past 30-years.

As more subsurface information is collected, the understanding of these materials continues to improve. This, along with improved technology for sample collection and testing, has resulted in more detailed site investigations, laboratory testing programs and data analysis. Laboratory index tests, consolidation and triaxial and simple shear strength test data and stress-strain responses, where used to develop soil constitutive relationships and to assign material strengths. Of special concern was to identify soil units that could potentially be driven into their normally consolidated state under static loading (and potentially experience undrained failure) and also to identify soils that could lose strength during cyclic loading.

The foundation unit that has a major influence on both the static and seismic stability of the TDF is the marine sandy-clay. An undrained shear strength (S_u) profile through this unit was developed at several locations by CPT and Shelby tube sampling and it generally shows a trend of decreasing S_u to about 10 m depth and subsequent linear increase beyond that depth. This, coupled with consolidation testing, implies desiccation as the likely primary cause of overconsolidation at shallow depth. Historical lowering of the water table may also have played a role.

In some areas, this marine clay was found to be stiff under initial site investigation but it was predicted to become normally consolidated under the weight of the tailings. Consideration of this has been incorporated into the various analyses.

2.1.2 *Seismic Behavior*

The seismic behavior of the soil units has been investigated through SPT and CPT site investigation programs, laboratory testing on undisturbed samples of cohesive materials and subsequent analysis of the collected data. This type of investigation and analysis is well understood.

The seismic behavior of the tailings has historically been more difficult to assess as SPT and CPT were found to be not as well suited to the assessment of this material. SPT is a technique designed for use in natural granular soils or soils with up to about 35 percent silt content. In these soils, pore pressure generated by SPT can be accounted for, or is not a major factor. However, the tailings contain over 80 percent silt and are unsuited to SPT testing. The tailings CPT soil behavior index indicates that the tailings classify as a clayey non-liquefiable soil under conventional CPT analysis. However, it is known that relatively low plastic milled tailings can behave very differently from natural soils that form the main basis for the soil behavior index.

To further assess tailings liquefaction potential, a series of tests were conducted on samples re-constituted in the laboratory. The tailings were tested in both cyclic triaxial and cyclic shear box apparatus, using material with as-placed moisture contents and at a starting density as low as 88 percent of standard Proctor density (SPD), which is below the specified minimum for placement (90 percent SPD). The laboratory tests indicate that the tailings resistance to liquefaction was marginal under the maximum design earthquake loading.

Following the above testing, another program to collect and test undisturbed in-situ tailings samples was completed. This included excavation and exposure of tailings that had been buried in the TDF for up to 15-years, including the "old" very lightly compacted tailings. Shelby tube samples were manually pushed into excavated surfaces and carefully shipped to the testing laboratory at the University of California Berkeley where consolidation, monotonic simple shear, cyclic simple shear and post-cyclic simple shear tests were completed.

Field density testing and laboratory consolidation and strength design found that the relative compaction of the older tailings increased with depth and the in-situ undrained strength and density was significantly greater than expected based on the average placement relative compaction of 90 percent in the old tailings and about 96 percent in the new tailings. While increasing density as the material consolidates under increasing load is expected, the consolidation testing results indicated that the tailings samples were over-consolidated and therefore consolidation alone could not account for the in-situ density and strength results and that other processes must contribute to this. These processes are possibly: matrix suction; secondary compression; and/or aging; and/or geochemical processes within the pile. The tailings have a significant gypsum content and observations of the pile during dry periods often showed a white precipitate forming on evaporative exposed surfaces, it is hypothesized that light cementation due to precipitation on these salts could account for the apparent overconsolidation and relatively high undrained strengths.

With respect to the laboratory results, they supported the observation that the tailings have a clay-like behavior under cyclic loading. This was noted in the stress-strain behavior and the development of shear strains over several cycles. The tailings appear to have relatively high resistance to cyclic loading, with the older tailings having grater cyclic resistance than the newer tailings. Compared to previous tests completed on reconstituted samples, the cyclic resistance observed from the in-situ samples was significantly greater (Figure 4) suggesting that in-situ fabric is an important component of the strength.

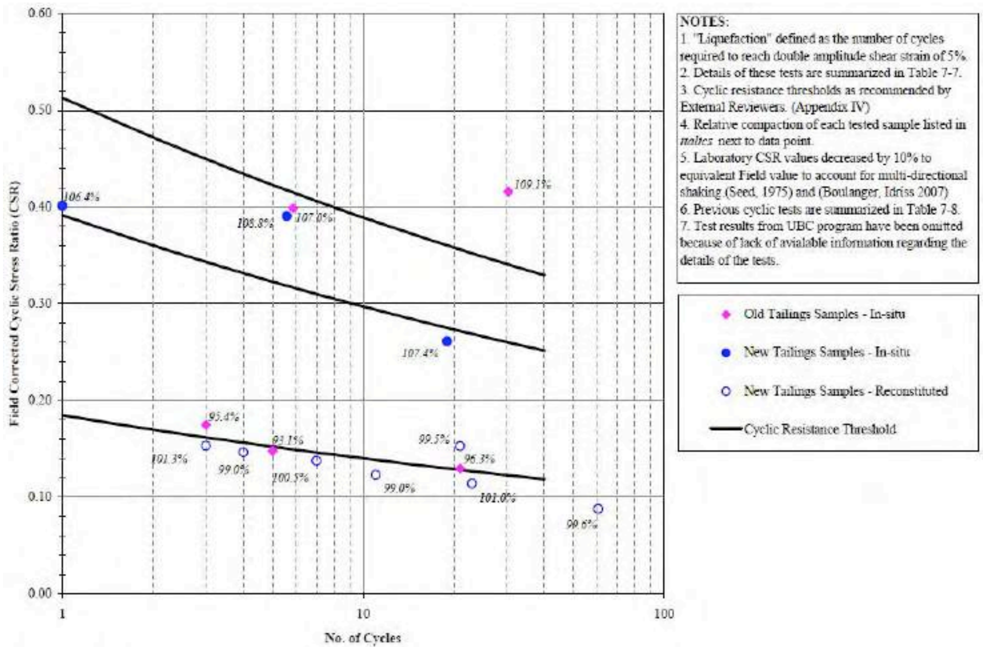

Figure 4 Cyclic Testing on New and Old Tailings

In general, the pre-cyclic shear strength of the tailings is relatively high when compared to other natural materials if placed with equivalent compactive effort. This is believed to be result from weak cementation of the particles as discussed previously.

A summary of the material strengths for the TDF is given in Table 1.

Table 1 Material Strength

Material	Unit Weight (kN/m^3)	V_{s1} (m/s)	Poisson's Ratio		Peak Friction Angle, ϕ (°)		Residual Friction Angle, ϕ (°)	Strain to Residual Strength (%)	Cohesion (kPa)
			Static	Dynamic	Static	Dynamic	Dynamic	Dynamic	
Tailings	20.1[1] (24.0)	223	0.3		35		-	-	40
Waste Rock	21.8	223	0.3		40		-	-	0
Peat	10.5	117	0.2		27		-	-	0
Clay – Desiccated (Upper)	18.9	322	0.49		0		-	-	144
Clay – Desiccated (Lower)	18.9	237	0.49		0		-	-	72
Clay – Normally Consolidated	18.9	196	0.3	0.49	S_u/σ'_{v0} = 0.24		S_u/σ'_{v0} = 0.12	5	0
Glacial Till	18.9	351	0.2		33		-	-	0
Bedrock	20.4	V_s = 762	0.3		N/A		-	-	N/A

[1]Dry unit weight. Value in parentheses is saturated unit weight.

2.1.3 *Deformation Modelling*

Because there is no water stored on top of the tailings, "failure" for the Greens Creek TDF is assessed in terms of deformation (rheology controlled slide) rather than the release of water and tailings typically associated with an embankment failure. While there is not expected to be a loss of life from such a potential "failure", the release (or sliding) of tailings from the TDF footprint has the potential to create environmental damage to the surrounding National Monumental wetlands, Tributary Creek and Cannery Creek.

Based on this, the hazard classification applied to the design of the TDF is "high". This classification influences the seismic design loading conditions and the facility is currently designed for a 2,500-year return period design earthquake, which produces a peak ground acceleration on firm ground of 0.34g from a maximum design earthquake of M7.6. Combining these seismic conditions with potentially liquefiable/softening materials presented in some areas within the TDF footprint resulted in the need for more complex modeling.

One such location is the current expansion (Stage 3 Phase 1) where the presence of both a deep bedrock trough and two foundation silty clay soil units noted as being potentially susceptible to strength loss during, or following, the design earthquake event were identified. It was thought that the bedrock trough could potentially provide some level of confinement due to its orientation compared to the TDF. This was considered in both the 2D and 3D modeling (static and dynamic) and it was found that the geometry and dimensions of the trough were indeed key to the displacement predictions.

The modelling results indicated that while there could be deformation under the maximum design earthquake conditions, it would not result in a "flow slide" and potential impacts of the deformation would be limited to:

- Damage to the perforated pipes in the drainage system: in the event that this occurred, the other elements of the drainage system (service layers and rockfill drains) are expected to continue to function as designed and the overall performance of the system will not be impacted.
- Damage to the wet wells where the water collected by the drainage system is: repair to the wet wells may be required following a seismic event but they are relatively accessible for maintenance.
- Movement of natural ground downstream of the TDF toe: up-thrusting and deformation of natural ground may occur, however deformations are predicted to be within the existing lease boundary.

2.2 *Environment*

2.2.1 *Water Management*

The TDF is underlain by a network of french drains that direct water from the base of the pile to the water treatment plant. The french drains are comprised of coarse sand and drain rock surrounding a perforated drain pipe. The layout of the french drains has developed over time and is generally sparser in the older areas of the TDF and more regularly spaced, and in some locations there are blanket drains, under expansions that have occurred since the late 1990's. To minimize environmental release of seepage, potentially not collected by the drain system, a series of soil-bentonite walls have also been installed.

The TDF is designed with "clean water" diversion ditches around the perimeter of the facility, and "dirty" ditches within the facility that collect water that has come in contact with the tailings. These ditches are designed for the conveyance of operational and storm flows and are lined to minimized seepage.

As noted in Part I, discharges from the mine are regulated under Greens Creek's Alaska Pollutant Discharge Elimination System (APDES) permit, which requires compliance with the Alaska Water Quality Standards (AWQS). To meet this standard, surface runoff and groundwater flows from the TDF and other facilities are directed to a lined centralized collection system and treated prior to discharge.

2.2.2 *Groundwater and Seepage Control*

During early development of the TDF, there was a reliance on the presence of low permeability soils, soil-bentonite walls, french drains and drainage ditches for seepage containment. In the late 1990's / early 2000's, expansions extended into areas where the thickness of the low permeability native soils was limited, or not present, and shallow bedrock was evident. This instigated the inclusion of a low permeability synthetic liner.

The liner system includes above and below liner drains, which allows collection of dirty water, reduces hydrostatic pressure in the tailings, and reduces the potential for leakage into the foundation. The low permeability of the tailings combined with the drainage provided by the above liner drain results in a piezometric surface within the TDF that is about 10 m above the base of the pile at the highest point in the pile, tapering towards ground surface at the perimeter.

The tailings, underdrain and liner effectively create a double liner system. The liner is comprised of (Figure 5):

- Sand Bedding Layer – to prevent the likelihood of liner punctures and tears from an irregular foundation.
- 80 mil HDPE textured geomembrane liner –to manage seepage of tailings contact water.
- Sand Interlayer (where required) - to increase liner interface friction for tailings pile stability.
- Geocomposite – to protect the liner from damage after installation and to provide a drainage flow path.
- Sand Service Layer – to provide further liner protection from vehicle traffic, prior to full scale tailings placement.

While this system has been successful in relation to seepage containment, it introduced a new design problem with the geocomposite to geomembrane interface residual friction strength being reduced and locally being the controlling stability layer. In critical locations, such as the embankment toe, a sand layer between the geocomposite to geomembrane has been incorporated.

Inclusion of the liner is a practice that continues to be applied for expansions of the TDF (Figure 6).

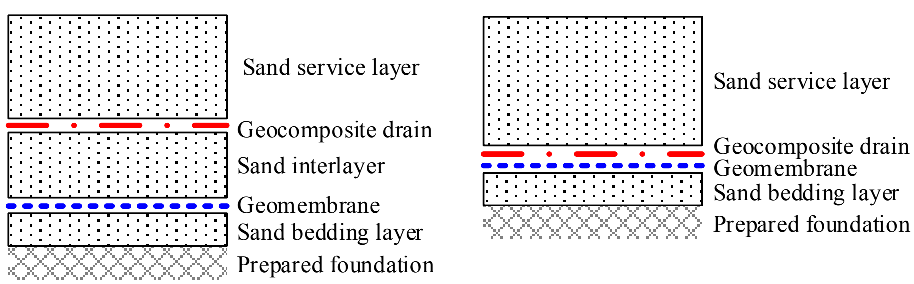

Figure 5 Typical Liner Configuration

Figure 6 Typical Liner Placement

2.3 *Construction Materials*

Materials available for construction of expansions at Greens Creek that cannot be sourced on-island need to be obtained from off-island sources and barged to the site. In determining what materials are available, the following is considered:

- Is the material to be placed within containment? If so, geochemistry of the material is not a key consideration because water that comes in contact with it will be collected by the drainage system and conveyed to the water treatment plant.
- Is the material to be placed outside containment? If so, this material will need to meet the geochemical requirements for discharge to the environment.
- How much material is required? There are a limited number of sources for material that meet the geochemistry requirements for placement outside of containment. This means that if large volumes of "clean" material is required, it will likely need to be sourced off-island.

2.3.1 *On-Island Materials*

On-island materials available for construction include sand, waste rock and tailings. There is a limited volume of sand available on-island so while it meets the geochemical requirements and can be placed outside containment, it is often not used for large construction projects and is retained for smaller operational activities.

Neither the waste rock nor tailings meet the geochemical requirements to be placed outside containment. The locations they can be used as construction materials is therefore limited to within the tailings area. They both, especially the tailings, have a structural function particularly on the perimeter slopes. The behavior of the waste rock and tailings are therefore key to the design.

Stability assessments for the TDF are based on the tailings parameters however it is noted that, if available, Operations can replace the tailings with waste rock without compromising safety factors. The in-situ properties of the tailings are given in Table 2.

Table 2 Tailings Properties

Property	Value
Gradation (Figure 7)	78% to 96% by weight passing No.200 sieve
Specific gravity of solids	3.13 to 3.50 (mean = 3.23)
Maximum dry density (standard Proctor, Figure 8)	18.9 to 21.5 kN/m^3 (mean = 20.3 kN/m^3)
Optimum moisture content (standard Proctor, Figure 8)	14.3% to 15.3% (mean = 14.8%)
Field hydraulic conductivity	2 x 10^{-7} cm/s

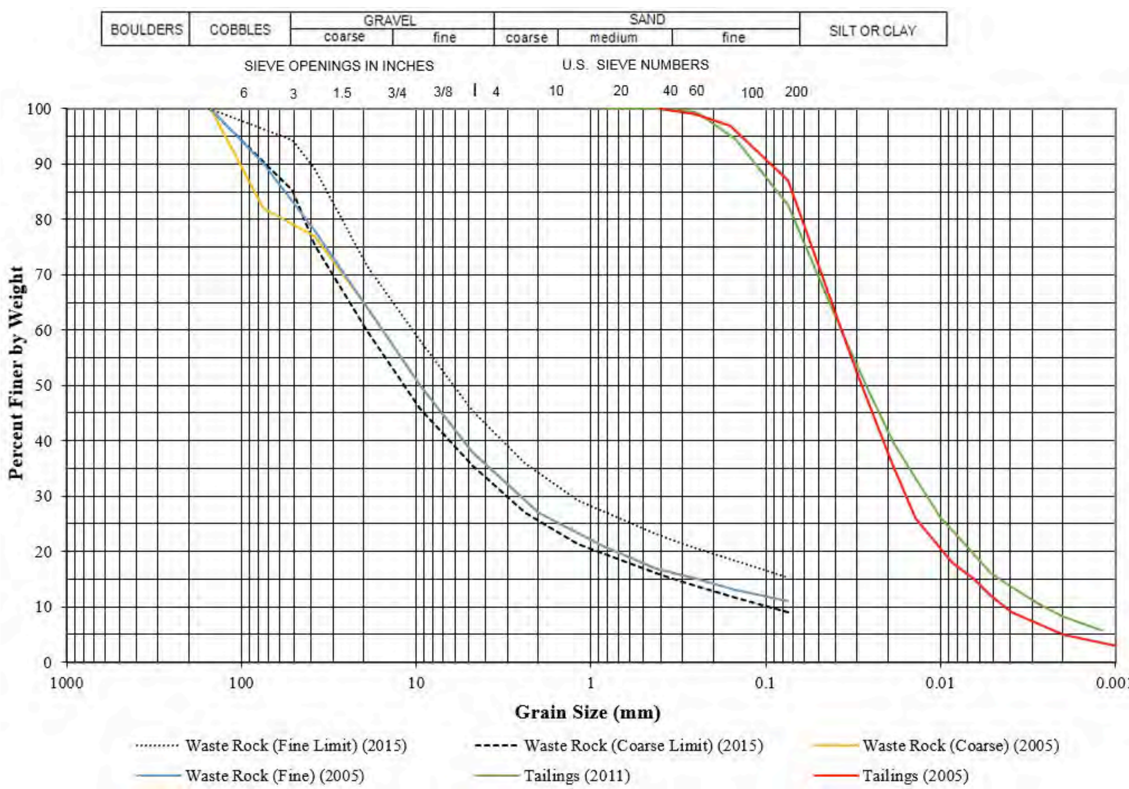

Figure 7 Tailings and Waste Rock Gradations

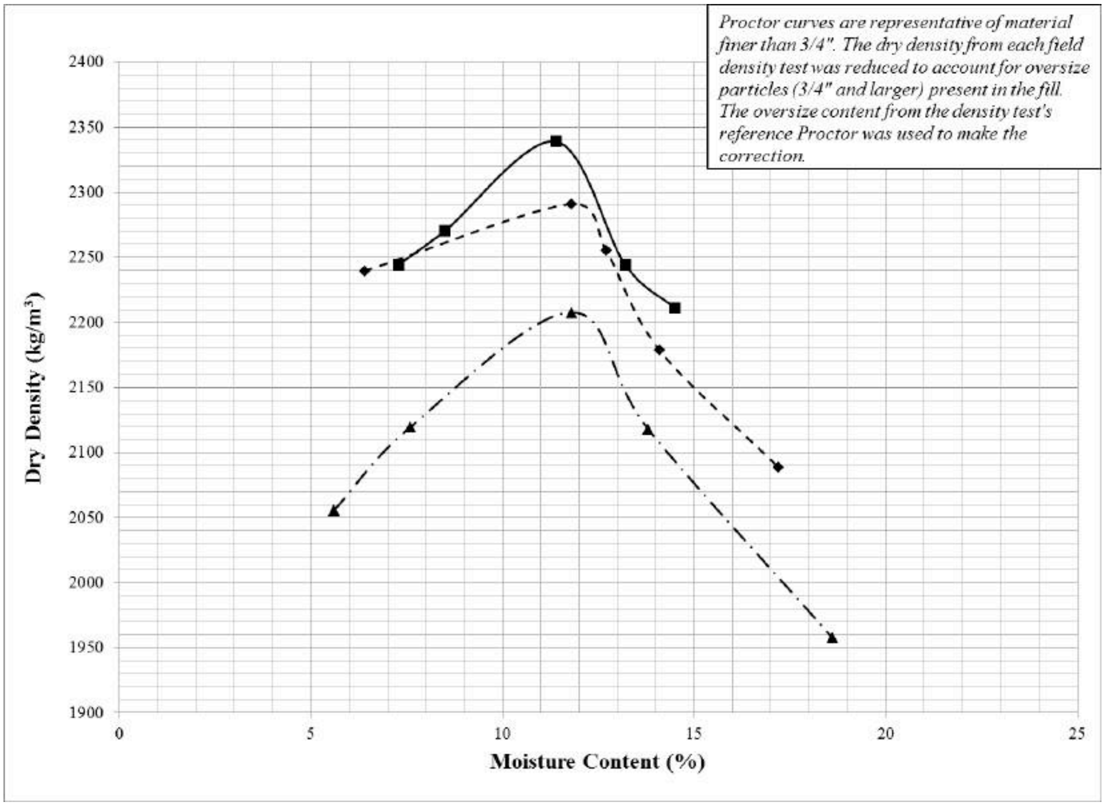

Figure 8 Tailings Proctor Curves

2.3.2 *Off-Island Materials*

Materials generally sourced off-island include the sand and gravels for the below liner drainage system as well as manufactured components such as drain pipes and wet wells. For the off-island granular materials, the source firstly needs to undergo geochemical characterization to confirm that the site-specific criteria will be met. Once this testing has been completed and a source approved, the time consuming and costly process of transporting the material commences. All off-island materials need to be barged to the Hawk Inlet port and unloaded for temporary storage at the port before being transported to the final construction location. The same transportation process applies to all manufactured materials.

The remoteness of the Greens Creek mine and the associated transportation challenges impact the design and construction in many ways. One example is the limited use of cast-in-place concrete. While there have been instances where cast-in-place concrete would have been a neat and convenient solution to a specific construction challenge (i.e., connection of the liner to a culvert headwall) alternate options were developed be the design team.

Other ways the transportation challenges impact design and construction include:

- Increased need for accurate quantities estimates to limit over- and/or under-ordering.
- Sequencing of construction activities to accommodate the ore concentrate shipping schedules.
- Coordination of storage and transportation of barged materials due to the limited space at the port and construction site.
- Sequencing the arrival and departure of large equipment to suit construction activities.

3 PERFORMANCE MONITORING

Because the pile is being continuously built, only a limited amount of performance monitoring can be done. Piezometers are placed throughout the pile to monitor water pressure and saturation and inclinometers are placed at the toe in key areas.

These data are used to calibrate stability and deformation models. To date, performance of the pile has been excellent with only occasional crack development and surface slumping, often related to wet weather material placement.

Drain flows and water quality are continuously monitored and, from a civil geotechnical design perspective, provide an indication of the continued functioning of the underdrains.

A key to monitoring is the frequent visual inspections and periodic engineering inspections.

4 CONCLUSIONS

The Greens Creek TDF is constructed in a high seismic and wet environment on highly heterogeneous foundation soils, some of which are susceptible to strength loss on seismic loading. Some foundation soils are loaded beyond their pre-consolidation pressure, which results in the potential for undrained shear failure.

The keys to the successful operation of the TDF are as follows:

- Tailings are carefully compacted in the outer structural zone, with inner areas left for wet or inclement weather placement.
- Under drains, combined with the very low hydraulic conductivity of the tailings, maintain a low water pressure in the pile, especially near the pile toe.
- Rapid aging of the tailings, possibly from chemical processes within the pile, produces an apparent over consolidation that greatly improves the tailings resistance to seismic loading.
- There are no design elements in the pile that are relied on to withstand large deformations (pipes are included only as a convenience and granular underdrains would still function). This means that relatively large deformations can be sustained by the TDF without significant loss of performance.

Some lessons learned from Greens Creek include:

- The tailings areas almost always grow larger than expected as more ore is discovered, hence designs need to have a high degree of flexibility.
- Gather more data than you think you'll need in peripheral areas to facilitate future expansions.
- Be flexible in your designs to facilitate future expansions
- Generate conceptual designs for multiple expansions well in advance of needing them to help site infrastructure; ponds, ditches, roads, pipelines, electrical infrastructure, etc.
- Study and understand environmental ramifications of expansions; i.e., dusting as the pile height increases.
- Understand closure complications well before you get there; pile geometry, slope length, vegetative effects, etc.
- Understand the long-term aging properties of the tailings since this can be an important contributor to the TDF performance over the operating life of the facility.

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Introducing: *Guidelines for Mine Waste Dump and Stockpile Design*

John Cunning
Golder Associates Ltd., Vancouver, BC, Canada

Mark Hawley
Piteau Associates Engineering Ltd., North Vancouver, BC, Canada

ABSTRACT: The authors are pleased to introduce the latest book sponsored by the Large Open Pit project, *Guidelines for Mine Waste Dump and Stockpile Design*, which is a comprehensive and practical guide to the investigation, design, operation, monitoring, and closure of mine waste dumps, dragline spoils, and major stockpiles. The Guidelines provide a summary of the current state of practice and are intended to help mine operators, geotechnical practitioners, and non-specialists improve their understanding of the factors that can influence the stability of waste dumps, stockpiles, and dragline spoils. These Guidelines have been developed by a consortium of geotechnical consultants and individuals including Piteau Associates Engineering Ltd., Golder Associates, Schlumberger Water Services, Sherwood Geotechnical and Research Services Inc., Dr. Oldrich Hungr of the University of British Columbia, and Dr. G. Ward Wilson of the University of Alberta. Sponsorship comes from the Large Open Pit project, initiated in 2005, which is an industry sponsored and funded international research and technology transfer project focused on the stability of large open pit mines.

1 INTRODUCTION

Mine waste dumps associated with large open pit mines are some of the largest man-made structures on Earth, with a potential impact to the safety of people, equipment, and the environment if not operated properly. While there are numerous successful mine waste dump operations worldwide, there are many cases of large-scale instability with significant adverse consequences. Comprehensive investigation, design, and monitoring programs are often carried out for the source pits and for nearby tailings facilities; however, similar programs for waste dumps are not always as thorough.

There are many steps required to advance a waste dump from the conceptual stage supporting an initial mine plan through to ongoing safe operations, and further work is required to implement closure. *Guidelines for Mine Waste Dump and Stockpile Design* has been prepared with the input of many mine waste management professionals and summarises the key steps in the development process. Examples of current industry best practices are also shared in an effort to advance our understanding of waste dumps, stockpiles, and dragline spoils with the goal of improving their reliability and safety. Figure 1 shows an active mine waste dump with a large mine haul truck dumping waste rock. Monitoring is being carried out for this waste dump and includes a wireline extensometer to measure ongoing deformations at the crest.

Figure 1. Mine waste dump under active operation, with simple wireline extensometer to monitor ongoing crest displacements.

1.1 *Good waste dump examples*

At the outset of this book project, an informal review found that worldwide there are many large open pit operations with a long history of mine waste dump operations. Examples include the Bingham mine in Utah, the Elk Valley coal mines in British Columbia (BC), and numerous large open pit operations in Peru and Chile, including the Pierina mine waste dumps shown on the book cover. Dragline operations are active at many sites in Australia, Canada, and the USA. Stockpiles are common as temporary dumps and are a typical part of many mining operations. All open pit operations, regardless of their size, require waste dumps. The book was prepared with the intention of covering the full range of mine waste dumps, dragline spoils, and stockpiles throughout their complete life cycle.

1.2 *Not so good examples*

Mine waste dumps can be very large structures, and the consequences of failure can be extreme. Much of our current understanding of mine waste dump behaviours was derived from investigations following significant dump failure events, including:

- Aberfan, UK, 1966 – dump failed and killed 116 children and 16 adults at a school in the town below the mine
- Balmer mine, near Sparwood BC – 1968 dump failed above Highway 3, striking a car and resulting in two fatalities
- Quintette mine, 1987 – dump failed, resulting in 5.6 million m^3 material with runout of over 2 km blocking local creeks

In response to past failures, a number of reports and interim guidelines have been produced to support dump design and operation. In the USA in 1975, guidelines were prepared by Mining Enforcement and Safety Administration (MESA, predecessor to Mine Safety and Health Administration) that included a design manual for coal facilities. In 1977 the *Pit Slope Manual* was prepared by the Canadian Centre for Mining and Metallurgy (CANMET 1977) and included a section on waste dumps.

In the 1990s in response to series of dump failures in BC, the British Columbia Mine Waste Rock Pile Research Committee (BCMWRPRC), composed of local mining companies, CANMET, and the BC Ministry of Energy and Mines, commissioned a series of interim guidelines that are still in use today. A total of 11 reports were issued under the auspices of this committee.

1.3 *The Large Open Pit project*

The Large Open Pit (LOP) project is an international research and technology transfer project focused on the stability of large slopes associated with open pit mines. It is an industry sponsored and funded project that was initiated in 2005 and managed by Dr. John Read under the auspices of Australia's Commonwealth Scientific and Industrial Research Organisation (CSIRO). At the time the guidelines for waste dumps book project was initiated, the LOP project included these partner sponsors: Anglo American plc; AngloGold Ashanti Limited; Barrick Gold Corporation; BHP Chile; BHP Billiton Innovation Pty Limited; Corporación Nacional de Cobre Del Chile (Codelco); Compañía Minera Doña Inés de Collahuasi SCM; De Beers Group Services (Pty); Debswana Diamond Company; Newcrest Mining Limited; Newmont Australia Limited; Ok Tedi Mining Limited; Technological Resources Pty Ltd (Rio Tinto Group); Teck Resources Limited; Vale; and Xstrata Copper Queensland.

One of the initiatives of the LOP project has been to develop a series of guidelines to capture and communicate the current state of practice in the investigation, assessment, design, and development of slopes associated with large open pits. The following LOP project sponsored books in this series have been published to date:

- *Guidelines for Open Pit Slope Design* (Read & Stacey 2009)
- *Guidelines for Evaluating Water in Pit Slope Stability* (Beale & Read 2013)
- *Guidelines for Mine Waste Dump and Stockpile Design* (Hawley & Cunning 2017)

The next book in the series, *Guidelines for Design of Open Pit Mines in Weak Rocks* (editors Martin & Stacey, in press), is expected to be published later in early 2018.

Guidelines for Mine Waste Dump and Stockpile Design draws on an experienced team of practitioners and was edited by Mark Hawley (Piteau Associates Engineering Ltd.) and John Cunning (Golder Associates Ltd.), with the assistance of an editorial subcommittee that included:

- Geoff Beale (Schlumberger Water Services)
- James Hogarth (Piteau Associates Engineering Ltd.)
- Andy Haynes (Golder Associates Ltd.)
- Peter Stacey (Stacey Mining Geotechnical Ltd.)
- Stuart Anderson (Teck Resources Ltd.)
- Claire Fossey (Golder Associates Ltd.)

The development of the book was supported by respected geotechnical and hydrogeological consulting companies and professors including:

- Piteau Associates
- Golder Associates
- Schlumberger Water Services
- Sherwood Geotechnical and Research Services
- Dr. Oldrich Hungr of the University of British Columbia
- Dr. G. Ward Wilson of the University of Alberta

1.4 *Survey of waste dumps and database of incidents*

A database of existing mine waste dump incidents was published in the 1992 BCMWRPRC guidelines. An updated summary of mine waste dump incidents in BC between 1968 and 2005 is included in Appendix I of the book.

As part of developing the book content, a survey was undertaken in 2013 to gather data on existing mine waste dump operations. Appendix II presents a summary of the survey data and an analysis of the key results.

1.5 *The Guidelines*

The mandate set out by the LOP project was to develop a practical reference book to help mine staff and consultants understand the myriad of factors that influence the stability of waste dumps, stockpiles, and dragline spoils. *Guidelines for Mine Waste Dump and Stockpile Design* is a comprehensive and practical guide which includes 16 chapters presenting the investigation, design, operation, monitoring, and closure of mine waste dumps, dragline spoils, and major stockpiles.

Brief descriptions of the key content in each chapter of the book are presented in the following sections.

2 BASIC DESIGN CONSIDERATIONS

This chapter presents an overview of design processes and main inputs by stage of design for waste dumps and stockpiles. Design of waste dumps and stockpiles requires the consideration of many interrelated factors that can change through the life of the mine. Key site selection factors are organised into the following groups:

- Regulatory and social
- Mining
- Terrain and geology
- Environmental
- Geotechnical
- Fill material quality
- Closure

The chapter includes a description of key objectives by stage of design following the stages in a mine project summarised in Figure 2. Site selection processes are described and a summary table of the suggested level of effort by component for each project stage is included.

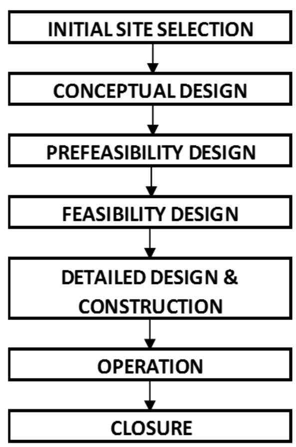


Figure 2. Key project stages.

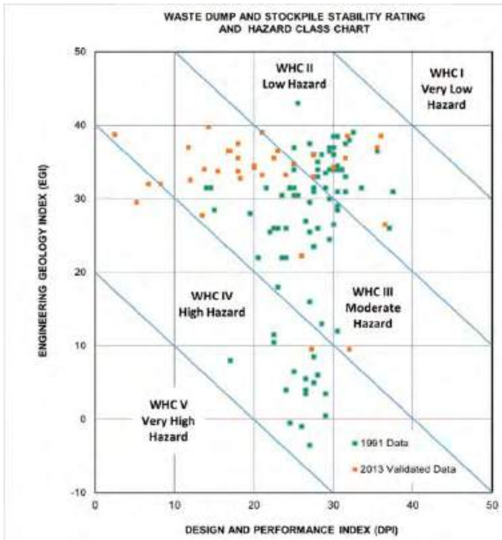
3 WASTE DUMP AND STOCKPILE STABILITY RATING AND HAZARD CLASSIFICATION

A new stability and hazard classification system, called the Waste Dump and Stockpile Stability Rating and Hazard Classification (WSRHC) system, is presented. This classification is based on a total of 22 factors organised into 7 groups. While this is a large number of factors, most are intuitive and should be easily obtained by the team familiar with the mine site and the planned waste dump design. The WSRHC system generates two indices:

- Engineering Geology Index (EGI)
- Design and Performance Index (DPI)

The combination of the two indices results in a Stability Rating (WSR) between 1 and 100. The numerical WSR scale is divided into five descriptive Hazard Classes (WHC), which range from very low to very high hazard (Figure 3). The rating and hazard class should be reviewed at each stage of the waste dump development.

An understanding of the waste dumps stability rating or hazard class can be used to define the level of effort that is appropriate for investigation, design, operation, and closure.



WSR	WHC	INSTABILITY HAZARD
80-100	I	Very Low Hazard
60-80	II	Low Hazard
40-60	III	Moderate Hazard
20-40	IV	High Hazard
<20	V	Very High Hazard

Figure 3.a) Waste Dump and Stockpile Stability Rating and Hazard Classification chart, and b) WSR rating ranges and WHC instability hazard classes (Hawley & Cunning 2017).

4 SITE CHARACTERISATION

Following an initial review of the WSRHC, initial site selection processes and site characterisation are undertaken for early project stages of mine waste dump, stockpile, or dragline spoil design. Chapter 4 describes typical site characterisation programs to support design. Site characterisation typically starts with conceptual, desk-top studies to support alternatives assessments at early project stages, followed by planning and undertaking of field investigations for advanced project stages. Key site characterisation study areas discussed include:

- Physiography and geomorphology
- Geology
- Natural hazards
- Climate
- Geotechnical

Planning and methods for field investigations that consider the level of design, dump classification, and risk are discussed. Many investigations are staged or phased as the overall mine project develops and the results of preliminary site characterisation are fed back into updates to the dump classification and used to advance to the next stage of design where further investigations may be required.

5 MATERIAL CHARACTERISATION

Along with site characterisation to establish the overall setting and context of the facility, field investigations to obtain in situ data and samples for further laboratory testing are carried out for material characterisation. Chapter 5 describes material characterisation to support planning and design of mine waste dumps and large stockpiles. Material properties are assembled to support the analyses and design processes. This includes gathering the key properties of the soil and bedrock foundation materials, and the properties of the waste dump and stockpile materials. The chapter discusses laboratory testing techniques, including the use of large triaxial tests on rockfill materials and interpretation of laboratory results.

6 SURFACE WATER AND GROUNDWATER CHARACTERISATION

Surface water and groundwater are important considerations in the planning, design, and operation of a mine waste dump or major stockpile. Chapter 6 describes characterisation work to support hydrologic and hydrogeologic evaluations both in and around planned areas. Planning for surface water and groundwater investigations with discussions on the level of effort by each project stage are included. Development of the conceptual hydrogeological model is reviewed and methods for the hydrogeological modelling of the dump, stockpile, and foundation are presented. Modelling is used to support understanding of pore water conditions in the dump and the foundation, and to support design of the diversion of underflow water management systems.

7 DIVERSIONS AND ROCK DRAINS

Following up on surface and groundwater characterisation, with waste dumps covering large areas of the mine site, a range of measures to control, divert, or manage water around or through the dump are required. Procedures for designing diversions are summarised. Rock drains are commonly used in waste dumps, and design principles based on the CANMET Rock Drain Research Program (1992–1997) are summarised. Figure 4 presents a rock drain being created through operations by end dumping. Other drainage elements, including drainage blankets, trench drains, chimney drains, and toe drains, are described.

Figure 4. Rock drain created by end dumping method (Hawley & Cunning 2017) photo by J. Hogarth.

8 STABILITY ANALYSIS

With the site and material characterisation data collected, and an understanding gained of the groundwater and surface water conditions, stability analysis is undertaken to objectively demonstrate that the facility meets or exceeds the minimum acceptance criteria. Chapter 8 describes the general factors affecting stability, reviews failure modes and analysis techniques, and includes a system for establishing and applying design acceptance criteria.

The suggested stability acceptance criteria are presented. Figure 5 shows an example of the approach for static analysis results which range depending on the combination of the consequences of failure and confidence in the analysis and supporting data.

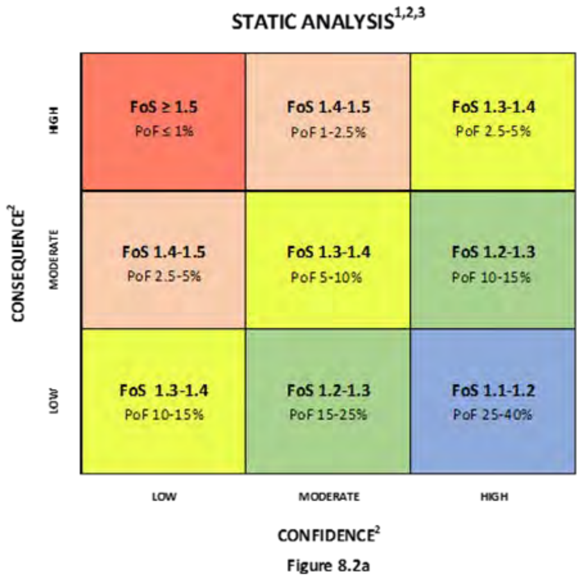


Figure 5. Example of static analysis acceptance criteria matrix (Hawley & Cunning 2017).

9 RUNOUT ANALYSIS

Appendix II of the book includes a summary of mine waste dump incidents in British Columbia, many of which resulted in substantial runout distance of waste rock dump material. Because mine facilities, the mine, or even public infrastructure can be located below the dump, and the potential for impact to the environment, an understanding of the runout potential is important. Chapter 9 describes various methods for the prediction of runout and includes discussions on:

- Material properties for runout analysis
- Slide initiation
- Failure propagation mechanisms
- Empirical runout analysis and prediction
- Dynamic runout analysis and prediction (2D and 3D)

The results of runout analysis can be used to establish site hazard and risk mapping, and in some cases, to size appropriate protection measures. Example of runout analyses are included.

10 RISK ASSESSMENT

As with other parts of the mining operations, there are risks with the operation of mine waste dumps, dragline spoils, and major stockpiles, and understanding these risks is the first step in developing effective mitigation measures. Risk assessment can be informed by the results of the stability and runout analyses and link to the dump stability hazard class or WHC. Chapter 10 presents a summary of the key elements of risk assessment and includes discussion on:

- Definition of risk
- Risk management process
- Analysis, mitigation, and management

11 OPERATION

Following completion of analyses and approval of designs, with risks understood and mitigation measures identified, the facility can go into operation. Chapter 11 provides a summary of recommended operation practices and includes discussions on:

- Dump and stockpile management plans
- Foundation preparation

- Water management
- Crest advance rate guidelines

Operations should be carried out with an understanding of the response framework for the monitoring plans (Chapter 12) and the closure plan (Chapter 16).

12 INSTRUMENTATION AND MONITORING

Instrumentation and monitoring of dumps are key to safe operations. Chapter 12 discusses the following key factors in instrumentation and monitoring:

- Visual inspections
- Displacement monitoring
- Monitoring guidelines
- Surface water and groundwater monitoring
- Example of trigger action response plan (TARP) or response framework, with links to the WHC

13 DRAGLINE SPOILS

Draglines are the largest piece of equipment at an open cut strip mine. This approach to mining results in a unique subset of mine waste management challenges, as the operation of the dragline often depends on the performance of the spoil.

Chapter 13 provides the operating characteristics and terminology for draglines. Dragline mining methods are described, including dragline operations and spoil management. The range of instability mechanisms and methods for stability analyses of these types of operations are discussed, and examples of stability analyses are provided.

14 MANAGEMENT OF ACID ROCK DRAINAGE

The Guidelines include a chapter on management of acid rock drainage (ARD) and metal leaching (ML) as related to mine waste dumps. Chapter 14 includes an overview of the ARD and ML processes which are important considerations in the design, operation, and closure of mine waste dumps. Numerous detailed guides on this topic exist, such as *The Global Acid Rock Drainage Guide* (INAP 2014), and geochemical expertise should be consulted for the geochemical stability aspects of the waste dump design, operation, and closure.

The chapter includes an understanding of the site climate as it interacts with the waste dump structure and hydrology including consideration of oxygen and water transport methods, which can be significant in waste dump structures and are often responsible for promoting the ARD processes. A review of methods for prevention and control through special handling techniques is included.

15 EMERGING TECHNOLOGIES

Technologies available to control ARD are highly developed and well understood as described in Chapter 14. However, problems with ARD from waste dumps still persist, largely because operators continue to use methods for constructing waste dumps that create structures that promote oxygen and water transport and interaction with sulphides. All ARD and/or ML problems found in existing and new waste dumps are created at the time of deposition based on the design principles and methods used for construction and closure.

Chapter 15 discusses the range of emerging technologies that should be considered to prevent ARD from occurring in the first place. The range of technologies discussed includes:

- Co-disposal techniques
- Waste rock in tailings
- Tailings in waste rock

- Layered co-mingling; cells
- Paste rock and mixtures of waste rock and tailings
- Blending
- Progressive sealing

16 CLOSURE AND RECLAMATION

Closure and reclamation should be considered as an integral component throughout the life cycle of a waste dump or dragline spoil. While this topic is presented as the last chapter in the book, planning for ultimate closure of the facility must be an active and iterative process throughout the design, operations, and closure cycle. Closure concepts should be in place when the conceptual design is prepared. Chapter 16 introduces the approach to closure and reclamation planning and provides an overview of key components of closure:

- Closure criteria and design life
- Geochemical stability
- Physical stability
- Land form and erosion control
- Revegetation

Planning for closure early in the design process can provide opportunities for progressive closure where completed portions of the dump can be resloped and revegetated concurrent with operations of other portions of the dump, as shown in the example in Figure 6.

Figure 6. Example of progressive reclamation concurrent with operations for a mine waste dump.

17 CLOSING

The editors would like to thank the LOP project for both the opportunity and the sponsorship to produce these Guidelines. The intent was to capture the current state of practice and help mine operators, geotechnical practitioners, and non-specialists improve their understanding of the factors that can influence the stability of waste dumps, stockpiles, and dragline spoils. The book has been in circulation since April this year. Each of the sponsor companies have received 50 copies of the book, which we understand have been distributed to their mine sites located around the world. We hope that the sites are finding this book of use, and we look forward to receiving constructive feedback on the book.

Many thanks to the large team that supported us in getting this book project completed and published.

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Management and Planning of Waste Disposal

Tailings History: 2016 & 2017

Jack A Caldwell
Robertson Geoconsultants, Vancouver BC, Canada

ABSTRACT: This paper continues a series of papers by the author over the past few years on the topic of the events and documents of the preceding year that affect the principles and practice of tailings facility design, construction, operation, management, and closure. Events and documents reviewed include: failure of tailings facilities in China, Israel, and the USA; a guidance document issued by the International Council of Mining and Metallurgy (ICMM) on tailings facility management; and new regulations in British Columbia regarding tailings facilities.

1 INTRODUCTION

Since preparing the paper for Tailings & Mine Waste 2016 on the same topic as this paper, tailings facilities have continued to fail at about the same rate as hitherto. In this paper, I discuss the failures in the past year that I find reported on the web. None of the information herein is based on personal knowledge; like me you may access the links I provide to see the original reports and more detail.

In addition to tailings failures, there have been tailings advances and successes. I discuss two significant new documents that provide guidelines intended to improve tailings management. And one significant announcement by a major mining company that they are moving to limit the use of water in tailings operations.

2 LUOYANG, CHINA

In August 2016 in Luoyang, China a tailings facility at an aluminum refiner broke. Details are sparse. Mining.com (2016) reports that at the time of failure the dam held 2 million tonnes of red mud and was about 1.5 kilometer long. Nobody was hurt, possibly because, as reported, the refinery (and presumably the dam) had been "shut down and villagers evacuated ahead of the dam burst." Pictures on the web show what appears to be a wide flow of tailings down the valley; the flow appears to extend to the edge of a village.

The Watchers (2016) states that 2 million cubic meters of "aluminum production byproducts" was released. The report includes a photo showing what appears to be flow from the dam. Also visible is the corner of the facility and what appears to be an embankment constructed of non-tailings material. Adjacent to the remaining embankment is what appears to be undisturbed (unfailed) tailings. There does not appear to be appreciable freeboard. Maybe this was a case of too much water on the dam. But this is speculation as there is nothing reported on the root cause of the failure.

3 MISHOR ROTEM, ISRAEL

The Landslide Blog (2017) reports that on June 30th, 2017 “the 60 meter high wall of a reservoir at a phosphate factory partially collapsed, letting lose 100,000 cubic meters (26.4 million gallons) of highly acidic wastewater in the Ashalim riverbed.” Watch the video at the link; it shows what appears to be a localized breach of a steep embankment. The breach to me looks as though it could be the result of overtopping. The report talks of “waste water.” The picture appears to show no significant flow of tailings from the facility. The upstream face of the embankment appears to have suffered upstream failure, possible as a result of rapid drawdown. I cannot tell if the embankment was constructed of tailings by an upstream method—there is no indication that I can see of centerline or downstream embankment construction. There are no further details on the cause of failure, so we shall have to await further reports to confirm or negate my speculation on the cause of failure.

4 MULBERRY FLORIDA

This story has not been recorded before now. I cannot recall the year or the location, other than it was in the dry part of the Transvaal in South Africa. I stood with a friend overlooking a conventional gold slimes dam. Except the center was a vast void. I was told that a sinkhole had formed beneath the dam and for years the operator had continued to pour slime (tailings) into the hole. I never found out what happened next.

This experience comes to mind when I read this (Wise 2017): “A 14 metre-wide sinkhole appeared in a phosphogypsum stack, opening a pathway for contaminated liquid into the underground; the liquid reached the Floridan Aquifer, a major drinking water resource. 840,000 cubic metres of contaminated liquid was released.”

A full and fascinating report, including the usual denials and refusals to release information, can be read at the Tampa Bay Times. Watch the video; it looks very much like what I saw a long time ago in South Africa.

5 TONGLVSHAN COPPER MINE, CHINA

From the silent kingdom, this is all I can find: “A tailings dam failure at Tonglvshan copper mine in Huangshi in China’s Hubei province resulted in two deaths in March 2017. The local government launched immediate safety inspections of all tailings dams in Huangshi.” (Wood McKenzie 2017). The website from which I copy this offers a two page report for five hundred pounds. I cannot afford that.

6 ICMM TAILINGS GUIDELINES

In December 2016 the International Council of Mining and Metals issued management guidelines for improved tailings practice (ICMM 2016). ICMM summarizes the guidelines thus:

“ICMM’s Tailings Governance Framework is embedded in a binding CEO-led position statement. Its purpose is to enable enhanced focus on those key elements of management and governance necessary to prevent catastrophic failures of tailings storage facilities. It also commits members to the continuous improvement in the design, construction, and operation of tailings storage facilities.”

The guidelines start by surveying tailings management practices by the members of ICMM (generally the world’s leading mining companies). The guidelines conclude:

- Some of the corporate documents were found to be comprehensive and in themselves examples of good practice.
- The majority of the member companies have corporate documents that substantially follow good practice.

- A minority of the member companies either have corporate guidance documents or adopt a surrogate that partly follows good practice. Hence most member companies either conform or partly conform to good practice

The minority not following good practice must either improve or the rate of tailings failures will not change. The guidelines document the standards of practice as established by many international and national codes. It is good to reiterate them so specifically in one place and with the authority of ICMM. Perhaps the most significant requirements, all too often not implemented are these:

- Owners and operators starting with the board of directors and chief executive officer should commit to assigning responsible competent persons and giving them authority and resources to implement systems required to manage their tailings facilities in such a way that the potential for a catastrophic tailings dam failure is minimized.
- Design, construction and operation, as a minimum, should take place under the supervision of a suitably qualified team led by an "Engineer of Record".
- Independent review by suitably qualified and experienced professionals should take place at appropriate milestones and intervals during each of the design, construction, and operation phases.

To my knowledge only British Columbia and Montana have regulations specifically calling for and Engineer of record and Independent Peer Review. Both jurisdictions adopted such regulations following the failure of Mount Polley's tailings facility in 2014.

7 APEGBC

In August 2016 the Association of Professional Engineers and Geologist of British Columbia issued Guidelines for Site Characterization for Dam Foundations in British Columbia (APEGBC 2016). This document was issued in response to recommendations following the failure of Mount Polley.

Terzaghi and Peck were the first to attempt to establish guidelines for site characterization. I am not sure they succeeded. Nor do these guidelines, I am afraid. For it all boils down to judgment. Nevertheless, the guidelines provide much valuable information and guidance—backed up the authority of the Association. They: spell out the roles of owners, developers, engineers, reviewers, regulators, and the many other parties involved in dam foundation characterization; describe the phases of work; recommend reports; discuss typical foundation geologies and characterization methods; list information sources; and much more. Certain I am that if you implement these guidelines your tailings facility foundations will be sound.

Most valuable in my opinion is this table—which I have already found occasion to use in connection with two projects.

Design Stage	General Objectives of Design Stage	Typical Site Characterization Activities
Scoping level design	Develop options for siting and design	Work is primarily based on existing information and table-top evaluations, but it typically includes a site visit for general reconnaissance of site conditions and mapping. Site geologic and other public information is used to develop an initial characterization of the potential site foundation conditions.
Prefeasibility design	Compare options to select the preferred site and design	Work typically includes terrain and bedrock mapping, some site-specific intrusive investigations, lidar, test pits, and geophysics.
Feasibility design	Support financing and environmental assessment estimates	Work includes a wide range of investigation methods, including intrusive investigations, in situ testing, geophysics, and laboratory testing. Extent of site investigations is increased to the level required for the complexity of the site.
Detailed design	Issue for construction drawings and specifications Address permitting requirements	It may be necessary to conduct additional site characterization to support aspects of the detailed design.

8 INDUSTRY ACTION

One of the more teasing promises of 2017 is the announcement by Goldcorp and FLSmidth that they are developing a system to co-mingle dewatered tailings with waste rock in a continuous process (Mining.com 2017). Of course filtered tailings is not new; comingling tailings and waste rock is not new; maybe a continuous process is a new goal. No details are announced in the news reports; but if they succeed, the implications could be significant: large mines will be able to avoid conventional slurry tailings and instead dispose of tailings with no water on the tailing and no water in the voids of the tailings; and all will be dilative.

They apply two new trade-marked names to the process: EcoTails and GeoWaste. Now I have been using the term GeoWaste for a long time to denote mine tailings facilities, waste rock dumps, and spent heap leach pads–see my courses on EduMine. There are companies called GeoWaste. So how can they trademark a name in common use by using it to describe a product they cannot yet make?

Bold I suppose. If they succeed where others have failed with comingling, this will be a noted success, whatever it is called. If they fail, the idea and the trademarks will fade into deserved obscurity.

9 ICMM WATER REPORTING GUIDELINES

Although not specifically about tailings, the ICMM guidelines *A practical guide to consistent water reporting* (ICMM 2017) will, I believe, impact tailings water management and reporting. This is what the guidelines set out to achieve:

- Defining an appropriate set of standardized water reporting metrics for the mining and metals industry.
- Outlining the minimum disclosure standard for member companies which sets a transparent benchmark for the industry.

- Providing practical guidance around preparing corporate water summaries and meeting the minimum disclosure standard.

I have already recommended to one client that they adopt the guidelines for their tailings facility. Standardized water reporting may also lead to proper water management at tailings facilities, and that can only help reduce the rate of failure of such facilities.

10 CONCLUSIONS

On the basis of what I write in this paper I think it is reasonable to conclude that the rate of tailings failure is not about to reduce any time soon. The ICMM findings that some of their member mining companies are not practicing effective tailings management is basis enough for this conclusion. In addition, as the failures in Israel, Florida, and China establish there are many industrial tailings facilities that are badly operated and probably unregulated.

Future failures cannot be blamed on a lack of guidelines. We now probably have too many to keep abreast of. Rather the failures will stem from management inattention, inexperienced designers, unsupervised operators, and inept regulators. The human component continues as it always has in engineering to be, in my opinion, the single-most significant factor leading to tailings failure. Let us discuss this opinion next year in the light of future failures.

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Strategic Developments in the Next Decade for Tailings

Jeremy Boswell & Iain Gidley
Thurber Engineering Ltd., Calgary, AB, Canada.

ABSTRACT: Developments in the field of tailings in the past three years (the tailings breach at Mount Polley in August 2014, and the flow liquefaction of the Fundão tailings dam in November 2015) have generated renewed scrutiny of tailings facilities. In addition, the subsequent changes to legislation and tailings guidelines, have demanded change in the design and management of tailings structures. This begs the question, where are we headed in the next decade? This paper considers some strategic changes that confront the international tailings industry.

Critical thinking in credible failure analysis is required to consider new and unusual mechanisms of failure and risk. Failure mechanisms which in the past may have been considered non-credible, may now require more attention and cannot be routinely dismissed. Concerns surrounding liquefaction risk have placed renewed emphasis on beaching, supernatant management, basin control and accurate recording of deposition history. The geotechnical performance of tailings structures may demonstrate a rising risk profile when compared to conventional water retaining structures, and consequently demands more stringent vigilance during the operational phase.

1 INTRODUCTION

Substantial changes are taking place in the tailings industry worldwide, particularly in the western hemisphere. It is not simply as a result of the direct impact of the Mount Polley and Fundão failures, which were both catastrophic in nature and unprecedented in size.

Rather, it is now as a result of some of the collaborative and innovative thinking that has transpired in industry over the past three years, led by organizations such as the Canadian Dam Association (CDA), Mining Association of Canada (MAC), The Alberta Dam Integrity Advisory Committee (AB DIAC), the Association of Professional Engineers and Geoscientists of British Columbia (APEGBC), the Geoprofessional Business Association (GBA), International Congress on Large Dams (ICOLD), United States Society on Dams (USSD), and the (US) Association of State Dam Safety Officials (ASDSO), and within the ranks of owners and operators of dams, consultants, contractors and regulators.

It is not the intention of this paper to summarize all of the developments in each of these organizations, the new codes, guidelines, regulations and laws, and yet more still in the process of being developed and published. Rather, the paper takes a strategic view of where we might be headed in the next decade, and singles out key individual topics which are likely to occupy markedly increased attention.

2 NEW AND UNUSUAL MECHANISMS OF FAILURE AND RISKS

Recent high profile international tailings flow liquefaction events have placed renewed scrutiny on tailings designs, and have drawn attention to new and unusual mechanisms of failure and risks (undrained failure after transition of a glaciolacustrine layer from an overconsolidated to a normally consolidated condition under loading at Mount Polley in British Columbia, and extrusion of soft layers causing collapse and subsequent flow liquefaction of adjacent sand tailings at Fundão, in Brazil).

While tailings failure investigations often focus on a geotechnical cause, the reality is that many factors may combine to create an environment where failure can occur. These contributing factors often go beyond the purely geotechnical – to such factors as increased rate of rise, inability to discharge water, insufficient construction material, requirement for more capacity, space constraints, budget constraints, design changes, silo thinking strictly within disciplines only, planning and scheduling conflicts, breakdown in communication, and others.

Analysis of previous flow liquefaction failures of tailings (Boswell and Sobkowicz, 2015 and other references cited in that paper) has usually found significantly more than a single cause of failure, and more than a single possible mechanism of failure. Indeed, 43 years after the Bafokeng failure, experts still disagree as to whether the primary mechanism was overtopping or piping (internal erosion). In fact, it may have been both. Detailed re-examination of the Merriespruit failure (Blight and Fourie, 2003, and others) shows a number of mechanisms at work simultaneously (overtopping, slope failure and headward slumping, and piping and liquefaction).

Mount Polley and Fundão have shed light on two potential failure modes, as described earlier. There needs to be an answer to the question: Could what happened at Mount Polley or Fundão happen here?

In addition, the design is required to answer to further questions in regard to credible failure modes (this list is not exhaustive):

1) How certain is the design team that all credible failure modes have been identified? The answer to this question would build upon and contribute to the results of the FMEA.
2) What was the rationale used to determine the weakest failure surfaces?
3) For the credible failure modes listed, what is the likelihood that the weakest failure surface has not been exhaustively identified (non-optimized solution)?
4) How have combination failure modes and mechanisms been considered?
5) How have sensitivity analyses been considered?

A systematic review of the credible failure modes should be undertaken to establish if anything has been neglected so far in the analysis, (e.g. the B.C. Ministry of Forest, Lands and Natural Resources Operations Chart of Global Failure Modes), such that the design team is able to answer each of the questions raised.

A recent project undertaken by one of the authors found evidence of multiple potential triggers for failure all as a result of a single large flood event (spillway failure, overtopping, piping, static or dynamic liquefaction, and embankment liquefaction or shear induced slope failure). Thus a linear consideration of a single risk and single remedy may be wholly inadequate in addressing all possible risks.

The Fundão failure highlighted the importance of considering the aggregate impact of both dynamic (earthquake) and static (rise in pore pressure, etc.) triggers to liquefaction. One or the other might just push the failure over the edge, acting in combination. Even small earthquakes can have major impacts on geotechnically fragile structures.

Perpetuity disallows the argument of unlikelihood: if you wait long enough, failure will occur. In earlier papers Boswell and Sobkowicz (2015, 2016) confirmed what Robertson (2011) had observed before: that in the face of perpetuity unlikelihood provides no defense at all.

It is critical that each corporate discipline (Mining, Tailings Planning, Operations, Construction, Water Resources, etc.) understand and take ownership of the scheme risks and the responsibilities which fall under their direct control, to supplement the efforts of a Geotechnical team.

The design tool to empower this collective team approach is a Failure Modes and Effects Analysis (FMEA), and one which has become synonymous with good practice in the oil sands and elsewhere. An FMEA uses a team-based approach to establish credible failure modes beyond those solely geotechnical in nature. Beyond the immediate technical risks, it also considers the impact of corporate changes (in shareholders, in management, in risk appetite) and wider industry developments (commodity prices, changes in legislation, etc.).

During the FMEA, the team is also able to develop mitigation strategies to reduce the risks to corporately acceptable levels. The mitigation strategies may require design changes, regulatory interactions and constraints to construction and operation of the structure. Early intervention has been found by experience to improve the design and add value. It is considered essential to involve all disciplines groups so that key risks are understood and managed accordingly.

The FMEA is fundamental to the MAC guidelines and is also a requirement for the delicensing of Oil Sands Tailings dams. In terms of the state of practice worldwide, the FMEA is now considered to be an integral part of the regulatory application and approval process.

The steps required for a comprehensive consideration of risk include the following steps, as described by Vick, (2017a, 2017b and 2014), and others:

- Risk identification.
- Risk characterization and analysis.
- Risk assessment and rating.
- Mitigation of risk.

The obvious benefits of an FMEA include the identification of cross cutting and multidisciplinary risks, the mitigation of risk to a corporately acceptable level, and the support provided to regulatory applications. An early FMEA process allows as much time as possible for mitigation measures and interventions to be introduced in advance, usually obviating larger costs associated with rework and delays.

3 STATIC LIQUEFACTION

There continue to be many far reaching effects from the Fundão tailings failure as the worldwide mining and tailings industries consider what happened, and deliberate as to how this can be avoided in future. Static liquefaction is now an essential consideration for tailings dam stability. The Fundão tailings structure failed in a liquefaction flow slide that initiated at the left abutment of the dam (Morgenstern et al., 2016). The panel tasked with review of the Fundão failure concluded that the dam failure occurred due to several factors and in particular, the flow liquefaction of the sand tailings. The failure was triggered by a lateral extrusion mechanism located in a slimes-rich zone in the embankment. The lateral extrusion of the slimes resulted in a reduction of lateral confinement of the overlying loose, contractive and saturated tailings sand, causing it to liquefy and which led directly to a flow liquefaction failure.

Since static liquefaction is a brittle failure mode, the traditional oil sands defense of the observational method provides no protection at all. It might even create a false sense of security - unless the instrumentation on a dam is targeted at the primary modes of credible failure.

As hydraulic structures are constructed at higher rates of rise the risk of producing liquefiable tailings continues to grow. This risk is further described in sections 3.1 and 3.2 below. In addition, the deposition of interbedded coarse and fine layers in both beach above water (BAW) and beach below water (BBW) deposition profiles in the oil sands introduces a further degree of variability in tailings behaviour seldom found elsewhere.

So what defense might be provided against static liquefaction and the triggers which might initiate this failure mechanism? Before answering that question, perhaps we should pause to consider some fundamentals as described in recent key publications.

Case studies have shown that tailings deposits may liquefy under static loading conditions (Fourie et al., 2001; Robertson, 2010). Static liquefaction describes an event by which saturated or sometimes partially saturated tailings undergo sudden loss of strength due to undrained loading triggered by events such as rapid rate of construction, prolonged precipitation (causing ponding, filling of tension cracks and an increase in phreatic surface levels), erosion of toe support, excavation and dumping of material on beaches. For liquefaction to occur, the tailings must be susceptible to excess pore water pressure buildup and strain softening with a resulting loss of shear strength. Tailings in this category include very loose cohesionless sands and silts as well as very sensitive clays (Robertson, 2010). Static liquefaction (Jefferies and Been, 2006), is recorded as a major design consideration for tailings structures.

The potential for flow liquefaction of tailings deposits is commonly assessed based on in situ testing data and screening criteria, such as those formulated by Fear and Robertson (1995) and Robertson (2010).

The Fear and Robertson criterion was later applied by Olson and Stark (2002, 2003) to an extensive database of case histories where flow liquefaction occurred.

The liquefaction criterion proposed by Robertson (2010) is the most recent development of earlier methodologies proposed by Professor P.K. Robertson. This criterion uses CPT data and involves correcting the tip resistance and sleeve friction measurements to account for stress level and grain characteristics (combined influence of fines content, mineralogy and plasticity). Robertson suggested that the boundary between contractive and dilatant soil behaviour can be represented by the normalized tip resistance, corrected for clean sands (Qtn,cs). A Qtn,cs value less than 70 represents soil (or tailings in this case) that are potentially contractive and susceptible to liquefaction. The Robertson criterion is also based on a number of case histories where CPT data is available.

Figure 1 below from Robertson (2010) shows the approach suggested by Robertson, in greatly summarized form. Normalized cone resistance is plotted against normalized friction ratio, in order to place data points on the plot, and so determine whether the material is liquefiable or not, and the extent thereof. The reader is urged to read the full paper, as well as the papers referenced above and subsequent papers by Sadrekarimi (2014, 2016), in order to appreciate the full scope of the approach, and how static liquefaction risk is assessed. It is of course essential to have access to good CPT testing data, as a point of departure.

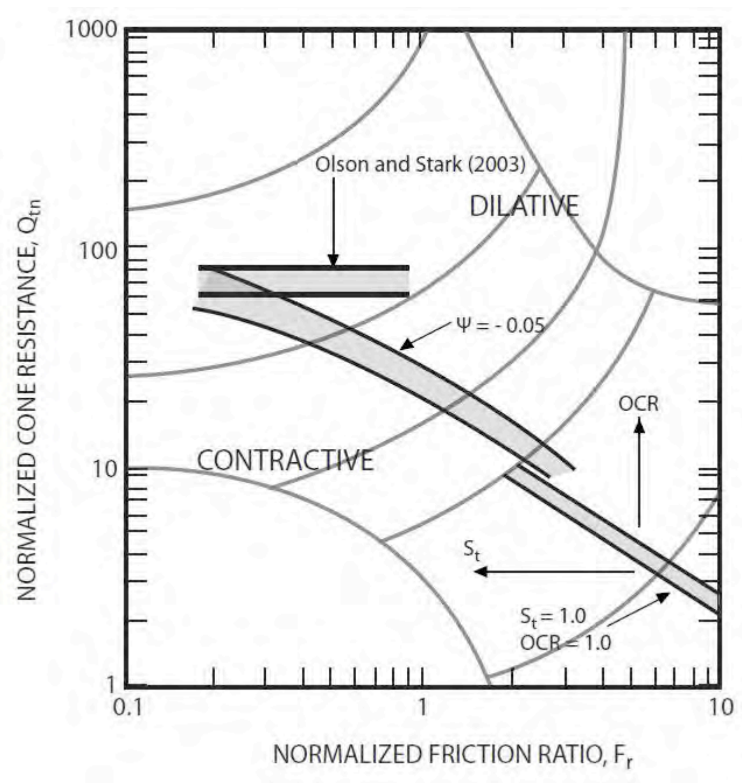

Figure 1 from Robertson (2010). Approximate boundary between dilative and contractive material response using normalized CPT parameters.

Among many important considerations, some of the primary defenses against static liquefaction are preventive, rather than curative: two of these important defenses are quite mundane and often overlooked. They are described in brief immediately below.

3.1 *The critical importance of deposition history*

Even for tailings deposits constructed very recently, available records of deposition are often unrecorded, unavailable or not suitable to fully reconstruct the deposition history. Improvements in aerial photography and the use of drone photography has provided some insight into the general history of the deposition and rate of rise. However, if the tailings deposit has demonstrated large and unexpected material variability, or shows the presence of weak layers within the deposit, the deposition history should be determined in greater detail.

The selection and slope stability analysis of critical cross sections requires significant interpolation of field data, construction and operational records, phreatic surfaces, and the use of engineering judgement in order to determine the extent of liquefiable tailings, the extent and character of weak clay layers, and the likely pore pressure response to triggering.

In the event that the actual geometry and performance of key weak layers within the structure is markedly different from that which has been inferred, the design may be too conservative, or worse still, non-conservative.

A series of telephone and site interviews using questionnaires, may be useful to amplify the understanding of the scheme deposition history, and in order to more accurately predict the structure behaviour. In addition, historical aerial photography (if possible at monthly intervals) may be examined in order to establish deposition in which supernatant was not decanted, the largest pond size, extent of accumulation of clay, bitumen, frozen ground and other interruptions of effective beaching, on the deposit, and degree of continuity between potential weak layers.

3.2 *Rate of rise*

Pollock, Mettananda and MacGowan (2014) show that sand tailings beach above water (BAW) deposition at annual rates of rise in excess of 10 metres per year, which is not track-packed, is liquefiable.

Exceeding the established safe rate of rise limits will increase the risk of creating weak or liquefiable layers in beach above water (BAW), or subaerial deposition, and may limit the future tailings storage capacity. Mitigation of problem layers after the fact is time consuming and expensive.

It is usually preferable but typically unachievable in practice, for deposition rates of rise to be slowing down, rather than accelerating. Within reasonable norms, the slower the structure is built, the higher its maximum potential final safe height is likely to be.

It is recommended that a safe rate of rise be established for a structure, and that instantaneous and monthly rates of rise be measured, and recorded, as well as lift thickness. Widely differing differential rates of rise in different areas of the structure should also be avoided, and replaced with consistent and regular deposition across the entire surface.

4 DAM SAFETY

Shortly after the Mount Polley Tailings Breach on August 4, 2014, a group of dam safety engineers convened to consider current and emerging issues related to the responsible operation of resource sector dams in Alberta. A workshop was convened in November 2014 by the Alberta Chamber of Resources, which led to the formation of the Alberta Dam Integrity Advisory Committee (DIAC). One of the earliest requirements to emerge for action was the definition of roles and responsibilities for an Engineer of Record (EOR).

Other industry and professional organizations involved in the design and management of mine tailings dams have convened separate committees to define the EOR and related roles in tailings dam safety. These include the Mining Dams Committee of the CDA, the Tailings Working Group of MAC, and the Tailings Engineer-of-Record Task Force of the Geoprofessional Business Association, which determined that assistance should be provided to their members to address the liability posed as EORs. Morrison and Hatton (2017) echoed this in publications either recently issued, or in the process of publication, GBA Tailings EOR Task Force (2017) at time of writing of this paper. Communication between these groups has facilitated an emerging consensus on the key elements of the EOR definition, and importantly, the role and limitations of the EOR role within a dam owner's organizational structure.

The concept of the Engineer of Record (EOR) has recently been applied to mining dams in various regulatory jurisdictions in Canada, the USA and elsewhere. The formal appointment of an EOR is a risk management tool that has gained importance as industry and regulators seek ways to improve the safety record of mine tailings dams, which have come under increased scrutiny following recent tailings dam failures in British Columbia and Brazil. The appointment of an EOR with clearly defined responsibilities for this role forms one essential component of a dam safety management system, which must have a defined chain of accountability and responsibility that includes the corporate leadership, operations management, and technical professionals.

A robust dam safety program is founded on accountability for dam safety that rests at the Owner's senior executive and board of directors' levels, and clearly defined responsibilities for managers and technical professionals. There is a list of key accountabilities and responsibilities that must be fulfilled in order to safely and effectively manage the design, construction, operation and closure/decommissioning of dams. Organizations may use different titles, and combine or split roles as necessary to fit the organizational size, scope and technical capability. However, an organization with a robust dam safety program will have all of these roles addressed in some manner within their organizational structure, or delegated to external consultants/contractors.

Boswell and Martens (2017) recently summarized the work that has been done to date by the Alberta Dam Integrity Advisory Committee (DIAC), towards providing greater clarity on the role of the Engineer of Record (EOR) and related dam safety roles for organizations and owners of tailings dams, tabled in a white paper prepared by the EOR subcommittee of DIAC in June 2017 (DIAC, 2017) and issued by the Alberta Chamber of Resources. The work drew heavily from the oil sands tailings industry, since the oils sands represent in both number and size the bulk of major tailings and mining dams in Alberta. The paper outlined the basic roles and responsibilities necessary for the safe management of dams in Alberta. It was intended to be a flexible framework, adaptable to the size and structure of an organization.

The Owner of a dam is ultimately accountable for the safety of the dam. To fulfill this accountability, the Owner requires a design prepared by a qualified and experienced design team, led by an individual with overall responsibility for the design. The Owner requires further technical support to verify that the dam is being constructed in accordance with the design specifications, is operated within the design limitations, is performing as expected, and to define remediation requirements if the dam performance does not meet the requirements. The individual engineer with the single point of responsibility for the design and technical support roles has been designated as the Engineer of Record.

The approach adopted by DIAC identified five key dam safety roles:

- Accountable Executive
- Operations Manager (Construction Manager/Project Manager)
- Dam Safety Responsible Engineer (DSRE) – (Chief Dam Safety Engineer)
- Engineer of Record (EOR)
- Design Engineer (DE)

Figure 2 below shows an example of an organizational structure and how these roles interrelate.

The overriding conclusion of the many revisions and updates issued recently by organizations tasked with dam safety is thus: assurance of tailings dam safety cannot be the job of a single individual, and the owner of a dam remains ultimately accountable for dam safety.

There is thus a list of key roles and responsibilities for dam safety which must be clearly defined and assigned to individuals, from an accountable executive in the dam owner's organization, to the operations manager and responsible technical professionals. The role of the EOR is central to the safe management of resource dams, but is only one component of a wider team with responsibilities and accountabilities for dam safety. The detailed and team-based nature of these responsibilities illustrates the integrated and sustained effort required in order to provide the assurance of dam safety in the long term. This is particularly true for tailings dams, in which the risk profile generally rises as the facility is constructed and operated to full height and capacity.

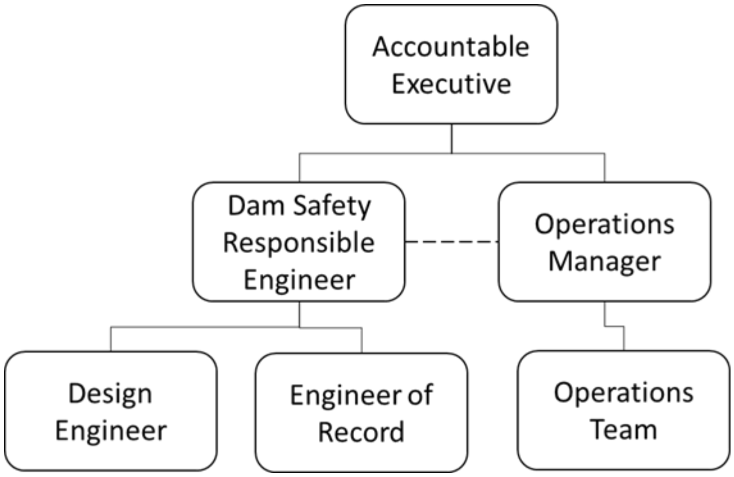

Figure 2. Dam Safety Accountabilities and Example Organizational Structure

5 WATER STORAGE

In the decades of the 1970s and 1980s, a primary defense against airborne environmental emissions from tailings (such as ionizing radiation, acid rock drainage, particulates and fugitive contaminants) was developed in the North American mining industry, led by the uranium sector: maintain a water cover or cap, over the tailings, in perpetuity.

However, earlier comments in Section 2 of this paper have described the difficulties associated with providing assurance of dam safety in perpetuity, notwithstanding how remote the possibility of failure might have been shown to be.

Mount Polley and Fundão are reiterating the lessons of the past 50 years: do not store water on tailings dams. Instead, store water in water dams, and aim at a zero pond approach for tailings containment. A new corollary is now being added: do not store liquefiable tailings, and aim for the storage of tailings which are inherently stable. Boswell and Sobkowicz (2015) list this as a Best Available Technology (BAT).

The ravages of climate change and other factors have shown in northern climates how difficult it is to maintain water storage and thermal stability at the same time. (Proskin et al, 2017). The message is abundantly clear: get rid of water storage on tailings.

6 FLOOD ANALYSIS AND SPILLWAY DESIGN

More rigorous consideration of credible failure modes and the triggers associated with them (as described in Section 2 above), have yielded other important learnings:

– Extreme flood events may provide multiple triggers for failure.
– Resulting risks and failure mechanisms may be additive, or even worse, multiplicative, rather than alternative.
– Spillway failure and other key component failures can lead to multiple triggers.

The events in February 2017 at Oroville Dam in the northwestern USA, Wikipedia (2017) have brought into sharp focus the importance of spillway design and maintenance. The 188 000 evacuees and their families are unlikely to ever forget what happened. Perhaps others will remember too, in the mining and tailings industries.

Hydrotechnical analysis and design remains a key feature of tailings design. It is a trite saying but nevertheless bears repeating: that all catastrophic flow tailings failures are caused by water.

7 FOUNDATION INVESTIGATION

One of the key findings of the Mount Polley Independent Panel Investigation was that the engineering properties and behaviour of a foundation unit were misidentified. Site investigations are intended to unravel millions of years of geologic deposition history. Often these histories involve environments significantly different from the current day. For example Fort McMurray, in Northern Alberta, at different times was once a large river, a tropical estuary, deep ocean and under kilometers of ice. Each one of these depositional environments created unique materials with different properties and behaviour. It is important to understand the many geologic environments that existed previously because it provides insight into the types of materials present, their pervasiveness and their geotechnical properties.

At Mount Polley a discontinuous glaciolacustrine layer (GLU) was found during the post-failure investigation to be in the worst location with regards to stability, i.e. it was present in the area where the maximum shear stress from the dam would be. The initial site investigation was "not sufficient" to identify the extent of the layer and appreciate its behaviour under load. Subsequent site investigations did look at the strength of the GLU, however they did not appreciate the stresses the material would face during operations. The average pre-consolidation pressure found during the post-failure investigation was in the range of 430 kPa with an initial over-consolidation ratio of 6. This implies that the average initial in-situ effective stress was around 70 kPa, which means that the foundation material would show over-consolidated behaviour, i.e. dilative, negative pore pressure generation during shear loading and drained behaviour, for the first 360 kPa of applied stress. After this point the applied pressure would exceed the pre-consolidation pressure and the GLU would behave in a normally consolidated manner, i.e. contractively, generating pore pressure during shear and exhibiting undrained behaviour.

At failure approximately 30 m of embankment had been placed, which would correspond to a stress increase in excess of 500 kPa at the level of the GLU. This is well in excess of the assessed pre-consolidation pressure of the material and likely meant that the material was behaving in a normally consolidated, i.e. undrained, manner.

In the wake of the Mount Polley investigation the Chief Inspector of Mines in British Columbia ordered 38 mines in the province to provide a letter of assurance that the conditions that existed at Mount Polley do not exist at their facilities. This example shows the importance of understanding the magnitude of the imposed stresses during construction as the material that failed was by all accounts very stiff and over-consolidated. Based on this observation therefore, no detailed investigation of the material was undertaken. However, since the structure was so large, the stresses imparted on the foundation materials we well beyond those previously experienced (i.e. the stresses exceeded those the material had previously been subjected to). Field investigations should be scaled based upon the zone of influence of the structure. The larger the structure, the larger the stresses and the further into the foundation the stresses will propagate, thus the greater the investigation required.

There are simple methods (pg. 136 Sabatini, et. al. 2002) to assess the pre-consolidation pressure of materials from in-situ test methods. These correlations can be used as a first approximation screening tool to identify when the foundation materials will approach these limits, and when further laboratory investigation is warranted. The objectives of the laboratory program also need to be well thought out, in order to answer such questions as: "Why are the materials being tested? What parameters are you looking for? How will those parameters be used?"

Laboratory investigations can be time consuming and complicated, and may pose several technical challenges such as pore fluid chemistry, stress history, size effects, material variability and difficulty in obtaining undisturbed samples. Between in-situ and laboratory testing, since only a very small volume of the foundation material is investigated, it is less likely that the very best and worst material has been found. It is important to understand the variability of the material and incorporate it into design. Given the potential difficulties it is important to have a statistically significant number of results that can then be compared against more plentiful in-situ data and other means of correlation or indexing against published data.

The complexity of the foundation materials for a large structure may demand much more than a single test investigation, in order to fully characterize and understand the behaviour. Instead, an iterative and integrated campaign approach is recommended, that evolves as new understandings are gleaned from previous test results and as design changes occur. Good design and economic design are derived from a full understanding of the hazards that are facing the structure. Typically, an increased investment in ground investigation results in a more cost effective design and at worst it may have cost a little more to gain only a reaffirmed understanding. Contrast this with a under scoped investigation that fails to adequately appreciate the complexities of the foundation materials and the cost of delays or increased potential for failure, or at worst, precipitates another failure such as occurred at Mount Polley.

8 INCREASED REGULATION, INSPECTION AND REVIEW

An important and most positive development in North America has been the increase in vigilance provided by regulators. New procedures and checklists have been developed, and additional staff have been recruited, in order to give effect to these heightened measures.

The USSD and ASDSO in conjunction with the Geoprofessional Business association (GBA) have convened seminars and workshops to draw attention to these new developments and to raise standards in the industry.

Locally, in Alberta, Alberta Environment and Parks (AEP) in association with the Alberta Energy Regulator (AER) and the Alberta Dam Integrity Advisory Committee have commenced a two day annual dam safety seminar in April each year.

In British Columbia it is now mandatory, to all intents and purposes, to employ an Independent Tailings Review Board for operating tailings facilities, BC (2016).

9 DECOMMISSIONING AND CLOSURE

A few observations in regard to tailings facility decommissioning and closure are perhaps necessary:

- The impetus (provided by high profile tailings failures) towards closure of non-operational facilities is growing.
- Active closure as a phase, is not only very expensive, it has become more risky.
- Passive closure is now more difficult to achieve and requires considerable planning, resources and expenditure.
- Closure costs and residual tailings risks are likely to be factored into future mining plan and ore and mine valuations.
- Delicensing of tailings dams appears to be an increasingly remote ideal, as stakeholders have been reminded in stark terms of the potential residual risks associated with tailings storage.
- Climate change is a necessary and important consideration in planning for closure, as noted by Rooney et al (2015), and many other authors, and is likely to substantially influence closure strategies.

10 INSTRUMENTATION WHICH TARGETS FAILURE MODES

Ralph Peck wrote in the foreword of Dunnicliff, 1988, that "(instrumentation) cannot guarantee good design or trouble-free construction. The wrong instruments in the wrong places provide information that may at best be confusing and at worst divert attention from telltale signs of trouble." When developing an instrumentation program every instrument installed in the ground must be done so with purpose. Answers should be provided to such questions as "Why is the instrument being placed where it is? What readings is it expected to give? What limits are there to establish troubling performance?"

Tailings impoundments often present additional challenges for instrumentation. In typical geotechnical works, construction takes place over one to two years, with some mega projects taking five to ten years. It is not uncommon for a tailings storage facility to be constructed over 30 years. This elongated construction period makes it more likely that the instrument or the instrument cable will be damaged, which is why multiple redundancies are critical for important instruments. An additional challenge stems from the fact that instruments that are placed in key locations tend to get damaged. When these instruments are replaced, the slope of the embankment or design of the structure may cause the new instruments to be installed in less significant locations away from where early movement and maximum stresses are located.

When thinking about the purpose and location of the instrumentation it is important to consider the potential failure modes. An embankment built on solid rock will have different areas for concern than an embankment built on shale. For large embankments, lateral foundation movement will generally initiate from a location where the highest shear stresses are, which is typically under the mid-slope of the embankment. As the embankment grows in size, the location will migrate either downstream or upstream, depending on the embankment construction method used, i.e. upstream or downstream construction. Maximum pore pressures and settlements are conversely found under the dyke crest where the maximum load is. Neither location is very conducive to replacing instrumentation. During the instrumentation design process it is therefore important to understand what the expectations for the instruments are. If the instruments are being used to control foundation performance and guide construction practices, then more redundancy is required. If the design is significantly robust and the loss or absence of instruments in place will not impact construction, then only sentinel instrumentation may be required.

The readings obtained from instruments are only one part of the greater picture however. Over reliance on instrument readings alone, can provide a false sense of security. A thorough understanding of the engineering conditions and anticipated behaviour is required, as instruments only monitor strata they are installed within. Measurements should be coupled with visual inspections. To borrow again from Peck, instrument readings are but one link in the chain, coupled with design understanding and visual observation.

Furthermore, instrumentation is also only a safeguard against behaviour that is monitorable or measurable. Brittle failure modes such as liquefaction and softening due to transition between over-consolidated and normally-consolidated behaviour are not capable of being monitored in a timeframe that allows mitigation or design changes to be implemented to prevent damage to the structure and possible failure. Failure modes that cannot be accounted for by instrumentation prevent application of the observational approach in their defense. These behaviours must be accounted for properly in the design process as, by definition, failure will happen too quickly to enact mitigation measures.

The Mount Polley Panel, 2015, stressed the need for sentinel instrumentation and the lack of sentinel instruments was found to be a shortcoming of the monitoring program at Mount Polley. The sentinel net of instruments provide early indications of lateral sub-surface movements and pore pressures. These instruments should be placed in locations, where early information may be obtained, and be spaced around the entire structure based on the expected size of the likely ultimate failure surface.

During design of a structure certain assumptions and simplifications are required. These are often necessary because structures are being built on unknown foundations, of materials with uncertain performance. Instrumentation is then installed to help validate the design assumptions and to help understand if they are conservative or not. The costs saved in more aggressive design must be offset by increased vigilance in monitoring.

11 CONCLUSION

One might be tempted to say with Dickens: “It was the best of times, it was the worst of times”. It has been argued by others that in the aftermath of Mount Polley and Fundão, (the worst of times) we have a window of about three years (the best of times) in which to effect real change in the way tailings is managed. After that, if history is any indicator, life will settle down again and the usual bad habits may unfortunately resurface – cost cutting, design changes, changes in personnel, apathy (“that will never happen here”), cut backs in training, regulatory vigilance and resourcing, and many other areas.

Or perhaps not. Perhaps we are going to observe a sea change. To forever depart from the “business-as-usual” approach so decried by the Mount Polley report. We have reason to be optimistic, based on the substantial changes described in this paper. Perhaps we do have a chance to rewrite history.

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Tailing management at the Kirunavaara iron ore mine

S. Töyrä
LKAB, Kiruna, Sweden

D. Lundell & A. Bjelkevik
Tailings Consultants Scandinavia AB, Stockholm, Sweden

ABSTRACT: The Kirunavaara iron ore mine is located in the sub artic climate zone. The presented case study focuses on challenges with tailings deposition in a cold climate, i.e. difficulties with slurry pumping and water circulations during winter. The design of the embankments have been altered to meet an increased production as well as new stricter dam safety requirements. Some embankments that were initially built with a till core have later been raised with an upstream method combined with spigotting. The water balance on a yearly basis is positive. The storage capacity in the system is however limited. The preferred situation for an upstream construction, on partly undrained tailings, is a minimum of free water combined with as many spigots as possible. The preferred situation regarding water supply is to minimize the risk of freezing, which requires a sufficiently large water volume and few discharge points to keep a concentrated flow.

1 KIRUNAVAARA IRON ORE MINE

1.1 *Background – Swedish tailings management*

Tailings management in Sweden has been – and in some cases still are – going through a transition from an older approach with lots of excess water circulating in the systems, to more optimized tailings depositions schemes with less water in the tailings storage facility (TSF). Initially, tailings dam design was largely influenced by the hydropower sector, resulting in TSF's with impermeable embankments and large volumes of free water, combined with one or few slurry discharge points (the latter usually as an alternating single discharge point).

However, the water situation has not been the driving force behind this transition. Instead a more optimized deposition scheme has usually been introduced to reduce the cost of embankment construction, through a more efficient use of available storage capacity (lower raises) and more cost-efficient design of the embankment structure itself.

The purpose of this paper is to give an example of problems that may arise due to this type of transition, which not only allows for an increase in tailings storage efficiency, but also completely changes the water balance.

1.2 *Introduction to LKAB and the Kiruna site*

Loussavaara-Kiirunavaara Aktiebolag, LKAB, mines iron ore at two underground mines in Kiruna and Malmberget as well as three open pit mines in the Svappavaara ore field. LKAB processes the iron ore into pellets in three steps; sorting, concentrating and pelletizing. The pellets are then shipped to customers around the world from the harbors in Narvik, Norway, and Luleå, Sweden. Due to the local conditions, the tailings are treated in different ways at the three mine sites. This paper will focus on the tailing management at the Kiruna mine site.

At the Kiruna mine site, there are one sorting plant, three concentrating plats and three pelletizing plants. Tailings occur as waste during the wet milling process in the concentrating plant. Tailings are transported to the TSF in three separate slurry pipelines, one from each concentrating plant. Other waste water (as well as tailings when the slurry pipelines are not operating) are discharged via an open trench, i.e. north or south trench. Water from the TSF is discharged into a clarification pond. The clarification pond is used as a second clarification step as well as process water storage.

Ground water is constantly pumped from the underground mine and this water is used in the process. The water balance is positive.

1.3 *Climate*

Kiruna is located in the northernmost part of Sweden, 145 km north of the Arctic Circle. The climate is subarctic with short summers and long winters. The average annual temperature is -1°C (30.2 °F) and Kiruna has approximately 200 days/year of snow cover. About 40% of the annual precipitation comes as snow. This means that the annual snow melt via the TSF (i.e. snow melt within the TSF, snowmelt in the catchment area as well as snowmelt from mining area) is a major part of the water being discharged in spring. The "water" that comes in to the system during the cold part of the year is, however, of no use until the temperature rises in spring.

2 THE GENERAL WATER MANAGEMENT SYSTEM AT KIRUNA

The mine in Kiruna is an underground mine, currently reaching 640 m below sea level. To keep the underground mine free of water the groundwater is constantly pumped to the ore refining plants. Process water is a combination of groundwater and recycled water from the clarification pond. An overview of the mining site and sounding can be seen in Figure 1.

The tailings is since 2014 dewatered and pumped as a slurry with 20-30% solids by weight. Excess water and waste water from all processing plants are collected and led in to one of two open trenches in which the water flows by gravity into the TSF at the eastern perimeter.

The TSF is the first clarification step of process water as well as the storage for tailings. The water is then discharged into the clarification pond south of the TSF. The clarification pond is the second and final clarification step. Water from the clarification pond is constantly pumped back to the processing plants for re-use and excess water is discharged to the recipient.

There are two more sources of water coming into the TSF. A nearby lake, Luossajärvi, is divided by a dam in order to keep the mining area dry. Surface water from the dry, or emptied, area of the lake is pumped to the TSF. There is also a small tailings- and clarification pond for tailings with high iron content next to the main TSF. This tailings are regularly dewatered and taken back into the concentration plant for re-processing. The water from this small clarification pond is pumped into the northern open trench and discharged into the TSF.

3 THE TAILING STORY FACILITY AT THE KIRUNAVAARA MINE

3.1 *General*

The TSF, Figure 2, was taken into operation in 1977. The tailings were from then to 2014 deposited by gravity through one of two open trenches at the eastern perimeter. The two fixed discharge points resulted in the creation of two sand cones in the eastern part of the TSF, with the coarse tailings close to the discharge points in the east and the finer tailings further out in the pond. Originally the free water pond in the TSF was covering most of the impoundment surface area except for the sand cones.

The surrounding dam walls are therefore built as cross valley dams the north and the south with design similar to water dams i.e. till core, filters and rock support fill. The tailings settling along the dams has been the fines as deposition has been towards the dams.

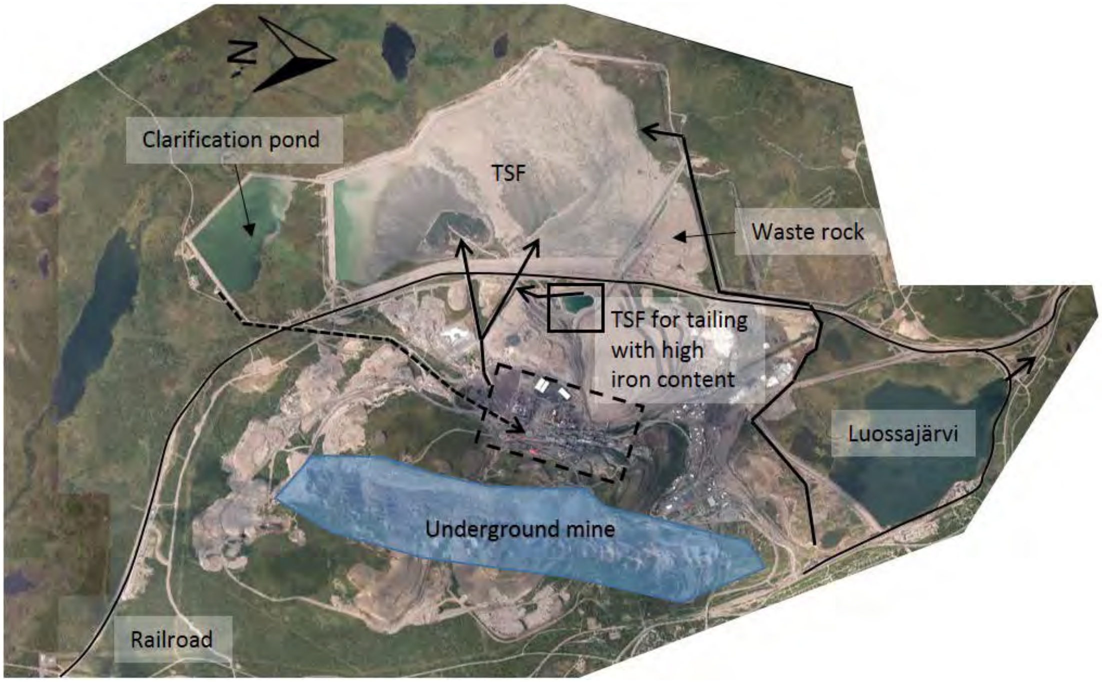

Figure 1 Overview of the water management system. The processing plants are located within the square with dashed lines. The open trenches south and north are marked as arrows from the process plants to the TSF. Pump line from Luossajärvi and the small TSF for tailings with high iron content is marked as arrows. Re-circling pump line is marked as a dotted arrow from the clarification pond to the plants.

The embankments were raised four times up to 2008 according to the center line method using the same type of design. By then the highest dam, the south embankment, was 15 m high from foundation to crest.

Due to ground deformations from the excavation in the underground mine the railway line to Kiruna had to be relocated. The new railway was placed along the east side of the tailings pond, between the plants/mine and the TSF. Due to limited space a new embankment, dam C2-B, on the east side of the TSF, partly through the TSF, was built between 2009 and 2011.

As the dams reached the final height (due to the width of the till core) and the iron ore production rate increased a decision was made 2011 to change the system for tailings management by changing the design of the embankments as well as the depositing method. During 2012-2014 the northern dam, dam C-C2, as well as a new dam, dam O-O2, on the northwest side, was changed into upstream construction with spigot deposition, which with time will change the shape of the TSF from sloping from east towards west into sloping from north to south. In 2015 all three slurry pipelines, one from each concentrating plant, were in use.

The water level in the TSF is controlled by a gated spillway in the Dam B-R-O close to point B, as seen in Figure 2. The water level in the clarification pond is controlled by two decant towers with fixed thresholds. One of the decant towers is also connected with the re-circulation pump station. A submersion of the dam crest close to point R acts as an emergency discharge if water levels gets critical.

In 2017 the area of the TSF is approximately 5.9 km^2, the total deposited volume of tailings about 36 Mm^3 and the water storage capacity at normal water level approximately 2.7 Mm^3.

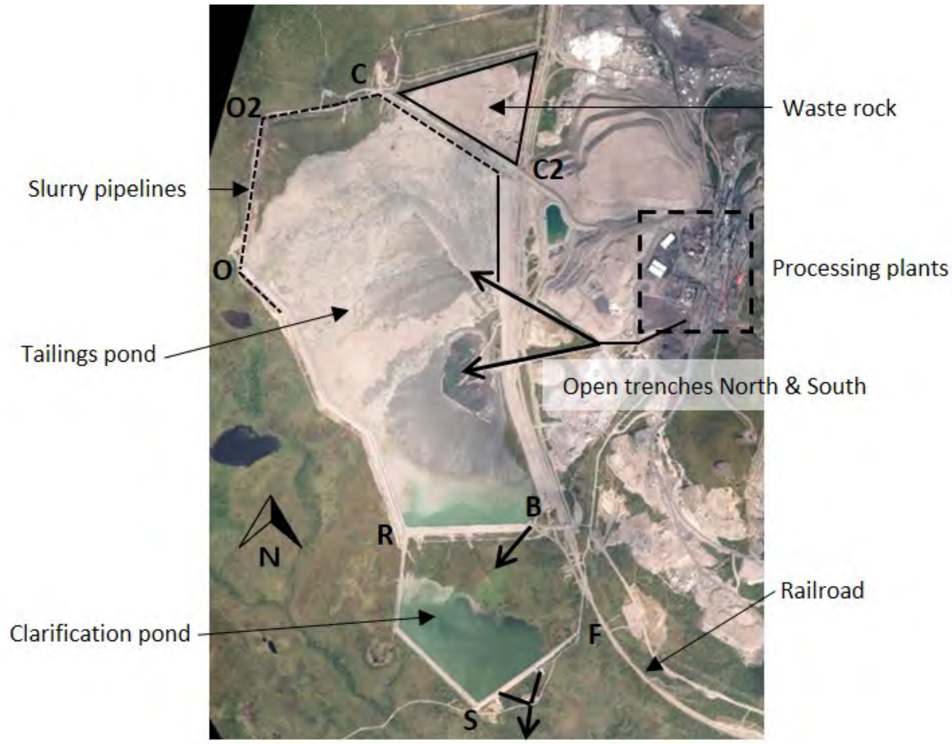

Figure 2 Overview of the tailings and clarification pond. Slurry pipelines are doted were deposition of tailings through spigotting take place.

3.2 *Dam O-O2*

The starter dam of Dam O-O2 was constructed in 2012. The design is upstream construction using tailings, filters and waste rock. The dam is connected to B-R-O, even though dam B-R-O has a different design, see Section 3.5. Tailings is spigotted from the crest of the dam. Dam O-O2 is 1163 m long with a maximum height of 6 m.

3.3 *Dam C-C2*

This dam is one of the original till core dams, which has been turned into an upstream dam from which tailings is spigotted. From the final crest of the original dam, constructed as a rock fill dam with a till core and one layer of filer material, the dam has been raised upstream in 2012 and 2013. The upstream raises are constructed of tailings, a filter and a layer of rock fill for erosion protection. Dam C-C2, is 1200 m long with the maximum height of 17 m.

3.4 *Dam C2-B*

This dam was built in the eastern perimeter of the TSF along with the railway. The dam is partly built through the TSF and therefore founded on tailings in the north part. The dam was constructed of tailings, three layers of filter and rock fill and is planned to be raised downstream with the final base already constructed during the first phase. Northern part of the dam from the northern trench to point C2 has been raised 2013. A till core is built in the connection to dam C-C2 to prevent rom water from flowing through dam C2-B into the rock fill of dam C-C2. Dam C2-B, is 2650 m long with a maximum height of 18 m at point C2.

3.5 *Dam B-R-O*

Dam B-R-O is also one of the original till core dams and since 2012 divided into two parts; one that is design for water upstream and the other part designed for having tailings upstream. Due to the original deposition method, the tailings upstream this embankment consist of the fine particles not sutible for upstream construction.

The original design, up to 2012, include a vertical till core, raised according to the center line method, two layers of filters downstream and rock fill upstream and downstream. From 2012 the whole length of the dam is rased downstream. For the south part, with water upstream, the raise consist of compacted tailings as the “sealing zone”, allowing for high seepage as the seepage water will be collected in the clarification pond downstream, with filters and rock fill downstream including a rock fill buttress of 300-500 mm rocks and erosion protection of rock fill on the upstream face. For the northern part with tailings upstream the design is similar except for no erosion protection on the upstream face.

At the moment tailings is not spigotted from the northern part of this dam eventhough the permit allowes for it. This is due to tailings setteling nicely along the dam when spigotted from Dam O-O2. Dam B-R-O is 3300 m long with a maximun height of 17 m in the south.

3.6 *Tailings deposition by spigots*

The tailings is, since 2015, deposited by spigots from the northern part of the TSF and the slurry pipelines are placed as shown in Figure 2. There is one pipeline from each of the three concentrating plants. Two of the pipelines has an inner diameter of 250 mm and the third 200 mm, where the latter is the longest. The pipelines have a length ranging from 2.5 km to 6.5 km. The spigots are placed with a spacing of 25-30 m and approximately 5-8 spigots are normally operating at the same time. Most of the top area of the TSF is 2017 covered by tailings and the tailings flow nicely in thin layers over the surface from the north to the south.

Tailings is preferable deposited far away from the plants, i.e. on Dam O-O2, during the warmer part of the year i.e. mid-April until mid-October. During winter, the pipelines are shortened and the tailings is deposited from Dam C-C2. This is due to the high risk of freezing and ice plugs in the pipelines in winter, especially during unexpected stops of the slurry flow.

The length across the TSF is about 3.5 km from the discharge points in the north to the spillway in the south. This is the distance the water have to flow during winter in order to maintain the recirculation of water. During winter, the number of spigots open are therefore reduced to about 2 spigots per slurry pipe line to increase the flow and reduce the risk of freezing.

The purpose of deposit the tailings in the north part of the TSF is to reshape the surface and increase the inclination from north to south so that the water easily flows over the tailing surface to the water pond where the spillways are located and then further into the clarification pond. In this way the capacity of the TSF is optimized as well as the water management.

4 CASE STUDY – TOWARDS LESS WATER IN CIRCULATION

The case study focuses on water management and difficulties with water loss due to freezing at winter, and describes the situation that arose during the winter 2014/2015, when changing from single point discharge along the eastern perimeter of the TSF, to deposition from spigots along the northern embankments. During this winter only one of three slurry line was taken into operation.

The current deposition scheme, described in section 3.1, comprises a change of the tailings surface by raising the northern part creating a surface area sloping from north to south instead of from east to west through spigotting from the northern embankments. The initial plan for this transition was to start spigotting from the northern part of the TSF already in 2012, and then start raising the northern embankments 2014, but as this didn’t happen (the change to spigotting) the original deposition scheme (single discharge points from the eastern embankments) was used past 2012. For this reason, the remaining capacity, rather limited already 2012, was pushed too far resulting in the southern part of the TSF intended for water storage was filled up with tailings which limited the water pond to a minimum affecting the first stage clarification step, which in turn resulted in some tailings spilling into the clarification pond.

The available capacity in the northern part of the TSF in 2012 was sufficient for the demand from 2012 to 2014 without raising any embankments, but the tailings could not reach that area by deposition from the two fixed single discharge points.

The start of spigotting from the northern embankments was delayed first because of restrictions in the environmental permit and later from delays in both design and construction of the slurry pipe line system. So, instead of changing to spiggotting already in 2012, spigotting could not start until in June 2014 for the first concentrating plant and not until 2015 for all three concentrating plants. The major consequence of the delay was that during this time tailings were mainly filling out the volume intended to be used for water storage, which in turn significantly reduced the surface for clarification, as seen in Figure 3.

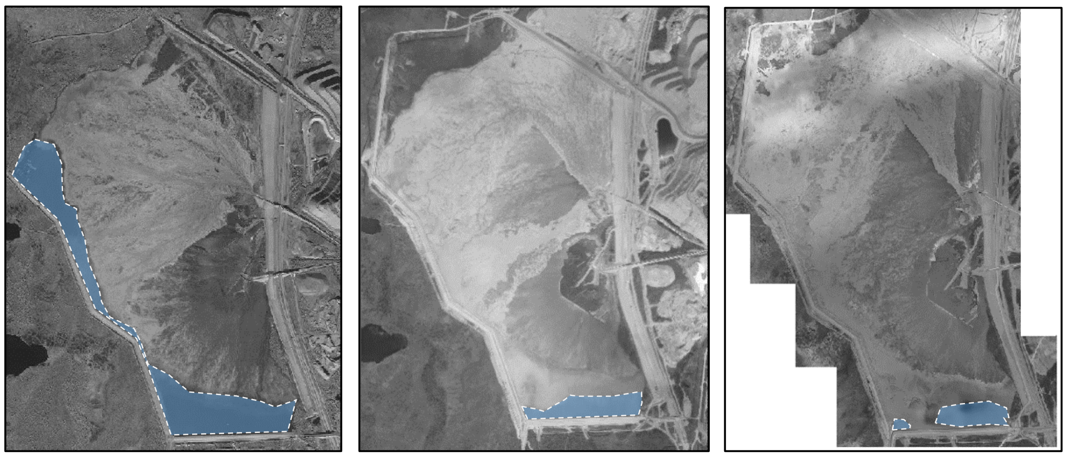

Figure 3. Reduction of available free water in the TSF, showing areas with significant water depths summer 2012 (left), summer 2014 (middle), summer 2015 (right).

In October 2014 it was concluded that, following the reduced storage capacity, not only clarification was limited, but the stored water volume in the system could even be too low in relation to expected losses due to freezing during the winter. There was a risk that freezing could occur to an extent that the recirculation of water would stop, which would affect the pellets production.

The available knowledge and experience of the water balance was not detailed enough to provide sufficient guidance and this resulted in the following objectives for deposition:

1. Minimize tailings transport from the northern part to the free water surface in the south, due to insufficient surface for clarification.
2. Minimize tailings transport from the northern part to the free water surface in the south, due to limited (and decreasing) available volume for water storage.
3. Maximize the deposition of tailings in the northern part of the TSF, where significant unused volumes are available.
4. Minimize the loss of water due to freezing, since the stored volume of free water is limited.

Point one to three would best be met through spigotting from multiple spigots from the northern embankments – resulting in the coarse fraction settling in the very most northern part of the TSF, and also the longest possible distance from discharge point to the free water surface. However, the last point, reduce the amount of freezing water, would best be met through deposition from a single discharge point located at the south-east side of the TSF. This would result in a concentrated flow and the shortest available distance from discharge point to the free water surface.

To optimize operations, four different alternatives for deposition were listed, ranging from "making use of the available northern volume with high risk of freezing" to "no use of the available northern volume and minimized risk for freezing". The available alternatives for deposition are shown in Figure 4. The remaining – more difficult – issue was then to determine what indicators would initiate a switch from one of these alternative to another.

Figure 4 Alternatives for deposition:

1) As many spigots as possible are used (some flow is still deposited from the eastern side, since the pumping system is not fully operational, meaning parts of the flow always have to be discharged via the trenches). A berm, earlier constructed to divert the flow from the single discharge point, is leading the flow from the eastern side further north.
2) Reducing the number of spigots used, still depositing parts of the flow from the north.
3) Concentrated flow from the northern discharge point on the eastern side.
4) Concentrated flow from the southern discharge point on the eastern side.

4.1 *Step 1: What water temperatures were expected?*

To get an idea of what water temperatures and widths of flow paths to expect a simple measurement of the water temperature was conducted at six locations in the TSF and one in the concentrating plant, KA1. The results are summarized in Figure 5 together with the general flow paths and their typical widths.

4.2 *Step 2: What is the risk of freezing – and what are the critical conditions?*

To facilitate planning, a simplified model was used to give an indication of the risk of freezing for the different deposition alternatives. Initially the shape of the channels formed on the tailings beach were assessed based on parable shape with the width 5,5 times the water depth, as suggested by Fitton (2007), which was then combined with the Manning formula to assess water velocities. With assistants from SMHI (Swedish Institute for Meteorology and Hydrology) the risk of freezing was assessed using HEC-RAS and the WQ-module, which may be used for calculation of heat transport. Radiation, sensible heat and latent heat was considered. The most important factors are wind and ambient air temperature, while atmospheric pressure and relative humidity are of less importance.

The results from the model are shown in Figure 6, and are presented as the distance the water is expected to flow before the temperature is reduced to less than 1 degree Celcius, depending on ambient temperature, the wind and the shape of the channel formed by the tailings (given by the current deposition alternative). It should be stressed that this simplified model was not expected to give a very accurate description of the situation, but instead used to assist in assessing and illustrating the risks related to the situation.

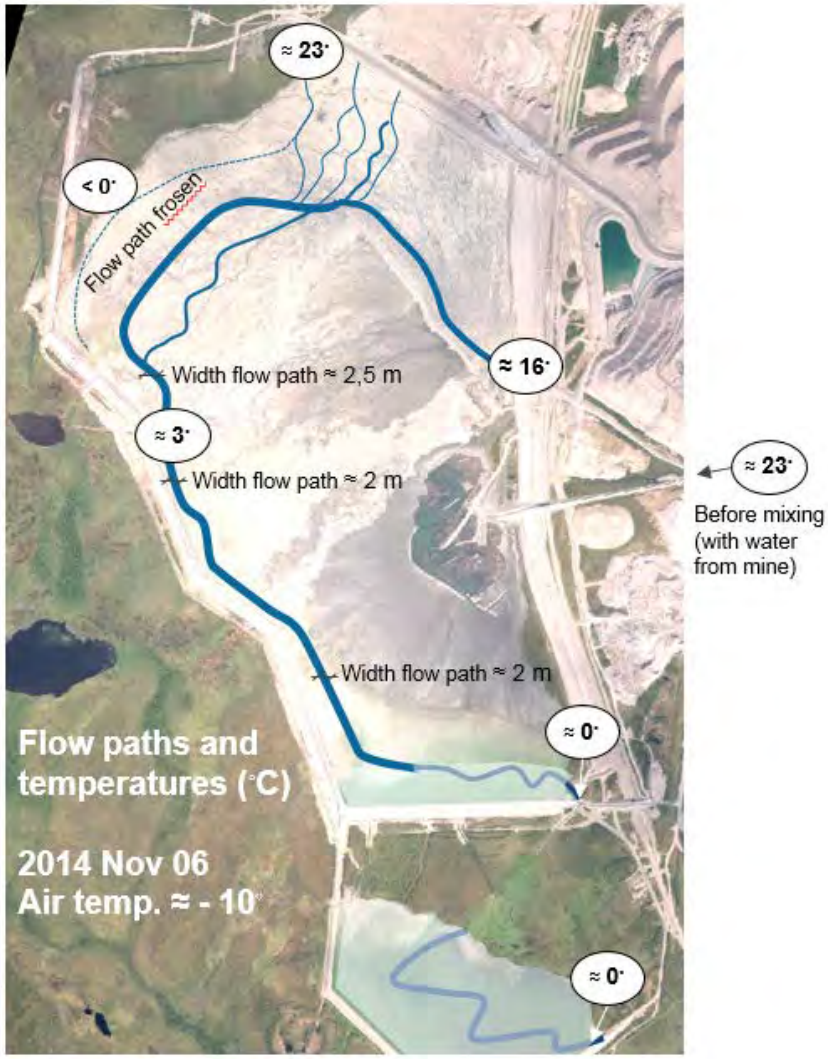

Figure 5 Measured temperatures in °C in different locations in the TSF as well as estimated width of the flow paths.

4.3 *Step 3: A plan for deposition, based on climate related indicators*

The model and observations on site both indicated that the flow from multiple spigots in the northern part of the TSF would risk freezing, while the concentrated flow from one of the single discharge points on the eastern perimeter would not be expected to freeze.

Based on this regular control of water temperatures and ice thickness was introduced. The later due to parts of the main water flow paths included areas with free water with rather shallow depths, which potentially could totally freez, i.e. all the way down to the bottom and thus blocking the flow path. The introduced control was combined with visual inspections regarding water freezing on the tailings surface.

Actions was decided to take place if; ice was forming in the channels of the water flow on the tailings surface, the ice grow thicker than 50 cm at the control locations, or the water level in the TSF started to sink (which would be the consequence of too much circulating water freezing instead of reaching the free water surface). The intended actions were to switch from one alternative to another, reducing the risk of freezing, according to Figure 4. The idea was simply to move from one alternative to another with less risk of freezing, until freezing stopped being a problem.

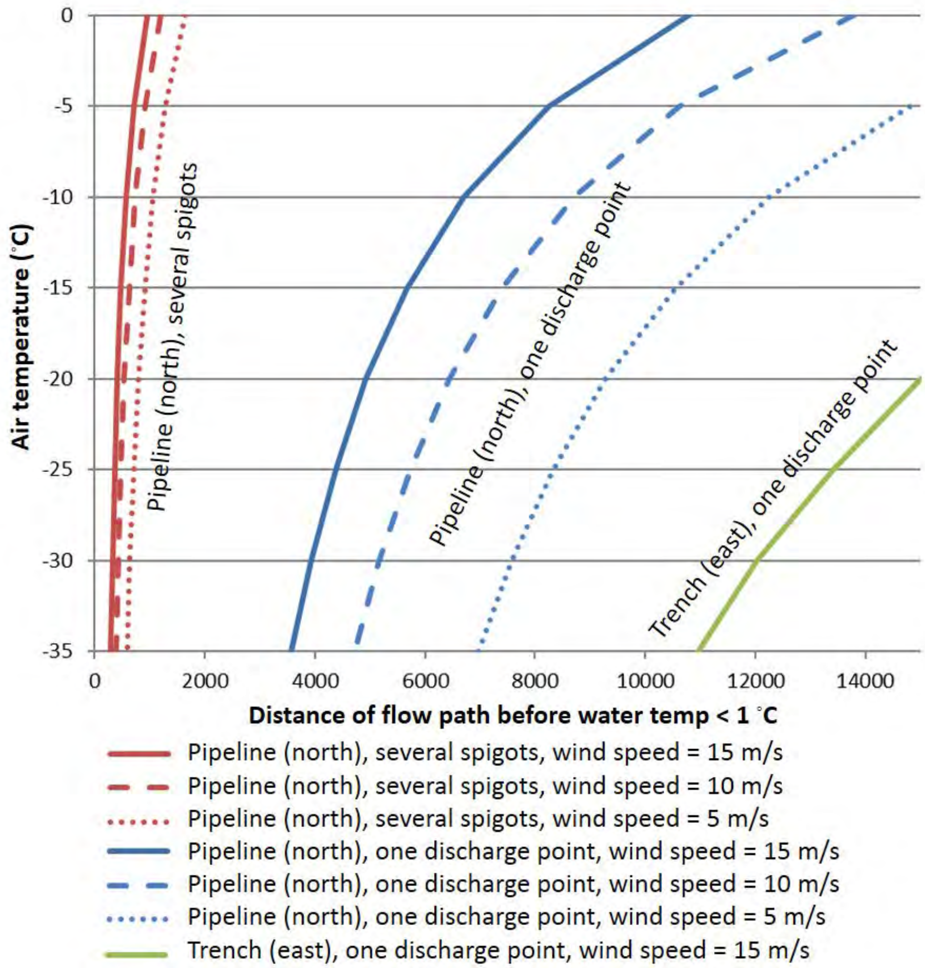

Figure 6 Critical distance for temperature loss in the flow paths as a function of air temperature and wind speed (SMHI, 2014).

4.4 *Results*

The main objective was met – the water level in both TSF and the clarification pond did drop, Figure 7, but not under critical levels. However, the situation with the extra control and extra measurements taken, combined with the action plan described above, did not result in a completely satisfying situation.

The initial thought, that actions were to be taken as soon as freezing occurred turned out inacceptable. Deposition according to alternative 1 until freezing was noticed and then moving on to alternative 2 etc., without going back to an earlier alternative would have resulted in too much tailings being deposited in the area of free water surface in the south part of the TSF.

Changing between the different alternatives involved a risk of losing water when changing from one alternative to another would result in a frozen flow path having to be melted while an open flow path would be left to freeze. Furthermore, the impact of wind was underestimated in relation to ambient temperature. Initially the ambient temperature (simply based on the weather forecast) was chosen as the main indication of the need to change deposition alternative. The wind had a lot greater impact than anticipated and drops in water level was observed during winter storms.

The combination of a system where actions came with a cost and where the governing condition was the combined effect of wind and temperature, in turn resulting in rapid changes that were rather complicated to predict, made operation very difficult.

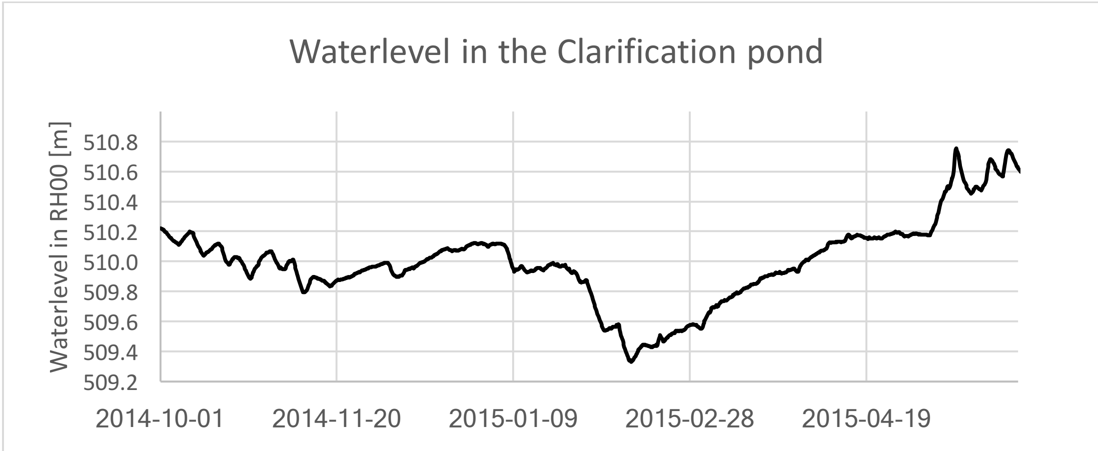

Figure 7 Water level variation in the clarification pond over the winter 2014/2015.

5 CONCLUSION

Conclusions were reached at different levels. For the presented action plan it was specifically concluded that assessing the risk of freezing in future situations (i.e. the combined effect of temperature and wind) was difficult and changes occurred too fast for actions to be taken.

On a more general level, for the described site, it was concluded that the loss of water was a significant risk to the operation of the processing plants. This had not been acknowledged as a real risk before, since the rather large inflow from the underground mine had always compensated for the amount of water freezing in the TSF. The impact of the changed deposition, both on the available volume for water storage and the risk of freezing of deposited tailings, had therefore not been considered fully.

Furthermore, it was concluded that a better understanding of the risk of freezing and a more accurate prediction combined with more sophisticated actions, would most likely still not have resulted in a satisfying situation. The only way to obtain an acceptable risk would instead be to simply increase the margins in the system. This was later done through rescheduling the raise of the southern embankments to increase the available storage capacity for water in the TSF. Raising the dams of the TSF along with the increased sloping of the tailings have now increased the margins in the system. The freezing problem was no major issue the winters that followed.

On an even more general level, that holds true for most tailings facilities in Sweden, this case was yet another example that the transition from one deposition scheme, with a large excess of water in circulation, to a more optimized scheme, with less water in the TSF, often gets to a weak point: If access to water has never been a problem, the water balance has usually not been given the attention necessary, and is most likely far from sufficiently accurate to be reliable. In the case of LKAB the company has now started work with gaining a lot more detailed understanding of the water balance, to avoid similar problems in the future.

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Changing our Thinking – From Tailings Processing to Engineered Fill

Janis Drozdiak
Golder Associates Ltd., Vancouver, BC, Canada

Jay Norwood
Golder Associates Inc. Walnut Creek, CA, USA

ABSTRACT: When it comes to tailings and tailings storage facilities, there is always a balance between the "wants" of the facility operators – simple/easy to operate, a facility which can accommodate any flow/concentration/PSD, low maintenance and inexpensive – versus the "needs" of the geotechnical engineers – efficient storage of tailings volume, geotechnically stable structure, resistance to the elements (rain, snow, flood, earthquake) and long term reliability. Tailings processing and delivery equipment has the mandate to connect these two potentially opposing objectives. In an effort to find balance, the concept of engineered fill can be introduced. Perhaps the most recognized use of engineered fill is that of paste back-fill where the tailings are manipulated in order to produce structural fill to support underground mining plans. Engineered fill also applies to surface tailings facilities with cyclone sand dams where the "design fill" (cyclone sand) is produced according to the geotechnical specifications. This paper discusses the expansion of the concept of engineered fill as a key element in the design of all tailings storage facilities. The site selection and the geometry of the tailings storage facility often set the requirements for the performance of the fill. With a clear concept of desired geotechnical results for the facility and the fill required to meet those, the processing equipment, deposition sequence and storage facility can be configured to produce this material.

1 INTRODUCTION

Tailings are a continuous by-product of the mining process with an equally continuous need to remove and deal with them in some manner. Tailings are produced in the plant and transportation to the disposal site is key to overall tailings system design. Most typical mine sites have a central location, be it a box, tank, sump, which will collect pretty much everything considered tailings. From this central location, the tailings are then transported in some manner and disposed of.

With tailings the typical "wants" from operations are the following:

- Easy to operate;
- Ability to handle any flowrate, fluctuations in solids concentration and oversize material;
- Low maintenance requirements;
- Low cost solution (money is made in the concentrator not in the tailings);
- Should never curtail or shut down production.

The needs for the tailings from a geotechnical standpoint are the following:

- Geotechnically stable structure;
- Resistant to the elements such as rain, flood or seismic events;
- Reliable long into the future – i.e. stand the test of time.

The connection between the "wants" of the operators and the "needs" of the geotechnical engineer is the tailings processing equipment with a view to make the operation as simple as possible while balancing the dam designer objective to make something as stable as possible.

2 ENGINEERED FILL

A shift in thinking is needed in the industry. Rather than consider the material exiting the plant as "tailings" or "waste", the concept of engineered fill should be substituted. By engineered fill, what is meant is managing the tailings properties as needed in order to produce the desired geotechnical result. From this thinking, the tailings processing and delivery method matches the configuration and geometry of the tailings storage facility itself, be it a big, flat impoundment, a valley fill or a cyclone sand dam.

2.1 *History of Engineered Fill*

Engineered fill is not a new concept in the industry. A commonly known example is that of paste backfill where the tailings are manipulated by processing equipment in order to produce structural fill for underground mining. By controlling the key parameters, such as slump (i.e. yield stress), binder content (e.g. cement ratio), cure rate of the material, and the placement and fill sequence, the performance criteria specified for short term and long term cure strength can be achieved. From this, the tailings transform from a waste product to an engineered fill that supports the underground mining plan.

Underground paste backfill material selection can include paste fill composed of mill tailings from the mill, modified tailings (i.e. with fines removed or coarse tailings added), reclaimed mill tailings (either from dedicated storage systems or from historic tailings deposits), sand (i.e. tailings with the fines removed through cycloning), tailings and sand mixtures in various proportions and tailings, sand and waste rock mixtures in various proportions. (Lee, 2015) The primary purpose of the selected backfill material is to provide a material that exhibits the strength properties in order to support both short and long term underground mining plans. Sub-requirements under the category of strength typically include unconfined compressive strengths, tensile strengths, Young's Modulus, strength vs time (both ultimate strength and the rate of strength development over time) and the geometry of the stopes and the surrounding rock (Lee, 2015). Selection of the paste plant equipment in order to make a proper "engineered fill" satisfying these criteria is the key to success.

An important lesson-learned in the paste backfill industry concerns selection of process equipment. Most early paste plants considered the addition of cement directly to the out-put of paste thickeners (deep-cone, steep bottom angle). Ore variability, concentrator operational variability and property changes from pre-development pilot testing would result in inconsistent thickener output. The only significant corrective action left to ensure the resulting plant output (engineered fill) met the performance criteria was to add cement. This would often increase the operating costs of paste fill past the viability of its selection. Current plant design includes at least partial filtration of paste thickener underflow to ensure the operator has the installed capacity to accommodate operational variability.

2.2 *Surface Engineered Fill*

The concept of engineered fill can also be applied to surface tailing facilities. Cyclone sand tailings dams are a form of engineered fill. In order to build a tailings dam with cyclone sand, the tailings must be processed in order to meet the dam building material specification. Plant tailings are processed by being cycloned into a coarse and fine fraction with the coarse fraction used as the dam building material.

This coarse fraction or sand must meet the sand specification set by the dam designer. Fines content is typically limited to no more than 10% to 20% as too many fines present in the sand prevent water from draining leading to permeability issues. As well, too many fines also change the sand deposition angle as fine particles flow further and therefore pushing the sand out further and reducing the slope. The objective of cyclone process is to provide construction material (engineered fill) that can be cost-effectively placed, drained, compacted, etc. to achieve the required civil properties of the storage dam.

Cyclone selection and configuration is a key component of this type of engineered fill. The coarseness of the grind is controlled by the mill in the plant, with tailings operators obviously having very little control over this. If the grind is expected to be fine, two stage cycloning is usually required in order to achieve the necessary fines content. The Cerro Verde mine in Peru is an example of this.

The specification of the cyclone sand must comply completely with the dam designers needs for compaction, dam slope and drainage. Geotechnical needs for dam stability include the sand placement sequence, rate of rise (i.e. can the sand production keep up with the dam raise schedule and the sand demand curve for dam building material versus the tailings storage curve. By using, the sand from the tailings a valuable geotechnical dam building material is conserved in order to benefit construction, deposition, consolidation and closure.

For this type of engineered fill, there are different types of tailings processing equipment configurations in order to meet the geotechnical building material needs. For a high sand demand, a fixed cyclone station is best as this type of arrangement has the most cyclone availability as there is no requirement to move cyclones to different locations. For a low sand demand, an on dam header mounted cyclone system is usually best. For a medium sand demand, a mobile cyclone is typically used.

2.3 *Thickening for Engineered Fill*

Although slightly radical, thickening of tailings can also be considered as engineered fill. To thicken or not and how much to thicken is a hotly debated issue in the mining industry today. Fundamentally, discussion of thickening tailings is usually centered on water reclaim and the cost of pumping large quantities of water back for use at the plant site. Key parameters for considering of thickening are water reclaim cost vs the cost of the thickener and the degree of thickening (i.e. type of thickener required – high rate, high compression and deep cone).

To bring the concept of engineered fill to thickening, beach slope is governed by the yield stress of the tailings (Luppnow et al 2009) which can be altered with the use of the correct thickener. Beach slope is determined by the slurry parameters which contribute to the yield stress. From a geotechnical perspective, the natural yield stress of the engineered fill is desired. Addition of flocculent in order to increase thickening and therefore the material's yield stress can be problematic as the material needs to be fully sheared at the point of deposition. Yield stress can decay in a pile which could result in slope instability which is undesirable for beach slope development. The deposition plan for the facility - single point discharge, multi-point discharge or widely spread layers also plays a role in the development of the desired beach slope.

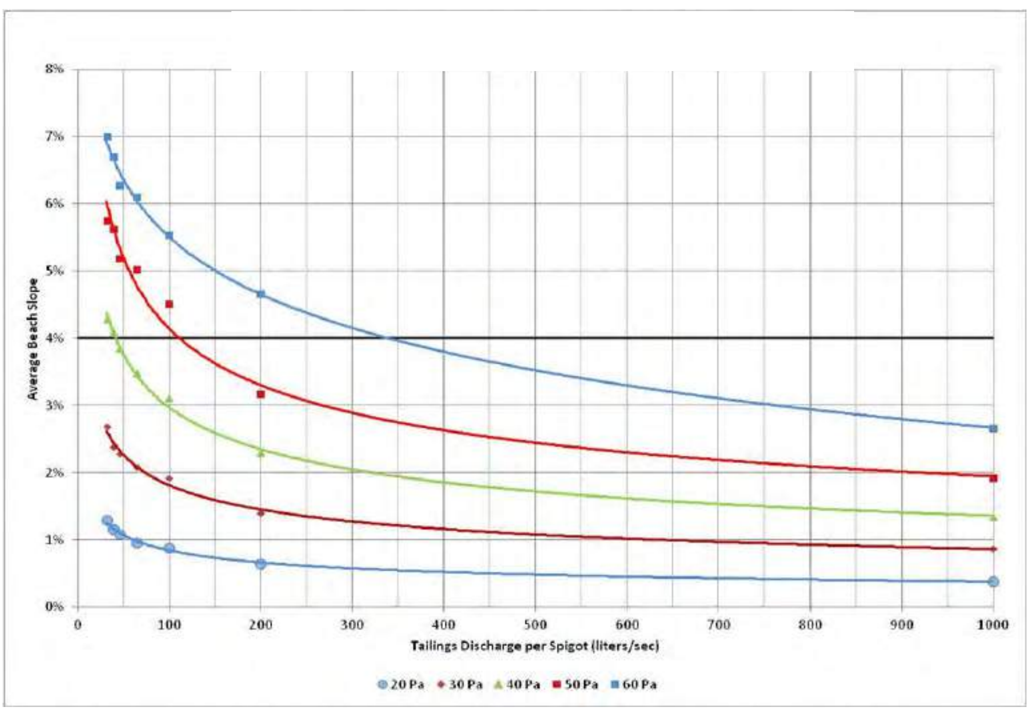

Figure 1 - Average Beach Slope vs. Tailings Discharge per Spigot

Figure 1 highlights the resulting average beach slope for different yield stress values versus the tailings discharge flowrate per spigot. It is important to realize that beach slope is determined by the material that comes out of the end of pipeline or spigot – i.e. at the point of deposition. (Li, 2011). Properties that come from the thickener outlet are not necessarily the same as those at the end of pipe due to the possibility of shearing of the material during transport. In addition, the underflow concentration is not equivalent to a yield stress value and the properties of the thickener underflow material cannot be controlled by the density of the material.

Selection of a thickener should be based on the engineered fill properties required at the end of the pipe rather than just buying a thickener in order to thicken. Figure 2 illustrates the effect of shearing time on the yield stress for different solids weight concentration that can be produced by a range of thickeners.

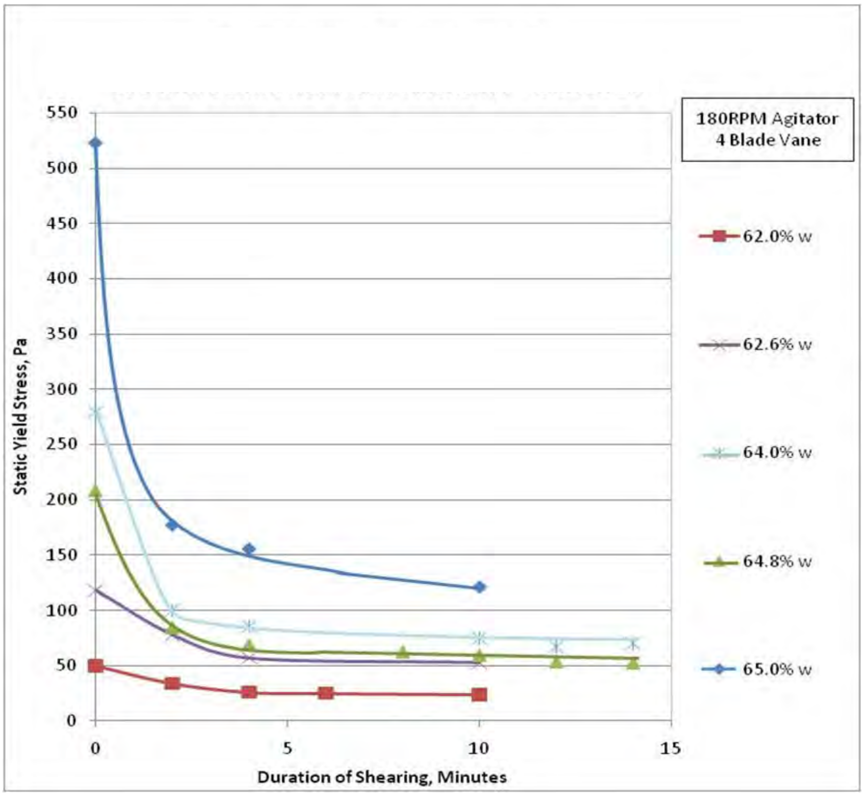

Figure 2 - Effect of Shearing Duration on Static Yield Stress

From Figure 2, the effect of shearing time on the static yield stress can be quite pronounced and consequently greatly affect the desired beach slope.

As discussed above, selection of thickened tailings as a management plan must consider the deposition plan required to achieve the target beach slope and thus provide the design tailings storage capacity. That is controlled by yield stress and discharge spacing (spigot size and number in operation). The yield stress of the slurry is controlled by the thickener.

The test work summarized in Figure 2 above was developed to understand what concentration range would be required to achieve 50 Pa yield stress at the pipeline/spigot discharge (fully sheared condition). In combination with testing of other samples from the proposed mine, it became clear that an in-situ thickener bed yield stress of ~200 Pa was required to consistently achieve 50 Pa at the end of pipe.

The 200 Pa yield stress exceeded the maximum value for the selected thickener vendors standard high compression thickener (HCT) but was far less than would be expected in a paste thickener. As a result, a custom thickener design was implemented with a residence time and bottom angle between the two "standard" models with the objective of producing consistent, end-of-pipe thickened tailings with a 50 Pa (yield stress) – engineered fill. The thickeners have been installed and are performing in line with design expectations.

It is important to also note that thickener control is a key component of ensuring delivery of the required tailings properties. Test work on tailings samples from pilot programs and commercial operations consistently demonstrates variability in the yield stress versus concentration curve across the ore body. Simple underflow withdrawal on underflow concentration (density) is not adequate to ensure the design properties are met. On-line measurement of yield stress is unreliable (at best) and might not provide accurate information as yield stress decay curves (as illustrated in Figure 2) are different from sample to sample.

The thickener operating objective is to produce "on-specification" engineered fill regardless of feed conditions. Underflow density, rake torque, bed depth, throughput, flocculent dosage and other variables must be integrated into the control strategy. Similar to the concentrator operations, ore variability must be learned and integrated into the thickener control philosophy.

2.4 *Engineered Fill and Storage Capacity*

With the use of thickened tailings as engineered fill, the storage capacity of the selected tailings impoundment can either be enhanced or reduced.

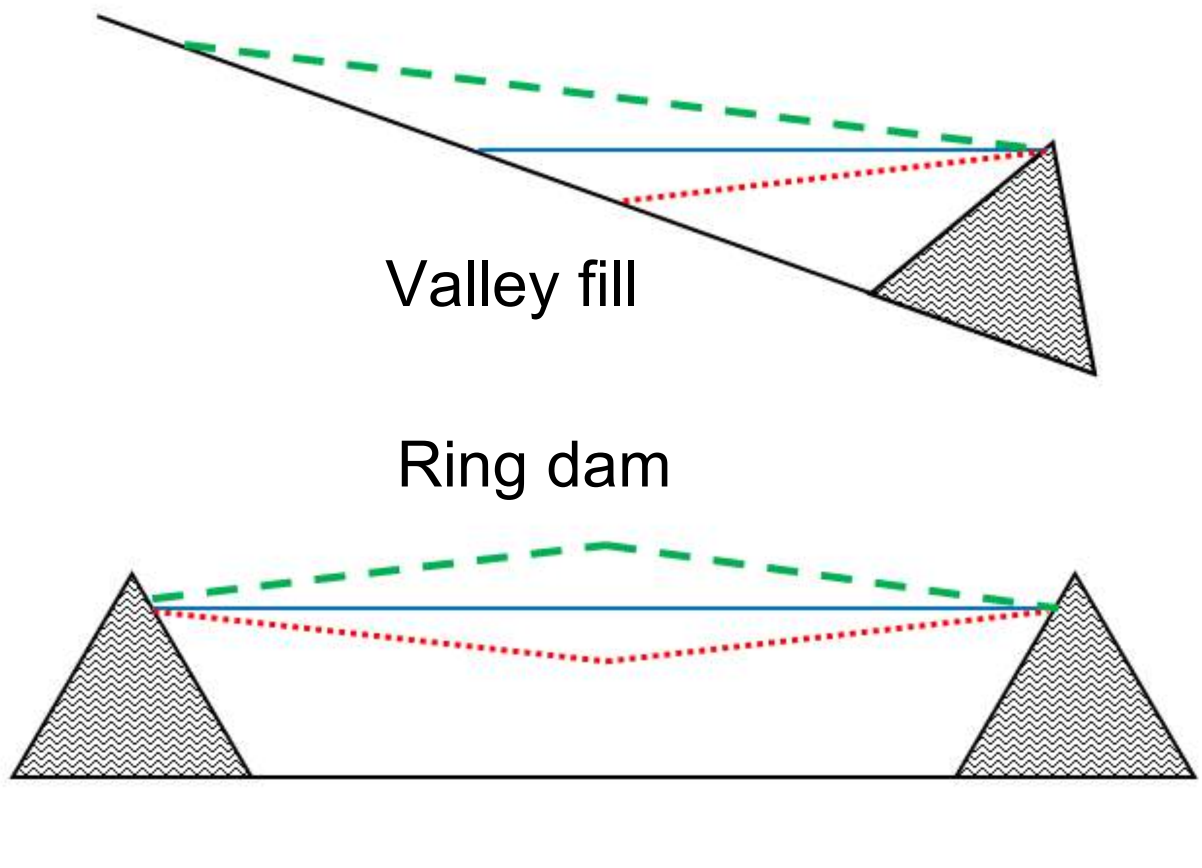

Figure 3 - Design Fill to Suit TSF Geometry

Figure 3 shows how selecting the configuration to suit the TSF geometry can aid the deposition plan. The red line illustrates how thickened tailings could hurt the storage capacity while the green line shows how thickened tailings could aid in increased storage capacity. The selection of the tailings processing equipment can also impact the selection – i.e. discharge from the back of a valley fill dam to increase storage capacity.

Central discharge of thickened tailings or paste can also add to the tailings storage volume if the chosen disposal site is flat. This method can also be used if the ground conditions are not stable enough to support a tradition tailings storage facility such as the case of the Kidd Creek tailings impoundment. The main disadvantage of a central discharge is the high flowrate of the discharge.

Site selection for the tailings storage facility in relation to the position of the process plant can have a large impact on the tailings processing system selection and where the components of the system are located. Planning the configuration and location of the tailings processing system can often be key to success in the early stages of mine life planning.

For example, if thickened tailings are selected as the engineered fill most appropriate for the tailings storage facility, it is often advantageous to have the discharge point from the thickeners close to the storage facility in order to minimize the pumping and power requirements to transport the thickened material. Depending on the process plant location, this may mean the thickeners are best located at the process plant (in the case of an adjacent impoundment) or at the tailings storage facility itself (in the case of an impoundment located significantly higher or farther from the process plant).

3 DELIVERY SYSTEM

The delivery system is the connection between the engineered fill and the tailings storage facility. Delivery systems commonly used vary from pipelines and pumping to conveyors to trucking. What system is most appropriate for the engineered fill depends on the key parameters of the fill and the chosen site for storage:

- Distance between tailings processing equipment and storage facility. There are limits depending on how much the tailings have been thickened. For conventional thickened tailings, the distance can be upwards to 50 km, while high density tailings have a limit of approximately 5 km and paste has a limit of 1.5 km per pump station. The longer the distance and thicker the material the great the cost in both equipment and power.
- Operating range of the plant site – i.e. mill shutdowns
- Deposition sequence into the tailings storage facility also needs to be considered. Where does the fill need to be placed? – i.e. one discharge location off the dam face, spigotting around the circumference of the facility or central point discharge and when does it need to be placed.
- Control of water - if any (depends on the type of engineered fill)
- Ultimate elevation of the facility
- Placement of the tailings processing equipment – i.e. either at the tailing storage facility or at the plant site.
- Geotechnical needs of the facility itself – designing the engineered fill to meet the needs of the facility and then the most appropriate processing equipment configuration and transportation system to get the fill where it is needed.

4 KEY CONFIGURATION PARAMETERS

Deposition planning between the geotechnical team and the tailings processing teams is critical. Geotechnical needs such as drainage, consolidation, curing and compaction, all need to be considered when selecting an appropriate engineered fill. Simple and repetitive sequences are best for deposition into the facility.

Planning for operational variability of the tailings itself – throughput changes, grind size variation, tailings properties, and weather issues – is also key to the success of the engineered fill for the tailings storage facility. Space allocation, pumping requirements (and associated power requirements), capital re-investments and closure planning need to all be considered during the configuration planning. Water pond management (if any) and the needs for reclaim water and closure also need careful consideration.

Starter dam optimization is a key part of the configuration parameters. Start-up water storage, storm water containment and containment of the initial tailings all need to be balanced with the height of the starter dam and the availability of the materials for dam construction. For example if the dam configuration is to use cycloned sand, the demand for that sand must be balanced with the starter dam height and when full production of sand can be achieved to ensure freeboard and containment requirements are met.

Selecting the most appropriate tailings processing equipment and the delivery system best suited to the tailings storage facility to produce an engineered fill, the wants of the operators and the geotechnical needs can be satisfied.

5 CONCLUSION

With a clear concept of desired geotechnical results for the facility and the performance requirements (flow, slope, consolidation, etc.) of the fill to meet these, the processing equipment, deposition sequence and storage facility can be configured to produce this material. In addition contingency in the selected equipment is key in order to handle operational variability.

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General guidance for the design of tailings dams in northern regions

S.A. Proskin and J.E.S. Boswell
Thurber Engineering Ltd, Calgary, AB, Canada

N. Sims
Northwest Hydraulics Consultants, Vancouver, BC, Canada

ABSTRACT: Developments in the field of tailings in the past three years have generated renewed scrutiny on tailings facilities. Tailings storage facilities in northern locations are no exception. Northern regions, such as Canada's northern territories, pose additional changes to geotechnical and hydrotechnical design. Seasonally frozen ground is encountered during winter conditions while other areas are underlain by permafrost with an overlying active layer. Surface water varies from ice covered to an open water season with significant seasonal fluctuations in flow rates. These regions are usually sparsely populated and are often isolated with limited access to equipment and materials. The combination of these conditions can have a significant impact on the construction, operation, closure and reclamation of northern tailings facilities. This paper describes some of the developments and experiences of the authors in the geotechnical and hydrotechnical dam safety review of tailings facilities. Geotechnical engineering considerations include use of frozen materials, permafrost foundations, changes in thermal regime, and credible failure modes. Hydrotechnical engineering considerations include those for freeze-up, break-up, and potential ice blockages affecting conveyance and downstream channels and the impacts of rain-on-snow events and snow melt. Finally, there are overall dam safety considerations related to safe access to the site, risks associated with water storage, and environmental aspects of reclamation and decommissioning.

1 INTRODUCTION

Recent high profile international tailings flow liquefaction events have placed renewed scrutiny on tailings designs, and have drawn attention to new and unusual mechanisms of failure and risks as described in more detail in a sister paper at this conference: Boswell and Gidley (2017).

Tailings facilities in northern climates are not exempt from this scrutiny. At heart of the challenge is a new dam safety imperative requiring drastic change in water storage: the removal of water from tailings facilities.

This paper re-examines some of the fundamentals associated with geotechnical design in northern regions and the application to tailings. It includes practical recommendations in regard to thermal aspects of tailings design in order to provide sustainable solutions for mining in the longer term.

2 GEOGRAPHY OF NORTHERN REGIONS

2.1 *Definitions*

Indigenous and Northern Affairs defines Canada's North as the Northwest Territories, Nunavut, and the Yukon Territory. However, northern parts of British Columbia, Alberta, Saskatchewan, Manitoba, Ontario and Quebec have a similar geography. The North accounts for 40% of Canada's land mass and it has world-class mineral, oil and gas deposits. The North is also at the forefront of climate change impacts and adaptation.

There are several definitions of the Arctic region. Some of the more convenient definitions include the region north of the Arctic Circle (66 33 N), the regions where the mean July temperature is 10 C, or it is northernmost limit of the tree-line (roughly following the July 10 C isotherm). Besides parts of Russia, Canada, United States, Greenland (Denmark), Finland, Iceland, Norway and Sweden, it also encompasses the Arctic Ocean and adjacent seas.

The defining characteristics of the arctic include extreme fluctuations between summer and winter temperatures, permanent snow in the high country, and permafrost. Summer long daylight hours somewhat compensate for the short summer. Winters have very short daylight periods. Above the tree-line the flora dwarf shrubs, sedges, lichens, and mosses forming a tundra and extensive peat areas. Land animals include the Arctic hare, muskox, and caribou herds that are preyed upon by Arctic fox, wolves and Grizzly bears. Polar bears hunt for marine animals from sea ice, such as seals.

2.2 *Permafrost*

Permafrost is perennially frozen ground remaining at temperatures below 0 °C (Harris et al. 1988). Based on this definition, the water phase should occur as ice but salinity and mineralogy can permit water to exist at sub-zero temperatures. The presence of ground ice in permafrost is the key element influencing the geotechnical behaviour of soils and bedrock. The large-scale landforms associated with permafrost terrains, such as ice patterned ground, are another consideration for construction design (Johnston 1981). Consequently, the distribution (or areal continuity) of permafrost of the polar north and northern Canada and Alaska have been mapped and classified (Brown 1970, Harris et al. 1988) with the regions summarized in Table 1. As shown in Figure 1 Canada's territories are underlain by continuous or discontinuous permafrost while isolated and sporadic permafrost is found in the northern parts of provinces (except the Maritimes). Subsea permafrost occurs offshore in the western arctic. Most of Alaska is underlain by permafrost as shown in Figure 2. The permafrost type and ground ice conditions should be investigated as part of the geotechnical design.

The active layer is the layer above the permafrost that thaws and freezes with the seasons. It represents the thermal equilibrium between the meteorological conditions and ground surface and subsurface conditions. The active layer depth and thermal regime are not only sensitive to meteorological conditions (temperature, snow cover) but also the present of peat, soil cover, water bodies, and vegetation. Changes to these, such as stripping peat, would lead to thawing while the site re-establishes the thermal equilibrium. This can take years while extensive thaw settlement and erosion take place.

Table 1. Permafrost extent classification

Permafrost Type	Area Underlain by Permafrost Percent	Ground ice content Percent
C-Continuous	90 to 100	10 to more than 20
E or D-Extensive discontinuous	50 to 80	0 to 15
S-Sporadic discontinuous	10 to 50	0 to 10
I-Isolated patches	0 to 10	0 to 10
U-None		
O-Subsea permafrost		
G-Permafrost associated with glaciers		
W-Inland water bodies		

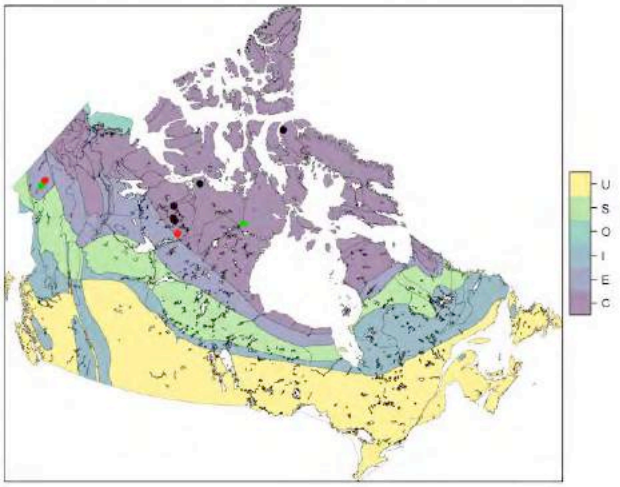

Figure 2 Permafrost distribution in Canada (see Table 1 for explanation of nomenclature)

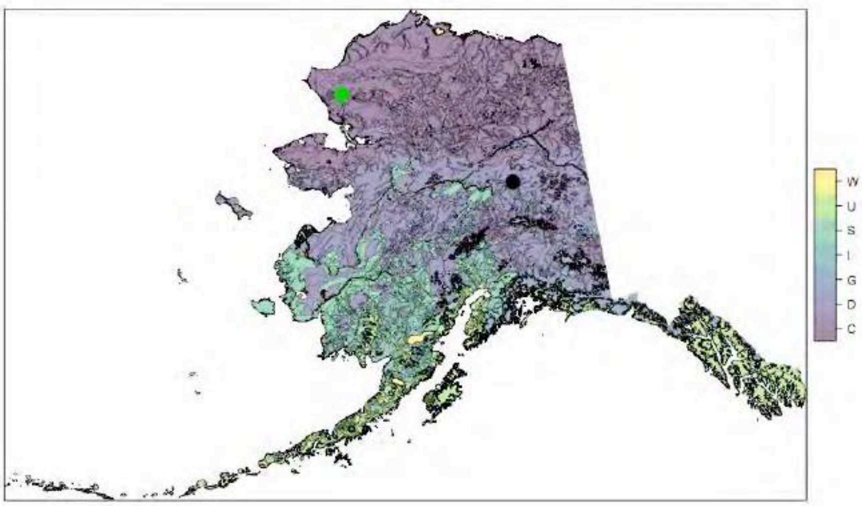

Figure 1 Permafrost distribution in Alaska (see Table 1 for explanation of nomenclature)

Saline permafrost, permafrost with pore fluid containing dissolved salts, exists across northern Canada (Hivon et al. 1989, Hivon and Sego 1993). The saline pore fluid alters the engineering properties of frozen ground mainly by lowering the freezing point of the pore fluid. In particular the creep and shear strength of saline permafrost are reduced (Nixon and Lem 1984). Locations with higher salinities are associated with marine submergence that occurred in recent geological history (usually associated with glaciation).

Taliks or unfrozen zones with permafrost can be found near/under water bodies and should be identified during the site investigation as these can serve as conduits for seepage in permafrost foundations.

Northern communities have small populations and impose constraints on site transportation and access to equipment and materials. Transportation options among communities and project sites northern Canada can be limited, seasonal and expensive compared to southern Canada. The Yukon has an extensive network of all-weather roads and includes the only road that reaches the Arctic Ocean (via NWT) (Highways and Public Works 2017). In the NWT, all weather roads connect Hay River and Yellowknife to northern Alberta. There is an extensive network of winter roads that are built each year to many communities and diamond mines (Proskin et al. 2011). Coastal communities rely on sea-lift for delivery of bulk goods and supplies during the short open water season. Otherwise most communities are served by air transport. Climate change may have substantial implications for northern transportation (Prowse et al. 2009).

Remote northern areas often have sparse coverage for meteorological, hydrotechnical, geotechnical and permafrost data. Consequently, site characterization is important to define site conditions early and this includes installing meteorological stations to gather climate data and hydrotechnical stations for gathering river flow and water levels. Local knowledge or indigenous knowledge may offer additional information about site conditions, previous land use and other insights that would supplement technical knowledge.

3 GEOTECHNICAL ASPECTS

This section reviews the geotechnical considerations that are particular to tailings dams and embankments in remote arctic settings. Soviet experience with water retention dams paved the way for earthfill construction in the arctic. Since the 1970s, mining activity in northern Canada and Alaska has led to the construction of tailings dams with those discussed here shown in Figure 3.

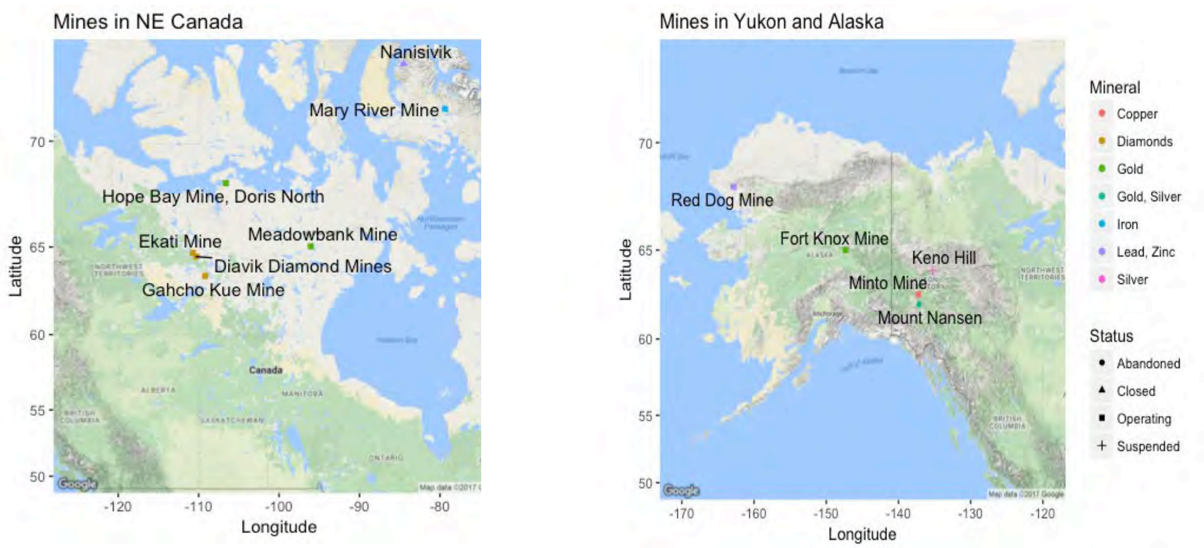

Figure 3 Mines in NU, NWT, Yukon and Alaska with frozen or unfrozen dams

3.1 *Initial Planning and Site Characterization*

Geotechnical site characterization is fundamental to identifying the geological and thermal regimes of the project site and providing data for initial siting and design. The major components are: initial office study and planning; field investigation(s); and thermal regime monitoring. The initial office study gathers general information from previous studies and public sources. This will assist in planning the initial field investigation by selecting candidate sites for the various geotechnical structures. Terrain analysis of aerial photography is useful in identifying permafrost features that will affect site selection and the performance of engineering structures. Coastal areas should be checked for the potential occurrence of saline permafrost. Field investigations will be planned taking the office-based studies that have identified information gaps and prioritized areas for focussed characterization. The logistics of the field investigation should also be considered at this stage in view of the required equipment, site access, and budget. Qualified personnel should have sufficient training and experience to conduct the field investigations in permafrost terrain and describing ice rich ground conditions.

Field investigations need careful planning and preparation to enhance data recovery while keeping costs under control. Methods of investigation may include geophysics, test hole drilling, test pitting and ground temperature instrumentation. Special drilling and sample recovery techniques are needed to recover and log the ground ice found in soil or rock. Ground ice is logged according to methods for describing the occurrence of ice within the samples (ASTM D4083-89 (2016) 2016). The upper surface of bedrock also needs careful investigation to ascertain if ice fills the discontinuities (Canadian Geotechnical Society 2006) that could later thaw. Geophysical methods that rely on seismic and electrical properties sensitive to ice content in the ground may have a role in delineating the horizontal extent of ground ice over a site (Pascale et al. 2008). Saline permafrost, if it exists, should be delineated across the site through measurements of salinity of the pore water. Andersland and Ladanyi (2004) provide more details and options for site characterization.

Thermal regime monitoring considers those field measurements that can characterize the site ground temperature conditions. Ground temperature cables, having been installed during the site characterization program(s), should collect data over a sufficient depth to define the active layer and the depth of permafrost as it relates to the excavations or foundation depths. Air temperature, precipitation, and snow cover are among the meteorological site conditions that will influence ground temperatures. A suitable meteorological station is recommended to collect reliable data for geotechnical, hydrotechnical and other purposes. Ideally advanced collection should take place 3 years ahead of construction to provide sufficient time to characterize the active layer and permafrost temperatures (Sayles 1984).

Geotechnical laboratory testing provides index data to classify the soils by standard methods and aid in quantifying the ice content and potential thaw settlement. Measuring the strength-deformation properties of frozen soils requires special sample preservation and shipping methods along with laboratory equipment that can test frozen specimens at sub-zero temperatures.
Several references provide guidance for site investigations (Johnston and National Research Council Canada 1981, Andersland and Ladanyi 2004, 2004, EBA Engineering Consultants Ltd. 2004, Canadian Geotechnical Society 2006, Canadian Commission on Building and Fire Codes and National Research Council of Canada 2015).

3.2 *Dam Design Concepts*

Construction and engineering experience for water retention and tailings dams in permafrost regions has developed into two categories of designs: (i) frozen and (ii) unfrozen structures. Frozen structures are designed to preserve foundation permafrost. They may have an unfrozen embankment or they may incorporate frozen elements within the embankment to provide seepage control and structural stability. Frozen structures are appropriate for sites with continuous permafrost and where the foundation soils would become unstable if allowed to thaw. They require passive or active measures to maintain the frozen state of the foundation permafrost and additional measures to build any internal frozen elements and keep them frozen. Unfrozen structures permit thaw of the foundation soils and do not rely on any internal frozen elements for seepage control or structural stability. Unfrozen structures are suitable for permafrost foundation soils that are thaw-stable or on sound bedrock. Thawing of the foundation permafrost occurs gradually and any thaw settlement needs to be accommodated by the dam while it maintains satisfactory performance.

3.2.1 *Unfrozen Dams*

Unfrozen design and construction methods are similar to southern construction design and construction except the thawing of the frozen ground in the dam foundation and slopes of the reservoir have to be considered. A particular concern is water seepage through the foundation and abutments.(Bogoslovskiy et al. 1963).

Sayles (1984) recommended the following general practices: (i) build impervious zone with self-healing soils so that it can accommodate differential settlement during foundation thaw; (ii) flatten embankment slopes; (iii) overbuild the height to account for anticipated settlement; (iv) undertake on-going repairs and re-build portions of the embankments undergoing settlement beyond limits; (v) incorporate sand drains in the thawing foundation to accelerate consolidation settlements; (vi) grout to stabilize thawing foundations; and (vii) account for icing of filters and drainage systems used to control seepage through and beneath the embankment.

The Red Dog Mine tailings dam was designed as an unfrozen dam on a permafrost foundation that was expected to undergo some thaw, settlement and creep deformation. Their design considered removing the ice rich soils or flattening the overlying embankment slopes to accommodate potential thaw deformation (Hammer et al. 1988).

3.2.2 *Frozen Dams*

Sites for frozen dams should be situated on continuous permafrost and where the foundation soils are unsuitable in a thawed state. The key requirement is preserving the frozen state of both the embankment and foundation since these are critical to seepage control. However, frozen materials require careful thermal and erosion controls to prevent thaw and loss of materials. Sayles reported (1984) experience with frozen embankments indicated those with permanent reservoirs should be situated in areas with a mean annual temperature colder than -8 °C. Even in these locales embankments higher than 10 m would require refrigeration to maintain the frozen state.

Frozen embankments may consist of an unfrozen upstream zone and a frozen downstream zone. The upstream zone acts as an insulating layering between the water reservoir and the frozen embankment. The frozen embankment performs as the seepage barrier and resists the horizontal loads.

Typically, unfrozen embankments on permafrost are limited to areas with cold permafrost where the water is to be retained by the embankment for a relatively short time.

Spillway and water outlet control structures in permafrost require special considerations. These should not be located within and or adjacent to ice-rich permafrost without refrigeration.

The Crescent Lake Dame, Thule, Greenland was one of the first projects to build a frozen core dam on permafrost in North America. Top of the water elevation was kept below the top of the frozen portion of the dam. These structures showed it was feasible to construct thermally and structurally stable dams with permanent water reservoirs because the pervious fill materials were sufficiently frozen to cut off seepage (Linell and Johnston 1973).

Frozen core dams have been built at mine sites in northern Canada. Several frozen core dams were built at Ekati on continuous permafrost as part of their tailings and waste water disposal plant These dams were built with a frozen gravel core that was designed to remain below - 2°C during the life of the project. (EBA Engineering Consultants Ltd. 2004). A frozen core tailings dam was built at the Doris North Project, Hope Bay (Rykaart et al. 2015).

3.2.3 *Thermal Analysis*

Thermal analysis is required when frozen fill or permafrost foundations are part of the design (Bogoslovskiy et al. 1963, Sayles 1987, EBA Engineering Consultants Ltd. 2004, Cassie et al. 2009, Pigage et al. 2012). Establishing the thermal regime requires deep ground temperature cables and several years of data because of the large seasonal and operational variations (Dufour and Holubec 1988). Commercial software packages are available that permit 1-D and 2-D thermal analyses using meteorological boundary conditions. Thermal modeling requires knowledge of the ground thermal properties (thermal conductivity, specific heat) and meteorological conditions (surface temperature, snow cover, solar radiation and wind speed). Climate change can be analyzed by adopting an appropriate warming scenario in terms of change of average monthly or annual air temperatures (EBA Engineering Consultants Ltd. 2004, Cassie et al. 2009, SRK Consulting 2014). Ground temperature data from the site is needed to calibrate the thermal model before it is used to simulate changes due to embankment construction and water/tailings impoundment(Weaver and Kulas 2003).

3.2.4 *Seepage Control and Water Management*

Seepage control and water management are closely associated with the thermal regime of either a frozen or unfrozen dam. In addition to design aspects of seepage, filter zones, erosion and differential settlement, there is added concern of thermal stability of permafrost soils and fills. For an unfrozen dam on permafrost, preserving the foundation may entail refrigeration of the foundation during construction and during operations. Furthermore, a positive seepage cut-off is sealed to the frozen foundation and extended up to the embankment crest. An economical cut-off alternative is to develop a frozen zone by letting embankment soils freeze when water is not being stored.

Liners have been used as the primary seepage control for unfrozen dams on permafrost. At Red Dog Mine tailings dam their unfrozen dam incorporated a geosynthetic liner on the upstream face and extended into trench to control seepage through and beneath the dam. A filter drain was also included to intercept any seepage past the liner. Considerable seepage occurred through the foundation when it thawed due to the heat from the tailings reservoir. To reduce the foundation seepage an upstream blanket of fine-grained soil was installed (EBA Engineering

Consultants Ltd. 2004). At Diavik Mine the processed kimberlite containment dams employ either an HDPE or bituminous geomembrane on the upstream face. The liners were keyed into a cut-off trench in permafrost on the upstream side of the rockfill shell (Cunning et al. 2008).

In warmer permafrost regions experience with Hess Creek Dam in Alaska and Kelsey and Kettle generating stations in Manitoba has shown seepage under permanent reservoir storage. In particular, the latter had undergone rapid thaw settlement as high as 1.5 m without failure. A series of sand drains were installed in the ice-rich foundation materials to assist in stability during thaw. There's a risk of failure developing as frozen shell separates from the settling material creating a seepage path (Linell and Johnston 1973).

Linell and Johnston (1973) noted Canadian embankments have shown significant longitudinal and transverse cracks to depths of about 3 m due to growth of ice lenses in frost susceptible core materials. The Waterloo Lake Dam in northern Saskatchewan experienced longitudinal cracking and silt outflows along the length of the crest (Solymar and Nunn 1982). It was postulated that frost penetration initiated ice lensing in the silt core leading to erosion and settlement of the core. Thermally induced stresses during winter have also contributed to cracking. This cracking should be anticipated and accounted for in design.

Unfrozen zones (taliks) within permafrost have been found on dam sites. At Lupin Mine Dam 1A seepage was observed that was attributed to flow through weathered bedrock within a talik (Dufour and Holubec 1988). A talik was found at Red Dog Mine in the middle of a creek within the tailings dam footprint (Hammer et al. 1988).

At the Mount Nansen mine tailings dam seepage control consisted of beaching the tailings on the upstream dam face. However, it did not develop sufficiently and excessive seepage caused the permafrost to thaw causing a positive feedback that increased seepage and induced downstream slope stability issues (EBA Engineering Consultants Ltd. 2004).

3.2.5 *Stability and Settlement Analysis*

In general, the stability analyses follow practices for southern dams. An upstream, unfrozen shell is susceptible to shear failure due to thawing generated excess pore pressures. Rapid draw down is another consideration. For unfrozen embankments, some additional consideration is given to the low shear strength that may occur at the interface of the thawing permafrost.

Unfrozen dams on a permafrost foundations can be assumed to undergo some thaw, settlement and creep deformation. Either the ice rich soils will have to removed or the overlying embankment slopes will have to flattened to accommodate potential thaw deformation (Hammer et al. 1988).

3.2.6 *Cooling Methods during Construction*

Frozen dams require extra care and effort to achieve the desired temperatures and cooling of the foundation and frozen dam elements. Construction during winter permits the use of natural cooling to preserve the foundation or to employ cooling systems. Natural cooling is most likely to work for small embankments with heights less than 10 m relying on winter subfreezing temperatures to freeze the exposed slopes and the dam crest (Sayles 1984). Techniques for augmenting natural cooling include snow clearing from the downstream face, building a shelter over the downstream slope to prevent snow from covering it and bermingthe down stream toe to prevent heat transfer from tail-water.

For embankments up to 25 m, more active techniques for cooling have been deployed, such as artificial refrigeration by circulating natural chilled air when the mean annual air temperature (MAAT) is less than -5 °C and refrigerated liquid brine, ammonia and carbon dioxide have also been used. These are usually distributed through the embankment fill into the permafrost foundation via vertical or horizontal pipes.

Frozen dams with frozen cores have been built during winter using equipment to place fill in lift and letting it freeze prior to placing the next lift. At Ekati the frozen core was keyed into the frozen foundation with an upstream sand zone, a downstream rockfill zone and a rockfill shell. Structures attained a height of 20 m and length of 250 m. At Doris North, Hope Bay an asphalt plant was used to moisture condition the non-saline crushed basalt and the fill was placed only when temperatures were colder than -10 ºC. Since the core was not completed in one winter season, it was covered/insulated with temporary crushed rock layer. This was removed next winter season and the frozen core completed.

3.2.7 *Monitoring and Instrumentation*

Early detection of unplanned thawing or seepage is critical in identifying early signs of the loss of permafrost. Ground temperature monitoring and seepage monitoring networks are required to confirm the functioning of the frozen cut-offs and preservation of the foundation permafrost. Ground temperature cables installed in boreholes or in trenches are the primary means of monitoring the thermal regime in the foundation and the dam. Frost gauges are some times used to determine the surface depth of frost. Vibrating wire piezometers monitor pore pressures in unfrozen portions and can provide additional temperature measurements with their built-in thermistor.

The time frames establishing stable ground temperature regimes can vary widely. At the Crescent Lake Dam in Greenland two winters were required to establish stable thermal regime in the 3.6 m high embankment and another two years when the height was increased by 2.4 m(Fulwider 1973). At Red Dog Mine their thermal analysis time frame has considered climate change and water impoundment effects over decades (Weaver and Kulas 2003).

4 HYDROTECHNICAL ASPECTS

The hydrotechnical aspects of design and monitoring of northern dams are similar to the aspects considered for dams located in other regions; however, with the added consideration of possible implications of climate change (deglaciation) and ice conditions (such as ice-covered channel blockages, ice jams, freeze ups, de-icing).

From a hydrological perspective, the event drivers for the development of an inflow design food (IDF) should consider possible deglaciation (due to climate change) and also scenarios such as rain-on-snow events. Hydrometric monitoring to support hydrological assessment and also operation may encounter difficulties with generating consistent rating curves for channels due to ice effects. Also, northern conditions may result in damage of hydrometric gauges by mobilized ice impacting gauges.

Key hydrotechnical considerations include on-site reporting of ice conditions. In addition, simulating of ice processes such as the formation of freeze-up, ice cover evolution, and break-ups as determined by climatic and hydrologic factors and developing measures to eliminate or decrease ice problems. Any potential conveyance systems associated with the mine should consider the potential for frazil ice to decrease conveyance and effectiveness of any pumps, valves, and syphon systems. Dam breach and inundation assessments must consider the channel downstream of the facility with respect to ice cover, ice jams and breakups as it may change the possible inundation extents of a failure. Also, specifically for any associated conveyance system (or dam breach path), there should be assessment of whether low velocity, shallow flow in cold weather could freeze developing think layers of ice blocking the channel (a process sometimes referred to as aufeis).

5 ENVIRONMENTAL AND SAFETY ASPECTS

Access to site is a key consideration for any project; however, it is typically considered a more challenging initiative for a northern dam given the remoteness, and sometimes extreme, rapidly changing weather conditions. Along with site access, the consideration of response times to site is an important long-term consideration from an operational and surveillance perspective. Potential loss of life following a potential dam breach is a consideration for all dam sites; however, may be less likely for a remote northern site. Potential damage of key infrastructure for accessing the site and release of tailings into the environment downstream are also very important considerations.

6 CONCLUSIONS

While the overarching geotechnical and hydrotechnical principles governing sound tailings design also apply in northern climates, there are additional considerations for northern tailings designs. A sound understanding and knowledge of permafrost engineering and the practical management of thermal issues becomes an important point of departure for northern tailings design. Besides the logistical issues associated with remote northern sites, there are usually limited field data for geotechnical and hydrotechnical design. Two or three years of meteorological, thermal regime and hydrotechnical data should collected in early stages of planning.

Frozen dams can take advantage of continuous permafrost and frozen materials but they require thermal analysis, specialized construction and operational constraints to preserve the frozen thermal regime contributing to seepage control and stability. Unfrozen dams built on thawing permafrost must consider the effects of thaw settlement and pore pressures on dam stability.

In addressing the challenges of tailings management in northern climates, it may be possible to kill two birds with one stone: the removal of water storage from tailings facilities will substantially improve the performance of a dam in terms of dam safety, while at the same time make it more straightforward to minimize impact on the thermal and environmental setting.

Key hydrotechnical considerations include on-site reporting of ice conditions; simulating ice processes such as the formation of freeze-up, ice cover evolution, and break-ups as determined by climatic and hydrologic factors; and developing measures to eliminate or decrease ice problems.

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Addressing climate change on mine closure projects

Clint Strachan
Stantec Consulting Services, Inc., Fort Collins, Co, USA

ABSTRACT: "Has climate change been adequately considered in your design?" is a question commonly asked in review of mine closure projects, where the design life is several decades or centuries. The answer for how climate change is considered in the design depends on the application. The data used to estimate the peak discharge through a spillway differs from the data used to estimate long-term infiltration through a soil cover. This paper discusses the factors in using climate data and selecting design storm events for mine closure applications from projects in North and Central America, and how climate change is addressed in the design.

The primary design applications for mine facilities water management are storage, flow, and infiltration or drainage. Each application uses different forms of climate data. Recurrence intervals used to represent extreme storm events are typically extrapolated from existing precipitation records. The relatively short period of detailed climate records and the selection of required recurrence intervals of 1,000 or 10,000 years requires careful extrapolation of existing climate data. In environments with snow accumulation, a rain-on-snow or rapid snowmelt event may require consideration.

These factors illustrate the need for sound engineering judgment in selection of design storm events, as well as how climate change is considered. This judgment includes assessing key factors in the design (flow, storage, infiltration), the period of record and quality of available climate data, and knowledge of extreme periods or events (wet or dry) in the site history.

1 INTRODUCTION

1.1 *Background*

Mining involves movement of material to expose and extract the desired ore, followed by concentration and recovery of the desired products using various water-based processes. This requires management of process water and water affected by mined materials, as well as separation of unaffected water. This management includes features for water storage or containment, diversion or conveyance, and control of meteoric water infiltration.

Design of these features requires some form of climatic input. In many locations, the design requirements are dictated by the governing regulatory agency regulations or by accepted technical guidelines. As described later in this paper, the climatic input for these features is quite often different in a closure scenario than an operational scenario, due to the difference in design life or period of performance.

While design of mine features involves knowledge of site geology and geohydrology, underlying foundation conditions and mine feature material characteristics, along with slope stability and settlement analyses, this paper focuses on surface water aspects, and the corresponding precipitation input.

1.2 *Design approach*

Mine features that convey discharge are designed with a different precipitation input than mine features that store water. This is summarized below.

For features that divert, convey, or discharge water (such as spillways, drainage channels, and reclaimed surfaces) the key design issue is conveying the peak flow or discharge. The feature geometry (cross section and bed slope) and resulting flow velocity are compared with the peak runoff from a particular storm event. The peak runoff is typically from an intense, short-duration storm (such as a 6 or 24-hour duration storm) or a rapid snowmelt or rain-on-snow event.

For ponds or impoundments that store water, the key design issue is providing storage capacity for the volume of water to be contained. The water storage capacity is compared with the total runoff volume from a particular storm event or from multiple events. The total runoff is typically from long-duration storms (such as a 72-hour duration storm), multiple storms (such as a rainy month or quarter) or the total volume of snowmelt runoff. For mine features that manage process water and meteoric water, water balance calculations are used to evaluate storage capacity over multi-year periods and under varying precipitation scenarios.

For reclaimed surfaces where infiltration of meteoric water is of concern, daily precipitation values are typically used with evapotranspiration and other factors. For this infiltration evaluation, wet and dry-year scenarios are used over decades of simulation.

2 DESIGN CRITERIA

2.1 *Design flood guidelines*

Precipitation events and resulting flood flows and volumes used to design conventional water-storage dams are the same as those used for tailings storage facilities and mine water-storage facilities. These flood flows are used to size spillways or evaluate dam freeboard capacity. Table 1 presents the guidelines from the Australian National Committee on Large Dams (ANCOLD) (adapted from ICOLD, 1992).

Table 1. ANCOLD guidelines for calculation of design flood.

Hazard category and description		Safety standard
Low	No loss of life, minor damage	100 to 1,000-year flood
Significant	Unlikely loss of life, significant damage	1,000 to 10,000-year flood
High	Loss of life, extreme damage	10,000-year flood to PMF*

*Probable maximum flood

Table 2 presents the guidelines from the Canadian Dam Association (CDA) (from CDA, 2007, 2013), which include additional hazard categories.

Table 2. CDA guidelines for calculation of design flood.

Hazard category and description		Safety standard
Low	No loss of life, minimal damage	100-year flood
Significant	Unspecified loss of life, minor damage	100 to 1,000-year flood
High	Loss of life $\leq$10, significant damage	1/3 between 1,000-year flood and PMF*
Very high	Loss of life $\leq$100, high damage	2/3 between 1,000-year flood and PMF
Extreme	Loss of life > 100, extreme damage	PMF

*Probable maximum flood

Table 3 presents similar guidelines from the U.S. Army Corps of Engineers (adapted from ICOLD, 1992). These guidelines include dam height and reservoir capacity in addition to hazard category.

Table 3. U.S. Army Corps of Engineers guidelines for calculation of design flood.

Hazard category and description		Dam size*	Safety standard
Low	No loss of life, minimal damage		
		Small	50 to 100-year flood
		Intermediate	100-year flood to ½ PMF
		Large	½ PMF to PMF
Significant	Unlikely loss of life, significant damage		
		Small	100-year flood to ½ PMF
		Intermediate	½ PMF to PMF
		Large	PMF
High	Loss of life, extreme damage		
		Small	½ PMF to PMF
		Intermediate	PMF
		Large	PMF

*Dam sizes:

Small: 7.6 to 12.2 m dam height, 620 to 1,230 m^3 reservoir capacity
Intermediate: 12.2 to 30.5 m dam height, 1,230 to 61,500 m^3 reservoir capacity
Large: $\geq$ 30.5 m dam height, $\geq$ 61,500 m^3 reservoir capacity

These guidelines include flood events ranging from the 50-year recurrence interval flood to the probable maximum flood (PMF). The more significant the consequence of failure is, the higher the safety standard is for design (ICOLD, 1989). The safety standards in these guidelines also incorporate the fact that water-storage dams are expected to be in operation for a long period of time.

For mine features, there may be varying periods of performance, from phases of operation to post-closure. In the United States, design criteria for mine facilities closure are based on the selected design periods and potential consequence of failure. Canadian mine closure requirements are similar to U.S. requirements, with mine closure plan regulations and requirements are administered by each province, but with common closure elements (McKenna et al., 2013).

In Idaho, requirements for closure include providing a stable, maintenance-free condition, with the minimum design inflow being the 100-year recurrence interval event (IDWR, 1993). In Arizona, the prescriptive design storm is the 100-year recurrence interval event, but with latitude (based on potential consequence of failure) to incorporate design storm events from the 25-year recurrence interval to the PMP, depending on potential downstream risk from failure ADEQ (1998).

The design life or operating period aspects are explained below.

2.2 *Precipitation frequencies and design*

For closed mine facilities with minimal active maintenance, the anticipated period of performance is assessed to select the design storm event for channel sizing and surface erosion protection selection, with consideration for the consequence of failure. The effect of period of performance on probability of failure is illustrated in Table 4 below, from probability calculations summarised in Vick (1990). The precipitation events in Table 4 are used to estimate peak discharge or storage values used for design.

Table 4. Probability of precipitation event being exceeded over a specific period of time.

Time period (years)	Precipitation event return period (years) 10	100	500	1,000	10,000
1	10*	1	0.2	0.1	0.01
5	41	5	1.0	0.5	0.05
10	65	10	2	1	0.1
20	88	18	4	2	0.2
50	99	39	10	5	0.5
100	100	63	18	10	1.0
1,000	100	100	86	63	10

*Probability values are in percent

For the 100-year recurrence interval event, the probability of being exceeded in one year is one percent. For the same recurrence interval, the probability of being exceeded over 10 years is 10 percent, over 50 years is 39 percent, and over 100 years is 63 percent. For example, a diversion channel designed for a 100-year recurrence interval storm has a 10 percent probability of being overtopped in 10 years and a 63 percent probability (greater than a 50/50 chance) of being overtopped in 100 years. If the channel is designed for the 1,000-year recurrence interval, the probability of being overtopped in 100 years is reduced to 10 percent. Reclaimed mine facilities with drainage channels or soil covers that will remain in place for decades (with minimal maintenance) may require design for more extreme storm events.

For a low-hazard category structure, the 100-year recurrence interval event may be an appropriate design event for an operating period of five to ten years. However, use of 100-year recurrence interval event for the same low-hazard category structure may not be an appropriate design event for a reclaimed or post-closure application, where the operating period is several decades.

3 PRECIPITATION ESTIMATES USED IN DESIGN

3.1 *Statistical calculation from precipitation records*

Recurrence interval storm events are estimated statistically from climate records. The applicability of a 100-year recurrence interval event used for design at a mine site depends on the proximity of climate recording stations to the site and the period of recorded data at the recording stations. Often a mine may have precipitation records at the site, but for a limited period of time. A nearby city may have precipitation records for many decades, but at some distance from the mine site. Judgment in selection of the appropriate precipitation record is required to estimate representative recurrence interval events for design. Extreme storms such as 500 or 1,000-year recurrence interval events require more careful estimation, since they are statistically extrapolated from available precipitation records that may be over only several decades.

3.2 *Precipitation estimates from meteorological data*

The probable maximum precipitation (PMP) for a specific location is calculated from meteorological factors (as outlined in WMO, 2009). The calculation is based on the maximum amount of moisture in the atmosphere at a specific location (moisture availability), and the amount of this moisture that can be converted to precipitation (storm efficiency) over a defined period of time (typically 6 or 24 hours). Due to varying types of storms (thunderstorms, tropical storms, and systematic storms), PMP totals vary with latitude and watershed area. For locations in the United States, PMP estimates are shown in graphical form from regional reports (for example NOAA, 1977, 1980, 1982), based on location, drainage basin size, and time of year. For other parts of the world, procedures for estimating the PMP outlined in WMO (2009) are used.

Because the PMP estimate is not based on the variability of available precipitation records, the PMP or fractions of the PMP are often used for extreme storm event estimates (as shown in Tables 2 and 3).

3.3 *PMP and PMF*

The precipitation depth and intensity from the PMP is used to estimate the resulting runoff rate or peak discharge and total runoff volume for the PMF. The PMF is estimated from hydrologic calculations involving the PMP and basin topography and surface conditions. Where both flow and precipitation data for a particular flood event are available, correlation between the PMP and PMF is possible.

3.4 *Infiltration and water balance modeling*

Evaluation of water-storage capacity includes the total runoff from extreme precipitation events (such as a 24-hour duration storm) as well as runoff from precipitation over a longer period of time. For tailings storage facilities and other water-storage features, this is commonly evaluated as a water-balance calculation, where other factors (evaporation, process-water input, water recycling, and pore-water entrainment) are included. The simulation period for the water balance may be over several years or the entire life of the mine. Precipitation data is typically entered on a monthly or daily basis. Where climate data is available, actual daily precipitation is used, and selected to represent average-year, wet-year or dry-year scenarios.

Evaluating the infiltration through a cover system is similar to the water balance calculation, but involving only the key the factors affecting infiltration through the cover: precipitation, runoff, evapotranspiration, and water retention in the cover. Due to the time necessary for parameters to equilibrate or respond to changes in input, the simulation period may be over years to decades. Actual daily precipitation is the typical input, selected to represent average-year, or multiple wet-year or dry-year scenarios. This precipitation record selection requires considerable engineering judgment to represent conservative but realistic long-term conditions.

4 ADDRESSING CLIMATE CHANGE

Design of facilities for water discharge, water storage, or control of infiltration are based on accepted methods, consequence of failure, and assessment of risk, as well as precipitation input. For facilities with a high hazard or consequence of failure, precipitation input tends toward extreme events, up to the PMP.

For facilities that require acceptable performance for a long period of time (such as in a post-closure scenario), precipitation input also tends toward extreme events, up to the PMP. For example, uranium tailings facilities in the United States have a performance standard of no tailings exposure with minimal maintenance for a period of 200 to 1,000 years (USNRC, 2005). Tailings storage facilities in Western Australia have a similar performance standard over a period of 100 to 300 years. For these scenarios, the PMP is the design storm for evaluation of performance.

This long post-closure period of performance has prompted questions about consideration of potential effects from climate change. Fossil records, evidence of previous floods, geomorphologic features, and glacier ice sampling demonstrate significant variations in climate and biology in the past. Studies of more recent history have indicated variations in temperatures and precipitation since the last major period of Holocene glaciation (CSIRO and BoM, 2016, Montgomery, 2012, WMO, 2009).

Predictions of future climate variations include higher temperatures, regions of higher precipitation, other regions of lower precipitation, and more intense precipitation (CSIRO and BoM, 2016, WMO, 2009). These potential changes coupled with actual climate data with relatively short periods of record indicate that adjustment of precipitation input is appropropriate. This would apply to recurrence intervals extrapolated from climate data (such as the 50 and 100-year recurrence interval storms) and daily precipitation records used for water balance and infiltration modeling.

Adjustment of the PMP due to climate change may not be warranted (Jakob et al., 2008, WMO, 2009). Due to the general meteorological factors used in estimating the PMP, warming temperatures may affect the moisture availability component of the PMP calculation, but not significantly.

5 CONCLUSIONS

The primary applications for mine facilities water management are design for storage, flow, and infiltration. Each application uses different forms of climate data, ranging from short-period, intense precipitation to months or years of daily precipitation. Recurrence intervals used to represent extreme storm events are typically extrapolated from existing precipitation records. The relatively short period of detailed climate records requires careful extrapolation of existing climate data.

For high-hazard category structures or mine features with a long period of performance, the PMP is the common precipitation input parameter. The PMP is not extrapolated from climate data, but is calculated from regional meteorological factors of moisture availability and storm efficiency.

The long post-closure period of performance for mine facilities has prompted questions about the potential effects from climate change, specifically warmer temperatures, more intense precipitation, and regions of either higher or lower precipitation. For storm event selection based on actual climate data with relatively short periods of record, adjustment for precipitation input for future climate change may be appropriate. This would apply to recurrence intervals extrapolated from climate data and daily precipitation records used for water balance and infiltration modeling. For PMP values, the potential effects of climate change may not be significant enough for adjustment, due to the general meteorological factors used in the calculation.

Regardless of considering climate change, sound engineering judgment is necessary in selection of appropriate design storm events for evaluation mine facilities for water storage, discharge, or infiltration.

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Economic evaluation for the disposal of slurry versus thickened tailings in Western Australia – A case study

A. Carneiro & A. Fourie
The University of Western Australia, Perth, WA, Australia

ABSTRACT: Tailings management challenges, particularly arising from water scarcity and tightening regulations, are key driving forces behind operational changes taking place in mining. As tailings storage facilities (TSFs) come under increasing scrutiny, operators are looking at alternative management strategies. High density tailings may be the best available option in many cases, but is commonly deemed undesirable because of economics, a conclusion that sometimes results from short-term profit-based valuations. This paper reports on an economic evaluation completed for comparing the life-cycle costs of disposing tailings as slurry versus thickened tailings in Western Australia. TSF designs were developed and costed. Key elements driving the costs of disposal are highlighted. The concepts of the West Australian Mining Rehabilitation Fund are introduced and considered in the estimates. Overall, comparative costs show that the thickened tailings option would be selected for this specific case. Although showing slight difference in the Net Present Cost, comparative capital, operating and closure costs outline significant dissimilarities between the alternatives. The paper concludes by discussing the importance of considering environmental, social, risk, and economic costs in the evaluation of tailings disposal methods for selecting the most cost-effective option.

1 INTRODUCTION

The business case for applying sustainable development principles and leading practices in tailings management is compelling. Increasing regulatory and social demands are driving forces behind operational changes taking place within the mining industry. As tailings storage facilities (TSFs) come under increasing environmental scrutiny, more operations are looking at the benefits of alternative tailings management practices. Integrated tailings management, including technical, economic, environmental, social, and risk aspects of the operation has been shown to be a potential solution to satisfy the escalating demands facing the mining community.

Currently, evaluation methodologies used to decide on the preferred tailings management option are limited, and have usually poorly addressed the issue of realistic financial provisioning for the responsible disposal of tailings. Though several studies cover comparative economic evaluation of tailings disposal methods, review of publically available feasibility projects and conference papers has revealed that they mainly account only for capital and operational expenditures, and only for certain items that are specific to a project. There is a recognized lack of economic data showing that it is economically effective to manage a TSF considering all the potential costs over the entire life-cycle of the facility.

Generally, the arguments in favour of the selected tailings storage option have been dominated by achieving operational simplicity, and the goal of financial performance, by lowering capital and operating costs. However, the use of current methodologies may result in underestimating the closure and rehabilitation costs, and overlooking non-technical issues that may pose significant overheads to the project. Hence, the currently available approaches used for the evaluation of tailings disposal alternatives rely on a narrow set of parameters for cost estimation, and may no longer point to the most cost-effective option.

In many cases high density tailings may be the best available option. However, using thickened tailings is commonly deemed undesirable because of economics, a conclusion that sometimes results from short-term profit-based valuations. Low-capital alternatives may in fact not have significant, or any, cost advantage at all if associated potential environmental, social and risk costs are not recognised. These factors are difficult to incorporate in cost estimates, but may impose significant expenses and impediments to a project.

This paper reports on a case study for the disposal of a typical non-acid generating gold tailings stream in the Goldfields of Western Australia (WA). An economic evaluation was completed, comparing the life-cycle costs of disposing the tailings as conventional slurry versus thickened tailings. A conceptual TSF design with associated equipment configuration and required infrastructure was developed for each option. Capital, operating, and closure cost estimates were prepared based on the designs. Key elements driving the costs of disposal are highlighted and discussed. The concepts of the recently implemented Mining Rehabilitation Fund (MRF) in WA are introduced and considered in the estimates. The paper concludes with a discussion about the importance of considering the totality of environmental, social, risk, and economic costs in the evaluation of tailings disposal methods for selecting the most cost-effective option.

2 REVIEW OF INCREASING TAILINGS MANAGEMENT CHALLENGES

The challenges to the management of tailings are ever increasing and include the need to balance economic, environmental and social issues. In the future there are likely to be rising financial and operational threats posed by water scarcity, and tightening regulations resulting from the socio-environmental consequences of ongoing catastrophic TSF failures.

Mining can use large quantities of water. Coupled with increasing water costs, potential unavailability of water has been identified by mining companies as one of the major challenges in the future. Given the longer-term trend of falling ore grades and higher volumes of ore production, even more water is likely to be required for mineral processing.

Many mines are located in water stressed regions that are particularly prone to water shortages, such as in WA and Chile. In the Chilean arid north region, mining operators are acutely aware of water constraints. The problem of water scarcity has driven some miners to using expensive desalination processes to treat seawater. Data published by Chile’s Mining Council reported on realized costs of removing all the salt from seawater in Chile to be around US$5 per m^3 (Jamasmie, 2013).

Challenges arising from dwindling water resources are not just related to the financial burden on mine operators. Conflicts emanating from social and environmental disputes over water resources in mining areas create pressures on government to tighten the policies for water usage. As a consequence, strict regulations may impose onerous obstacles to permitting processes, and even make projects unfeasible due to difficulties obtaining a social license to operate.

The potential (and actual) failure of TSFs is another reason why pressures are being placed on tailings management strategies. The failure or poor performance of TSFs can have dramatically negative impacts on the corporate bottom line and reputation, sometimes even being impossible to remediate.

The mining industry has experienced several significant dam failures in recent history. Lately, reported significant accidents involving tailings facilities occurred in Israel, China, Brazil, Australia, and Canada. In Israel, it occurred in June 2017. Part of a 60 m high wall of a reservoir at a phosphate factory collapsed releasing acidic wastewater in a riverbed (Petley, 2017). In China, it occurred in Luoyang, Henan Province, in August 2016. Red mud generated at an alumina refinery rushed down flooding 2 villages home of over 300 people (Petley, 2016). In Brazil, it occurred in November 2015, when the Fundão dam operated by the mining company Samarco breached. Approximately 32M m^3 of tailings were released completely destroying the downstream village of Bento Rodrigues, home of around 600 people. Tailings flowed on down rivers to the sea more than 600 km away. In Australia, it occurred in July 2015, when a wastewater dam collapsed releasing fine coal particles into the Blue Mountains National Park near Lithgow

(Brown, 2017). In Canada it happened in August 2014, when the Mount Polley dam failed releasing approximately 25M m^3 of mine tailings in a source of drinking water, being considered the largest mining disaster in Canadian history (Hoekstra, 2016).

Research conducted by Bowker & Chambers (2015) projected 11 “Very Serious” failures for the decade of 2010-2020, at a total cost of $6 billion. However, data published by the World Information Service on Energy, in which 90 major TSF failures have been reported over the last 50 years, shows that failures have been occurring at a frequency of around 2 per year. Moreover, only rehabilitation and repair costs to cover the damage caused by the Samarco’s dam failure in Brazil are of the order of $US2.3 billion (News.com.au website, 2016).

As a consequence of recent, well-publicized tailings dam failures, TSFs have come under increasing environmental scrutiny. In addition to the overwhelming economic loss incurred from a TSF failure, far more reaching are its effects on driving public perception, increasing regulatory burden and government oversight. For instance, following the Samarco dam failure, considered Brazil's worst environmental disaster, mining became the subject of intense scrutiny. In Brazil, a bill (PL 3676-2016) is being considered by the government to ban construction of dams using the upstream method. For the state of Minas Gerais, where around 900 of the 1,200 dams existing in Brazil are located, this measure represents serious material risk to the feasibility of many mining projects.

In Canada, the government of British Columbia introduced regulatory changes in response to the failure of the Mount Polley tailings dam. Among the changes is the requirement for new mines to provide an alternate assessment of best available technology for tailings disposal in their provincial applications. It follows the recommendation of a government-appointed engineering panel that called for a “move away” from storing mine tailings under water and behind earth and rock dams (Vancouver Sun website, 2016).

Increasing regulations related to mine closure are also a reflection of the extent to which public perception and economic loss resulting from poor management of TSFs have on changes to governance requirements and practices. In Western Australia, in response to the potential increase in unfunded rehabilitation liability for mines that have been abandoned, the government recently implemented the Mining Rehabilitation Fund (MRF). Introduced in July 2013, it is a pooled fund to which mining operators contribute. Participation in the MRF is compulsory. It was established to replace the system used worldwide of applying Unconditional Performance Bonds (UPBs) against tenements as security for compliance with environmental obligations. The MRF requires mining companies to make annual contributions to the fund, calculated on the amount and type of area disturbed as a result of mining activities. The introduction of the MRF does not absolve operators of their legal obligations to carry out rehabilitation works. Instead, it is seen as an incentive to mining companies for undertaking progressive rehabilitation, as the sooner a company fulfils its environmental obligations, the lower is its annual levy payable to the fund.

In summary, meeting the increasing challenges to the management of tailings may be achieved by enduring value, encompassing sustainable development principles, and applying leading practices to the evaluation of possible alternative methods for surface tailings disposal. The continuous growth of the mining industry while maintaining credibility before the public depends on having strong commitment to the development of technology to improve water efficiency and lower the risks of catastrophic TSF failures. Evaluation of tailings disposal options need to look beyond short-term expenses, considering the totality of environmental, social and economic costs over the long-term.

3 COST COMPARISON OF CONVENTIONAL SLURRY TAILINGS VERSUS THICKENED TAILINGS DISPOSAL – A CASE STUDY

3.1 *Tailings physical characteristics and operating parameters definition*

The first step undertaken to conduct this comparative study was to research information about gold mine operations in WA. Publicly available data supported the adoption of reasonable physical characteristics for a hypothetical tailings stream. Operating data was selected with the purpose of making this study easily transferable to other similar operations. Tailings are assumed to be from gold ore, processed in a mineral processing plant comprising crushing, grading, flotation, thickening, and carbon-in-leach circuits. Among other relevant characteristics that affect sizing of thickeners, pumps and pipelines, particle size distribution was assumed as 75-80% less than 75 µm, and specific gravity of tailings solids as 2.79.

The proposed location for the TSF is in a semi-arid region, characterized by hot dry summers and mild winters with a mean annual rainfall of 266 mm and average annual evaporation of 2,500 mm. Thus, under these semi-arid conditions characterized by relatively low rainfall and high evaporation, the site water balance is negative. The selected site for building the TSF is assumed to be at flat topography 1 km away from the thickener. Make-up water required for mineral processing will be sourced from a catchment point located 5 km from the plant. Operating parameters are based on an annual tailings production rate of 2M dry tonnes, resulting in a total of 30M dry tonnes of tailings solids to be disposed of over the 15 years operating LOM. For the purpose of this study, it is considered that the plant will not vary the volume of tailings produced per year, and that the physical and geochemical characteristics of the tailings will remain unchanged over the LOM.

3.2 *Tailings storage facilities design considerations*

Design considerations were determined based on intensive literature review, discussions with experienced geotechnical engineers, and a fieldtrip to a gold mining operation in Kalgoorlie. Access to reports detailing final design and construction considerations of existing TSFs in WA ultimately supported selection of design parameters. Conceptual TSF designs were developed considering assumed operational details and storage requirements for the project. The designs considered two surface disposal methods, differentiable by the degree of dewatering applied to the tailings slurry at the processing plant, before transport and disposal. The two options considered were conventional slurry at a solids content of 55% by mass, and thickened tailings at a solids content of 65% by mass. Based on assumptions and selected design criteria, the TSF geometry along with associated infrastructure and equipment configuration for each alternative was determined.

3.2.1 *TSF design for conventional slurry tailings*

The TSF designed for the containment of the conventional slurry tailings is a paddock-type dam comprised of 1 square cell covering a footprint of 94 ha. Tailings will be discharged by spigotting from a ring dyke. The total design storage is 21.5M m^3 based on an assumed tailings dry density of 1.4 t/m^3.

The slurry will be pumped to the TSF by one of 2 pump trains, arranged in a duty/standby configuration, with each pump train consisting of 5 centrifugal pumps in series. Tailings will be transported via a 200 mm diameter HDPE pipeline and deposited sub-aerially from a slurry ring, located on the perimeter wall. A series of discharge points along the embankment will be used to dispose the slurry into conductor pipes to deliver tailings to the beach level.

Major infrastructure components include a 6 m high earthfill starter embankment constructed with outer slopes at 1V:3H (18°) to facilitate contour ripping and the establishment of vegetation on the side slopes. Proposed infrastructure designed for seepage control includes the construction of an underdrainage system consisting of collector drains and an underdrainage tower. For water management, the design considered the construction of a decant system to remove supernatant as well as storm water from the surface of the facility. The decant structure comprises an 8 m wide decant road that gives access to the centre of the TSF where underdrainage and decant towers are to be located. The suggested decant tower is constructed of vertically-stacked, 1.8 m diameter slotted reinforced concrete pipe sections, while the underdrainage tower is built of 0.9 m diameter solid concrete pipe sections. Designs consider both structures resting on reinforced concrete slabs, and surrounded by an external zone of rockfill to retard the inflow of sediment into the towers. Decant water will be returned to the mill for re-use in the process via a 160 mm diameter HDPE pipeline. Make-up water needed for mineral processing will be pumped from a natural source via a 160 mm diameter HDPE pipeline. Designs also considered the construction of a TSF access ramp, access road from the plant to the TSF, pipeline corridors, and a topsoil stockpile.

During the operating life of the facility, the design assumed that TSF capacity will be increased by raising the embankment using the upstream method of construction, at a rate of 2 m per year. The coarser fraction of the tailings sourced from the adjacent tailings beach will be used as construction material. The decant road will also be raised at a rate of 2 m per year using harvested tailings, but following the concepts of the centreline method of construction.

The life cycle of this proposed design comprises the acquisition of equipment at year 0, followed by 2 years of construction, and 15 years of mining operation. At year 17, the dam will be raised to the final proposed height of 30 m. After 3 years drying, at year 21 it is assumed that the surface of the TSF will be trafficable for construction equipment to perform rehabilitation work. It consists of hauling and placing hard rock for filling the gap at the top of the TSF, spreading topsoil to cap the facility, and deep ripping and spreading of seeds for the revegetation of disturbed areas. After an assumed 3 years for regulator approval of rehabilitation strategies, another 5 years of post-closure environmental monitoring to meet closure criteria and completion of the project's LOM were assumed. The assumption is thus that it will take 29 years from the start of the project to final relinquishment.

3.2.2 *TSF design for thickened tailings*

The TSF designed for the containment of the thickened tailings is a paddock-type facility with circular shape covering a footprint of 324 ha. Tailings will be discharged into the TSF using the Central Thickened Discharge (CTD) method forming a self-supporting cone of 1,000 m radius. Based on an assumed average tailings beach angle of 2%, in its final profile the cone will be 20 m high. Tailings deposition will occur by radial discharge from 8 spigots located within the central area of the facility. The total design storage is 20M m^3 based on an assumed tailings dry density of 1.5 $t/m^{3.}$

Thickening to about 65% solids by mass will take place in a high-compression thickener at the processing plant. The tailings will be pumped to the TSF by a piston-diaphragm positive displacement pump coupled to 2 charge pumps arranged in a duty/standby configuration. Tailings will be transported in a 200 mm diameter HDPE pipeline and deposited sub-aerially from the spigots connected to the end of the tailings distribution pipeline.

Major infrastructure components include a 2 m high earthfill environmental perimeter embankment, and an 8 m high x 8 m wide x 920 m long spine access way to the discharge point, both constructed with outer slopes at 1V:3H (18°). As thickened tailings can be considered as a dense, non-segregating slurry, the TSF was designed as a passive system with regards to seepage control. Thus, no underdrainage system was designed to intercept seepage or consolidation water. Moreover, as the tailings are planned to be discharged in thin layers, evaporation is expected to occur fast enough to prevent bleed water. Proposed infrastructure designed for storm water management includes the construction of a lined pond designed to accommodate a total freeboard of 500 mm above the normal operating surface water level plus a 1 in 100 year, 72-hour rainfall event, considered as 174 mm for the site in question, plus the total volume of collected runoff during the 3 wettest months of the region. The calculated volume of precipitation runoff collected in the pond will be returned to the mill for re-use via a 200 mm diameter HDPE pipeline for approximately 2 months per year. Make-up water needed for mineral processing for the remaining months will be pumped from a natural source via a 200 mm diameter HDPE pipeline. Designs also considered the construction of a TSF access ramp, access road from the plant to the TSF, pipeline corridors, and a topsoil stockpile.

The TSF capacity will be increased by raising the spine access way, and consequently the tailings deposition point, using the centreline method of construction. Tailings sourced from the adjacent tailings beach will be used as construction material. As thickened tailings are assumed to be deposited by radial discharge forming a self-supporting cone of gentle slope angle, the frequency of raising the deposition point does not follow a constant rate. In the first years of operation, the height of the tailings cone will increase rapidly, reducing the rate of rise as the layer of tailings are expected to cover larger surface areas in the following years. Thus, the raising of the spine access way is planned to follow the rate of rise of the cone. Raises of 2 m are intended, but not at regular time intervals.

The life cycle of this proposed design comprises the acquisition of equipment at year 0, followed by 2 years of construction, and 15 years of mining operation. At year 14, the spine access way, and thus the deposition point, will be raised to their final height. Mine closure and rehabilitation activities will take place in year 18, mainly consisting of hauling and placing topsoil to cap the TSF, hauling and placing hard rock to fill the pond, and deep ripping and spreading of seeds for the revegetation of disturbed areas. This will be followed by 1 year for regulator approval, and a further 5 years of post-closure environmental monitoring to meet closure criteria and completion of the project's LOM. The assumption is thus that it will take 24 years from the start of the project to final relinquishment.

3.3 *The cost estimates*

The database of costs used for evaluating the two tailings disposal strategies has been developed mainly based on obtained third-party contract rates. It reflects current market pricing in WA. The Net Present Cost (NPC) analysis for this study assumed a discounted cash flow (10%) over the LOM. For comparison purposes, the cost estimates considered the main associated capital, operating, and closure costs to reflect the difference between the alternatives. They are not representative of all the costs to produce and store the tailings.

The capital expenditures (CAPEX) were estimated based on fixed, one-time expenses to be incurred on the purchase and installation of equipment, and on the construction of infrastructure required prior to the commencement of operations. Earthworks for both options comprise site preparation (e.g. topsoil stripping from TSF footprint area), and the construction of the perimeter embankment. Earthworks differ for building the underdrainage and decant systems for the slurry option, and for building the tailings deposition system (e.g. spine access way), and a collector pond for the thickened tailings option.

The operational expenditures (OPEX) have been estimated based on annual costs for increasing the TSF capacity, pumping tailings, pumping return and make-up water, and for paying the MRF levy. As they were assumed to be similar, annual maintenance, labour, and general and administrative costs were not accounted for in the estimates.

As mentioned before, mining companies in WA are required to make annual contributions to the MRF. The amount payable is based on the type of disturbance, in accordance with the "Rehabilitation Liability Categories". Each disturbance type is related to the description of infrastructure or land, and corresponds to a category with an associated unit cost rate as specified in the MRF regulations. The area in hectares covered by each disturbance type is then used to calculate the Rehabilitation Liability Estimate (RLE). The RLE is calculated by multiplying the size of each disturbance type by the associated unit rate. For instance, an area disturbed by a TSF class 1 (characterized by its highest embankment being at least 5 m high), falls into category "A", with an associated unit rate of $50,000 per hectare. The amount of levy payable is assessed as the RLE multiplied by the current Fund Contribution Rate (FCR) of 1%.

The closure and rehabilitation expenditures were estimated based on one-time expenses to be incurred on rehabilitation works, and on annual costs of post-closure environmental monitoring. General management costs, and costs associated with demobilization of plant and equipment were assumed to be similar for both options, and were not accounted for in the estimates.

A literature review enabled development of the costing database. Despite the lack of publicly available data on tailings costs, some information was easily accessed. The *Cost Estimation Handbook* (AusIMM, 2013) offers cost indicators for various unit rates. In addition, ready data on TSF closure costs can be found in the *Rehabilitation Cost Estimation Tool* (RCE tool) (NSW Government website), and the *Mining Financial Assurance Calculator* (Queensland Government website).

3.4 *Assumptions*

The cost estimates focused on the physical structure required to store the tailings, and did not consider stability issues. Regarding relevant inclusion and exclusion of costs, as the Goldfields region's groundwater is hypersaline and of little economic value, the cost of make-up water is just the operating cost of pumping it from a natural source to the plant. Moreover, supply of construction material was not accounted for in the estimates. Although different volumes would be needed for each TSF, what ultimately affects the costs of each option, it is assumed that sufficient quantity will be available on site. In addition, in spite of the recognised importance of seepage interception systems for preventing the naturally saline groundwater rising into the potential root zone of vegetation, closure cost estimates did not consider the construction of dewatering bores and trenches for managing groundwater levels after TSF rehabilitation. Not all unit rates could be determined. For example, the unit rate for using tailings to annually increase TSF capacity was not obtained. However, the cost to win, load, haul, spread, moisture condition, and compact low permeability fill for constructing the embankment was known. The same rate for construction using tailings was thus used. Although the cost of harvesting tailings from the beach and compacting it in the walls may be cheaper because of the shorter distance for hauling, it was assumed this rate is inclusive of the ongoing relocation of the piping required to accommodate the raised embankment.

4 RESULTS AND DISCUSSIONS

4.1 *Net Present Cost Comparison*

The results of this work are based on the completion of NPC analyses for the disposal of a typical gold tailings stream in WA. Comparative evaluations are based on the costs of disposing the tailings as conventional slurry versus thickened tailings. For each option, a conceptual TSF design with associated equipment configuration and required infrastructure was developed. Capital, operating, and closure cost estimates were prepared for each alternative. A chart comparing the results of the total NPC, CAPEX, OPEX and closure costs is shown in Figure 1.

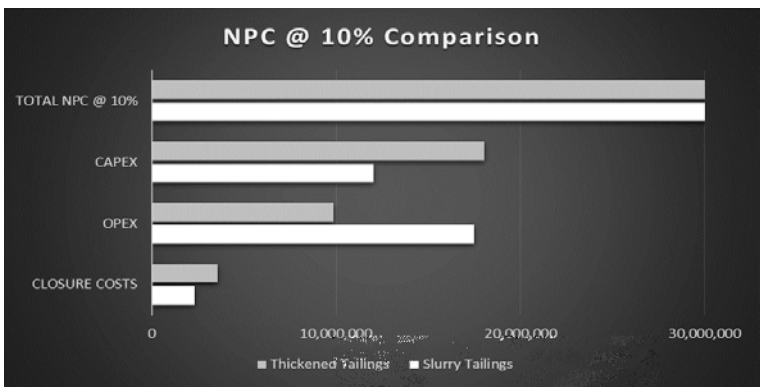

Figure 1. Comparative NPC, CAPEX, OPEX and closure costs.

Overall, the estimated total NPC of both options is the same, being only 1% higher for the slurry option. Comparison of CAPEX shows higher costs for the thickened tailings system, while comparison of OPEX shows higher costs for the slurry option. Capital investment for the CTD option is 50% higher than the required investment for the slurry alternative. However, the total operating cost of the slurry option is 78% higher than the total operating cost of the thickened tailings option. On the other hand, closure cost estimates show that rehabilitating the thickened tailings facility, although involving simpler TSF rehabilitation works, costs 62% more than rehabilitating the TSF for the slurry option.

4.2 *Comparative evaluation of capital costs*

The comparison of the total capital cost required for each alternative indicates that, due to the differences in dewatering technology, tailings pumping system, and disposal method, the initial investment is higher for the thickened tailings option. A graph showing the cost comparison of the items costed in the CAPEX estimates can be seen in Figure 2. As reported in the literature, the up-front capital associated with dewatering technologies to produce high density tailings is higher. For this specific case, the cost of supply and installation of the high-compression thickener was estimated to be $1.4M, compared to $0.8M for the conventional thickener, representing an additional cost of 75%.

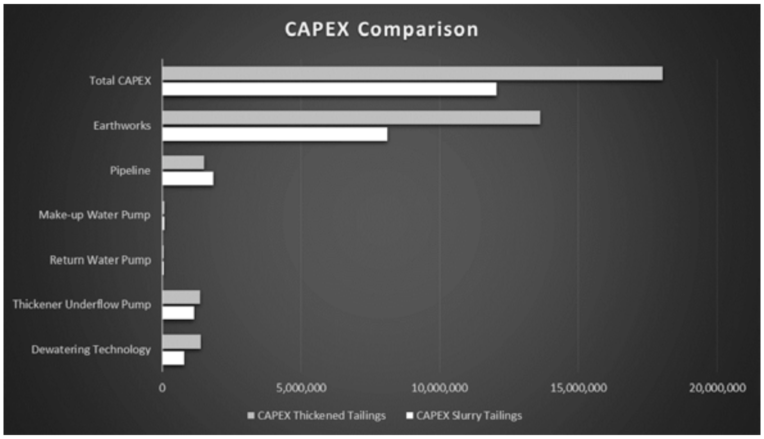

Figure 2. Comparative capital costs.

Comparative CAPEX for the supply and installation of thickener underflow pumps shows that the NPC of the thickened tailings option is 18% higher than the NPC of the slurry option. The cost of supply and installation of one PD pump was estimated as 18% higher compared to the supply and installation of 10 centrifugal pumps. Even considering installation cost of centrifugal pumps as 100% of the cost of supplying them, compared to 50% considered for installing PD pumps, CAPEX of PD pumps is significantly higher. However, the short life span anticipated for the centrifugal pumps, requiring spares worth 1/3 of the pump supply cost per year, escalates the operating cost of the slurry option throughout the LOM.

The capital cost of supply and installation of pumps for return and make-up water is the same for both options. According to quotations obtained from third parties, supply costs of centrifugal water pumps do not vary significantly with relatively small differences in pumping capacity. However, it should be noted that this does not reflect the actual savings on water costs if more dewatering is applied to the tailings at the plant. As the slurry tailings have lower solids content, the volume of water discharged with it is higher. Even considering the return of decant water, some is expected to be retained in the tailings, or lost through seepage and evaporation. Thus, it was expected higher required make-up water and consequently higher CAPEX with pumps for the slurry option.

CAPEX for supplying and installing pipelines is higher for the slurry option.. The investment in pipeline for the slurry tailings was estimated to be 22% higher than for the thickened tailings option. This difference is due to the extension of the slurry ring along the perimeter embankment needed for tailings discharge. Moreover, slurry deposition also require a significant number of spigots and conductor pipes to deliver tailings to the beach level.

On the other hand, comparative evaluation of the NPC of earthworks for building the TSFs shows that the costs of the CTD option is 68% higher than the costs of the slurry option. Identified key elements driving the costs of earthworks are the footprint area and the construction of embankments for tailings containment. The area to be occupied by the CTD alternative is more than 3 times larger than the area required for the slurry option. Thus the NPC of site preparation for the CTD option, mainly consisting of topsoil stripping, was estimated to be 3 times the NPC of the slurry alternative. The savings in topsoil stripping realized by the slurry option is somewhat offset by the large volume of construction material needed for building the starter embankment. During the construction phase, the dam volume required for the slurry option was estimated to be approximately 600,000 m^3, whereas the estimated total dam volume required for the thickened tailings option decreases to 127,000 m^3. It represents a required capital cost for embankment construction for containing slurry tailings more than 3 times higher.

Also driven by the large footprint area required for the CTD option, earthwork costs associated with the construction of a pond next to the TSF offset the savings realized with dam construction. Thickened tailings are considered to be non-segregating and to produce minimal bleed water. Hence, underdrainage and a decant system was not considered necessary. However, due to the vast surface area of the tailings cone, capacity for significant storm water management needs to be provided. A lined pond was designed and costed for the CTD option. Comparative capital costs shows that the costs of constructing a pond for the thickened tailings option is more than 7 times the costs of constructing underdrainage and decant systems for the slurry solution.

The capital costs of the CTD option also include the construction of an access spine way to the tailings discharge point to be located in the centre of the facility. This cost represents 18% of total CAPEX associated to earthworks required for the CTD option, and is not required for the slurry option.

4.3 *Comparative evaluation of operational costs*

Overall, comparative evaluation of OPEX shows that operating the TSF for containing slurry is 78% more expensive than operating the TSF for containing the thickened tailings. A graph showing the cost comparison of the items costed in the OPEX estimates can be seen in Figure 3. High OPEX for the slurry option is primarily due to the complex work of regularly raising the perimeter embankment, and pumping tailings using 5 centrifugal pumps in series. It should be noted that OPEX incurred on pumping tailings and water include the cost of supplying pump spares. It was considered that centrifugal pumps require spares worth 1/3 of the pump cost per year, whereas PD pumps require spares worth 3% of the pump cost per year.

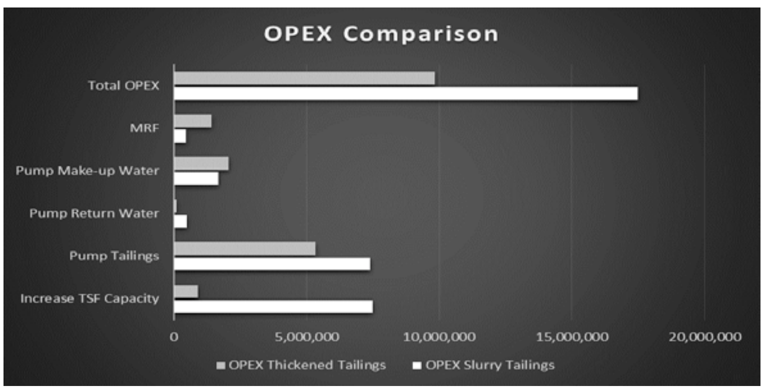

Figure 3. Comparative operating costs.

For this specific case study where the TSF is to be built in a flat terrain, operating costs of increasing TSF capacity of the slurry option is significantly higher. It was estimated that the NPC for raising the dam throughout the LOM is more than 8 times the NPC for raising the tailings deposition point for the CTD option. The CTD option was designed considering that the thickened tailings will form a self-supporting cone requiring no containment wall. It was assumed the construction of an environmental embankment for water management purposes only. Conversely, the TSF for the slurry option was designed for containing wet tailings that require a perimeter embankment to retain slurry and decant water.

Although representing around 50% of the operating costs considered for both options, costs of pumping slurry is 38% higher than the costs of pumping thickened tailings. This difference in costs is given by the difference in pumping systems adopted for each option. At lower solids content, slight variations in density do not create a big difference in rheology, hence centrifugal pumps were considered for the slurry tailings. As solids content increases, the rheology becomes more sensitive to smaller variations in density, an effect that may be an issue for centrifugal pumps. PD pumps were therefore selected for the thickened tailings. In spite of having similar discharge pressures, the power consumption of the 5 centrifugal pumps in series is higher than the power consumption of a single PD pump, manifesting as higher OPEX for the slurry option. The costs of spares of centrifugal pumps, considered annually in this case study, are also higher as they have shorter operating life.

Comparative evaluation of OPEX for pumping return water shows that the cost of the slurry option is more than 4 times the cost of the CTD option. For this specific case study and based on existing TSFs in the Goldfields, it is considered that 55% of water discharged with the slurry tailings will be collected and returned to the mill for re-use. For the CTD alternative, bleed water is not expected as thickened tailings are assumed to be a dense, non-segregating slurry to be discharged in an arid region. However, the large surface area of the thickened tailings cone enables the collection of a high volume of runoff to be pumped back to the plant as return water. Annual operating costs associated with return water for the CTD option is lower because as the site water balance is negative, return water will be possible just 2 months per year.

Comparative evaluation of OPEX for pumping make-up water shows that the cost of the thickened tailings option is 24% higher than the cost of the CTD option. However, as mentioned before, this comparison does not reflect the actual difference in water savings if more dewatering is applied to the tailings at the plant. In this study, a relatively high percentage of decant water for the slurry alternative is considered to be recovered and pumped back to the mill. It should be noted that considering lower recovery of return water, what is realistic for the assumed site location, the volume of water discharged with the slurry is higher, as is the requirement and associated costs of make-up water for the slurry option.

In this specific case study, the annual cost of pumping make-up water represents 10% and 21% of total estimated OPEX for the slurry and thickened tailings options, respectively. If mining operators would in future be required to pay for the use of groundwater this cost element would significantly impact the total operating costs. A benefit-cost analysis for a case where water costs $\$3/m^3$ resulted in the costs associated with make-up water representing 50% of total OPEX for the slurry option, and 71% for the thickened tailings option. NPC of supplying and pumping make-up water for the thickened tailings option was estimated to be of the order of \$19M, compared to \$15.5M for the slurry option.

A comparative evaluation of the levy payable to the MRF shows that the NPC of the CTD option is more than 3 times the NPC of the slurry option. This difference is due to the large footprint area required for the CTD option, the greater area occupied by the overburden stockpile, and the area disturbed by the pond that is classified as category "A" with the associated unit rate being the highest one.

A comparison of the rehabilitation costs used in the MRF regulations and the rehabilitation costs resulting from this specific case study was carried out. For calculating the levy payable to the MRF, the unit rate for rehabilitating the area disturbed by TSF class 1 is \$50,000/ha. Allowing for project management and surveying, contingency, and a monitoring period of 10 years, the estimated rehabilitation costs for the TSF for containing slurry would be of the order of \$290,000/ha. For the CTD option, it would be of the order of \$70,000/ha.

Based on the rehabilitation costs realised in this case study, a scenario in which the unit rate for the area disturbed by a TSF class 1 increases to \$100,000, and the FCR rises to 1.5% was analysed. Annual levy payable to the MRF for the CTD option would represent 1/3 of all the operating costs considered in the OPEX estimate, i.e. comparable to the OPEX for pumping tailings. Regarding the significant effect the annual contribution to the MRF would have on operating costs in this case, an important point should be highlighted. As per the MRF regulations, the unit rate for land under rehabilitation significantly decreases to \$2,000/ha. Conceiving progressive rehabilitation may, therefore, become crucial to reduce operating costs, especially given the issues of tightening regulations discussed in this paper.

4.4 *Comparative evaluation of closure and rehabilitation costs*

A comparative evaluation of the closure cost estimates shows that the total NPC for rehabilitating and monitoring the thickened tailings TSF is 56% higher than the NPC for rehabilitating and monitoring the slurry option. NPC of rehabilitation works for the thickened tailings are of the order of \$3.6M compared to \$2.4M for the slurry tailings option. The main element driving the rehabilitation cost for the CTD option is associated with hauling and placing topsoil to cap the vast surface area of the cone. The difference in closure costs between the options was expected, as the majority of the rehabilitation works depends on the size of the footprint. For both options, rehabilitation works predominantly consist of hauling and placing topsoil to cap the TSF, and deep ripping and spreading of seeds in all disturbed areas. However, it important to be highlighted that the difference in rehabilitation costs is small compared to the significant difference in footprint area between the two alternatives.

Because of the difference in water content of the deposited tailings, different approaches were considered for the reclamation of each TSF. As the slurry option involves the disposal of relatively wet tailings, additional work was costed for covering the facility. On the other hand, rehabilitation of the TSF for containing thickened tailings consists of straightforward earthworks, virtually as soon as tailings deposition ceases.

Differences in the consistency of the tailings also reflect on the required revegetation procedures. For both alternatives, the placement of 300 mm of topsoil for capping the TSF and enhance vegetation establishment was considered. However, for the slurry option, the placement of a layer of hard rock on top of the TSF and on the side slopes of the embankment was assumed, in order to reduce erosion, and impede capillary action and the upward movement of salt within the profile.

Moreover, the life cycle considered for discounting the rehabilitation costs is different due to the differences in tailings solids content. A period of 3 years drying is allowed for the slurry TSF, so that heavy earth moving equipment can begin placing cover layers. Then, after rehabilitation works are completed, a period of 3 years waiting for regulator approval was allowed for. Monitoring costs are accounted for in the estimates for this period of 3 years, and for the following 5 years of post-closure monitoring. For the CTD option, after performing rehabilitation works, monitoring costs are considered for 1 year for regulator approval, and for the following 5 years of post-closure monitoring.

It should be noted that the

In a possible scenario, in which the thickened tailings facility can be progressively restored so that a lower volume of topsoil will be enough for the establishment of vegetation, the NPC of rehabilitation works would be significantly reduced. On the contrary, in a possible scenario where the slurry tailings facility does not meet closure criteria within the considered 5 years of post-closure monitoring, and more time, or even more work, is needed to achieve acceptable rehabilitation, the NPC would possibly increase.

In this case, which is perhaps a more realistic closure scenario, the significant difference in footprint area would not affect the costs of rehabilitating the TSFs as before. Instead, rehabilitation costs would reflect the difference in disposing wet and high density tailings.

5 CONCLUSIONS

Perhaps it is inevitable that challenges to the management of tailings will intensify the pressure on the mining industry to become more sustainable. As with the issues raised in this paper, the continuous growth of the industry depends on having a strong commitment to the development of technology to improve water efficiency, and lower the risks of catastrophic TSF failures.

A conclusion to be drawn from this specific case study is that the key elements driving the costs to dispose tailings are related to the storage of low density slurries, e.g. high costs associated with the large footprint required for tailings storage, the high volume of construction material required for building retaining embankments, the high volume of make-up water needed for mineral processing, or the complex work to rehabilitate the site at closure.

The use of high density tailings may possibly tackle many of the above issues. However, using dewatered tailings is often deemed undesirable because of economics. This conclusion has sometimes resulted from evaluation methodologies that do not incorporate all the costs that may potentially affect the finances of a project. Taking due account of relevant environmental, social and risk costs is crucial to the selection of the most cost-effective option. These factors are difficult to incorporate in cost estimates, but may impose significant expenses and impediments to a project.

Identifying the elements driving the costs of disposing tailings is part of ongoing research aimed at addressing the long-standing challenges of financially quantifying the "real" costs of disposal. Cost comparisons should look beyond short-term expenses, considering the totality of environmental, social, risk, and economic costs over the long-term. For example, in the lead up to the selection of a tailings disposal option, the life-cycle environmental impacts of each alternative should be understood and quantified. Costs associated with stakeholders' acceptance should also be included, possibly through quantifying the costs associated with delays in the permitting process, or related to solving disputes with specific stakeholders and obtaining a social license to operate. Potential failure costs should also be included, possibly by quantifying the clean-up and repair costs incurred from the failure of projects with similar characteristics.

Therefore, a tailings disposal method should be selected based on the assessment of the accumulated costs, using an integrated approach. In the future, the benefits of high density tailings storage will become more prominent, particularly as management challenges increasingly pose pressure on mining operators to seek alternative strategies for tailings disposal. The attractiveness of reducing costs in the short term must be carefully weighed against the possibility of increasing environmental, social, and risk costs throughout the LOM and beyond. In this context, there is an urgent need to adopt more holistic methodologies for comparative evaluations. This serves to enable decision-makers to decide on the best practice for tailings disposal efficiency to ensure sound environmental and social performance.

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Developing Technical Guidance for Leaching Operations, for Flow Regimes and Economic Recovery

David Gilbert
ASARCO Ray Mine, Kearney, AZ, USA

ABSTRACT: The objective of a leach operation is closely tied to operational goals of the mining companies. Operations departments are involved in the distribution piping, pumping, but often don't follow a regulated program bound by physical constraints of the heterogenous aquifer problem. Additionally, cure-rinse cycles are estimated and not guided by applicable drying-wetting knowledge as gained with instrumentation. There is a minor effort in pre-construction planning and modeling of the respective leach volume. By standardizing the pre-leach planning, and modelling, the mineral value return can be optimized and mine resources more efficiently allocated. This paper will describe the sequence of technical activities during all phases of injection leaching for operational guidance and better economic return value to SXEW operations.

1 INTRODUCTION

The dump leach processes are often performed using ad hoc methodologies. The planning sequence consists of placement, leach-rinse, and sampling for recovery. This article will demonstrate how a project methodology can assist and add value to an operations function.

Pre-construction planning, modelling, and analysis will engage SXEW operations to optimize the operating function. Preconstruction planning consists of material characterization and physical parameters of flow through the dump. Water balance applications and monitoring are more effective as a tracking device.

1.1 *Pre-construction Planning*

Simple scheduling tools and pre-construction meetings will allow for optimal use of resources, including better flows through the leach and dump recovery. Part of the planning function includes material sampling, visual reconnaissance of the dump, and operations interviews with SXEW staff. A sample layout is prepared showing leach application methodology (sprinkler, injection), well construction, and leach monitoring type (moisture sensors, piezometers). A base case slope stability analysis is performed as well as pertinent geophysics to rock deposition area (RDA). Spacing of injection wells should include any geophysics results and best engineering judgement for the RDA material. Engineering judgement can include previous visual observations of exterior slope "popouts" and appropriate geotechnical stability analyses for the dump. As such, appropriate setback distances can be established for injection wells (see Figure 1). Additionally, pressure sensitivity and depth to first screened interval can be done using a heave calculation for the material.

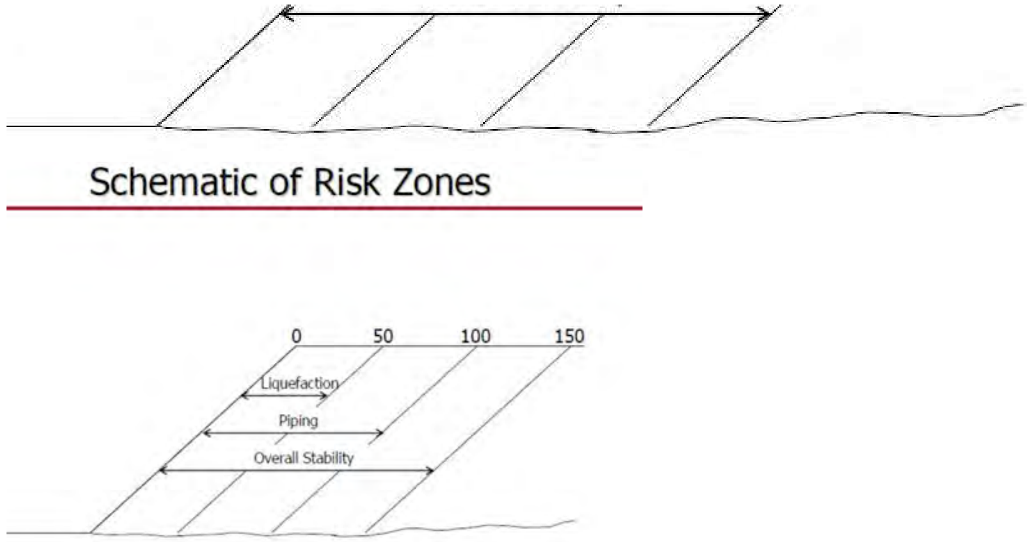


Figure 1 – Setback distances from dump face for geotechnical criteria

1.2 *Construction and Monitoring*

Construction diagrams, borehole installation guidance and monitoring criteria are recorded and prepared into a construction plan using inputs from pre-construction analysis and characterization. Construction plans include well diagrams in section and plan (see Figure 2). All plans should be documented and activity registers prepared using a checklist format in preconstruction, and operational guidance for construction. Similarly, instrumentation is drawn from stability analysis, historical precedent, and saturation (moisture sensors) and potentiometric profile (piezometers).

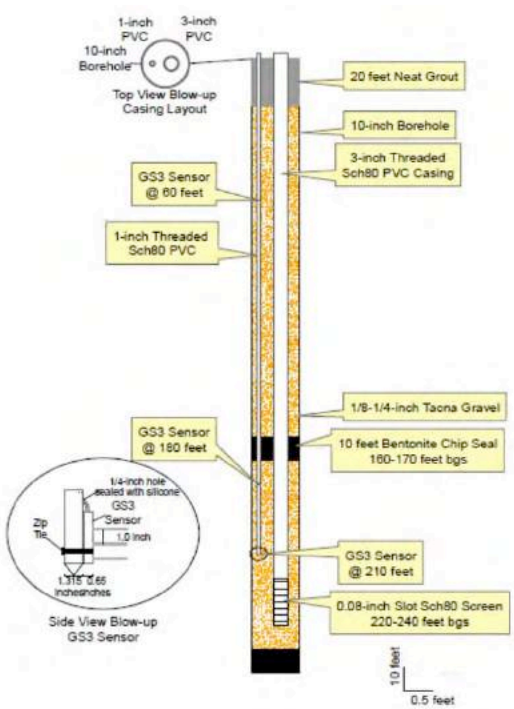


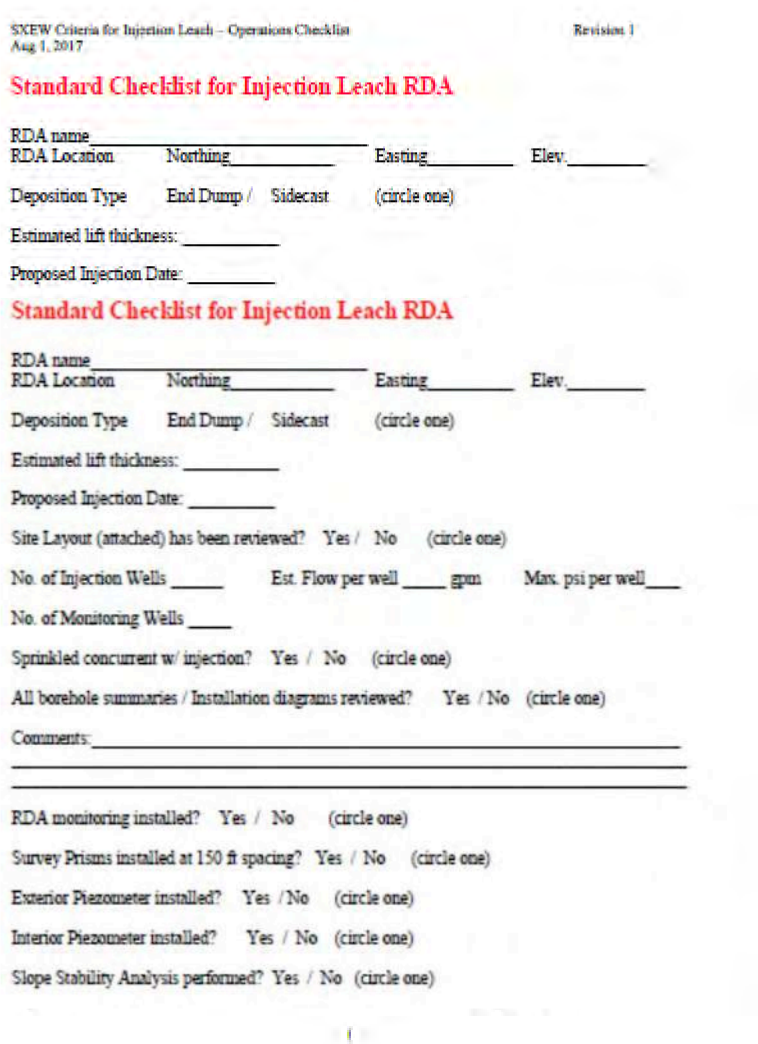
Figure 2 – Typical installation diagram for borehole sequence

2 DISCUSSION

This section will describe in further detail the importance of planning, and monitoring functions.

2.1 *Preconstruction Planning*

Prior to installation of a plumbing layout, all site characterization and analysis should be organized and compiled, so as to direct the initial design of plumbing layout, screened intervals, operating flows and pressures and monitoring. Site characterization should dovetail into preconstruction after thorough review of geophysics (hydraulic mound and dispersion), hydraulic function (conductivity), depth to vadose zone, and possibly metallurgical guidance (ore recovery). A preconstruction plumbing layout in plan, and cross section for injection wells should be considered part of the project file. Plumbing layouts should include monitoring locations, instrumentation, on planning will form the basis plexity and number of injection e combined. All preconstruction ons personnel before drilling and performing geophysical analysis input on depth to screened interlation requirements are met (see



SXEW Criteria for Injection Leach – Operations Checklist Revision 1
Aug 1, 2017

Standard Checklist for Injection Leach RDA

RDA name____________________
RDA Location Northing________ Easting_______ Elev._______

Deposition Type End Dump / Sidecast (circle one)

Estimated lift thickness: ________

Proposed Injection Date: ________

Standard Checklist for Injection Leach RDA

RDA name____________________
RDA Location Northing________ Easting_______ Elev._______

Deposition Type End Dump / Sidecast (circle one)

Estimated lift thickness: ________

Proposed Injection Date: ________

Site Layout (attached) has been reviewed? Yes / No (circle one)

No. of Injection Wells _____ Est. Flow per well ____ gpm Max. psi per well____

No. of Monitoring Wells ____

Sprinkled concurrent w/ injection? Yes / No (circle one)

All borehole summaries / Installation diagrams reviewed? Yes / No (circle one)

Comments:____________________________________

RDA monitoring installed? Yes / No (circle one)

Survey Prisms installed at 150 ft spacing? Yes / No (circle one)

Exterior Piezometer installed? Yes / No (circle one)

Interior Piezometer installed? Yes / No (circle one)

Slope Stability Analysis performed? Yes / No (circle one)

2.2 *Construction and Monitoring*

After characterizing the site using investigative techniques, the drilling begins and should be supervised by a qualified geologist or engineer familiar with the testing, modelling, and leach thickness (to bedrock and/or original ground)at each individual location. Depending on location, cuttings and other physical sampling should seek to support the findings of the investigative phase.

If original installation assumptions are found to be false, operations staff should be informed, and a revised installation sequence should be prepared and signed off. During drilling, blow counts should be recorded for possible adjustments to instrumentation or screened interval (see Figure 4). Usually there is a pattern, whereby top of previously placed lifts are harder, with higher blow-counts.

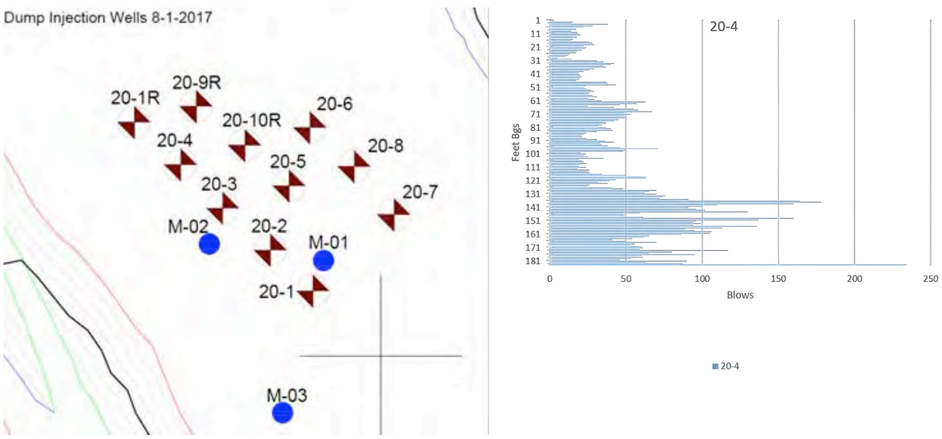


Figure 4 – Correlation of blowcounts to end dump lift thickness at RDA

Monitoring locations and instrumentation should be installed and baseline references established by taking preliminary readings prior to leach injection (see Figure 5). During this phase it is possible that conductivity tests (slug tests, falling head) should be carried out in the field for later inputs into hydrogeological model.

Figure 5 – Performing baseline readings for moisture sensors

Construction diagrams, borehole installation guidance and monitoring criteria are recorded and prepared into a construction plan using inputs from pre-construction analysis and characterization. Construction plans include well diagrams in section and plan. All plans should be documented and activity registers prepared using a checklist format in preconstruction, and operational guidance for construction. Similarly, instrumentation is drawn from stability analysis, historical precedent, and saturation (moisture sensors) and potentiometric profile (piezometers).

2.3 *Geotechnical, Flow Monitoring and Other Considerations*

Geotechnical background for each RDA can be optimized by examining the unsaturated flow concepts associated with the timing sequences of cure-rinse cycles. Lessons learned might seek to develop further understanding of physical properties of the dump including gradations with depth (see Figure 6) and volumetric water content of the dump at any point in time. Installation of flowmeters, real-time moisture sensors (see Figure 7), precludes need for costly geophysics snapshots (see Figure 8). Flow characteristics can also be aided by correct installation of screened intervals, representative to dump cross section (see Figure 9).

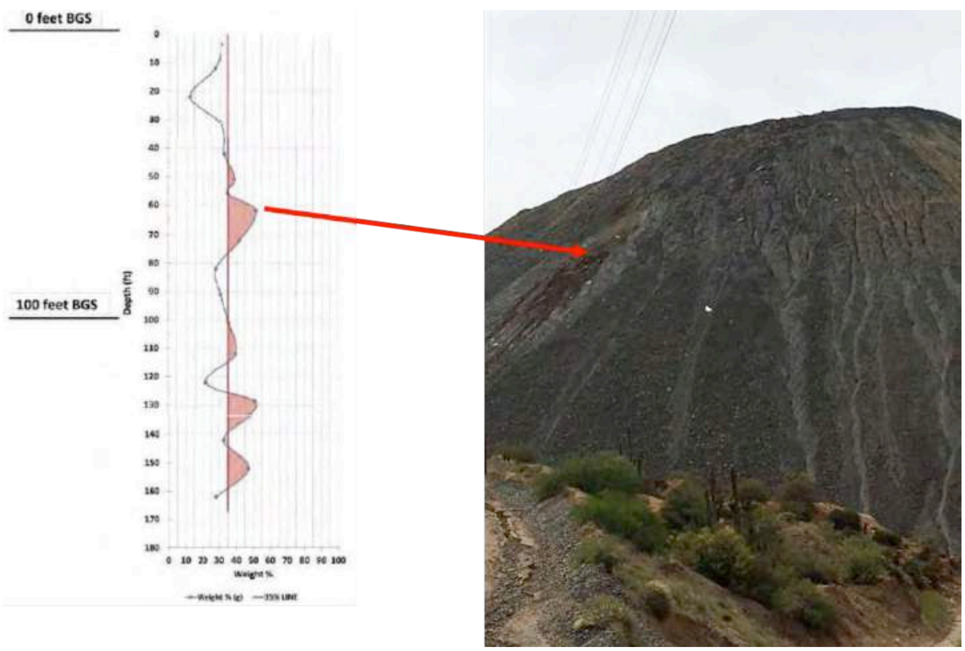


Figure 6 – Bridging of fines layer creates erosion pathway and sliver popout

Figure 7 – commercially available moisture sensors and flowmeters to evaluate fluids dispersion through RDA

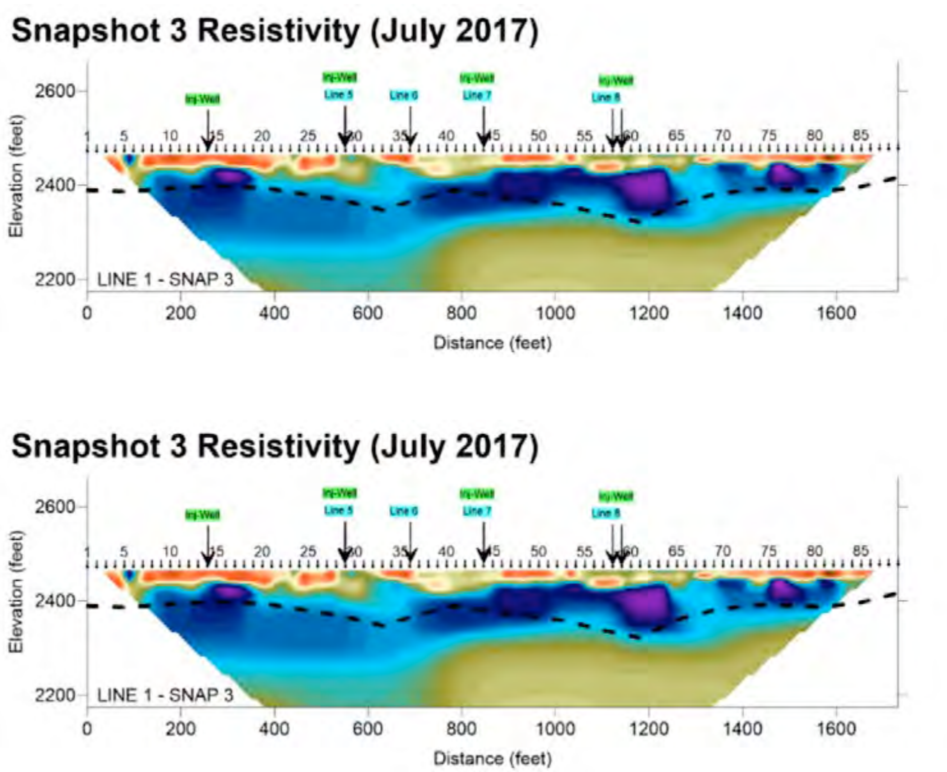


Figure 8 – Comparison of Geophysics to determine flow radius of injection well and dispersion

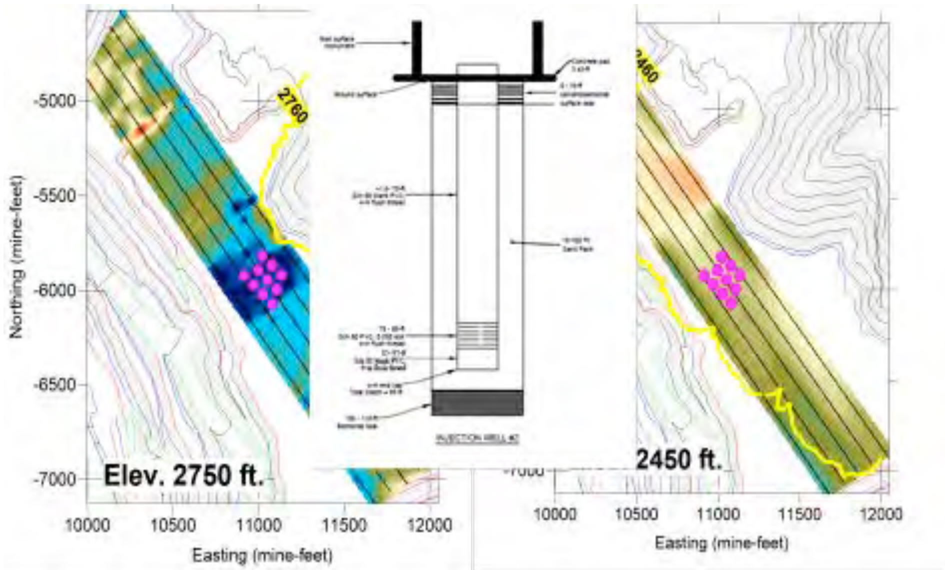


Figure 9 – Injection well spacings and depth to screened intervals characteristic of RDA geophysics

Water balance accounting is made more difficult by heterogenous nature of dump materials, and time to create preferential erosion pathways which can be measured. A more representative approach might be to understand original ground topography, as related to collection ponds and use stage capacity relationships for measurement over time (see Figure 10).

Figure 10 – Potential consideration of stage capacity, as related to flows in hydrologic sub-basins

3 CONCLUSION

Building on each successive phase of injection leach planning, and adopting a quality approach to the production aspects will allow for better monitoring, and correct understanding of leach volume. Standardizing certain aspects frees up time for investigation and discovery, and the subsequent development of the body of knowledge for predicting economic recovery. Future opportunities include better characterization of the unsaturated flow concepts tied to economic recovery, and optimization of dispersion. Possible applications include controlled shock blasting, and air injections between injection cycles. Dump Leach development is best suited to the proper engineering judgement (Peck, 1984) during all phases of the operation. Once a physical understanding is modelled, economics can be evaluated for improving operations strategy.

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Hydrogeological Aspects of Waste Disposal

Using Bentonite-Polymer Composite Geosynthetic Clay Liners to Contain Bauxite Liquor

Kuo Tian
Department of Civil, Environmental, and Infrastructure Engineering, George Mason University, Fairfax, VA, USA

Craig H. Benson
School of Engineering, University of Virginia, Charlottesville, VA, USA

ABSTRACT: Experiments were conducted to evaluate the effect of a hyperalkaline bauxite liquor from aluminum refining on the hydraulic conductivity of geosynthetic clay liners (GCLs) used in composite liner systems. Tests were conducted on two GCLs: one containing conventional sodium-bentonite (Na-B) and the other containing a bentonite-polymer (B-P) composite. Both GCLs were prehydrated to 70% water content by misting with tap water. The Na-B GCL was approximately four orders of magnitude more permeable to the bauxite liquor than to DI water ($1.5x10^{-7}$ m/s vs. $2.1x10^{-11}$ m/s). In contrast, hydraulic conductivity of the B-P GCL to bauxite liquor was low and similar to the hydraulic conductivity to DI water ($1.4x10^{-11}$ m/s vs. $1.1x10^{-11}$ m/s). Suppression of osmotic swelling of the bentonite is the primary factor responsible for the high hydraulic conductivity of the conventional Na-B GCL to bauxite liquor relative to DI water. Low hydraulic conductivity of the B-P GCL is attributed to polymer clogging intergranular pores between bentonite granules. The findings suggest that B-P GCLs can be viable for composite liners containing bauxite liquor.

1 INTRODUCTION

Geosynthetic clay liners (GCLs) are factory-manufactured barriers comprised of a layer of sodium-bentonite (Na-B) sandwiched between two geotextiles (Bradshaw and Benson 2014, Scalia et al. 2014, Tian et al. 2016). GCLs are common elements in waste containment facilities due to their low hydraulic conductivity to water (typically < 10^{-10} m/s) and ease of installation (Shackelford et al. 2000). The low hydraulic conductivity of GCLs is primarily due to osmotic swelling of Na-B, which creates narrow and tortuous flow paths (Bradshaw and Benson et al. 2014).

The hydraulic conductivity of GCLs can be affected by chemical interactions between the Na-B and liquid to be contained (Jo et al. 2001, 2005, Kolstad et al. 2004). GCLs permeated with more aggressive leachates having higher ionic strength and/or a predominance of polyvalent cations can be orders of magnitude more permeable than GCLs permeated with deionized (DI) or tap water due to suppression of swelling of the Na-B (Jo et al. 2001, 2005, Kolstad et al. 2004, Shackelford et al. 2010, Scalia et al. 2014, Tian et al. 2015, 2016). Consequently, recent studies have focused on using polymer-modified bentonite to improve the chemical compatibility of GCLs with aggressive liquids like bauxite liquor (Katsumi et al. 2008, Scalia et al. 2014, De Camillis et al. 2016, Tian et al. 2016, 2017). GCLs containing bentonite-polymer (B-P) composites have been evaluated for containment of leachates from mine waste (Shackelford et al. 2010), coal combustion products (Salihoglu et al. 2016), and low-level radioactive waste and mixed waste (Tian et al. 2016). Tian et al. (2016) proposed that a polymer clogging mechanism controls the hydraulic conductivity of B-P GCL to aggressive leachates.

Bauxite liquor can be particularly aggressive with GCLs. Benson et al. (2008) evaluated the hydraulic conductivity of two conventional Na-B GCLs to a bauxite liquor from an aluminum refining operation. One GCL contained granular bentonite and the other powdered bentonite. The leachate had pH 12.2 and an ionic strength of 774 mM, with Na (409 mM) and Al (129 mM) being the predominant metals. The hydraulic conductivity of one GCL containing granular bentonite increased to as high as 1.8×10^{-8} m/s when permeated with bauxite liquor, whereas the hydraulic conductivity of the GCL with powdered bentonite decreased to approximately 2.5×10^{-12} m/s. The increase in hydraulic conductivity was attributed to reduction in swelling of bentonite granules in the bauxite liquor. Precipitation of aluminum complexes in the fine pores of the powdered bentonite was hypothesized to clog flow channels, resulting in lower hydraulic conductivity.

The objective of this study was to evaluate the effect of bauxite liquor on the hydraulic conductivity of two geosynthetic clay liners (GCLs) being considered for a composite liner at an aluminum refinery. Tests were conducted on a GCL containing conventional Na-B and a GCL containing a bentonite-polymer (B-P) composite. The bauxite liquor used in the experiments has pH 13.0 and an ionic strength of approximately 700 mM. Hydraulic conductivity tests were conducted in flexible-wall permeameters. Before permeation, the GCL specimens were prehydrated with tap water to 70% water content to mimic subgrade hydration that would occur in situ (Bradshaw and Benson 2014). Deionized (DI) water was also used for permeation as a control. Swell index tests were conducted on the Na-B and B-P with bauxite liquor and DI water.

2 MATERIALS AND METHODS

2.1 *Geosynthetic clay liners*

Two GCLs were evaluated in this study. One GCL contained conventional Na-B and the other a B-P composite. The B-P composite material was created using the slurry polymerization process described in Scalia et al. (2014). Each GCL consists of granular material (bentonite granules or granules of bentonite-polymer composite) sandwiched between a non-woven geotextile (top) and a woven geotextile (bottom). For both GCLs, the bentonite is encased in polypropylene geotextiles bonded by needlepunching.

Physical properties of the GCLs are summarized in Table 1. Polymer loading was determined using the loss on ignition method. The major mineral components in each bentonite, determined by X-ray diffraction, are summarized in Table 2. Montmorillonite is the predominant mineral in each bentonite; quartz, feldspar, calcite, and cristobalite are present in measureable quantities.

Table 1. Physical properties of the GCLs used in the study.

GCL	Initial Thickness (mm)	Mass Per Unit Area (kg/m2)	Polymer Loading (%)
Na-B	5.1	3.7	-
B-P	4.8	4.4	14.6

Note: Polymer loading is determined using loss on ignition test per ASTM D7348.

Table 2. Mineralogy of bentonite in GCLs.

GCL	Montmorillonite (%)	Calcite (%)	Feldspars (%)	Cristobalite (%)	Quartz (%)	Other (%)
Na-B	77	1	5	2	15	0
B-P	75	1	4	0	16	3

2.2 *Bauxite liquor*

The bauxite liquor used in the testing program was obtained from an aluminum refinery and analyzed for elemental concentrations by inductively coupled plasma-optical emission spectroscopy (ICP-OES). The bauxite liquor has pH 13.0, ionic strength of 700 mM. Al (~119 mM) and Na (~456 mM) are the predominant metals.

2.3 *Prehydration*

Prior to permeation, the Na-B and B-P GCL specimens were prehydrated with tap water to simulate hydration after installation associated with water uptake from a subgrade (Bradshaw and Benson 2014). The GCLs were hydrated by misting with tap water to 70% water content, a typical water content for a GCL hydrated on a subgrade for 60 days (Bradshaw and Benson 2014). Prehydration of the GCL was anticipated to enhance the chemical compatibility by inducing osmotic swell of the bentonite and formation of a polymer hydrogel prior to contact with bauxite liquor (Bradshaw and Benson 2014).

2.4 *Swell index*

Swell index tests were conducted on bentonite from the Na-B and B-P GCLs according to ASTM D5890 with bauxite liquor and DI water. Air-dried Na-B and B-P were ground using a mortar and pestle until 100% passed the No. 200 sieve. Approximately 90 mL of hydrating liquid was added to the 100 mL graduated cylinder, followed by 2.0 g of dry and ground material added slowly in 0.1 g increments. Hydrating liquid was used to rinse particles adhering to the sidewalls and to fill the cylinder to the 100 mL mark. After 24 h of exposure, the swell index was recorded in mL/2.0 g.

2.5 *Hydraulic conductivity*

Hydraulic conductivity tests on the GCL specimens were conducted in flexible-wall permeameters using the falling headwater - constant tailwater method described in ASTM D6766. Backpressure saturation was not applied to simulate the *in situ* condition and to avoid unrealistic changes in geochemistry. After prehydration, the effective confining stress was set at 20 kPa and the hydraulic gradient was set at 125. Influent was contained in 50 mL burettes sealed with parafilm to prevent evaporation and carbonation. Effluent was collected in 60-mL polyethylene bottles sealed with parafilm.

Equilibrium was defined using the hydraulic and chemical equilibrium criteria in ASTM D6766. The criteria in D6766 require no temporal trend in the hydraulic conductivity measurements, hydraulic conductivity falling within 25% of the mean for three consecutive measurements, incremental effluent volume (Q_{out}) within 25% of the incremental influent volume (Q_{in}) for at least 3 measurements, and the ratio Q_{out}/Q_{in} exhibiting no temporal trend. Chemical equilibrium is defined as the electrical conductivity and pH of the effluent (EC_{out} and pH_{out}) showing no temporal trend and falling within 10% of the electrical conductivity and pH of the influent (EC_{in} and pH_{in}).

3 RESULTS AND DISCUSSION

3.1 *Swell index*

Swell index of the Na-B and B-P to DI water and bauxite liquor is shown in Fig. 1. The swell index of Na-B was 25.0 mL/2.0 g in DI water and 9.0 mL/2.0 g in bauxite liquor. Swell index of the B-P was 74.5 mL/2.0 g in DI water and 17.5 mL/2.0 g in bauxite liquor.

The change in swell index of the Na-B reflects how the high ionic strength of bauxite liquor (700 mM) suppresses osmotic swell, as the magnitude of osmotic swelling is inversely proportional to the concentration of cations in the pore water (Norrish and Quirk 1954). The lower swell index of the B-P in bauxite liquor illustrates that swelling of the polymer hydrogel is also sensitive to the solution chemistry.

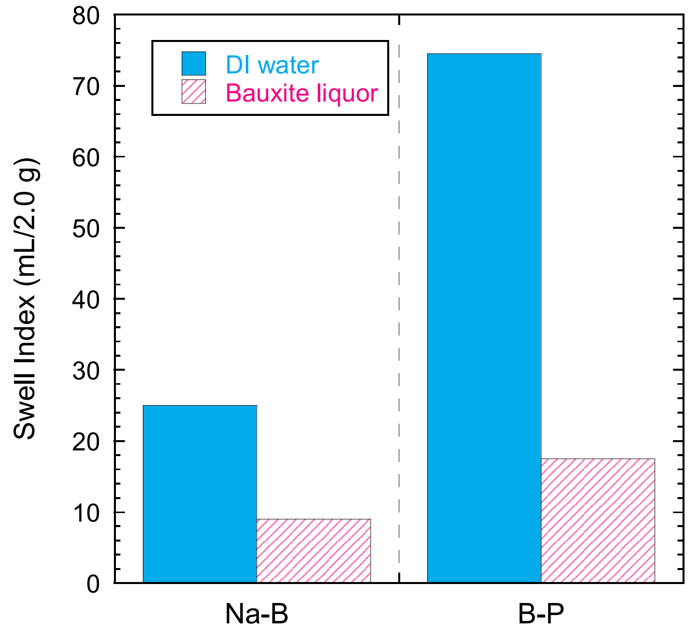

Figure 1. Swell index of Na-B and B-P to DI water and bauxite liquor

3.2 *Hydraulic conductivity tests*

Hydraulic conductivity tests were conducted on the Na-B and B-P GCLs with bauxite liquor and DI water for 21–280 d. Na-B GCLs permeated with bauxite liquor met the termination criteria in ASTM D6766, whereas the B-P GCLs had not reached chemical equilibrium on termination. Pore volumes of flow (PVF) were calculated based on the final properties of each GCL specimen (e.g., thickness, diameter, and mass).

Table 3. Hydraulic conductivity of GCLs permeated with bauxite liquor and DI water.

GCL	Permeant Solution	Test Duration (d)	Prehydration Water Content (%)	Hydraulic Conductivity (m/s)
Na-B	Bauxite Liquor	21	70	$1.5x10^{-7}$
Na-B	DI Water	266	-	$3.4x10^{-11}$
B-P	Bauxite Liquor	280	70	$1.1x10^{-11}$
B-P	DI water	280	-	$1.4x10^{-11}$

Hydraulic conductivity of the Na-B and B-P GCLs permeated with bauxite liquor as a function of PVF is shown in Fig 2. Hydraulic conductivity of the Na-B GCL increased to $1.5x10^{-7}$ m/s immediately, whereas the hydraulic conductivity of the B-P slowly increased from $2.5x10^{-12}$ m/s to approximately $1.4x10^{-11}$ m/s during the first 8 PVF, and then remained relatively constant.

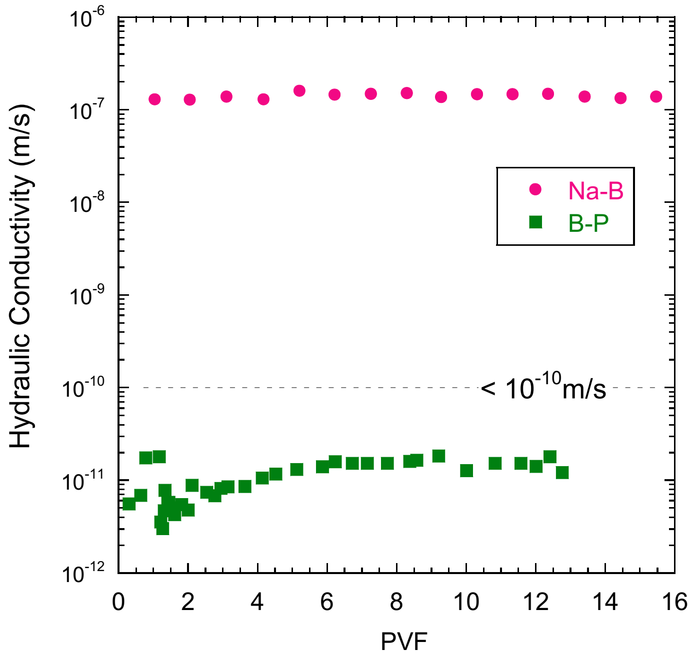

Figure 2. Hydraulic conductivity vs. pore volumes of flow for Na-B and B-P GCLs permeated with bauxite liquor.

Hydraulic conductivities of both GCLs to bauxite liquor and DI water are shown in Fig. 3. The hydraulic conductivity of Na-B to bauxite liquor is approximately four orders of magnitude higher than the hydraulic conductivity to DI water (1.5×10^{-7} m/s vs. 3.4×10^{-11} m/s). Reduction in swelling of the bentonite is the primary factor responsible for the higher hydraulic conductivity of the conventional Na-B GCL to bauxite liquor relative to DI water (e.g., swell index in bauxite liquor = 9.0 mL/2.0 g vs. 25.0 mL/2.0 g in DI water, Fig. 1). This finding is consistent with Jo et al. (2001), which showed that the hydraulic conductivity of Na-B GCLs increases as the swell index drops below 16 mL/2.0 g.

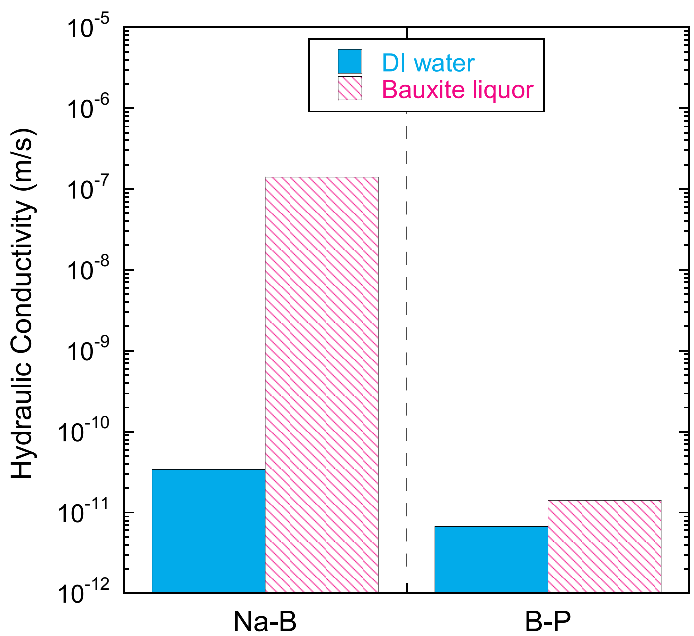

Figure 3. Hydraulic conductivity of Na-B and B-P GCL permeated with DI water and bauxite liquor.

In contrast to the Na-B GCL, hydraulic conductivity of the B-P GCL to bauxite liquor was similar to the hydraulic conductivity to DI water (1.4×10^{-11} m/s vs. 1.1×10^{-11} m/s) (Fig. 3). The low hydraulic conductivity of the B-P GCL is attributed to polymer clogging intergranular pores controlling flow of bauxite liquor through the GCL, as described in Tian et al. (2016, 2017). The polymer forms a three-dimensional hydrogel structure that occupies the intergranular pores space when bentonite swelling is suppressed (Scalia et al. 2014, Tian et al. 2016), resulting in low hydraulic conductivity. These findings suggest that B-P GCLs can be viable for composite liners for bauxite liquor disposal facilities.

4 CONCLUSION

This study evaluated the impact of bauxite liquor on the hydraulic conductivity of two commercially available GCLs. One GCL contained conventional sodium-bentonite (Na-B) and the other a bentonite-polymer composite (B-P). Prior to permeation, the Na-B and B-P GCLs were prehydrated with tap water to 70% water content to simulate subgrade hydration.

Hydraulic conductivity of the conventional Na-B GCL increased immediately when permeated with bauxite liquor. The final hydraulic conductivity was approximately four orders of magnitude higher than the hydraulic conductivity to DI water (1.5×10^{-7} m/s vs. 3.4×10^{-11} m/s). Suprresion of swelling of the Na-B in bauxite liquor is the primary factor responsible for the higher hydraulic conductivity. Prehydration to 70% water content had no beneficial effect on the hydraulic conductivity to bauxite liquor.

Hydraulic conductivity of the B-P GCL to bauxite liquor was very low and similar to the hydraulic conductivity to DI water (1.4×10^{-11} m/s vs. 1.1×10^{-11} m/s). The low hydraulic conductivity of the B-P GCL is attributed to polymer hydrogel clogging intergranular pores that conduct flow in the Na-B GCL. This finding suggests that a B-P GCL prehydrated to 70% water content may be viable barrier for lining disposal facilities containing bauxite liquor.

ACKNOWLEDGEMENTS

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Salinity effects on the consolidation behavior of kaolin

Wesley Herweynen, Christopher A. Bareither & Joseph Scalia IV
Colorado State University, Fort Collins, CO, USA

ABSTRACT: The objective of this study was to evaluate the effects of salinity on consolidation and hydraulic conductivity of kaolin via seepage-induced consolidation tests. The influence of salinity was assessed by (i) adding calcium chloride to kaolin during hydration of slurry specimens and (ii) hydrating slurried specimens of kaolin in tap water and subsequently introducing calcium chloride to the kaolin during seepage. An increase in salinity led to increased consolidation settlement for all experiments. However, kaolin slurries initially hydrated with tap water and subsequently seeped with calcium chloride yielded larger consolidation settlement than kaolin slurries hydrated in the salt solutions. This behavior was attributed to flocculated clay structures in tap water slurries that had greater potential for void volume change with increasing effective stress. Comparable hydraulic conductivities were measured at similar void ratios for all specimens, with the lowest hydraulic conductivities corresponding to the lowest void ratios obtained with high salt solutions.

1 INTRODUCTION

Mine tailings are the mill rejects and residuals from the processing of ore to extract useful and precious minerals, metals, and other resources (Wang et al. 2014). Tailings typically consist of a range of materials from high plasticity clay to non-plastic sand. As produced, mine tailings typically have high moisture content, low solids content, and can be pumped in the form of a slurry for storage in a tailings storage facility (TSF). With the ever increasing demand for mineral products and the depletion of high grade ore deposits, mining of low grade ores has become standard practice. Consequently, the volume of tailings produced is increasing, on the order of millions of tonnes being produced annually to add to the already billions of tonnes being stored globally (Wang et al. 2014).

Storage of such large quantities of high water content mine tailings not only incurs high costs for mine operators, but also presents significant environmental concern and potential liabilities. Therefore, the dewatering of mine tailings remains a primary objective for mine operators. Wang et al. (2014) discussed the current theoretical understanding and industrial best practices on treating mine tailings for improved water recovery. Altering the solution chemistry of mine tailings through the addition of flocculants, such as inorganic salts, is one of the best available technologies for improving the geotechnical and rheological behavior of mine tailings.

Sedimentation is the tendency of soil particles to settle out of suspension and is a function of particle size and shape, and fluid density and viscosity (Gorakhki and Bareither 2016). As particles settle out of suspension to form a sediment, excess pore water pressure develops due to self-weight consolidation of the mine tailings (Williams et al. 2013). As pore water pressure dissipates, tailings volume decreases and effective stress increases. Adding inorganic salts to mine tailings can induce flocculation of suspended particles, which improves sedimentation and consolidation, enhances water recovery from mine tailings, and increases storage capacity in TSFs.

The effect of soluble salt concentration on sedimentation is well understood (e.g., Olson and Mitronovas 1962; Mitchell and Soga 2005; Palomino and Santamarina 2005). Williams et al. (2013) studied the effect of commercially available salts [$NaCl$, $CaCl_2$, $MgCl_2$, KCl, $CaSO_4$, $Al_2(SO_4)_3$] on the sedimentation behavior of four sodium bentonites, calcium bentonite, and kaolinite. The study demonstrated that increasing the concentration of dissolved salts generally improved sedimentation behavior. However, the degree of improvement was dependent on the clay mineralogy and the type of dissolved salt. Gorakhki and Bareither (2015) studied the effects of soluble salt concentration on sedimentation behavior of a low-plasticity kaolinite and high-plasticity bentonite. An increase in salinity yielded higher sedimentation rates and higher final solids content in the bentonite. In contrast, higher salt concentrations for kaolin yielded lower sedimentation rates, whereas the final kaolin solids content was unaffected by salinity. These studies demonstrated that the influence of salinity on sedimentation of fine-grained soils is dependent on clay mineralogy.

Numerous studies have also been conducted on the effects of soluble salt concentration on consolidation behavior of fine-grained soils. Olson and Mitronovas (1962) conducted one-dimensional consolidation tests on calcium and magnesium illite specimens sedimented from suspension with varying pore fluid soluble salt concentrations. Results showed that the relationship between void ratio and effective stress is affected by soluble salt concentrations of the pore water, but only to the extent that the pore fluid chemistry changed the initial void ratio after sedimentation. The final void ratio under vertical effective stresses of 2800 kPa was unaffected by soluble salt concentration of the pore fluid.

Barbour and Yang (1993) investigated the influence of pore fluid chemistry on the compressibility and permeability of two calcium montmorillonitic clayey soils. Permeation with a salt solution produced a 1.0-3.5% reduction in volume. This chemically-induced consolidation resulted in overconsolidation of the clayey soils. Once the chemically-induced preconsolidation stress was exceeded, the clayey soils consolidated along a similar virgin compression curve to soils consolidated with fresh water. Pore fluid salinity was also shown to have limited influence on hydraulic conductivity of consolidated clay specimens when initially prepared as slurry.

The objective of this study was to investigate the effect of salinity on consolidation behavior of kaolin clay via seepage-induced consolidation tests. Salinity was varied in the kaolin specimens via two methods: (i) preparing kaolin slurry with varying strength calcium chloride solution and subsequently permeating with the same calcium chloride solution; and (ii) preparing a kaolin slurry with tap water and subsequently permeating to chemical equilibrium with varying strength calcium chloride solution. Thus, a key objective was to evaluate the effects of mixing versus permeating with a saline solution on the consolidation behavior of slurried kaolin. This research effort was developed to provide baseline data for subsequent testing on mine tailings as well as to assess how exposure of slurry to saline solution at different stages of the slurry can influence consolidation behavior. The settling and consolidation behavior of tailings are key factors for mine operators in predicting long term storage capacity of TSFs. Therefore, having a thorough understanding of this behavior is essential for efficient, optimized, and cost-effective tailings storage and management.

2 SEEPAGE-INDUCED CONSOLIDATION TEST

The seepage induced consolidation test (SICT) was originally developed to evaluate consolidation and hydraulic conductivity properties of soft, slurry materials at low effective stress (e.g., Znidarčić et al. 1992; Fox and Baxter 1997). A new SICT was designed and constructed at Colorado State University (CSU) based on original characteristics of the experiment, but modified to include a larger diameter specimen (152-mm diameter) and capability to test with non-standard solution chemistry. Thus, the SICT apparatus at CSU has the fundamental benefits of a traditional SICT test (e.g., testing slurry materials over a low stress range), but also allows permeation of non-standard chemical solutions during consolidation and hydraulic conductivity measurement.

A schematic of the SICT apparatus is shown in Fig. 1 (Tian 2017). The apparatus includes a 152-mm diameter specimen cell, flow pump capable of controlling flow rate over the range of permeabilities applicable to tailings, vertical loading piston with load cell, a Mariotte bottle for constant head on the top of the specimen, and top and bottom pressure transducers for monitoring pore pressure. Detailed discussion of the design, calibration, and assessment of this new SICT is in Tian (2017).

The SICT testing procedure consisted of three steps. In the first step, the void ratio at zero effective stress (e_0) was determined. This corresponded to the void ratio at the end of sedimentation and beginning of consolidation. The e_0 was determined via depositing slurry into the specimen cell to an initial thickness of approximately 50 mm. The slurry was allowed to settle and supernatant water was subsequently removed. The final thickness of the sedimented slurry was measured and an average water content and e_0 were computed assuming a saturation of 100%. The 152-mm-diammeter slurry column assisted in reducing the error in mass-volume calculations to determine e_0.

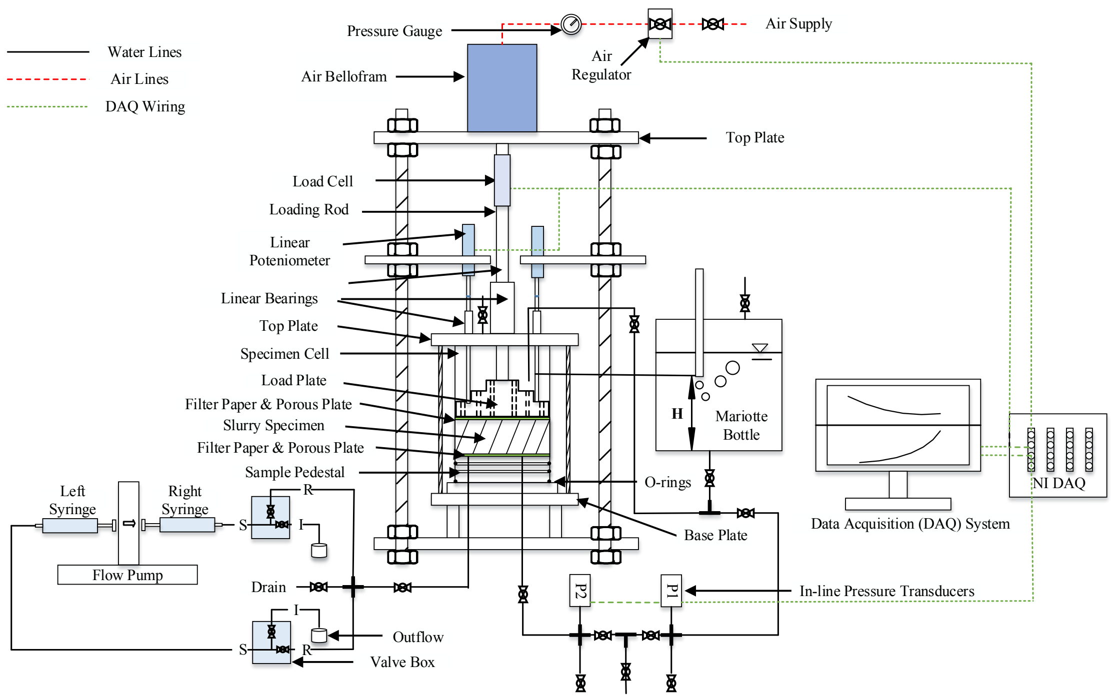

Figure 1. Schematic of the seepage-induced consolidation test (SICT) apparatus.

In the second step, the flow pump was used to create downward seepage within the slurry and induce consolidation. A constant flow rate was imposed on the specimen until steady-state conditions were achieved, which was identified as a constant pressure difference across the specimen. The pressure difference was computed via the pressure transducers at the top and bottom of the specimen. Steady-state was also confirmed as no subsequent change in vertical deformation measured with two linear potentiometers (Fig. 1). A steady-state void ratio (e) and saturated hydraulic conductivity (k_s) were calculated for each imposed flow rate. The k_s was computed by Darcy's Law using the pressure difference and specimen height at steady-state. Furthermore, the seepage force varies across the specimen at steady-state conditions, which inturn produces a decreasing e with depth in the specimen from the top surface. The void ratio at the mid-depth was adopted herein as a measure of the average void ratio of the specimen at steady-state. This second step was repeated for at least two additional flow rates to obtain e and k_s at three different flow rates that corresponded to three different effective stresses (σ').

The third step involved vertical load application via a load piston (Fig. 1). Vertical force was generated by a pneumatic air cylinder connected to the load piston. The third step was initiated once sufficient consolidation was achieved during the seepage phase, and the first load applied via the air cylinder was typically in the range of $\sigma' \approx 10$ kPa. Steady-state consolidation during each vertical load increment was verified via measurements of pore pressure and vertical displacement. The e at steady-state was computed directly from mass-volume relationships assuming that the σ' applied via the load piston was transferred equally with depth in the specimen and e did not vary with depth. The k_s at steady-state was measured via applying a small flow rate that would not induce further consolidation, but was large enough to induce a measureable pressure difference across the specimen to compute k_s. This third step was repeated until a final, target σ' was reached. At the end of an experiment, soil was exhumed from the specimen to measure the final water content and compute a final *e*.

Data analysis for the SICTs was performed based on the theory described by Liu and Znidarčić (1991) with implementation in Microsoft Excel. Liu and Znidarčić (1991) proposed the following two relationships to define the compressibility (*e*-σ') and hydraulic conductivity (k_s-*e*) relationships for a SICT test:

$$e = A\left(\sigma' + Z\right)^B \quad (1)$$

$$k_s = C \cdot e^D \quad (2)$$

where A, B, Z, C, and D are fitting parameters. These fitting parameters were determined via the Solver function in Excel to minimize the sum of squared residuals between the measured data and predicted *e*-σ' and *e*-k_s relationships.

3 MATERIALS AND METHODS

3.1 *Materials*

Kaolin clay was used in this study, which consisted predominantly of kaolinite minerals. Kaolin slurry was prepared with a solids content of 35%, which was approximately three times the liquid limit and allowed the material to easily be poured into the SICT specimen cell. Salinity of the pore fluid was varied via addition of calcium chloride dihydrate ($CaCl_2$). Experiments were conducted with tap water and $CaCl_2$ solutions prepared to 0.1 M and 1.0 M. Kaolin slurries were either prepared with tap water and hydrated for 24 h, or prepared with one of the $CaCl_2$ solutions and allowed to hydrate for 24 h. Seepage solutions used during SICT also included tap water and the two $CaCl_2$ solutions. The two $CaCl_2$ solutions were prepared via dissolving a known mass of $CaCl_2$ into a known volume of tap water. Solutions were checked for consistency in preparation by measuring the electrical conductivity (EC).

3.2 *Test Methodology*

A summary of the experiments conducted for this study is in Table 1. Experiments included two stages: (i) hydration and (ii) seepage. The main variable between any of the five experiments conducted was solution chemistry of the hydration and seepage solutions. Otherwise, a consistent SICT methodology (as described previously) was implemented for all experiments. A final σ' = 40 kPa was applied in all five experiments via the load piston.

Kaolin slurry was prepared in the target hydration solution and subsequently subjected to a seepage solution. The experiment that included tap water as the hydration and seepage solution (T1) was considered the baseline case, and was used for comparison to all other experiments that included varying salinity for hydration and/or seepage. Two experiments were conducted with kaolin slurry hydrated in tap water and then subjected to 0.1 M or 1.0 M $CaCl_2$ seepage solutions (T4 and T5). In these two experiments, all plumbing lines between the flow pump and specimen were saturated with tap water. The $CaCl_2$ solution was introduced during the first seepage phase, and the two experiments were run until physical and chemical equilibrium conditions were achieved. Chemical equilibrium was determined by regularly measuring EC of the effluent until effluent EC was stable and within 95% of the influent EC.

Table 1. Seepage-induced consolidation tests conducted with varying hydration and seepage solutions.

Test	Hydration Solution	Seepage Solution
T1	Tap	Tap
T2	0.1 M $CaCl_2$	0.1 M $CaCl_2$
T3	1.0 M $CaCl_2$	1.0 M $CaCl_2$
T4	Tap	1.0 M $CaCl_2$
T5	Tap	1.0 M $CaCl_2$

The other two experiments conducted for this study included kaolin slurry prepared via hydration in either the 0.1 M or 1.0 M $CaCl_2$ solutions and then the same solutions were used during seepage (T2 and T3). In these experiments, all plumbing lines between the pump and specimen were saturated with the same salt solution. The SICTs were run normally in these cases such that only physical equilibrium (pore pressure and vertical displacement) was evaluated for steady-state.

4 RESULTS

The *e*-σ' relationships for the three experiments conducted with consistent hydration and seepage solutions (T1, T2, and T3) are shown in Fig. 2a. Preparation of kaolin specimens via hydrating in varying strength saline solution resulted in different e_0 (Fig. 2a), which is shown as *e* plotted on the y-axis of Fig. 2. The addition of salt decreased e_0, and the 1.0 M $CaCl_2$ yielded the lowest e_0 and densest specimen after sedimentation. The *e*-σ' relationships for the experiments that included kaolin hydrated with tap water and subsequently seeped with the 0.1 M and 1.0 M $CaCl_2$ solutions are shown in Fig. 2b. The three kaolin specimens that were hydrated identically in tap water yielded nearly identical e_0 (Fig. 2b).

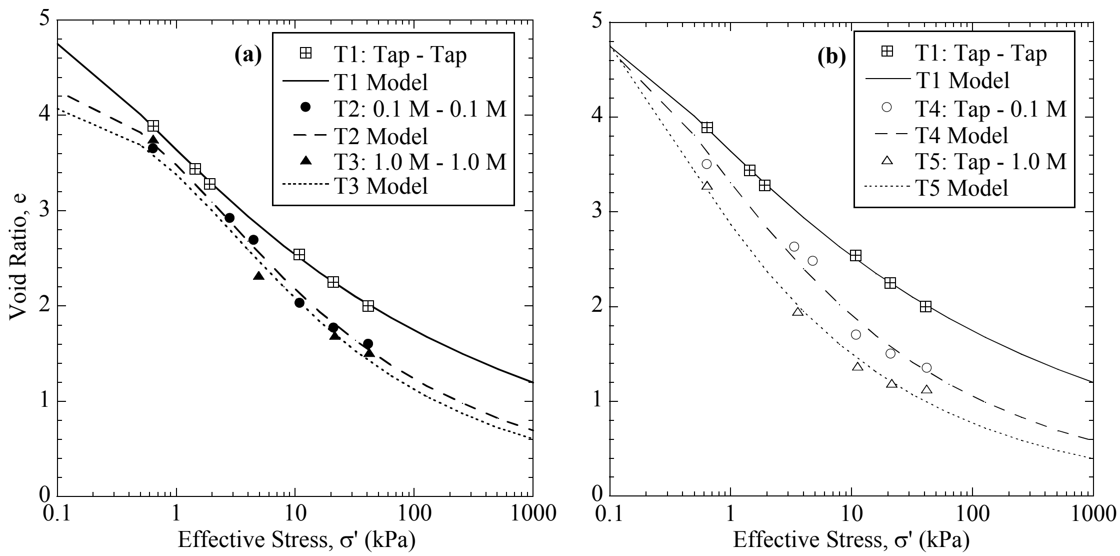

Figure 2. Relationships of void ratio versus effective stress for (a) kaolin specimens that included the same hydration and seepage solutions (T1, T2, and T3) and (b) kaolin specimens that included tap water as the hydration solution and 0.1 M or 1.0 M $CaCl_2$ as the seepage solution.

The *e*-σ' relationships for all SICTs shown in Fig. 2 display characteristic consolidation behavior, whereby *e* decreased with increasing σ'. The experiments conducted in consistent hydration and seepage solutions (Fig. 2a) show similar consolidation behavior, but lower overall *e* for a given σ' for the $CaCl_2$ solutions (T2 and T3) compared to tap water (T1). Thus, the addition of a $CaCl_2$ solution to slurried kaolin followed by subsequent seepage and vertical loading increased consolidation. In a similar manner, introducing salinity to kaolin during the seepage phase also led to increased consolidation (Fig. 2b). The data shown in Fig. 2b were for experiments initially hydrated in tap water and then seeped with the three different solutions (Table 1). An increase in salinity of the $CaCl_2$ solution from 0.1 M to 1.0 M during the seepage phase led to increased consolidation in the kaolin specimens. Thus, introducing salinity to kaolin following initial slurry sedimentation can also lead to increased consolidation.

The *e*-σ' relationships from Fig. 2 are reproduced in Fig. 3 to compare the effects of hydration and seepage on consolidation for similarity in salinity of the salt solutions. The *e*-σ' relationships for the pure tap water (T1), pure 0.1 M $CaCl_2$ solution (T2), and the specimen hydrated in tap water and seeped with 0.1 M $CaCl_2$ solution are shown in Fig. 3a, whereas the corresponding relationships for the 1.0 M $CaCl_2$ solution are shown in Fig. 3b. The general observation between the two $CaCl_2$ solutions are similar in that higher consolidation settlement was observed for kaolin specimens prepared in tap water and subsequently seeped with the salt solutions.

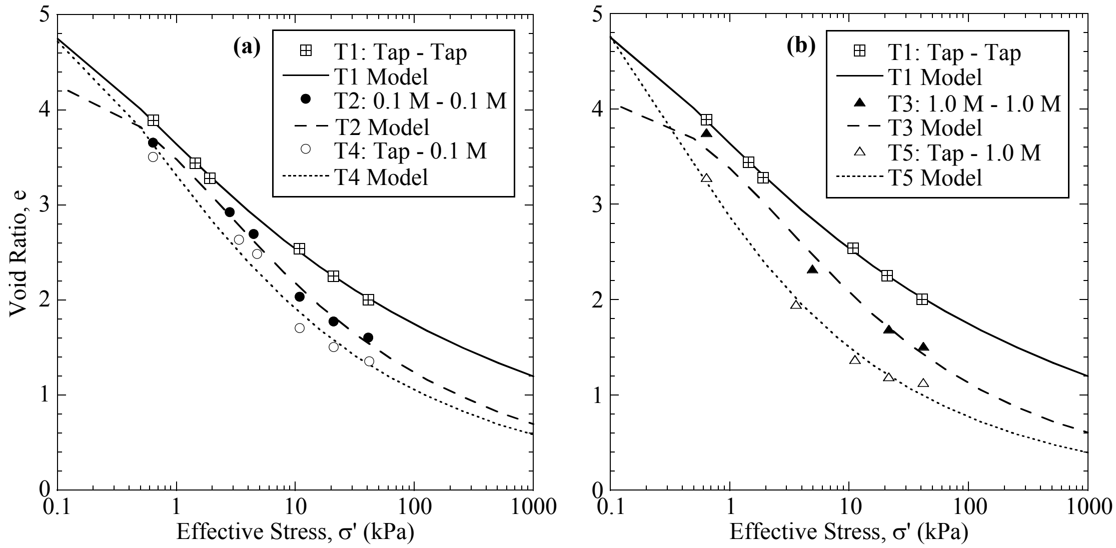

Figure 3. Relationships of void ratio versus effective stress for (a) kaolin specimens that included 0.1 M $CaCl_2$ solutions (T2 and T4) and (b) kaolin specimens that included 1.0 M $CaCl_2$ solutions. The experiments with tap water used for the hydration and seepage solution are included for comparison.

At low salt concentration (i.e., in tap water), dispersed kaolin forms aggregates (Yang and Barbour 1992) comprising edge-to-face flocculated clay platelets, commonly described as a "house of cards fabric" (Lambe and Whitman 1969). Thus, initially at low σ', the tap-water prepared kaolin had high *e* (i.e., T1, T4, and T5 in Fig. 3). As additional σ' was added, the aggregates compressed together, which reduced interaggregate pores (Yang and Barbour 1992) and decreased *e*. In contrast, kaolin dispersed in high concentration solutions (e.g., 1.0 M $CaCl_2$) has an increased tendency towards formation of aggregates comprising face-to-face platelet stacks, analogous to a house of cards constructed from small decks of cards. At the initial low σ', the kaolin had lower *e* (Fig. 4) due to the increased density of the card-deck tactoid (assemblages of clay platelets) comprising the kaolin aggregates. As σ' increased, the 0.1 and 1.0 M $CaCl_2$ prepared aggregates were more stable than the aggregates initially prepared in tap water, which led to more resistant aggregated structures for kaolin prepared in salt solutions that were more resistant to densification.

The lowest σ' data points shown in Fig. 3 for each test correspond to the first applied seepage phase. These data points were generated via seepage with tap water or a $CaCl_2$ solution to chemical equilibrium. The kaolin specimens that were prepared with tap water but then permeated with $CaCl_2$ solution simultaneously underwent chemical consolidation during seepage-induced consolidation (Yang and Barbour 1992) resulting in decreased *e* (Fig. 3). Chemical consolidation resulted in lower *e* compared to kaolin initially prepared with $CaCl_2$ at low σ'. This phenomenon was attributed to a greater sensitivity of aggregates comprising a house-of-cards fabric to chemical consolidation than the more stable aggregates of salt-equilibrated house of card-deck fabric under the same applied stress.

A compilation of e-k_s and k_s-σ' relationships for all five SICTs conducted for this study is shown in Fig. 4. The general trend in the e-k_s and k_s-σ' relationships is similar for all SICTs and k_s ranged between approximately 10^{-7} m/s and 10^{-9} m/s for σ' ranging from 1 to 40 kPa. Hydraulic conductivity was approximately the same for a given value of *e* regardless of pore fluid salinity (Fig. 4a). However, the k_s-σ' relationships illustrate that increasing salinity of the pore fluid reduced k_s for a given σ'. This observation can be explained via the *e*-σ' relationships (e.g., Fig. 2), whereby an increase in pore fluid salinity increased consolidation and yielded lower *e*.

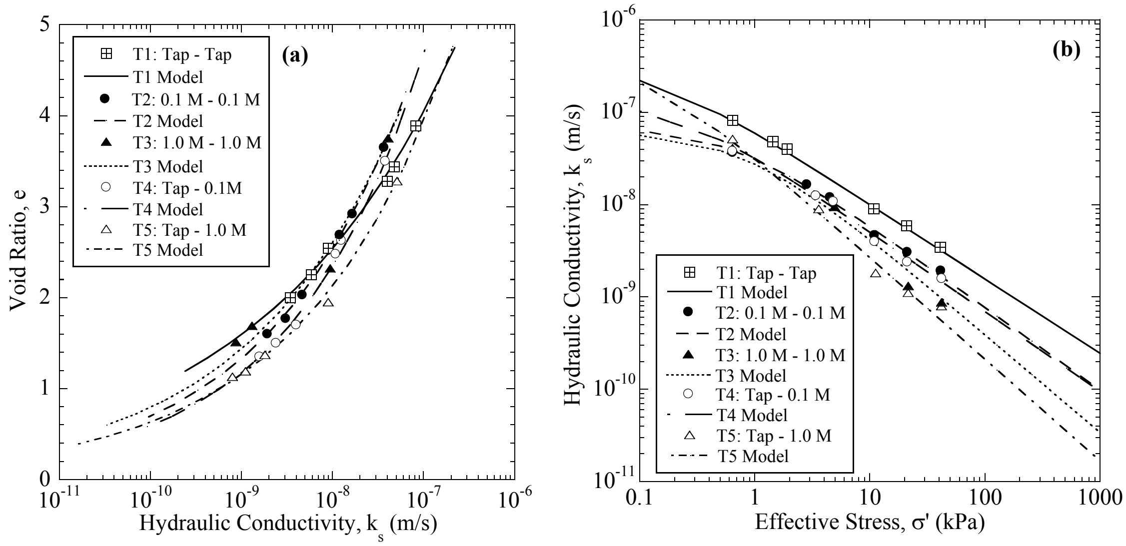

Figure 4. Relationships of (a) void ratio versus hydraulic conductivity and (b) hydraulic conductivity versus effective stress for all seepage-induced consolidation tests conducted in this study.

5 PRACTICLE IMPLICATIONS

To replicate field conditions, pore fluid chemistry in laboratory consolidation tests must match process water. Otherwise compressibility and permeability behavior may not be representative of what will occur in the field. In addition, data shown herein illustrate that laboratory consolidation tests should also mimic any anticipated changes in pore fluid chemistry that may occur during operation of a TSF.

The data herein support that the addition of salt to the tailings stream may result in improved dewatering. However, this does not account for potential negative affects arising from increased salt concentration, such as environmental concerns or damage to liners. Addition of salt after tailings sedimentation may also be more effective at maximizing the consolidation in a TSF. However, the increase of mine tailings salinity following sedimentation in a TSF to enhance consolidation requires seepage through the mine tailings to distribute salinity and induce chemical consolidation.

6 CONCLUSIONS

The effect of salinity on the compressibility and permeability of kaolin clay was evaluated using a seepage-induced consolidation test (SICT). The following observations and conclusions were drawn from this study.

- An increase in pore fluid salt concentration increased consolidation settlement for specimens hydrated in tap water, 0.1 M $CaCl_2$, or 1.0 M $CaCl_2$ that were tested with the same chemical solution during seepage. These experiments illustrated the effect of increasing salt concentrations on consolidation settlement of kaolin.
- Specimens that were hydrated with tap water and subsequently subjected to 0.1 M and 1.0 M $CaCl_2$ solutions also exhibited increased consolidation settlement relative to the pure tap water specimen. These specimens also exhibited larger consolidation settlement when compared to specimens initial hydrated in a comparable strength $CaCl_2$ solution. Higher consolidation settlement in the tap-water prepared specimens that were subsequently seeped with salt solutions was attributed to an initial flocculated clay structure that allowed for larger chemical-induced consolidation.
- Pore fluid salt concentration was shown to have limited influence on hydraulic conductivity at a given void ratio. However, hydraulic conductivity decreased for a given level of ef-

fective stress with increasing salt concentration. This phenomenon was attributed to enhanced consolidation with increasing salt concentration that yielded lower void ratios (i.e., denser kaolin specimens) for a given applied effective stress.
- Addition of $CaCl_2$ increases kaolin consolidation. However, pore fluid chemistry in laboratory consolidation tests should match field conditions. Any anticipated changes in pore fluid chemistry that may occur in the field should also be included in testing.

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Insights into rainfall-induced tailings run-out

M.A. Llano-Serna & D.J. Williams
The University of Queensland, Brisbane, QLD, Australia

L.C. Hernandez
University of Brasilia, Brasilia, DF, Brazil

ABSTRACT: Globally, there are on average at least two tailings dam failures annually, resulting in loss of life, damage to infrastructure and environmental impact downstream. Despite the importance of being able to postulate tailings dam-break and run-out, available guidelines specific to tailings dams are outdated and unreliable. The more established guidelines are based on approaches developed for water storage dams. However, released tailings will behave very differently from released water. Further, there is a lack of tailings run-out data available with which to validate and calibrate analysis methods. High rainfall in January 2016, following the failure of the Fundão tailings dam at Samarco in Brazil in November 2015, caused the mobilisation of an estimated further 7 Mm^3 of tailings from the Fundão storage. Rainfall data for this period are available, and the regression and mobilisation of the tailings were captured on video. This paper focuses on the use of a particle-based approach called the Material Point Method (MPM) to simulate this rainfall-induced tailings run-out event. One of the main advantages of the MPM is that it allows the use of conventional geotechnical constitutive models to solve large deformation problems, as experienced during tailings run-out. The MPM framework is used in this study to provide insight into the kinematics of tailings regression and run-out, based on the rainfall-induced event at Samarco. The MPM-based methodology proposed shows considerable promise. The validation exercise involved in this study aims to develop further the methodology to enable its wider application to the simulation and prediction of tailings run-out.

1 INTRODUCTION

Recent years have seen a number of tailings dam failures in developed countries and involving global mining companies that have attracted much adverse publicity globally. In particular, these include tailings dam failures at Mount Polley, Canada in 2014, and at Samarco, Brazil in 2015. Other recent tailings damn failures include Luoyang, China in 2016, and southern Judean Desert, Israel in 2017.

Azam & Li (2010) found that the annual failure rate for major tailings dams over the last one hundred years is more than two orders of magnitude greater than the annual failure rate for conventional large water retention dams. Robertson (2011) suggested that the risk of tailings dam failures is increasing by a factor of 20 every third of a century. Therefore, there are significant benefits, and clear international industry and community needs, for a significant improvement in the capability to simulate and predict tailings dam failures and tailings run-out. Emphasis should be placed on the prevention of tailings dam failures rather than reacting after a catastrophe, and on anticipating the risk of a tailings dam failure, in order to minimise the potential impact and cost of tailings dam failures and tailings run-out (Rico *et al.* 2008).

Conventional tailings dam-break and run-out analyses follow the steps: (i) selection of the dam-break mechanism, (ii) estimation of the volume of tailings released, (ii) review of downstream flows and the potential for tailings transport, (iii) routing the water and tailings downstream, and (iv) sensitivity analyses to gain confidence in the results. So-called "sunny-day" (caused by dam failure) and "rainy-day" (flood-induced) failures are generally both considered (CAD, 2007 revised 2013). The reliable simulation and prediction of tailings run-out requires an ability to model the kinematics of a mudflow, to delineate the area and elevation that would potentially be affected. This is required to allow the potential human, infrastructure and environmental losses to be quantified.

A report of dam failures worldwide by Rico *et al.* (2008) highlighted that 25% of incidents are related to meteorological causes such as unusual rainfall events or periods of snow. Tailings run-out on dam failure is beyond the current capacity to reliably predict, but is desperately needed. This study focuses on the mobilisation of an estimated further 7 Mm^3 of tailings from the failed Fundão storage in Brazil following high rainfall in January 2016.

2 OVERVIEW OF CASE STUDY

The Fundão tailings dam failed in November 2015, in a flow slide initiated at the dam's left abutment. The subsequent run-out of tailings extended a distance of 500 km downstream of the storage, killed 19 people, and produced incalculable damage. The detailed technical report on the Fundão tailings dam failure by Morgenstern *et al.* (2016) employed several forensic methods to describe the technical causes of the catastrophe. The failure was described as the consequence of a chain of events and conditions. These included changes to the original dam design, and the encroachment of slimes into the sand dam within the requirement for a 200 m sand beach. Design changes included a setback of the left abutment, which exacerbated the encroachment of slimes into the sand dam. At several times well prior to the failure, piping, seepage, cracking and slumping were observed on the downstream face of the dam.

Subsequent to the failure of the Fundão tailings dam in November 2015, high rainfall during January 2016 resulted in the mobilisation on 27 January 2016 of an estimated further 7 Mm^3 of tailings from the remaining Fundão storage. Figure 1 shows the location of the Colégio Caraça weather station (20°05′49″S, 43°29′17″W, 1300 MASL) 13 km to the north of the Fundão Dam, which is used as a reference in this study. Figure 2 shows the daily precipitation recorded at the Colégio Caraça weather station from 1 December 2015 through to the mobilisation of further tailings 27 January 2017.

Figure 1. Location of Colégio Caraça weather station relative to Fundão Dam (source: Google Earth Pro®, July 2016).

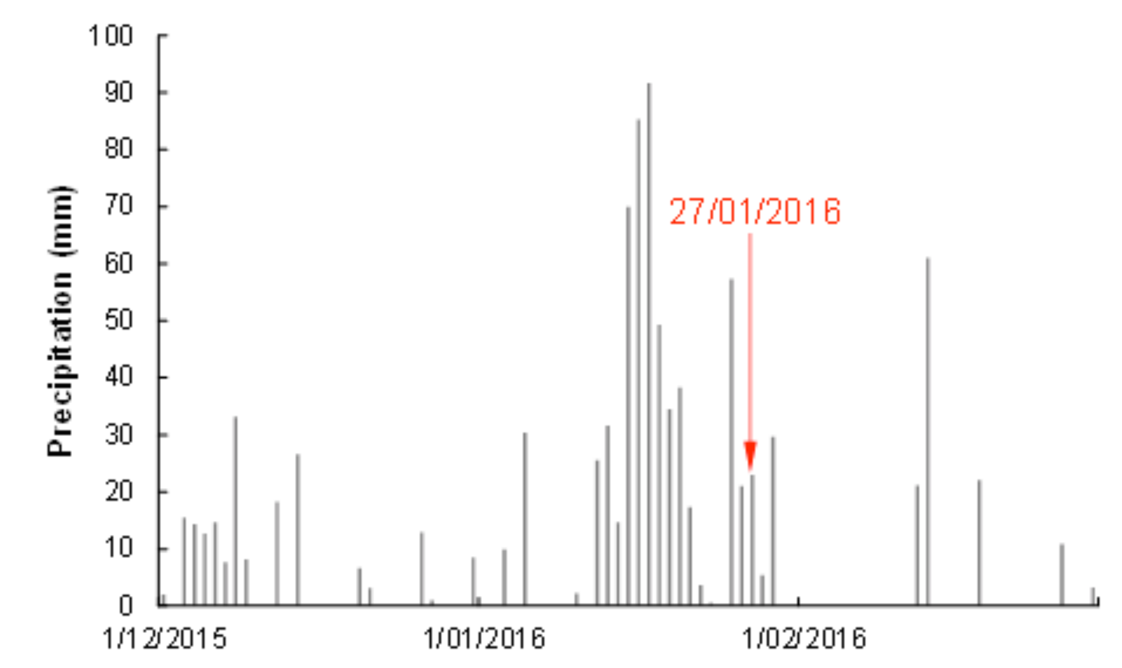

Figure 2. Daily precipitation recorded at Colégio Caraça weather station (modified from ANA, 2016).

During January 2016, almost 570 mm of cumulative rainfall was recorded prior to the mobilisation of further tailings from the remaining Fundão storage. The wet season in the Fundão region usually starts in October and goes until March, with historical monthly rainfall totals of around 500 mm (DFBRD, 2017). In January 2016, this monthly total was recorded in only 16 days.

It is interesting to note that the mobilisation of tailings occurred 10 days after the peak daily rainfall of 92 mm shown in Figure 2. Coincidently, 9 days before the November 2015 failure of the Fundão tailings dam a significant rainfall event occurred, as reported in Appendix K of the report by Morgenstern *et al.* (2016).

Conventionally, high rainfalls are considered in a rainy-day failure scenario, with emphasis on overtopping due to the storage capacity of a tailings dam being exceeded. The case study considered herein highlights the need for a predictive tool capable of modelling the stresses induced by increased pore water pressures and the potential for run-out under extreme rainfall events.

3 RUN-OUT MODELLING USING MATERIAL POINT METHOD

The modelling of a run-out process may be classified numerically as a large deformation problem. The complexity of the phenomena lies in the extreme deformations that occur during run-out, causing compression and localised failure due to induced stresses reaching upper limits. These kind of problems remain largely unsolved to date. Recent advances in computational capacity allow the use of particle-based methods to tackle these problems. Popular choices include the discrete element method (DEM, Cundall & Strack, 1979), smoothed particle hydrodynamics (SPH, Gingold & Monaghan, 1977), and the material point method (MPM, Sulsky *et al.* 1994).

The MPM can be understood as an advance on the widely known and widely used finite element method (FEM). When using finite elements, the stiff relationship between Gauss points and the mesh limit the modelling capabilities to small strains. The basic principle of the MPM is shown in Figure 3. The material points carry the state variables and material parameters, while the mesh is used for calculation purposes only, where the constitutive relationships are solved. Note that the material points represent a portion of the discretised volume rather than a soil grain, in contrast to the DEM.

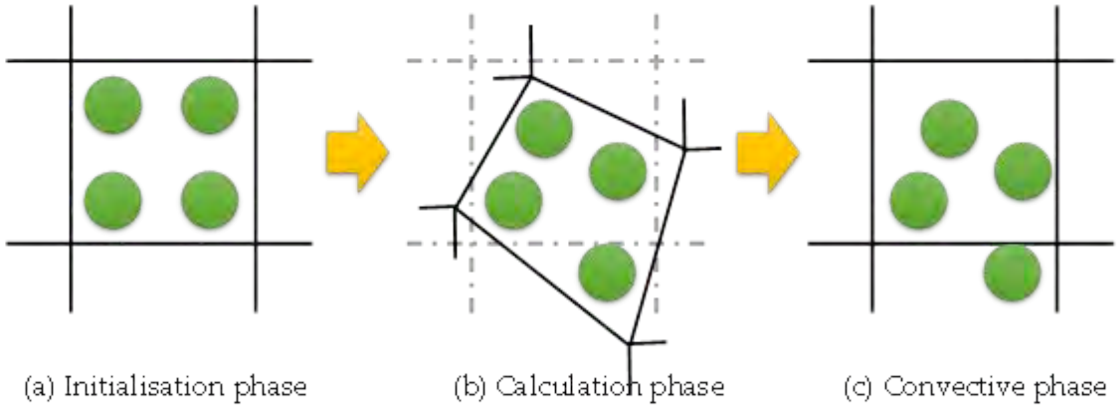

Figure 3. Calculation process during MPM.

At the beginning of each computational step, relevant information from the continuum is transferred to the computational grid (see Figure 3a). The governing equations of motion are solved and the nodal values updated based on the calculated strains and stresses (see Figure 3b). In the final computational step, the background grid is reset to its initial configuration. The material points keep their updated positions, the deformed grid is discarded, and the iterative process may be continued (see Figure 3c).

3.1 *Numerical model*

The geometry of the numerical model is shown in Figure 4. The soft tailings mass is placed above a stiffer medium represented by a surface with a smooth contact. The frictional effects between the run-out tailings and the sliding surface are disregarded herein. The material parameters were defined with the information reported by Morgenstern *et al.* (2016).

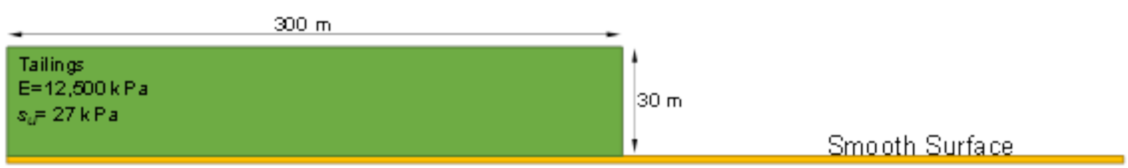

Figure 4. Geometry and material parameters adopted in numerical model.

4 RESULTS

The boundary condition problem was solved using the dynamic explicit code Anura3D. Figure 5 shows the time lapse of the run-out, expressed in terms of the incremental pore water pressure calculated during run-out. It is noted that the calculated pore water pressures were highest at the advancing toe of the run-out. In the early stages (t<105 s), the kinematics of the process seem to remain stable. However, in the later stages, the accumulated pore water pressures increase at a higher rate, producing the "splash-like" behaviour seen in the recorded videos of the run-out. Also, between t=105 s and 140 s, the continuous loss of confinement generates a series of "shock waves", resulting in increased pore water pressures back from the toe of the run-out, implying a decrease of the mean effective stress.

The arrows in Figure 5 show the advance of the cone of depression, which is produced by the continuous regression of the crown of the tailings deposit as the continuous failure occurs at the toe of the run-out. Similar behaviour has modelled and described by Llano-Serna *et al.* (2017). It is expected that as the model is run for longer periods of time, the cone of depression would attain angles of less than 5° (Martin *et al.* 2015).

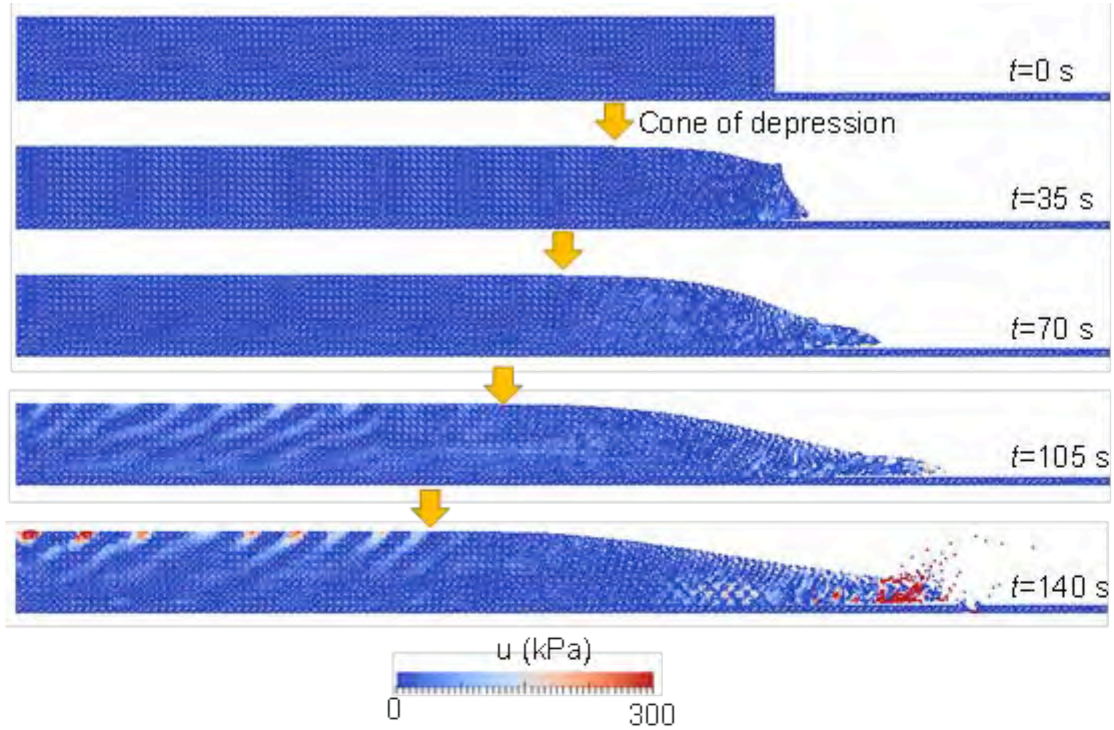

Figure 5. Time progression of run-out analysis, represented by increments of pore water pressure during run-out.

5 CONCLUSIONS

This paper presents the application of the MPM to describe the kinematics of run-out processes involved with tailings dam failures. The observed mobilisation of an estimated further 7 Mm^3 of tailings from the failed Fundão storage in Brazil following high rainfall in January 2016 motivated the study. Tailings dam incidents related to meteorological causes such as unusually high rainfall highlight the need for a numerical tool able to describe the kinematic behaviour of tailings run-out produced by increased pore water pressures. The present study is a step forward in this direction.

To implement a single tool to model rainfall, seepage, increasing pore water pressures and run-out is challenging. The main reasons for this are the accelerated time scales over which tailings run-out occurs, and the enormous deformations involved. Whereas rainfall may accumulate over many hours, days or even weeks, the onset of run-out takes only hundreds of seconds. The explicit nature of the MPM adopted herein presents limitations. Longer periods of time imply extensive computational cost. However, the results are promising. Further development of the modelling currently underway is expected to further extend its capabilities, with view to its more extensive and widespread application.

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Rock Drain Seepage and Related Water Quality Impacts in Rose Creek, Faro Mine Complex

Paul Ferguson & Christoph Wels
Robertson GeoConsultants Inc., Vancouver, BC, Canada

Carrie Gillis & Dustin Rainey
Assessment and Abandoned Mines, Government of Yukon, Whitehorse, YT, Canada

ABSTRACT: In 2013, dissolved (zinc) Zn concentrations in the North Fork of Rose Creek (NFRC) increased substantially due to increased seepage loads from the Sulphide Cell within the nearby Intermediate Waste Rock Dump (WRD). Loads from the Sulphide Cell appeared to have increased due to higher recharge rates in 2013 due to natural climatic variability. This paper describes Zn loads and concentrations in the NFRC before and after recharge rates increased in 2013, and evaluates the effectiveness of seepage interception measures in this area. Options for further reducing Zn loads to the NFRC are briefly described, as are other options being considered for site-wide closure.

1 INTRODUCTION

1.1 *Background*

The Faro Mine Complex (FMC), in Canada's Yukon Territory, was one of the largest lead (Pb) and zinc (Zn) producers in the world. Mining ceased there in the 1990s and the site has since been under care and maintenance by the Government of Yukon (YG). Groundwater and surface water at the FMC is impacted by Acid Rock Drainage (ARD) from ~70 million tonnes (t) of sulphidic tailings in the Rose Creek Tailings Facility (RCTF) and ~320 million t of waste rock in the Waste Rock Dumps (WRDs), including the Intermediate WRD (see faromine.ca).

The Intermediate WRD is located south of the Faro Pit near the North Fork of Rose Creek (NFRC) (Figure 1). Seepage from the Intermediate WRD reports to groundwater and is routinely captured by the S-Cluster Seepage Interception System (SIS) before it reaches the NFRC. The S-Cluster SIS consists of several pumping wells and a shallow interceptor trench and has been operated near continuously since 2009 (see SRK, 2011; RGC, 2014).

Seepage from the Intermediate WRD also reports to the NFRC upstream of the S-Cluster SIS via the so-called 'rock drain'. The rock drain is located beneath a haul road that was built over the pre-mining course of the NFRC. In 2013, dissolved Zn concentrations in the NFRC increased to more than 1 mg/L Zn. Peak concentrations occurred during winter low flow conditions that year. Synoptic flow and water quality surveys in November 2013 indicated that the high Zn concentrations were related to increased seepage loads to the NFRC from the rock drain (see RGC, 2015).

Seepage to the rock drain likely originates from the Sulphide Cell within the Intermediate WRD. This type of seepage was identified upgradient of the S-Cluster SIS during drilling and has been intercepted by pumping well PW14-06 since January 2016. If not intercepted, this seepage would report to the S-Cluster SIS (see RGC, 2017). Seepage recovery from pumping well PW14-06 is therefore opportunistic, and is not done to reduce Zn concentrations in the NFRC. The North Fork Rock Drain (NFRD) SIS has operated immediately downstream of the rock drain since 2015 (see BGC, 2015).

Figure 1. Faro Mine Site (FMC) Layout.

1.2 *Study Objectives*

In 2017, RGC reviewed groundwater quality near the Intermediate WRD and surface water quality conditions in the NFRC between stations NF2 and X2 in the S-Cluster Area (Figure 2). Objectives were to:

- Determine whether the Zn loads that bypass the S-Cluster SIS via groundwater affect Zn concentrations in the NFRC.
- Determine whether the interception of Zn loads by the NFRD SIS and/or pumping well PW14-06 have improved Zn concentrations in the NFRC.
- Provide recommendations on how to further reduce Zn loads to the NFRC.

Monitoring data collected between January 2010 to March 2017 were reviewed to achieve these objectives. This period was selected because it includes a period when only the S-Cluster SIS was operated (January 2010 to March 2015) and periods in 2015 and 2016 when the NFRD SIS and/or PW14-06 were also being operated. Flows and water quality in the NFRC at station NF5 are also reviewed. This station was established in October 2015 after Zn concentrations in the NFRC had decreased.

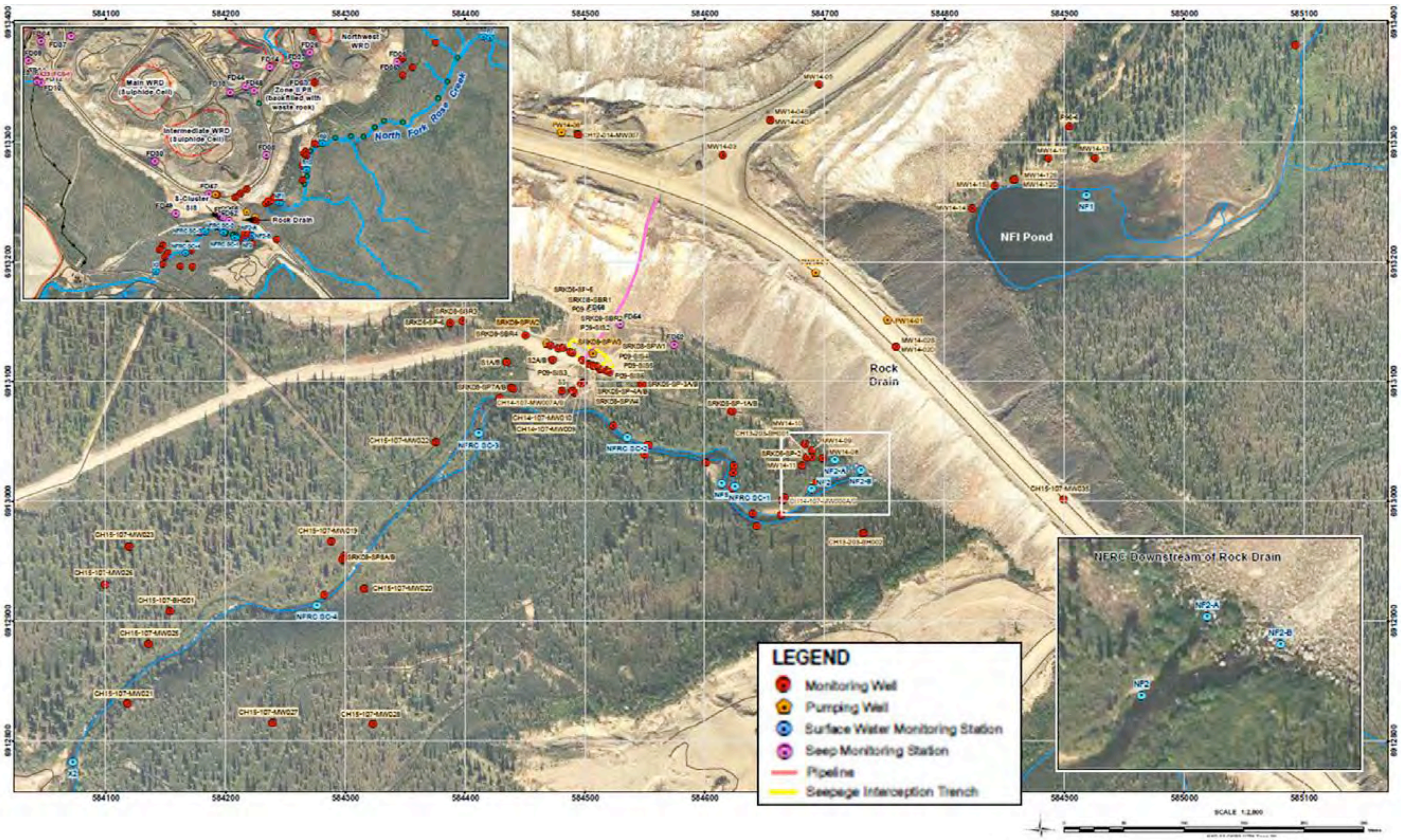

Figure 2. Groundwater Monitoring Wells and Surface Water Monitoring Stations near the Rock Drain and S-Cluster SIS

2 METHOD AND APPROACH

Surface water quality is routinely monitored at numerous locations along the NFRC. Flow measurements and water quality observations (i.e. SO_4 and dissolved Zn) for the following routine monitoring stations along the NFRC were compiled for this study:

- NF1. NFRC upstream of the NFRD near the NF1 pond.
- NF2A. NFRC downstream of the 'rock drain' (northern creek bank)
- NF2B. NFRC downstream of the 'rock drain' (southern creek bank).
- NF2. NFRC downstream of NF2A and NF2B (northern creek bank).
- NF5. NFRC upstream of S-Cluster Area (near NFRC SC-1).
- X2. NFRC downstream of the S-Cluster Area near the mine access road.

Flows in the NFRC are routinely monitored at NF2 and X2 with staff gauges. NF2 water represents combined flows from NF2A and NF2B. Flows from NF2A and NF2B are not well-mixed at NF2 due to an island between the stations that separates NFRC flows into two discrete streams (see Figure 2 inset). This is consistent with findings from RGC (2015), wherein Zn concentrations varied by several orders of magnitude at different locations within the NFRC channel at NF2. Station NF5 was established downstream of NF2 where the NFRC is thought to be thoroughly mixed (see EDI, 2017). Flows in the NFRC at X2 represent NF2 flows and additional flows of groundwater to the NFRC within the S-Cluster Area.

Flows and concentrations in the NFRC were used to estimate SO_4 and Zn loads in kg/day. To estimate loads intercepted in the S-Cluster Area, RGC compiled monthly flows from the S-Cluster SIS via pumping well SRK08-SPW3 (the central sump), pumping well PW14-06, and the NFRD SIS. These flows are recorded with totalizers (in m^3) as part of routine monitoring at the FMC.

3 RESULTS

Selected flows and concentrations of SO4 and Zn for the period January 2010 to March 2017 are shown in Figures 2, 3, and 4 and summarized in Tables 1, 2, and 3. These data, and other supporting data (e.g. intercepted Zn loads, flows and seepage water quality at X23, etc.), are discussed below.

Table 1. Selected/Representative Loads in the NFRC at NF2, 2010 to 2017

Flow Condition	Date	NFRC at NF2				
		Flow, L/s	[SO_4], mg/L	[Zn], mg/L	SO_4 Load, kg/d	Zn Load, kg/d
2009/2010 Water Year						
Low Flow	14-Apr-10	163	21	0.017	296	0.2
High Flow	8-Jul-10	1,556	9	0.005	1,210	0.6
2010/2011 Water Year						
Low Flow	13-Apr-11	240	17	0.022	353	0.4
High Flow	20-Jun-11	3,042	7	0.005	1,814	1.3
2011/2012 Water Year						
Moderate	14-Nov-11	856	20	0.016	1,509	1.1
High	5-Sep-12	1,372	16	0.006	1,884	0.7
2012/2013 Water Year						
Low Flow	12-Dec-12	570	19	0.011	911	0.5
Low Flow	9-Jan-13	441	20	0.023	762	0.9
Low Flow	5-Feb-13	356	19	0.018	587	0.5
Low Flow	6-Mar-13	450	20	0.015	759	0.6
Low Flow	4-Apr-13	200	22	0.012	375	0.2
Low Flow	8-May-13	435	20	0.067	740	2.5
2013/2014 Water Year						
High Flow	10-Jul-13	1,682	13	0.032	1,948	4.6
High Flow	6-Aug-13	1,233	15	0.082	1,609	8.7
High Flow	12-Sep-13	2,548	10	0.047	2,268	10.4
High Flow	2-Oct-13	1,739	15	0.086	2,314	13.0
Low Flow	4-Dec-13	751	26	0.58	1,693	37.7
Low Flow	8-Jan-14	311	34	1.22	921	32.8
Low Flow	5-Feb-14	550	33	0.88	1,581	41.8
Low Flow	5-Mar-14	472	42	1.11	1726	45.3
Low Flow	2-Apr-14	260	37	1.26	833	28.3
2014/2015 Water Year						
High Flow*	7-Jul-14	2,030	11	0.15	1,982	26.4
High Flow	17-Jul-14	1,296	19	0.77	2,116	86.2
High Flow*	5-Aug-14	1368	-	-	-	-
High Flow*	20-Aug-14	1205	-	-	-	-
High Flow*	22-Sep-14	2737	-	-	-	-
Low Flow	12-Feb-15	118	40	0.95	409	9.7
2015/2016 Water Year						
High Flow+	6-Oct-15	2526	16	0.10	3,516	21.8
Low Flow	8-Feb-16	554	25	0.015	1,188	0.7
High Flow	18-May-16	3,021	5	0.12	1,376	31.1
2016/2017 Water Year						
Low Flow	14-Feb-17	322	29	0.15	806	4.2

* Survey completed by EDI

+ Flows measured during CH2M Hill's lithium dye tracer test

Table 2. Loads Intercepted in the Rock Drain and S-Cluster

Month	S-Cluster SIS (via SRK08-SPW3)					Pumping Well PW14-06					NFRD SIS (at NF2A)				
	Flow, L/s	$[SO_4]$, mg/L	[Zn], mg/L	SO_4 Load, t/month	Zn Load, t/month	Flow, L/s	$[SO_4]$, mg/L	[Zn], mg/L	SO_4 Load, t/month	Zn Load, t/month	Flow, L/s	$[SO_4]$, mg/L	[Zn], mg/L	SO_4 Load, t/month	Zn Load, t/month
July-15	1.7	11,200	731	51	3.3	-	-	-	-	-	-	-	-	-	-
August-15	1.4	11,500	803	43	3.0	-	-	-	-	-	-	-	-	-	-
September-15	1.7	10,400	755	45	3.3	-	-	-	-	-	-	-	-	-	-
October-15	1.7	10,500	663	47	3.0	-	-	-	-	-	-	-	-	-	-
November-15	1.5	8,720	544	34	2.1	-	-	-	-	-	-	-	-	-	-
December-15	1.6	10,000	1,000	42	4.2	-	-	-	-	-	-	-	-	-	-
January-16	1.3	12,067	863	30	2.2	0.2	18,733	1,897	7	0.7	5.0	*15*	*0.5*	0.1	0.00
February-16	1.3	11,575	867	38	2.8	0.2	18,025	1,667	9	0.9	5.3	*15*	*0.5*	0.2	0.01
March-16	1.1	11,002	876	32	2.6	0.2	16,640	1,655	8	0.8	5.5	*15*	*0.5*	0.2	0.01
April-16	1.4	4,899	603	18	2.3	0.1	18,133	1,695	7	0.6	-	-	-	-	-
May-16	1.6	12,233	710	52	3.0	0.1	19,200	1,846	3	0.3	-	-	-	-	-
June-16	1.5	11,750	641	45	2.4	0.2	17,450	1,635	8	0.8	-	-	-	-	-
Average Flow or Concentration	**1.5**	**10,487**	**755**	**-**	**-**	**0.2**	**18,030**	**1732**			**5.3**	**15**	**0.5**		
Total Load Intercepted, 2015/2016	**-**	**-**	**-**	**477**	**34.2**	**-**	**-**	**-**	**42**	**4.1**	**-**	**-**	**-**	**0.6**	**0.02**
% Contribution	**-**	**-**	**-**	**91.8%**	**89.3%**	**-**	**-**	**-**	**8.1%**	**10.6%**	**-**	**-**	**-**	**1.3%**	**0.5%**
July-16	1.4	11,700	840	43	3.1	0.2	17,975	1,682	9	0.8	-	14	0.2	-	-
August-16	1.5	10,540	1,010	40	3.9	0.2	16,680	1,634	7	0.7	-	12	0.1	-	-
September-16	1.1	9,000	642	26	1.8	0.1	17,300	1,680	3	0.3	-	15	0.2	-	-
October-16	1.0	11,800	706	30	1.8	0.2	*17,817*	*1,712*	*8*	*0.7*	-	22	0.3	-	-
November-16	1.1	11,300	677	31	1.9	0.2	*17,817*	*1,712*	*8*	*0.8*	-	24	0.4	-	-
December-16	1.0	9,530	607	24	1.6	0.2	18,000	1,570	8	0.7	3.3	35	0.9	0.3	0.01
January-17	*1.2*	*10,645*	*747*	*32*	*2.3*	*0.2*	*17,598*	*1,665*	*7*	*0.7*	5.3	47	2.1	0.7	0.03
Average Flow or Concentration	**1.2**	**10,645**	**747**	**-**	**-**	**0.2**	**17,598**	**1665**	**-**	**-**	**4.3**	**24**	**0.6**	**-**	**-**
Total Load Intercepted, July 2016 to January 2017	**-**	**-**	**-**	**228**	**16.3**				**50**	**4.8**				**1.0**	**0.04**
% Contribution	**-**	**-**	**-**	**81.6%**	**77.3%**	**-**	**-**	**-**	**18.1%**	**22.6%**	**-**	**-**	**-**	**0.3%**	**0.2%**
Total Load (projected), 2016/2017	**-**	**-**	**-**	**456**	**32.7**	**-**	**-**	**-**	**101**	**9.5**	**-**	**-**	**-**	**2**	**0.1**

Notes:

SO_4 and Zn concentrations are average values from weekly monitoring data. Flows (in L/s) correspond to monthly flows from totalizers.

Water quality for October and November 2016 were not available, so the SO_4 and Zn concentrations for these months (italicized) are the average values for January to September, 2016

SO_4 and Zn concentrations at NF2A were not available for the months of January, February, or March 2016. Concentrations for these months (italicized) are the average values from July to December, 2016

Table 3. Selected/Representative Loads in the NFRC at X2, 2010 to 2017

Flow Condition	Date	NFRC at X2				
		Flow, L/s	$[SO_4]$, mg/L	[Zn], mg/L	SO_4 Load, kg/d	Zn Load, kg/d
2009/2010 Water Year						
Low Flow	14-Apr-10	207	28	0.031	500	0.6
High Flow	8-Jul-10	1,538	15	0.023	1,993	3.1
2010/2011 Water Year						
Low Flow	13-Apr-11	138	26	0.038	311	0.5
High Flow	20-Jun-11	3,472	9	0.008	2,610	2.3
2011/2012 Water Year						
Moderate	20-Oct-11	667	26	0.015	1,487	0.9
High	5-Sep-12	1,442	12	0.008	1,495	1.0
2012/2013 Water Year						
Low Flow	12-Dec-12	732	25	0.015	1,599	1.0
Low Flow	8-Jan-13	335	25	0.042	713	1.2
Low Flow	5-Feb-13	399	24	0.016	842	0.6
Low Flow	6-Mar-13	300	26	0.017	671	0.4
Low Flow	4-Apr-13	176	29	0.019	439	0.3
Low Flow	8-May-13	414	25	0.056	884	2.0
2013/2014 Water Year						
High Flow	10-Jul-13	1,701	12	0.016	1,778	2.3
High Flow	6-Aug-13	1,070	15	0.028	1,423	2.6
High Flow	12-Sep-13	-	12	0.058	-	-
High Flow	2-Oct-13	1,663	15	0.087	2,097	12.4
Low Flow	4-Dec-13	604	28	0.44	1,460	22.8
Low Flow	8-Jan-14	354	33	0.71	1,009	21.7
Low Flow	5-Feb-14	302	42	0.94	1,091	24.6
Low Flow	5-Mar-14	571	45	1.05	2202	51.8
Low Flow	2-Apr-14	362	46	1.13	1440	35.4
2014/2015 Water Year						
High Flow*	7-Jul-14	1,700	13	0.17	1,836	25.0
High Flow	17-Jul-14	1,455	14	0.24	1,798	29.8
High Flow*	5-Aug-14	990	-	-	-	-
High Flow*	20-Aug-14	1213	-	-	-	-
Low Flow	12-Feb-15	154	41	0.95	550	12.6
2015/2016 Water Year						
High Flow+	6-Oct-15	2745	17.3	0.10	4,103	23.7
Low Flow	8-Feb-16	-	33	0.27	-	-
High Flow	17-May-16	-	6	0.04	-	-
2016/2017 Water Year						
Low Flow	6-Feb-17	315	35	0.36	941	9.8

* Survey completed by EDI

+ Flows measured during CH2M Hill's lithium dye tracer test

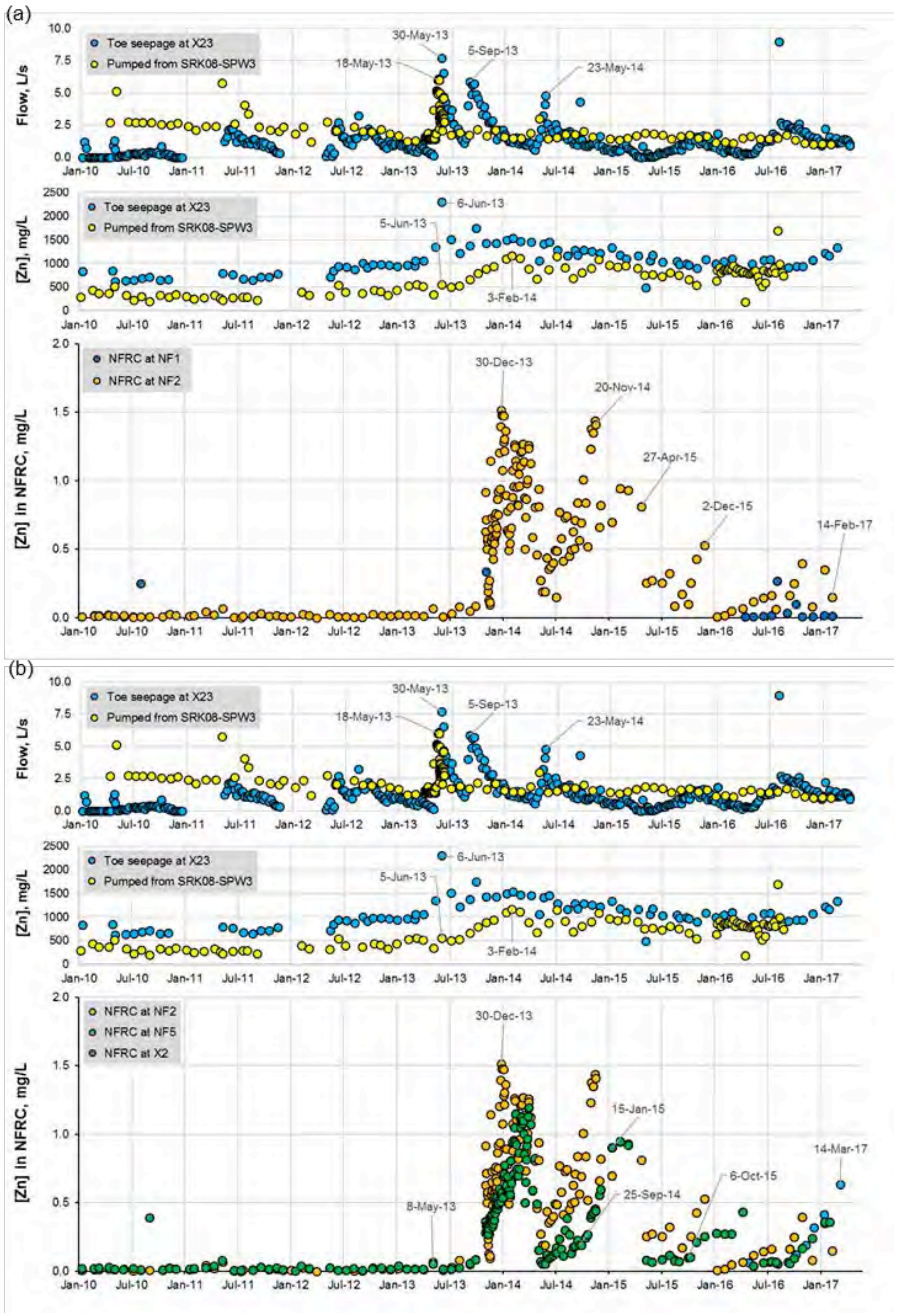

Figure 3. (a) Zn concentrations in the NFRC at NF1 and NF2 and (b) Zn concentrations in the NFRC at NF2, NF5, and X2. Zn concentrations and flows in toe seepage from Main WRD at X23 and flows from the S-Cluster SIS at SRK08-SPW3 also shown.

4 DISCUSSION

4.1 *Zn Concentrations and Loads in the NFRC at NF2*

At NF2, Zn concentrations were highest in the NFRC during low flow conditions in 2013/2014 (see Figure 3a). Zn concentrations first increased during the 2013/2014 water year[1] and peak Zn loads of 30 to 40 kg/day Zn were observed from December 2013 to July 2014 (see Table 1). The highest Zn load at NF2 was about 86 kg/day on July 17th, 2014. However, this large load is considered an outlier, and is attributed to an unrepresentative Zn concentration at NF2 on this day (due to incomplete mixing). More reliable flow estimates from Environmental Dynamics Inc. (EDI) (EDI, 2014) suggest ~26 kg/day Zn in July 2014 (see Table 1)[2].

Since 2013/2014, the highest Zn concentration observed during each subsequent winter low flow period has decreased, i.e. to 0.15 mg/L Zn in February 2017, and Zn loads in the NFRC at NF2 have decreased by an order-of-magnitude. RGC (2015) attributes the high Zn loads in the NFRC at NF2 in 2013/2014 to unusually large seepage loads from the Intermediate WRD (Sulphide Cell) to the rock drain reach. RGC (2015) estimated that 25 to 30 kg/day Zn reported to the NFRC between NF1 and NF2 from the "rock drain seep".

The large Zn load to the NFRC in the rock drain area in 2013/2014 was likely caused by an increase in seepage rates from the Intermediate WRD (Sulphide Cell) and/or higher source term concentrations due to increased recharge to the Intermediate WRD by infiltrating rainfall and snowmelt. Increased recharge and associated contaminant "flushing" in 2013/2014 is supported by the following observations during the same period (see Figure 3):

- Higher seepage flows to the S-Cluster SIS from the Intermediate WRD.
- Higher flows of toe seepage from the Main WRD at X23.
- Elevated Zn concentrations in toe seepage from the Main WRD at X23.

Also, precipitation records for the Faro Airport show that the amount of precipitation (mainly rainfall) that occurred from June to October 2013 was 30% higher than the long-term average for this period. Together, these observations suggest larger seepage flows from the Intermediate WRD (Sulphide Cell) and/or higher Zn concentrations in that seepage due to the flushing of Zn and other ARD-related constituents in 2013/2014.

The 2013/2014 flushing event also appears to have caused a plume of more impacted groundwater to be transported from the toe of the Intermediate WRD to the S-Cluster SIS over a period of eight months in late 2014 and early 2015 (see Zn concentrations at SRK08-SPW3 in Figure 3). This groundwater was transported in the shallow sub-surface between the toe of the Intermediate WRD and the S-Cluster SIS, hence the increase in Zn at SRK08-SPW3 over weeks and months as the plume arrives.

Since 2013/2014, Zn loads to the NFRC have gradually decreased due to the occurrence of more typical climatic conditions (and lower recharge to the WRDs) and/or lower dissolved Zn concentrations in seepage from the Intermediate WRD (Sulphide Cell) since the large-scale flushing of ARD products in 2013/2014. Lower Zn loads in the NFRC since 2013/2014 could be related to one (or both) of the following:

- Natural variation in the magnitude of seepage flows from the Sulphide Cell and/or the Zn concentration in this seepage.
- Lateral movement of seep(s) beneath the rock drain due to changes in the local flow field and/or the re-establishment of a previous, preferential flowpath that prevails during periods of lower recharge.

The first explanation implies that less concentrated seepage from the Sulphide Cell has continued to affect NFRC water quality since peak loads occurred in 2013/2014. The second explanation implies that the high Zn load from the Intermediate WRD (Sulphide Cell) that occurred in 2013/2014 has persisted in subsequent years but no longer reports to the rock drain or the NFRC. In RGC's opinion, the first explanation (i.e. a reduction in Zn load from the Sulphide Cell since 2013) is more plausible than a change in the direction of seepage flow.

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[1] A water year is defined here as the period from July 1st to June 30th of the following year

[2] Flows from EDI (2014) for July are based on multiple velocity measurements across the NFRC during the Rose Creek Monitoring Program (see RGC, 2015, for further details).

The reduction in load in the NFRC since 2013/2014 is likely caused by natural variations in WRD seepage due to climatic variability and is therefore unrelated to operations and/or seepage interception at the site. Zn concentrations in the NFRC could therefore increase again if seepage rates from the Sulphide Cell and/or source term concentrations were to increase. Should this occur, pumping concentrated seepage upgradient of the rock drain would likely be required to reduce loads to the NFRC and decrease Zn concentrations.

4.1.1 *Intercepted Zn Loads in the S-Cluster Reach*

The following seepage interception systems are presently operated in the rock drain and S-Cluster reach:

- S-Cluster SIS (see SRK, 2010; RGC, 2014). Consists of a shallow interceptor trench and pumping well SRK08-SPW2. Seepage is delivered to a central sump and pumped from the SIS by SRK08- SPW3. The S-Cluster SIS has been operated near-continuously since February 2009.
- Pumping well PW14-06 (see RGC, 2017). Located upgradient of the S-Cluster SIS near the haul road. This well is screened from 51.1 to 61.8 m bgs across the interface of waste rock, shallow overburden, and weathered bedrock. Pumping well PW14-06 has been pumped continuously since January 2016.
- NFRD SIS (see BGC, 2015). Constructed in March 2015 to capture impacted seepage flows from the rock drain at NF2A. It was operated for three months in 2015 and four months in 2016 (January to March, 2016, and December 2016).

Table 2 summarizes SO_4 and Zn loads intercepted by the S-Cluster SIS, pumping well PW14-06, and the NFRD SIS in 2015/2016 and during the first six months of the 2016/2017 water year. Some key observations from Table 2 (and Figure 3) are summarized below:

- The S-Cluster SIS intercepted ~34 t of Zn from groundwater in 2015/2016. The large Zn load intercepted by the S-Cluster SIS is related to a consistent flow (~1.4 L/s) of ML/ARD-impacted groundwater from the SIS that contains ~10,000 mg/L SO_4 and up to ~750 mg/L Zn.
- Together, the S-Cluster SIS and pumping well PW14-06 intercepted ~21 t Zn during the first six months of 2016/2017. ~77% of this Zn load was intercepted by the S-Cluster SIS. This percentage is consistent with RGC (2017) and will likely be representative of the entire 2016/2017 water year if the S-Cluster SIS and pumping well PW14-06 are both operated continuously. In 2015/2016, the S-Cluster SIS accounted for ~90% of the captured load because operation of pumping well PW14-06 only began in the middle of the 2015/2016 water year.
- Pumping well PW14-06 captured smaller flows of more highly-impacted seepage that corresponded to ~20% of the Zn loads captured by the S-Cluster SIS. Seepage captured by PW14-06 likely originates from the Intermediate WRD (Sulphide Cell within the Intermediate WRD) and flows to this well via a natural drainage channel that follows pre-mining topography (see RGC, 2015).
- The loads intercepted by pumping well PW14-06 are more likely to report to the S-Cluster SIS than to the NFRC (see RGC, 2015). Operating this pumping well may therefore reduce bypass to groundwater downgradient of the S-Cluster (at X2) but is unlikely to improve NFRC water quality conditions in the rock drain reach. Insufficient monitoring data at NF2 and X2 is available since start of operation of pumping well PW14-06 in 2016 to confirm this contention.
- The NFRD SIS captures relatively large flows of surface water; however, the captured flow typically contains less than 1 mg/L Zn. When operated in 2016, flows from the NFRD SIS accounted for more than two-thirds of the monthly volume of water intercepted in the S-Cluster reach but less than 0.5% of the intercepted Zn load.

4.2 *Zn Loads to NFRC between NF2 and X2*

Routine monitoring data suggest that Zn concentrations in the NFRC at X2 are typically lower than they are in the NFRC at NF2 (Figure 3). This discrepancy is evident throughout the year and is most likely explained by incomplete mixing in the NFRC at NF2. This is consistent with detailed profile sampling across the NFRC at NF2 by EDI in 2014, during which the west bank of the NFRC was biased high due to flow from NF2A (see EDI, 2014).

A less likely explanation for lower Zn concentrations at X2 would be discharge of unimpacted groundwater, i.e. dilution, along this reach. However, groundwater in this reach tends to be more impacted than stream water and, even if unimpacted, the groundwater discharge rate providing dilution would have to be substantial (i.e. 25 to 50% of streamflow). Flow rates this high are implausible based on the hydraulic properties of the aquifer in this area and have not been observed in streamflow measurements.

In 2016, an additional monitoring station NF5 was introduced downstream of NF2 by YG to provide a more representative (i.e. better mixed) sampling station in the NFRC between the toe of the rock drain and the S-Cluster SIS. Unfortunately, NF5 is not sampled on the same day as NF2 and X2 so water quality data and/or loads at these stations could not be directly compared. Detailed synoptic surveys (including dye tracer test) were completed by CH2MHill across the reach between NF2 and X2 (including stations at SC2, SC3, and SC4) during high flow conditions in October 2015. This survey (data not provided) shows no significant increase in Zn concentrations, nor Zn load in the reach between NF2 and X2. These data, which are considered more reliable than routine monitoring data, imply that Zn loads that bypass the S-Cluster SIS are too small to noticeably increase the Zn concentration in the NFRC at X2 during high flow periods. This is consistent with findings from RGC (2014).

It should be noted that recent Zn trends in NFRC (Figure 3) indicate a noticeable increase in Zn concentrations between stations NF2 and X2 during winter baseflow conditions (Jan-Mar) for the last three years (2015-2017)[3]. However, this increase is primarily due to an abrupt decline in Zn concentrations at NF2 rather than an increase at X2. This is likely artefact of non-representative sampling at station NF2 during mid-winter conditions when it is difficult to locate the sampling location through ice and snow. This explanation is consistent with recent sampling completed during the winter of 2016/2017 at the new sampling station NF5.

4.3 *S-Cluster SIS Bypass*

In 2015/2016, the monthly load intercepted by the S-Cluster SIS ranged from 2.1 to 4.2 t/month and it was typically about 3 t/month (see Table 2). A Zn load of 3 t/month corresponds to ~100 kg/day. Groundwater quality observations and hydraulic performance monitoring suggest that some S-Cluster SIS bypass does occur (see RGC, 2014). The magnitude of the flows and loads that bypass the S-Cluster SIS are not, however, well-constrained, nor is it clear whether bypassing flows and loads report to the NFRC at X2 or to NFRC downstream of X2.

Previous experience suggests that 5 to 10% S-Cluster SIS bypass is plausible. These percentages imply that 5 to 10 kg/day Zn could bypass the S-Cluster SIS towards the NFRC. The limited data available for station NF5 (and the data for NF2 that are considered reliable) suggest that loads of this magnitude do not report to the NFRC between these stations and station X2. Moreover, most of the groundwater downgradient of the S-Cluster SIS is characterized by less than 1 mg/L Zn (see RGC, 2017). This concentration implies ~60 to 120 L/s of bypass to the NFRC. These flows seem unreasonably high for bypass, and lower flows would imply higher Zn concentrations that are not widely-observed downgradient of the S-Cluster SIS. Together, these observations suggest that S-Cluster SIS bypass is likely less than 5 to 10 kg/day Zn or that Zn has been retarded and/or attenuated along the flow path in the aquifer between the S-Cluster SIS and the NFRC. Future NFRC monitoring at NF5 and X2 on the same day is needed to resolve this issue and further constrain S-Cluster SIS bypass.

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[3] A notable exception is the field survey taken on January 19th, 2017, which did not show any significant increase in zinc concentrations and/or zinc load in the reach between NF2 and X2. Additional synoptic monitoring at NF5 and X2 will be required to further clarify the factors controlling zinc loading in the S-Cluster reach.

4.4 *Zn Loads and Concentrations in the NFRC at NF5 and X2*

Zn concentrations at X2 peaked during winter low flow conditions in 2013/2014 as they did at NF2 (Figure 3). At station X2, the NFRC is well-mixed (unlike at NF2), and the Zn loads in the NFRC accurately represent the Zn load from the rock drain and a relatively small, additional Zn load from S-Cluster SIS bypass. In 2014/2015, 25 to 30 kg/day Zn was observed in the NFRC at X2 in July 2014. The only higher Zn load since 2010 was observed during low flow conditions in March 2014 (Table 3).

Loads of 25 to 30 kg/day Zn in 2014/2015 are considered more representative of peak Zn loads in the NFRC than the loads in March 2014 because these loads were confirmed by flow data collected by EDI in July 2014 and by CH2M Hill in October 2015. These loads are also comparable to loads calculated from routine monitoring data in late 2013.

During low flow conditions in February 2017, a load of ~10 kg/day Zn in the NFRC at X2 was observed. ~10 kg/day Zn was also observed in the NFRC at NF5 during low flow conditions in March 2017. These data suggest that ~10 kg/day Zn is likely representative of the load to the NFRC from the rock drain seep during recent (and more typical) low flow conditions.

In February 2017, 0.36 mg/L Zn was observed in the NFRC at X2. This concentration is about an order-of-magnitude higher than the CCME guideline for Zn (0.03 mg/L Zn). In March 2017, 0.64 mg/L Zn was observed in the NFRC at NF5 due to lower flows in the creek (and therefore less dilution of the ~10 kg/day Zn load).

During low flow conditions, Zn loads to the NFRC from the rock drain seep would have to be reduced to less than ~1 kg/day Zn by intercepting seepage upgradient of the rock drain. Our analysis suggests that improved seepage recovery by the S-Cluster SIS and/or pumping well PW14-06 would not significantly improve NFRC water quality.

Preliminary load balance calculations suggest that 10 to 25 kg/d Zn could be intercepted upgradient of the rock drain by pumping ~1 to 3 L/s of groundwater containing 100 mg/L Zn near the NF1 pond. This concentration was observed at well MW14-14 in September 2015 and is five time higher than the Zn concentration initially observed in 2014 (see RGC, 2015). Considering the potential for intercepting relatively small flows of highly-impacted groundwater, pumping wells near well MW14-14 are being considered.

5 KEY FINDINGS

Key findings from this study are summarized as follows:

- Zn loads (and concentrations) in the NFRC at NF2 and X2 are approximately equivalent during high flow conditions. This implies that the Zn load bypassing the S-Cluster SIS and discharging to NFRC during these periods is relatively small in comparison to loads in the NFRC discharging beneath the rock drain. Therefore, Zn concentrations in the NFRC at X2 and NF5 (where the creek is well-mixed) primarily reflect seepage loads from the rock drain and it is at these stations that future flows and water quality should be monitored to detect changes in the seepage load from the rock drain.
- Zn loads to the NFRC from the rock drain seep are estimated to range from 10 to 25 kg/d Zn. The higher load was observed in 2013/2014 and 2014/2015 when seepage loads from the Sulphide Cell to the rock drain increased. This increase was likely caused by increased recharge to the Intermediate WRD in 2013/2014 due to climatic conditions that year. Changes in Zn in the NFRC at X2 therefore reflect natural variability in flow rates and/or source term concentrations for the rock drain seep originating from the Intermediate WRD (Sulphide Cell).
- Together, the S-Cluster SIS and pumping well PW14-06 likely intercept about 40 to 45 t/year Zn (or 110 to 123 kg/day Zn) from groundwater reporting from the Intermediate WRD to the NFRC. The S-Cluster SIS accounts for 80% of this load and the other 20% is intercepted by pumping well PW14-06. Pumping well PW14-06 is inferred to reduce loads to the S-Cluster SIS. It is not, however, believed to reduce loads that report to the NFRC in the rock drain area. Turning off this well would likely not affect NFRC water quality.
- The NFRD SIS intercepts only a very small Zn load upstream of NF2 at NF2A. The system has not operated continuously since it was constructed in early 2015 and, when it has oper-

ated, has intercepted relatively large flows of surface water that contains less than 0.5 mg/L Zn. In 2015/2016, the NFRD SIS intercepted less than 0.5% of the combined Zn load intercepted by this system and the S-Cluster SIS and pumping well PW14-06.
- Intercepting 1 to 3 L/s of highly-impacted groundwater upgradient of the rock drain (near the NF1 pond) could potentially reduce Zn concentrations to the NFRC by 10 to 25 kg/d Zn. This load reduction could decrease Zn loads in the NFRC at NF5 (and X2) to less than 1 kg/d Zn and may allow CCME guidelines for Zn to be achieved in this reach of NFRC during the climate/recharge conditions that were observed at the FMC since 2010.

6 PATH FORWARD

A site-wide closure plan for the FMC is being developed by YG and Indian and Northern Affairs Canada (INAC) as part of the Northern Contaminated Sites Program. The closure plan will address water quality conditions in the NFRC, amongst other issues that pertain to the physical and geochemical stability of tailings and waste rock, and will be submitted in 2018.

During closure planning, YG will (i) monitor NFRC flows and water quality conditions at NF5 and X2 and (ii) evaluate the feasibility of additional seepage interception upgradient of the rock drain along the western perimeter of the NF1 pond. These activities are being undertaken to support the selection of a preferred approach to achieving CCME guidelines for the NFRC, which could include additional seepage interception and/or raising and lining the creek bed. For further details, see faromine.ca and www.aadnc-aandc.gc.ca.

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Investigation of the role of hardpans on the geochemical behavior of the Joutel mine tailings

Abdellatif Elghali, Mostafa Benzaazoua & Bruno Bussière
Université du Québec en Abitibi-Témiscamingue (UQAT), Rouyn-Noranda, QC, Canada

Dirk Schaumann
Fibics Incorporated, Ottawa, ON, Canada

Shaun Graham
ZEISS Natural Resources Lab, Cambridge, MA, USA

Thomas Genty, Josée Noel, Chris Kenedy & Jean Cayouette
Agnico Eagle Mines Limited, Rouyn-Noranda, QC, Canada

ABSTRACT: This paper focuses on the mineralogical and geochemical characterization of oxidized, unweathered tailings and hardpans formed in the Joutel Eagle-Telbel mine tailings impoundment (Québec). To highlight geochemical behavior of oxidized and unweathered tailings, oxidized and non-oxidized tailings were sampled, analyzed and subjected to kinetic testing. Two scenarios were tested: oxidized tailings alone to simulate surface runoff due to presence of hardpans, and oxidized tailings above unweathered tailings to simulate vertical infiltration. The mineralogical characterization of hardpan samples revealed a composition of 46% iron oxy-hydroxides, 19% siderite, 4% pyrite and non-sulphide gangue minerals. The 11-months column test results showed that the leachate from the oxidized sample were consistently acidic (pH 2.44); the average concentrations were 781 mg/l S, 550 mg/l Ca, 31 mg/l Mg, 106 mg/l Fe and 11 mg/l Zn. However, the leachates provided by vertical infiltration scenario were consistently neutral (pH = 7.3), with average species concentrations of 1653 mg/l S, 523 mg/l Ca, 817 mg/l Mg, 0.88 mg/l Fe and 0.67 mg/l Zn.

1 INTRODUCTION

Sulphides within mine tailings can oxidize and generate acidity, sulphates and metals leaching. When carbonates are also present in the tailings, they neutralize, through dissolution reactions, the acidic tailings pore water. These oxidation/neutralization and hydrolysis mechanisms cause the precipitation of various secondary phases. In some specific cases, massive precipitation of secondary phases leads to the formation of "cemented" layers of precipitated minerals called hardpans (Blowes *et al.*, 1991; Blowes *et al.*, 1998; McGregor *et al.*, 2002; Pérez-López *et al.*, 2007).

Hardpans are naturally formed beneath tailings surface and their mineralogy is variable depending on the initial chemical and mineralogical composition of tailings. Hardpans may occur as continuous or discontinued layers in tailings storage areas, typically contained primary minerals, poorly crystalized phases, amorphous ferric oxides, metal sulphates, etc. (Lottermoser *et al.*, 2006). Hardpans are usually formed in the vadose zone where there is a geochemical contrast of pH, Eh and chemical concentrations between oxidized and unweathered tailings. Hardpans formation may be accelerated using alkaline amendments to reactive tailings (Pérez-López *et al.*, 2007). Precipitation of secondary minerals reduces the porosity of the initial tailings. These cemented layers are compact and impermeable compared to the initial tailings materials, which influences water and oxygen fluxes (Blowes *et al.*, 1991). The geochemical behavior of tailings impoundments is then influenced by various mechanisms related to the hardpans: i) precipitation of secondary phases decrease contaminant concentrations by co-precipitation and adsorption mechanisms, ii) protection of the underlying tailings by inhibiting oxygen diffusion, and iii) deviation of water fluxes. Deflecting the vertical infiltration water flow to surface runoff isolates the unweathered tailings.

Joutel Eagle-Telbel is a closed mine site located north of Abitibi-Témiscamingue (Québec). Joutel's tailings were characterized as "uncertain" regarding potential for acid generation. Results of kinetic tests on unweathered and oxidized tailings confirmed that their long-term environmental behavior is uncertain for most samples, and acid generating for some others (Benzaazoua *et al.*, 2004). The results confirm a high spatial variability of chemical and mineralogical tailings characteristics. Recently and in some specific zones of the impoundment, acidity has been observed in the supernatant waters. However, the final effluent shows a neutral behavior. To explain this phenomenon, various materials consisting of oxidized and unweathered were sampled, characterized and submitted to laboratory kinetic leaching tests. Leaching tests consisted on two scenarios: i) a column with only the oxidized tailings to simulate surface and sub-surface runoff due to presence of hardpan and ii) a column with oxidized tailings above unweathered tailings to simulate vertical infiltration. The project hypothesis is that the hardpan when it exists deflects the water vertical infiltration to surface runoff.

2 MATERIALS AND METHODS

2.1 Materials sampling

Samples used in this project were collected in acidic zones based on previously determined paste pH. Samples were collected from different vertical horizons (i.e. oxidized, hardpan and unweathered tailings) in a trench dug out in the selected zones (Figure 1). A composite sample was made for each horizon based on samples collected on different locations. The samples were homogenized in the laboratory using quartering.

Figure 1: Tailings sampling and horizons

2.2 Physical, chemical and mineralogical analysis

The tailings grain size distribution was evaluated using a laser analyzer (Malvern Masetrsizer S). Specific gravity (Gs) was determined using micromeritics Helium Pycnometer, and the specific surface area (SSA) was evaluated with a micromeritics analyzer using B.E.T method.

The bulk chemical composition of the samples was analyzed using Perkin Elmer Optima 3100 RL ICP-AES following a total HNO3/Br2/HF/HCl digestion. The total sulfur and inorganic carbon contents were determined by using induction furnace (ELTRA CS-2000). The chemical analysis of the leachates samples from column leaching tests were done using pH, Eh and conductivity meter. The elemental composition was determined using ICP-AES on acidified samples (2% HNO_3). The acidity and alkalinity were evaluated using automatic titror.

Major minerals within oxidized and unweathered tailings were analyzed by X-ray diffraction (XRD; Brucker D8 Advance, with a detection limit and precision of approximately 0.5 %, operating with a copper cathode, Kα radiation) using DIFFRACT.EVA software and quantified using TOPAS v4.2.

Hardpan samples were analyzed using automated mineralogy system and light microscopy. The objectives of this characterization were to determine the mineralogical composition, mineralogical association and stoichiometry of minerals forming hardpan sample.

A large-area light-microscopy image mosaic of one hardpan sample was acquired with coaxial reflected light using a Zeiss AXIO Zoom.V16. Large-area SEM image mosaics of a polished 30 mm diameter epoxy mount of a hardpan sample was acquired with the ZEISS Atlas 5 software by using a ZEISS EVO MA 15 tungsten filament SEM at Fibics Incorporated (Ottawa, Canada). The light-microscopy image mosaic was imported in the Atlas 5 correlative workspace and aligned with the sample in the SEM chamber. The data set was exported to an autonomous series of files called the Browser-Based Viewer. The Browser-Based Viewer dataset of the hardpan sample can be viewed at the following link:
http://www.petapixelproject.com/mosaics/UQAT/Hardpan-03-BBV-Export-LM-BSD/Hardpan-03-BBV-Export-LM-BSD/index.html.

Element distribution maps and point analyses of several regions of interest were acquired with the Bruker Esprit 1.9 software on the Zeiss EVO MA 15 SEM at 8.5 mm working distance and an acceleration voltage of 20 kV. The acquired element maps and point analyses were exported from the Bruker Esprit software.

The automated mineralogy analysis was carried out at Fibics Incorporated (Ottawa, Canada) using Zeiss Mineralogic Mining on a ZEISS EVO MA 15 tungsten filament SEM equipped with two Bruker 6│30 Xflash energy dispersive x-ray spectrometers (EDS). Mineralogic Mining version 1.02 was used for this analysis with its fully quantitative EDS mineral classification protocol, where minerals are classified based on the weight percent contribution of elements present, and thus the mineral's stoichiometry. The full mapping analysis mode was used with an EDS analysis carried out every 4 µm of the sample surface with approximately 4000 counts per EDS spectrum for quantification with an electron beam dwell time of 0.07 seconds. The automated mineralogy analysis was acquired at 8.5 mm working distance, an acceleration voltage of 20 kV, and a probe current of 2.0 nA.

2.3 *Static and kinetic leaching tests*

Acid potential (AP) of oxidized and unweathered samples was calculated using sulfur sulphides which is the difference between total sulfur and sulphates. Neutralization potential (NP) was analyzed using titration method as described in (Bouzahzah *et al.*, 2015). The net neutralization potential (NNP) was defined as the difference between NP and AP; the neutralization potential ratio (NPR) as the ratio between NP and AP (Miller *et al.*, 1991).

Kinetic tests were done using Plexiglas columns of 14-cm diameter and 1-m height. Two scenarios were tested: the first consisted only of the oxidized sample, and the second consisted of an oxidized sample above unweathered sample. The column with only oxidized tailings was mounted to simulate surface runoff due to the presence of hardpans. The column with oxidized sample above unweathered sample was used to simulate vertical water infiltration in the absence of hardpans. These scenarios were chosen due to the difficulty to generate a hardpan artificially at the laboratory scale. The columns were monthly flushed with 2 L of deionized water and columns were allowed to dry under ambient air between two flushes. The leachate waters were collected and analyzed after 4 hours of contact with the samples.

3 RESULTS AND DISCUSSION

3.1. Results

3.1.1. Physical characterization

The physical characteristics of the samples studied were very variable (Table 1). The D60 and D90 represent the grain size for 60 and 90%, respectively, passing on the cumulative grain size distribution curve. The oxidized sample had a finer particle size compared to unweathered sample. The D60 and D90 for the oxidized and unweathered samples are respectively 13 µm, 88µm, 37 µm and 163 µm.

The specific surface area of oxidized sample was of about 38 g/m² while of only 0.7 g/m² for unweathered sample. High specific surface area lead to high exposure rate of minerals. The specific gravity (Gs) of unweathered sample (Gs=3.35) was greater than that of oxidized sample (Gs=2.83). The variation of physical parameters between oxidized and unweathered samples is mainly due to oxidation. In fact, oxidation when occurring reduces de particle size of minerals (sulphides, carbonates) and decreases the specific gravity of sample by depleting sulphides.

3.1.2. Chemical and mineralogical characterization

Chemical and mineralogical compositions are summarized in Table 1. The chemical composition of the solids was different for the three samples due to their different mineralogical composition. The concentration of sulfur increases from oxidized tailings (%S=3.9) to unweathered tailings (%S=6.39). The hardpan sample presents an intermediary concentration of sulfur (%S=5.42). This is explained by sulphides oxidation process. Sulfur from oxidized sample is oxidized and leached, and then precipitated as iron oxy-hydroxide and gypsum in the hardpan's zone. Hardpan's sulphides are coated and passivated by precipitation of these secondary minerals. Generally, coating of sulphides (Figure 2) reduces significantly oxygen diffusion (Blowes *et al.*, 1991).

Concentrations of magnesium in oxidized sample, hardpan and unweathered sample are respectively 0.045, 0.47 and 1.75%. This element is mostly linked to carbonates (dolomite, ankerite). Iron is slightly enriched in unweathered sample (25.78%) and hardpan (25.39%) comparatively to oxidized sample (20.17%). The oxidized sample is more enriched in Al (3.62%) followed by hardpan sample (2.61%) and then unweathered sample (1.5%). Calcium concentrations are similar in three horizons.

Mineralogical characterization of oxidized and unweathered samples was performed using X-ray diffraction and hardpan was characterized using Mineralogical Mining. Sulphides distribution presents an increasing of concentrations as a function of depth. The sulphides are mostly a reflection of pyrite content. Sulphides in the oxidized horizon are almost depleted due to 25 years of oxidation and the sulphides in the unweathered horizon are protected against oxidation by the water table.

Oxidation of sulphides within upper surface tailings favors carbonates dissolution. Therefore, carbonates distribution follows the same trend than sulphides distribution. Carbonates are thus almost depleted in the oxidized horizon while they are preserved in the unweathered horizon. In general, carbonates and sulphides in the hardpan are protected due to massive precipitation of secondary iron oxy-hydroxides (Figure 2).

Table 1: Physical, chemical and mineralogical compositions of studied samples

Physical characteristics				
	D60 (μm)	D90 (μm)	SSA (m²/g)	Gs
Oxidized	13.2	37.15	0.7	2.83
Unweathered	87.6	163.57	38.07	3.35

Chemical composition (%)								
	Al	Ca	Mn	Mg	Fe	S	Zn	Cu
Oxidized	3.62	3.82	0.12	0.045	20.17	3.90	0.010	<ND
Hardpan	2.61	3.26	0.39	0.47	25.39	5.42	0.011	0.003
Unweathered	1.5	3.53	0.74	1.75	25.78	6.39	0.011	<ND

Mineralogical composition (%)					
	Sulphides	Carbonates	Fe-oxi-hydroxides	Gypsum	Others
Oxidized	ND	1.76	23.74	19.3	55.2
Hardpan	4.51	19.65	45.78	0.26	29.8
Unweathered	7.40	42.15	<ND	<ND	50.45

Static tests					
	NP (titration)	NP (car-bonates)	AP	NNP	NPR
Oxidized	3	20	5.5	-2.5	0.5
Unweatherd	228	538	193	35	1.2

ND: detection limit

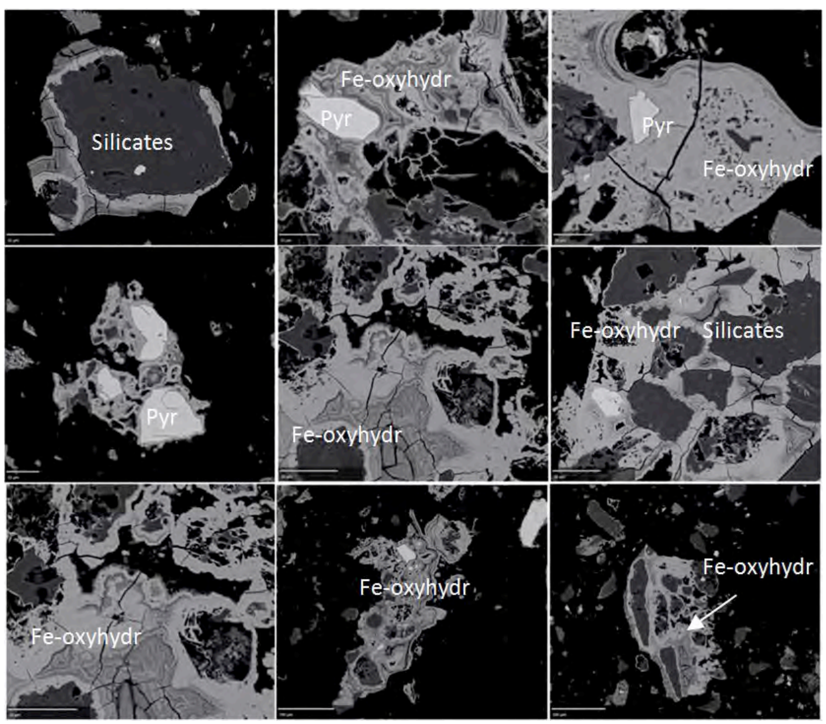

Figure 2: BSE images showing textures of secondary iron oxy-hydroxides within the hardpan sample (Fe-oxyhydr = iron oxy-hydroxide, pyr = pyrite)

Hardpans are formed in the tailings impoundment due to oxidation/neutralization and hydrolysis reactions. This layer is characterized by its low saturated hydraulic conductivity (McGregor *et al.*, 2002) due to the cementation of particles through the precipitation of secondary minerals reduces tailings porosity and the permeability. Therefore, the mineralogic characterization is crucial to understand hardpan formation and its effect on the geochemical and hydrogeological behaviour of tailings impoundments. The detailed mineralogical characterization of hardpan is presented in Table 2 based on the analysis of more than 70,000 particles.

Table 2: Mineralogical characterization of hardpan using Mineralogic Mining

Mineral	Formula	Weight %	Wt% Average Composition
Hematite	Fe_2O_3	32.23	Fe 68.54; O 31.46;
Goethite	FeOOH	2.97	Fe 60.17; O 39.83;
Goethite-Silicate	-	2.22	O 36.64; Fe 28.85; Si 21.23; Al 7.74; Na 4.96; K 0.31; Mg 0.26;
Other iron oxide	-	8.36	Fe 79.9; O 20.1;
Pyrite	FeS_2	3.97	Fe 51.02; S 48.98;
Pyrrhotite	FeS	0.54	Fe 77.82; S 22.18;
Dolomite	$CaMg(CO_3)_2$	0.09	O 50.8; Ca 29.82; Mg 11.69; Fe 7.67; Al 0.02;
Calcite	$CaCO_3$	0.01	Ca 50.81; O 47.59; Fe 1.6;
Ankerite	$Ca(Fe,Mg,Mn)(CO_3)_2$	0.52	O 44.36; Ca 26.52; Fe 20.05; Mg 6.19; Mn 2.88;
Siderite	$FeCO_3$	19.03	Fe 56.35; O 36.19; Mg 3.96; Mn 3.22; Al 0.28;
Gypsum	$CaSO_4.2(H_2O)$	0.26	O 39.41; Ca 37.87; S 22.72;
Quartz	SiO_2	12.45	Si 51.75; O 45.76; Fe 1.69; Al 0.75; K 0.05;
Albite	$NaAlSiO_3$	11.75	O 42.67; Si 33.37; Al 12.6; Na 9.55; Fe 1.75; K 0.04; Ca 0.02
Illite	$(K,H_3O)(Al,Mg,Fe)_2(Si,Al)_4O_{10}[(OH)_2,(H_2O)]$	1.62	O 44.69; Si 34.39; Al 14.09; Fe 3.82; K 2.96; Mg 0.03;
Orthoclase	$KAlSi_3O_8$	1.57	O 42.81; Si 29.35; Al 17.87; K 4.86; Na 2.95; Fe 2.17;
Biotite	$K(Mg,Fe)_3[AlSi_3O_{10}(OH,F)_2$	1.49	O 39.38; Si 18.76; Al 18.24; Fe 17.62; K 5.77; Mg 0.23;
Muscovite	$KAl2(Si_3Al)O_{10}(OH,F)_2$	0.51	O 42.97; Al 24.24; Si 23.53; K 7.27; Fe 1.68; Mg 0.31;
Chamosite	$(Fe^{2+},Mg,Fe^{3+})_5Al(Si_3Al)O_{10}(OH,O)_8$	0.31	O 39.1; Fe 32.85; Si 14.28; Al 10.23; Mg 3.24; K 0.3;
Rutile	TiO_2	0.10	O 51.72; Ti 48.28;

Iron oxy-hydroxides (Figure 2) are precipitated and form the cement matrix surrounding the grains. Most of the secondary iron phases present is hematite (Table 2). The mineralogical associations of iron oxy-hydroxides are presented in Figure 3. Hardpan iron oxy-hydroxides are present as liberated and mostly as binary association to other iron oxy-hydroxides. They are more associated to carbonates (8%) than sulphides (2%). This leads to passivation of sulphides and carbonates. Sulphides within hardpan sample are almost all associated with iron oxy-hydroxides (47%pyrite and 33% pyrrhotite). Generally, association of iron oxy-hydroxides with sulphides is known as sulphide's coating.

The results of the static tests on oxidized and unweathered samples are presented in Table 1. Neutralization potential (NP), AP and NNP are presented in kg of $CaCO_3$/t. Neutralization potential analyzed by titration is smaller than the value calculated by total inorganic carbon concentration. This is due to the presence of iron carbonates which reduces the real NP of tailings (Bouzahzah *et al.*, 2015). Neutralization potential of oxidized sample is lower than its AP which may lead to an acidic behavior of this sample. The unweathered tailings have 35 kg $CaCO_3$/t of difference between its NP and AP. This sample is uncertain regarding acid generation (Miller *et al.*, 1991) .

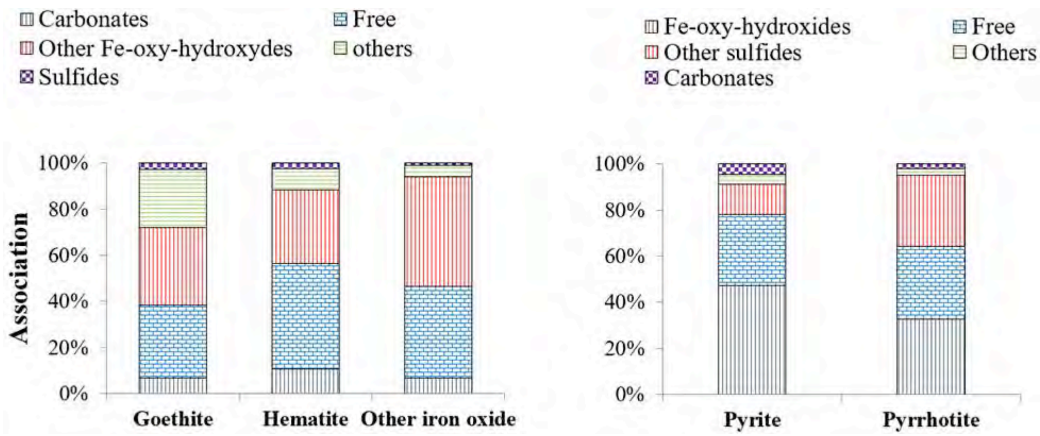

Figure 3: Mineralogical associations of secondary iron phases and sulfides

3.1.3. Leaching tests

Results of the geochemical quality of column tests leachates are shown in figure 4. The columns leaching tests were run for 11 months. The preliminary results showed an acidic behavior for the oxidized tailings: pH of oxidized tailings leachates was between 2.15 and 2.77 with a tendency of slight increase. However, when leachate water from oxidized sample is allowed to infiltrate vertically into unweathered sample, the acidic leachates are neutralized by carbonates dissolution within unweathered tailings. pH of column leachates stayed neutral between 6 and 8.

The electrical conductivity of the leachates from the oxidized tailings is slightly lower than that from oxidized tailings above the unweathered tailings. It varies between 2.89 mS/cm and 5.79 mS/cm while that from vertical infiltration varies between 2.65 mS/cm and 6.4 mS/cm. This may be explained by more extensive dissolution and oxidation reactions due to presence of unweathered tailings.

Acidity and alkalinity are two parameters that describe the capacity of a solution to acidify and neutralize a solution. The acidity measurement obtained from the surface runoff leachates is higher than that measured for leachates from vertical infiltration. Acidity of surface runoff leachates varies between 250 mg $CaCO_3$/l and 1550 mg $CaCO_3$/l. The acidity of vertical infiltration leachates is low and varies between 15 mg $CaCO_3$ and 168 mg $CaCO_3$. During the vertical infiltration, the acidity produced in the oxidized tailing is neutralized by carbonates within the unweathered tailings. The alkalinity of surface runoff was zero during the entire column test duration. The alkalinity of vertical infiltrated water was between 40 mg $CaCO_3$ and 179 mg $CaCO_3$.

The leachates chemical compositions for the two scenarios were different. Leachates from surface runoff scenario are characterized by high concentrations of Ca, Al, Fe and Zn while the leachates from vertical infiltration contain more S and Mg. The concentration of S leached from the oxidized sample ranged between 735 mg/l and 1112 mg/l and that from the scenario of vertical infiltration was between 926 mg/l and 2200 mg/l. The calcium concentrations from the surface runoff scenario varied between 619 mg/l and 894 mg/l and those from vertical infiltration scenario ranged between 423 mg/l and 638 mg/l. The average concentration of Fe leached from the oxidized sample was about 106 mg/l and that leached from vertical infiltration scenario was about 0.88 mg/l. The average concentration of Mg leached from the oxidized sample was low and equalled to 31.4 mg/l, while the leachates form vertical infiltration through the unweathered sample contained high concentrations of Mg and varied between 372 mg/l and 1110 mg/l. A higher concentration of Zn was leached during the surface runoff scenario than during the vertical infiltration scenario with average concentration of 11.13 mg/l and 0.67 mg/l, respectively. Similar results were obtained for Al. The concentration of Al was higher in the leachate from surface runoff scenario (10 mg/l and 50 mg/l) than that measured in the leachate from the vertical infiltration scenario (0.014 mg/l and 0.072 mg/l).

3.2 *Discussion and On-going Works*

The oxidized tailings contain mainly secondary phases while most of sulphides and carbonates were dissolved. The unweathered sample showed high concentrations of carbonates (mainly siderite) and sulphides (pyrite). The hardpan sample showed a mineralogical composition formed by a mixture of oxidized and unweathered tailings minerals. It is formed through the precipitation of secondary phases forming crusts around grains and ultimately forming a cement matrix around them which reduces the porosity of these horizons. The formation of hardpan and the reduction of the porosity influence considerably the hydrogeological properties of these tailings horizons. As a result, the hydraulic conductivity of this tailings horizons decreases leading to a change of the local water flow at the Joutel tailing impoundments which in turn influences the geochemical behavior of tailings (McGregor *et al.*, 2002).

The oxidized sample which was run to simulate surface runoff due to presence of hardpans generated leachates with high concentrations of Ca, Fe, Al and Zn. These elements are associated with secondary minerals. In general, Ca and S originate from the dissolution of gypsum. The mineralogical characterization of oxidized sample showed 19.3% gypsum. Fe and Zn are related to secondary iron oxy-hydroxides. Precipitation of secondary iron phases leads to trapping of wide range of contaminants (McKenzie, 1980; McGregor *et al.*, 1998; Fukushi *et al.*, 2003). These elements are released during iron oxy-hydroxides dissolution (Sheoran *et al.*, 2006).

The acid generation in the oxidized sample is mainly related to secondary iron phases (El Adnani *et al.*, 2016). The acidity is maintained during all kinetic tests because of the absence of neutralizing minerals which is confirmed by absence of alkalinity in the leachates from oxidized sample. The neutralization potential of oxidized tailings is depleted after more than 25 years of field exposure and the same tendency for sulphides.

The oxidized sample above unweathered sample was run to simulate vertical infiltration of water flows due to absence of hardpans. The geochemical monitoring of this column test showed a non-acid generating behavior. The acidity generated by the oxidized tailings is neutralized by unweathered tailings carbonates. Carbonates dissolution is the main pH buffering within the unweathered tailings. This is confirmed by Ca and Mg leaching during this scenario. During surface runoff scenario, there are some contaminants leached such as Al and Zn. These contaminants are present in very small concentrations during vertical infiltration scenario. It may be that, during pH buffering by carbonates dissolution, these elements are co-precipitated or adsorbed by the new formed iron secondary phases.

Evaluation of hydrogeological properties of oxidized, unweathered and hardpan samples is ongoing to demonstrate the variation of permeability and water retention curves. Hydrogeological modelling using Hydrus 1D is ongoing to evaluate the water content and water flow in the tailings impoundment in presence and absence of hardpans.

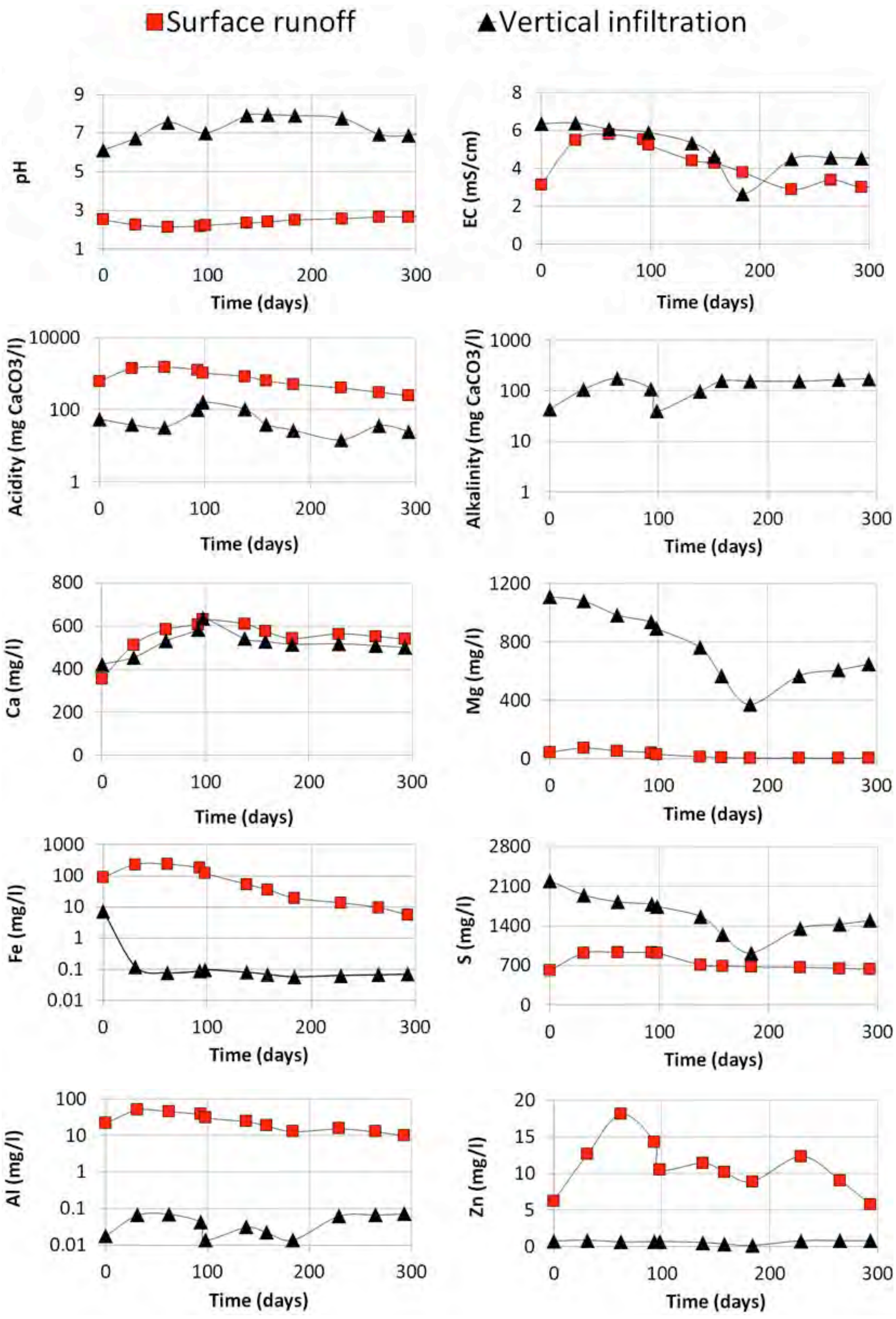

Figure 4: Geochemical monitoring of column tests

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GUNNAR MINE TAILINGS REMEDIATION PLAN – WATER SHEDDING COVER SYSTEM AND LANDFORM DESIGN

Kristie Bonstrom & Denise Chapman
O'Kane Consultants, Saskatoon, SK, Canada

Chris Reid & Alexey Klyashtorin
Saskatchewan Research Council, Saskatoon, SK, Canada

ABSTRACT: Saskatchewan Research Council (SRC) is acting as project manager for Cleanup of Abandoned Northern Sites (CLEANS) on behalf of the Saskatchewan Ministry of Economy. Remediation of the Gunnar uranium site, located on the shores of Lake Athabasca, falls under the Cleanup of Abandoned Northern Sites (CLEANS) project on behalf of the Saskatchewan Ministry of Economy. A case study of the project is presented in this paper. Remedial action for the Gunnar tailings deposits is driven by human and ecological health risk posed by exposure to gamma radiation. O'Kane Consultants Inc. with geochemistry support from EcoMetrix Inc. completed this detailed tailings remediation plan for SRC. The objectives of the remediation plan was to develop a design that meets design criteria and also meets the expectations of regulators, the federal and provincial governments and the Aboriginal communities of the Athabasca region. The detailed design includes a water-shedding cover system and landform, surface water management system, and revegetation plans for the Gunnar Main Tailings, Langley Bay, Gunnar Central, Beaver Pond and Catchment 3 Secondary Tailings Deposit. A key tool for assessing the design throughout the process is the failure modes and effects analysis (FMEA). This was completed for the Gunnar Tailings Remediation Plan to ensure all design criteria are met and to identify key risks to the design.

1 INTRODUCTION

The Gunnar uranium mining and milling site is located on the north shore of Lake Athabasca in northern Saskatchewan. The site, operated by the former Gunnar Mines Limited, commenced uranium production in 1954 and ceased mining operations in 1963. Uranium ore originating from the open pit and sub-surface mines at Gunnar generated approximately 8.5 million tons of rock that had been mined and processed. Mining resulted in directing approximately 2.2 to 2.7 million m^3 of waste rock and over 5 million tons of unconfined tailings to nearby valleys, depressions, and lakes, covering a total of over 70 ha of land. The location of the Gunnar site and a layout of the Gunnar site features is shown (FIGs.1 and 2).

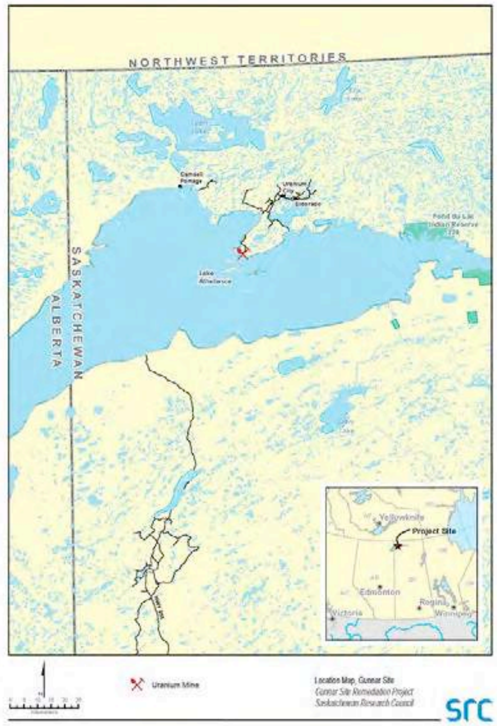

Figure 1. Location of the former Gunnar Mine site (SRC, 2013).

Figure 2. Gunnar Mine site features (SRC, 2013).

The Gunnar Uranium Mine is part of the Cleanup of Abandoned Northern Sites (CLEANS) project. In 2006, Saskatchewan Research Council (SRC) was requested by the Government of Saskatchewan to manage the clean-up of the former Gunnar Mine and Mill site as part of Project CLEANS. In February 2009, the Federal Minister of Environment announced the Former Gunnar Mining Site Rehabilitation Project Proposal would need to proceed as a Comprehensive Environmental Assessment. In 2015, SRC received final regulatory signoff on the Environmental Assessment from the Government of Saskatchewan. The project moved forward in 2015 with development of remediation plans for the site that was separated into two main areas: the Tailings Areas and the Other Site Aspects. O'Kane Consultants Inc. was retained to develop the Tailings Remediation Plan for the Gunnar Mine tailings areas and following sections discuss targets and plans for the Tailings Areas only. The Tailings Remediation Plan ('the Plan') includes the construction of cover systems and landforms as well as a surface water management system for the exposed tailings deposits at the Gunnar Mine site. Remediation plans were developed for the three main (primary) tailing deposits along with minor (secondary) tailing deposits (see Table 1).

Table 1. Gunnar tailings areas included in the tailings remediation plan (OKC, 2013).

Tailings Area Location	Design Surface Area (ha)
Primary Deposit	
Gunnar Main	48.0
Gunnar Central	11.0
Langley Bay	18.5
Secondary Deposit	
Beaver Pond	2.7
Catchment 3	3.4

2 TAILINGS REMEDIATION PLAN DESIGN DEVELOPMENT

2.1 *Development of Gunnar Mine Site Overall Objectives and Design Criteria*

A rigorous stakeholder engagement and consultation program was carried out by SRC prior to the development of any remediation plans for the site. The program was an essential part of the remediation plan process to ensure that any plans developed met the appropriate site-specific criteria for the Gunnar site. Key stakeholders for the project were the federal and provincial governments, the regulators, as well as Aboriginal groups of the local communities in the Athabasca region. Consultation involved numerous meetings and workshops, as well as the studies and interviews conducted as part of a Traditional Knowledge Survey and the Socio-Economic Assessment. A key outcome of the stakeholder consultation program was the development of an overarching land-use objective: "to manage the use of the land and renewable resources of the Athabasca in an integrated and environmentally sound manner to ensure ecological, economic, social, cultural, and spiritual benefits for present and future generations" (SRC, 2013). The Gunnar site falls into the Special Management Zone of the Land Use Plan, where the protection of cultural places and wildlife habitat is paramount.

As part of the Environmental Assessment process, and in conjunction with stakeholder and Aboriginal input, the criteria that were developed were Site Specific Remedial Objectives (SSRO's). That is, site-specific criteria for all of the identified constituents of potential concern. Because of regionally elevated background conditions, the development of SSRO's are a more realistic measure of protecting human and environmental health at the Gunnar site as opposed to generic numeric standards. These were carried through as remedial objectives during the engineering design process. SSROs were calculated using a species sensitive distribution (SSD) approach, consistent with the methodology used by the Canadian Council of Ministers of the Environment (CCME). Concentrations providing protection to 80% and 90% of the species were considered for the selection of SSROs for the Gunnar site. For larger water bodies, such as St. Mary's Channel and Langley Bay, a 90% protection level was implemented. Given its limited size and function in the broader aquatic environment, an 80% protection level was recommended for Zeemal Bay. SSROs developed for the Gunnar mine site during the EIS process are found in Table 2.

Table 2. Gunnar surface water quality SSROs (SRC, 2013).

Constituent of Potential Concern	SSRO for St. Mary's Channel/ Langley Bay (µg/L)	SSRO for Zeemel Bay (µg/L)
Arsenic	100	390
Cadmium	0.30	0.85
Copper	5	12
Lead	13	35
Uranium	90	200

The primary objective of the project was to develop detailed remediation designs using the criteria developed in the EA process for the exposed tailings deposits at the Site (Table 3). The Tailings Remediation Plan for the deposits was completed to meet design criteria developed during the EIS process. The general design criteria were: limit external radiation exposure to levels as low as reasonably achievable (ALARA principle), meet surface site specific water quality remedial objectives (SSROs), improve air quality by eliminating long-lived radioactive dust (LLRD), ensure traditional land uses can occur adjacent to the landforms as well as limited activity on the landforms, design a water shedding landform, design a surface water management system to handle 1:200 year event peak flows, and establish a self-sustaining native plant community.

Table 3. Gunnar tailings remediation design criteria (SRC, 2013).

Parameter	Criteria
External radiation exposure	Reduce gamma dose rate radiation to 1.14 µSv/h (1 µSv/h above the local natural background) for the average of measurements taken over a 1 ha area and 2.64 µSv/h (2.5 µSv/h above the local natural background) as a maximum spot measurement.
Surface water quality	Meet site-specific remedial objectives (SSROs) in St. Mary's Channel and Langley Bay (see **Error! Reference source not found.** in Section **Error! Reference source not found.**).
Groundwater quality	Groundwater quality to be compared to 2010 interim Tier 2 commercial / industrial guidelines developed on behalf of Environment Canada. Radionuclides to be compared to 2010 Alberta Tier 1 Soil and Groundwater Remediation Guidelines.
Air quality	Keep concentrations of particulate matter (PM) emissions during closure phase to <10 µm below the 24-hour criteria of 50 µg/m^3 and PM ≤2.5 µm below the Canada Wide Standard of 28 µg/m^3.
Land use	Ensure traditional land uses can occur adjacent to the site. Prevent the construction or operation of permanent or temporary residences on remediated mine waste deposits.
Landform	Design landform to be water-shedding and increase the distance between the rooting zone and water table / capillary fringe to prevent COPC efflorescence and limit the effects of solute uptake.
Surface water management	Design surface water management system to handle peak flows from the 1 in 200 year event (see Section **Error! Reference source not found.**).
Vegetation	Establish a self-sustaining community of plant species native to the region.

2.2 *Failure Modes and Effects Analysis*

A failure modes and effects analysis (FMEA) was completed for the Gunnar Tailings Remediation Plan to ensure all the above design criteria were met and to identify key risks to the remediation designs. Potential failure modes identified embodied the key physical, chemical, and biological processes relevant to the site that could potentially influence the long-term integrity or performance of the remediation designs. The results of the FMEA on the remediation plan were used to guide key support studies required for refining the detailed designs to mitigate the risk.

Failure modes having been assigned the highest risk identified in the FMEA process were concerned with the unknown quantities of suitable material to complete the cover system designs and maintaining cover integrity from frost heave (physical process) and capillary rise of affected water (chemical process). Additional studies were completed to address key risk areas in support of the detailed design development.

The FMEA was re-evaluated following completion of the studies identified to address the risk during the evaluation. Based on the results of the detailed design support studies, increased confidence was gained in estimations for the FMEA compared to the estimates following the development of the initial remediation plans. In particular, the risk rating for each of the failure modes rated as high during the initial FMEA on the preferred remediation plan was reduced to moderate or moderately high, both of which now fall in the As Low as Reasonably Practicable (ALARP) area of the risk matrix used in the analysis. In general, the additional information provided in the assessments allowed improved estimates of likelihood (by reducing the likelihood). The FMEA is a key tool for reclamation and closure design as it provides a documented process of what potential failure modes are identified, how the risk is assessed, and plans for mitigation.

3 TAILINGS REMEDIATION DESIGN DETAILS

The Tailings Remediation Plan includes the construction of cover systems and landforms as well as a surface water management system for the exposed tailings deposits at the Gunnar Mine site. The plan includes plans for revegetation of the areas and a monitoring plan to track design performance. Specific details for each tailings area of the Gunnar site can be found in OKC (2015). A general overview is provided below.

3.1 *Gunnar Main and Gunnar Central Design Detail Overview*

The Gunnar Main and Gunnar Central Tailings areas consist of similar design elements. The tailings area landforms were created to be water-shedding with positive surface drainage to eliminate standing bodies of water on the landform to reduce meteoric water from infiltrating and contacting the tailings surface, reducing potential loads. Waste rock fill was designated to be used where substantial fill requirements were needed as it was determined that using waste rock fill would not result in SSRO exceedance. Waste rock was also used in the design to create working platforms and to create a coarser-textured material layer over the finer-textured tailings material, thereby limiting the upward migration of tailings pore-waters and contaminants of potential concern (COPCs).

An earthen or soil cover system a minimum 0.6 m thick will be placed on the areas for remediation in-place to mitigate ecological and human health risks to meet the gamma radiation design criteria. A two layer cover system was designed to optimize the volume of available borrow material of suitable texture to reduce erosion potential. The bottom 0.3 m layer consists of 0.3 m medium-textured borrow material and 0.3 m coarser-textured material. The 0.3 m layer of coarser-textured material will be placed at the cover system surface as a vegetation growth medium and erosion resistance layer.

The drainage channel originating in Gunnar Main and travelling through Beaver Pond runs through the Gunnar Central area along the northeast extent of the area. The channel collects surface waters from the reclaimed tailings and natural catchment area, routing surface waters off the landforms to reduce net percolation. The channel was designed using the 1 in 200-year storm event criteria based on climate data from a nearby station that was adjusted for climate change. The final surface of the covered areas will be vegetated with native plant species following recommendations from extensive studies conducted by SRC (OKC, 2017).

3.2 *Langley Bay Design Detail Overview*

The tailings mass within Langley Bay area is saturated and an alternative remediation design was developed for the area. The remediation design leaves the tailings in-place and includes a minimum of 1.5 m till cover system and defined shorelines with rock armor protection for protection of the tailings deposit across the entire range of historical lake water levels. Waste rock was not used as fill to create the landform for this area to reduce the risk of loading to the adjacent Langley Bay. A minimum 1.5 m till cover system, consisting of a 1.0 m medium-textured layer overlain by a 0.5 m coarser textured layer, provides adequate protection of the saturated tailings mass. Final elevation of surface is a minimum of 0.3 m above the maximum historical water level greatly diminishing the probability of flooding the entire landform. A constructed surface water channel between Back Bay and Langley Bay provides long-term management of Back Bay waterbody.

The effects of differential settlement and freeze/thaw cycling or frost heave were considered in the development of the Langley Bay cover system design given that they were identified as failure modes during the FMEA process having a high risk rating. The nature of the materials available for remediation and the location of the water table near the surface of the tailings, creates conditions where both differential settlement and freeze / thaw cycling could potentially affect the final landform. Analysis of consolidation and differential settlement indicated that differential settlement will likely be less than 0.1 m and won't affect surface water drainage patterns. The use of medium and coarser-textured materials, which are generally sandy with low proportion of silt and clay materials, will limit the occurrence and magnitude of frost heave affecting the cover system surface.

Additional mitigation measures incorporated in to the design include development of the Main and Back Bay overflow channels effectively isolate the landform from surface water runon from the adjacent natural catchment. Runoff and subsequent ponding in areas with differential settlement or frost heave will be limited to snowmelt and large rainfall events in localized areas of the landform. Localized ponding on a small area of the Langley Bay landform might occur, but generally surface water will drain on the landform due to the overall gradient of the surface and the relatively short flowpaths to engineered drainage channels. The FMEA concluded that risk that localized ponding could lead to deterioration of cover system performance and increased concentrations of COPCs in Langley Bay was low and in the broadly acceptable range.

3.3 *Secondary Tailings Deposit Design Overview*

Secondary tailings deposits for remediation for the Gunnar site are known as Beaver Pond and Catchment 3. These areas contain areas where tailings have be re-deposited from the primary areas. The Beaver Pond area is a result of an overflow of tailings from the Gunnar Main area and settled within a ponded area. In addition, till overburden was excavated from the current dammed channel in 1954, creating a surface water flowpath from the beaver dams to Gunnar Central area. The Catchment 3 area is thought to be a result of tailings migration from Gunnar Main through a berm break subsequent redistribution through surface water movement as well as through windblown erosion. The secondary deposit tailings area extents will be further confirmed as the construction of the areas is completed; however, the remediation of the design for these areas are briefly summarized in the following sections.

3.3.1 *Beaver Pond Design Detail Overview*

The detailed design included development of an engineered surface water flow channel providing sufficient flow capacity and armoring protection to convey the 1:200 year design storm event. Excavation to develop the channel will remove tailings housed within the Beaver Pond area to the greatest extent possible, based on available topography and the results of 2015 bathymetry measurement completed by Saskatchewan Research Council. Excavated tailings will be re-located to Gunnar Main tailings area. Any additional tailings located outside the channel footprint encountered during construction will be excavated and re-located to Gunnar Main tailings area. Recontouring and vegetation of the area will be implemented as required to blend with the surrounding landscape.

3.3.2 *Catchment 3 Area*

Catchment 3 tailings are located directly adjacent to eastern edge of the Gunnar Main tailings area. Similar to remediation strategies at Gunnar Main and Central, after clearing and grubbing of vegetation on the tailings surface, waste rock will be placed to create the landform and will be covered with a 0.6 m thick cover system. The waste rock thickness within the landform varies, but the design provides a minimum of 0.5 m of waste rock over the defined extents of the tailings areas. The cover system placed on the waste rock surface consists of a 0.3 m layer of medium-textured material overlain by a 0.3 m coarser-textured layer at surface. Similar to the Beaver Pond area, the area will be contoured and revegetated to blend with the natural surroundings.

3.4 *Comparison of Design Details to Design Criteria*

The suitability of the cover system, along with the underlying waste rock fill (where required), was evaluated against several parameters such as providing a barrier to gamma radiation, elimination of airborne tailings dust, reduction of net percolation from current levels, surface erosion, effects of freeze/thaw cycling and differential settlement, limitation of capillary rise, and sustainability of vegetation. Table 4 provides a summary of how the Tailings Remediation Plan designs address the design criteria.

Table 4. Gunnar tailings remediation plan compared to design criteria (OKC, 2017).

Parameter	Criteria	Remediation Design
Gamma Radiation	Reduce gamma radiation to 1.14 μSv/h	0.6 m of local till material reduces gamma to below criteria
COPC Loadings	Meet site-specific remedial objectives (SSROs)	Predicted loadings show that COPCs will be reduced from current conditions and will meet SSROs for Langley Bay
	Reduce COPC loadings to Langley Bay	
Air Quality	Eliminate tailings dust emissions	Placement of cover system will eliminate tailings dust emissions
Land Use	Ensure traditional land uses can occur adjacent to the site	Placement of cover system and subsequent revegetation will ensure traditional land uses can occur
Landform	Design landform to be water-shedding and increase distance between plant roots and water table	Water shedding landform utilizes minimum 0.75% hillslope gradients and minimum 0.5% channel gradients. Cover materials in areas with shallow water table chosen to minimize upward migration of COPCs
Surface Water Management	Design surface water channels/streams to handle flows from 1 in 200 year event	Surface water channels and outlets on all landforms were designed to handle the peak flows from a 1 in 200 year event including climate change
Vegetation	Establish a community of plant species native to the region	Revegetation plan uses species native to the region

A monitoring program was developed as part of the full Tailings Remediation Program to demonstrate performance and track the evolution of the designs. The monitoring program is an essential part of the design and is part of any closure design to provide evidence that the final remediation plan performs in accordance with the decommissioning and reclamation plan, is on the appropriate trajectory and achieves the predicted stability.

4 CONCLUSIONS

Detailed designs for the exposed tailings areas at the former Gunnar Mine site in Northern Saskatchewan was developed. Paramount to finalization of the plan was the process that was followed through design development of the area. Stakeholder engagement at the beginning of the overall closure process for the site was key to development of closure objectives and appropriate design criteria that is reasonable and achievable for the site. The incorporation of a FMEA through the design process at the conceptual level provides a framework to be able to identify potential failure modes for the designs that are important to project owners and stakeholders and implement mitigation measures to reduce the risk. This process is then used at the detailed design stage following completion of the mitigation plans to ensure that the areas of high risk have been lowered to an acceptable level. The Gunnar Mine Tailings Remediation Plan provides an example of the importance of stakeholder input and consideration of site-specific characteristics in meeting closure objectives.

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Identifying Water Flow Paths Through, Under & Around Tailings Impoundments

M.J. Kennedy, R.R. Blanchard, V.O. Kofoed, M.L. Jessop & D.F. Bierman
Willowstick Technologies, Draper, UT, USA

Cecil Urlich
AECOM, Seattle, WA, USA

ABSTRACT: Tailings impoundments continue to undergo failures at an unacceptably high rate, including failures at operations owned by high profile mining companies. These failures are often the result of a combination of design, construction and operations actions that are controlled by humans and must be better coordinated and managed in the future. The consequence of failure can be widespread flows of tailings, and this can be extreme in terms of life loss, environmental damage, social license to operate, company value, and mining industry sustainability. Therefore, it is necessary that the mining industry strive for zero failures of tailings facilities. Any additional technology and information that enables an owner of a tailings impoundment to be more certain of its condition and thereby reduce the risk of failure is of tremendous value to reliable tailings and mine water management. The Willowstick method has been successfully used to identify water flow paths through, under and around tailings impoundments in plan and elevation. This provides additional information to supplement existing analysis, and to further improve tailings impoundment safety through more robust designs, if necessary. This paper, by the use of two tailings impoundment case studies, illustrates the procedure, findings, and the benefits of this methodology. The findings include tailings impoundments where new groundwater flow paths were identified. The impoundment designers were able to update the designs to mitigate for the new information and design a safer and stable tailings impoundments with reduced risk of failure and in accordance with the goal of zero failure.

1 INTRODUCTION

Tailings dams and storage facilities are unique and different than water supply reservoir dams for many reasons: they store tailings (waste liability) versus water (resource asset); their construction and design evolve as the mine lives advance and tailings volumes increase while codes, regulations and technologies change; they embrace tailings properties, disposal methods and water management in dam design; and they store tailings in perpetuity.

Properly planned, designed, constructed, operated and maintained tailings dams provide safe and effective tailings storage facility (TSF) structures. Dams are raised by optimizing upstream, centerline and downstream methods to fit site conditions, land constraints and mine operations. TSFs use a host of tailings and water management methods to optimize storage space, reduce operation costs, protect the environment, and maintain indefinite dam stability and safety.

Tailings dams are dynamic structures that grow in size and complexity over their operating life and must be maintained after closure. Their designs evolve with time and not always as originally envisaged because of many factors that range from global events, commodity prices and regulations that mine owners have no control over, to local operation, community, environmental, and other challenges.

Evolving designs continue to combine proven existing and new (disrupt mining) technologies to enhance the positive, progressive and collective ability to effectively manage tailings dams and TSFs.

2 NON-INTRUSIVE INVESTIGATIVE TECHNOLOGY

This paper describes the Willowstick geophysical technology that uses a low voltage, low amperage, alternating electrical current to energize subsurface water by placing electrodes in tailings water and seepage discharge. With one electrode placed in the tailings and the other placed in contact with water below the tailings dam, an electrical circuit is established between the electrodes (Figure 1).

As with all electrical currents, this circuit generates a magnetic field that is measured and mapped from the surface. The collected data is used to render two- and three-dimensional (2D and 3D) maps and Electric Current Distribution (ECD) models of seepage paths (Figure 2). The technology maps and models preferential groundwater flow paths, just like an angiogram that enables doctors to "see" blood vessels in the human body.

The application of the technology to tailings dams is based on the principle that water increases the conductivity of earth materials through which it flows. As the signature electric current travels between electrodes strategically placed upstream and downstream of the tailings dam, it concentrates in the more conductive zones, or the areas of highest transport porosity, where tailings water preferentially flows out of the TSF as seepage through, under, or around the dam.

An electric circuit is established in the water of interest. Measuring the resultant magnetic field at the surface reveals the electric current flow and distribution. Data is processed and compared to a predicted magnetic field from a theoretical homogenous earth model to highlight deviations from the "uniform" model. 2D maps and 3D models are generated and combined with known sub-surface data to enhance preferential seepage path definitions (Figures 2).

The graphic shading in the following black-and-white figures is in shades of gray. Within the Ratio Response Map (Figure 2) dark gray shows actual flow that is less than flow predicted by the "uniform" model. Medium gray, within the dotted lines, shows actual flow that is more than flow predicted by the "uniform" model, representing a seepage path.

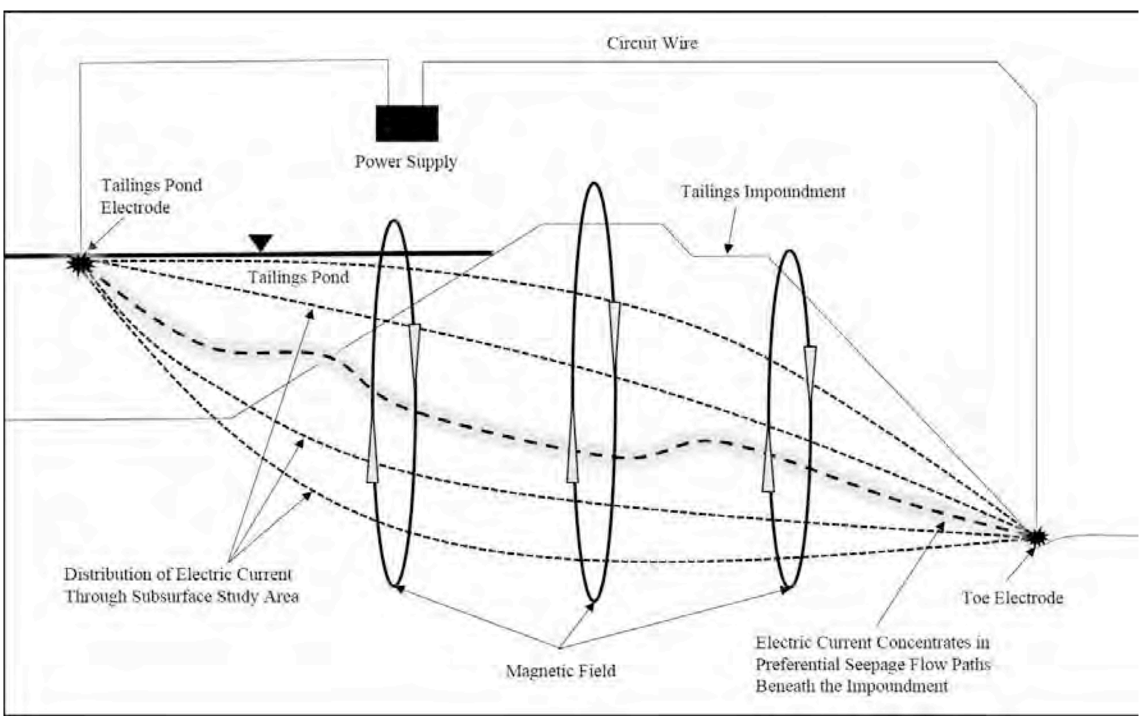

Figure 1. Survey Setup

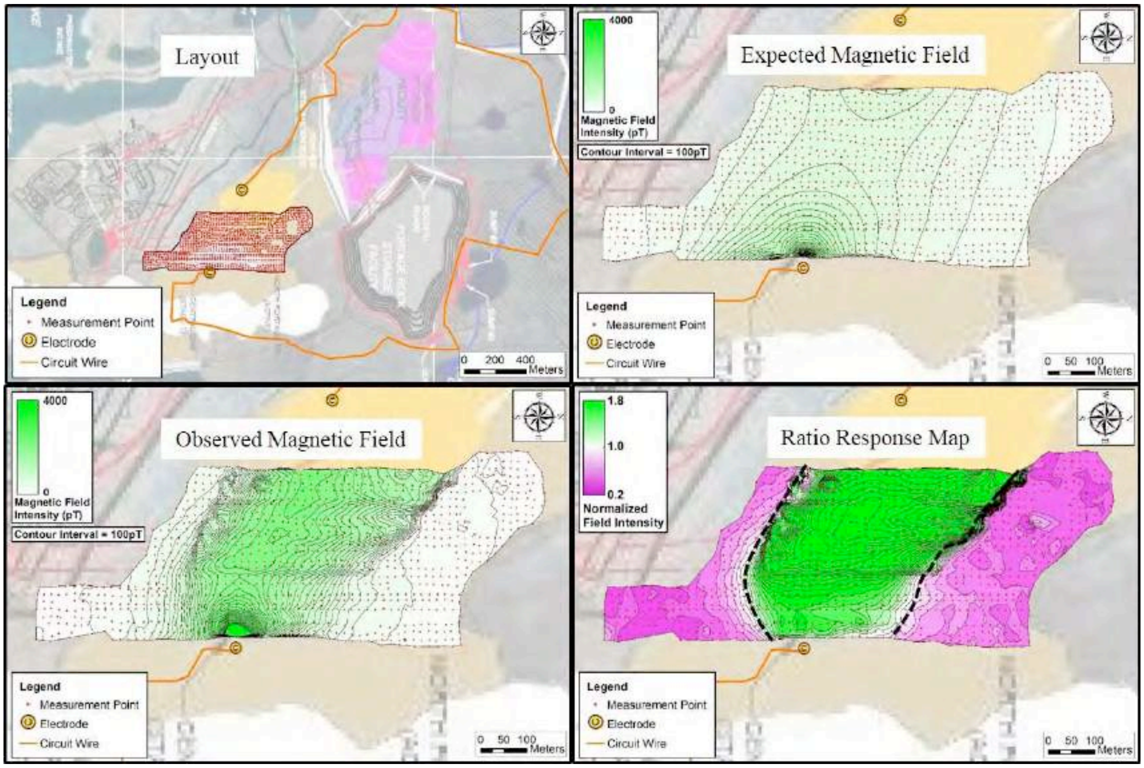

Figure 2. Steps to get data results and two dimensional maps

This paper, through two tailings dam studies, illustrates the procedures, findings and benefits of this technology. Results range from confirmation of design decisions to identifying new seepage paths. In the latter case, the designs are now being updated based on the new information to provide a more safe and stable tailings dam and reduce the risk of failure.

3 TAILINGS DAM STUDY 1

Tailings Dam Study 1 involves a newly constructed tailings dam downstream of an original active TSF and upstream of the open pit (Figure 3). While the new dam was being built, seepage was observed at its downstream toe. The Willowstick method was conducted to delineate the seepage paths and any risks that they might pose to the new dam and the open pit.

The Willowstick surveys identified primary and secondary seepage paths under the dam (Figures 4 and 5). The plan and profile views show seepage path widths and depths. The light-grey lines are the interpreted seepage paths passing under the dam.

The results confirmed that there are no other seepage paths of any significance through or under the dam. This provided confidence that piping and internal erosion were not concerns for the dam itself because the identified seepage paths are in the foundation materials under the dam. This means the geotechnical investigations for piping potential can focus on the foundation materials.

A Geotechnical investigation was completed by drilling into the seepage path target areas identified by the survey. The drilling confirmed the survey findings. Remedial measures are now being planned that will cost significantly less than if the survey had not been completed.

Figure 3. Tailings facility seepage area site plan

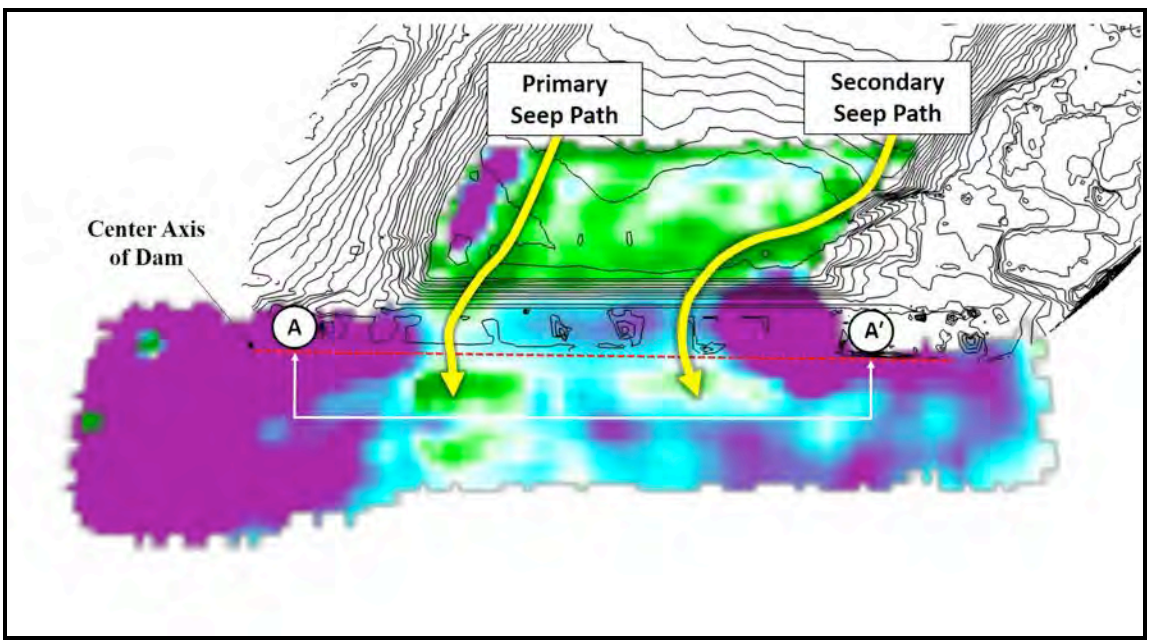

Figure 4. A primary and secondary seepage path identified under the dam

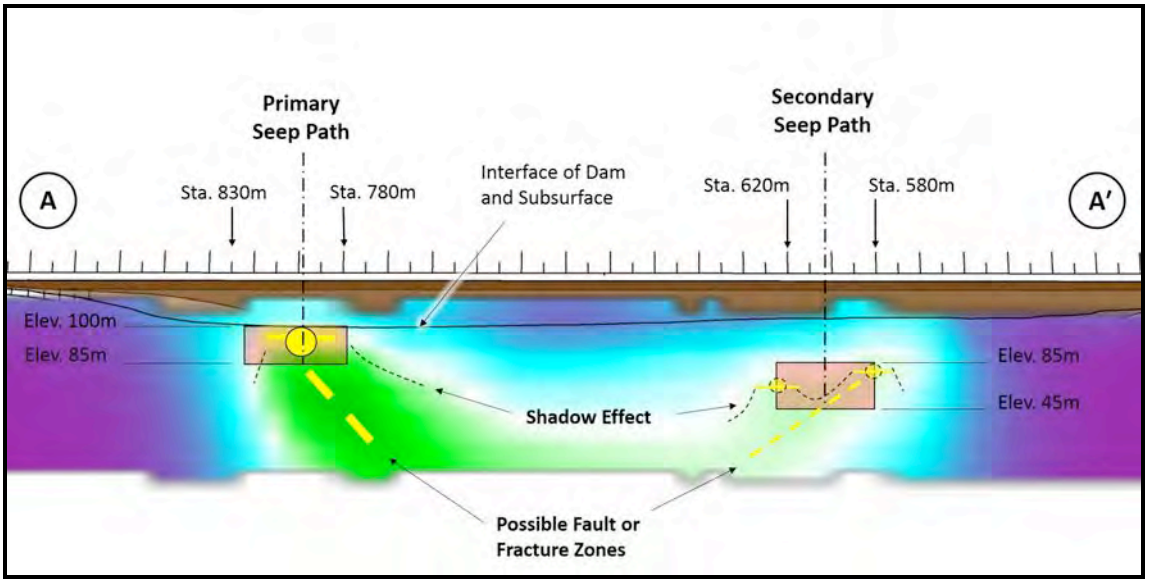

Figure 5. Seepage path elevations

4 TAILINGS DAM STUDY 2

Tailings Dam Study 2 involves a TSF with a tailings dam in a valley. A seepage collection pond is downstream of the tailings dam. A waste rock dump occupies one of the valley sides of the TSF. A 2-kilometer long drainage collection trench is aligned along the toe of this dump (Figures 6).

Seepage from the TSF collects in a collection system and is pumped back to the TSF. Additional contribution to the pumped back water is surface runoff from the collection system catchment area. There was concern that some "surface runoff" could also be seepage from the TSF through the left abutment of the tailings dam, and that there could be seepage from the TSF flowing under the collection system and to the environment.

Water that escapes from the waste rock dump drainage collection trench also enters the TSF. However, as the TSF rises over time, the hydraulic gradient could switch from the TSF to the waste rock dump.

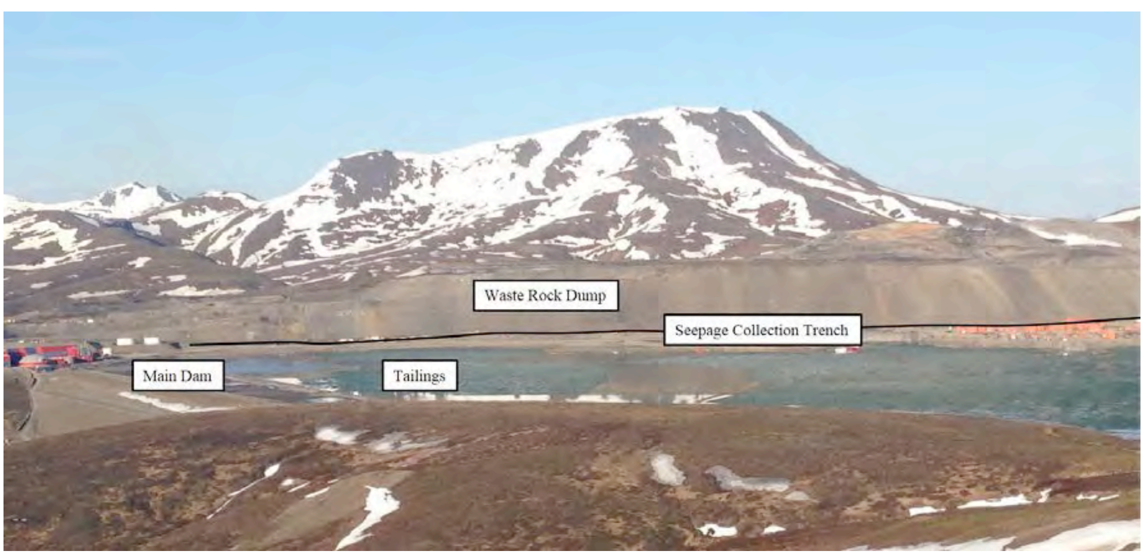

Figure 6. Tailings facility and waste rock dump

The survey was conducted in two phases: The purpose of Phase 1 was to identify and delineate seepage patterns through, under, and around the tailings dam. This will enable them to stabilize the tailings dam and raise it, if necessary. The purpose of Phase 2 was to identify any deeper seepage paths that might pass under the waste rock dump drainage collection trench and flow into the TSF.

During Phase 1 the surveys identified three major seepage paths. The first seepage path was found flowing under the wing wall section of the tailings dam (Area 1 in Figure 7). This seepage path is broad at first but narrows as it progresses down the hillside. Eventually it drains into the TSF, 5 to 6 meters below the ground surface in a buried former drainage channel. The second seepage path began in the TSF following the original stream channels under the TSF (Area 2 in Figure 7) and it converges to a single path in a rock underdrain which was built in the original creek channel connected to a seepage collection pond. There is no sign of seepage bypassing the seepage collection pond and the survey confirmed that the TSF and seepage collection system were performing as designed. The third drainage path unrelated to TSF seepage was identified (Area 3 in Figure 7). Originating from the east hillside just downstream of the tailings dam (white dashed lines in Figure 7). Eventually, this survey confirmed that all of the seepage paths were captured in the seepage collection pond.

No other seepage paths were identified through, under or around the tailings dam and seepage collection pond areas. The survey confirmed the design assumptions and performance expectations, and enabled planning to start for future tailings dam raisings and collection pond relocations to accommodate the increased footprint of an expanded tailings dam.

For the Phase 2 survey, solid lines show primary seepage paths. Light dash lines show secondary seepage paths (Figure 8). Dashed circles show electric current preferentially flowing through and under the trench. Primary seepage paths were found to align with original drainages shown on pre-mining maps. Secondary seepage paths are under the trench but are shallower than primary seepage paths. The bottom of the trench intercepts the uppermost part of the seepage paths.

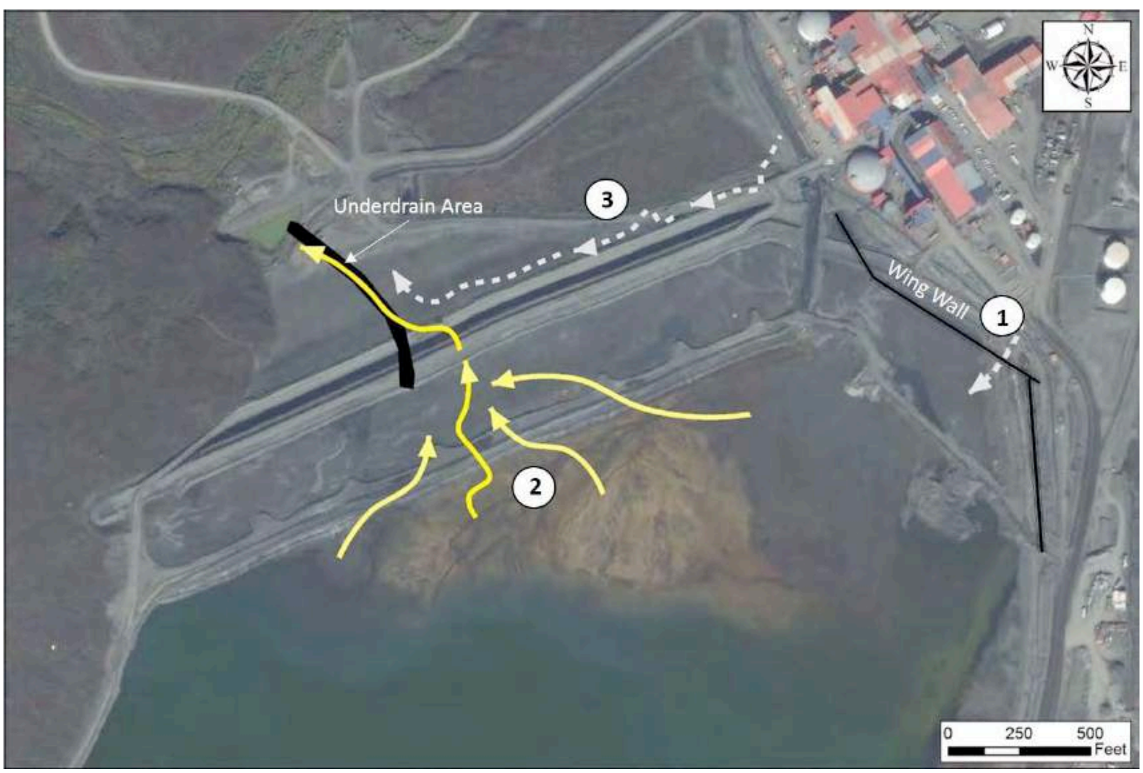

Figure 7. Plan view of Phase 1 flow paths

The survey found that seepage from the waste rock dump was flowing under the dump along the top of the original ground surface. The model identified the location and depths of the primary seepage paths that were found to be just under the base of the drainage collection trench (Figure 8 and 9). Figure 10 shows a seepage paths' depth within the 3D model.

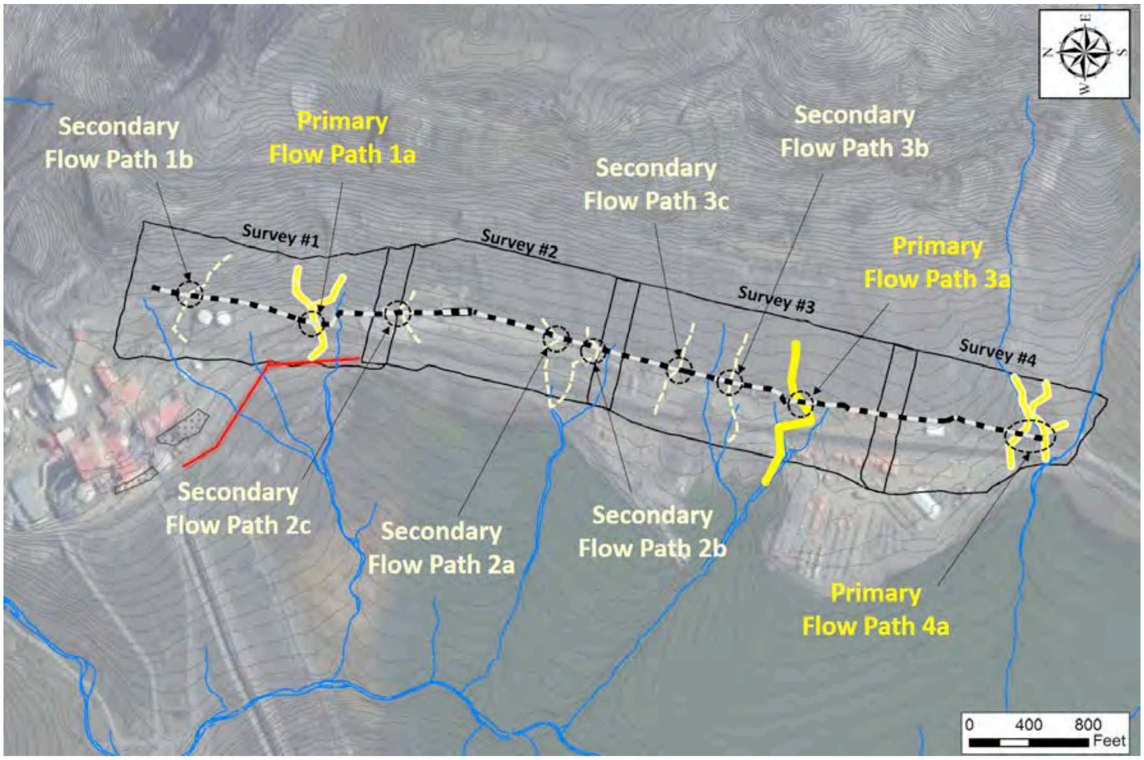

Figure 8. Plan view of Phase 2 flow paths

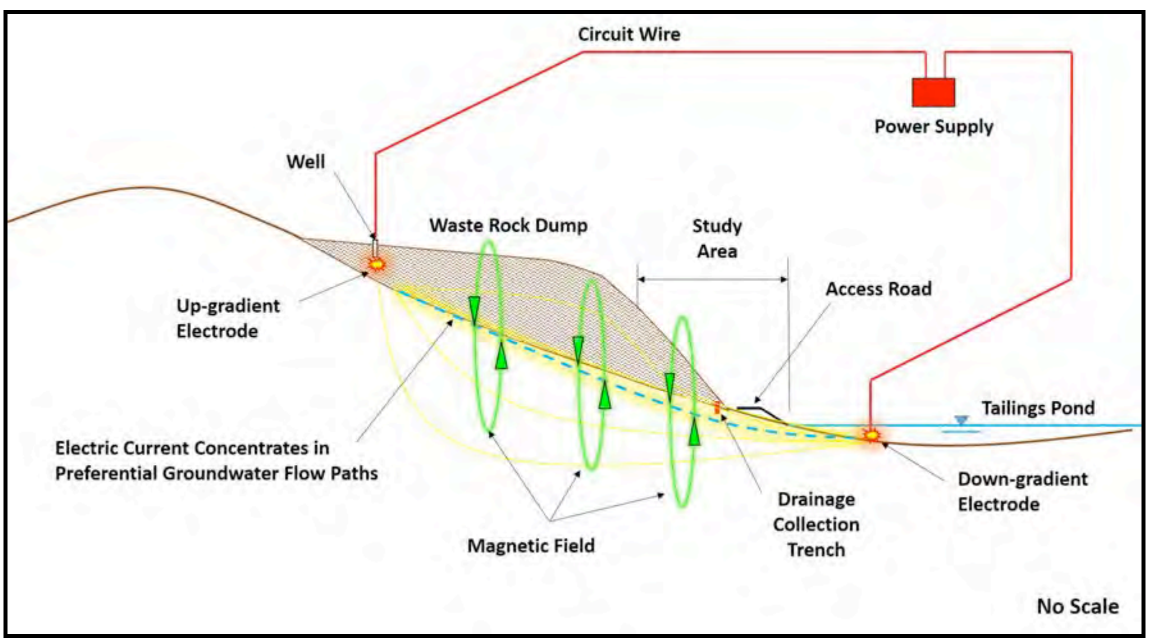

Figure 9. Cross section of survey layout

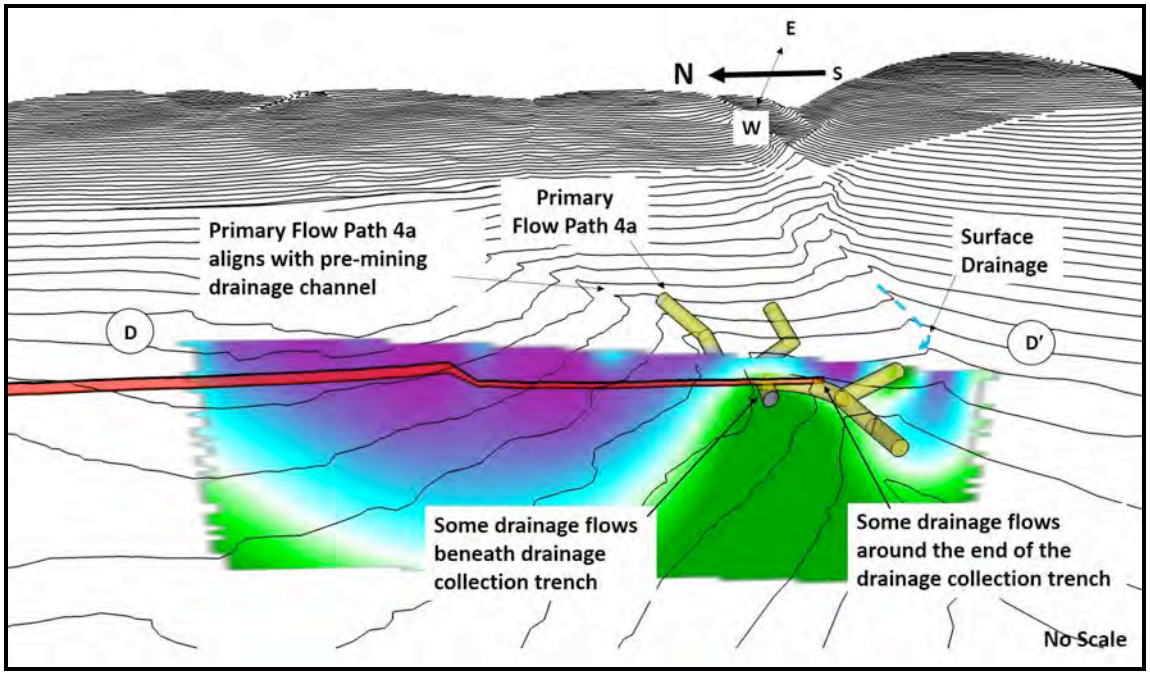

Figure 10. Model slice with seepage flow path

5 CONCLUSIONS

The two tailings dam studies show how the non-intrusive Willowstick method can be used to supplement known geological, geotechnical, hydrological and groundwater information to enhance the knowledge of existing seepage conditions.

This knowledge can then be used to cost-effectively support the design of tailings dam stabilizations and future tailings dam raising including for ultimate closure and post-closure, and to provide long-term safety and stability for TSFs and tailings dams.

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Waste Monitoring and Disposal

Amphibious robot for reclamation work on soft tailings deposits

Nicolas A. Olmedo
Copperstone Technologies, Edmonton, AB, Canada

Michael G. Lipsett
University of Alberta, Edmonton, AB, Canada

Stephen Dwyer & Jamie Yuen
Copperstone Technologies, Calgary, AB, Canada

ABSTRACT: Unmanned amphibious robots are an enabling technology that allows operators of tailings facilities to conduct work on previously inaccessible terrain. Robotic work can include deployment of standard geotechnical equipment, such as cone penetrometers and shear vane viscometers, installation of equipment and sensors, such as placement of wick drains and piezometers, and surface and subsurface sampling. Robotic systems can be used to map an environment in terms of soil properties and to map the depth of the deposit through bathymetric survey. Seed broadcasting and seedlings planting are also possible on saturated tailings for dewatering and strengthening through root bedding as a preliminary step in reclamation. In this paper we present the development of robotic systems for monitoring tailings deposits and opportunities for further work on deploying robots in tailings service.

1 INTRODUCTION

Advances in robotic systems and remote sensing technologies are necessary to improve condition monitoring and performance of environments affected by industrial operations. A key challenge for the mining industry is conducting reclamation work and monitoring hazardous and inaccessible locations, such as mine waste deposits. Fluid mine waste, such as oil sands tailings need to be monitored for complying with legislated requirements (Wills, 2016), improving remediation efforts (Hold, 1993), monitoring environmental conditions (Plumlee, 1994), and improving the performance of mining processes (Lipsett, 2014).

In Alberta, Canada, bitumen extraction processes generate tailings that do not readily consolidate. The material is collected in tailings ponds that settle and form Mature Fine Tailings (MFT) consisting of 30-35 %wt solids, which require further treatment. Flocculant addition and centrifugation are currently the most common treatment processes. The resulting output material is deposited on Dedicated Disposal Areas (DDAs), which need to be regularly monitored. Government legislation outlines constraints on the total volume of tailings allowed.

Current methods to characterize and monitor DDAs include manual geotechnical sampling and measurement campaigns (Beier, 2013). Samples and soil characteristics, such as shear strength, are collected to help minimize long-term storage of the material and identify hazards (Lipsett, 2009) and to determine when reclamation work can begin. Typically, manual geotechnical campaigns are limited to areas accessible to human workers, such as floating docks or areas trafficable by large manned vehicles. Risks to workers and equipment sinking are possible due to the nature of the operations and uncertainty and variability of the terrain. Current surveys with barges, excavators, and boats are costly, intermittent, and may put personnel at risk. Additionally, there is generally very limited surveying of areas inaccessible to workers. Timely environmental monitoring is necessary to improve mining operations.

Reclamation work can be challenging on terrain that is difficult for humans and traditional vehicles. As with geotechnical surveys, reclamation work is constrained to areas that are accessible and trafficable. In some cases, treatment of tailings for eventual reclamation needs to start before the deposits have developed a crust and gained sufficient strength to support workers and earth moving equipment. New platforms for implementation and deployment of treatment materials and methods for dewatering, such as vegetation, are required to start reclamation work as fast as possible at acceptable cost.

Unmanned robotic systems are being proposed to complement manual surveys. Tracked and wheeled robotic systems have been field tested in real industrial operations (Olmedo, 2016a). These systems were instrumented to be remote controlled; and they collect samples and estimate soil properties from wheel-soil interactions (Figure 1). The prototype robots were able to navigate deposits that had developed a weak crust, therefore were limited to terrain with a bearing capacity of 15 kPa. However, in many cases, the deposits that need to be investigated are areas that have recently poured tailings with no bearing capacity at all.

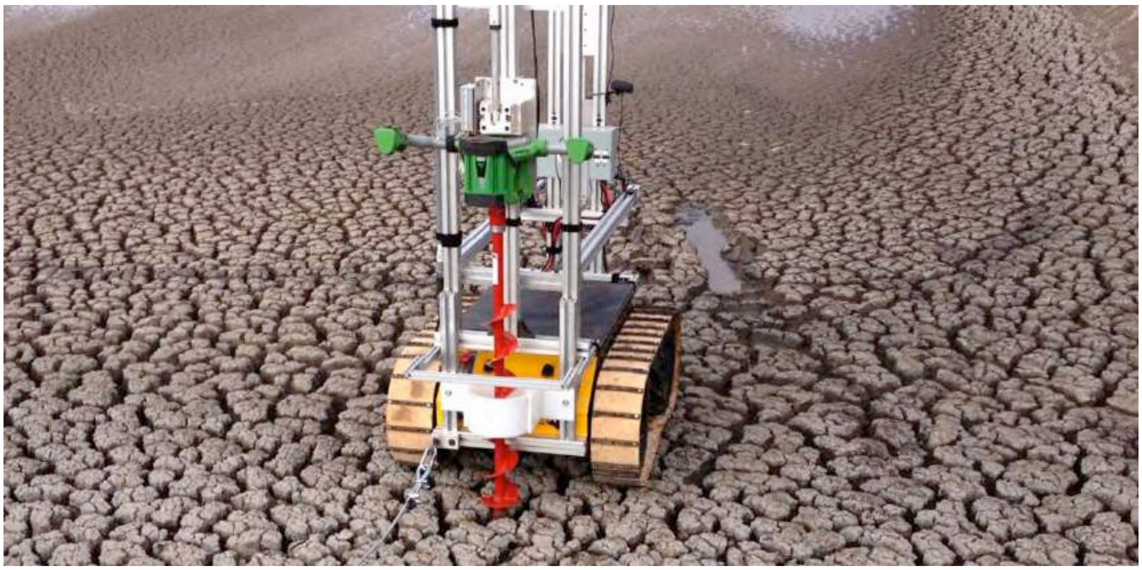


Figure 1. RTC-I rover for tailings characterization collecting a sample from an oil sands tailings deposit.

New systems are required to access DDAs with no bearing capacity. Additionally, platforms that can carry heavy payloads are required to deploy equipment for standard geotechnical testing such as cone penetrometer test (CPT) and vane shear tests (VST). For reclamation work, the platforms need larger payload capacities to deploy reclamation materials. In many cases, the equipment size necessary for carrying people makes amphibious navigation difficult for anything except narrowly targeted campaigns. For this reason, robotic solutions are attractive.

This paper describes recent work on amphibious robots for environmental monitoring (Olmedo, 2016b), by first summarizing the design and results of past field trials of an amphibious robot prototype for monitoring harsh terrains including water, mud sloughs, snow, grass, and treated tailings. Then, the paper describes recent developments on a new amphibious robot for reclamation work. Finally, the paper discusses lessons learned and identifies opportunities for further improvements in the technology.

2 DEVELOPMENT OF CST-AR1: AMPHIBIOUS ROBOT FOR TAILINGS MONITORING

Copperstone Technologies developed the original proof-of-concept amphibious robot prototype (CST-AR1) to navigate a wide variety of terrain and deploy standard geotechnical equipment (Figure 2). The robot was designed to overcome the trafficability limitations of commercially available vehicles and other robotic platforms, and to be sufficiently ruggedized to operate in rough industrial settings.

AR1's locomotion system employs a twin-screw propulsion system. Sealed aluminum cylinders with helical flights are rotated to move through soft and hard ground. These screws provide sufficient buoyancy to float on water or be able to move on deposits with no bearing capacity

(Figure 3). Aluminum cones are used on the front and back of each screw to displace material and minimize drag in the direction of motion of the vehicle.

Figure 2. Initial CST-AR1 field trials in a mud slough, Calgary, Alberta, Canada.

Two electric DC motors drive each screw. Aluminum towers support the motors, gearbox, and screws. The gearbox was designed to be adjustable and provide several transmission options. The robot can be reconfigured to balance between speed and torque in order to optimize performance on different terrains. The redundancy of actuators increases the operating range of the system and provides a recovery option if one of the motors fail.

An adjustable aluminum chassis was built to support a payload deployment tower in the centre of the robot and two marine batteries. The chassis was built to support the large lateral reaction forces experienced due to the counter rotation of the screws. The batteries were selected to be sufficiently ruggedized for harsh environments and to provide a runtime of an 8-hour work shift in water. For cohesive and hard ground the runtime decreases slightly due to increased motion resistance on the screws.

High power motor controllers were used to drive the scrolls in open-loop and closed-loop control modes. For closed-loop control, velocity feedback from ruggedized optical encoders was used to control speed of the scrolls. A disadvantage of closed-loop was that the motors needed to be load balanced in order to prevent a single actuator from carrying most of the load and experiencing over-current. Typically, open loop control of both motors results in a more even distribution of current requirements.

The robot was instrumented with a computer data acquisition system and control. An on-board Linux computer using the Robot Operating System (ROS) (Quigley, 2009) was used to map high-level control commands from the operator to low level commands to the motor controllers. Additionally, fail-safe routines were automated in case of loss of communication or responsiveness of any software component. The operator provided high level commands using a joystick. A 2.4 GHz wireless network was used to transmit commands and feedback, including video and telemetry data such as battery level indicators. The communication system maintained reliable signal strength during all field tests, with a maximum tested distance of 2 km line-of-sight. Two emergency stop buttons were included to disconnect power to the motors. Blinking light indication patterns were used to warn operators and people working around the robot if the robot was powered, emergency-stopped, or ready to move.

Figure 3. Initial CST-AR1 field trials in a lake, Calgary, Alberta, Canada.

The payload deployment system is comprised of a support tower and a rack and pinion mechanism (Figure 3). AR1 was integrated with a standard mud sampler and a standard cone penetrometer. Both payloads could be deployed up to 3m. For deeper deployment, an indexing magazine has been designed but has yet to be prototyped. The rack and pinion mechanism was controlled to lower the payloads at a constant speed. For CPT the penetration rate of 0.02 m/s was used. Optical encoders on the motor driving the pinion were used to estimate the depth of sampling and penetration. The maximum normal force possible was limited by the weight of the robot.

Field demonstrations of CPT were completed using a cone penetrometer from ConeTec (ConeTec, 2016). The cone data acquisition electronics and computer were integrated to AR1 to operate the cone remotely and log the measurements collected. Soil sampling was achieved using a standard mud sampler. The system was limited to one sample per trip. The design of an automatic interchangeable sampler magazine has been completed but has not been prototyped.

3 FIELD TESTING OF CST-AR1

CST-AR1 locomotion and payload deployment was tested in five environments: grass, water, snow, mud slough, and treated MFT. The tests aimed to provide mobility performance data to improve the design and for proof-of-concept deployment of payloads from unmanned amphibious systems.

Mobility on hard ground (Figure 4a) can be achieved in two ways. First, rotating the screws in opposite directions provides screw-type locomotion, which is very power intensive on hard ground due to high friction forces on the blades of the screws and the cylinders, especially during turning. A second option is to roll sideways, which consumes significantly less power, but due to the twin-screw configuration it does not permit turning. A combination of forward screw motion and lateral rolling can provide adequate maneuverability on hard ground.

Improved performance was observed on deformable terrain such as mud sloughs (Figure 4b) and snow (Figure 5a). Lower torque was required to rotate the screws, and turning was easily achieved by rotating the screws at different speeds. The system experienced small resistance while turning achieving high maneuverability. The small loads resulted in lower current required to operate the motors, increasing the endurance of the system. The systems ruggedness was tested in freezing weather.

(a) (b)

Figure 4. (a) CST-AR1 field trials on a grass and (b) mud sloughs.

Minimum motor torque was required for movement in water (Figure 3). In that case, high rotation speeds were necessary to generate enough water displacement. The robot needed to be adjusted to a high-speed and low-torque configuration. The system demonstrated good stability, even with the payload deployment tower fully raised. On water, soft mud, and snow, rotating the screws in the same direction resulted on turning on the spot, rather than rolling sideways as would happen on hard ground.

Indoors tank tests and outdoors field trials have been conducted on MFT and other treated tailings. Results from outdoor field trials and CPT will be reported on a separate publication. Indoor tank trials (Figure 5b) demonstrated sampling and locomotion for treated MFT. The material tested had almost no bearing capacity and minimal adhesion to the screws; therefore the performance was very similar to water. Sampling was limited to the depth of the tank. Samples were collected and stored for laboratory analysis.

(a) (b)

Figure 5. (a) CST-AR1 field trials on a snow and (b) treated MFT.

4 DEVELOPMENT OF CST-AR2

Two key limitations were identified with AR1. First, the payload capacity was limited to approximately 40 kg. This was sufficient to carry standard geotechnical equipment and their control and data acquisition components, but is not sufficient for reclamation work that can require payloads of at least 100 kg. Second, turning with a twin-screw propulsion mechanism is challenging in hard ground and it consumes a lot of power. While regular operations are expected to happen in soft deformable terrain, there are many instances when the robot will need to maneuver precisely on hard ground, such as loading and unloading from a trailer and navigating on terrain accessible to the operators. A new solution for turning was required to increase maneuverability on hard compacted sand and gravel.

AR2 was developed to address these two limitations. In order to achieve higher payload capacities, the robot was increased in size. AR2 was approximately 40% larger in volume than AR1, with a total weight of approximately 450 kg (Figure 6). Larger screws were used to increase the payload capacity to 100 kg on water. Stability on water was maintained and the center of mass was sufficiently low to allow climbing of slopes of approximately 35 degrees, which is necessary to climb slopes surrounding DDAs.

Figure 6. CST-AR2 field trials on deformable terrain, Calgary, Alberta, Canada.

Mobility on hard ground was improved by using a double-twin-screw-type propulsion design (Figure 7). In this configuration four screws are synchronized to provide screw-type mobility on deformable terrain, or are synchronized as four large wheels to allow sideways rolling. The screw flight arrangement balances reaction forces to provide smooth predictable motion on all terrains. A key advantage is that differential steering (skid-steering) is possible while in rolling configuration. Field testing on deformable loose sand and had compacted soil demonstrated the advantages of screw motion and rolling motions. Increased maneuverability was achieved; the robot operator was able to precisely control the position and orientation of the robot during loading and unloading off a trailer, simplifying and reducing risk in the operation.

AR2 adopted several subassemblies and components from AR1. Similar DC motors with gearboxes were used to drive the robot. One motor was used per screw. The same power system, motor controllers, and on board computer systems were used. A new ruggedized commercially available controller was integrated. This provided customizable control for a wide range of payloads, and allowed the robot to be operated without the on-board computer, increasing the reliability of the system.

AR2 was designed to carry three new payloads: i) seedling planter, ii) seed broadcaster and, iii) wick drain deployment system. These payloads can be used for reclamation work, in which a deposit can be covered in vegetation, and to increase dewatering of DDAs.

A computer rendering of the seedling planter is presented in Figure 8a. The seedling planter was designed to drop standardized commercially available seedlings onto soft tailings. The planter consisted of six deployment belts, which were indexed to release a row of seedlings. The belts were spaced to place the plants approximately 25 cm apart. On soft material, the plants were buried by pressing them into the ground with the robot screws.

Figure 7. CST-AR2 field trials on hard terrain, Calgary, Alberta, Canada.

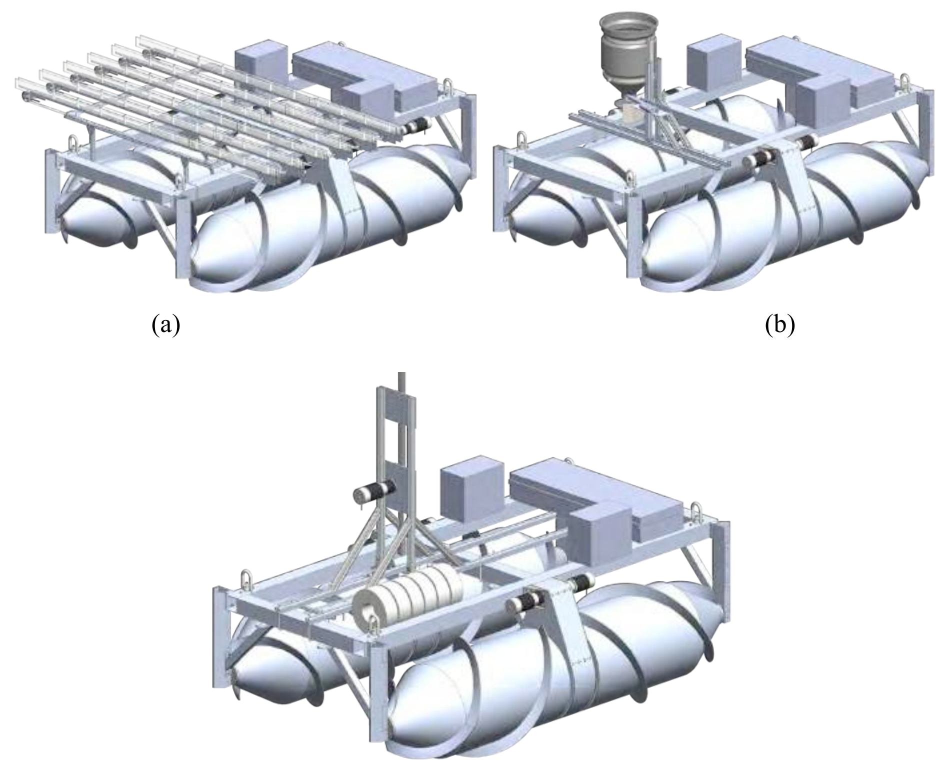
The seedling planter was built to carry in between 250 and 500 seedlings per trip. The area of the belts was the limiting constraint, not the weight of the seedlings. A multi-level belt system was been conceptualized but has not been built. For harder soils an indentation mechanism needs to be integrated to guarantee sufficient burring of the seedlings.

A commercially available seed broadcaster was modified for remote operation and integrated to AR2 (Figure 8b). The robot operator adjusted the gate opening by controlling a linear actuator, and the speed of the broadcaster by controlling a DC motor. Trial and error tests were necessary to find the correct combination for even spreading of the seed. Visual feedback was used to determine adequate seed dispersion. The maximum mass per trip was limited by the size of the broadcaster.

A wick drain deployment system was designed and integrated to AR2 (Figure 8c). Wick drains are typically installed by attaching them to anchor plates that are pushed into the soil. The payload deployment tower developed for AR1 was modified to lower a mandrel up to 4m deep and push customized wick drain plates. A mechanism was built to index up to 12 wick drains per trip. Each wick drain was pre-attached to an anchor plate. The anchor plates were designed to deform as they were pushed through the soil and to release from the mandrel when it was retrieved.

The three AR2 payloads were field tested with favorable results on a test cell DDA. The results of the field trials will be presented in a separate publication following the evaluation of the effects of the vegetation and wick installations on the dewatering performance of the test DDA.

Currently all payloads are controlled by the robot operator. This was adequate for initial proof-of-concept demonstrations where there performance of the equipment had reasonable uncertainty. Following successful demos, a key improvement is the automation of all payloads in combination of autonomous navigation of the rover. In that case, better performance is expected due to increased repeatability of operations. Fully automatic operations will require unmanned reloading of material. Future autonomous rover can be designed to collect material (seed, seedlings, wick drains) in automatic reloading stations and cover large areas without human intervention.



(a) (b)

(c)

Figure 8. CST-AR2 payloads (a) seedling planter, (b) seed broadcaster, and (c) wick drain deployment system.

5 CONCLUSIONS

The design and development of two amphibious robots has been completed. Field tests with AR1 demonstrated the feasibility of sampling and cone penetrometer testing in harsh environments. Better payload capacity and mobility were achieved with AR2. Three payloads for reclamation work were commissioned and integrated to AR2. Successful planting of seedlings, broadcasting of seed and installation of wick drains motivate further development of robotic systems. Future work will address challenges of full autonomous operation.

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50 Years of Successful Learning by Experience: Suncor Tailings Geotechnical

E.C. McRoberts
Amec Foster Wheeler Environment & Infrastructure, Burnaby, BC, Canada

T.I. MacGowan & N. Eenkooren
Suncor Energy Inc, Calgary, AB, Canada

G.W. Pollock
Amec Foster Wheeler Environment & Infrastructure, Edmonton, AB, Canada

ABSTRACT: Suncor Oil Sands (then the Great Canadian Oil Sands) opened on 30 September 1967 with a mining and refining operation producing 45,000 bbls/d; today, Suncor base mine achieves production in the order of 315,000 bbls/d. This order of magnitude increase did not come easily, as there were many technical, economic, environmental, regulatory and even political challenges during the 50 years which have been successfully managed by Suncor. Perhaps, not least of which were the challenges faced by the several generations of geotechnical engineers who had to not only deal with the usual range of technical factors–but who had to develop and maintain technical skills and techniques in the face of the other challenges. A key component of geotechnical learning from experience is the appropriate application of Peck's Observational Method and successful learning through understanding the imperatives of the business and operational planning, and managing the risks of short term gain versus long term pain. This paper will describe 50 years of successful learnings in mining dam design under these influences at Suncor.

1 INTRODUCTION

Suncor Oil Sands (then the Great Canadian Oil Sands) opened on 30 September 1967, with the first tailings structure construction started in 1966. The operation started with one pond with a planned containment height of 12 m to a changed planned final height of 98 m. Along the journey from one tailings pond to eleven (nine operational and two partially reclaimed) and from 45,000 barrel per day (bpd) to 315,000 bpd, the geotechnical designs have been challenged by the imperatives of the business and operational plan, the pressures on operations when production is threatened, changing regulatory environments, the risks of short term gain versus long term pain as well as all the geotechnical issues with containing significant volumes of fluids on weak soils. At one point early in the history of Pond 1, had hydraulically placed sand as containment not been deployed, the future learnings may not have been possible. This paper summarizes some of the learnings over the 50 year operational history. This paper was also taken as an opportunity to attempt to create a comprehensive list of geotechnical papers written on the many tailings structures of Suncor, although as it turns out it is not exhaustive.

2 MULTI-DISCIPLINARY DESIGN

Over the last 50 years geotechnical engineers working on Suncor mining leases have seen a range of new tailings technologies come and go. But the corner stone of mine development remains the same as developed at Suncor. To use overburden soils or segregated sand tailings streams to build retention dams, and deal in some manner with exceptionally intractable properties of the fines stream termed mature fine tailings (MFT), slimes or sludge per general terminology in the mining industry, all the while ensuring that design meets the type and availability of overburden material. These competing needs of development within the mine set up a situation for a truly multi-disciplinary team.

One of the lessons within a multi-disciplinary team is that tension between research, planning, and geotechnical groups can be necessary and productive. Over the years this tension has led to many cost savings and innovations, as mining methods and equipment types have changed, and as new tailings technologies are developed. The geotechnical perspective is that research and tailings planning by its nature edges towards optimism; whereas geotechnical, especially given the implications of being wrong, is much less sanguine. The concepts occasionally put forward by tailings research require significant lead time as, while attractive on the drawing board, they can often flounder on the shoals of geotechnical engineering and operational realities.

When a new technology is introduced "downstream" of extraction, variability inherent in requirement for continuous bitumen production militates against consistent tailings production. The geotechnical engineer must be cautioned against designing for the expected nominal operating condition. Understanding the influence of variability on the design is critical to understand what changing factors can impact the design. As the tailings stream reporting to the dam structures may not be consistent, designing for variation or some degree of uncertainty will necessitate the healthy tension between design, planning and operations to determine the optimum design/risk. This may also require operational commitments on factors outside of the geotechnical engineers' control. Designs must accommodate upsets, and current experience suggests variability will be the rule for any step-out technology in-line with extraction.

The exigencies of the business plan reflecting optimization of dyke placement schedules, changes in pit sizes and bitumen production driven by major swings in oil prices, and demands for storage resulting from unforeseen tailings performance are pressures bearing on the geotechnical designer – who may at the same time be wrestling with new and problematic site conditions. This is often reflected in requirements to modify or change a dam's design section (even before construction has begun) and to raise dams under construction to higher elevations. Pollock et al, (2014) discussed the influence of changing mine plans and new technologies on specific geotechnical designs at Suncor.

Extraction processes, reflecting variability in ore type, fines contents and process-ability, combined with start-ups and shut-downs requiring flushing of lines, result in a highly variable end-of-pipe whole tailings stream. The original method of dealing with wet segregating tailings with compacted cells and beaches offers a highly robust method of dealing with these unavoidable variations, but with the creation of mature fine tailings (MFT) – which we will return to.

Planners are faced with balancing the waste streams created by mining ore including, fluid fine tailings (FFT), water, MFT, coarse tailings, and overburden. They must balance ease and short hauls against the cost of tailings containment. Not an easy endeavor when facing economic pressures to reduce operating costs. Overburden short hauls can be very attractive in the short term. These competing needs drive various designs for the geotechnical engineer and can result in a design three times-build once reality

Given the cost of dealing with tailings, the geotechnical engineer faces on-going challenges to defend the design bases. Experience indicates that issues are the rule not the exception.

3 TAILINGS

In a conventional extraction scheme whole tailings are pumped at relatively low solids contents and the tailings stream separates on deposition into mostly sand and some trapped fines, with the remainder of the fines and some sand collecting in a pond as sludge or MFT. Suncor (then G.C.O.S) pioneered the first tailings containment facility at Tar Island Pond, Pond 1. In the early stages of project development the creation of MFT was a surprise. That, and the fact that achieved beach slopes were much flatter than bench scale flume testing resulted in a structure originally conceived to be built out of overburden to a height of 12 m, finally topping out at around 98 m by incorporating into the design upstream sand construction. This represented the first learning relating to surprises in tailings planning and the unreliability of blindly scaling up flume tests to commercial scale – a lesson that keeps being re-discovered 50 years on.

3.1 *Fluid Fine Tailings*

The most intractable component of fluid fine tailings is MFT, a witches' brew of highly dispersed clay minerals and bitumen both of which can engage in complex ionic exchange reactions that defy any geotechnical attempts to predict behavior by the application of Terzahgi's principle of effective stress. The water based extraction process uses warm water and agitation to clean bitumen from oil sands which results in clay dispersion. Unfortunately, the effect on the clays is difficult to reverse. It is no exaggeration to say that the problems with MFT which were first evident in Tar Island Pond very early on, and after the expenditure of billions, remains today likely the leading topic for research and development at Suncor and other oil sands leases.

McRoberts (1999) reported on the performance of MFT in Pond 1A at Suncor focusing on the importance of creep in understanding MFT behavior. More recently Wells (2011) updated this case record and concluded that after the initial densification of MFT transferred into this pond that no additional densification over the last quarter century has occurred, and that self-weight settling and consolidation is too slow to measure. McRoberts (1999) showed that application of the principle of effective stress with measured MFT properties could not explain performance and the same result was clear in the well reported 10 m column of Syncrude MFT at the University of Alberta, and concluded that creep, dominated the performance., Scott et al (2013) This is not surprising, Terzaghi (1952) comments that the variety of processes constituting the secondary time effect (i.e., creep) indicates that these phenomena are produced by at least two independent processes governed by different laws (i.e, not his principle of effective stress).

The creation of MFT points to an important factor in FFT, the bulking factor. That is to say the net volume occupied by tailings wastes is about 1.4 of the original volume. Complicating the bulking factor of FFT, is the need for a clean water cap, for clarification, on several ponds for process water return to extraction. As well, all precipitation that contacts oil sands, dyke drainage waters, run-off from waste dumps are collected and recycled. All are stored in tailings ponds and increase water in inventory. Mudding these waters with their accompanying ionic content, they can also strongly impact both settlement and consolidation characteristics of clay sized tailings, MFT consolidation. The requirement for clarification of fine clays to meet process recycle water requirements of low clay content leads to significant pond areas. In some cases multi-use of pond space and especially with clarification requirements means that water is generally impounded against the upstream face of the retention dam. This can be readily accommodated with an upstream method and subaerial beaches but introduces design and construction complications for overburden structures, particularly with internal erosion and piping. If an understanding of the water balance, including FFT and other complicating factors, is not well understood, dam structures can be under pressure to change designs to allow for increased fluids with less beaches; historically, this has been a pressure on dam structures.

3.2 *Coarse Tailings*

As a result of the learning of the presence of MFT, and complicated by lack of storage space elsewhere the height for Tar Island Dyke, Pond 1, went from 12m high to ultimately 98 m. This was the first ex-pit structure and there was no in-pit space available. There was also insufficient space and overburden material available to sustain the operation until a new tailings structure could be constructed in-pit. Pond 1 therefore set the stage for a 50 year, so far, odyssey of the competing needs of containment, design and availability of footprint and material.

In the late 1960's, starting with Tar Island Dyke many of the dams built were designated as Modified Upstream construction. These techniques developed by Dean R.M. Hardy, Sig Winzer, Suncor's Mine Engineer, and an unsung BC dredge spoil disposal engineer at Tar Island Dyke are still being applied today.

3.2.1 *Hydraulic Sand Construction*

Working in the 1960's Dean RM Hardy recognized the potential for liquefaction of the downstream section of Tar Island Dyke; in this regard the failure of the Fort Peck Dam in the 1930's as well as other hydraulic fill dams in the USA served as examples. He consulted with Professor Arthur Casagrande to develop a cross section of sufficient compacted cell width to contain beached tailings sands if it liquefied; see Hardy (1974), Mittal and Hardy (1977), and Chan et al

(1983). This basically introduced the precautionary principle, still generally followed today in hydraulic fill construction in the oilsands, that liquefaction should be assumed however potentially unlikely the triggering event might be.

There have been several liquefaction events in oil sands tailings ponds in the early days of lease development. These failures occurred on the upstream or pond side, when either beach or compacted cell sands were rapidly placed over loose beach below water (BBW). Less well known in Pond 1 were failures of very flat beaches above water (BAW) placed over BBW. Some of these failures are discussed by Mittal and Hardy (1977) and in more detail by Plewes et al (1989) who provide operational guidance in avoiding these types of beach failures. In Pollock et al. (2014 and 2014a), more recent designs of tailings structures have doubled the normal rate of rise of sand, increased the lift thickness from 3 m to 5 m and have built cell in winter, thereby breaking some of the guidance previously provided. But breaking rules generally require some offset, which took the form of engineering, drains, larger cells, dozer compaction and longer drainage time, all with significant success to date. One structure presently has 75 m of placed sand in 5 years which ultimately will have 130 m of sand in an initially inpit structure.

The upstream method has proved to be robust and economical. The major drawback is that unacceptable amounts of MFT have been generated. As well, methodologies to reduce MFT volumes are expensive, whether through removal or reclamation. For some upstream ponds the plan is to remove the MFT leaving a vast bowl formed from tailings sand that must eventually include building-in an outlet for run-off. These plans will no doubt provide a new range of geotechnical challenges once they reach the implementation phase in dealing with the drawdown of the ponds, and the long term stability of major planned breaches of the former supporting shell.

3.2.2 *Liquefaction*

If sands do not meet certain minimum levels of densification, they are considered vulnerable to liquefaction and must be assigned a liquefied shear strength. Assigning an appropriate strength to liquefied sand has been the topic of considerable debate in geotechnical practice, and is still not fully resolved. Given that the onset of this condition cannot be practically managed by the observational method, prudence is required in setting liquefied design strengths.

Ongoing research over the last 20 years indicates the ability of loose sand below water (BAW) to self-densify with increasing effective stress, McRoberts (2005). This is an important factor that can be considered in design, but must be proved-out under site specific conditions, and generally results in improving the long term stability as opposed to an initial design parameter.

The seismic risk levels are low in McMurray reflecting the tectonically quiescent nature of the interior craton. Structures designed to accommodate the possibility of static liquefaction and which meet standards for static stability will always prove to be able to withstand realistic seismic loadings. Current regulatory practice appears to be driving detailed seismic analysis for all tailings dams which is essentially redundant for structures designed to withstand static liquefaction.

3.2.3 *Seepage*

The provision of internal drainage has been a central feature of slopes constructed from cell sand. The main failure mechanism for uncontrolled seepage on sand slopes is spring sapping leading to gully formation. Oil sands tailings are highly erodible and once gully formation begins the focusing of subsurface flow into a gully from both seepage and runoff results in rapid progression of a gully. Other surface disruption mechanisms include heaving due to pressure build-up under frost penetration layers and lower permeability reclamation covers. In order to provide local surface stability against these mechanisms, internal drainpipes are required to keep the phreatic surface well contained within the slope, and where they have been deployed good performance has been achieved.

All tailings types, including sand, are also potentially vulnerable to internal erosion. There has been no evidence of such a mechanism developing in oil sands tailings slopes with 'roofing'; that is, where the sands above the phreatic surface are able to collapse into the erosion "pipe". However designs which contemplate overburden over sand on a downstream seepage face could readily permit a roofing mechanism to develop, and should be avoided.

Control of these mechanisms can be obtained by internal drains installed to maintain the phreatic surface well within the sand slope, and where flow is directed to appropriately designed drains. For the filtering component of the drain, design is vital to ensure that sand loss into the drainpipe does not occur. This controls the potential for the internal erosion mechanism. The first drainpipes installed in Tar Island Dyke consisted of perforated galvanized metal pipe with a coke surround. The coarse coke produced at Suncor was selected as an economic alternative to gravel, and met the filter rules for tailings sand. Practice at Suncor then evolved to use HDPE pipes and a filter fabric and common practice is to now use the pipe/fabric system, sometimes with a backup granular surround.

Seepage analysis in support of drainage designs is common practice. In most cases however flow is dominated by downwards seepage during cell and beach placement, and not flow from the pond. For most sand structures the main flow out of the dyke is not from the pond. In fact the presence of MFT and floating bitumen creates seals against the beach limiting the flux out of the pond and into the system. Generally, it will be found that after cell construction ceases the flow to the drainage systems is primarily due to the storativity of the cells and beach, infiltration of fresh water, and some flux through the developing pond bottom seals.

5 RISK MANAGEMENT

Most of Suncor's structures are major tailings dams, the failure of any one of which is not ponderable. At the same time there is continued pressure to innovate, drive down costs, and deal with environmental issues. But the geotechnical engineer must deal with uncertainty in a risk adverse manner. While it has been said many times, the fundamental method of proceeding is the observational method, Peck (1969) which provides a powerful method for dealing with geotechnical uncertainty – if properly followed. The method allows the design to be based on reasonable parameters, to monitor, and to execute pre-planned contingency measures if performance is outside of tolerable limits. The method also explicitly requires that the monitoring and execution plan can be implemented within the constraints of the mine and tailings plans. That is, if the contingency plan cannot be practically executed due to timing, lack of resources or alternative tailings storage the method cannot be adopted, and a more conservative design must be implemented. It is also stressed that the designer must clearly communicate the need for a comprehensive monitoring plan, and timely observations and interpretation of key performance indicators. This was not often fully understood – and occasional instrumentation was greatly reduced to save money – after the savings for an observational approach were first taken to the bank. Inherent in the observational method is an understanding of worst case parameters for which mitigations must be developed. McRoberts et al (1995) report on the application of the method for Tar Island Dyke.

The observational method is a powerful tool for design, but it is not suitable for considerations of internal erosion / piping within overburden structures, for some issues around the control of static liquefaction and brittle type failures. Ultimately, if it cannot be monitored, it must be designed for. Many foundation issues encountered in oilsands lend themselves to the observational approach and it has been successfully applied at most structures.

6 FOUNDATION

The presence of what geotechnical engineers preferentially term a "clay-shale" such as the Clearwater Formation (Kc), as well as certain clay rich facies in the McMurray Formation (Km) above and below ore, contains highly sheared zones with very low residual strengths. The original west bank leases where Suncor's early mining began had limited zones where Clearwater existed and the ore body was found directly on top of limestone. This is just as well as Skempton's (1964) definitive assessment of shear strain effects leading to residual strength was just beginning to be understood in practice. These shear zones have been induced by glacial drag as well as structural downwarping of the basal limestone formation. Syncrude's experience with initial failures at the Mildred Lake Settlement basin (MSLB) rapidly informed the local geotechnical knowledge base, and by the time major Clearwater and McMurray shear zones were encountered on East-bank Steepbank and Millennium Leases, a sound data base of knowledge existed. Problems with these units are managed by either very flat slopes or excavation and backfilling of shear keys, or both, along with the Observational Method. Treen et al (2002) discuss the design of a shear key for the Wood Creek Dam, part of Dyke 11A, which was raised to a height of 120 m over a foundation of weak basal clays with residual friction angles of 6.5 to 12 degrees. A shear key which has yet to be required as monitoring of the area has not indicated unexpected performance.

Not only are the Kc found to be pre-sheared and exhibit low strength near the residual angle, these clays can exhibit a range of pore pressure induced by construction and loading. Adopting conservative strength and pore pressure parameters and a minimum acceptable Factor of Safety (FoS) of 1.3 in accordance with Canadian Dam Association standards can in principle readily lead to slopes as flat as 17H:1V to 20H:1V, especially if the Quaternary cover in the passive toe area is relatively thin. In-pit dams can also require flatter slopes to manage weak basal clays or pond muds within the Km, and Paleo-clays at the limestone contact. Because of the large body of knowledge, experience with the movement and the use of the observational approach, the slopes used in design tend to be somewhat steeper with contingency planned.

The on-going foundation movements experienced at Tar Island Dyke, see Chan et al (1983), revealed highly slickensided shear zones in a Holocene clay deposited post glacially in the Athabasca river valley. No slickensides had been noted in the original circa 1965 investigations for a planned 12 m dyke and while the importance of such features might not have been appreciated in 1965 there was no credible mechanism for a pre-construction (i.e. geological) origin for the slickensides. This case record is reported in McRoberts et al (1995) who concluded that differential settlement of an originally lightly overconsolidated clay, but now brittle normally but highly consolidated clay lead to the development of strain localization, shear bands and therefore significant post-peak weakening.

Several strategies have evolved to deal with the design uncertainties introduced by designing based on stability and FoS, and monitoring based on deformations. The current state-of-art is lacking a well-developed, coupled analysis between deformations and stability for construction over clay-shales. How much deformation is considered excessive involves a considerable degree of judgment and prudence, given the consequences of failure. However, one strategy which can at least reduce initial capital expenditures is to start with relatively steeper slope as indicated by case records. The structure is monitored and in the event of movements, pre-planned contingency measures are executed. This approach requires that toe areas reserved for stabilization berms are clearly identified. The other option available is stepping out over upstream beaches into pond and this option has been used, however this reduces available tailings volumes.

7 CONSTRUCTION

7.1 *Heavy Haulers and Overburden Construction*

The last 40 years has seen the development of bigger mine trucks with both positive and negative geotechnical implications for overburden construction. Excavating many of the dry fills such as Kc and clay rich Km intraburden by large shovels with cuts 10m high or more results in a fill with many high strength blocks of cobble to boulder size clays. While some of this blocky fill is broken down by the D-10 Type Dozers used to spread fill, trafficking by loaded heavy haul trucks is necessary to meet compaction specifications and ensure a tight structure-less low permeability fill. This cannot be readily accomplished by smaller mine trucks. On the other hand, fills wet of optimum water content are highly sensitive to pore pressure generation and heavy hauler loading can quickly result in rutting and trafficability problems. Experience indicates that construction guidelines developed for conventional earthworks can be misleading in marginal fill types using heavy haulers, and when combined with optimistic interpretations of borrow quality can lead to considerable start-up difficulties.

A significant lesson learned in the development of all-year overburden placement for dyke construction was trafficking drier fills with heavy haulers result in a far superior density product than could be achieved with conventional compaction procedures. Use of heavy haulers to place fills wet of optimum resulted in rutting, which tended to limit the upper end of allowable moisture content. But, this was also viewed as an advantage as it decreased the sensitivity of induced pore pressures under high dyke loadings – which led to the rule of thumb that if you cannot drive a heavy hauler over the fill you should generally not be trying to build structural sections of a dam with it.

Overburden obtained by pre-stripping the opening mine cut is a potential source of material for starter dam construction as this material must in any event be moved to access the ore body. In addition CT dominated tailings plans requiring sand to sequester MFT and to cap the CT means that in-pit dams require considerable select overburden for zoned fills. There is considerable pressure on the geotechnical engineer to use all fills coming from these operations, including construction of in-pit dams. However, overburden quality varies from mine to mine, and a lack of understanding of local conditions generally has, and still does lead to over-optimistic assumptions on borrow availability. This in turn drives tailings storage demand to requiring tailings sand for dyke construction, directly impacting sand available for other uses.

An overburden starter dam usually provides several functions. Initially, the first external dam more than likely has to retain a significant volume of water. As governed by licensing approvals, water can only be obtained during high flow periods from the Athabasca River, and main tributaries, therefore a substantial inventory must be carried prior to start-up of extraction for a new mine. As tailings are placed behind the overburden dam, some time must elapse before tailings beaches can be raised above water [BAW]. Until this time and resulting elevation water is impounded directly against the overburden structure. Therefore internal erosion or piping of an overburden dyke is a design concern, requiring a chimney drain, or other control measures. In addition BAW sands are needed in order for compacted cells to stepout over tailings in the upstream direction. Beaches placed below water [BBW] are exceedingly loose and subject to liquefaction. Consequently, the overall tailings dam section will contain loose liquefiable sands within the design section. The second function of the overburden dam is to provide sufficient width and strength, in conjunction with the compacted cell sand to retain potentially liquefied BBW. Finally, the overburden must be internally self-supporting. For sites with weak Kc clays for example, this means that the overall slope angle may be relatively flat [say 12H:1V to 20H:1V] and therefore relatively poor overburden can be tolerated in some parts of the starter dam. Although, as discussed above, slopes this flat are generally managed through experience, the understanding of other existing slopes' performance and the observation method.

Overburden required within a tailings retention dam must meet geotechnical specifications in terms of mobilized strength. The types of overburden vary from lease to lease depending on the Quaternary geology and the presence or absence of Kc formation over the ore body. Once a mine is developed beyond the opening cuts then intraburden within the ore body also becomes available. A wide range of wet to dry fills are therefore possible. Generally footprints for tailings retention are always constrained by setbacks from natural features, mine pits, and other facilities. The geotechnical designer is asked to construct dam slopes as steep as possible with the most immediately at hand overburden. Where overburden fills are wet relative to optimum water contents and foundations conditions are poor, some amount of wet fills can be tolerated. On the other hand a site with good foundations but wet fills and limited tailings space requires steep slopes which may mean that a lot of wet fill must be rejected and wasted to dumps. This in turn puts additional pressure on dumps, and competing demand for plating to raise the dumps. The lesson here especially for planners is that not all overburden is created equal as it were, and overburden might be a real asset in one area; but a curse in another.

7.2 *Winter Construction*

The winters in Fort McMurray are long and cold, and the summers are often wet. Limiting construction to the traditional summer months was an early premise for construction of overburden dams in Fort McMurray, reflecting conventional practice. However, this is a severe limitation, as either equipment would be left idle for at least the four core coldest months or overburden material suitable for dyke construction would be wasted. Both of these have substantial economic implications.

The use of cold weather winter fill was pioneered at Suncor, see McRoberts et al (1983) and has been used extensively since that time at Suncor, as well as other leases. This approach began at Suncor following from the observations that shells of winter constructed waste dumps had more favorable geotechnical properties (SPT blow counts and pore pressures) than summer fills built in the wet weather with more conventional lighter highway type equipment. The development of heavy haul trucks has favored this application as insitu ground temperatures can be retained within the large volume of fill being transported, and the fill can be quickly compacted by the haul trucks once it is spread by dozer.

For low permeability overburden structures the time to develop a steady-state seepage regime is likely in the order of many decades or longer. The real threat is flow through construction defects, more likely to occur with winter construction or frozen material. Cracks can develop with any configuration short-circuiting the presumptive transient and steady state flow paths to drainage boundaries. The primary control system is a chimney filter-drain that intercepts all possible defect dominated flow paths, and an offtake system that ensures the chimney can drain. The chimney basically must function as a "crack-stopper" in the sense that the filter component stops eroded fines at the overburden to filter contact.

7.3 *Abutments*

Abutments designs are primarily an issue for in-pit dykes. Abutments can either be the pit wall or in some cases a previously constructed dyke where a T-Alignment must be used based on the mine plan. One interesting aspect of the oilsands is the effect of insitu stress release. Insitu Ko stresses are considered to exceed unity and possibly significantly higher. Stress release consequent on mining can cause both shear band propagation along weak facies, and gas exsolution. These mechanisms can result in exfoliation type cracking behind the oilsands face creating jointing that offers potential leakage paths from the pond to the downstream.

With the advent of shovel mining the typical pit slope consists of a stair-step series of falldown shovel slopes, and mine benches. In highly gassy ores, the oil sands slopes can flow or creep likely increasing the mass permeability of the slope thus affected. One measure that is taken in design is to flatten the abutment slope and creating a notch within which the low permeability element of the dam core can be inserted. This requires advance planning to ensure that the necessary setbacks are available.

Typically fill compaction is provided by heavy haul trucks. Given the size of these trucks, special provisions are required so that the truck wheels can compact right up to a prepared mine face. This requires that the fill dozer places a small ramp up the slope and backs up the ramp, into the slope.

7.4 *Hydraulic Sand*

As discussed briefly in 3.2.1, several construction controls have been in place for tailings construction to deal with potential liquefiable failures. Key learnings from Plewes et al (1989) were around lift thickness, rate of construction, drainage, overboarding of tailings onto beaches and deposition into water. At the time, winter construction was also not undertaken because of the fog created from the hot tailings stream into the cold weather. With the high rate of rise required for Sand Dump 8, all but deposition into water were changed (Pollock et al. 2014a).

Early issues with drainage and pushing up frozen saturated side dykes revealed that side dykes that provided containment, must be pushed up while the sand is not frozen as the hot water can melt the frozen lumps leading to a small break in a dry dyke.

8 DESIGNING FOR CLOSURE

In a typical operating mine, tailings can be found both externally or in-pit. In new currently licensed greenfield operations an external tailings pond is necessary in order to store tailings until in-pit space is available. For closure, external tailings ponds must be converted to a terrestrial landscape. Removing MFT for closure results in either infilling with more sand or breaching the dyke to eliminate future fluid storage potential. The first tailings area in Alberta's oil sands region to be fully reclaimed is Pond 1, which was accomplished by transferring MFT for treatment elsewhere and simultaneously infilling with coarse sand tailings. Suncor's Pond 5 surface was subsequently made trafficable by installing a semi-anchored floating engineered cover that includes, geogrid covered with coke while leaving some MFT with soft tailings in place and working to reduce the water in the this material with wick drains, discussed in Pollock et.al (2010). While proving to be effective, it has been time consuming and expensive. Closure challenges relating to consolidation behavior of the soft tailings remain ongoing for geotechnical engineers.

With Suncor's more recent tailings reduction operations, Suncor changed the tailings approach and designed directly with the end in mind. The end being a readily reclaimable, decommissionable tailings sand structure. With the fluid pond kept small in a sand dump, and TFT immediately removed, MFT was generated and treated elsewhere. The unimpeded perimeter drains would more effectively provide the necessary drainage for the structure without the usual MFT layers blocking the drainage paths as discussed in Section 3.2.3. The final closure scenario for this pond is simplified to just removal of the relatively small volume of fluids and reclamation of what is then a solid, stable landform.

9 INFLUENCE OF REVIEW BOARDS & REGULATORY AGENCIES

In the very early days, circa 1967, the only informed opinion as to the quality of water that could be discharged appears to have been the local Board of Health Medical Doctor. Things then changed and for the better. By 1977 the first dam safety design review in the oilsands was Department of Environment commissioned the first design review in the oilsands, undertaken by Professor N.R.Morgenstern for Alberta Environment , and was the predecessor for the required design safety review (DSR) now regularly being undertaken. This first review significantly strengthened the ability of the geotechnical engineer to insist on the budgetary support for appropriate monitoring with such new-fangled devices such as slope inclinometers and pneumatic piezometers. Today this is standard practice and well supported by Lease Operators. Subsequently Canadian Dam Association (CDA) guidelines and the Mining Association of Canada (MAC) guidelines came into existence and which more specifically addressed the role of geotechnical engineers and senior management, respectively. At the same time the Department of Environment established and strengthened a wide range of environmentally focused regulations – but that is another story to be told.

Initially at Suncor, external reviewers were consulted by Suncor and its consultants on an informal basis depending on specific project needs. This process was eventually formalized at Suncor by the creation of the Mine Development and Reclamation Review Board (MDRRB). As the MDRRB currently meets twice a year, there is significant review of the design during the process of development which provides timely input. Waiting until the design is complete for review can lead to delays in the project to deal with technical issues or unrealized potential. The power of external cold eyes review is now well understood by all concerned and is a major learning over the past several decades.

10 CLOSURE/SUMMARY

There have been many geotechnically related learnings that have helped oilsands be successful as an economic driver of the Province of Alberta and contributor to the Canadian GDP. Many of which are discussed within the paper and show the diverse issues that must be successfully navigated to provide the robust tailings structures necessary. It also shows that while some issues remain, there have been significant improvements made in geotechnical engineering, with more to come. Lease Operators today well appreciate the potential downsides of geotechnical related risk taking. However constant vigilance is required to maintain the success of the last 50 years.

One of the most important learnings can be seen in failures of other structures elsewhere in the world. This is that a highly integrated team effort and the success of an individual geotechnical structure relies on the operational discipline of planning, technology, operations, geotechnical engineering and regulatory bodies.

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There are too many people who participated in the learnings of the past 50 years to possibly enumerate. Some have passed on. To those around today, you know who you are, thank-you.

LIMITATIONS

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Log Pipeline Technology for Clay-Based Tailings

Baha Abdulnaga
Splitvane Engineers Inc., Sumas, WA, USA

ABSTRACT: The technology of log pipelines was developed extensively in the 1960's to the 1990's in Alberta, Canada and Missouri, USA for the transportation of goods in capsules and coal logs over long distances but was never considered for mine tailings. The concept was revisited by Splitvane Engineers Inc in the last three years. Efforts to apply this technology for the placement of clay-rich tailings, two new parameters emerged to maintain the log, the shear strength and the plasticity index. The importance of the Plasticity Index was measured through ablation tests on 100 mm diameter clay logs.

1 INTRODUCTION

The mining industry is facing the challenge of reducing quantities of water used to store tailings and mine waste. A number of tailings dam failures have caused havoc and numerous fatalities, and environmental disasters across all continents in the last 50 years by pouring large volumes of water and solids. Numerous alternative technologies have emerged from thickened tailings, flocculated tailings, paste filling, dry stacking to reduce water volumes.

To understand the correlation between the plasticity index and resistance to ablation of a log of clay, an ablation machine with a rotating drum was built. Binderless logs with a diameter of 100 mm where tested at for clays with the plasticity index of 7.8% (clayey sand), 32% (grey clay) and 43% (red clay) at various drum tip speed magnitudes, to simulate the the Liu's lift-off velocity for flow in tailings pipelines.

Although the pneumatic capsule and hydraulic log pipeline technology was developed for moving goods and coal over long distances, it can be adapted for the placement of certain tailings. Canadian oilsand clays typically have plasticity index values between 22% and 33%, while Phosphatic clays can reach values of 97% to 239%. Most tailings placement sites are within few kilometers of the mineral process plant. Understanding the degradation of the log at 85% of the lift-off velocity opens new opportunity to this 60 years old technology that has not been applied yet for solving some critical problems of tailings placement.

2 CONVENTIONAL EQUATIONS FOR HYDRAULIC CONVEYANCE OF RIGID LOGS

The concept of capsule pipeline was extensively investigated by the Alberta Research Council between 1958 and 1978 (Brown, 1987). The concept was further developed as a concept to convey logs of coal by the University of Missouri (Liu 2005).

Some concepts were tested for coal:

- Highly compacted logs without binder (Binderless logs)
- Compacted logs with asphalt as a binder

Binderless logs require very good control of the particle size and the moisture content. This concept would not apply readily to mining tailings and waste as most processes whether by filtration or thickening leave a non-negligible amount of moisture. Dry tailings or waste are limited to some coal and mineral sand processes.

Binderless coal log fabrication occurs at high pressures of 10,000 to 20,000 psi (70 MPa to 140 MPa). Intervoid air must be removed as it tends to compress and resist binding. Ejection of the log from the briquetting press must be done carefully to avoid cracking the briquette.

Liu (2005) defined 4 regimes for hydraulic conveyance of capsules:

- Incipient Velocity V_i, at which the capsule starts to slide along the pipeline floor, but the average velocity of the carrier liquid, is still above the velocity of motion of the capsule. Liu defined this state as *Regime 1*.
- As the velocity increase into *Regime 2*, the capsule moves. The total drag is a combination of pressure drag and skin drag due to shear. The drag is balanced by the friction force encountered by the sliding capsule. As the velocity increases, the capsule starts to experience a hydrodynamic force, a sort of lift that combines with the buoyancy force to reduce the normal force on the wall of the pipeline, due to the weight of the capsule.
- Once the velocity reaches a critical value V_{crit}, the flow enters into *Regime 3*, the contact force becomes smaller than the drag force. The capsule starts to accelerate and the shear force develops opposite to the direction of motion
- With further increase in velocity, the capsule lifts-off and ceases to be in contact with the bottom wall of the pipe. The flow enters into *Regime 4* at a velocity called lift-off velocity V_L. Experiments on coal logs indicate that the log move then at 15% higher velocity than the fluid bulk velocity.

In the lift regime, the capsule becomes unstable; it may roll against the walls of the pipe and endure damage. Liu (2005) therefore recommended limiting flow to 85% of the lift-off velocity.

2.1 *Incipient Velocity*

The cross sectional flow diameter being D_i, the pipe inner area is

$$A_p = \frac{\pi}{4} D_i^2 \tag{1}$$

The log frontal area A_c for a cylindrical capsule (or log) is defined in terms of the capsule or log diameter D_c

$$A_c = \frac{\pi}{4} D_c^2 \tag{2}$$

A diameter ratio k is defined as

$$k = \frac{D_c}{D_i} \tag{3}$$

To avoid jamming at bends k<0.90.

Defining L_c as the length of the log, the log aspect ratio is defined as

$$a = \frac{L_c}{D_c}, \tag{4}$$

To avoid roll-over of logs in the pipeline that may cause blocks, 1.2<a<4.

Logs with a>4 are difficult to manufacture.

The incipient velocity (Liu 2005) is calculated within 10% error margin as

$$V_i = \sqrt{\left(\frac{\rho_c - \rho_f}{\rho_f}\right)\frac{2gL_c(\sin\alpha + \phi\cos\alpha)}{C_D}} \tag{5}$$

Where
α = angle of inclination of pipe (positive for upward, negative for downward)
ϕ= coefficient of contact friction between capsule or log and pipeline, usually considered to be in the range of 0.45 to 0.55
ρ_c = density of capsule
ρ_f = density of fluid surrounding the capsule in the pipeline
C_D = drag coefficient for capsule or log

$$C_{Di} = \left[1 + K_c + f_a\left(\frac{L_c}{D_i - D_c} - 3\right)\right]\left(\frac{D_i^2}{(D_i^2 - D_c^2)}\right)^2 \tag{6}$$

The Darcy-Weisbach friction factor f_a is determined by using the Reynolds Number for a co-annular flow or by computing the equivalent Hydraulic diameter

$$D_H = D_i - D_c$$

The Reynolds Number is defined as

$$\mathrm{Re}_H = \frac{\rho_f V_f D_H}{\mu_f} \tag{7}$$

V_f= fluid velocity
μ = fluid kinematic viscosity

The term K_c is called the head loss coefficient and is defined in terms of the contraction coefficient for the flow around the log or capsule

$$K_c = \left(\frac{1}{C_c} - 1\right)^2 \tag{8}$$

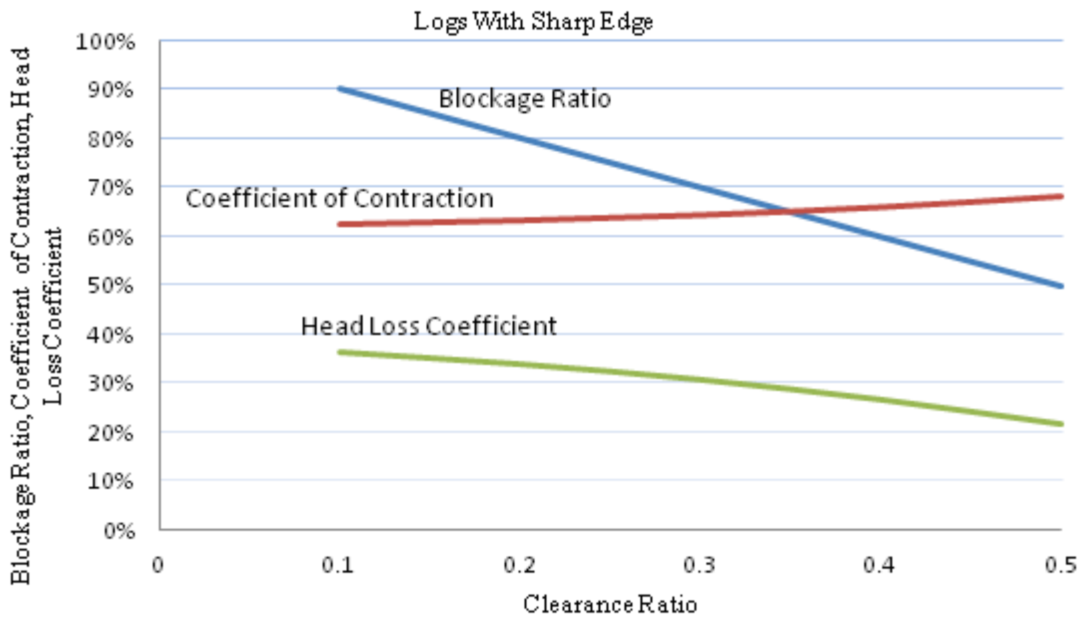


Figure 1. Contraction Coefficient C_c and head loss coefficient K_c and blockage Ratio for logs with sharp edged (90^o corner) entrance (upstream end) – plotted after data from Liu (2005)

2.2 *Critical Velocity*

The critical velocity occurs at the transition from *Regime 2* to *Regime 3*

$$V_{crit} = \sqrt{\left(\frac{\rho_c - \rho_f}{\rho_f}\right)\frac{2gL_c(\sin\alpha \; + \phi\cos\alpha)}{kaf_a}\left(\frac{D_i^2 - D_c^2}{D_i^2}\right)} \tag{9}$$

2.3 *Lift-off Velocity*

Liu's equation for lift off velocity

$$V_L = 7.2\sqrt{\left(\frac{\rho_c - \rho_f}{\rho_f}\right)gak(1-k^2)D_i} \tag{10}$$

V_L= lift-off velocity (m/s)
g = acceleration due to gravity (9.8 m/s^2)

3 ABLATION TESTS ON LOGS OF CLAYS

Numerous clays have a specific gravity of 2.65. Considering a capsule or rigid log of clay with a specific gravity of 2.65 at the diameter ratio of 0.9, Liu's Lift-Off Velocity is plotted in Figure (2) against pipe diameter, for aspect ratios of 1 to 4.

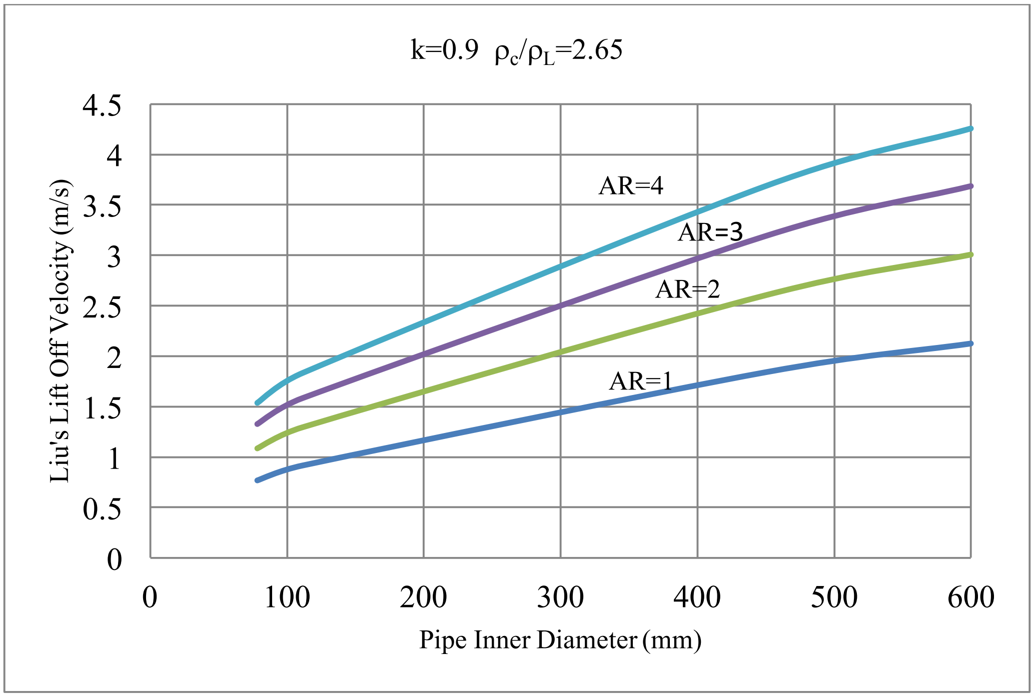


Figure 2. Calculation of the Liu Lift Off Velocity for rigid capsules at the specific gravity of 2.65 moving in a fluid of specific gravity 1.0 and an aspect ratio from 1 to 4, with a diameter ratio of 0.9

Logs of clays should not need not to be pressed at the very high pressure associated with logs of coal at an expensive cost. For clays and silts, it is critical to test for the liquid limit (L_L) and the plastic limit (P_L).

The liquid limit is defined as the moisture content in soil above which it starts to act as a liquid or act as a slurry mixture, and below which it acts as a plastic. To conduct a test, a sample of clay is thoroughly mixed with water in a brass cup. The number of bumps required to close a groove cut in the pot of clay in the cup is then measured. This test is called the Atterberg test.

The plastic limit is defined as the limit below which the clay will stop to behave as a plastic and will start to crumble down. To measure such a limit, a sample of the soil is formed into a tubular shape with a diameter of 3.2 mm (0.125 in) and the water content is measured when the cylinder ceases to roll and becomes friable.

The difference between the liquid and plastic limits is defined as the plasticity index

$$P_I = L_L - P_L \tag{11}$$

Clays with a diameter smaller than 2 μm contribute to the high plasticity index. Plasticity Index signifies the range of moisture content in which the clay keeps its integrity as a plastic lump. In their natural status dredged material contain water content ω_n equal or greater than the plastic limit.

Liu (2005) derived the equations for friction losses and pressure increase due to a number of capsules in a pipeline. These equations will need to be modified for tailings to account for some ablation and release of fines into surrounding slurry. Such modifications will be the subject of a future paper.

The ability of clays to maintain the form of a ball during dredging was analyzed by the US Army Corps of Engineers (1994) by preparing artificial clays, but the results were not deemed definitive by the American Society for Testing of Materials according to Leschinsky et al (1992). ASTM chose to issue a note rather than a standard and recommended further tests on natural clays. The note considered that balls of clays may form and maintain themselves when

- Plasticity Index >25%
- Shear strength > 25 kPa

The results of tests conducted by the US Army Corps of Engineers cannot be used for the analysis of flow of logs in a pipeline since the results are independent of the distance pumped. To understand the relationship between the Plasticity Index and the ablation of natural clays, a wet ablation machine with a rotating drum (Figure 3) was built at the slurry lab of Splitvane in the USA. The drum diameter was set at 445 mm.

Figure 3. Rotating Drum Ablation Machine for testing logs of clay at the slurry lab of Splitvane Engineers Inc.

One of the drawbacks of the drum test is that it introduces a new centrifugal not encountered in the pipeline. This force reduces the positive effect of buoyancy and hydrodynamic lift normally encountered in a pipeline. Drum tests do not cover any damage through pumps or valves. Drum tests are therefore considered as preliminary and should be followed up by pipeline field tests.

Three natural samples of clays were selected (Table 1) for the plasticity index in the range of 7% to 43%. Samples with a diameter of 100 mm (Figure 4 and 6) were prepared by pressing in a 12 ton briquetting press. The pressure is therefore infinitesimal smaller than used for preparing the coal log in Liu's tests to reduce cost of preparation of tailings. These logs were prepared from naturally dredged sources, without the addition of binders, flocculants or coagulants and were not subject to chemical changes associated with mineral processing.

The data was plotted against the tip speed of the drum. When the sample is fresh and has not been ablated the center of gravity of the 100 mm log is at 88% of the tip speed, but this ratio increases as the log ablates.

Table 1. Plasticity Index of Natural Clays tested in the Splitvane ablation machine

Sample	Type of clay	Moisture content	Specific Gravity	Liquid Limit	Plastic Limit	Plasticity Index
1	Sandy Clay	24.9%	2.66	22.5%	18.3%	7.3%
2	Grey Clay	24.5%	2.49	49.4%	17%	32.4%
3	Red Clay	28.4%	2.62	59.7%	16.9%	42.8%

Sample (1) consisting of sandy clay ablated fast at the tip speed of 1.4 m/s. It lost 65% of its mass within the first period corresponding to 275 m.

Based on the calculations of the Liu Lift-Off Velocity (Figure 2), the tests targeted tip velocity magnitude was from 1.0 to 4.0 m/s.

(a) Before Ablation (b) After ablation at 3.8 m/s

Figure 4. Grey Clay Log at start and at end of ablation test at 3.8 m/s drum tip speed

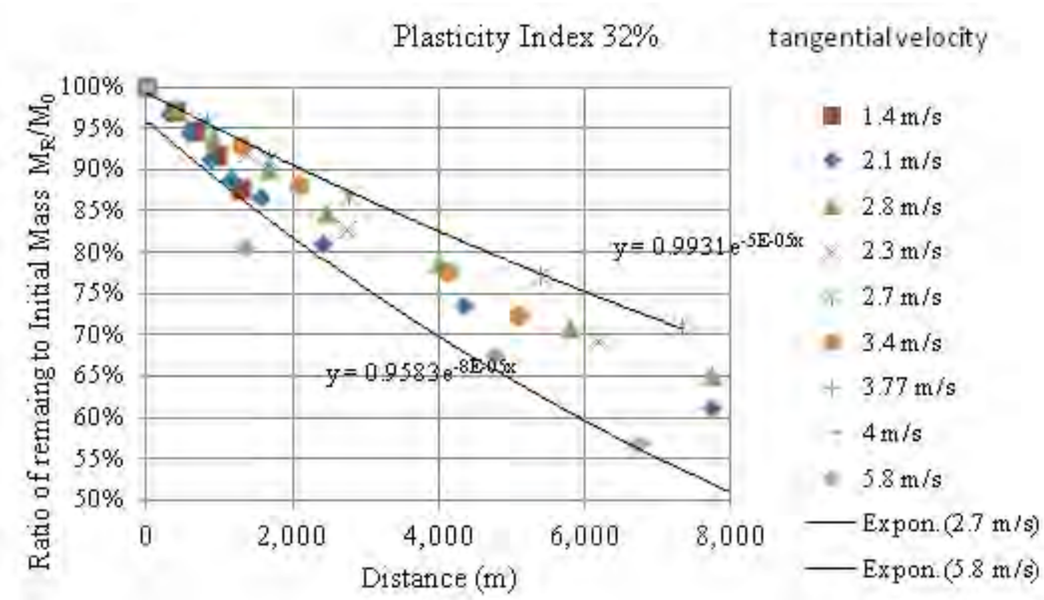
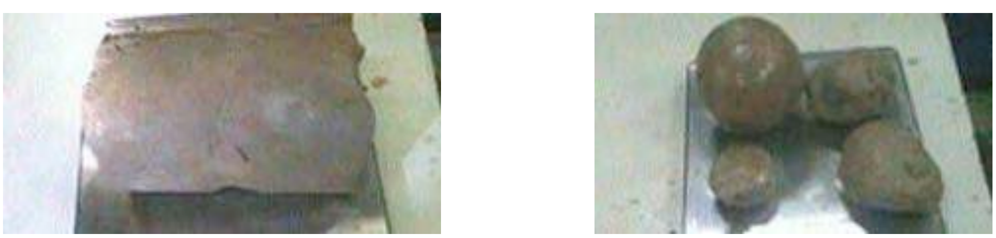


Figure 5. Ablation of log of grey clay at the Plasticity Index of 32% as a function of velocity and conveyance distance.

The Grey Clay Sample, with the Plasticity Index of 32% was able to retain 75% of its mass for the first 5 km and 65% of its mass up for a simulated pumping distance of 8 km up to the speed of 3.4 m/s (Figure 5).



(a) Red Clay Log With Embedded stones and shells (b) Failure of Log due to embedded stones after testing at drum tangential speed of 3.8 m/s

Figure 6. Disintegration of Log of Red Clay at the drum tangential speed 3.8 m/s due to embedded small stones.

Initially the Red Clay Sample (3) contained some coarse sand and sea shell debris that accentuated degradation at the higher speeds despite the higher plasticity index. Samples with embedded coarse material failed at 3.8 m/s (Figure 6) and had to be repeated with samples free of contaminants.

Red clay with embedded stone degraded much faster at 4.2 m/s than sample free of stones and shells (Figure 7). The sample with stones lost 25% of its mass within the first 3 km, while the sample free of stones did not reach this loss until the simulated distance of 6800 m had been reached.

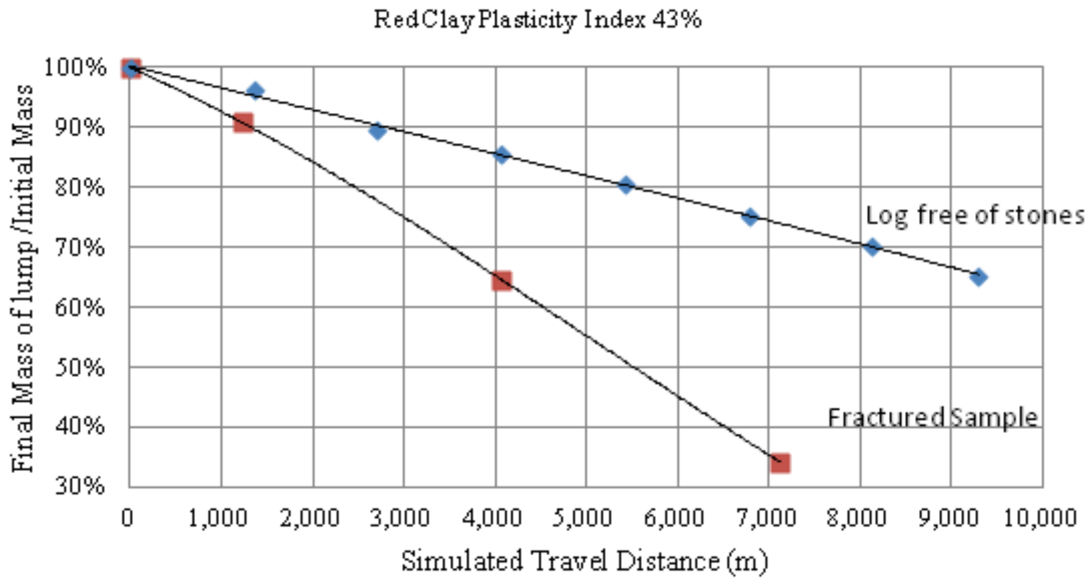


Figure 7. Collected Data for Red Clay with Plasticity Index of 43% - Ablation at 4.2 m/s is accentuated by small stones that create cracks as they try to free themselves due to the centrifugal forces in the rotating drum.

The higher Plasticity Index of Red Clay at 43% maintained the log shape longer than Grey Clay with Plasticity Index of 32% at the tangential speed of 4.2 m/s (Figure 8).

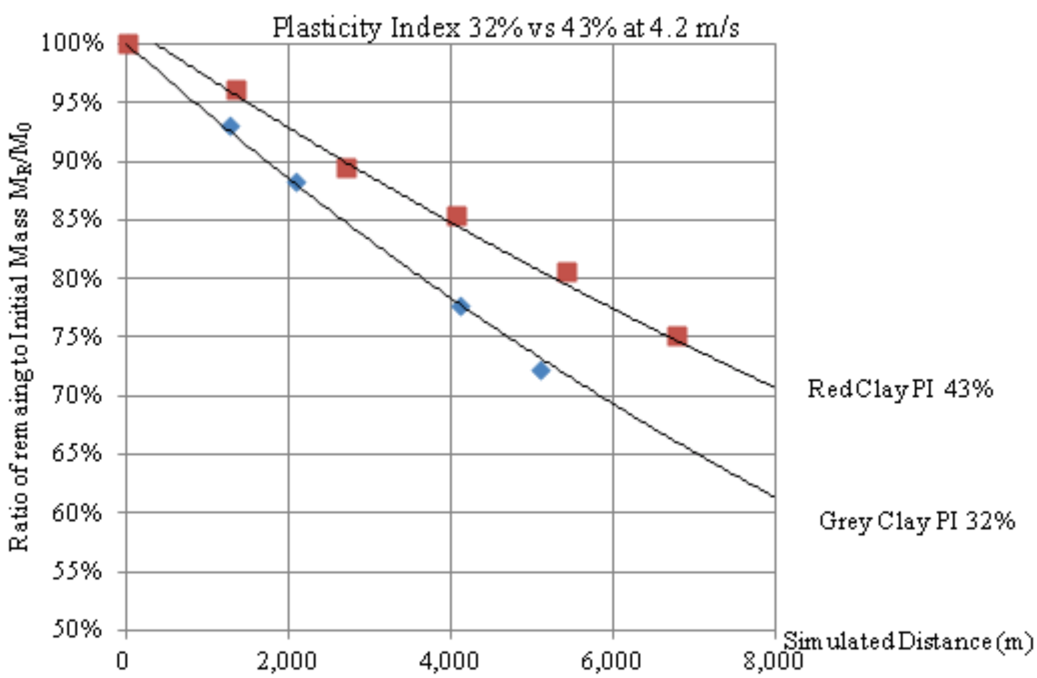


Figure 8. Comparison between Grey Clay with Plasticity Index of 32% with Red Clay with Plasticity Index of 43%, showing that higher plasticity index reduces ablation

4 DISCUSSION AND CONCLUSION

Clays are important in many tailings flows. Wissa et al (1982) measured the index properties of 12 samples of phosphatic clays from the phosphate mining industry in Florida, USA. The clay portion with diameter minus 2μm varied between 41% and 67%. The Corresponding Plasticity Index varied between 97% and 239%. Yao (2016) reported that clays are found in oilsand tailings, reporting that Mature Fine Tailings (MFT) develop a Plasticity Index between 22% and 33% with clay content between 45% and 50%, Thickened Tailings (TT) develop a Plasticity Index of 28% to 31% with clay content between 10 and 22%. Abulnaga (2002) discussed soft high clay in Peruvian copper tailings.

Ablation tests on medium plasticity clays at Splitvane Slurry Lab, in the range of the plasticity index of 32% to 43% show that logs have to the potential to maintain 75% of their shape in the first 5 km of flow at 85% to 100% of the Liu lift off velocity in pipes up to 600 mm Nominal Bore. Fines released during ablation have the potential to form slurry in the pipeline changing the Reynolds Number, Hedstrom Number and affecting pressure losses.

Reports of blockage of balls of clays in dredging and phosphate hydrotransport pipelines should not be confused with log pipeline hydraulics. Blockage in dredging occurs above the Lieu's lift off velocity and the balls become instable, roll, tumble and eventually form a bed resisting motion or block the pipe. It is therefore critical to test the flow of logs of clays at 85% of the Liu's equation for lift-off velocity and apply appropriate lessons from these industries.

There is also a risk of confusing the conventional Durand equation for deposition velocity of slurry with Liu's lift off velocity. The two regimes are different, the first referring to a stratified flow of slurry, while the latter a co-annular flow of a moving train of logs.

To protect the log of clay from ablation until it reaches the tailings placement area, some steps can be taken at the stage of preparation of logs by applying chemicals that boost the plasticity index as those used in the chemical industry (Bain 1971), applying a coat of a water repellent chemical to retard penetration of water into the log, removing coarse material and embedded stones, addition of binders, applying higher pressures at the stage of briquetting, but these steps may increase the cost of the technology.

The potential for conveying logs of clay based tailings in smaller diameter pipelines, with an important reduction of the volume of water for placement of the tailings justifies further research and development. By comparison with its competitors such as paste, clays of logs do not depend on the topography of the site. Logs of clay do not require complex and very long conveyors as used for dry stacking. Pumping schemes for feeding the logs into the pipeline have been studied and developed for forty years by the Alberta Research Council and the University of Missouri, and can be adapted for clay log pipelines opening new opportunities for mine waste placement.

Further recommended work includes testing the logs to modify the equations for the four regimes defined by Liu, confirming the lift-off velocity and the equations for friction and pressure losses in the presence of a train of logs.

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Waste Co-Disposal

Mine waste case examples of stacked tailings and co-disposal

Ben Wickland
Golder Associates Ltd., Vancouver, BC, Canada

Sue Longo
Golder Associates Ltd., Calgary, AB, Canada

ABSTRACT: This paper lists recent mining projects using "dry" methods for disposal of tailings. A total of 18 projects that have implemented, or planned to implement tailings, dewatering by filtering for on-land stacking, are summarized for total tonnage, ore type, method for thickening and deposition, with observations on utility of the approach. Similarly, 39 projects that have implemented, planned, or trialed co-disposal of tailings with waste rock are summarized. These projects demonstrate an increase in the consideration and use of alternative mine waste disposal methods compared to tailings deposited as slurry with a water pond behind dams.

1 INTRODUCTION

The objective of this paper is to present an overview of on-land "dry" mine waste disposal of filtered tailings, stacking and co-disposal of tailings, and waste rock. Sites that have implemented or planned to implement each technology are listed here with comments on utility. The paper is intended to provide a frame of reference for operators, regulators, and other stakeholders on alternative technologies to slurry tailing impoundments.

The impetus for change and consideration of alternate technologies stems, in part, from site specific drivers, such as available land for tailings impoundments, or water recovery in arid climates. A more widespread driver is the control of risk and liability associated with tailings dams. Loss of life and catastrophic environmental damage remains a fundamental concern around the safety and stability of tailings dams. Liabilities associated with failures appears to be dramatically increasing for recent and very public failures of tailings dams, resulting in multi-billion dollar fines and criminal charges to mining company executives. The panel investigation of the Mount Polley failure provided recommendations on the Best Available Technologies (BAT) to manage the risks around tailings dams. The recommendations included elimination of the water pond, promotion of unsaturated conditions in the tailings (with control of drainage), and also promotion of dilatant conditions by compaction to prevent liquefaction of tailings (IEEIRP 2015).

Two key ways to achieve the Mount Polley recommendations are through filtration technology and co-disposal. Filtration of tailings to remove water is an option to mitigate or eliminate the risk of tailings dam failure, and associated catastrophic loss of the water pond and liquefiable tailings. Co-disposal of tailings with waste rock is another option that allows on-land disposal without a dam or water pond. Both technologies in the form of case examples are presented in this paper to describe the state of practice, along with lessons learned, and a discussion.

First, terminology and definitions for alternative technologies are provided for context.

1.1 *Terminology*

Terminology and definitions for tailings disposal technologies can be confusing and contentious, varying from place to place and from practitioner to practitioner, e.g. tailing versus tailings, and tailings storage facility (TSF) versus tailings management facility (TMF). Variance in terminology can hinder common understanding of key issues, so definitions are provided here for the purposes of this paper. The following definitions are generic for each material type:

Slurry tailings: low solids content with water being the transport mechanism, no or little mechanical dewatering applied, has a critical flow velocity and no yield stress, segregating where coarse material forms a beach on deposition, and fines/slimes migrate towards a large pond.

Thickened tailings or high density slurry: higher solids content than slurry with mechanical dewatering being applied, at the lower edge of non-segregating behaviour, may have critical flow velocity, has a high slump value (~10 inch) and a yield stress, may have some water bleed, and may have a small pond.

Paste Tailings: normally free standing, possesses a yield stress, has no critical flow velocity, exhibits plug flow characteristics, has a measurable slump (7 to 10 inches), is homogeneous and non-segregating, minimal to no water bleed, fines are the transport mechanism, no pond.

Filtered (or stacked) Tailings: a cake like material, but are not “dry” per se; the material often retains significant moisture content that must be drained. Subject to static liquefaction and local flow failures. Transport mechanism is typically by truck or conveyor, i.e. non-pumpable.

In Table 1 below, a summary of solids contents and yield stresses are shown for each tailings classification. These ranges of solids contents and yield stresses are general ranges that would apply for the majority of tailings types, but not all will fit into these ranges.

Table 1: Summary of classifications

Classification	% Solids by Weight	Yield Stress	Transport
Slurry	10-30	0-20 Pa	Centrifugal pump
Thickened Tailings	50-60	20-80Pa	Centrifugal or Positive Displacement (PD) pump
Paste	65-75	80-100 Pa +	PD pump
Filter Cake	75-85	150 Pa+	Truck or conveyor
Centrifuge Cake	80-90	150 Pa+	Truck or conveyor

Figure 1 below shows a generic plot of shear stress versus other parameters that have come to define the different levels of dewatering. The actual value in solids content associated with each transition are often material dependent, so there is no hard and fast rule about when a slurry becomes non-segregating or when thickened tailings turns into a paste. It depends on several factors: the index properties of the material (e.g. particle size distribution, pH, and mineralogy); and mineral processing practices, such as flotation and chemical amendments.

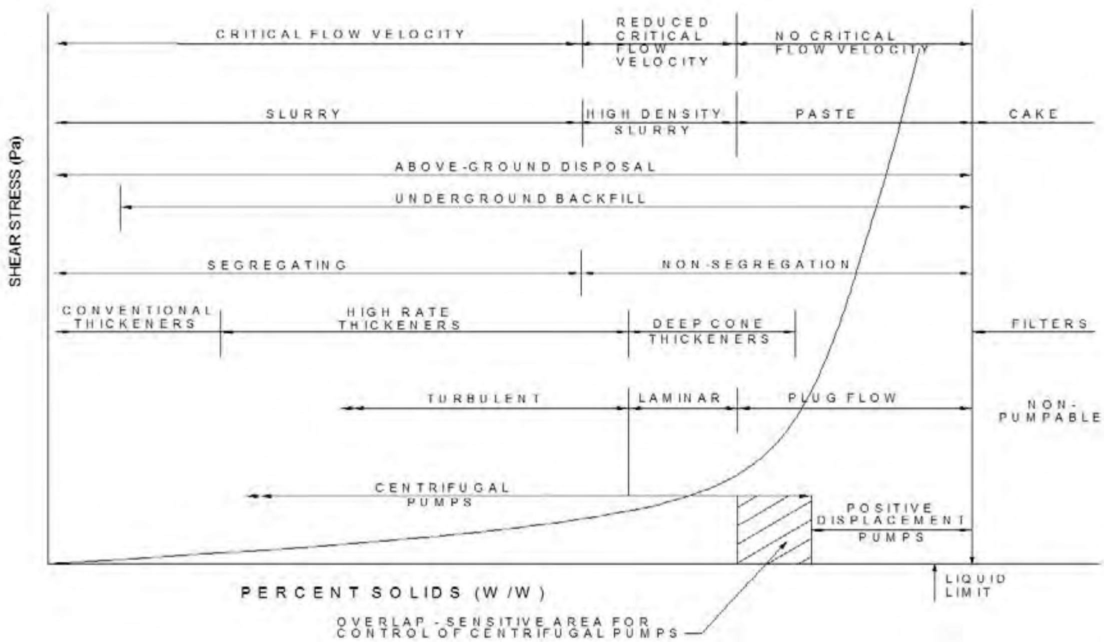

Figure 1. Tailings Shear Stress versus Everything (Jewell, Fourie. 2006)

The key takeaway from Figure 1 is that it must be understood what the material's curve looks like and how it will vary over the life of mine.

"Co-disposal" is a general term used to describe disposal of two or more mine waste types, such as waste rock and tailings. Table 2 provides a general overview of the forms of co-disposal distinguished by degree of mixing, after Wickland et al. (2006). The first published description of on-land of waste rock and tailings co-disposal that the authors could find is attributed to Nigel Skermer in Brawner (1978), who presented a flow sheet and concept description, "Combined Waste Rock – Tailings Storage Concept." Co-disposal has been actively used in underground mining operations for decades where development rock (waste rock) is placed in mined out stopes, and then filled with hydraulic fill or paste backfill. Underground co-disposal serves a different purpose than on surface co-disposal, but the idea of mixing or placing two materials together is not new.

Table 2: Forms of Co-Disposal (modified from Wickland et al. 2006)

Co-disposal Type	
Homogeneous mixtures – Waste rock and tailings are blended to form a homogeneous mass.	Increasing degree of mixing ↑
Pumped co-disposal – Coarse and fine materials are pumped to impoundments for disposal.	
Layered co-mingling – Alternating layers of waste rock and tailings.	
Tailings are placed in cells made of waste rock.	
Waste rock is added to a tailings impoundment.	
Tailings are added to a waste rock pile.	
Waste rock and tailings are disposed in the same topographic depression.	

Various trade names have been applied to different forms of co-disposal. For example, Golder Associates' trade name for co-disposal of tailings and waste rock is PasteRock™, pictured in Figure 2. Other names have included: blending, mixtures, co-mix, co-mixing, co-deposition, in-pit co-deposition, in-pit co-disposal, operational co-deposition, layered co-deposition, co-placement, pumped co-disposal, combined pumping, co-mingling, layered co-mingling, integrated discard disposal, rock integrated tailings system, integrated disposal, geo-waste, and tailings co-disposal™. For the purpose of this paper, methods of disposing waste rock and tailings are all called co-disposal, with additional adjectives to distinguish the type.

Figure 2: PasteRock^{TM} from Porgera co-disposal column study. Source: B. Wickland.

2 FILTERED TAILINGS EXAMPLES AND LESSONS LEARNED

As mentioned earlier with co-disposal, tailings filtration has been used for a long time in backfill applications, but is more recently being explored for surface disposal. In backfill applications, the tailings are typically dewatered by a thickener/filter combination to reach the solids contents required for structural fill underground. In some examples below, the mine operation has utilized the dewatering equipment for both backfill and surface disposal. Material is thickened and filtered, and used as backfill, as required; when not needed, the filter cake is trucked or conveyed on surface to a tailings storage facility. This allows use of the equipment full-time, not only during backfill operations while minimizing storage space on surface.

When combining with backfill is not possible, the filtration option should be considered with site specific circumstances where:

- Seismic zones prevent having any ponded water on the tailings
- Water recovery is paramount
- Climate conditions are such that operation in the winter is problematic for slurry tailings
- Space restrictions require minimization of footprint deposit

Several types of filters exist:

- Vacuum belt
- Vacuum disc
- Rotary drum
- Pressure

Figures 3-6 show photos of each type of filter.

Figure 3. Vacuum disc filter. Source: Confidential client.


Figure 4. Vacuum belt filter. Source: Confidential client.


Figure 5. Rotary drum. Source: Confidential client.


Figure 6. Pressure filter. Source: Confidential client.

Each filter has a different footprint, setup, and range of operation that should be considered when choosing a filter technology. For example, if space is restricted, using a long vacuum belt filter may not be possible; or if the mineralogy and/or PSD is such that vacuum filtration will not work, pressure filtration may be the only option. As reference, some performance parameters for filter cake are: cake loading rate, cake thickness, cake moisture content, ease of cake release, and blinding potential. In most applications, thickeners are used in front of a filter to maximize the gravity separation component (read less expensive), which minimizes the number and size of filters required. Examples of filtered tailings are listed in Table 3. Varying types of filters and deposition schemes are highlighted in various climates.

The list is not intended to be comprehensive, and is based on literature review and on private studies where permissions were granted. Several private initiatives by resource companies were known to the authors at the time of writing, and are not included here.

Table 3. Filtered Tailings Disposal

Project and Location	Implementation Date	Filter Type	Deposition Strategy	Climate
Mandalay, Guatemala	Design	Pending	Pending.	Wet
Platreef, South Africa	Design	Thickener to Vacuum disc filter	Conveyors and stackers. Initial deposition will be trucks for drainage areas and compacted areas.	Summer rainfall, heavy thundershowers
Peabody Wilpinjong, NSW, Australia	2015	Pressure filter	Filter cake mixed with coal rejects and trucked and end dumped into mined out voids	Semi-arid
OCP, Morocco	2015/ 2016	Pressure filter	Conveyor, stacker, and reclaimer.	Dry
La Coipa, Chile	1990	Thickener and Belt filter	Conveyed, stacked, and dozer spread.	Dry (desert)
Cobre Las Cruces, Spain	2013; 2015-2016	Started with Belt filter and changed to Pressure filter	Stacking by truck.	Moderate precipitation, warm
Jinfeng, China	2017	Pressure filter	Trucked, tipped spread, and compacted in 1 m lifts. Initially compaction was by smooth drum roller, but subsequently changed to a tyned roller.	Wet
White Mountain, China	2017	Pressure filter	Delivered to the TSF by conveyor. The tailings are deposited over the TSF surface via a string of secondary conveyors and spread using high mobility excavators. The tailings do not dry back much, and there is no further compaction.	Cold
Green's Creek, USA	Early 2000s	Pressure filter	Cake drops into a concrete vault and a loader loads trucks. The flow is split into the backfill feed or the surface disposal feed. The surface disposal feed is trucked, spread into cells, and roller compacted.	Wet and cold

Project and Location	Implementation Date	Filter Type	Deposition Strategy	Climate
Tambomayo, Peru	2017	Thickener and Pressure filter	Trucked and dried for 7-10 days, deposited and compacted.	Dry
Confidential client, Mexico	Design	Thickener and Pressure filter	Conveyed, stacked, and compacted.	Dry
Cerro Lindo, Peru	2010	Belt filter	Trucked, windrowed for 2 days, and then spread and compacted with vibratory rollers.	Dry
Raglan, Canada	2000-2001	Thickener to Pressure filter	Trucked to the TSF and dozed and compacted	Moderate precipitation, cold
El Sauzal, Mexico	Closure (operated for ~10 years)	Thickener to Pressure filter	Tailings conveyed to drying area and spread with a dozer. Once dry, they are pushed over the edge of the drying embankment and loaded into trucks. Truck transported to dry stack area and compacted. Separated into structural zone and non-structured zone (50 cm lifts vs. 5 m lifts).	Dry
Molycorp, USA	2014	Pressure filter	Blended bentonite and tailings paste or non-blended tailings filter cake. System is split into surface disposal and backfill.	Arid
Pogo, USA	2006	Pressure filter	Cake is trucked and compacted in lifts in the shell of the TSF area for structural stability; the remaining is dumped inside the TSF shell. Filtered tailings are placed in 30 cm lifts on surface.	Cold and wet, Arctic with permafrost
Confidential Client, USA	2014	Pressure filter	Overland conveyor to load out, and then dumped and spread in TSF in cells. Lifts in outer shell were 30 cm lifts and interior were 1 m.	Cold (full four seasons)
Kupol (Kinross), Russia	2017	Pressure filter	Truck haul, dozing and compaction.	Arctic, extensive permafrost

2.1 *Filtered Tailings Lessons Learned*

The biggest lesson learned from virtually all of the above examples is the importance of getting the filter cake water content to geotechnically suitable water content, and the development of shear strength. Optimum water content (OWC) is generally the target for cake moisture, but even OWC does not guarantee it is geotechnically suitable. Filter cake deposits will generally still require drainage, and should not be considered “dry” just because they are cake. Thick lifts may not drain quickly, and potentially can statically liquefy. Thinner lifts are typically required, 1 m or less with a cycle of placement, drainage, and then spreading and compacting. Contingencies are required for rainy or wet conditions. Sites may have two deposition schemes, depending on the season, i.e. winter vs. summer.

One mistake that could easily be made is to calculate OWC in geotechnical terms, and then compare that to filter cake water content, which is typically defined in metallurgical terms. This can result in water content targets that are off by 10% or more.

In terms of filter performance, the biggest issue typically arises around designing for an average tailings material, rather than for the worst case scenario tailings material. This can potentially result in under sizing of filter units, which can result in missing the target cake water content. Other lessons learned are as follows:

- Clay causes major issues, and variability makes it worse
  - Design for the worst case
  - Expect more fines
  - Smooth out variability
  - Include provisions for additional capacity beyond original design
- Cloth type and changes drive performance
  - Assess compatibility of the filter materials, e.g. plates, discs, and cloth with the tailings materials
  - Protect the filter system from the elements, e.g. dust and sun
  - Optimize the filter cloth selection
- Maintenance is your best bet
  - Include a trash screen to prevent larger particles from entering the filters and causing damage
  - Include a thorough preventative maintenance program
- Instrumentation and programming are key with multiple filters
  - Run the system on automatic; this will minimize wear

In the system design it is also easy to miss the importance of the transition points, which typically can cause delays and bottlenecks. Process design must consider the needs of the transportation system design, and from there the deposition strategy. For instance, if the filter cake misses the OWC target, and then the cake may be too sticky to come out of the truck bed at the deposit area, so a contingency plan is a necessity. Conversely, if the cake comes out too dry, there may not be enough moisture to compact it properly.

With any tailings system design, the strategy must account for long-term flow of water, geotechnical performance, and geochemical performance.

3 CO-DISPOSAL EXAMPLES

New projects are now considering/proposing co-disposal to manage physical and chemical risks posed by mine waste storage. Advantages may include: higher density, lower volume, reduction in the flux of oxygen and water through the waste, higher strength, and resistance to liquefaction. Advantages of highest density occur when mixtures are homogenous, and the tailings fill the waste rock voids (Wickland et al. 2006).

Table 4 presents projects that have implemented, proposed, considered, or published trials on co-disposal. The list is not intended to be comprehensive and is based on literature review, as well as private studies where permissions were granted. Several private initiatives by resource companies were known to the authors at the time of writing, and are not included here. Additional detail of specific examples are provided in Habte and Bocking (2017).

Table 4. Co-disposal Examples

Project/Location	Description
Implemented	
Daggafontein, South Africa	Soil and waste rock are mixed and used as a closure cover on a tailings storage facility – predecessor of PasteRock (Gowan et al. 2010).
Mt. Thorley, Australia	Dewatered tailings are added to coarse rejects on a conveyor, and then transported in trucks for dumping at the same time as mining spoil (Gowan et al. 2010).
Jeebropilly Colliery, Gordonstone Colliery (Kestrel), Burton, Charbon,	Coarse and fine coal rejects are mixed and pumped to an impoundment (Gowan et al. 2010, Williams and Gowan 1994)

Project/Location	Description
Coppabella, Cumnock, Hail Creek, Moorevale, Moranbah North, North Goonyella, Stratford, Lake Vermont, Sonoma, Australia	
Agua Blanca, Spain	Thickened tailings are discharged directly onto layers of waste rock within a lined tailings impoundment (Pers. Comm. Ward Wilson).
Cerro De Maimon, Dominican Republic	Layered co-disposal of mine rock placed over desiccated thickened tailings in 1.5 m to 2.0 m lifts (Li et al. 2011).
Coal Mines (Confidential), South Africa	Tailings slurry pumped into centre of dump, compacted discard forming a wall around the central slurry impoundment. Slurry may also be pumped onto un-compacted discard resulting in a matrix (SADME 2001).
Unnamed Polymetallic Mine, South Africa	Overburden and waste rock are used to create cells for disposal of tailings, including base drains to remove tailings consolidation water (Habte and Bocking 2017).
Nunavik Nickel Mine, Canada	Thickened tailings deposited in lined waste rock containment cells (Bertrand et al. 2009).
Snap Lake Mine, Canada	Processed kimberlite is placed in unlined storage cells composed of grits and waste rock.
Neves Corvo Mine, Portugal	Mine rock used to construct storage cells for tailings paste over an existing conventional tailings impoundment (Junqueira et al, 2009).
Illawarra Coalfields, Australia	Cells constructed in a waste rock dump filled with tailings (Gowan et al. 2010).
Oaks Mine, South Africa	Waste rock used to build paddock type cells, into which tailings are disposed (Dunn 2004, see also Habte and Bocking 2017).
Greens Creek Mine, USA	Filtered tailings, mine production rock, and other materials, including ditch sediments, are trucked and end dumped at the waste site at a variable ratio, and then compacted with a vibratory roller (HGCMC 2015).
Dunka Mine, USA	Waste rock deposited in a tailings impoundment on mine closure.
Navachab Gold Mine, Namibia	Filtered tailings co-disposed with waste rock in active waste rock dumps (Habte and Bocking 2017).
La Quinua Mine, Peru	Thickened tailings discharged to an under-drained area within a leach pad (Habte and Bocking 2017).
Tarong Coal Mine, Australia	Rejects end dumped and placed in a void, and then tailings pumped into the void (Gowan et al. 2010).
Kidston Gold Mine, Australia	Waste rock and thickened tailings were placed in an open pit from opposite sides of the pit rim (Wilson 2001).
Proposed or Considered	
NICO Project, Canada	Thickened tailings and mine rock to be placed in alternating 5 m thick layers with perimeter embankments for containment (Habte et al. 2014).
Esquel Gold Mine, Argentina	Tailings to be disposed with waste rock or leach ore (Leduc et al. 2004).
Shakespeare, Canada	Thickened tailings to be placed in waste rock cells.
Krumovgrad Gold, Bulgaria	Paste tailings to be placed in cells constructed from mine rock (Eldridge et al. 2011).
Sites with Published Trials	
Brukunga Remediation Project, Australia	Tailings and mine rock mixed with limestone (Donald et al. 2015).
Ulan Coal Mines, Australia	Coarse rejects pushed onto wet coal tailings and mechanically mixed.
Douglas Colliery, South Africa	Coal tailings slurry poured over 0.3 m thick layers of coarse rejects.
Copper Cliff, Canada	Tailings, slag, and waste rock were mixed to form PasteRock, and then placed in lined test cells.
Porgera Gold Mine, Papua New Guinea	Fully mixed tailings and waste rock placed in 6 m high columns (Wickland 2006).

3.1 *Co-disposal Lessons Learned and Best Practices*

General findings from consulting practice with co-disposal are listed here, focused on the mixed forms with the potential for chemical and physical stability.

1) The most common form of implementation of co-disposal to date has been pumped co-disposal (13+ sites), which continues to be used globally. Constraints include particle size that may be pumped (50 mm is common); hydraulic sorting of coarse and fine particles on deposition results in a beach and slime ponds that require management, i.e. a dam.
2) The next most common form of implementation is depositing tailings in cells made of waste rock at 7 sites. Cells are earthwork intensive, forming smaller dams that may or may not retain water, and reduce the potential volume of tailings that may be released.
3) Full scale implementation of fully mixed co-disposal of waste rock and tailings has not been implemented at a hard rock mine. The barrier to implementation of mixed co-disposal to date is cost. Capital expenditures for co-disposal may include dewatering equipment, such as thickeners and filters, as well as stacking and conveying equipment. Mining projects are more frequently considering life cycle costs, which, including closure, have made co-disposal more attractive with time. Methods for mixing are now available.
4) There is no secret formula for the co-disposal of mixtures of waste rock and tailings. There are published methods to design a mixture based on the properties of the mine waste, to achieve maximum density for maximum strength, minimum hydraulic conductivity, and least water stored (Wickland et al. 2006).
5) Dewatering of tailings is required for co-disposal with waste rock for on-land stacking. Low viscosity slurry runs out of the voids of waste rock, off the belt, and out of the back of the truck.
6) Proponents considering co-disposal should be aware of the >50 available papers, and at least six graduate studies of co-disposal. Re-inventing the wheel is costly (re-naming the wheel is another matter).
7) Laboratory testing of co-disposal samples containing waste rock is complicated by larger particle sizes. The rule for geotechnical testing is: the maximum particle size should be no greater than 1/6th the diameter of the testing device, e.g. shear box.
8) Gap graded mixtures of waste rock and tailings can lose fines, or are not internally filter compatible.
9) Drainage of lifts of co-disposal of mixtures of waste rock and tailings may be required after placement, depending on mixture design. Similar to filtered tailings, high initial moisture content may require a drain down period to achieve strength suitable for trafficability, and to receive the next lift. Mix design is key.

3.2 *Discussion*

This paper has presented several projects where alternatives to slurry tailings discharge with a water pond and a dam have been tried or considered. The lists are not exhaustive, and are intended to provide insight into the state of practice for filtered tailings and co-disposal as best available practices.

Implementation of filtered tailings is ongoing at projects with relatively low tonnages. Technology is still being developed, and is not currently being utilized at higher tonnage rates, due primarily to the cost of commercially available technology in 2017. Stacking at high rates is possible, but the lift height for tailings may be limited due to the drainage requirements and the compaction required for placed lifts, so as to limit static liquefaction.

Co-disposal is a relatively new idea for surface disposal. This emerging technology has been selectively implemented where there is an advantage to do so, primarily as pumped co-disposal of coal washery wastes, as waste rock used to build cells to retain tailings solids, and in some other forms. Co-disposal as fully mixed waste rock and tailings has the potential to achieve long-term physical and chemical stability without a tailings dam or water pond. Barriers to implementation include capital and operating cost, due to the requirement for mechanical equipment to size the rock, dewater the tailings, mix, and place.

Filtered tailings and co-disposal are relatively new technologies and as such, must go through development and refinement on the way to widespread implementation, as illustrated in Figure 7.

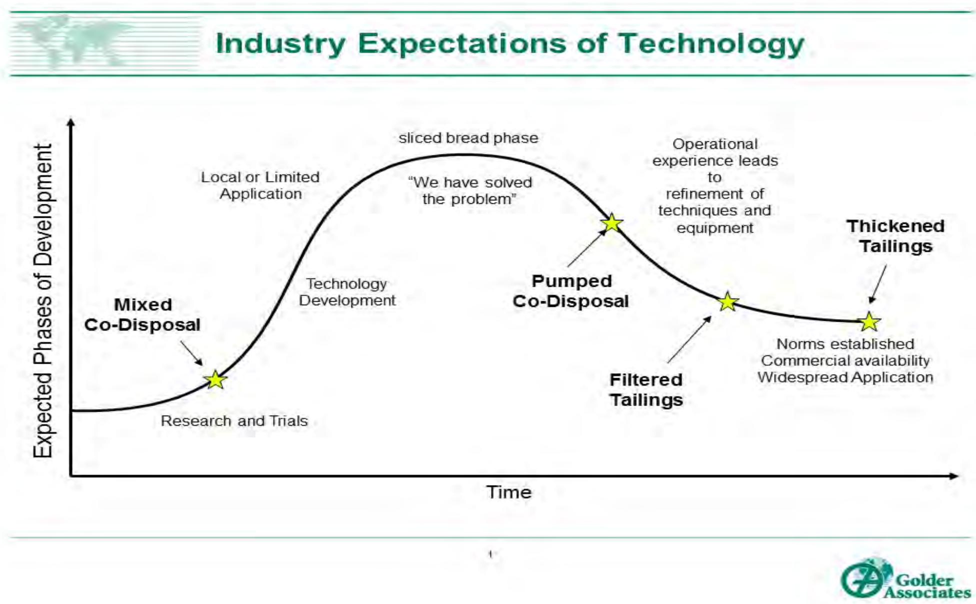

Figure 7. Industry expectations for implementing new technology.

To close on a sober note, the reader is cautioned that dry facilities still carry risk of catastrophic failure. The Aberfan coal spoil failure occurred in 1966. Some 195 reported incidents occurred at waste rock dumps between 1968 and 2005 in British Columbia, Canada alone (Hawley and Cunning 2017). Failures were often attributed to low strength foundation conditions exacerbated by water, steep foundation conditions, and high loading rates during operation. Dry or wet, geotechnical engineering of on land mine waste disposal systems is required to manage the risks.

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Evaluation of the use of paste rock as cover material in mine reclamation

Sandra Pouliot & Bruno Bussière
RIME UQAT, Rouyn-Noranda, QC, Canada

Anne-Marie Dagenais,
Golder and Ass., Montreal, QC, Canada

G. Ward Wilson
University of Alberta, Edmonton, AB, Canada

Yanick Létourneau
LaRonde Mine, Agnico Eagle Mines Ltd, Rouyn-Noranda, QC, Canada

ABSTRACT: The mining industry faces increasing challenges related to mine site reclamation with increasing impacted footprints and the quantity of materials required to put covers in place. The research presented here is complementary to other technical pre-reclamation studies to evaluate the application of paste rock, a mixture of waste rock and mill tailings material, available at the LaRonde mine (Agnico Eagle mines Ltd), with or without a limestone amendment and/or compaction. The combination of tailings and waste rock material to create a homogeneous mixture that has the appropriate hydrogeological properties is attractive because it offers significant environmental advantages (Wickland, 2006). The new mixture possesses both the geotechnical characteristics of waste rock, and the hydrologic characteristics of mill tailings (Wilson, 2008). Previous field test measurements (Wilson, 2008) have shown that the infiltration rates and drainage are reduced when the mixture is used to build cover systems. This present study for the use of paste rock relies on results from laboratory material characterization, an eighteen months monitoring of in-lab column tests, and the construction and monitoring of a field test cell at the LaRonde mine in 2015. More specifically, results of permeability tests, oxygen consumption tests, columns hydrogeological behavior (suction and water saturation profiles), and water quality are presented in this article.

1 INTRODUCTION

Tailings deposited in surface impoundments are often prone to oxidation when they contain sulphide minerals and when they are exposed to water infiltration and atmospheric oxygen. In many cases, sulphide oxidation reactions lead to the formation of acid mine drainage (AMD), when the neutralization potential is insufficient compared to the acid generation potential of a given tailings (e.g., Blowes & Ptacek, 1994). In these cases, appropriate measures must be taken to avoid environmental impacts. Covers with capillary barrier effects (CCBE) and a moisture retaining layer (MRL) at a high degree of saturation can be used to control oxygen migration (e.g. Aubertin et al., 1995,1996; Bussière et al., 2003). This type of cover relies on a phenomenon called the capillary barrier effect that develops when a fine-grained material overlies a coarse-grained material.

Cover systems used to limit acid generation from mine tailings storage facilities (TSF) are usually composed of natural materials. The availability of suitable, naturally-occurring materials can be a concern, particularly in terms of stripping and transportation costs when the required volume to cover large TSF. The construction costs can greatly increase when these materials are not located within close proximity to the site to be reclaimed (Bussière et al., 1999). Moreover, concerns related to the social acceptability and the environmental impact of stripping sites of natural materials put pressure on the mining industry to use alternative materials for their cover systems. Consequently, mining companies are increasingly interested in using mine waste rocks and tailings in substitution to natural materials (Aubertin et al., 1995 Bussière, 2007, Kalonji et Al., 2017)).

Paste rock is a mixture of waste rock and mill tailings material generated by a mine operation. The combination of tailings and waste rock material to create a homogeneous mixture that has appropriate hydrogeological properties to become cover materials is attractive because it offers significant environmental advantages (Wickland, 2006). Indeed, if well designed, the new mixture can possess both the geotechnical characteristics of waste rock, and the hydrogeological characteristics of mill tailings (Wilson, 2008).

The LaRonde mine (Agnico Eagle mines Ltd) is a polymetallic mine located in the greenstone belt of the Abitibi region in the province of Quebec, Canada. The mine has more than 180 hectares of surface tailings impoundments containing sulphide rich, acid generating material. The possibility of using paste rock with available mine materials is being evaluated as a reclamation option for the tailings storage facility (TSF). Because of its acid generating tailings and waste rock material, the paste rock is being evaluated as cover materials, with or without a limestone amendment and/or compaction.

This study presents the evaluation of the use of paste rock, made with available mining materials at the LaRonde mine (Agnico Eagle Mines Ltd), to make an adequate cover system to control the generation of acid mine drainage. The objective is to assess the performance of limestone amended and non-amended paste rock made entirely of mining materials using in-lab column tests. The four columns tested were monitored for a period of eighteen months by analyzing their hydrogeological and geochemical behavior. More specifically, oxygen consumption tests, suction and volumetric water content saturation measurements, and water quality of the effluent after each flush are used to assess the performance; the main results are presented in this article.

2 MATERIALS AND METHODS

2.1 *Materials preparation*

Both the waste rock and the tailings used in this study were sampled at the LaRonde mine (Agnico Eagle Mines Ltd). Note that the particle size of waste rock samples was truncated to 50 mm before being transported to the laboratory. From the literature review (Wickland 2006, and Wilson 2008), the ideal ratio in dry weight to make the paste rock mixture should be around 4 to 5: 1 (waste rock: tailings). Several tests with different ratios (5.1:1 to 3.6:1) were produced in the laboratory using a small cement mixer to obtain cohesion between the tailings and waste rock. Permeability tests, as described below, were used to select the 4.3:1 (waste rock: tailings) as the optimal ratio for the LaRonde mining material. The mining materials used were characterized using the methods presented in the following sections to obtain their main physical, geochemical and hydrogeological properties.

2.2 *Materials characterization*

First, the particle size distribution of the tailings was obtained using a Malvern Mastersizer laser particle size analyser. The waste rock particle size distribution was determined by sieving according to ASTM standard D422 (2007) for the coarse fraction, and by a Malvern Mastersizer laser particle size analyser for the fine fraction (particles smaller than 0.425 mm). In the case of the paste rock, a mixture of tailings and waste rock, the particle size distribution was obtained using the same steps as the waste rock. The specific gravity (G_S) of each material was determined by an immersion basin for particles over 5 mm, and a helium pycnometer (Micromeritics AccuPyc 1330) according to ASTM standard D854-10 (2012) for smaller particles. Proctor compaction tests on tailings and waste rock were performed according to ASTM standard D698-12 to determine the optimum Proctor value (ρ_{opt} and w_{opt}).

The saturated hydraulic conductivity (k_{sat}) was determined for the tailings, the paste rock mixture, and waste rock. Tailings k_{sat} was evaluated using a rigid wall permeameter according to ASTM standard D5868-95 (07), which was then compared to values predicted by the model by

Mbonimpa et al. (2002a). Because of the coarse grain material, k_{sat} of the waste rock and paste rock were evaluated using the constant head permeability test, both in large high-density polyethylene (HDPE) columns (80 cm in height and 30 cm in diameter) as per standard ASTM D2434-68 (06). The values were then compared to values from the prediction models from Shepherd (1989) and Chapuis et al. (2004).

The tailings WRC was determined using a pressure cell (Tempe Cell) following ASTM standard D6836-02. The available literature does not present a test to determine the WRC for coarse grain material with the presence of fine silt-like particles such as the paste rock mixture presented in this study. A large size pressure cell, following the mechanism of a Tempe Cell, close to ASTM standard 6836, was presently developed.

The materials used in this study were also tested in the laboratory for their geochemical and mineralogical characteristics. First, total sulfur-carbon was measured with an Eltra CS 2000 carbon sulfur determinator, a high temperature furnace, in order to estimate the acid generation potential as well as neutralization potential of the tailings and the waste rock. The concentrations in heavy metals (As, Be, Bi, Sb, Se, and Te) were measured with a ICP-AES spectrometer for each material. Assays with an X-Ray diffractometer (XRD) were also completed for tailings and waste rock samples to obtain a semi-quantitative estimation of their mineral composition (see Kalonji et al. 2017 for more information materials characterization methods).

2.3 *In-lab columns construction and instrumentation*

Each of the in-lab columns were mounted in black HDPE (High Density PolyEthylene) with an internal diameter of 0.30 m, and a height between 0.9 and 1.2 m, depending on the thickness of the cover system. A 0.1 m space was made available at the top to allow for oxygen consumption tests and the realization of wetting-drainage cycles. At the bottom of the columns, a base equipped with a ceramic plate was installed and connected to a flexible tube to simulate the water table below the column (this system allow to produce a suction at the base of the columns).

The parametric settings of the four laboratory columns tested (C1, C2, C3, and C4) are summarized in table 1. One will observe that the moisture retaining layer (MRL) is made of paste rock at an optimum ratio of 4.3:1 waste-rock to tailings in all columns. Two of the columns (C1 and C2) assess the paste rock cover in a CCBE configuration with a waste rock capillary break, while the other two columns (C3 and C4) test a bi-layer cover configuration. Because of the high sulfur content of the LaRonde mining material, the paste rock used as MRL of two of the columns (C2 and C3) was amended with a fine limestone gravel as a neutralization agent. The waste rock material used for the top draining layer was identical for all columns (0-50 mm), and all were compacted.

Table 1. Parametric settings for the laboratory experimental columns

Column	Limestone Amendment	Compaction	Draining layer (0.30 m)	MRL (optimum ratio) (0.50 m)	Capillary break (0.40 m)
C1	no	yes	Waste rock 0-50 mm	Paste rock (4.3: 1 - *waste rock: tailings)*	Waste rock 0-50 mm
C2	yes	yes	Waste rock 0-50 mm	Paste rock (4.3: 1 - *waste rock: tailings)*	Waste rock 0-50 mm
C3	yes	yes	Waste rock 0-50 mm	Paste rock (4.3: 1 - *waste rock: tailings)*	no
C4	no	yes	Waste rock 0-50 mm	Paste rock (4.3: 1 - *waste rock: tailings)*	no

Instruments were installed throughout the experimental columns as shown in Figure 1. The GS-3 probes were used to determine the volumetric water content in all the layers. For each materials, a calibration curve was developed to improve the measurements precision (± 0.03). Suction values were measured using a Watermark probe and tensiometers; only the Watermark probe results are presented in this paper. Several references are available on the use of these instruments including Kalonji et al. (2016). The columns also allowed to perform oxygen consumption tests that were performed on a regular basis, at the end of each flush. The test assesses the capacity of the cover to control oxygen migration. More information on OC tests can be found in the work by Elberling et al. (1994) and Mbonimpa et al. (2011).

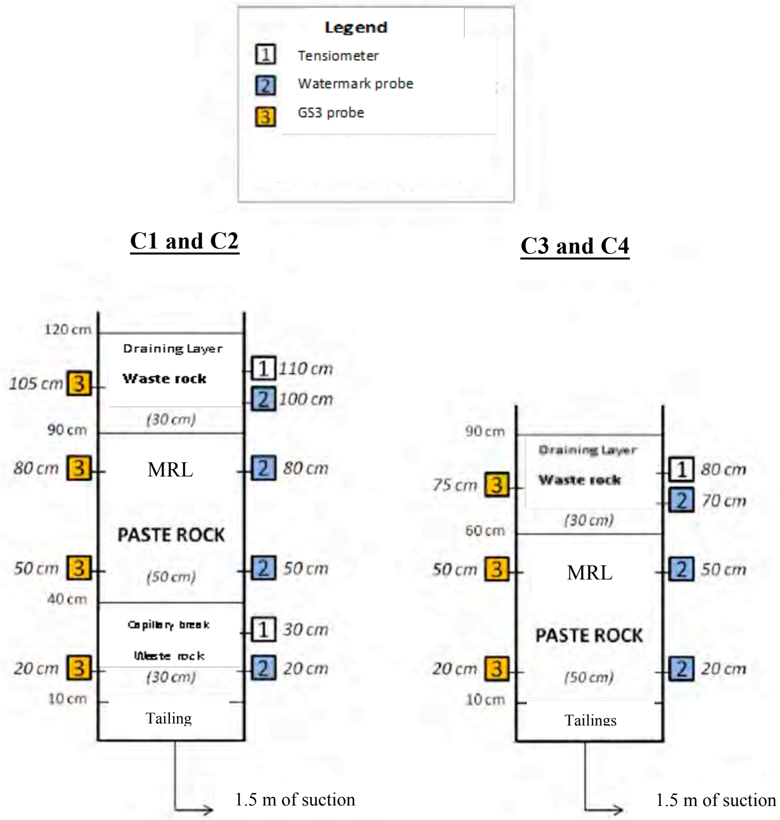

Figure 1. Experimental laboratory columns

3 RESULTS

3.1 *Materials characteristics*

This study focused on both hydrogeological and geochemical behavior of the different mining materials. Table 2 shows some of the important physical characteristics of the LaRonde tailings, paste rock mixture, and waste rock used in study. The specific gravity of the materials are related to their sulphide content with higher values for the tailings (3.29) compared to the waste rock (2.73). Based on the particle size distribution, the tailings are classified a plastic silt (ML) and the waste rock samples as well-graded sand (SW) (USCS, McCarthy, 2007), with variable proportions of fine particles. The tailings k_{sat} varied between 3.9×10^{-8} and 5.7×10^{-8} m/s, while that of the waste rock varied between 1.6×10^{-5} and 7.1×10^{-4} m/s, and measured at 2.8×10^{-8} m/s for the paste rock. For the tailings, the Air Entry Value (AEV) of the two WRC were approximately between 20 to 30 kPa (Tempe Cell measurements). WRC representative of the tailings measured in a Tempe Cell, and fitted with the modified Kovács predictive model (Aubertin et al. 1998, 2003), are shown in Figure 2. The AEV for the LaRonde mine waste rock was measured in previous work by Kalonji et al (2017).

Table 2. Basic and hydraulic properties of materials used in experimental columns

Properties/Parameters	Materials used in experimental columns		
	LaRonde tailings	Paste rock (sample 2)	LaRonde Waste rock
%S	19.08		1.065
%C	<0.05		0.44
Grain size			
D_{10} (mm)	0.00437	0.180	0.11
D_{50} (mm)	0.02526	20	9.11
D_{60} (mm)	0.03475	24	13.50
C_U (D_{60}/ D_{10})	0.00795	0.133	122.73
Saturated hydraulic conductivity, k_{sat} (m/s) porosity in brackets.	3.86E-08 to 5.65E-08 (0.42-0.46)	2.75E-08	1.59E-05 to 7.13E-06
Solid grain density, G_s	3.29		2.73
Air Entry Value, ψ_a measured in the lab (kPa),	20-30	*Testing in progess*	0.43*

*From Kalonji et al (2017)

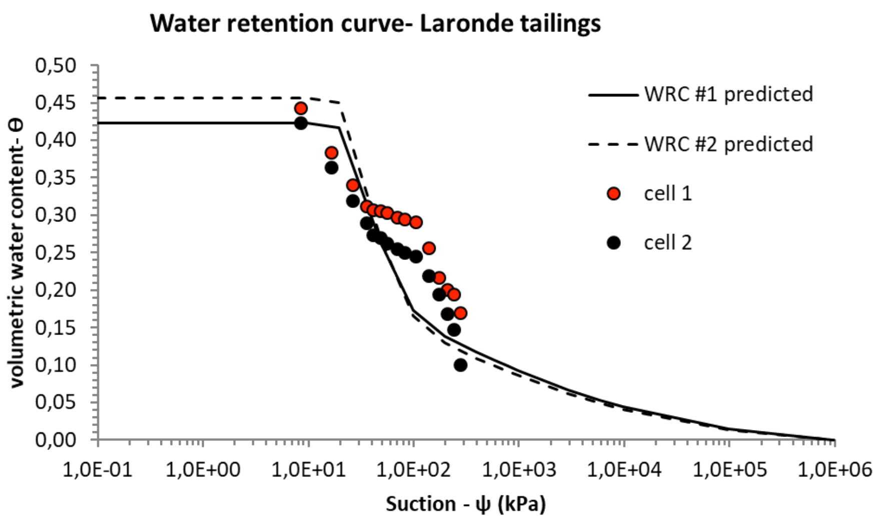

Figure 2. Water retention curve measured vs predicted for LaRonde tailings

With the sulfur and carbon values, the net neutralization ratio (PN/PA) was calculated at 0.00698 for the tailings and 1.101 for the waste rock, clearly demonstrating both materials are potentially acid generating (PAG) since PN/PA < 2. For this reason, a ¼ inch limestone gravel was added to the paste rock mixture in columns C2 and C3, and to the field test cell (CR-4) to increase the neutralization potential and theoretically make the material non-acid generating.

The XRD analysis for the semi-quantitative mineral composition showed that the tailings are mainly composed of silicates such as quartz (42%), albite, and muscovite, and the presence of pyrite in a proportion of 30%. As for the waste rock, that is rich in quartz (50%), carbonates (4.6%), and sulfides (1.35%).

3.2 *Hydrogeological results*

The hydrogeological data (VWC and suction) for the in-lab columns are presented in the following graphs (figures 3 to 6). For the CCBE columns 1 and 2, the suction and VWC sensors 1 to 4 are placed respectively 100 cm (1), 80 cm (2), 50 cm (3), and 20 cm (4) from the bottom of the column. As for bilayer columns 3 and 4, the suction and VWC sensors 1 to 3 are placed at 70 cm (1), 50 cm (2), and 20 cm (3) from the bottom of the column.

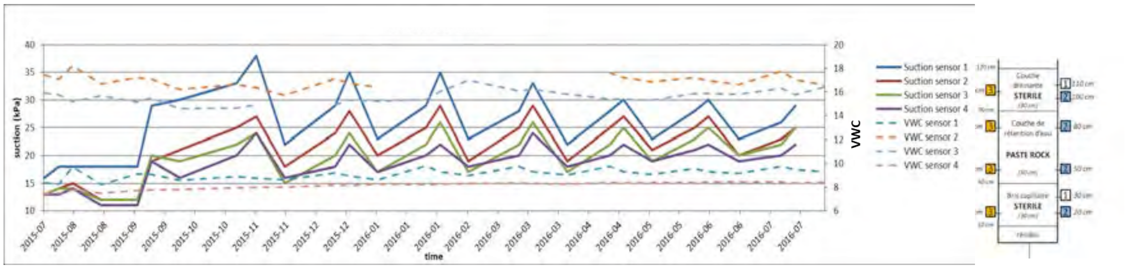

Figure 3. Volumetric water content and suction measured for column 1

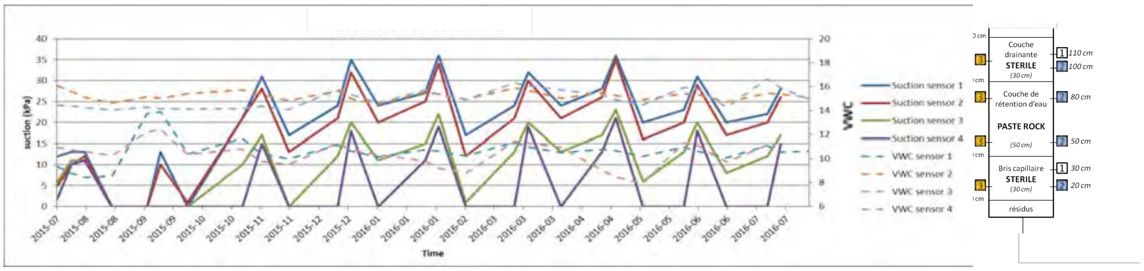

Figure 4. Volumetric water content and suction measured for column 2

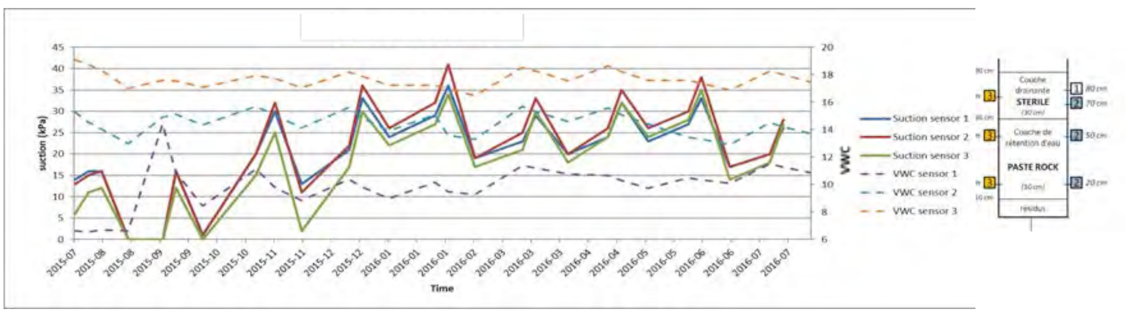

Figure 5. Volumetric water content and suction measured for column 3

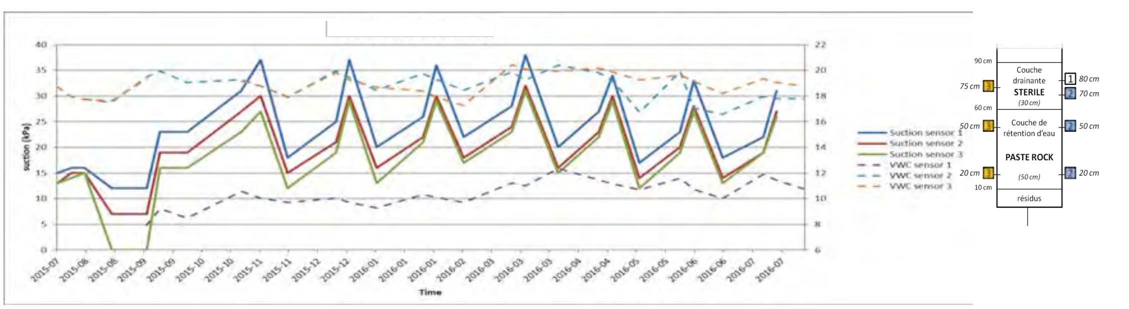

Figure 6. Volumetric water content and suction measured for column 4

The results for columns C1 and C2 show expected behavior for a CCBE cover configuration, where the higher volumetric water content (VWC) are observed in the moisture-retaining layer (MRL) made of paste rock, which correspond to suction values below the air entry value (AEV) allowing for the material to keep its saturation. For example, in column #1, the VWC in the paste rock (MRL) stays between 16-18% while the suction (< 35 kPa) was close to the AEV of tailings (20-30 kPa), which is expected to be lower to the one of paste rock due to the difference in porosity; the porosity of tailings (typically between 0.40 and 0.50) is more important than the one of paste rock (<0.20).

For the bi-layer columns without a capillary break (C3 and C4), the measured volumetric water contents (VWC) are higher with the depth of the cover and the suction is slightly higher, rising close to 40 kPa.

When the results for the amended columns are compared to non-amended ones, lower VWC values and higher suction values are observed. This is most likely explained by the addition of the limestone amendment that raises the density and lowers the porosity of the paste rock mixture. This will in turn allow for a better water retention characteristics, or said differently, the VWC will be higher for a given suction.

3.3 *Oxygen flux results*

The oxygen fluxes measured with oxygen consumption tests for the four laboratory columns over a period of 500 days are presented in the Figure 7.

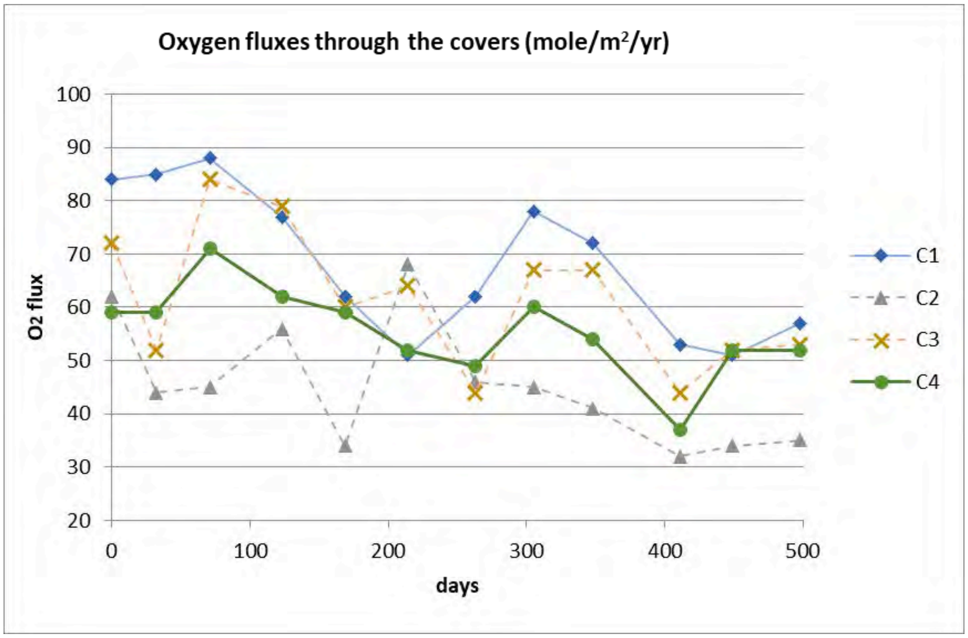

Figure 7. Oxygen flux measured in the different columns

The lowest fluxes are observed in the amended column with a waste rock capillary break (C2). In both amended columns (C2 and C3), the limestone most likely limited the reactivity of the sulfides present in the paste rock, which in turn lowers the oxygen fluxes. The oxygen flux values measured range from 30 to 80 mol/m²/yr. The paste rock cover contributed to limiting the reactivity of the Laronde tailings present at the bottom of each column. Other studies by Bussière et al (2004), demonstrated that the Laronde tailings have a reactivity between 365 -1500 mol m²/yr.

3.4 *Geochemical results*

The leachate samples from each column following each monthly rinse cycle were analysed for pH, Eh, conductivity, alkalinity, acidity, total sulfur, and several metals (Al, As, B, Ba, Be, Bi, Ca, Cd, Co, Cr, Cu, Fe, K, Li, Mg, Mn, Mo, Na, Ni, Pb, Sb, Se, Si, Sr, Te, Ti, and Zn). The objective was to assess the capacity of each cover configuration to limit acid mine drainage (AMD) and metal leaching. Some of the results, representative of the trends observed are shown in Figure 8, 9 and 10 below.

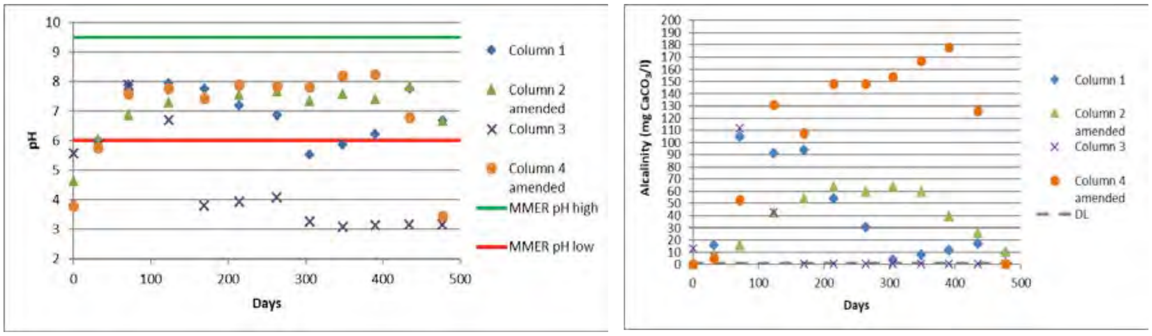

Figure 8. Results for pH and alkalinity measured from the leachate of the four laboratory columns

The first observation by looking at the pH and alkalinity results is that column 3 is the first to drop in pH and alkalinity over time, followed by column 1. This is most likely because they were not amended by limestone. Column 1 shows a slightly better performance than #3, probably due to the capillary break in its configuration that reduces more efficiently oxygen migration. The amended columns #2 and #4 keep their pH above the federal MMER limit, but show a downward trend starting around 400 days. All columns have lost their alkalinity by 500 days. Clearly at that point, the amendment is not able to counterbalance the acidity generated by the mining materials.

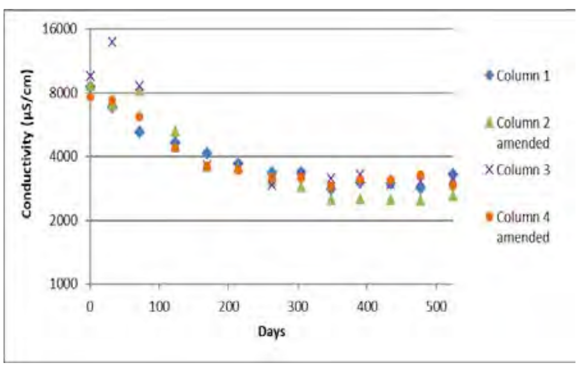

Figure 9. Conductivity measurements from the leachate of the four columns

As for the conductivity measurement, there is a downward trend for all columns until 300 days. After that, the values stay the same or they rise slightly; at the end of the test, the highest value is observed in the leachate of Column 1 while the lowest is for the leachate of Column 2. This, added to the pH and alkalinity profiles show a change in the trends of the results at around the same time.

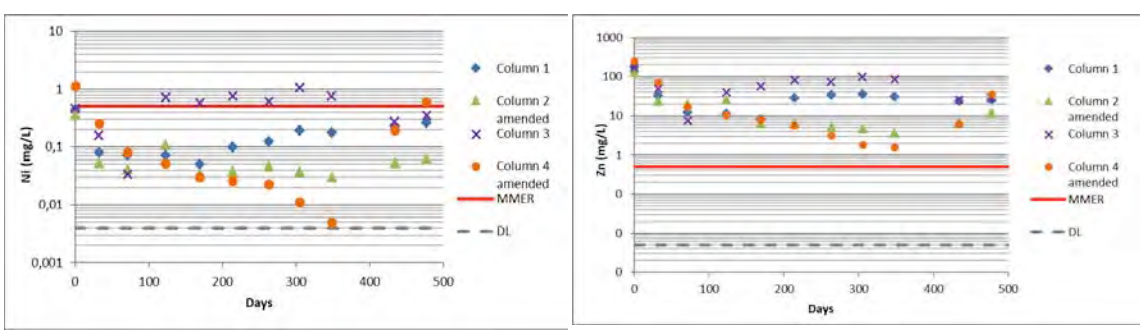

Figure 10. Results for nickel and zinc from the leachate of the four columns

Results in metal concentrations for nickel and zinc show a better performance for the amended columns up to 400 days. Because of the limestone amendment, there was a downward trend in nickel and zinc for both columns C2 and C4, lower than the regulatory limit for nickel and almost reaching the regulatory limit for zinc until 325 days, when the alkalinity was lost in the system. After that time, the concentration in both Ni and Zn increased significantly.

4 CONCLUSION AND UPCOMING WORK

An experimental protocol using in-lab columns was conducted in order to evaluate the use of paste rock as an adequate cover to prevent acid mine drainage (AMD). Results of the hydrogeological monitoring of the in-lab columns demonstrated that the paste rock cover, more specifically in a limestone amended CCBE configuration, was successful in maintaining an adequate volumetric water content (VWC) and suction to limit the migration of oxygen in the cover. The geochemical monitoring showed however that the beneficial effect of the limestone amendment to maintain a neutral pH and sufficient alkalinity to prevent acid mine drainage is only present for a certain period. The results show depletion of alkalinity and a rise in the metal leaching after the depletion (after 325 days).

In order to further interpret the results obtained so far from the in-lab columns, results from the columns will be compared to a full season of results from a field experimental test cell. More specifically, a field experimental cell with a paste rock cover was constructed at the LaRonde mine in the fall of 2015 with a design based on previous work (Bussière et al. 2007). The cell as the form of a reverse truncated pyramidal shaped basin insulated from the hydrogeological system via a geomembrane. A drain was installed at the center of the 1 m^2 bottom of the cell was deployed horizontally in order to emerge outside the cell. The outlet of the drain connected to a flowmeter allows monitoring water volume and quality. Volumetric water content (VWC) sensors and suction sensors were positioned in the different layers of material and connected to a datalogger. A pore gas sampler connected to oxygen sensors in the different layers will allow measurement of vertical oxygen concentration profiles. The results and interpretation will be made available in an upcoming publication.

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Co-disposal Practice in Mine Waste Management

Kebreab B. Habte & Ken Bocking
Golder Associates Ltd., Mississauga, ON, Canada

ABSTRACT: Mining operations leave behind large volume of tailings and waste rock waste streams for perpetuity, which may pose significant risk to the mining companies, the local community, and the environment. Despite the continuous improvement in their design and operation, the failure rate of tailings disposal facilities remains higher than that of water reservoir dams and waste rock dumps. On average, three tailings disposal facilities are failing every two years. Waste rock dumps, on the other hand, are comparatively stable but could have a significant environmental impact if they have metal leaching and acid generating potential. There is a growing trend by the mining industry to identify alternatives to the traditional methods of separate disposal of slurry tailings and waste rock. One of the options available is the co-disposal of tailings and waste rock. Co-disposal may increase the complexity and the cost of operation; however, it has also the potential to improve stability, reduce acid rock drainage, reduce dusting and erosion, reduce disposal footprint area, allow progressive closure, facilitate consolidation of the tailings, and reduce life-cycle cost. This paper provides examples and details of the implementation of tailings and waste rock co-disposal.

1 INTRODUCTION

Mining operations produce large volumes of waste rock and tailings as waste products which will be left behind for perpetuity. To meet the growing demand for minerals, mining companies are even now developing lower grade ore bodies, significantly increasing the tailings and waste rock that need to be managed. Most jurisdictions require these waste streams to be disposed in a manner to ensure their long-term physical and chemical stability.

Typically, waste rock is end-dumped as a dry material in very thick lifts. Waste rock dumps are generally very stable as the waste rock particles are coarse and angular. McLemore et al., (2009) reported about 29 major waste rock dump failures since 1960. Almost 75% of these failures were on sedimentary waste rock dumps and the main cause of failure seemed to be related to foundation condition.

Tailings are commonly deposited as a slurry with 25% - 45% solids content. Tailings slurry deposition results in particle segregation and release of a significant amounts of water. Coarse and high specific gravity particles settle near the discharge point in the tailings beach area where as fine and light particles settle further away in the slimes area. The tailings beach is often unsaturated and prone to oxidation if the slurry contains sulphide minerals. On the other hand, the slimes area is often covered by a large water pond and prone to instability and liquefaction. Since 1960, about 276 tailings disposal facilities failures have been recorded worldwide (WISE, 2017; CSP, 2017). The main causes of tailings facility failure have been overtopping, slope stability and earthquake (ICOLD, 2001; Strachan and Goodwin, 2015). The majority of the failures were in operating facilities and many of these were related to the water management. The run-out distance of a tailings facility failure is generally much higher than that of waste rock dump failure, because impounded water can potentially transport tailings over long distances.

Water dams and waste rock dumps have much lower failure rate than tailings dams. Current trends in risk-mitigation include: reducing the size of tailings pond, dewatering the tailings (to non-segregating thickened tailings, paste, or filter cake), and co-disposal of tailings and waste rock. These trends are aligned in their objectives with the most recent recommendation presented in the Mount Polley Independent Review Panel Report (IEEIRP, 2015).

Dumps containing sulphidic waste rock are particularly prone to the generation of acid rock drainage (GARD, 2009), which can have serious environmental impacts. The rate of acid generation is usually controlled by the rate of flux of oxygen to the reactive rock surfaces. End-dumping of waste rock tends to create a boulders base layer overlain by alternating angle of repose layers of coarse and fine particles. Through advection, air easily flows into the rubble base layer and upwards into the overlying alternating layers of coarse grain particles. Inclusion of tailings in the voids of the waste rock dump can reduce advection of air and thereby slow the rate of acid generation. Placement of tailings on or within waste rock dumps can also limit the infiltration of water thus limiting the volume of contaminated seepage released from the toe of the dump.

The purpose of this paper is to provide examples and details of the implementation of tailings and waste rock co-disposal.

2 WHAT IS CO-DISPOSAL?

In mine waste management, the term "co-disposal" refers to the disposal of tailings and waste rock streams in one integrated disposal facility. Wickland *et al*. (2006) described various forms of co-disposal depending on the degree of mixing. The waste streams could be intentionally mixed so that the tailings can fill the void spaces of the waste rock. The waste streams can be deposited together so that some degree of mixing takes place by virtue of the deposition process, or the waste streams can be placed in an integrated disposal facility with very little to no mixing. There is inconsistency in literature regarding the terminology used to describe the forms of co-disposal as discussed in Wickland and Longo (2017). Generally, co-disposal can take any of the following forms:

- Waste rock inclusion within a tailings disposal facility;
- Tailings disposal facility inclusion within a waste rock dump;
- Tailings storage in cells constructed of, or encapsulated within waste rock;
- Waste rock piles encapsulation with tailings;
- Layered co-disposal of tailings and waste rock;
- Fully mixed placement ("co-mingling") of tailings and waste rock; and,
- Pumped co-disposal of coarse and fine tailings.

Co-disposal can take place on various surface topographies and also within mine voids (such as open pit).

3 WHY CO-DISPOSAL?

Co-disposal should be considered by each project/mine when evaluating mine waste management options. A detailed trade-off study that takes into account life-cycle cost and risk should be carried with other mine waste management options prior to committing to co-disposal. Generally, co-disposal could be selected to improve physical stability of the tailings; to reduce acid rock drainage and metal leaching from the waste rock; to reduce the footprint area of the waste disposal facilities; to allow progressive closure, to control freeze-drying, dusting and erosion of tailings; to accelerate consolidation of the tailings; and to reduce life-cycle cost.

4 HOW COMMON IS CO-DISPOSAL?

Co-disposal is not a new concept. It is commonly used to manage coarse and fine tailings in coal and diamond mines. There are also a number of metalliferous mines that have successfully used the concept, although the facilities are not typically described as co-disposal facilities. Wickland and Longo (2017) listed the mines that are known to have trialed, planned or operated using co-disposal. The following sections provide brief descriptions of some existing and planned co-disposal facilities.

5 CO-DISPOSAL CASE STUDIES – EXISTING FACILITIES

5.1 *Snap Lake Mine, Canada – Coarse Tailings Inclusion within Fine Tailings Disposal Facility*

Snap Lake is an underground diamond mine located about 220 km northeast of Yellowknife, Northwest Territories, Canada. The mine is located in the Canadian arctic in the region of continuous permafrost. The average annual precipitation, lake evaporation and temperature of the mine site are approximately 370 mm, 300 mm and -6 ºC, respectively. The mine has been in operation since 2008. The mill processes kimberlite at a rate of 3,150 t/d.

The mine developed a co-disposal facility to dispose all its waste streams: 8.0 Mt (6.2 Mm^3) of fine tailings, 14.8 Mt (8.2 Mm^3) of coarse tailings and grits, and 0.87 Mt (0.44 Mm^3) of waste rock (Bedell and Flemming, 2010). The fine tailings, coarse tailings and grits are non-acid generating and approximately 50% of the waste rock is potentially acid generating. Co-disposal was selected to reduce life cycle cost, improve stability and enhance the potential for progressive reclamation.

The co-disposal facility covers a total footprint area about 90 ha with a maximum height of about 40 m. The facility was developed in three phases of cells and each cell was also internally partitioned as shown on Figure 1. This was to allow progressive placement of non-reactive waste rock closure cover as each cell is filled. Coarse tailings, grits, waste rock and quarried rock are used to construct the perimeter embankments of the three cells and also the divider berms inside the cells. These embankment materials are placed in 0.3 m lifts and compacted using smooth drum vibrating rollers or dozers. Potentially acid generating materials are excluded from within 50 m horizontally of the downstream toe of the perimeter embankments of the facility or within 3 m vertically from closure cover surface. The exterior side slopes of the perimeter embankments are graded to a 3H:1V gradient and covered with 0.5 m thick inert rock fill for erosion protection.

The fines tailings have been deposited at 43% to 53% solids content. The coarse and grits are dewatered to a solids content of about 84% in the process plant and are hauled and placed in the co-disposal facility using conventional earth moving equipment. The perimeter embankments of the facility are relatively free-draining, allowing the tailings water to drain freely while retaining the fine tailings. Perimeter ditches and sumps are provided to collect the seepage water from the facility. Water collected in the perimeter sumps is regularly pumped into a water management pond. No permanent pond is maintained inside the co-disposal facility itself; however, mobile pumps are used to pump out whatever water that ponds temporarily inside the cells.

Figure 1: Snap Lake Co-disposal Facility

5.2 *Navachab Gold Mine, Africa – Tailings Disposal Facility Inclusion in a Waste Rock Dump*

Navachab Gold Mine is an open pit mine located approximately 170 km northwest of Windhoek, Namibia. The mine is situated in Namib Desert with annual average rainfall around 215 mm and annual evaporation around 2,400 mm. The mine has been in operation since 1989. The mill processes ore at a rate of 4,000 t/day. Both the tailings and waste rock are non-acid generating.

In 2004, the mine commissioned a new tailings disposal facility (TSF 2), adjacent to one of the waste rock dumps as shown on Figure 2. The facility is located in a closer proximity to the mill and the open pit than the previous tailings disposal facility. The facility was developed over two valleys that slope to the north and to the south. Weathered waste rock from the open pit operation was end-dumped to form a horseshoe shaped embankment to provide containment for part of the facility. To the east a hill provides topographic containment. The south valley is the deepest and at this location the embankment is approximately 55.0 m high. The side slopes of the embankment vary from 1.8-2.2 (H):1(V). The average crest width of the embankment is 30.0 m.

A 5 m wide engineered starter embankment was constructed against the upstream slopes of the perimeter embankment on the north and south sides. The maximum height of the engineered starter embankment was 25 m. An upstream toe drain was provided on the starter embankment. Riser pipes with submersible pumps were provided to pump out the seepage collected by the toe drain. The engineered fill for the starter embankment consists of a 1 m wide sand and gravel filter zone and/or a 210 g/m^2 non-woven geotextile filter and a 4 m to 5 m wide 200 mm minus weathered waste rock fill compacted in 0.3 m lifts.

Tailings slurry was processed at the upstream side slopes along the full length of the waste rock embankment using regularly spaced cyclones. The cyclone tailings underflow was used to provide a 30 m wide coarse filter zone between the cycloned tailings overflow and the waste rock embankment. The tailings pond was initially in the low spot of the valley in the south side but, with time, it was pushed against the hill to the east. Water was returned to the plant with pumps on a floating barge. Cycloned tailings deposition continued above the elevation of the waste rock embankment for a period of time prior to closure.

In 2012, the mine commissioned a belt filter to dewater the tailings as the tailings disposal facility reached its ultimate capacity. The filtered tailings are now being co-disposed with the waste rock in the active waste rock dumps.

Figure 2: Co-disposal Facility of Unnamed Mine in Africa

5.3 *The Oaks Mine, South Africa – Tailings Disposal Facility Inclusion in a Waste Rock Dump*

The Oaks mine is an open pit diamond mine located 360 km northeast of Johannesburg in Limpopo, South Africa. The mine was in operation between 1998 and 2008. Over these operational years, the mine processed 2.4 Mt of kimberlite ore. The mine site is located in a semi-arid climate with average annual precipitation and evaporation of about 380 mm and 1,500 mm, respectively.

The mine co-disposed all of its waste streams: waste rock, fine tailings, coarse tailings and grits (Botham, 2011; Dunn, 2004). All of the waste streams are non-acid generating. Co-disposal was selected to reduce life cycle cost, to improve long-term stability and to facilitate progressive rehabilitation. The general arrangement of the co-disposal facility is shown on Figure 3. The total footprint area of the facility is approximately 49 ha and it consists of three fine tailings cells and one coarse tailings and grits cell. Most of the waste rock was used as a perimeter embankments for the fine tailings cells and for the coarse tailings and grits cell. The exterior side slopes of the perimeter embankments were placed at the angle of repose (1.5H: 1V). The maximum height of the co-disposal facility was 40 m. The fine tailings were thickened to approximately 50% solids content and pumped using centrifugal pumps. The fine tailings are end discharged from the perimeter embankments. A conveyor belt was used to transport the coarse tailings and grits.

The co-disposal facility was progressively closed. A number of rehabilitation activities were completed during operation. The exterior side slopes of the facility were re-graded to a 3H: 1V slope and benches were provided every 10 m vertical height to help control soil erosion and improve long-term slope stability. After the fine tailings cells were completely filled, they were covered with a layer of coarse tailings and grits for closure. The top surface of the facility was gently sloped and provided with berms to promote infiltration. A topsoil cover was placed over the final geometry of the facility. The topsoil was ameliorated to ensure that it was chemically neutral and suitable for vegetation growth. The topsoil was seeded with perennial grass species indigenous to the general area to establish a sustainable vegetation cover. In addition, storm water catchment berms were constructed along the perimeter toe of the facility to capture run-off water.

Figure 3: The Rehabilitated Oaks Mine Co-disposal Facility

5.4 *Inclusion of Coal Tailings Disposal Cell inside a Coarse Discard Dump in South Africa*

A significant number of coal mines in South Africa include coal tailings disposal cell(s) inside a compacted coarse discard dump (SADEM, 2001; Cameron, 2010). Such co-disposal facilities are located in close proximity to the process plant to reduce haulage and pumping distances. The two waste streams each have a very good potential for re-mining, and therefore they are intentionally not mixed. Often the tailings cell is located in the periphery of the co-disposal facility for ease of operation and re-mining. The coarse discard is used as a perimeter containment.

Coal discards are potentially acid generating, metal leaching and are prone to spontaneous combustion, due to their high permeability. The co-disposal facilities are designed to reduce the volume of water that could infiltrate through the coarse discards and report to the toe as seepage. The seepage is often of poor water quality and requires expensive treatment. Seepage through the coarse discards is typically reduced by compacting the coarse discards in thin lifts, progressively rehabilitating the exterior side slops of the coal discards, discharging the fine discard slurry from perimeter spigot points to maintain a central pond, placing the slurry spigot discharge points away from the perimeter containment wall, and maximizing the recovery of the supernatant water to maintain a small pond and a good length of fine discard beach. The base of the facilities are covered with relatively low permeability soil to reduce the seepage that may infiltrate into the environment. The facilities are provided with a perimeter toe drain to collect the seepage. Some diamond, chrome and vanadium mines in South Africa also use similar co-disposal concept for their coarse and fine tailings. Typical examples of such facilities are presented on Figure 4.

In some mines, shallow cells are created inside coal discard dumps for the disposal of fine tailings. The cells are encapsulated with coarse discards before a new cell is created on top. Illawarra Coal Mine in Australia is one example that is known to have used this co-disposal concept (Gowan *et al.*, 2010).

Figure 4. Examples of Fine Tailings Disposal Cells inside Coarse Tailings

5.5 *Unnamed Mine, South Africa – Tailings Disposal Facility Inclusion in Waste Rock Dump*

An operational polymetallic mine in South Africa started a major expansion in 2006. The mine is located in a mountainous topography with steep valley sides and a number of streams. The mean annual precipitation of the site is around 1,000 mm. Part of the expansion included developing a co-disposal facility to store 18 Mt (9.7 Mm^3) of potentially acid generating tailings within a 136 Mm^3 overburden and weathered waste rock dump.

The co-disposal facility is located in a valley. The facility requires constructing three cross-valley dams (central dam, upstream dam and downstream dam) using overburden waste and waste rock to create two tailings disposal cells as shown on Figure 5. For start-up, the central dam was constructed. The central dam was raised in the downstream construction method. Once the tailings level started to get high, the upstream dam was constructed to its ultimate height. At a later stage, the downstream dam was also constructed. Tailings deposition will eventually bury the central dam. The downstream dam is raised in a downstream construction method. The dams were constructed from bottom up in 1.0 m to 1.5 m lifts. The overburden and waste rock were was hauled using the mine trucks. Water trucks, dozers and graders were used to moisture condition and prepare the fill material for compaction using sheepsfoot and smooth drum vibratory rollers.

Surface flows occur along the co-disposal valley, particularly during the summer rainfall season. Blasted waste rock was placed along the valley to channel out the spring water from the facility. A series of drains was also installed at the base of the tailings disposal cells to remove tailings consolidation water. A water diversion ditch was also constructed to divert clean water away from the co-disposal facility.

Tailings slurry is discharged from spigot points along the dams at about 45% solids content. The tailings deposition plan is aimed at pushing the tailings pond against the natural topography. A floating reclaim pump barge was used to reclaim the supernatant water for re-use to the mill.

Figure 5: Co-disposal Facility of an Unnamed Mine in South Africa

5.6 *Neves Corvo Mine, Portugal – Inclusion of Waste Rock inside a Tailings Disposal Facility*

Neves Corvo is an underground copper and zinc mine located about 220 km south of Lisbon, Portugal. The mine is located in a semi-arid climate with an average annual precipitation of 485 mm, and an average annual evaporation of about 1,315 mm. The mine has been operating since 1988. The mine processes ore at a rate of about 6,850 t/day.

The waste rock and the tailings are acid generating. The tailings were originally deposited as a slurry under water (about 0.5 m to 1.0 m thick) to control oxidation in a 190 ha subaqueous tailings disposal facility created by constructing a 42 m high rockfill dam across a valley. The waste rock was stockpiled separately. Since 2012, a co-disposal facility was developed on top of the previous subaqueous tailings disposal facility to store an additional 27 Mt of thickened tailings and 10 Mt of waste rock (Lopes *et al*., 2015) as shown on Figure 6. Co-disposal was selected as it provided a cost effective way of extending the life of the tailings disposal facility and will also create a stable long-term landform.

The first stage of the co-disposal facility development entailed maintaining the perimeter dam at its previous elevation and sequential construction of 15 cells using internal dykes of waste rock. The dykes are built by first advancing a narrow front of waste rock at about 5 m to 6 m above the pond level and then widening to a minimum crest width of 6 m and lowering to a typical height of 4 m above pond level. Thickened tailings at 60% to 70% solids content are deposited within each cell. The cells are developed in a clockwise fashion as new cells are required, displacing and directing the water cover towards the final cell at the southeast corner, which will act as a sedimentation pond before the water is discharged into a new water reservoir to be constructed outside the facility.

The second stage of development of facility will entail building a stack of thickened tailings above the existing cells, via upstream deposition from the perimeter, to a maximum height of 30 m above the perimeter dam. Waste rock is being used for sequential construction of 2 m high deposition berms at 40 m spacings (5% overall stack slope) and for a 1m thick base cover for the final closure cover.

Figure 6. Neves Corvo Co-disposal Facility

5.7 *La Quinua Mine, Peru – Inclusion of a Tailings Disposal Facility inside Heap Leach Pad*

La Quinua is one of Minera Yanacocha's open pit gold mines located approximately 45 km from Cajamarca in northern Peru, at an altitude of 4,700 m. The average annual precipitation of the site is around 1,380 mm. The mill produces tailings at a rate of 13,700 t/day.

In 2008, the mine commissioned a 45 Mt tailings disposal facility fully contained inside a 530 Mt heap leach pad (Kerr *et al.*, 2010) as shown on Figure 7. Both the tailings disposal facility and the heap leach pads are developed in stages, at the ultimate stage reaching maximum heights of 115 m and 130 m, respectively.

Leach ore is used as a perimeter embankment for the tailings disposal facility. The leach ore is placed in 16 m thick uncompacted lifts. The loosely placed leach ore is subject to application of large quantities of leach solution and percolation flows. To promote vertical downward drainage and to reduce the lateral spread of flow, coarser leach ore is placed at the bottom and finer leach ore is placed at the top of the perimeter embankment.

The tailings are thickened to 67% solids content and then deposited from multiple discharge points to push the tailings pond towards the south-central area of the facility. A blanket drain system is provided above the liner system of the facility directly below the tailings pond area to drain the water by gravity into a downstream reclaim pond. Decant towers are also provided as a back-up if the blanket drain system becomes clogged.

Figure 7. La Quinua Co-disposal Facility

5.8 *Nunavik Nickel Mine, Québec – Inclusion of Tailings Cells inside a Waste Rock Dump*

The Nunavik Nickel Mine is located 1,800 km northwest of Québec, Canada. The mine lies within the Canadian Arctic in the region of continuous permafrost. The average annual precipitation, lake evaporation, and temperature of the mine site are 600 mm, 225 mm, and -9 ºC, respectively. The mine has five ore bodies, four of which are mined out through open pit operations and one through underground operation. The ore bodies are processed at the Expo site at a rate of 4,500 t/day. The mine has been operating since 2011.

Both the tailings and the waste rock are potentially acid generating. The tailings are co-disposed within the Expo Pit waste rock dump. The co-disposal facility consists of two tailings disposal cells and one waste rock cell as shown on Figure 8. The perimeter and internal embankments of the two tailings disposal cells are constructed in two stages using run-of mine waste rock. The waste rock embankments are constructed in 1 m lift compacted using smooth drum vibrating rollers. The crest width of the embankments at the ultimate stage is 10 m and the side slopes are 3H:1V. A 10 m wide bench is provide at the middle of the interior side slope of the embankments and two to three 10 m wide benches are provided at the exterior side slopes of the embankments. The base and the interior side slopes of the tailings disposal cells are fully lined with 1.5 mm white LLDPE geomembrane liner. A 550 g/m^2 non-woven geotextile, screened esker (<20 mm), and a random esker (<150 mm) transition are placed between the geomembrane liner and the waste rock embankment. The excess Expo waste rock, (i.e., the waste rock left after the construction of the embankments for the tailings cells), is end discharged into a standalone cell adjacent to one of the tailings disposal cell.

This tailings are dewatered in a deep cone thickener to 65% solids content. The thickened tailings are discharged from multiple spigot points located on the embankments. The co-disposal facility will be progressively closed with a geomembrane cover as each cell is filled. The geomembrane will be covered with a protective layer of coarse granular material.

Figure 8. Nunavik Co-disposal Facility

5.9 *Cerro de Maimon Mine, Dominican Republic – Layered Co-disposal of Waste Rock and Tailings*

Cerro de Maimon is an open pit gold mine located 75 km northwest of Santo Domingo, Dominican Republic. The site is seismically active. The mean annual precipitation and pan evaporation at the mine site are 2,010 mm and 1,710 mm, respectively. The mine has been in operation since 2008. The mill processes ore at a rate of 2,500 t/day.

The co-disposal facility of the mine allows layered co-disposal of a potentially acid generating weathered waste rock and a potentially acid generating tailings as shown on Figure 9. The co-disposal facility is located on a hillside (Wislesky and Li, 2010). Inert waste rock from the mine operation was used to construct a horseshoe shaped perimeter embankment. The facility is internally divided into three cells to provide flexibility for co-disposal. The perimeter embankment is an engineered structure constructed in thin lifts. The base of the facility is lined with clayey soil and the interior side slopes of the perimeter embankment are lined with geomembrane liner. The tailings are dewatered to solids contents of 55% to 60% to form non-segregating thickened tailings before they are pumped out for disposal.

Field trial tests showed that placing a 2 m thick lift of waste rock over a 1 m thick lift of freshly deposited tailings is the ideal layered co-disposal scheme for the site (Li *et al.*, 2011). A significant increase in density and shear strength of the underlying tailings was noted during the trial. The tailings were moderately desiccated over a short period of time after deposition. The waste rock was end dumped using articulated trucks and spread using low ground pressure dozers. The field trial also found that a minimum of 1.3 m thick waste rock is required to safely operate the co-disposal equipment. The geomembrane liner system of the facility was covered with selected materials to protect it from damage during the placement of the waste rock.

The selected co-disposal concept was observed to provide significant quantifiable benefits including: significant increase in density and shear strength of the tailings deposit, and acceleration of tailings consolidation by shortening the drainage paths. The increase in density of the tailings would also decrease the liquefaction potential of the tailings during seismic events.

Figure 9. Cerro de Maimon Co-disposal Facility

5.10 *Serra Azul Mine, Brazil – Layered Co-disposal of Coarse and Fine Tailings*

Serra Azul is an open pit iron ore mine located approximately 60 km southwest of Belo Horizonte, Minas Gerais, Brazil. The mine site is located in a humid subtropical climate with average annual precipitation in excess of 1,000 mm. The mine processes at a rate of 16,450 t/day.

In 2013, the mine commissioned a coarse and fine tailings co-disposal facility against a steep hill slope (Figure 10). The selected co-disposal concept eliminates water ponding on top of the facility, ensures long-term physical stability, and eliminates the need for purchasing additional land (Lugão *et al.*, 2013). The co-disposal facility contains a rockfill starter embankment with upstream transition and filter zones. The base of the facility was covered with a blanket drain and bottom drains using jig tailings (< 6 mm). Transition and filter zones are provided upstream of the starter rockfill embankment. The perimeter embankment was raised in the upstream construction method using jig tailings. A reclaim pond is provided at the downstream toe of the facility to collect seepage and run-off water from the facility.

The tailings (< 1 mm) are separated into coarse (underflow) and fine (overflow) tailings using cyclones. The coarse and fine tailings are co-disposed upstream of the perimeter embankment. The co-disposal area is partitioned using the coarse tailings. Typically, the cells are 100 m long, 25 m wide and 1 m deep. The berms of the cells are approximately 8 m wide with 1.5H:1V side slopes. The cells are first filled with a 0.5 m thick layer of fine tailings and then with a 0.5 m thick layer of coarse tailings. The fine tailings are trucked to the cells after undergoing settling and dewatering in a series of ponds located at the toe of the co-disposal facility. The cells/berms are raised in 1 m intervals. Waste rock is also planned to be deposited on top of the co-disposal facility once it reaches closer to its ultimate height.

Figure 10. Serra Azul Co-disposal Facility

5.11 *Greens Creek, USA – Co-mingling of Filtered Tailings and Underground Waste Rock*

The Greens Creek is an underground silver mine near Hawk Inlet, Alaska, USA. The annual average precipitation and temperature at the mine site are 1,450 mm, and 5.8°C, respectively. The mine has been operating since 1989. The mill generates 1,800 t/d of tailings. The tailings are dewatered using filter press to 88% solids content. Approximately 50% of the filtered tailings are returned underground for use as structural backfill and the remaining tailings are trucked approximately 13 km to a surface tailings disposal facility.

The tailings disposal facility has the capacity to store 9.6 Mt (4.5 Mm^3) of tailings and 2.3 Mt (1.2 Mm^3) of waste rock (HGCM, 2013). The facility covers a total footprint area of 13 ha and it is about 25 m high. Both the tailings and the waste rock are potentially acid generating. Since 2009, when waste rock became available, it has been co-mingled with the filtered tailings at the tailings disposal facility to reduce reclamation cost and to reduce impacts on environment (Figure 11). The co-mingling process involves end dumping of filtered tailings and waste rock at a ratio ranging from 1:1 to 3.2: 1 and spreading the wastes with a bulldozer and then compacting with a smooth vibratory roller. The co-mingled material typically gets compacted by at least two back and forth passes with a dozer and at least one back and forth pass with a smooth drum vibratory roller. The same compaction technique is used when the filtered tailings are disposed separately.

Figure 11. Greens Creek Co-disposal Facility

5.12 *Kidston Gold Mine – In-Pit Co-disposal of Tailings and Waste Rock*

Kidston Gold Mine is a closed open pit mine located approximately 280 km northwest of Townsville in Queensland, Australia. The mine operated between 1985 and 2001. The mine processed ore at a rate of 13,700 t/day. The mine is located in a semi-arid climate with average annual precipitation and evaporation of about 700 mm and 1,650 mm, respectively.

The mine had two open pits, Wises Hill and Eldridge. Mining started in Wises Hill Pit and was completed in 1996, leaving a final void of 240 m deep and covering a total surface area of 52 ha. Eldridge Pit was developed between 1995 and 2001 to a maximum depth of 260 m and covering a total surface area of 54 ha.

Approximately 35 Mt of waste rock and 27 Mt of tailings generated from the Eldridge Pit were co-disposed inside the Wises Hill Pit (Williams and Currey, 2002). The waste rock was end-dumped from the northeast corner of the pit and thickened tailings were concurrently deposited from the northwest corner of the pit. For most part, the tailings and waste rock occupied separate spaces in the pit, although some mixing occurred at the interface. The potentially acid generating waste rock was placed below the final pit water level to reduce oxidation. The tailings were dewatered to 68% solids content before they were disposed in order to provide sufficient re-turn water to the mill without pumping water from the pit.

The co-disposal offered a number of advantages including: limiting waste rock haul distances, guaranteeing water return, enhancing the recovery of cyanide from the process water, reducing the disturbance footprint and facilitating the rehabilitation of the Wises Hill Pit.

Figure 11. Kidston Mine Co-disposal Facility

5.13 *Pumped Co-disposal of Coarse and Fine Tailings*

Combined pumping of coarse coal discards and fine tailings streams through a pipeline for disposal was pioneered in 1990 at Jeebropilly Colliery in southeastern Queensland, Australia (Morris and Williams, 1997). Since then, a number of coal mines in Australia, USA and Indonesia have adopted the technology. The Australian coal mines that are known to have used pumped co-disposal include: Burton, Charbon, Coppabella, Cumnock, Hail Creek, Lake Vermont, Sonoma, Gordonstone (Kestrel), Moorevale, Moranbah North, North Goonyella, Stratford and others (Gowan *et al.*, 2010).

The coarse rejects (< 50 mm) and the fine tailings (< 1 mm) are mixed in a plant and pumped for disposal into mine voids or surface disposal facilities with perimeter embankments. The combined waste stream is typically pumped using centrifugal gravel pumps at a velocity ranging between 3.5 m/s to 4.0 m/s. Pumping at this high velocity requires a significant amount of power. The high velocity and the abrasive nature of the rejects material cause high pipeline wear. The particles also have a tendency to segregate during deposition.

One alternative to pumped co-disposal of coal rejects is to dewater the fine tailings to a filter cake consistency and then mix it with the coarse rejects on a conveyor belt. Examples of mines that are using or planning to use such a co-disposal concept includes: Dartbrook, Mt. Thorley and Wilpinjong coal mines in Australia, and Trend coal mine in Canada.

6 PLANNED CO-DISPOSAL FACILITIES

In recent years, there has been a surge of metalliferous mining projects planning to use concepts of co-disposal to manage their waste streams. Few of these co-disposal facilities which are known to be close to implementation are briefly described below.

- Krumovgrad Co-disposal Facility, Bulgaria – The facility will be developed against a steep mountain to dispose about 7 Mt of tailings and 15 Mt of waste rock, both inert (Eldridge *et al.,* 2011). Shallow cells will be created in the interior of the facility which will be filled with paste tailings and encapsulated with waste rock. The exterior of the facility will contain a continuous waste rock zone with upstream transition and filter zones. Seepage will be collected at the downstream toe of the facility.
- NICO Co-disposal Facility, NWT, Canada – The facility will be developed against a hill to dispose about 30 Mt of tailings and 97 Mt of waste rock, both potentially acid generating and metal leaching (Habte and Bocking, 2014). Shallow cells will be created in the interior of the facility which will be filled with thickened tailings and encapsulated with waste rock. The exterior of the facility will contain a continuous waste rock zone with upstream transition and filter zones. Seepage will be collected at the downstream toe of the facility.
- Brukunga Co-disposal Facility, Australia – About 8 Mt of waste rock and 3.5 Mt of tailings excavated from the historical pyrite mine and mixed with crushed limestone will be co-mingled and compacted inside the facility which has clay core containment dams (Brett *et al.*, 2011). The co-mingled waste will be covered with a crushed rock overlain by soil cover suitable for vegetation growth, with recreated creeks over the final surface, able to feed water into the permeable crushed rock to maintain the saturation of the co-mingled waste.
- Unnamed Co-disposal Facility in North America – About 12 Mt of thickened tailings and 4 Mt of waste rock, both potentially acid generating, will be co-disposed in a lined facility. The waste rock will be used to construct a 30 m wide perimeter embankment and to encapsulate the top tailings surface during closure. Supernatant water will be decanted from the tailings surface and seepage from the facility will be conveyed into an outside reclaim sump. During operation, another 21 Mt of potentially acid generating waste rock will be stockpiled abutting to the co-disposal facility. During closure, the stockpile will be relocated and backfill the open pit below the predicted minimum water level.
- Unnamed Co-disposal Facility in Africa #1 – About 18 Mm^3 of tailings and 14 Mm^3 of waste rock, both non-acid generating, will be co-disposed in a geomembrane lined facility. The waste rock will be used to construct the perimeter embankment and internal berms. Seepage from the facility will be conveyed into an outside reclaim pond.
- Unnamed Co-disposal Facility in Africa #2 – Two tailings disposal cells, with a total storage capacity of 120 Mt, will be constructed in series inside a 540 Mt waste rock dump. The perimeter embankments of the cells will be constructed in thin lifts from bottom up using waste rock. The base and side slopes of the cells will be lined with geomembrane. Once the cells are filled with tailings they will be encapsulated with waste rock.

7 DISCUSSION

Despite the common perception, there are a number of mines that are currently using the concepts of co-disposal; however the facilities are typically not clearly reported as being co-disposal facilities. The paper has compiled and provided brief descriptions of a few such facilities and a few planned facilities.

The majority of the co-disposal forms currently in practice have only a minor degree of physical mixing between the tailings and waste rock. Nonetheless, by simply disposing of two or more types of mine waste in a single facility, these forms of co-disposal will provide better log-term physical and chemical stability than the current practice of separate disposal of the waste streams. The majority of the co-disposal experience that exists between coarse tailings and fine tailings in coal, diamond, and iron ore can easily be transferred to hard rock metalliferous mines, (with the possible exception of pumped co-disposal).

Co-disposal can be achieved at various solids content of tailings (i.e., varying from slurry to thickened tailings to filtered tailings). The decision should be based on site specific conditions, including: available property footprint, climate, water scarcity, seismicity, the metallurgical process used and economic factors.

To reduce the risk of a potential overtopping failure, the operational philosophy of a co-disposal facility should generally aim at reducing the volume of water ponded on top of the facility. Ideally, this can be done by providing a separate water reclaim pond outside the footprint of the co-disposal facility itself. It is also beneficial to promote consolidation of the tailings, and to provide drainage systems to easily remove consolidation or supernatant water from the facility.

Some forms of co-disposal (such as those involving placement of filtered tailings) will have high capital and operating costs, such that their use will have to be limited to special circumstances. On the other hand, there are a number of co-disposal forms which could be much more cost-effective than the traditional separate disposal. Mines should generally consider various forms of co-disposal during a mine waste management options study. The economic evaluation between the various options should be based on life-cycle cost. Not all co-disposal forms are contingent on the presence of a high waste rock to tailings ratio. The site specific conditions should be carefully assessed to identify the optimal co-disposal concept.

Development and operation of a co-disposal facility will be outside of the range of experience of most mine waste management personnel; hence switching to co-disposal will require training and adjustment. It may require more equipment and personnel than separate disposal facilities. It may require the mining and milling plans to be more integrated. Typically, waste rock disposal is currently handled by the mining team and tailings deposition is handled by the milling team. For co-disposal to be successful these two teams may need to work more closely together.

Operation of a co-disposal facility will be sensitive to the concurrent availability of tailings and waste rock, and the ratio between these two streams will typically vary through the life-of-mine. The co-disposal facility should also be flexible enough to accommodate changes in the mining and milling plans due to market volatility or mine priorities.

8 CONCLUSIONS

The paper showed that co-disposal has been successfully used in different mines, including hard rock metalliferous mines, located in different climatic regions of the world. Co-disposal, in its various forms, should be considered by each mining project when evaluating mine waste management options.

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Characterization of Tailings

Coarse and fine tailings slurry separation in iron ore mines

Philippe Rio Roberge & Frédéric Choquet
WSP Canada Inc., Montreal, QC, Canada

ABSTRACT: Iron ore mines have specific constraints for tailings management: they produce large quantities of tailings covering a wide range of particle size distribution. Regulatory agencies increased restrictions on Tailings Storage Facility (TSF) footprints and market cycles have put pressure on mining companies to reduce their operational costs. Therefore engineering companies must come up with approaches to safely manage tailings within these constraints. This paper discusses coarse and fine tailings slurry separation as an effective solution to optimizing tailings management at iron ore mines. Custom made tailing management plans must be developed to forecast construction requirements and operational procedures must follow the tailings management strategy to be successful. Overall, this approach is beneficial in reducing the environmental risk and impact and is an economical advantage for mine operators. Coarse and fine tailings slurry separation is presently being used on Labrador Trough mine sites. .

1 INTRODUCTION

Coarse and fine tailings slurry separation in large output iron mines as an effective solution to optimize tailings management is discussed. As a matter of fact, iron ore mines produce significant quantities of tailings whereas regulatory agencies have increased restrictions on TSF footprints and market cycles have put pressure on mining companies to reduce their operational costs. Therefore engineering and mining companies must come up with approaches to safely manage tailings.

The paper presents slurry separation as a result of the ore process while illustrating the impacts that tailings separation can have on the project's environmental risks, environmental impact and construction costs.

Finally the strategies developed to manage fine and coarse tailings and the resulting impact on tailings management are described.

2 WHAT IS TAILINGS SLURRY SEPARATION?

2.1 *The separation process*

Tailings slurry separation is the action of creating two tailings slurry streams, one coarser and one finer, to be independently pumped to the TSF.

The stream separation is often achieved in the iron recovery process. At certain steps in the concentration process, the rejected stream from the iron recovery line is sent to a gravity separator. From this separator, the stream still containing iron, called the higher grade concentrate stream or underflow, is then recirculated in the recovery process and the reject stream, called the coarse tailings stream or overflow, is sent to an independent pump box.

The fine tailings comes from the final iron recovery reject stream. This tailings stream containing mostly fine tailings that goes to a tailings thickener where process water can be reclaimed and fine tailings sludge can then be pumped to the TSF.

2.2 *Cut-off Point*

The cut-off point set during the gravity separation process is obviously adjusted based on iron recovery rather than the preferred tailings particle size distribution. However, additional separation equipment can be added in the tailings stream to further segregate the tailings and obtain a desirable Particle Size Distribution (PDS). Figure 2.1 presents the PSD of coarse and fine tailings streams.

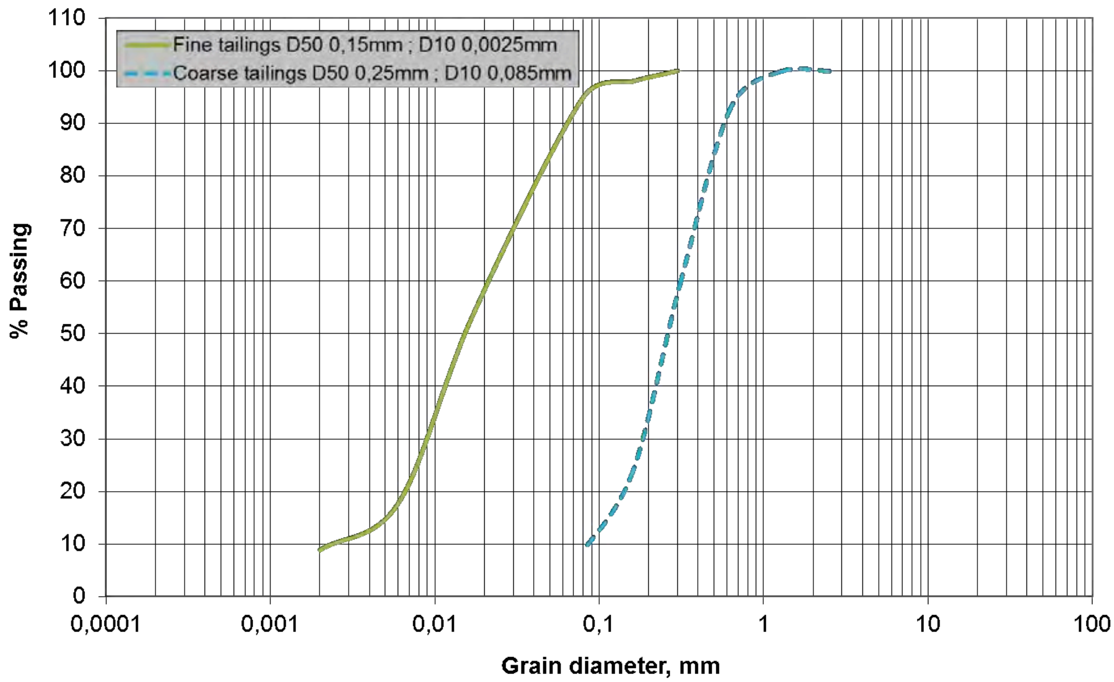

Figure 2.1. Coarse and fine tailings PSD

2.3 *The pumping*

Mixed tailings slurry pumping is generally preferred as it creates a more homogeneous slurry that increases pumping efficiency and reduce operating costs. When separating coarse and fine tailings slurry, the coarse PSD stream is difficult to pump as it requires higher velocities to prevent solid particles from settling in the pipeline. The finer particles conveyed through a different pipeline, can no longer be beneficial to combined slurry pumping by creating a cushioning effect to hold the coarser particles into suspension as they travel through the pipeline. Operators have to constantly monitor pressures and speeds in order not to sand the pipelines. Therefore pumping costs go up and training as well as maintenance and inspections are increased. The separation strategy must therefore justify the increased pumping investment and changes in standard operation procedures by increasing TSF safety and reducing construction costs.

3 WHY SEPARATE TAILINGS SLURRY

Segregating coarse and fine tailings allows enhancement of the coarse tailings properties. The TSF owner can draw benefits from two properties: draining capacity and sedimentation speed.

3.1 *Reducing the environmental impact*

As coarse tailings sediment faster than when mixed, their deposition slope is steeper than mixed tailings. The same amount of coarse tailings can therefore be impounded in a smaller footprint. Fine tailings generally represent a smaller portion of the slurry production. Segregated tailings impounded separately, can have a footprint smaller than if mixed altogether.

Impounding two types of tailings separately can also prove efficient in minimize the impact of tailings storage. If the owner decides to store segregated tailings separately, each storage facility can be adapted and located according to the topography, environmental constraints and its operation can be optimized regarding the properties of each tailings.

If the two tailings flows are discharged into the same facility, the coarse tailings can then be used to confine the fines either at the center or close to the topography. The owner would take advantage of the steeper deposition slope of the coarse tailings to confine the fine tailings.

The rapid draining of coarse tailings also enhances the recovery of transport water. This has an impact on the water consumption during winter as less water remains frozen in the tailings. Ice lenses are less likely to occur and the amount of melting water returning in the water system during the spring melt is proportionally reduced. The capacity of the process water pond can be optimized and the operation of the water management system is simplified as the volume to manage is slightly reduced.

However, as the pumping system is doubled to pump two types of tailings slurry, the overall length of piping is increased, increasing the probability of leaks and the amount of monitoring and surveillance.

3.2 *Reducing the environmental risk*

The rapid sedimentation speed of coarse tailings results in a smaller sedimentation volume of water for coarse tailings when they are impounded separately from fine tailings. The pond which collects runoff and contact waters from the tailings can be relocated outside of the TSF as primary sedimentation will occur faster within the TSF. This opportunity meets the first recommendation of the Report on Mount Polley Tailings Storage Facility Breach (Morgenstern et al. 2015).

Earthquakes are associated with the release of seismic waves which cause increased shear stresses on the embankments and increased pore pressures in saturated tailings. Liquefaction may also be caused by mine blasting or nearby motion and vibrations of heavy equipment. Tailings and the dam may liquefy during seismic events.

When separating coarse from fine tailings, the draining capacity of coarse tailings results in a desaturation (i.e. decreasing pore pressures) of the biggest portion of the tailings stack. This also goes in the direction of the recommendations provided in the Report on Mount Polley Tailings Storage Facility Breach (Morgenstern et al. 2015).

Coarse tailings can also be used to build a deposition beach along a dyke. These beaches if only composed of coarse tailings can provide drainage for the fine or mixed tailings confined by the dyke. The coarse tailings as presented in Figure 2.1 can act as a filter for the fine tailings. When adequately densified, coarse tailings are less prone to liquefication (Anderson and Eldridge, 2010).

By segregating fine from coarse tailings, the owner reduces the amount of material prone to liquefaction. The liquefiable tailings are better located (either in a certain portion of the TSF or in a separate TSF). It becomes easier to manage the risks and to monitor each tailings according to its specific behavior.

3.3 *Reducing the construction costs*

Coarse tailings are mainly composed of coarse sand. This material can be used as a mass backfill to raise peripheral dykes and dams of a TSF. The segregation done at the plant helps control the quality of the discharged material made available for construction.

Coarse tailings can be discharged where needed for the construction. They can be discharged either to form a borrow zone that will be excavated and hauled, as hydraulic fill material used for upstream raises or as a stabilization beach along dykes or dams.

In the first case, sand is excavated and transported by trucks from a stockpile built inside the TSF. From the pile, tailing sand is hauled to the worksite where it is spread and compacted as a conventional granular backfill. The provision of a reliable (in terms of quantity and quality) backfill material borrow zone to the contractor in charge of the earthworks plays an important role in the control of the construction costs.

The flexibility of the pumping allows the creation of the stockpile where it fits the best within the TSF depending on available access and location of the earthworks. But with a robust pumping system, it is possible to fill the stockpile outside of the construction period or when manpower availability is not critical for the TSF maintenance or operations. As coarse tailings drain quickly, the stockpile can even be built up in the winter, outside of the construction schedule at a northern mine site. Experience on northern Quebec sites revealed little to no ice accumulation was found in coarse tailings discharged at a borrow zone during the winter. However, when the summer excavation reached the mixed tailings at the bottom of the borrow zone, lumps of frozen fine tailings (resulting from a previous mixed slurry discharge) were present.

Coarse tailings can be discharged along the crest of the dykes to be raised as a hydraulic fill. In this scenario, two options can be used.

1) Using the fact that coarse tailings drain quickly, the discharged tailings can be placed continuously with heavy equipment. The material can be used for the upstream raises of peripheral dykes. Once compacted, the material will be the mass backfill of the mechanically raised dykes.
2) The steep deposition slope adopted by the coarse tailings will provide appropriate deposition beaches (spigotting). These beaches keep water from the the sedimentation pond or the discharged slurry away from the dykes crests. This configuration lowers the water table through the retaining structures and generally improves their stability. An example of spigotting is presented in Choquet et al. 2015.

The rapid settlement of coarse tailings enhances the performance of the mine team dedicated to spigotting. As the slope angle is steeper than a mixed slurry, less material is required to fill a same stretch of beach. The rapid drawdown of the water table inside the deposited beach makes it more readily available for heavy equipment traffic, pipeline installation or a second phase of spigotting.

Despite the higher pumping costs and pumping investment required, the segregation of coarse and fine tailings can reduce the construction costs of a TSF over the long term as follows:

- Provision of a reliable source of backfill material in terms of quantity and quality;
- Borrow zones that can be filled according to manpower availability
- Faster hydraulic fill for the structures raised by spigotting

4 MANAGEMENT STRATEGIES

Depending on project specific constraints, coarse and fine tailings management strategies can be adjusted to achieve specific objectives. However, the design of the TSF containment and conveyance structures must take the management strategy into consideration. This is the key aspect in reducing costs and optimizing the operation. If the operations team does not correctly describe the operation and development strategies to the design team, containment and conveyance structure designs will not be optimized to the actual operation.

4.1 *Managing fine tailings*

Fine tailings management can be done in the same disposal area or in a separate area. In both cases, the fine tailings deposition plan requires little field intervention in the short term.

When disposing in the same area, a specific location must be chosen to deposit the fine tailings. This zone is selected by its topographical features that will allow safe storage with minimum construction and operational requirements. Features like topographic containment, ground slope, soil conditions, end of pipe distance from the settlement pond and the downstream environment are taken into consideration when selecting the location of fine tailings disposal.

The decision of storing fine tailing in a separate TSF or cell from the coarse tailings, is taken based on multiple factors. These include: detrimental effects to the optimal tailings management plan of depositing both streams in the same TSF, embankment design required to safely manage the fine tailings, available footprint to manage tailings and water management requirements.

4.2 *Managing coarse tailings*

As discussed previously, coarse tailings can be used for three reasons:

- Hydraulically placed borrow materials for standard excavation, transport and backfill operations,
- Spigotted beached material for dyke stabilization,
- Spigotted and dozed material for upstream raises.

The efficient use of coarse tailings any one of or all of these reasons depend on project specific needs and require specific operational procedures and techniques.

The coarse tailings management strategy is likely to require more field and mill operator effort to effectively utilize this material. Mill operators will be required to activate and shut off pipelines on a more frequent basis to deliver the material to the selected areas. Field operators will be required to operate equipment and supervise the deposition on a more frequent basis to optimally shape the material. The combination of both is called operational flexibility. For coarse tailings deposition to be optimally operated, the tailings pumping and field operations must be able to use operational flexibility.

Pumping challenges have been described in section 2.3. Field challenges come from increased activity related to optimized material shaping and by risk of deposition overtopping. When managing a coarse tailings stream, one must understand that the deposition cone buildup occurs rapidly. Techniques and procedures must be developed and applied systematically for the operation to remain safe while being very efficient.

4.3 *Seasonal management*

In cold climates, thought has to be put into the TSF execution plan when working with coarse and fine tailings slurries.

Winter conditions are far from ideal when working around coarse tailings hydraulic deposition. As a result, the rapid build-up of the deposition cone requires constant supervision, tailings shaping and/or frequent end of pipe handling. The consequence of a pipeline sanding is increased when the slurry freezeup occurs within a few hours of pump stoppage. Winter construction work may not be optimal as hydraulically placed borrow material becomes difficult to work with or it is improper for use as construction material.

To overcome these challenges, designers and operator can either stop the tailings separation process by mixing the coarse and fine feeds together or they can push on by increasing mechanical effort and operational awareness to handle the coarse tailings during the winter.

4.4 *Operation, maintenance and surveillance manual*

When operating two tailings slurry streams, the reduction of embankment construction costs as well as the reduced environmental risk comes with additional supervision and maintenance responsibilities. The advantages of optimizing the deposition strategy based on utilizing a coarse tailings stream generally means more human involvement in both the field the mill. Field personnel must manipulate pipelines and spigot ends on the existing embankments, creating situations where incorrect installation and supervision can lead to health and safety issues as well as a containment breach. At the same time, mill operators must operate pump sets and valves to direct the tailings streams where required, creating situations where pipes burst, pipeline sanding and pump box overflow can occur leading to down time.

Therefore, the TSF OMS manual recommended by the Mining Association of Canada must become a reference manual for operators to constantly remind them of the objectives, procedures, risks and emergency measures to be observed on a daily basis. The manual must constantly be updated with the latest operation procedures, training and knowledge transfer is crucial for the operators to guarantee sustainable operations over the long term. The participation of all parties in a tailings management team (mine, mill, environment, TSF field operations and designers) is required for the management strategy to be successful. Yearly objectives must be set, KPI's developed and deposition progress monitored. Only then can operators achieve constant results by overcoming the challenges, adapting to change, and training for the future.

4.5 *Reference project*

The Bloom Lake iron ore mine (Québec Iron Ore Inc. 2017), previously developed by Consolidated Thompson, operated by Cliffs Natural Resources and now owned by Québec Iron Ore Inc, a subsidiary of Champion Iron Limited, utilizes coarse and fine tailings slurry separation on a year round basis. The innovative strategy and methods developed for this project required coarse and fine tailings deposition in separate management facilities due to the limited footprint permitted by the provincial government. The steep deposition angle of the coarse tailings is integrated in the construction and deposition strategy in order to optimize the limited space available. The fine tailings are contained in a dyke surrounded storage facility as their angle of deposition is much lower. Figure 4.1 presents a view of Bloom Lake coarse tailings deposition. One can observe on the left hand side of the picture that coarse tailings have an approximate angle of deposition of 6%.

Figure 4.1. Bloom Lake coarse tailings deposition

5 CONCLUSION

The iron ore tailings separation is based on the segregation which takes place during the ore process. It essentially consists in keeping the coarse and fine tailings streams produced at the concentrator in two separate streams. Tailings can then be discharged into the same TSF or in separate TSFs, depending on the objectives.

Despite the increase in pumping cost and investment, the separation can result in lower environmental impacts, lower environmental risks and a reduction of construction costs.

Tailings separation improves the properties of the coarse tailings which represent almost 80% of the tailings production. Due to their enhanced draining capacity and steeper deposition slope, coarse tailings can be used to create:

- a borrow zone of mass backfill to be excavated during the construction period,
- hydraulically placed backfill raised with heavy equipment to build upstream raises of dykes,
- deposition beaches which will improve the performance of the retaining structure.

All of these can either improve the cost control throughout the construction period, facilitate the environmental risk management during over TSF lifespan and/or decrease the environmental impact of the project during the preliminary engineering phases.

Managing separated tailings helps reduce the water volume of water stored with the tailings and improve the overall desaturation in the TSF. The technology goes towards the recommendations of the Report on the Mount Polley Tailings Storage Facility Breach.

To some extent, tailings separation is an intermediate step between conventional mixed hydraulic deposition and dry stacking. It provides a simple but efficient way to reduce construction costs, improve construction performances and operational risk management. The concept is currently in application in Labrador Trough iron mines.

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Design, manufacture and calibration of sensors for monitoring settling, consolidation and desiccation of tailings

C. Zhang, D.J. Williams, & X. Lei
The University of Queensland, Brisbane, QLD, Australia

ABSTRACT: In order to acquire continuous and real-time data on the settling, self-weight consolidation and desiccation of tailings, a holistic monitoring system has been developed. It comprises in-house manufactured moisture and suction sensors, balances, a data logger, and a web interface, plus a camera for capturing images of the tailings surface as it undergoes desiccation. The in-house manufactured monitoring system is substantially less expensive and more user-friendly than proprietary equivalents, and is capable of the long-term measurement of volumetric water content, suction and mass balance, and of capturing images of the tailings surface. The new sensors were calibrated together in two types of tailings, in a laboratory desiccation apparatus, under the same sequence of settling, self-weight consolidation, initial desiccation, re-wetting and re-desiccation, which would occur in the field. The two tailings samples tested were clayey red mud and sandy coal tailings. Initial desiccation, and re-wetting and re-desiccation, experiments were carried out using both tailings samples to test the performance of the sensors and the monitoring system. The data collected also defined the drying soil water characteristic curve (SWCC) of the two tailings samples, which were compared with those obtained using the Fredlund SWCC device.

1 INTRODUCTION

Most tailings storage facilities (TSFs) employ the sub-aerial deposition of tailings as a slurry, in which the deposited tailings subsequently experience settling, self-weight consolidation and desiccation. Optimising the efficiency of tailings desiccation is the key to improving their shear strength, and hence the operational safety and subsequent rehabilitation of the stored tailings (Shokouhi *et al.* 2017). Desiccation of the tailings also reduces the volume required for their storage. Ideally, tailings deposition should be carefully designed so that: (i) the thickness of each deposited layer is limited to allow capillary forces to effectively deliver tailings water up to the surface where it can be evaporated, and (ii) the tailings deposition cycle time is sufficiently long to maximise desiccation. To achieve these outcomes, it is important to understand tailings behaviour during settling, self-weight consolidation and desiccation, particularly the change over time in the evaporation rate and its exponential drop-off with depth, and the resulting change over time in the volumetric water content and suction profiles with depth. In this study, an integrated tailings monitoring system is developed to acquire continuous and real-time profiles with depth of volumetric water content, suction, and water loss due to evaporation, either in the laboratory or in the field. The system comprises sensors, a balance, a data logger, a web user interface, and a camera. The system is adapted for a laboratory desiccation apparatus used for calibrating multiple sensors at one time, and for capturing the tailings desiccation behaviour. This paper describes the monitoring system, the desiccation apparatus and the desiccation experiments. The results of the four tailings desiccation experiments are presented, and a discussion and conclusions are presented that outline the capability and limitations of the monitoring system and future plans for its application in the laboratory and in the field.

2 EQUIPMENT DESCRIPTION

2.1 *Tailings monitoring system*

The tailings monitoring system incorporates a new moisture sensor and two new suction sensors that have been manufactured in-house. The new dielectric moisture sensor measures the volumetric water content indirectly through the dielectric permittivity of the medium (see Figure 1A). The dielectric constant of water of about 80 is substantially higher than that of dry tailings of about 3. All of the electrical components of the sensor are carefully sealed, to ensure the longevity of the sensor in hypersaline, acidic or alkaline environments. Unlike the commercially available time domain reflectometry-based moisture sensor, which is affected by changes in both volumetric water content and salinity, as occurs on desiccation, the new moisture sensor measures volumetric water content irrespective of salinity up to that of seawater.

The new dielectric suction sensor is based on the new dielectric moisture sensor, and has two porous ceramic plates attached on the two flat surfaces of the sensor (see Figure 1B). At equilibrium, the suction in the ceramic plates is the same as that in the surrounding tailings. Since the particle size distribution and the SWCC of the ceramic plates are known, the volumetric water content in the ceramic plates measured by the sensor can be converted to suction, which is the same suction as that of the surrounding tailings.

The new thermal suction sensor measures the tailings suction indirectly from the rise in temperature on heating (Figure 1C), and also provides the ambient temperature. Being able to generate heat as well as measure ambient temperature, the electrical module of the sensor is slotted in a porous ceramic cylinder of which the particle size distribution and the SWCC are known. Heat produced by the electrical element in the centre of the sensor starts to dissipate to the surrounding tailings through the enclosing ceramic cylinder. Thermal equilibrium is achieved in 30 s, when the temperature of the sensor and that of the surrounding tailings equilibrate. As the bulk thermal conductivity of the ceramic cylinder increases monotonically with increasing volumetric water content, low suction (corresponding to a high volumetric water content) would lead to low temperature rise in the tailings on heating. The advantage of these two new suction sensors, compared to the commercially available suction sensors is that suction measurement range of the sensor can be varied by changing the porous ceramic. Specifically, ceramics with large particles (e.g., silica ceramic) are able to measure a lower suction range, and vice versa.

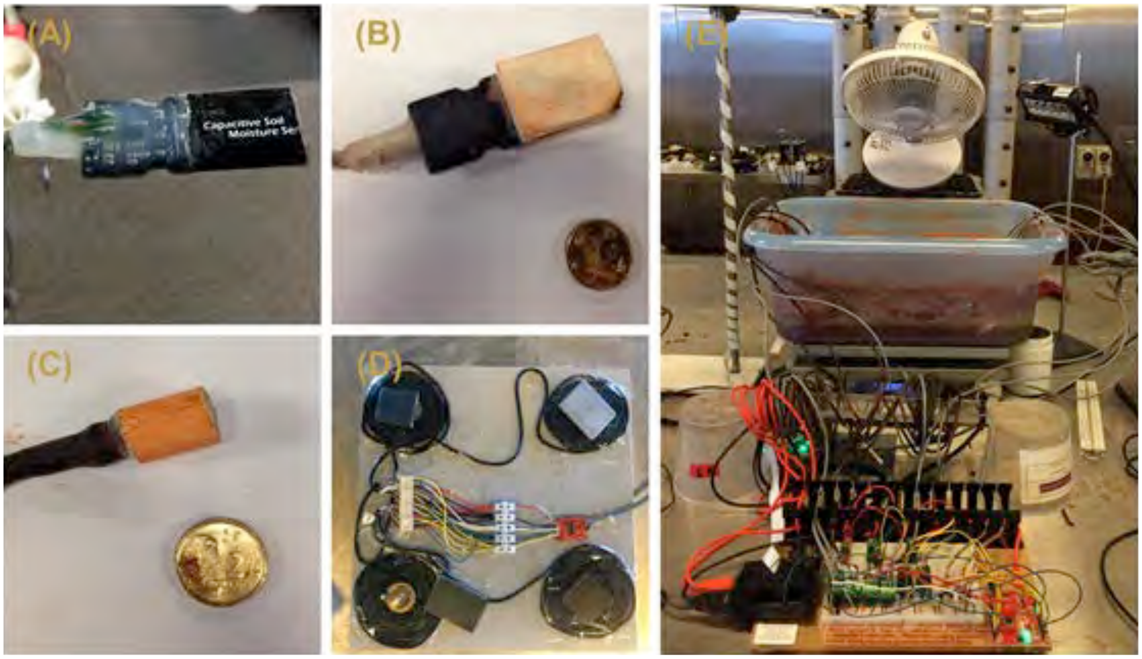

Figure 1. Tailings monitoring system: (A) dielectric moisture sensor, (B) dielectric suction sensor, (C) thermal suction sensor, (D) load cell balance, and (E) tailings desiccation apparatus.

The integrated monitoring system is also equipped with an in-house manufactured balance, which replaces conventional scales. The new balance comprises four button load cells between pairs of square PVC plates. The new balance measures the mass of the desiccation apparatus to an accuracy of 10 g and a capacity up to 100 kg (see Figure 1D). The in-house balance is less affected by wind than scales would be, has low power consumption due to the absence of unnecessary control panels and LED displays, can be stacked for added redundancy, and is relatively inexpensive to manufacture. This in-house balance is capable of working either in the laboratory or in the field, for measuring the loss of moisture on desiccation.

All of the sensors, the balance, and the camera are controlled by a new in-house manufactured data logger, equipped with 15 analog and 57 digital channels. The new data logger is driven by open-source hardware and software, and is able to acquire data individually and sequentially from each sensor type and each individual sensor at a specified time interval. In addition to data acquisition, each sensor channel provides a relay that powers each individual sensor in turn when data acquisition is due. This results in low power consumption, and little heating of cables and sensors that may cause zero shift and non-linear calibration. In addition, the monitoring frequency can be varied and the data delivered via an internet browser in real-time, enabling access to the system for all interested parties.

Commercially available data loggers are dedicated to a particular sensor type and are driven by propriety software, resulting in multiple data loggers being required for multiple sensor types, and limited scope for real-time data downloads. They also continuously power all sensors connected to them, leading to heating and excessive power demand.

2.2 *Tailings desiccation apparatus*

The holistic monitoring system was adapted to a tailings desiccation apparatus used for calibration purposes. The apparatus comprises a container measuring 420 mm long by 280 mm wide by 100 mm high; large enough to accommodate multiple sensors of various types. The container is placed on a mass balance for continuous monitoring of the loss of moisture due to evaporation.

Tailings can be placed in the apparatus at any consistency, allowed to settle and consolidate under its self-weight, and then dried using either a heat lamp or fan, or both, to obtain the drying SWCC of the tailings. The thickness of the tailings in the container is set at 40 mm, to ensure relatively uniform drying through its thickness, while being thicker than the zone of influence of the sensors. All the sensors are embedded in the middle of the tailings layer to provide average readings.

In all of the experiments using the desiccation apparatus reported herein, seven sensors were employed: two dielectric moisture sensors, two dielectric suction sensors, and three thermal suction sensors. An electrical fan was used to accelerate the desiccation process. A high-resolution camera was mounted about 400 mm above the tailings surface to capture the desiccation process, including shrinkage and cracking (Konrad & Ayad, 1997) and salt precipitation (Fisseha *et al.* 2010) of the tailings at four-hourly intervals. A flash was incorporated to ensure a uniform light source for the photographs, including at night. All sensors, the mass balance, the camera, and the light source, were controlled by a dedicated data logger.

2.3 *Four desiccation experiments*

Two different types of tailings were used to calibrate the sensors: (i) red mud from Queensland Alumina Limited, located in Gladstone, Australia, and (ii) coal tailings from Jeebropilly Coal Mine, located in the Ipswich Coalfields of south-eastern Queensland, Australia. Two desiccation experiments were carried out on each tailings. Both tailings were deposited in the desiccation apparatus at an initial solids concentration of 25% by mass, similar to the consistency that they are deposited into their respective TSFs.

The red first red mud desiccation experiment involved the initial desiccation of the red mud slurry. Due to its fine practical size distribution and low initial dry density, the red mud underwent considerable shrinkage and cracking on initial desiccation. The experiment was continued until there was no further desiccation with time.

The second red mud desiccation experiment followed re-flooding of the container with 4000 mL of deionised water, amounting to the water lost during initial desiccation. Some of the sensors were located in the previous randomly located desiccation cracks, while others were within uncracked blocks of red mud. This enabled the different behaviour of the cracked and uncracked red mud to be followed on re-flooding and re-desiccation.

Similarly, the first coal tailings experiment involved its initial desiccation of the slurry, while the second desiccation experiment followed re-flooding of the container with 4000 mL of deionised water containing 10 ppt concentration of sodium chloride designed to explore the impact of salt on the sensor readings.

In all desiccation experiments, the calibration procedure comprised: (i) calculation of the actual volumetric water content and suction of the tailings based on the weight loss measured by balance and the SWCC obtained using the Fredlund SWCC device, (ii) measurement of the volumetric water contents and suctions over time using the sensors, (iii) comparison between the actual values of volumetric water content and suction with the raw readings from the sensors to obtain calibration factors, and (iv) comparison between the drying SWCCs obtained from the sensors with that obtained from the Fredlund SWCC device.

3 EXPERIMENTAL RESULTS

Figure 2 shows the data and photographs of the surface obtained during the red mud initial desiccation Experiment 1. As supernatant water was initially abundant on the surface, a relatively high evaporation rate was observed during the first 3 days, with some fluctuations due to the daily changes in temperature and humidity. At the start of the second day, the red mud started to desaturate, as identified by the drop in the volumetric water content and marked by the onset of cracking (see Figures 2E and 2F). The maximum evaporation rate occurred at the start of the third day when the red mud was unsaturated and cracks were well developed. This was attributed to the newly formed cracks enlarging the surface area for evaporation, to be greater than that from the top surface alone. This high evaporation rate did not last for long, and the rate declined rapidly from immediately after the maximum as the available moisture diminished. Accompanying the fast decline in volumetric water content, the suction increased steadily and salt crystals started to appear on the surface of the red mud. The red mud became completely desiccated, with the evaporation rate and volumetric water content reaching close to zero, and the suction remaining at a constant high value.

During the initial desiccation of the red mud, all of the dielectric moisture sensors captured very well the desaturation of red mud, despite one of the sensors (dielectric moisture sensor B) becoming partially exposed in a crack at the end of the experiment. The readings from the thermal suction sensor agreed very well with the suctions based on the balance and the Fredlund SWCC. However, the readings from the dielectric suction sensors showed a poorer match to those based on the balance and the Fredlund SWCC for suctions lower than 10^5 kPa. This is probably due to the high air-entry suction of 10^3 kPa of the ceramic plate used in these sensors. The thermal suction sensor had a ceramic cylinder with an air-entry suction of only 50 kPa, allowing suction measurement from this value and above.

Figure 3 shows the data and photographs of the surface obtained during the red mud re-desiccation Experiment 2, following re-flooding after the end of Experiment 1 and waiting 2 days for the red mud to re-wet. The re-flooding resulted in a reduction in suction, but neither broke the blocks nor narrowed the cracks. The added water largely filled the cracks and covered the surface. On re-desiccation, the changes over time of evaporation rate, volumetric water content and suction were similar to those obtained on initial desiccation, with some unique features induced by the pre-existing cracks. As deionised water covered the entire surface at the beginning of Experiment 2, the evaporation rate in the first 2 days was higher than that in Experiment 1, which involved salty supernatant water. The moisture sensor partially exposed in the crack (dielectric moisture sensor B) showed desaturation earlier than the sensor fully embedded in the red mud (dielectric moisture sensor A), reflecting the early loss of water from the crack. Although some suction sensors were visible in the cracks (dielectric suction sensor B and thermal suction sensor C), their readings remained close to the sensor completely embedded in the red mud (dielectric suction sensor B), although the thermal suction sensor recorded lower suctions.

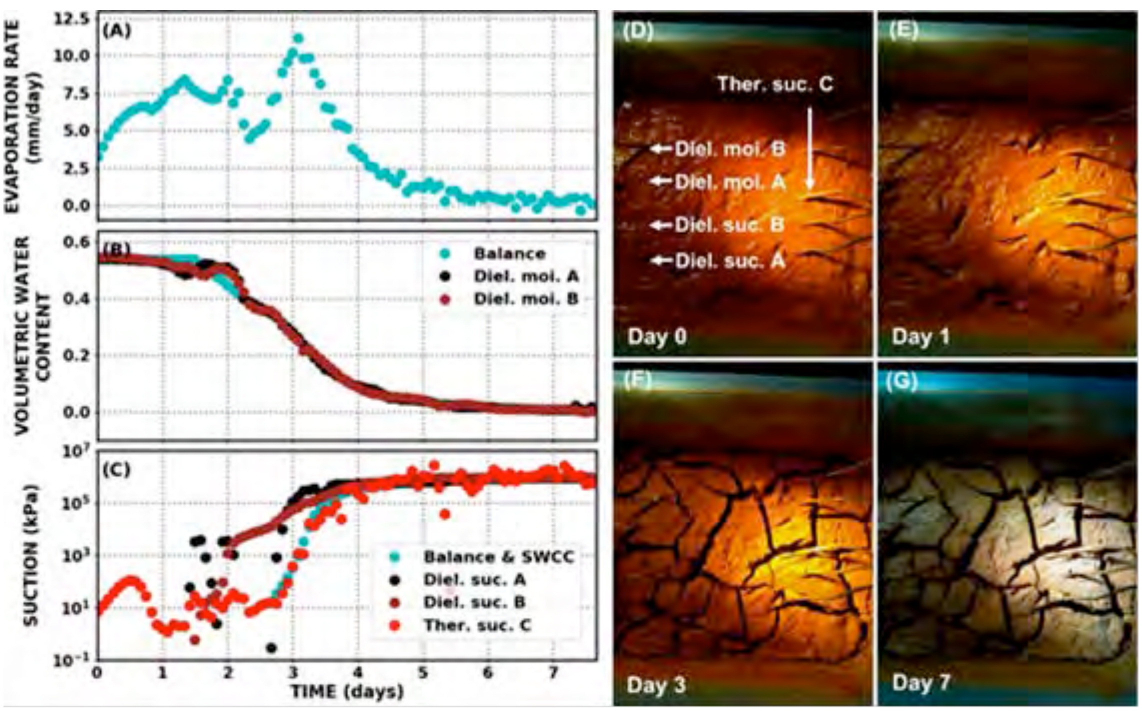

Figure 2. Results of red mud initial desiccation Experiment 1: (A) evaporation rate obtained from balance, (B) volumetric water content obtained from two dielectric moisture sensors and balance, (C) suction calculated from balance and Fredlund SWCC and obtained from suction sensors, (D) photograph of surface at beginning of experiment, (E) photograph of surface at onset of cracking, (F) photograph at end of crack formation, and (G) photograph at end of experiment.

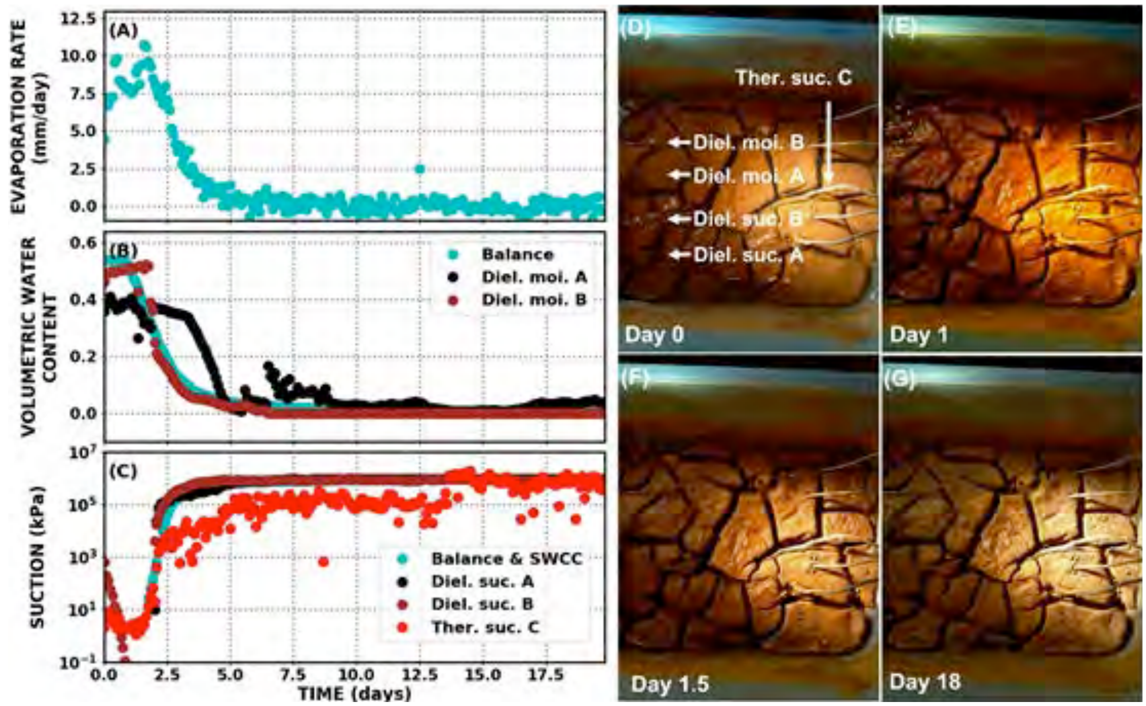

Figure 3. Results during red mud re-desiccation Experiment 2: (A) evaporation rate obtained from balance, (B) volumetric water content obtained from two dielectric moisture sensors and balance, (C) suction calculated from balance and Fredlund SWCC and obtained from suction sensors, (D) photograph of surface at beginning of re-desiccation, (E) photograph when water remained in crack, (F) photograph when water in crack was completed evaporated, and (G) photograph at end of experiment.

Figure 4 shows the data and photographs of the surface obtained during the coal tailings initial desiccation Experiment 1. The evaporation rate was highest at the beginning of the experiment, followed by a sharp decrease in evaporation rate and volumetric water content. Due to their sand-size, the coal tailings did not develop significant cracks on the surface, although dry patches of relatively lighter colour appeared (see Figure 4E) and subsequently enlarged (see Figure 4F), indicating an increase in salinity. All of the moisture and suction sensors captured relatively well the drying behaviour of the coal tailings, as demonstrated by their consistency with data obtained from the balance and the Fredlund SWCC.

Figure 5 shows the data and photographs of the surface obtained during coal tailings re-desiccation Experiment 2. It followed re-flooding using saline water after the end of Experiment 1 and waiting 2 days for the coal tailings to re-wet. The effect of the added saline water is seen to be an initial evaporation rate of about half that obtained in Experiment 1 with less saline pore water, and evaporation extended for longer. Once the tailings dried, significant salt crystalisation was observed on the tailings surface (see Figure 5F). The presence of salt also affected the responses of the dielectric moisture sensors (see Figure 5B), which showed a delayed drop in volumetric water content by 4 to 6 days compared to that shown by the balance. While the thermal suction sensors reproduced well the suctions based on the balance and the Fredlund SWCC, the presence of salt led to the dielectric suction sensors showing a delayed increase in suction (see Figure 5C).

Figure 6 compares Fredlund SWCCs with those obtained by a combination of volumetric water content from the balance and the suctions measured using the two suction sensors, for both the red mud and the coal tailings. The Fredlund SWCCs for the two tailings are smooth curves, which the respective SWCC data measured in the desiccation apparatus straddle. The SWCC data from the two suction sensors show a similar amount of scatter, and the matches between the Fredlund SWCCs and the SWCC data measured in the desiccation apparatus are reasonable, despite the effects of cracking. The addition of salt in the coal tailings re-desiccation Experiment 2 results in greater deviation of the dielectric suction SWCC data from the Fredlund SWCC, due to the delayed response of the dielectric suction sensors. The relatively fine-grained red mud had a higher air-entry suction (~10 kPa) than the relatively coarse-grained coal tailings (~1 kPa). At the same suction, the red mud had a relatively higher volumetric water content than the coal tailings. Both the dielectric and thermal suction sensors overestimate the air-entry suction, particularly for the red mud. This is probably caused by the higher air-entry suction of the ceramics than that of the tailings.

4 DISCUSSION AND CONCLUSIONS

Understanding the combined settling, consolidation and, in particular, the desiccation of tailings is the key to improving field dewatering efficiency as well as optimising the tailings deposition thickness and cycle time to a tailings storage facility. The monitoring system described herein is capable of following these processes continuously and in real-time, and can be deployed in the laboratory or in the field. The desiccation apparatus provides a means of calibrating the in-house manufactured sensors described herein.

Initial desiccation and re-desiccation experiments were carried out in a laboratory desiccation apparatus on relatively fine-grained red mud and relatively coarse-grained coal tailings to test and calibrate the in-house moisture and suction sensors, and to obtain drying SWCCs for the two tailings.

As found in other studies (e.g., Mortl *et al.* 2011; Malicki & Walczak, 1999), the readings from the dielectric moisture sensors were influenced by salinity. Improved accuracy could be achieved if the salinity were known. The readings from thermal suction sensors were confirmed to be independent of tailings salinity. The drying SWCC data obtained from the desiccation apparatus straddled reasonably well the drying SWCC measured using the Fredlund device, although the data over-estimated the air-entry suction determined using the Fredlund device. The estimation of the air-entry suction could be improved by reducing the air-entry suction of the ceramics applied in the suction sensors or by deploying multiple suction sensors with different measurement ranges.

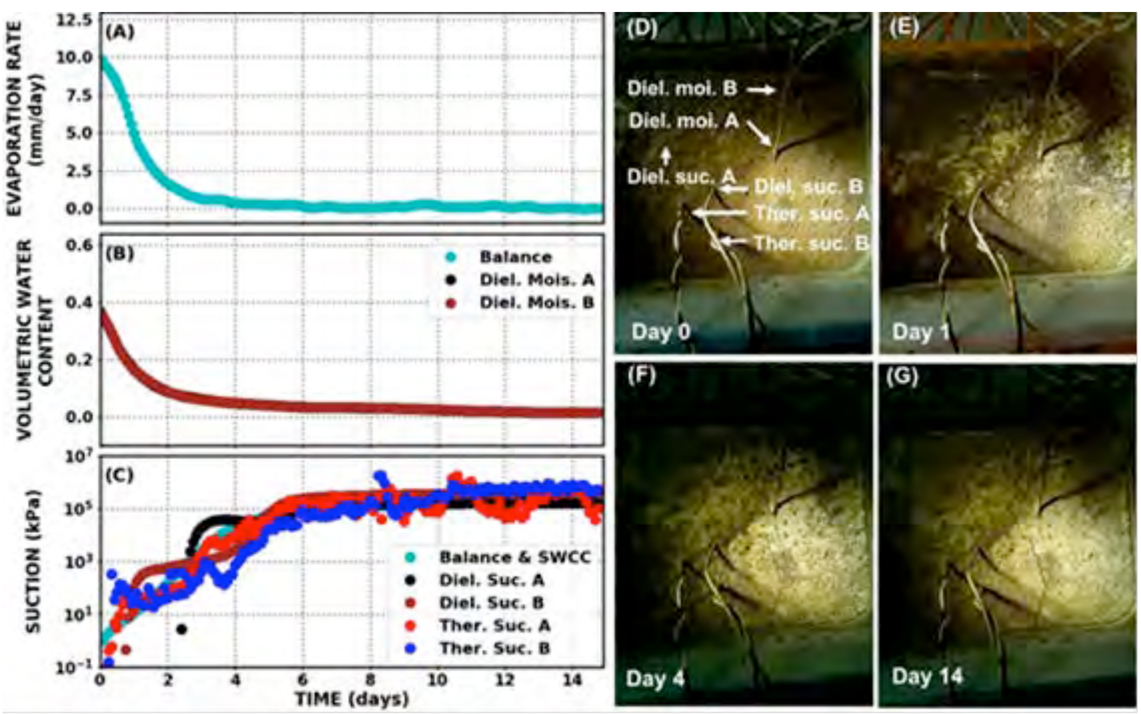

Figure 4. Results during coal tailings initial desiccation Experiment 1: (A) evaporation rate obtained from balance, (B) volumetric water content obtained from two dielectric moisture sensors and balance, (C) suction calculated from balance and Fredlund SWCC and obtained from suction sensors, (D) photograph of surface at beginning of desiccation, (E) photograph when dry patches started to appear on surface, (F) photograph when salt started to crystalise on surface, and (G) photograph at end of experiment.

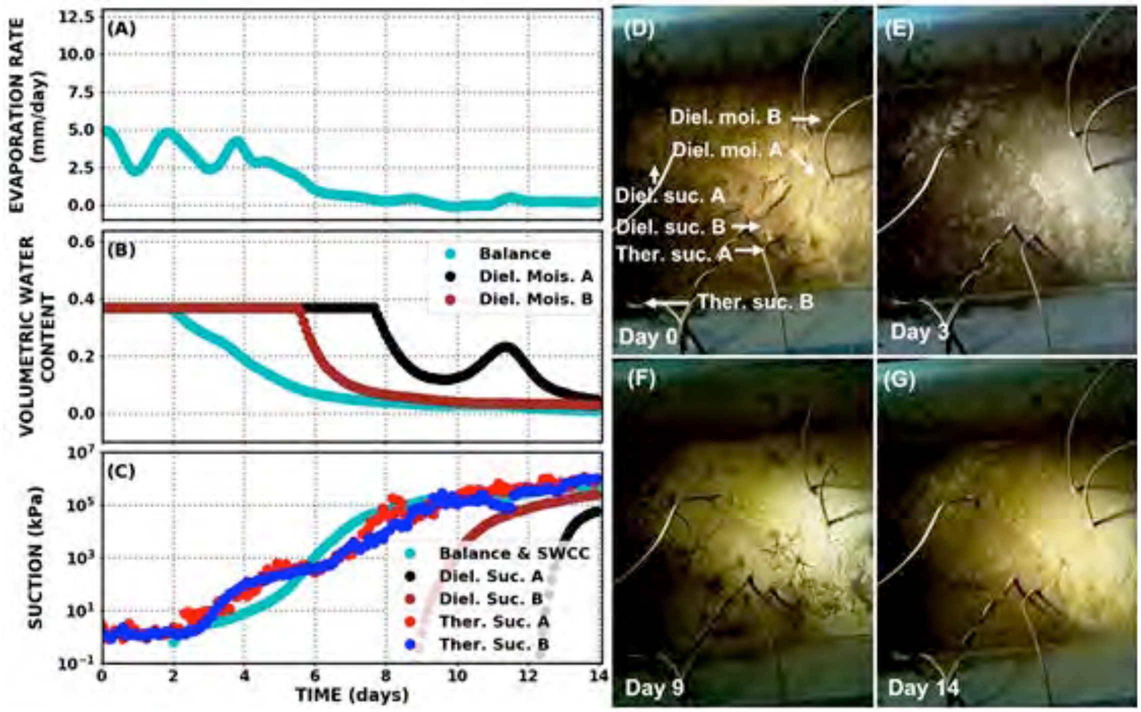

Figure 5. Results during coal tailings re-desiccation Experiment 2: (A) evaporation rate obtained from balance, (B) volumetric water content obtained two dielectric moisture sensors and balance, (C) suction calculated from balance and Fredlund SWCC and obtained from suction sensors, (D) photograph of surface at beginning of re-desiccation, (E) photograph when surface became exposed, (F) photograph when significant salt had crystalised on surface, and (G) photograph at end of experiment.

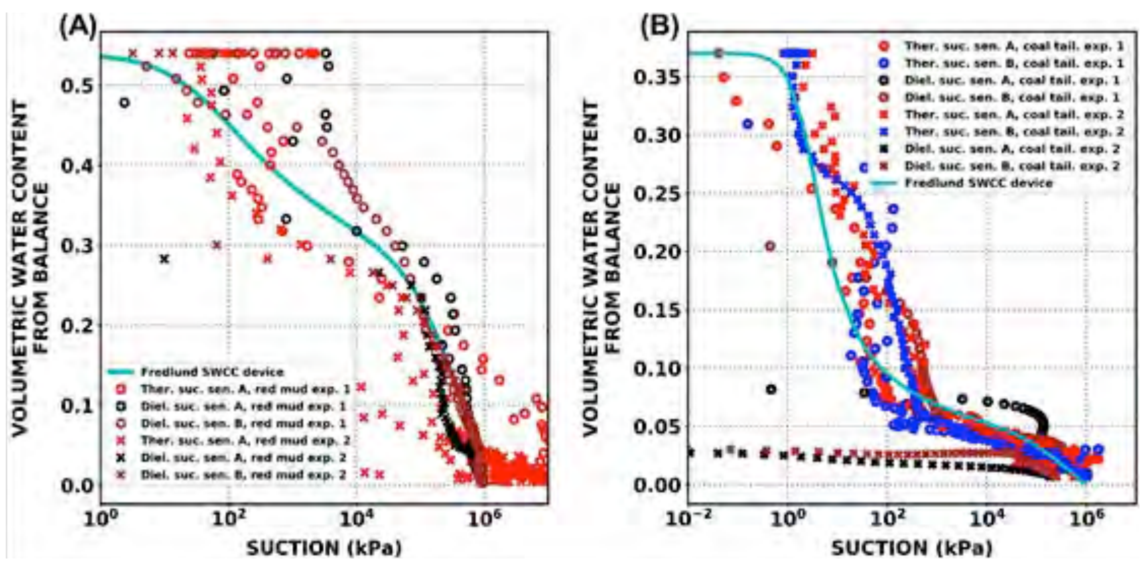

Figure 6. Comparison between Fredlund SWCCs and those obtained by a combination of volumetric water content from balance and measured suctions for: (A) red mud, and (B) coal tailings.

The future development of the tailings monitoring system will incorporate an in-house developed salinity sensor that is able to measure soil salinity over the entire range from fresh water to the solubility limit (265 ppt). The monitoring system has been integrated with a purpose-built, instrumented column 1,400 mm in height and 200 mm in diameter, which has been deployed under controlled conditions in the laboratory, and in the field with a weather station to monitor the climatic drivers of desiccation. The column is mounted on an in-house manufactured balance to monitor directly the water loss (or gain) over time. The instrumented column is a robust tool for optimising the tailings deposition thickness and cycle time to a tailings storage facility.

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State parameter as a geological principle in tailings

David Reid
Golder Associates Pty Ltd, Perth, WA, Australia

Mike Jefferies
Retired, formerly Golder Associates, Canada and United Kingdom

ABSTRACT: It has been previously observed that, for hydraulic fills with similar index properties and depositional environment, a similar range of in situ state parameter is often observed. The effect of depositional environment (i.e. depositional energy) on relatively clean sands was studied in detail during 1980s construction works in the Beaufort Sea, and the principle has been extended to the states of tailings. To provide further information and insight into this phenomenon, cone penetration test (CPT) data from a wide range of tailings deposits have been collected for comparison, with data sourced from both from the authors' company files for projects they have been involved in, and published data in the literature. The deposits assessed include relatively coarse, non-plastic, sub-aerially deposited facilities in arid environments (i.e. with drying), low plasticity silty materials pluviating through decant ponds, and some filtered stacks. The resulting trends suggest an increase in density with deposition energy, particularly with respect to materials pluviating quiescently in decant ponds compared to those deposited sub-aerially. The general trend of looser states with increasing plasticity of fines (and noteable exceptions) are discussed. The filter stacks examined surprisingly show slightly dilative in situ states, despite being deposited with minimal compaction – a field deposition method seemingly similar to the laboratory moist tamping method, used to create loose contractive specimens.

1 INTRODUCTION

1.1 *General*

Tailings strength, under both static and earthquake loads, depends on its state parameter. The state parameter can be measured by cone penetration test (CPT) soundings within an existing impoundment, but that does not help when designing/planning a new storage facility. The difficulty for design or planning is that, while standard laboratory tests provide properties that control how a tailings consolidates, there are no tests (or theory) that predict/measure the initial tailings condition (particularly fabric) as it changes from a slurry to a soil – the 'starting point' for standard geotechnical calculations is unknown. There is a further consideration that laboratory-scale sedimentation tests are known to not replicate the depositional environment, with misleading results in at least the case of some silty sands and sandy silts; the difficulty in predicting state, and hence behavior, in the design stage for hydraulic fills has previously been discussed by Reid and Fourie (2014).

It is obviously helpful for design/planning of any storage facility if existing experience for the in situ state of tailings could be synthesized in terms of the tailings type (gradation, mineralogy, specific gravity) and depositional environment (sub-aerial beach, sub-aqueous). This paper presents such a synthesis, premised around a geological principle that the depositional environment has a strong effect of the achieved in situ state parameter (e.g. high energy situations, such as wave action on a beach sand, produces a denser state than pluviation through a water column).

1.2 *Framework*

The paper is based on CPT data from a variety of sites, and includes both tailings and natural soils.

CPT soundings will show increasing penetration resistance with depth, all else equal, because confining stress increases with depth. Understanding soil behaviour from CPT data thus requires removal of the confining stress effect – generally referred to as 'normalization'. This paper uses the standard normalized (dimensionless) groups derived from the measured data of Q, F_r, B_q. Plotting CPT data as $Q(1-B_q)+1$ versus F provides a convenient way of summarizing CPT data in what is referred to as a 'soil behaviour type' (SBT) plot, this name arising because different soil types plot in different zones.

A further utility of the SBT plot is that, by assuming typical properties for each soil type, it can be contoured by lines of equal 'state parameter'. The state parameter ψ is defined as the void ratio difference of a soil to its critical void ratio at the same stress (Been and Jefferies 1985), and is a controlling factor for much soil behaviour. The contours are developed using the using the equations given by Plewes et al. (1992). Further detailed numerical work by Shuttle and Cunning (2008) provided more insight and defined a line – colloquially known as the "green line" - that distinguished between potential-flowslides versus loss-of-stiffness on the SBT diagram.

Thus, simply presenting CPT data in a SBT diagram can reveal a lot about tailings and this form of presentation is a good way to summarize the various case histories considered – and is the approach followed here. A quick caveat is needed, however, as tailings tend to be more angular particles than natural sands and this can lead to tailings appearing to be denser than they are; a SBT is a starting point to appreciate trends, but further quantitative evaluation will be needed for stability assessments.

1.3 *Data presentation*

CPT data in tailings shows natural variability. The data shown in this paper is generally presented in one of two forms:

- Individual data points across a relevant depth range, when a fairly focused depth interval (usually with accompanying laboratory testing) is examined
- An extent representing the range between the 10^{th} and 90^{th} percentile of the data in the X and Y axis.

It is noted that the percentile range indicated has been selected only to enable presentation of the general trends observed at a site, and is not a suggestion as to the relevant characteristic state. Detailed examinations of the effects of material variability have suggest that the 80^{th} percentile is a reasonable characteristic state (Jefferies and Been 2015), as applied in recently well-documented geotechnical practice (Morgenstern et al. 2016).

In additional to the discussions and graphical data in the text, index test data and critical state parameters for the tailings and soils considered are outlined in Appendix A.

1.4 *Site selection*

Sites presented are either tailings the authors have directed the testing and characterization of, or relevant sites from the literature that assist in highlighting the themes of the discussion. Most of the sites had triaxial testing carried out in parallel to the CPT investigation, to assist in refinement of in situ state determination based on modelling and/or estimate of in situ density from sampled water content adjacent to CPTs.

2 SANDS AND NON PLASTIC SILTY SANDS/SANDY SILTS

2.1 *Beaufort Sea hydraulically placed sands*

Perhaps the first detailed field-scale examination of the effect of different placement methods on resulting ψ were studies on Beaufort Sea hydraulically placed sands (Jefferies et al. 1988). Construction of artificial islands for oil exploration involved different sand placement techniques depending on application. The use of different placement techniques for essentially identical soils allowed assessment of the effects of depositional method on resulting ψ. Two primary forms of placement are outlined:

- Bottom dumping: a large quantity of sand is released from the bottom of a hopper-dredge to fall into the island being constructed and retained by previously constructed bunds. This process results in high depositional energies. The sand is, however, still pluviated through a water column with no air-drying or rolling across a beach.
- Pipeline: where the sand is discharged from a pipeline near the deposition area. For most Beaufort Sea construction the end of the pipeline was below water level so there was no sub-aerial beach. This results in lower depositional energy compared to bottom dumping. Pipeline discharge was controlled to allow bund construction that formed the island slopes and also used to infill the cores of the caisson retained islands.

An example of the different outcomes from the two placement methods is illustrated by data from the Tarsuit P-45 caisson retained island in Figure 2. Both placement methods used the same borrow source, and the as-placed sandfill contained generally less than 3% fines (a result of washing during borrowing). The bottom dumped berm material achieves significantly denser state – indeed, the berm fill achieves a sufficient density to obviate strain softening behavior, whereas portions of the core fill could be at risk of strain softening under some situations. It is emphasized that the Ersksak sand used in both core and berm was as similar across both phases of the works as is likely to occur in any field-scale geotechnical comparison. Clearly, the depositional process, particularly depositional energy, can have a major effect on resulting ψ – a concept related to the "infinity of normal consolidation lines", developed partially from observations of the Beaufort Sea works (Jefferies and Been 2000).

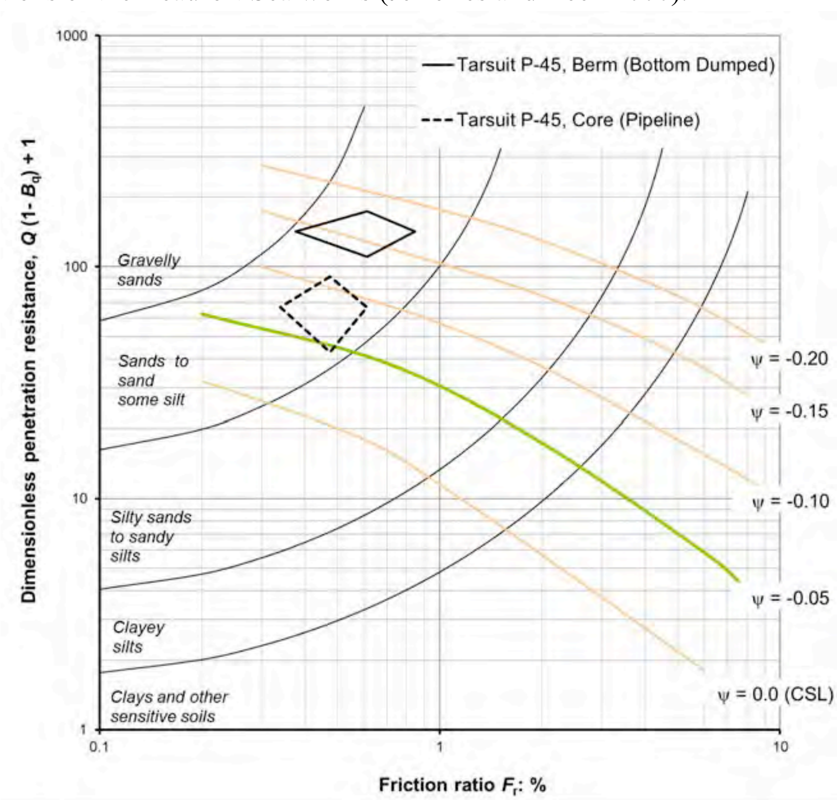

Figure 1. Effects of deposition method for Beaufort Sea hydraulically-placed sand

2.2 *CANLEX project*

CANLEX, A major collaborative research project in the 1990s involved the study of liquefaction characteristics of a number of sands and silty sands – including four TSFs where the material was pumped as a slurry onto tailings beaches. The materials were primarily quartz sands, with fines contents generally ranging from 5 to 15% - higher than Beaufort Sea works, yet coarser than the majority of tailings. While placement as a slurry on a tailings beach is similar in some ways to pipeline discharge, two differences include the process not being under water, and the generally higher fines contents of even the "sandy" tailings studied in the CANLEX project (with a related effect on Ψ). The results from the four TSFs are presented in Figure 3 alongside the Tarsuit P-45.

From the perspective of the historical development of methods to derive Ψ from CPT results (at a screening level), the CANLEX project indicated the Plewes et al. (1992) method performed better than the method of Been and Jefferies (1992), likely a result of better empirical correlations of CSL slope to CPT results (Reid 2015).

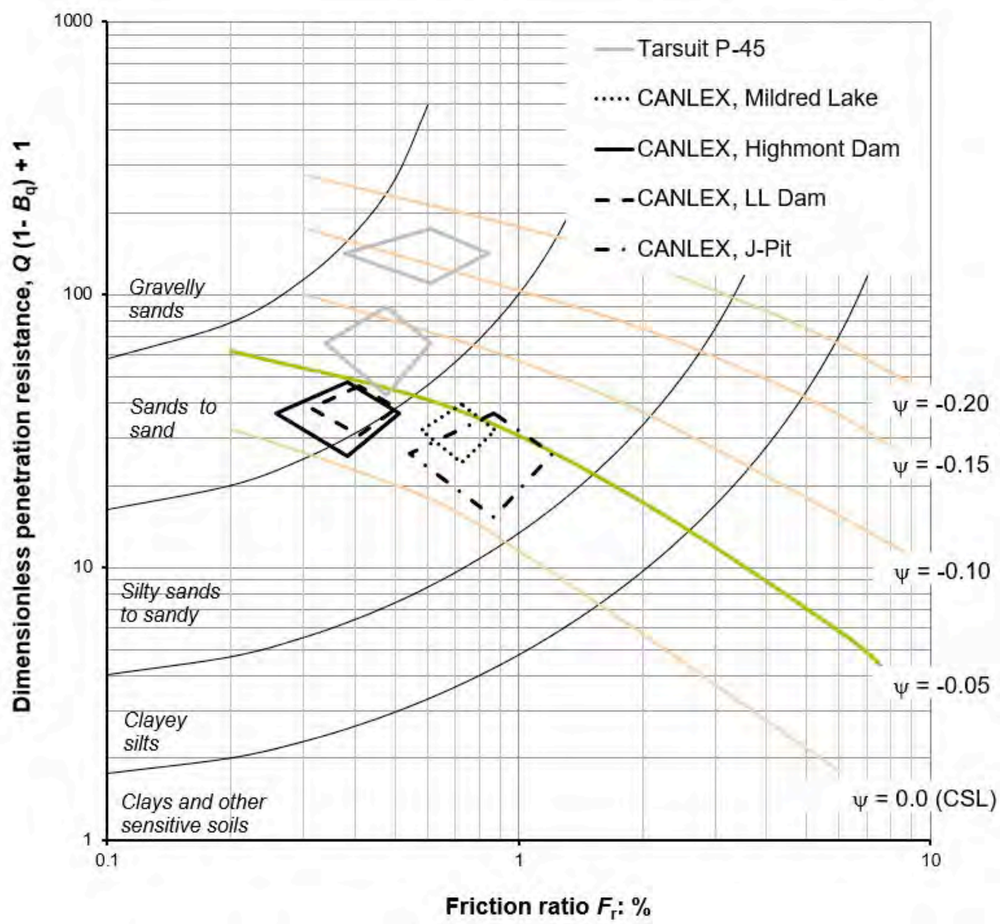

Figure 2. Comparison of CANLEX tailings sites to Beaufort Sea sands

2.3 *Fundao and TVA Kingston*

The Beaufort Sea and CANLEX sites comprised relatively low fines contents (<15%). To assess the state of tailings with higher quantity of non-plastic fines, (more "typical" of most tailings), two TSF failure case histories and another published example are added to an expanded Figure 4:

- Fundão TSF: CPT results from probes F01 and F02 through the sandy tailings upstream of the left abutment are presented (Morgenstern et al. 2016). Index testing of the sandy tailings indicated a range of fines contents ranging from 30 to 70%, with the blend used for CSL determination during the investigation having 40% fines. CPT data from elevations 860 to 880 m was selected.
- TVA Kingston Dredge Cells: An ash storage facility that failed in 2008, with significant root cause analyses carried out (AECOM 2009). The cause of failure identified by AECOM was static liquefaction of contractive ash material, triggered by creep-based deformation and/or failure of underlying slimes. The ash material was generally silt size – typically 70 to 80% fines content. CPT data were available in disturbed, failed ash (Cell 2), and in nearby unfailed areas (Cell 1). Owing to the likely disturbance in failed

areas, the data presented in Figure 4 are from CPT 303 in Cell 1, across depths 21 to 30 m.

- Rose Creek: A non-plastic sandy silt (60% fines), which had pluviated through a historic decant pond. Characterisation of the material state was outlined by Shuttle and Cunning (2007, 2008).

Deposition processes on the beach at Fundao and TVA Kingston were likely to be generally similar to that at the CANLEX sites – i.e. subaerial deposition as a slurry runs down a tailings beach. Although the failed portion of the TVA Kingston facility included a zone where finer-grained slimes pluviated through water, such depositional conditions do not appear relevant to the CPT 303 location based on the AECOM (2009) review of historical aerial photographs. Alternatively, the Rose Creek data presented is for a zone where the finer-grained fraction of the tailings pluviated through a decant pond during deposition.

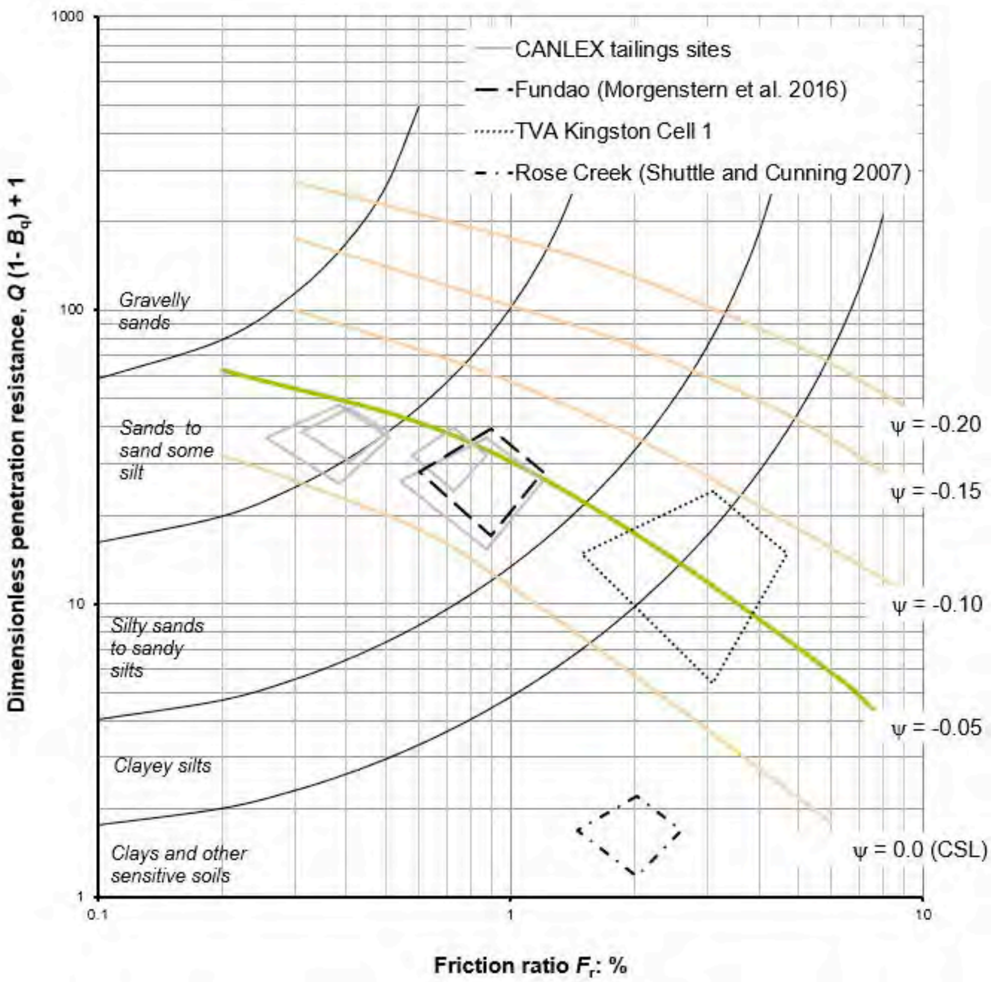

Figure 3. Comparison of CANLEX tailings sites to tailings with higher silt content: Fundao, TVA Kingston, and Rose Creek

As indicated in Figure 4, Rose Creek, which pluviated through the decant pond exhibited significantly looser states than the other examples. While pluviation through a decant pond has similarities to deposition hydraulic fill sands (previously discussed), a silty material when reaching a decant pond would be in a much lower energy environment than any conceivable hydraulic fill exercise. Whereas slow increase in height is a benefit for a TSF, for a hydraulic fill works the rates of rise, and related quantity of material deposited in an area per unit time (~energy) is much higher. Further, a TSF decant may be quite turbid, with finer silts/slimes taking some time to fall out of suspension. It is noted that there have been some tentative, yet fascinating studies on the effect of turbid water on resulting densities of pluviated material (Vaughn 1999).

3 PLASTICITY

3.1 *General discussion*

The previously outlined materials were non-plastic. However, a number of tailings contain appreciable plasticity. This is particularly the case for slimes deposited near or within the decant pond, which are finer-grained. Such materials are also more likely to have been deposited through quiescent pluviation. Although decant material are generally not relevant to TSF stability at the time of deposition, the continued expansion and raising of many facilities often results in the perimeter eventually being raised over and becoming relevant to perimeter stability (for example, Anderson and Eldridge 2010).

With respect to in situ state, it could reasonably be expected that as plasticity increases, the accessible range of Ψ would increase (i.e potentially becoming more contractive). Such a suggestion is implicit in proposed methods to classify soils as "clay like" or "sand like" (for example, Boulanger and Idriss 2007). The increased likelihood of a positive Ψ for plastic materials can be seen, indirectly, through the discussion on the mechanical properties of "clays" vs. "sands" in geotechnical textbooks. The strength of clays is generally described be a discrete undrained strength (s_u), whereas sands are analyses by means of effective frictional angle – although the limitations of extending this framework too far is clear (for example, Davies et al 2002).

3.2 *Data*

The following data are presented for tailings with a range of PIs, and where different depositional conditions occurred (i.e subaerial vs. pluviated). Each data set relates to a discrete depth range of CPT results, to correlate to the laboratory testing of an adjacent sample.

- The Rose Creek data previously presented is included for comparison
- Gold Tailings A – a sandy silt with PI from <5, deposited subaerially on a beach with some air drying following deposition.
- Gold Tailings B, a sandy silt with PI from 9 to 13, from two locations:
  - B1: deposited subaerially on a beach with some air drying following deposition. ($\sigma'_v \sim 260$ kPa)
  - B2: pluviated through a decant pond, negligible drying likely to have occurred. ($\sigma'_v \sim 480$ kPa)
- Molybdenum Slimes: A silts with trace sand, PI of pluviated through a decant pond, no drying. Previously outlined by Anderson and Eldridge (2010).

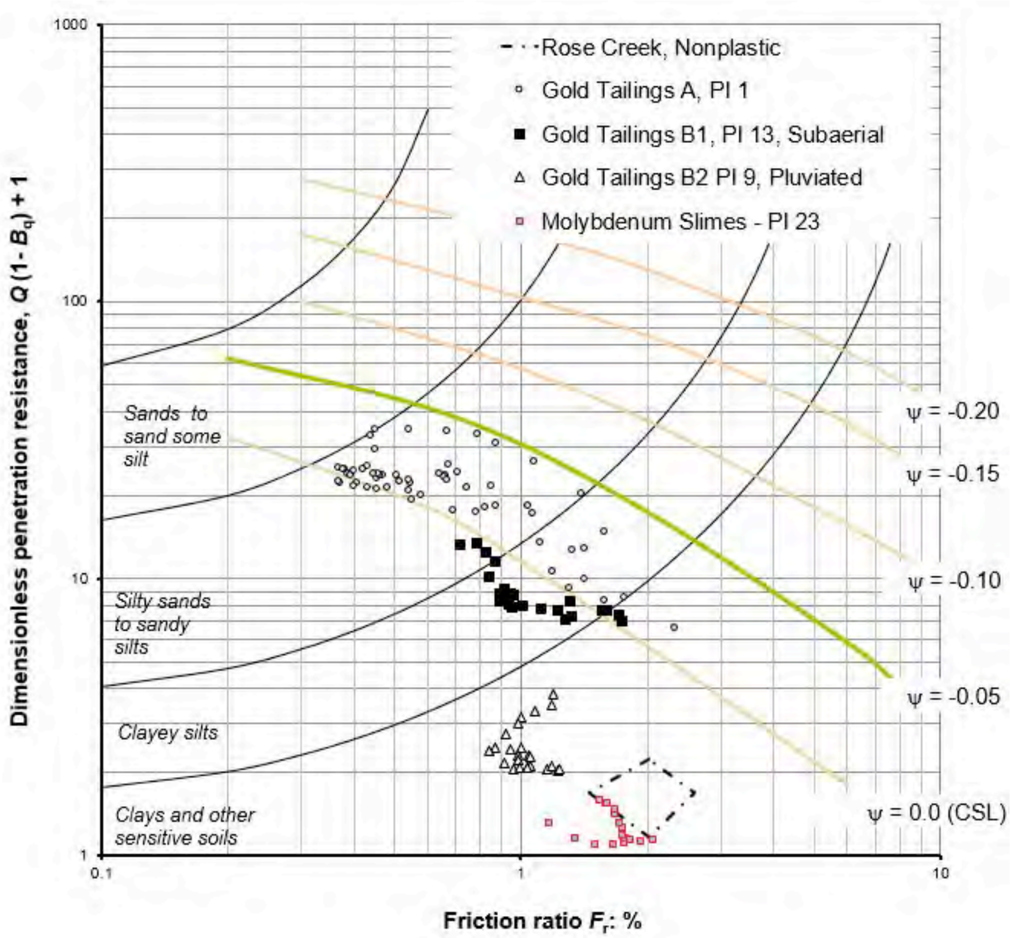

Figure 4. Tailings with appreciable plasticity, and different depositional conditions

Similar to the Rose Creek data previously presented, comparison of Gold Tailings B1 and B2 highlights the different NCLs resulting from different depositional conditions – with the behaviours not converging up to the vertical effective stresses present during CPT probing (~200 to 400 kPa)

The different states seen for identical materials suggests some difficulty in applying normalized methods for such material. For example, remoulding in the laboratory from a non-segregating slurry is unlikely to achieve densities as loose as those observed from pluviation through a decant pond. While methods to pluviate soils slowly through water columns in the laboratory have been developed (Bjerrum and Rosenqvist 1956, Høeg et al. 2000, Donahue et al. 2008), they are time consuming, and perhaps most importantly can create non-homogenous layered samples. This layering (and its effect on mechanical properties) may or may not reflect natural layering processes in the field – as suggested by different mechanical behavior when prepared at the same void ratio as undisturbed specimens demonstrated by Høeg et al (2000). The comparison of Gold Tailings B1 and B2 may also relate to the significant differences seen for clayey materials depending on the remoulded water content (Hong 2010) – which itself presents significant difficulties when designing a TSF in the design stage, as remoulded slurry consistency will affect subsequent behavior, and the slurry density at deposition varies for most tailings.

The results also point to the importance of site specific testing, to, at minimum, obtain CSL slope. For example, both the SBT and F_r of Rose Creek and Endako are quite similar, despite quite different compressibility.

3.3 *Filtered Tailings*

The increasing popularity of filtering dewatering technology, and investigations of such deposits, means a number of examples of CPT probe data through filter stacks are now available. Two examples are presented to compare the in situ states of the filter stacks to the previous data.

- <u>*Filtered A:*</u> An unsaturated stack comprising sandy silts and silty sands, as outlined by Robertson et al. (2017). The stack was of significant depth, with stresses of >1 MPa relevant to much of the available CPT data.
- <u>*Filtered B*</u>: A silty sand stack from our project files. Owing to some historic higher than desired moisture contents of the deposited material zone from approximately 6 to 20 m depth was in a saturated state – this zone was selected for analysis herein.

Both sites are noted to have undergone minimal compaction after placement of the material.

Examination of the state of filter stacks in an unsaturated state required caution, as suctions may affect both the CSL location (for materials with fines), and CPT tip resistance if suctions are sufficiently high (Yang and Russell 2015). These issues as they apply to tailings with a fluctuating phreatic level are discussed by Russell and Reid (2016). For the purposes of this paper, the stack outlined by Robertson et al (2017) appeared to be of sufficiently low suctions such that CPT results gave appropriate response (with respect to the saturated conditions of the material, were it to rewet), while for the stack from our project files we focused on the saturated region to avoid unsaturated effects.

The data from the two sites discussed above are compared to other non-plastic silt/sands in Figure 5. Interestingly, the two filtered states show generally denser states than subaerially deposited tailings of similar material type. This is curious, as placement of an unsaturated material with minimal compaction is probably the closest field analogue to moist tamping – the sample method that produces the loosest possible specimens in the laboratory. While the dilative condition of either material is surprising, the looser state of Filtered Stack A compared to B may be affected by the higher stresses relevant to that case history, combined with a non-parallel CSL and NCL.

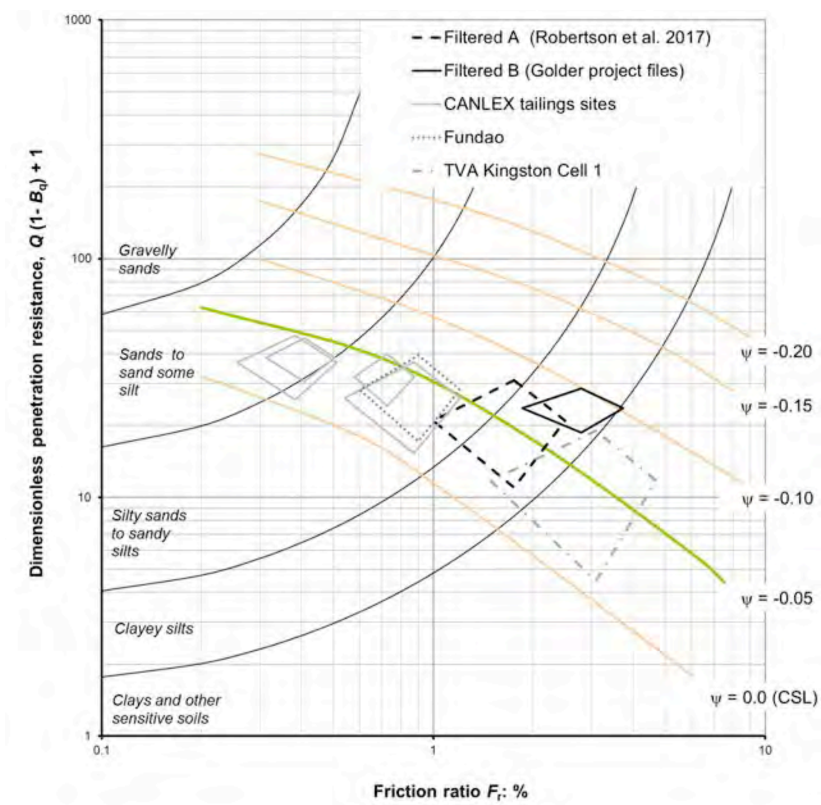

Figure 5. Comparison of silt/sand filtered tailings to subaerially deposited non plastic tailings

4 CONCLUSIONS

CPT data from TSFs with a range of different depositional conditions and material types was interpreted and compared to assess the effects of such factors on resulting Ψ in situ. The results agreed with typically-assumed generalisations of soil behaviour, in that material with higher fines content, and in particular with higher plasticity, generally exhibited looser states. Similarly, materials with higher silt content deposited in a low depositional energy, i.e. pluviated through decant ponds appeared to exhibit the loosest states regardless of plasticity. The limitations of reliance on index tests alone to assess likely state and/or in situ strengths was highlighted based on the results. Filtered tailings materials were seen to give denser states than may have been assumed, given the minimal compaction and moisture condition typical to that tailings deposition method.

APPENDIX A – SOIL AND TAILINGS REFERENCE DATA

Table A1. Index test data and critical state parameters for tailings and soils discussed

Site / TSF	D_{50}	% < 75 um	PI (%)	M_{tc}	Γ	λ_{10}
Tarsuit P-45 (Erksak Sand)	200-300	1-4	NP	1.27	0.875	0.043
CANLEX: J-Pit	170	~15	NP	1.32	0.92	0.035
CANLEX: Mildred Lake	160	~10	NP	1.62	0.92	0.035
CANLEX: Highmont Dam	250	~10	NP	1.70	0.98	0.068
CANLEX: LL Dam	200	~8	NP	1.60	0.98	0.068
TVA Kingston*	-	70-80	NP	1.20	0.95	0.046
Fundão**	88	40-45	NP	1.33	0.865	0.055
Rose Creek	55	60	NP	1.25	1.08	0.159
Molybdenum Slimes	5	99	23	1.37	2.06	0.541
Gold Tailings A	<38	75	1	1.33	0.927	0.120
Gold Tailings B1	<38	86	13	1.34	0.92	0.104
Gold Tailings B2	<38	80	9	1.30	0.95	0.104
Sth. America Filtered	65	-	NP	1.35	0.72	0.090
Golder Proj. Files Filtered	20-50	70-85	0-3	---- Not tested ----		

* Cell 2 material tested by AECOM (2009), CPT303 located in Cell 1 (unfailed portion)

** Gradation of the material tested by review panel

NP: Non plastic

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Waste Characterization

Relationship between In-Line Polymer Dose and Tailings Index Properties

David Anstey & Raymond Guang
Golder Associates Ltd., Calgary, AB, Canada

ABSTRACT: Since the mid-2000s, in-line polymer treatment of tailings has become an increasingly common technology for dewatering tailings. The polymer is typically applied to the tailings stream immediately upstream of the discharge point resulting in accelerated dewatering, increased beach slope, reduced segregation, and improved water clarity within the tailings storage facility. The technology has been implemented for a large range of mined commodities and is now considered as part of the routine screening process for tailings management options for new mines. However, the dose requirements for in-line polymer vary widely across the mining industry and tailings types. Polymer dose has been compared to tailings index properties, such as particle size and Atterberg Limits, across a range of mining operations, applications and tailings types. The relationships developed are intended to provide the designer with a check for the expected range of polymer doses applicable to a particular tailings and can be used to support preliminary tailings options assessment for new projects.

1 INTRODUCTION

Since the mid-2000s, in-line polymer treatment of tailings has become an increasingly common technology for dewatering and managing tailings. As of 2014, the technology had been implemented at more than 30 operating sites world-wide (Riley & Utting 2014). It is now considered as part of the routine screening process when considering tailings management options for new mining projects.

In-line polymer addition typically comprises the injection of a polymer solution directly into the tailings stream at or close to the point of discharge. The location of injection may vary from several hundred meters upstream of the discharge point to the plunge-pool immediately downstream, depending on the tailings properties and amount of mixing required. In some applications, a dynamic mixer is used to optimize mixing of the polymer with the tailings.

In-line polymer addition may be used to control tailings beach slope (by influencing tailings rheology), reduce segregation, increase the rate of sedimentation and consolidation, and/or improve supernatant water clarity (da Silva 2011). The type of polymer selected will depend on the application and the properties of the tailings to be treated. Most frequently, polymers used for in-line applications take the form of high-molecular-weight polyacrylamides formed from acrylamide monomer. Different functional groups are attached to the backbone of the polymer to adjust its properties for specific applications or minerals.

The polymer dose required to create the desired effect may also vary significantly between different tailings types and applications. In some applications, it may form a significant portion of the operating costs for tailings management. Polymer dose is, therefore, an important factor in deciding whether this technology is suitable for a particular project.

Estimating the optimum in-line polymer dose for a particular tailings is not particularly difficult. With as little as 40 L of tailings slurry, polymer screening tests can be carried out at bench-scale to assess various polymers and their resulting rheology, net water release or release-water clarity. These screening tests can provide an estimate of the preferred polymer and optimum polymer dose in a matter of hours. However, engineers are sometimes called upon to provide a judgement on the optimum dose that may be required for a particular tailings, often in the absence of a tailings sample or specific laboratory testing. An example is as an input to a scoping study for a project, where high-level capital and operating costs are required and in-line polymer addition may be one of several options being considered. In such circumstances, it may be useful to have a broad understanding of the approximate range of polymer doses applicable to a particular mined commodity or range of tailings properties.

2 FACTORS INFLUENCING FLOCCULATION

The effect of polymers on the flocculation of tailings has been well studied and understood since the 1950s (Henderson & Wheatley 1987). Flocculation occurs when individual particles in suspension form aggregates by the action of polymeric chains attaching and bridging the particles, overcoming the repulsive forces between particles. The effectiveness of a polymer in flocculating a tailings sample has been demonstrated to be a function of:

- The polymer chemistry
- The presence of coagulants that destabilize colloidal dispersion and their order of addition (Sabah & Cengiz 2004)
- Mineral surface charge and chemistry, including the activity of clay minerals (Stocks, 2006)
- Particle size, shape and surface texture (Stocks, 2006)
- Shear conditions and mixing in pipelines and on the beach (Revington et al. 2012, Henderson & Wheatley, 1987, Stocks 2006)
- Slurry density (or solids content), which affects the ability for the polymer to disperse and mix through the tailings sample
- Polymer dilution, which will also affects its ability to disperse and mix with the tailings (Pillai 1997)
- pH (Atesok et al. 1988, Pillai 1997)
- Water chemistry (Stocks 2006).

3 OPTIMUM POLYMER DOSE

In the typical flocculation process an optimum polymer dose will exist. When the polymer concentration is lower than optimum, there are insufficient polymer chains in solution to attach to and bridge all of the particles, leaving some particles in suspension. When polymer concentration exceeds the optimum, the surface of the particles become saturated by the polymer chains and steric stabilization occurs, where the ability of the particles to aggregate is impeded.

In-line polymer addition typically occurs in a highly dynamic environment with a tailings slurry that is often at a much higher solids content than is typical of conventional tailings thickening. Rather than aggregating and settling particles in the water column of a thickener, the aim is to rapidly aggregate particles, increasing their yield stress, arresting their flow and dropping them out of the dynamic suspension. These conditions result in optimum polymer doses that are typically higher than those required for thickener applications (Riley & Utting 2014).

The authors have collected data on the optimum polymer dose reported for in-line polymer addition across a range of tailings types, polymers and applications. These optimum doses have been estimated from various bench-scale and flume-scale tests, field trials and commercial operations.

At a bench scale, the optimum dose is estimated by mixing various polymer doses with subsamples of the tailings and observing the resulting water clarity, rate of dewatering and/or yield stress of the samples. From this process a single optimum dose is estimated for the sample and polymer tested.

Flume tests are sometimes run using a single dose derived from bench-scale tests. More sophisticated flume tests may include a variation in properties that result in adjustments to the dose as the test progresses. Field trials and operations will usually involve a feedback loop where visual signs of flocculation are observed at the discharge and the dose is adjusted accordingly. This leads to a range of optimum doses corresponding to a range of tailings solids contents and index properties. For the purposes of current analysis, where a range is reported, the authors have selected the average dose and corresponding average tailings properties for analysis.

4 ANALYSIS

4.1 *Data*

Data have been compiled from testing of 89 tailings samples, comprising 42 oil sands samples, 35 iron ore (magnetite, hematite and blends), 4 mineral sands samples and one or two samples from each of gold, gold-copper, copper, alumina, laterite nickel and coal. The information includes 52 bench-scale tests, 30 flume tests, three field trials and four commercial operations. The data has been obtained from a combination of publically available regulatory applications, private test data and published literature (Cooling & Beveridge 2016, da Silva, 2011, Guang et al. 2014, Mizani et al. 2013, Riley et al. 2015)

Not all of the fields analysed were available for all of the samples. Data was most often provided for particle size distribution, specific gravity of solids and solids content. Limited data was available for Atterbeg Limits, methylene blue index, mineralogy and water chemistry. The following sections present the correlation between optimum polymer dose and commonly reported tailings properties. Polymer dose is reported as dry grams of polymer per tonne of dry tailings solids (g/tonne).

4.2 *Solids Content*

The optimum polymer dose is compared to the solids content of the tailings slurry prior to polymer treatment across a range of mined commodities in Figure 1.

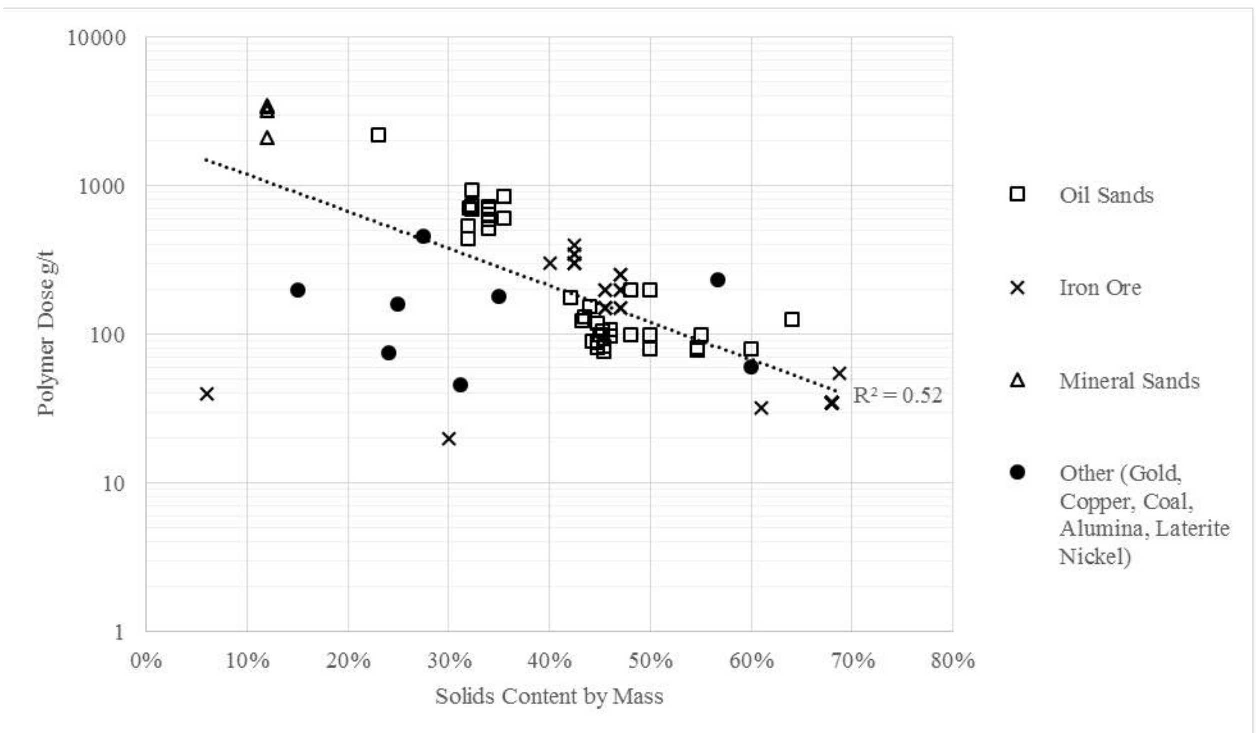

Figure 1. Optimum Polymer Dose versus Solids Content and Commodity

We might expect that a higher tailings solids content would make it more difficult for the polymer to disperse and mix with the tailings, increasing the optimum dose required for treatment.

Figure 1 indicates that this is not the case. Rather, Figure 1 indicates that solids content and polymer dose have an inverse relationship, with a coefficient of determination (R^2) of 0.52 when plotted on a log-linear scale.

In most cases, tailings are thickened at the processing plant in a conventional, high-rate or paste thickener prior to deposition. The solids content that can be achieved in the thickener depends on the mineralogy, particle size, and clay content of the tailings as well as other factors that are common to the efficacy of in-line polymer addition. The trend observed in Figure 1 supports the conclusion that tailings that are 'difficult' to thicken upstream require higher in-line polymer doses. Conversely, tailings that dewater easily in a thickener will require lower doses of polymer in-line. It is likely that there is a correlation between the flocculant dose used in thickeners and in-line polymer dose, although this relationship has not been explored as part of the current analysis.

Some of the scatter observed below the linear regression in Figure 1 is representative of tailings that undergo little or no thickening as part of their processing prior to deposition.

We might expect that commodity type would be a proxy for mineralogical influences on flocculation. This is because geological formations and process technologies are often common to mines for a particular commodity. Figure 1 indicates that optimum polymer dose can vary significantly between tailings samples for a single commodity. This suggests that factors other than commodity type play a significant role in determining optimum polymer dose.

4.3 *Particle Size Distribution*

Optimum polymer dose was compared to the silt and clay size fractions of the tailings samples, represented by the proportion passing 75 µm, 44 µm and 2 µm. For the 75 µm, and 44 µm comparisons, four outlying points were excluded from the correlation. These four points were from a composite tailings created in the laboratory from a combination of 'slimes' and 'sands'. The resulting 'synthesized' tailings was strongly gap graded and so skewed the results for the particle sizes in question. Polymer dose is plotted against proportion passing 75 µm, 44 µm and 2 µm in Figures 2, 3 and 4.

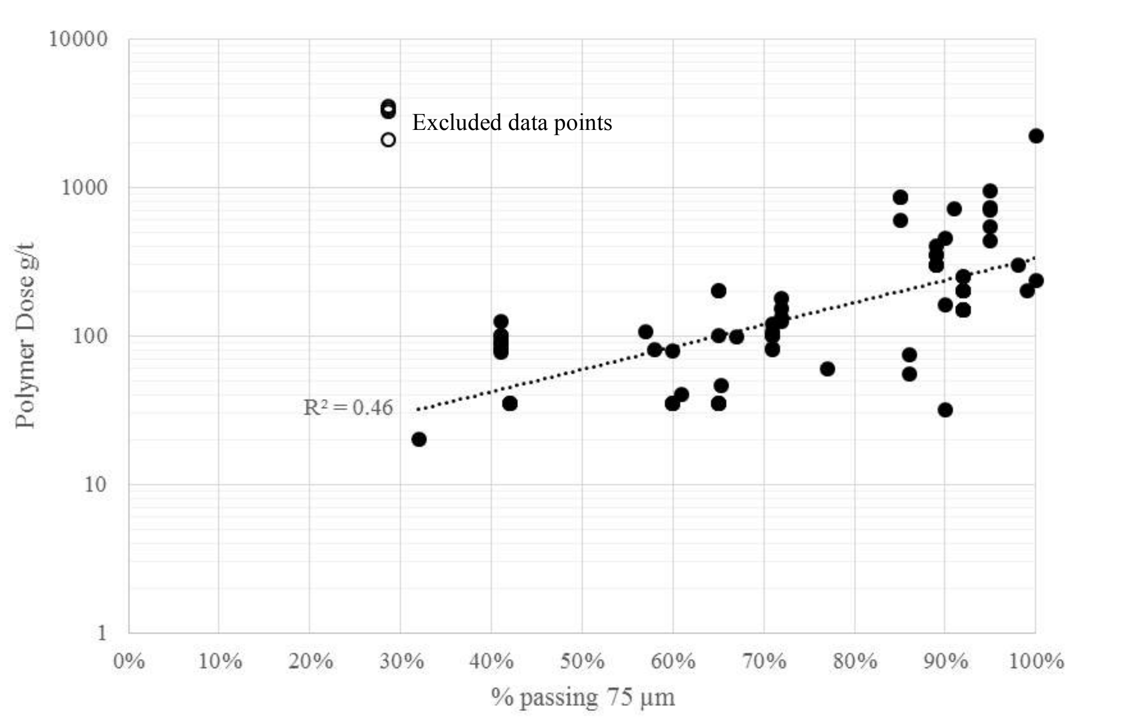

Figure 2. Optimum Polymer Dose versus Percentage Passing 75 µm

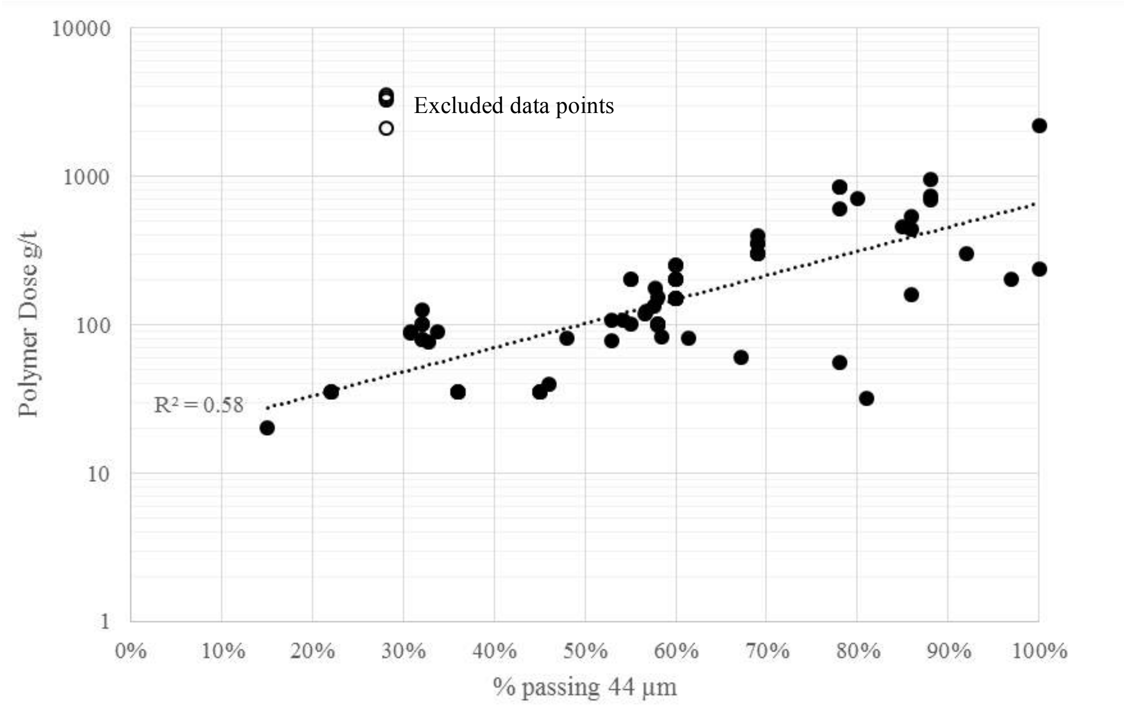

Figure 3. Optimum Polymer Dose versus Percentage Passing 44 µm

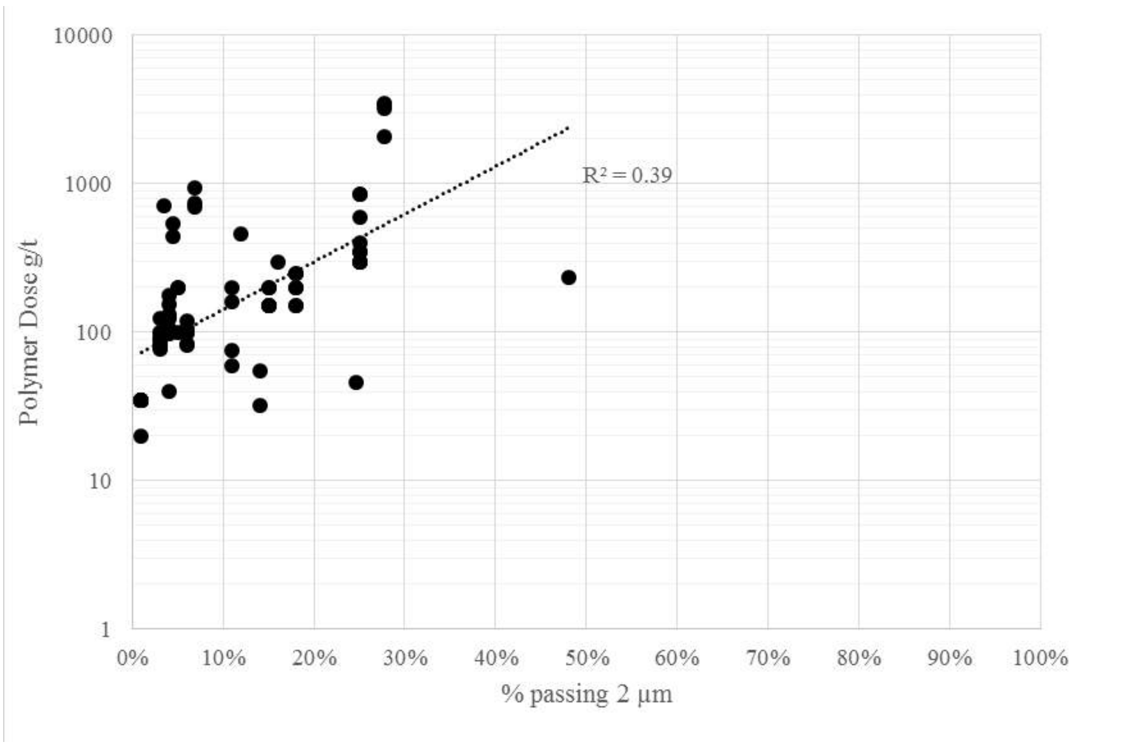

Figure 4. Optimum Polymer Dose versus Percentage Passing 2 µm

A moderate relationship is evident between optimum polymer dose and the proportions of particles passing 75 µm, 44 µm and 2 µm. The relationship is strongest for the proportion passing 44 µm. This reflects the importance of finer particles in influencing polymer dose.

4.4 *Atterberg Limits*

Optimum dose is plotted against Liquid Limit and Plasticity Index in Figures 5 and 6.

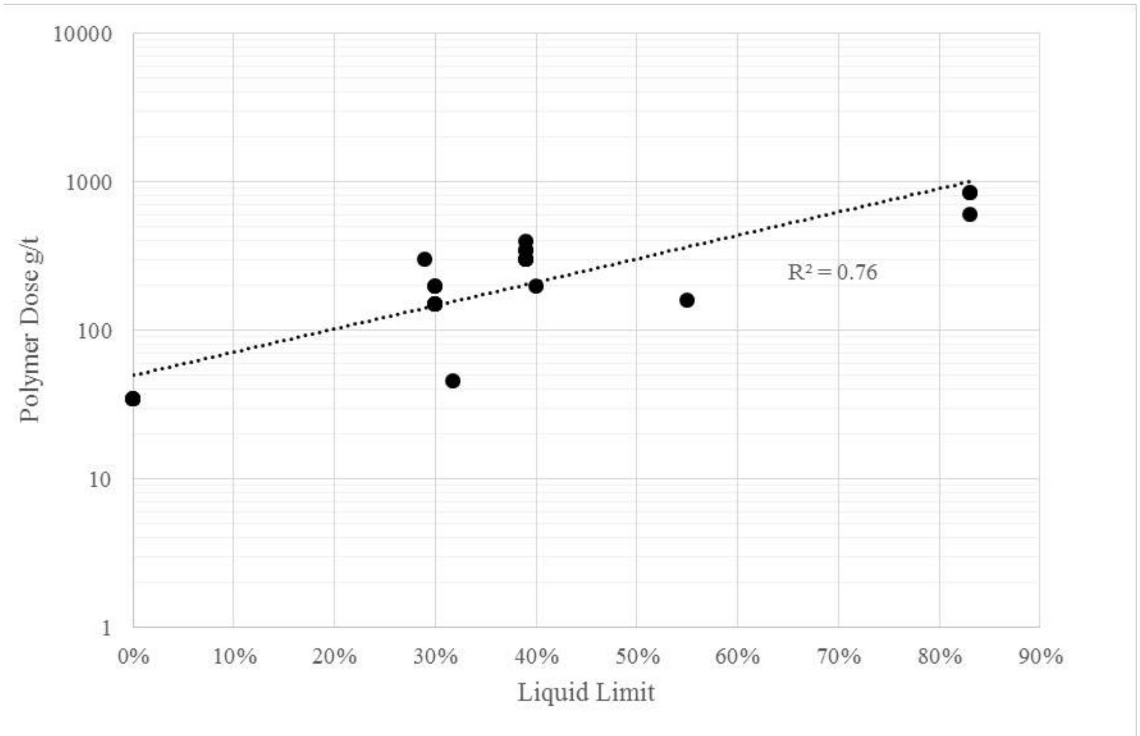

Figure 5. Optimum Polymer Dose versus Liquid Limit

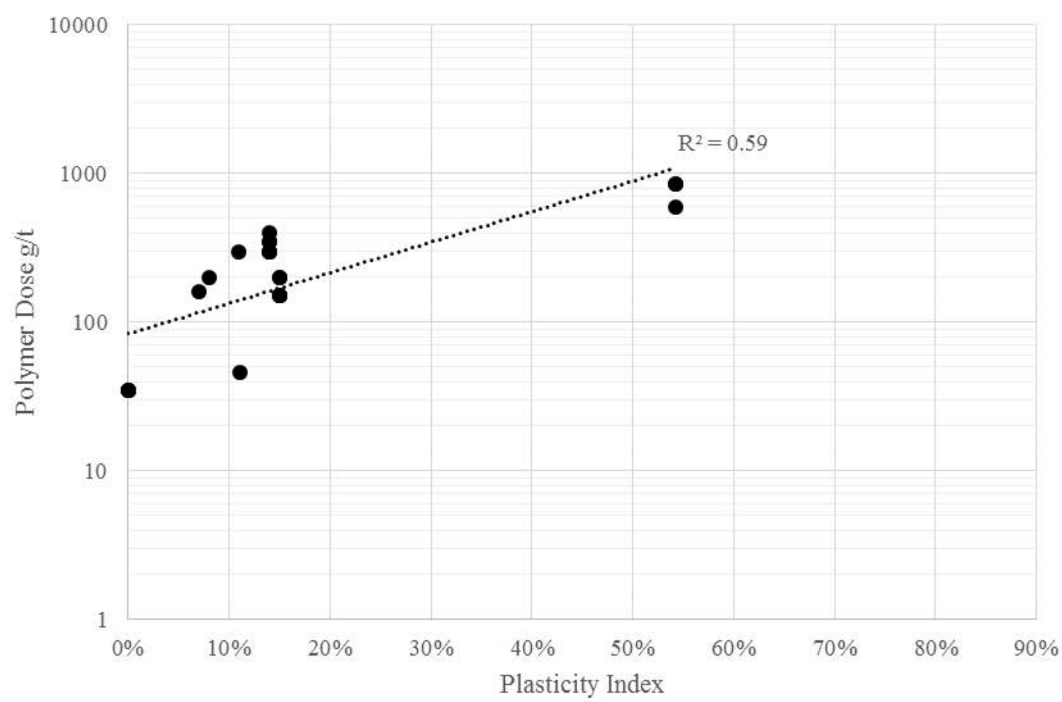

Figure 6. Optimum Polymer Dose versus Plasticity Index

Acknowledging that there are limited data points on which to base a relationship, there is nonetheless a moderately strong relationship between optimum polymer dose and both liquid limit and plasticity index. This is perhaps not surprising since plasticity index is proportional to the cation exchange capacity and activity of the clay minerals (Kaminsky 2008). The quantity of clay and its cation exchange capacity is a large determinate of the amount of polymer required to flocculate the tailings.

4.5 *Methylene Blue Index*

Methylene Blue Index (MBI) is commonly used as an indicator for the activity of clays in oil sands tailings. The test is described by the American Society for Testing Material (ASTM) C387-09, Standard Test for Methylene Blue Index of Clay.

The relationship between MBI and the cation exchange capacity of clay minerals has led to the suggestion that this test provides a better and more direct measurement of the potential reactivity of tailings containing clays than tests such as Atterberg Limits (Boxill, 2011). Regrettably, MBI results were only available for a portion of the oil sands tailings samples analyzed and none of the tailings samples generated for other commodities. The total number of samples with MBI values are therefore limited. The correlation between optimum polymer dose and MBI is presented on a log-linear scale in Figure 7.

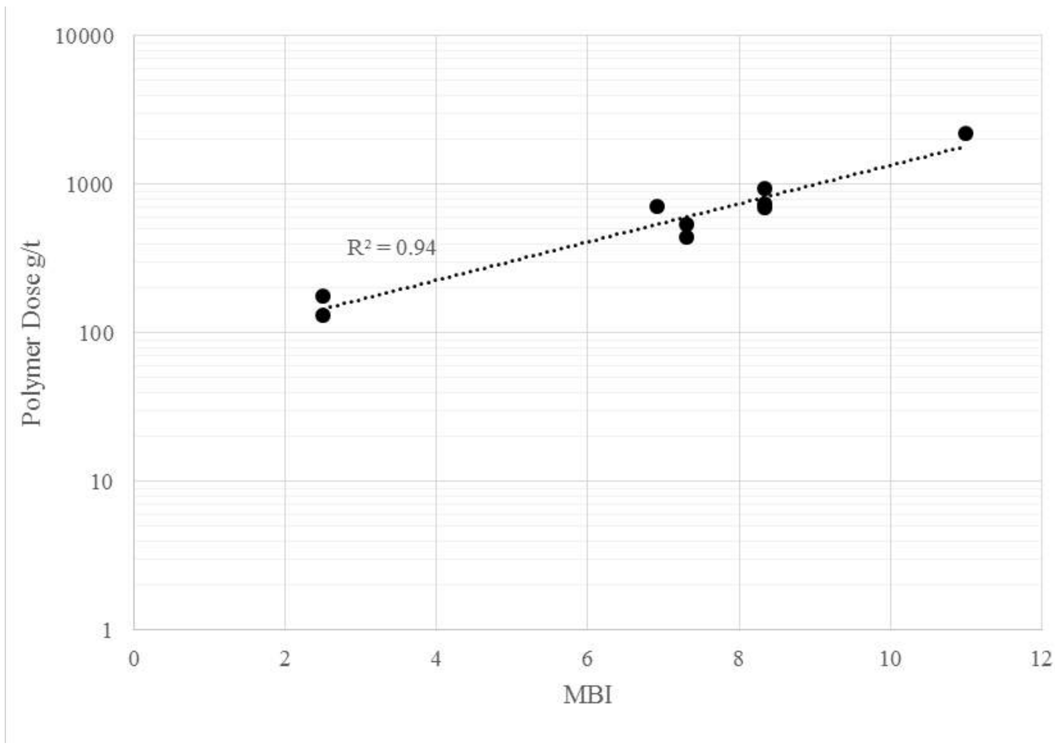

Figure 7. Optimum Polymer Dose versus Methylene Blue Index

Figure 7 indicates a very strong relationship between MBI and polymer dose for the oil sands tailings samples tested. This supports the conclusion that clay activity is an important factor in polymer consumption. However, the correlation between polymer dose and MBI may be overstated by Figure 7, because of the absence of data for commodities other than oil sands. As shown in Figure 1, oil sands samples also have a strong relationship with solids content when other commodities are excluded.

4.6 *Poor Predictors of Optimum Polymer Dose*

No correlation was identified between optimum polymer dose and specific gravity, particle diameter passing 80%, 50% or 10% (P80, D50 or D10), fines over fines plus water (FOFW), or plastic limit. There was insufficient data available to draw meaningful conclusions regarding the relationship between polymer dose and tailings pH, water chemistry or zeta potential.

The relationship between polymer dose and application was also assessed. The expectation was that steepening the tailings beach slope may require an increased polymer dose relative to improved dewatering or water clarity applications. However, no clear relationship was evident between polymer dose and application for the data analyzed. It appears other factors are more important in determining optimum polymer dose. The data set did not contain any examples where polmer dose was assessed for a single tailings sample across multiple applications.

5 LIMITATIONS

There is significant scatter observed when comparing optimum polymer dose to typically available tailings index properties. Some of the potential reasons are discussed below.

Solids content, and silt and clay fractions are correlated with the quantity and activity of clays in a sample but are not a direct measure of clay activity. While this accounts for the observed variation in the plots, large variations in polymer dose are also observed for tailings types with little or no clay mineralogy. In this case, the proportions of other minerals, such as quartz, mica, and precipitates (gypsum, metal hydroxides) may play a role.

There is limited data available for the index properties of many of the tailings samples reported so the data presented may fail to capture the true variation in index properties. This is particularly true of operational facilities, where a tailings sample from the discharge point may be tested for index properties annually or even less frequently. But it can also be the case for bench-scale testing where one sample is characterized with the expectation that it is reasonably characteristic of the other samples used to determine optimum dose. In the laboratory, the focus is most often on the selection of the optimum polymer dose (based on visual inspection and indicator tests) rather than how this corresponds to the tailings index properties.

In operation, the reported polymer dose may not be the optimum dose for the current tailings properties. The dose may be set based on earlier test work or correlations with slurry density, developed as part of earlier trials. Tailings properties will change over time and work may not have been carried out to characterize and update polymer dose based on these changes.

Polymers also differ from one another. Suppliers spend a great deal of time and money researching and developing new and differentiated products. As a result, the quantity of polymer required to have the same effect will vary even for the same tailings sample. This scatter is demonstrated in Figure 8, which depicts the range of optimum doses determined for different polymers tested on identical tailings samples.

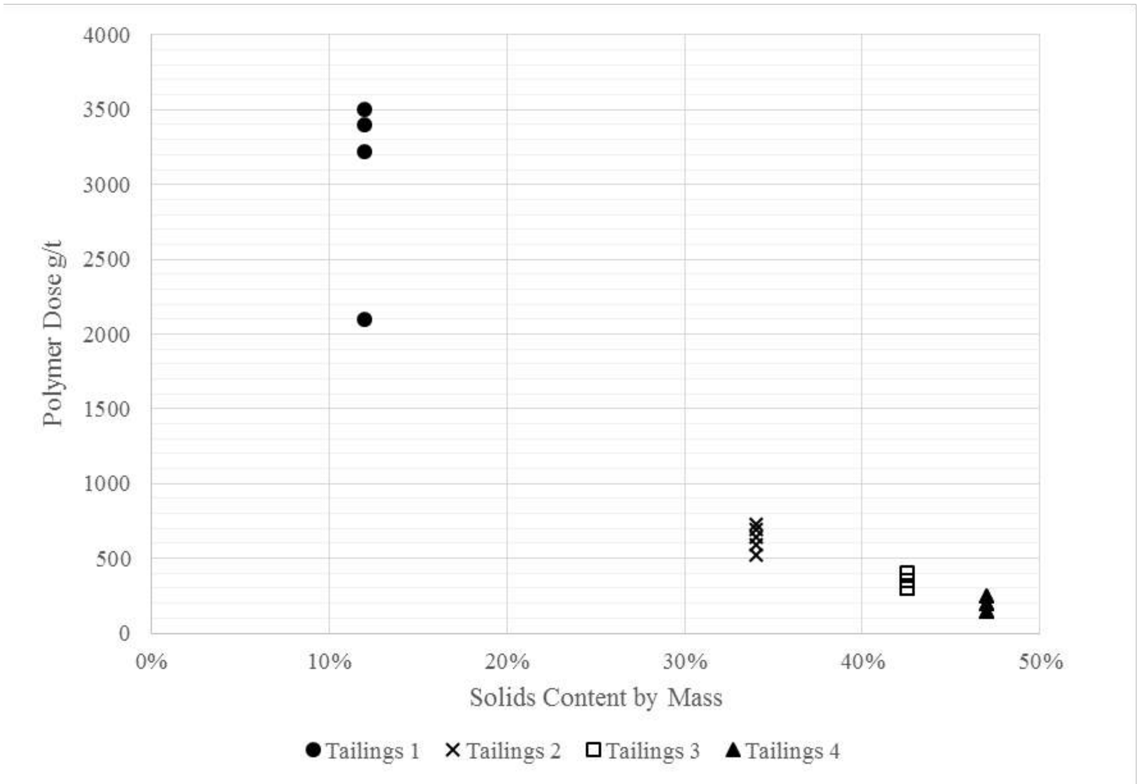

Figure 8. Variation in Optimum Polymer Dose for different Polymers

6 CONCLUSIONS

Analysis of in-line polymer treatment of 89 tailings samples demonstrates that optimum polymer dose can vary significantly between tailings samples for a single commodity.

Several common tailings index properties show some correlation with optimum polymer dose. This is particularly true of index properties that are representative of the fine particle fraction and clay activity of the tailings. Of the properties analyzed, optimum polymer dose has the strongest relationship with MBI, liquid limit and the fraction passing 44 µm. Notwithstanding, it is fair to conclude that that tailings index properties are limited in their ability to accurately predict optimum polymer dose for in-line polymer applications. Even when identical tailings samples are tested against multiple polymers, the optimum dose can vary significantly. This highlights the importance of bench-scale tests in selecting a polymer type and estimating the optimum dose for design.

The data set used for this study is heavily weighted towards laboratory-scale data and two industries (oil sands and iron ore mining). Operational data on optimum dose was limited to only four tailings samples. The relationships presented would benefit from the collection of more data points and from the analysis of a broader suite of parameters.

One avenue for further investigation is to separate the tailings samples into dominate mineral types before analyzing against particle size distribution. In many instances, the dominant mineral type and grind size for a particular deposit may be known long before any tailings samples are generated for a project.

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SWCC: Experimental results of copper tailings and spent ore

José Riquelme & Cristian Godoy
SRK Consulting, Santiago, Chile

ABSTRACT: Modelling flow into mining geotechnical structures, such as heap leach, spent ore dumps or a tailings storage facility, should consider the fact that constitutive materials may be found in an unsaturated regime. A fundamental parameter for doing this type of analysis is the soil-water characteristic curve (SWCC), however, its empirical determination is not frequent due to availability and time restrictions. The current state of practice consists in the determination of these curves through indirect estimations which are not necessarily meant to be applied on particular materials as those from mining process.

This article presents the results of an experimental study on SWCC determination for spent ore and tailing materials, comparing their results with different estimation methods. Given the inability of the estimations of reproducing the actual behavior of those materials, an alternative focus is proposed to experimentally determine the SWCC using only three measurements, saving time and testing costs.

1 INTRODUCTION

Geotechnical structures associated with mining works are generally in a state of partial saturation, so any study of infiltration or flow into them must consider aspects of the mechanics of unsaturated soils. The soil-water characteristic curve (SWCC), which defines the ability of the soil to retain or release water, plays a fundamental role in the performance of these analyzes, so that its determination is vital. Determining empirically the SWCC can take from few days to several months, depending on the material; Due this, and the scarce availability of equipment in laboratories of soil mechanics, estimates are usually used from more accessible parameters.

In the present study, the results of an experimental campaign for the determination of moisture retention curves through pressure plate tests of materials from different mines, in particular, tailings and spent ore - for which there is not enough information available in the literature. In addition, the predictive capacity of three different estimation methods was investigated and, finally, a methodology is proposed to determine the suction curve that allows a reduction of the times and resources associated with pressure plate tests, proposing appropriate execution parameters for each type of material studied.

2 SOIL WATER CHARACERISTIC CURVE (SWCC)

The moisture retention curve (SWCC) is a graphical representation of the mathematical relationship between suction within a soil with water content or degree of saturation [1]. Originally developed for soil studies in agriculture. The SWCC represents the water storage capacity of a material versus different states of matrix suction. This curve generally presents two strong slope changes, the first change being the air entry value (AEV), which represents the suction at which the soil passes from a saturated to a partially saturated state and is associated with negative pore pressure. The second slope change represents a state of equilibrium, which is associated with a residual moisture content. Fig. 1 shows the shape of a type curve. In the agricultural field it is common to carry out tests to determine the moisture content at two typical suction values of 30 kPa and 1500 kPa, which represent the field capacity and the permanent wilting point respectively. Fig. 1 shows the typical shape of sandy, silty and clayey soils.

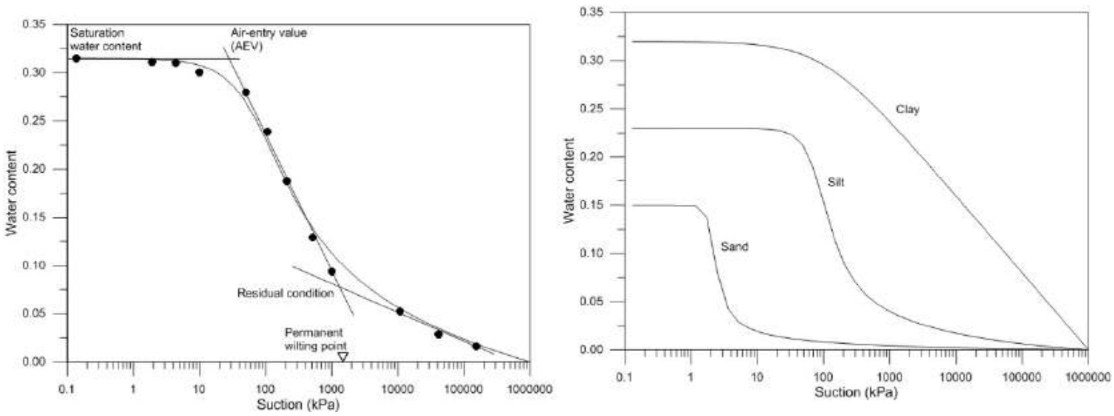

Figure 1. SWCC Example (Left). Typical Shape of sandy, silty and clayey soils (Right).

3 EMPIRICAL DETERMINATION SWCC CURVES

Empirical curves were determined from five samples from tailings storage facilities and nine from spent ore dumps, which originated from mining operations located in the north of Chile. The basic properties and particle size distributions of these materials are presented in Table 1 and, in Figs. 2 and 3, respectively.

Table 1. Properties of studied material.

Type	Name	γ_d (t/m^3)	Gs	e	n	d_{max} (mm)	% Gravel	% Sand	% Silt	% Clay
		(1)	(2)	(3)	(4)	(5)				
Tailing	Tailing 1	1.50	2.75	0.83	0.45	0.4	0	34	45	21
	Tailing 2	1.35	2.79	1.07	0.52	0.4	0	19	63	18
	Tailing 3	1.82	2.93	0.61	0.38	0.4	0	33	59	8
	Tailing 4	1.6	2.83	0.77	0.43	0.4	0	35	56	9
	Tailing 5	1.51	2.65	0.75	0.43	2	0	36	52	12
Spent ore	Spent ore 1	1.96	2.79	0.42	0.30	76.2	62	26	8	4
	Spent ore 2	1.84	2.79	0.52	0.34	25.4	55	30	12	3
	Spent ore 3	1.75	2.79	0.59	0.37	4.8	0	67	26	7
	Spent ore 4	1.60	2.79	0.74	0.43	4.8	0	67	26	7
	Spent ore 5	1.59	2.72	0.71	0.41	38.1	44	43	10	3
	Spent ore 6	1.50	2.66	0.77	0.44	50.8	56	28	11	5
	Spent ore 7	1.49	2.62	0.76	0.43	50.8	56	27	12	5
	Spent ore 8	1.46	2.66	0.82	0.45	38.1	56	26	14	4
	Spent ore 9	1.47	2.62	0.79	0.44	38.1	56	30	11	3

Note: (1) dry unit weight
(2) Specific gravity of soil solids
(3) Void ratio
(4) Porosity
(5) Maximum particle size

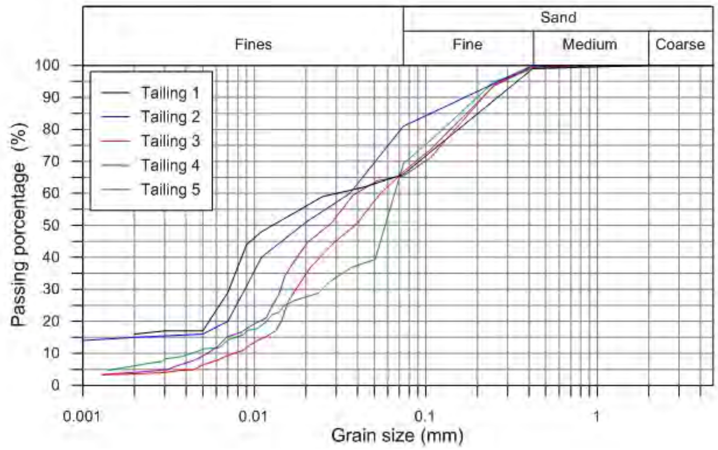

Figure 2. Particle size distribution of tailings.

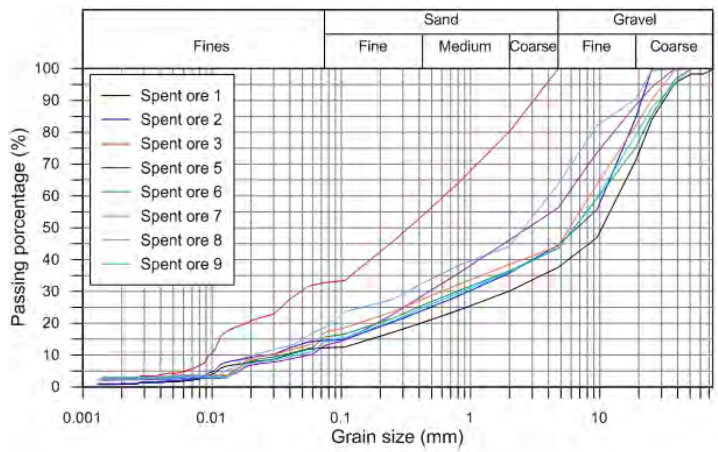

Figure 3. Particle size distribution of spent ore

Experimental determination of swcc was carried out using the pressure plate method [2], commonly used in Chile for agricultural studies (not so in soil mechanics laboratories). In this test saturated samples are placed inside a pressure chamber, then suction is applied until the sample removes retained water, reaching a state of equilibrium. Finally the gravimetric moisture content is determined by wet and dry mass difference.

In this study to construct the SWCC curve in the whole spectrum of possible suctions tests were executed at different suctions depending on the material. Fig. 4 shows samples and the equipment used in the execution of the test.

Figure 4. Pressure plate equipment used to obtain SWCC

Experimental results of the soils tested in the framework of this study can be observed in Fig. 5, below. It is possible to note the difference between results obtained for tailings with respect to spent ore, following the first the typical form of S, while latter have a relatively flat shape.

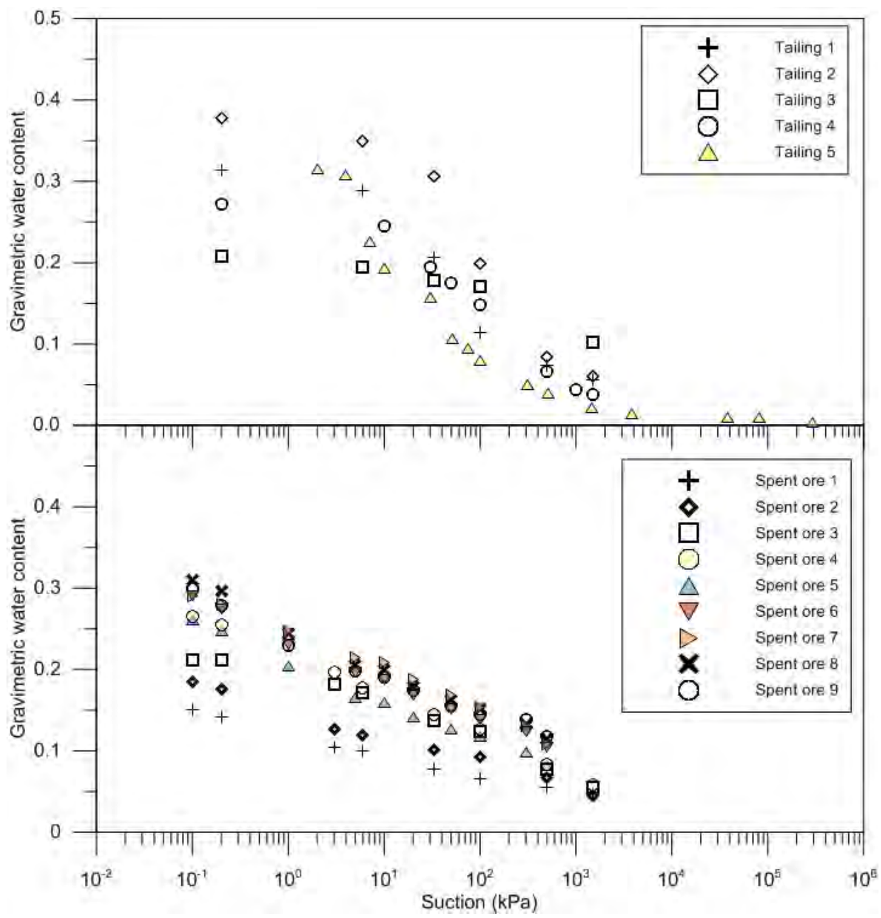

Figure 5. Experimental SWCC Tailing (upper) and Spent ore (lower).

In order to obtain a continuous representation of the SWCC, the experimental results were adjusted following the parametric models of van Genuchten [3] and Fredlund and Xing [4].

$$w(\psi) = w_r + (w_s - w_r)\left[\frac{1}{\left[1+(a_{vg}\cdot\psi)^{n_{vg}}\right]^{m_{vg}}}\right] \quad (1)$$

$$w(\psi) = w_s\left[1 - \frac{ln\left(1+\frac{\psi}{h_r}\right)}{ln\left(1+\frac{10^6}{h_r}\right)}\right]\left[\frac{1}{\left[ln\left[e+\left(\frac{\psi}{a_f}\right)^{n_f}\right]\right]^{m_f}}\right] \quad (2)$$

Where:

$w(\psi)$	Water content as a function of suction
ψ	Suction, capillarity or negative pore pressure
w_s	Saturation water content
w_r	Residual water content
a_f, n_f, m_f, h_r , a_{vg}, n_{vg}, m_{vg}	Fit parameter of the models

4 COMPARISON EXPERIMENTAL CURVES WITH SWCC ESTIMATION METHODS

Due to the relative difficulty present in the experimental determination of the SWCC, several methodologies have been developed to estimate it from some properties of the material to be studied. In the present study three different estimation methods have been considered, the first one corresponds to the one proposed by Fredlund & Wilson [5], which considers mainly particle size distribution properties and volume-mass relations, and making use of neural networks makes estimation. The second methodology used corresponds to the one proposed by Vereecken [6], which through a regression model determines the curve using as input parameters sand and clay percentages, the density and the carbon content of the material. Finally, Aubertin methodology [7] using the modified Kovacs method to estimate the porosity, density, void ratio, D10 and D60 of the material. For the application of the different methods of estimation and adjustment of curves, the Soilvision software [8] was used.

In order to quantify the differences between different estimates with respect to the experimental data, the Nash-Sutcliffe efficiency coefficient [9] was used, which measures the adjustment quality between two models and varies between $-\infty$ and 1; In this case, it has been considered that efficiencies above 0.9 are acceptable.

For this verification, six tailings from the literature corresponding to those described in references [7], [10] and [11] have been included in addition to the five tailings tested, and were named Relave 6 to Relave 11.

Table 2 shows a summary of the efficiencies of each estimation method with respect to the experimental data, it is possible to observe the great variability present in each case, however, the estimation through the Fredlund & Wilson method is the one that presents values better on average for the cases under study.

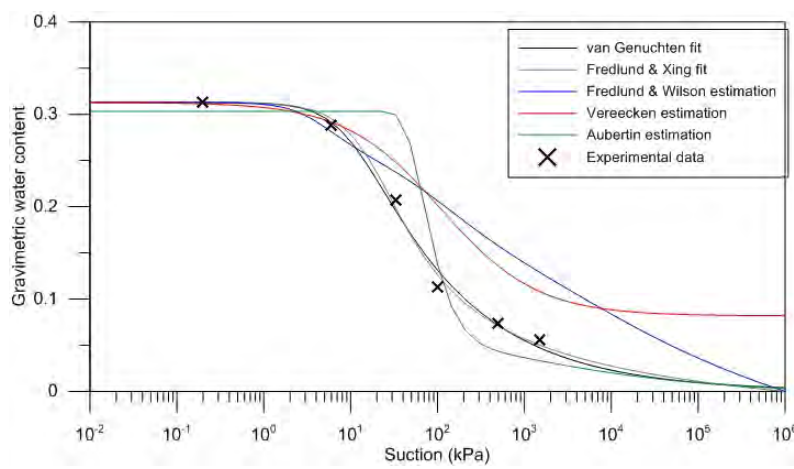

Figure 6. Comparison between fits, estimations and experimental data, tailing 1.

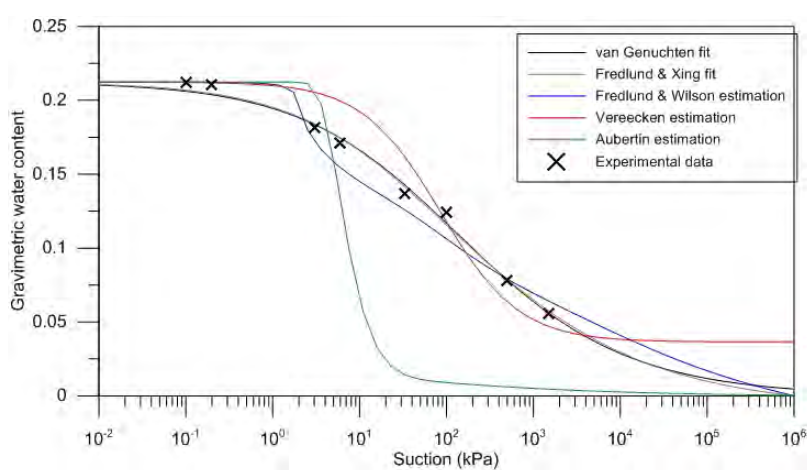

Figure 7. Comparison between fits, estimations and experimental data, spent ore 3.

Table 2. Estimation efficiency for each method.

Type	Name	Fredlund & Wilson	Vereecken	Aubertin
Tailing	Tailing 1	0.64	0.74	0.82
	Tailing 2	0.74	0.90	0.92
	Tailing 3	0.59	0.31	-5.08
	Tailing 4	0.82	0.97	0.40
	Tailing 5	0.93	0.94	0.51
	Tailing 6	0.93	0.95	0.71
	Tailing 7	0.76	0.78	-5.25
	Tailing 8	0.75	-0.45	-3.72
	Tailing 9	0.72	0.87	0.35
	Tailing 10	0.82	0.92	-0.77
	Tailing 11	0.78	0.92	0.06
Spent ore	Spent ore 1	0.95	-0.22	-2.30
	Spent ore 2	0.86	0.26	-1.94
	Spent ore 3	0.96	0.90	-0.58
	Spent ore 4	0.93	0.75	-0.11
	Spent ore 5	0.83	0.25	-3.04
	Spent ore 6	0.79	0.33	-4.82
	Spent ore 7	0.90	0.60	-6.17
	Spent ore 8	0.65	0.27	-4.59
	Spent ore 9	0.76	0.13	-5.62

5 ALTERNATIVE WORK FOCUS

Considering the values obtained in Table 2, the motivation to search for new alternatives that give similar results to the experimental results arises, without having to invest too much time or resources in its execution. Following the previous philosophy, it is proposed to perform a partial experimental determination of the retention curve, complemented by an adjustment method to fill the experimental gaps. In particular, it is proposed to perform the pressure plate test for only three points.

The points proposed for this methodology correspond to a low suction, whose associated humidity can be approximated to the saturation humidity of the soil; A high suction, equivalent to the point of wilting; And finally, an intermediate point, which in this case would be represented by the air entry value (AEV). The latter value was calculated for both the Fredlund & Xing and van Genuchten experimental fit, results are presented in Table 3.

Table 3. AEV for experimental fit VG y F&X

Type	Name	van Genuchten	Fredlund & Xing
		AEV (kPa)	AEV (kPa)
Tailing	Tailing 1	5.9	7.5
	Tailing 2	12.1	13.3
	Tailing 3	44.9	25.8
	Tailing 4	8.5	8.6
	Tailing 5	11.5	10.2
	Tailing 6	2.1	3.0
	Tailing 7	1.8	2.3
	Tailing 8	83.2	106.7
	Tailing 9	18.6	19.0
	Tailing 10	25.9	21.5
	Tailing 11	26.1	19.2
Spent ore	Spent ore 1	0.17	0.19
	Spent ore 2	0.27	0.18
	Spent ore 3	2.79	1.87
	Spent ore 4	0.42	0.33
	Spent ore 5	0.18	0.18
	Spent ore 6	0.20	0.15
	Spent ore 7	0.47	0.29
	Spent ore 8	0.13	0.16
	Spent ore 9	0.13	0.13

Average AEV for tailings corresponds to 22 kPa and for spent ore corresponds to 0.5 kPa. By adjusting these suction values to those typical of the laboratory, the values recommended in Table 4 are obtained.

Table 4. Selected suction values.

Point	Spent ore	Tailing
	Suction (kPa)	Suction (kPa)
1	0.1	0.1
2	1	30
3	1500	1500

Subsequently, the adjustment of van Genuchten and the adjustment of Fredlund & Xing with respect to the experimental curve were verified. For the tailings, both adjustments presented efficiency values close to 1. For spent ore only the Fredlund & Xing adjustment presents efficiency values close to 1, while van Genuchten delivers low values for four cases. The above is summarized in Table 5.

Table 5. Fit efficiency for each method.

Type	Name	van Genuchten	Fredlund & Xing
Tailing	Tailing 1	0.99	1.00
	Tailing 2	0.99	1.00
	Tailing 3	0.99	1.00
	Tailing 4	1.00	1.00
	Tailing 5	0.99	1.00
	Tailing 6	0.99	0.99
	Tailing 7	0.99	1.00
	Tailing 8	1.00	0.97
	Tailing 9	1.00	0.99
	Tailing 10	0.99	1.00
	Tailing 11	0.99	1.00
Spent ore	Spent ore 1	0.20	1.00
	Spent ore 2	0.34	0.99
	Spent ore 3	1.00	1.00
	Spent ore 4	0.99	0.99
	Spent ore 5	1.00	1.00
	Spent ore 6	0.27	1.00
	Spent ore 7	0.46	0.99
	Spent ore 8	0.93	1.00
	Spent ore 9	0.97	1.00

By way of example, Figure 8 shows fitting for tailings 1 and spent ore 3.

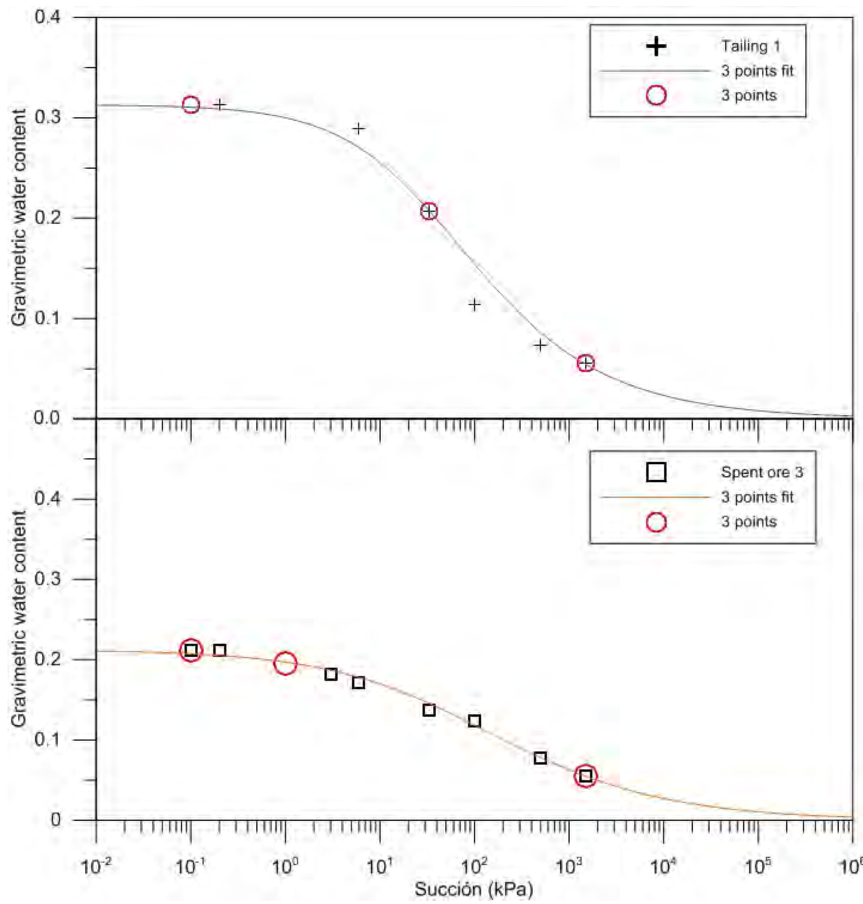

Figure 8. Example of fitting for tailings 1 (upper) and spent ore 3 (lower).

6 CONCLUSIONS

Moisture retention curves (SWCC) were experimentally determined for spent ore and tailings from different mining operations in Chile. These curves can be used as references for studies of unsaturated flow in deposits of tailings, spent ore dump and heap leach, with similar materials.

Experimental results obtained were compared with different estimation methods, finding that none represents the behavior satisfactorily for all the cases studied, according to the calculated efficiency. However, Fredlund & Wilson estimation proves, in general, to be superior to the others studied.

When the total determination of soil water characteristic curve is not possible, it is recommended to use the proposed three-point adjustment method, which includes the following suction values in addition to the saturation point of the sample: For tailings 30 KPa and 1500 kPa, while for spent ore 1 kPa and 1500 kPa. This alternative approach yields higher efficiency values than indirect estimates, representing a more reliable.

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Development of a sampling strategy for mine waste characterization

I. Demers, H.S. Amara, M. Gasnier & J.P. Bernard
Research Institute in Mines and Environment, Université du Québec en Abitibi-Témiscamingue, Rouyn-Noranda, QC, Canada

ABSTRACT: Decisions on mine waste management are often based on characterization results obtained from samples; therefore the quality and representativeness of those samples are critical. Presently, there is no solid sampling protocol specifically for mine waste. The objective of the project is to develop a sampling protocol for tailings based on Gy's Theory of Sampling. A sampling grid was implemented on a section of an inactive tailings impoundment and surface samples were collected systematically. Chemical analyses were performed by XRF and induction furnace, and particle size distribution was obtained. Mapping and interpolation using kriging highlighted the variability in each parameter, and simulations with a different number of samples showed how a different sampling grid affects the overall interpolation results. A suggested sampling protocol involves the determination of heterogeneity first using rapid analyses and experimental variograms, then establishment of the number of samples and their location for a more detailed characterization.

1 INTRODUCTION

Mining operations generate large amount of waste which may have significant impact on the environment. A good knowledge of the chemical, physical and mineralogical features of the waste material is essential for a proper management and reclamation of mine waste and mine-impacted land. Mine waste may contain soluble and deleterious metal elements, residual processing chemicals, or radioactivity. Sampling campaigns are generally required at the pre-feasibility and feasibility stages, and should be performed throughout the life of mine to better characterize the ore, and the waste streams.

The validity of any environmental evaluation is directly related to the representativeness of the samples used in the study. For studies performed on tailings ponds or waste rock piles, there is no clear sampling procedure, nor a precise number of samples required. Tailings ponds and waste rock piles can be highly heterogeneous depending on the ore type, mining, processing and deposition method employed, so sampling these infrastructures is not straightforward.

MEND reports 4.1.1 (Canect 1989) and 4.5.1-1 (SENES 1994) presented some sample collection methods for tailings and waste rock piles but gave minimal information on how many samples to collect and on how to select adequately the location of the samples. Some important guidelines were mentioned; such as to separate the area to sample in sectors according to anticipated variations in particle size and/or geochemistry.

BC AMD Task Force proposed a relationship between the mass of the geologic unit (e.g. waste rock pile) and the number of sample to collect (SRK et al. 1989, 1990), but is not uniformly accepted. Indeed, the linear relationship may suggest too few samples for lower unit tonnages, and may also need to be adapted to each site (Robertson and SRK 1990).

From a fundamental perspective, the theory of sampling (TOS) developed by Pierre Gy provides an estimation of the errors associated with the sampling of heterogeneous materials (Gy

1979, 2004a, b; Petersen et al. 2005; Pitard 1993). The global estimation error (GEE) implied in the sampling process of a zero-dimension lot can be expressed as the sum of the total sampling error (TSE) and the total analytical error (TAE). TSE's contribution to GEE is more significant than TAE, since TSE can be 100 to 1000 larger than TAE (Petersen et al. 2005). The focus should therefore be on minimizing TSE, which is related to material heterogeneity and the sampling process. The sampling process can introduce two types of errors: correct sampling errors and incorrect sampling errors. Even if sampling is done correctly, the fundamental sampling error (FSE) and the grouping and segregation error (GSE) remain. FSE is related to the material heterogeneity while GSE is related to spatial distribution and the sampling process itself (Boudreault et al. 2012). Incorrect sampling errors (ISE) can be minimized or avoided with an adequate sampling procedure and sampling tools. The relationships between the sampling errors are summarized in equations 1 and 2.

$$GEE = TSE + TAE \tag{1}$$

$$TSE = FSE + GSE + ISE \tag{2}$$

Gy's equation (Petersen et al. 2005, Wills and Finch 2016), used to estimate the FSE, can be expressed as:

$$\sigma^2 = \frac{(M - m_s)}{M m_s} C d^3 \tag{3}$$

where σ^2 is the variance, M is the mass of the lot (g), m_s is the mass of sample from the lot (g), C is a sampling constant based on the mineralogy, and d is the largest particle size (cm), often taken as D_{95} (size at which 95% of the particles pass through the sieve).

The sampling constant C is determined from:

$$C = fglm \tag{4}$$

where f is a particle shape factor, g is a particle size distribution factor, l is a factor related to liberation, and m is a factor related to the mineralogy.

In the case of a sampling campaign on a tailings impoundment covering several tens of hectares, it can be assumed that M is much larger than m_s, therefore equation 3 can be simplified to:

$$\sigma^2 = \frac{C d^3}{m_s} \tag{5}$$

The sampling of mine waste brings complex issues, mainly related to the mass of the lots. Works on sampling procedures were performed in other industrial fields, such as contaminated soils (Boudreault et al. 2016), geotechnical characterization (Phoon and Kulhawy 1999) and recycled material sorting (Maris et al. 2014), but few research was done on mine waste. Considering than stakeholders have higher expectation from the mining industry, and require more precise information, the development of sound sampling practices should be investigated.

2 OBJECTIVE

The objective of the project is to develop a sampling protocol based on Gy's Theory of Sampling (TOS) (Gy 1979), which could be applicable to mine tailings and waste rock, to improve representativeness of environmental assessments. More specifically, this study investigates the number and position of sampling points on a tailings pond during a sampling campaign.

3 MATERIALS AND METHODS

3.1 *Site description*

The study area is part of the Westwood-Doyon Mine, a gold mine owned and operated by Iamgold Corporation, located approximately 40 km east of Rouyn-Noranda, Quebec, Canada (Figure 1). Tailings pond #1 was selected for the study because it has been inactive for nearly 30 years and the tailings surface is stiff enough to walk on and sample. Tailings pond #1 contain tailings from the previous Doyon mine which were exposed to the atmosphere and generated acid mine drainage. Although significant oxidation occurred on the tailings, no hard pan was formed.

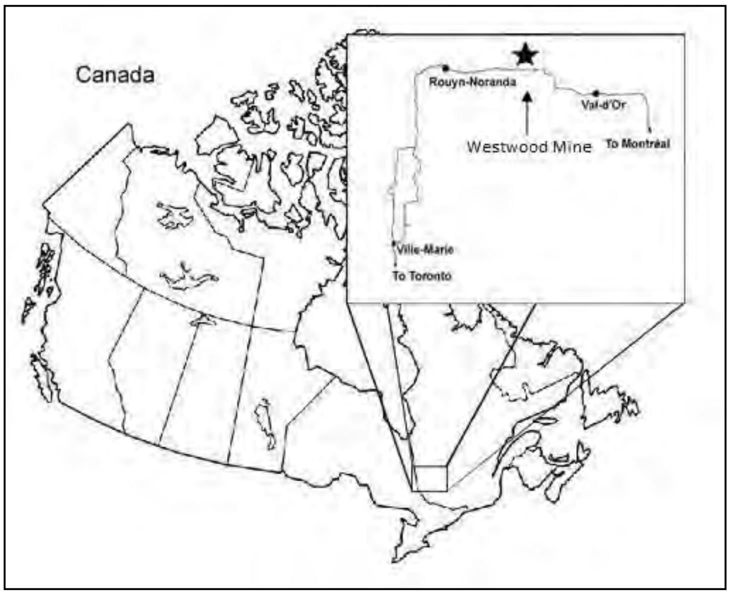

Figure 1: Location of sampling site

3.2 *Sampling procedure*

The sampling campaign focused on surface sampling of a portion of Tailings pond #1. A 480 m by 120 m grid was implemented on the site, with square mesh of 30 m by 30 m, as shown in Figure 2. The edge (point 0) was chosen arbitrarily to consider the lot as a zero dimension lot and to obtain bias-free samples. A total of 85 samples were collected from the tailings surface using a 350 ml cylinder with sharpened edges (so that particles have equal chance of being in or out the sampler); the sampler was rinsed after each sampling to prevent contamination.

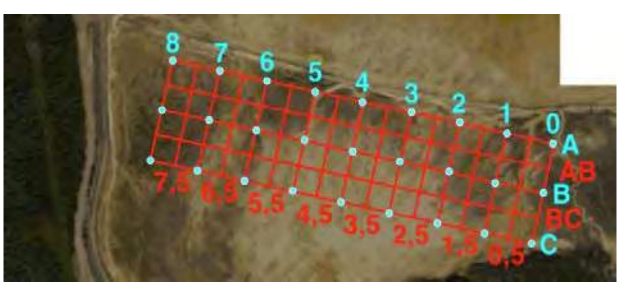

Figure 2: Sampling grid placed on the tailings impoundment, each intersection is a sampling point

3.3 *Analytical procedure*

Once all the samples were collected and stored in air-tight bags, they were transported to UQAT's laboratory for physical and chemical analyses. To obtain information on replicability, each sample was sub-sampled and analyzed in triplicate. This study focused on particle size distribution, sulfur, iron and calcium content.

3.3.1 *Sample preparation*

The first step of sample preparation is to dry the samples in an oven at 60°C for 24 to 48 hours. Then, the particles are passed through a 300 µm screen to break the lumps and remove any unwanted foreign particle. Then the samples are homogenized by rolling on a paper 60 times (15 times each corner). A sub-sample is placed in small bags for further analyses.

3.3.2 *Samples physical characterization*

Particle size distribution was obtained with a Microtrak S3500 laser particle size analyzer, which measures particles sizes from 0.02 to 2800 µm. From the particle size distribution, D_{50} were obtained, representing the size at which 50% of the particles pass through the sieve. This measure is also useful to evaluate the material's homogeneity. Coefficients of uniformity (C_U) were also calculated, using $C_U = D_{60}/D_{10}$, where D_{60} and D_{10} are the size at which 60% and 10% of the particles pass through the sieve.

3.3.3 *Samples chemical characterization*

Total sulfur and carbon content was obtained with an Eltra C/S-800 induction furnace. The results are provided with a 5% measurement precision. Sulfur content is a relevant parameter to consider for mine site reclamation since it can be related to acidification potential. Carbon content can be related to neutralization potential; however the values measured in the present study were low and therefore not discussed in this paper.

The chemical composition of the main elements in the samples was obtained with a Niton XZ3t 900 portable x-ray spectrometer. The focus of the study was on three major elements: iron, calcium and sulfur.

These analytical methods may not be the most accurate; however one of the objectives of the study was to define a sampling strategy based on rapid and convenient measurement methods that would be potentially applicable in field sampling campaigns.

3.4 *Geostatistical procedure*

The geostatistical procedure involved two main steps, lot characterization and sampling grid optimization.

3.4.1 *Lot characterization*

The first step involves the estimation of the lot heterogeneity using experimental variograms. The estimation variance was reduced using kriging, which is an unbiased interpolation method and an appropriate linear estimation tool under two conditions: (i) valid stationary hypothesis: the parameters values does not depend on the position of the samples, but only on the distance between the sampling points; (ii) correct modeling of the experimental variogram. From this first step the heterogeneity of the sampling area and its parameters, and a map of the parameters distribution are obtained.

3.4.2 *Sampling grid optimization*

Using information obtained from the lot characterization step, the number of sampling points is optimized by establishing a new sampling grid. Differential mapping was then performed to estimate the error between the new grid and the complete lot grid.

4 RESULTS

4.1 *Tailings characterization*

Table 1 presents a summary of the samples characterization results. Particle size distribution, expressed as D_{50}, shows a significant range between minimum and maximum value, with the mean value (28.38 µm for D_{50}) much closer to the minimum value. C_U values are between 5 and 8, which is slightly lower than typical tailings once deposited as reported by Bussière (2007) as between 10 and 30. The different values may be explained by the weathering of the Doyon tailings since deposition, which may have removed and favored certain particle sizes by percolation and/or oxidation and precipitation of secondary minerals. Sulfur, calcium and iron show relatively low variance and coefficient of variations, meaning that the chemical composition is relatively homogeneous, as expected from a tailings impoundment. However, peaks were observed in a few samples, especially for S (2.751%) and Fe (5.42%).

Table 1: Main sample characterization results

Parameter	**Min value** (µm for D_{50} and C_U, % for S, Ca, Fe)	**Max value** (µm for D_{50} and C_U, % for S, Ca, Fe)	**Mean** (µm for D_{50} and C_U, % for S, Ca, Fe)	**Median** (µm for D_{50} and C_U, % for S, Ca, Fe)	**Variance** (µm^2 for D_{50} and C_U, %^{2} for S, Ca, Fe)	**COV** (%)
D_{50}	18.37	97.26	28.38	25.5	129.62	6.79
C_U	5.2	8.69	6.49	6.32	0.73	0.69
S	0.656	2.751	1.375	1.349	0.188	0.353
Ca	0.54	1.24	0.93	0.95	0.03	0.147
Fe	2.46	5.42	4.05	4.00	0.377	0.483

Gy's equation (Equation 5) was then used to verify that the sample mass collected is adequate to minimize the fundamental sampling error. To calculate C, the factors f, l, g, and m need to be defined. The sulfur content was used as the desired property to obtain from the sampling. However C is related to mineralogy, so it was assumed that all sulfur in the tailings are present as pyrite. The mean S content from Table 1, 1.375%, was transferred to a pyrite content of 5.15%. Using pyrite density and gangue density (2.7 g/cm^3), m was determined to be 86.41 g/cm^3. Considering the particle size distribution, g was estimated as 0.25 and l as 0.15. Factor f is equal to 0.5 for a non-gold ore (Wills and Finch, 2016). The calculated value for C using equation 4 is 1.62 g/cm^3.

The largest particle size of the sample, estimated by D_{95}, was 124.5µm. The maximum variance can be chosen as a 1% error, or $\sigma = 0.01$. Using all these values, the minimum sample mass can be obtained from equation 5. The result is that a sample mass of 32.6 g would be enough to reach the desired variance. However, to account for other errors involved in sample preparation, it is suggested to double the m_s value. A sample size of approximately 65 g would be required. In the field campaign performed in Doyon tailings impoundment, the sample size was close to one kg, therefore it was judged sufficient to minimize the FSE.

4.2 *Lot heterogeneity and mapping of characterization results*

Omnidirectional experimental variograms were produced using the characterization results and the spatial positioning of the samples. An example is presented in figure 3, for sulfur content. No nugget effect was found for the elemental parameters. The C_U variogram did not show a range, so for this paper it will not be further discussed. For each variogram, the model, sill and range were obtained and reported in table 2.

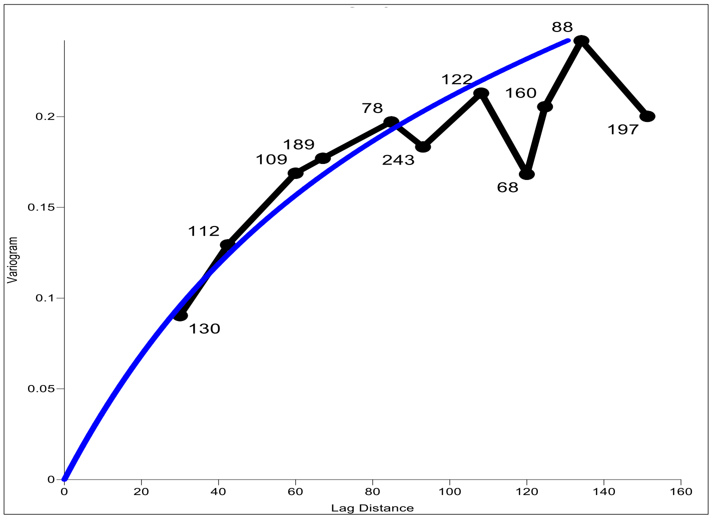

Figure 3: Experimental variogram for sulfur concentration in tailings

Table 2: Parameters of the variograms using the entire data set

Parameter	Model	Range (m)	Sill
C_U	linear	-	0.7
S	exponential	55	0.16
Ca	exponential	50.5	0.023
Fe	spherical	61	0.562

For the three elemental parameters, the range is between 50 and 61 m, which represents the distance between each point where the covariance is nil, or the samples are not correlated. The sill, from 0.562 to 0.023, is the largest variance between two samples.

Using the above information, and normal kriging, maps of the sampling area were drawn with Surfer to position the measurements and extrapolate the data between the sampling points. Results are presented in figure 4. The sulfur map shows a “hot spot” in the upper-center of the grid, and lower values around the edges. The sulfur “hot spot” corresponds to high Fe concentrations, indicating a possible area of higher pyrite content. A very low Fe value was observed in the upper-left area, which may or may not be realistic. Iron and calcium appear in higher concentrations in the left portion of the grid.

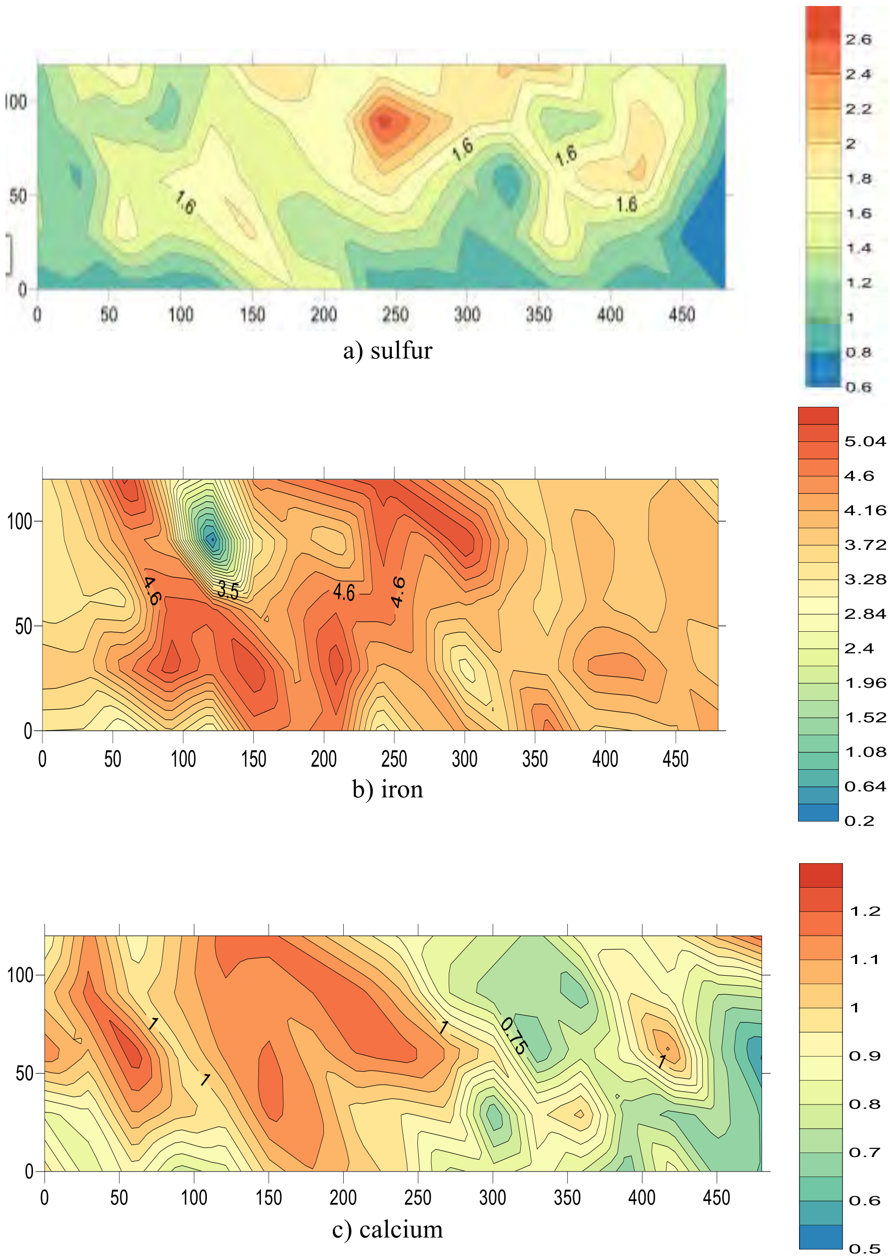

Figure 4: Extrapolation maps for sulfur (a), iron (b) and calcium (c) using normal kriging for all samples collected in the sampling grid

5 DISCUSSION

5.1 *Optimization of number of sampling stations*

The sampling grid may be optimized based on the results presented above. As seen in table 2, the range for sulfur, calcium and iron are 55, 50 and 61 m, respectively. It is postulated that the distance between each sample could be increased from 30 m to 60 m without much impact on the overall results, since points 30 m apart are likely related. Maps were drawn using only the blue points on Figure 2, which reduces the number of data points from 85 to 27. Figure 6 presents the updated maps.

The sulfur map still shows the “hot spot” in the center of the field and the edges still show the lower concentrations. The updated Fe map dropped the very low value in the upper-left area, so the color scale is different, nevertheless the low concentrations around the edges are similar and areas of higher concentrations correspond. Ca concentrations show less variations in the updated map compared to the full data map.

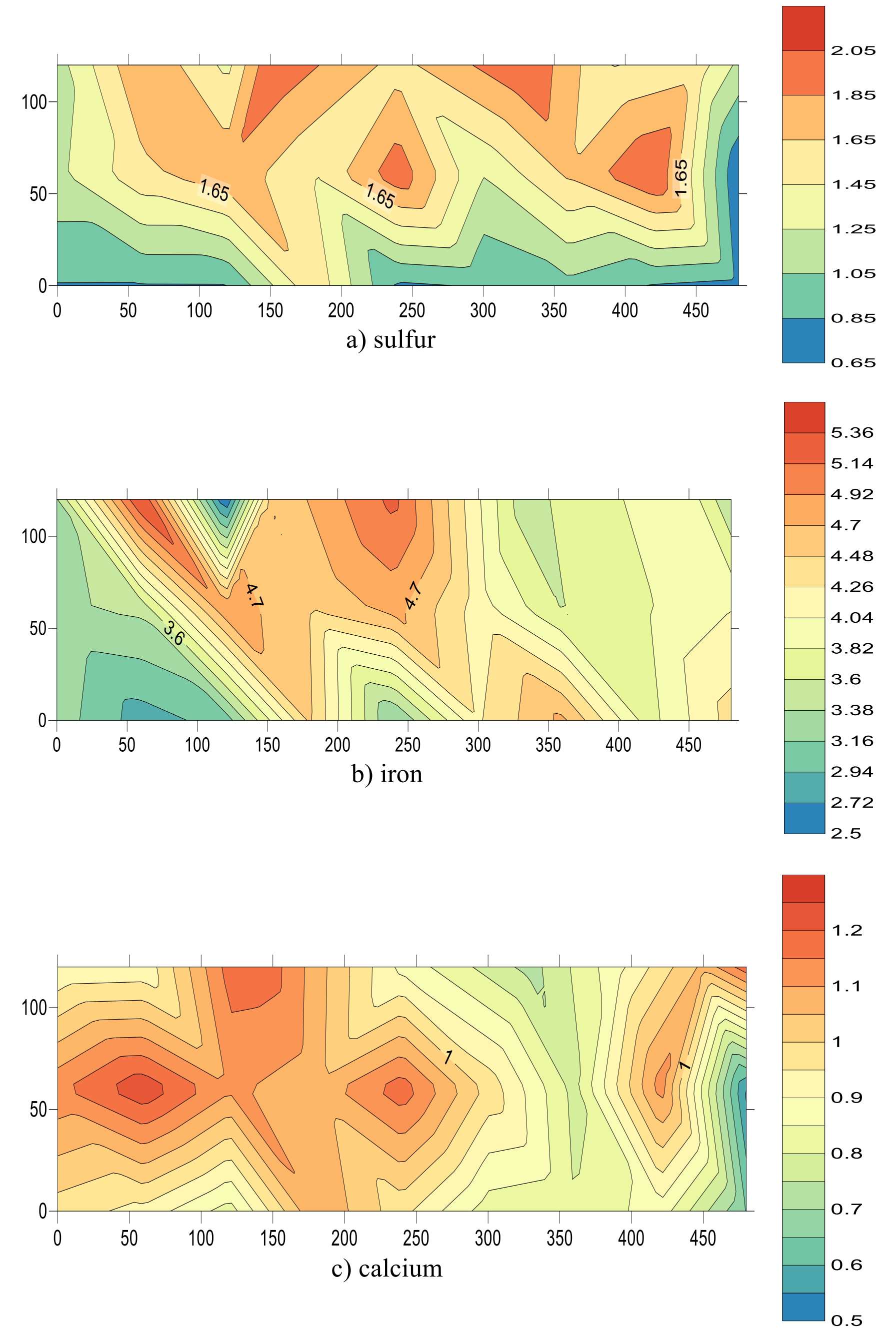

Figure 5: Extrapolation maps for sulfur (a), iron (b) and calcium (c) using normal kriging for 27 samples (60 m x 60 m) collected in the sampling grid

To validate the observed differences and similarities between the scenario with 85 samples (30 m x 30 m) and the scenario with 27 samples (60 m x 60 m), differential maps were produced. Figure 6 presents the differential map for sulfur only (for space constraints). The relative error on sulfur measurements is mostly between 0 and 10%, with a mean gap of 3.6%. Calcium also presented a relative difference mostly between 0 and 10%, for a mean difference of 4.4%. Iron was the element with the largest relative differences, from 10 to 50%.

The person responsible for sampling should therefore select the acceptable relative error and build the sampling grid accordingly. The study here was done on a relatively small portion of the tailings impoundment and can help to select the desired sampling strategy for a full impoundment campaign.

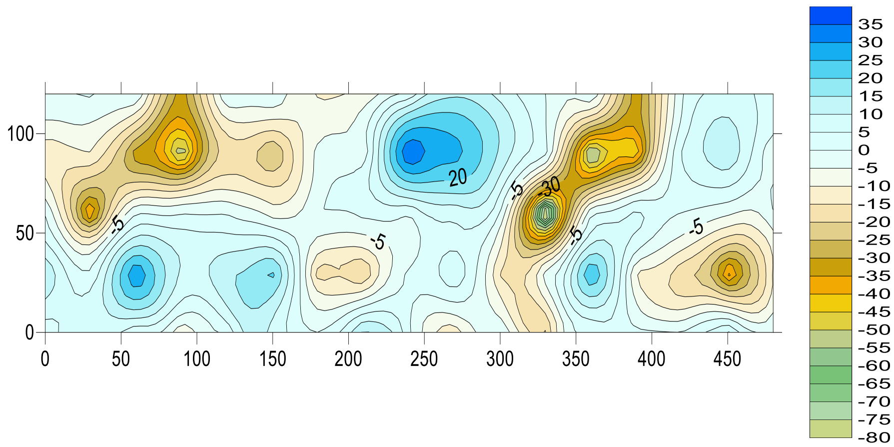

Figure 6: Differential map for sulfur concentration on the tested area

5.2 *Optimization of location of sampling stations*

To set up a sampling grid, a preliminary sampling campaign could be performed to evaluate heterogeneity, or more specifically, to determine the correlation distance between a pair of points. A suggested procedure is to start from a random point on the area to be sampled, and then collect samples every 2^n steps, in the x-direction and a perpendicular y-direction. Rapid characterization methods are proposed, such as portable XRF, to obtain values for several elements of interest. Experimental variograms can then be constructed. Results from this procedure are presented in table 3. Even with a small number of samples (17 in this case), the same model (exponential) was obtained for sulfur and calcium. However, no conclusion can be drawn for iron, which was shown in 5.1 to be more sensitive to the number of data points.

Table 3: Parameters of the variograms using 17 data points

Parameter	**Model**	**Range (m)**	**Sill**
S	exponential	72	0.04
Ca	exponential	90	0.045
Fe	linear	-	0.15

6 CONCLUSIONS

Sampling mine waste for environmental characterization is not an easy task. There is no agreed upon protocols to select the number and location of samples specifically for tailings and waste rock sampling. The work involved a sampling campaign on a portion of a tailings impoundment to validate the sample mass collected using Gy's theory of sampling, and to test the application of basic geostatistics to optimize sample number and location on a target area.

Results indicated that the minimum sample mass for a low variance should be approximately 65 g to evaluate sulfur content, which is less than what was really collected. The fundamental sampling error is considered minimal. Experimental variograms were constructed for C_U, S, Ca, and Fe. From the modelling of the variograms, the spacing between points could possibly be increased from 30 m to 60 m (range). Differential maps, interpolated using normal kriging, showed a relative difference generally between 0 and 10% for Ca and S values, and from 10 to 30% for Fe values.

Finally, the development of a sampling plan for a tailings impoundment may follow these steps:

1) Evaluate heterogeneity using experimental variograms, and extract the range and sill from the models;
2) Establish the sampling grid according to information found in step 1;
3) Use standard extrapolation methods, such as normal kriging, to estimate the parameters value throughout the sampled area.

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Evaluation of mechanical and hydraulic properties of geosynthetic clay liners for mining applications

C.A. Bareither, S. Ghazi Zadeh, J. Conzelmann, J. Scalia IV & C.D. Shackelford
Colorado State University, Fort Collins, CO, USA

ABSTRACT: The hydraulic conductivity and internal shear strength of needle-punched, reinforced geosynthetic clay liners (GCLs) were evaluated with varying non-standard chemical solutions. Hydration solutions used to evaluate hydraulic conductivity included (i) de-ionized water, (ii) tap water, (iii) control water, (iv) synthetic gold mine process solution (Au-PS), and (v) synthetic bauxite mine process solution (B-PS). Hydraulic conductivity experiments revealed that low hydraulic conductivities can be achieved in strong chemical solutions; however, there is potential for preferential flow to occur along needle-punched fiber bundles in stronger chemical solutions with high peel strength GCLs. Hydration solutions used to evaluate internal GCL shear strength included de-ionized water, Au-PS, and B-PS. A reduction in internal shear strength was measured in the B-PS relative to de-ionized water, whereas GCLs hydrated in Au-PS exhibited an increase in internal shear strength. Increased shear resistance was attributed to cation exchange in the bentonite, whereas B-PS likely reduced interface friction between the reinforcement fibers, geotextiles, and bentonite.

1 INTRODUCTION

Geosynthetic clay liners (GCLs) are effective barrier materials for liner and cover systems in waste containment applications. Exposure to non-standard chemical solutions can alter the chemical and mechanical properties of both the bentonite and geotextiles comprising a GCL. Liquid wastes and leachates in waste containment systems can hydrate GCLs with different non-standard solutions via direct contact or through punctures in an overlying geomembrane. Non-standard solutions encountered in mining applications can have a broad range of ionic strength, pH, chemical constituents, and ratio of monovalent and divalent cations (RMD). For example, heap leach operations in copper mining generate extremely acidic solutions containing high sulfate, chlorine, phosphate, and oxidizing agents (e.g., Hornsey et al. 2010; Shackelford et al. 2010) whereas highly alkaline solutions are characteristic of bauxite mining (Gräfe et al. 2011).

The layer of sodium (Na)-bentonite within a GCL forms a low hydraulic conductivity (*k*) ($\leq$ $2\text{-}3\times10^{-11}$ m/s) layer when hydrated with a dilute solution (e.g., tap water, TW), and this hydrated layer acts as a barrier for fluid flow and contaminant transport (Shackelford et al. 2000). The effectiveness of GCLs in containment applications has been demonstrated for systems with low ionic strength, *I*, solutions and leachates (Shackelford et al. 2000; Jo et al. 2004). The low *k* of Na-bentonite results from the osmotic adsorption of water around montmorillonite platelets (montmorillonite is the dominant mineral of bentonite). Osmotically associated water yields swelling of the immobile phase, reducing pathways for flow and chemical transport. However, for osmotic swelling to occur, monovalent cations (e.g., Na^+) must dominate the montmorillonite exchange complex, and the permeant solution must have low *I*, although the definition of low depends on the specific combination of bentonite (and additives, if any), liquid chemistry (i.e. concentration and pH), and effective stress (σ'). Unfortunately, many liquids, such as brines generated in mining and energy production applications, may exhibit high *I* or extreme pH (< 3, > 12) that can result in high *k* of GCLs, particularly at low σ'. Thus, to anticipate GCL performance in mining applications, project specific hydraulic compatibility tests are necessary. ASTM D6766 (*Standard Test Method for Evaluation of Hydraulic Properties of Geosynthetic Clay Liners Permeated with Potentially Incompatible Solutions*) provides a method for measuring *k* of a GCL with potentially incompatible solutions.

The assessment of internal shear strength of GCLs in varying chemical solutions is relatively limited. Müller et al. (2008) evaluated the long-term shear strength of GCLs submerged in deionized water (DIW) and TW, and reported that failure regularly occurred in specimens submerged in DIW (i.e., low *I*). However, GCL specimens submerged in TW generally resisted failure due to cation exchange that increased the internal shear strength of bentonite and reduced tensile stress transferred to the needle-punched reinforcement fibers. The internal friction angle (ϕ') for sodium montmorillonite reported by Mesri and Olson (1970) was ≤ 4°. However, the exchange of divalent calcium (Ca^{2+}) for monovalent Na^{+} in montmorillonite can increase ϕ' to approximately 10-15°, due to reduced hydrated water in the diffuse double layer (Mesri and Olson 1970). Although an increase in shear strength due to cation exchange is beneficial for slope stability, the exchange of divalent-for-monovalent cations can increase *k*, by factors ranging from one to several orders of magnitude depending on the extent of cation exchange (e.g., Shackelford et al. 2000). Thus, cation exchange in bentonite within the GCL has mechanical advantages but hydraulic disadvantages for long-term performance as a barrier material.

The use of GCLs in mining applications can expose GCLs to aggressive, non-standard chemical solutions. Thus, an assessment of the effects of exposure to non-standard chemical solutions on hydraulic and mechanical properties of GCLs is needed for design. The objective of this paper is to provide an overview of a research program focused on the evaluation of hydraulic and mechanical properties of GCLs for mining applications. Data are presented from multiple hydraulic and strength tests along with concise observations and conclusions.

2 MATERIALS AND METHODS

2.1 *Geosynthetic Clay Liners*

Three commercially-available, needle-punched GCLs were included in this study. Properties of these GCLs are summarized in Table 1. GCL-1 consisted of a layer of natural Na-bentonite sandwiched between two nonwoven geotextiles, and bonded by needle punching. GCL-2 consisted of a layer of natural Na-bentonite sandwiched between a woven carrier geotextile and a nonwoven cover geotextile. GCL-1 and GCL-2 were both non-heat-treated GCLs. GCL-3 consisted of a layer of powder bentonite sandwiched between a woven carrier geotextile and nonwoven cover geotextile. Heat treatment was used by the manufacturer of GCL-3 to thermally-bond needle-punched reinforcement fibers to the carrier geotextile. GCL-1 had a high peel strength (2170 N/m), whereas the peel strengths were lower and comparable between GCL-2 (720 N/m) and GCL-3 (740 N/m). The percent area covered by reinforcement fibers bundles was estimated for GCL-1 (5.3%) and GCL-2 (1.0%) (percent area covered by reinforcement fiber bundles for GCL-3 was unavailable). These percentages were useful for assessing the influence of fiber bundles on hydraulic behavior of the GCLs.

Table. 1. Properties of the geosynthetic clay liners (GCLs)

Parameter	GCL-1	GCL-2	GCL-3
Manufacturer reported peel strength (N/m)	2170	720	740
Percent of surface area covered by fiber bundles [a]	5.3	1.0	NM
Average dry bentonite mass per area (kg/m^2)	5.6	5.2	3.4
Swell index in de-ionized water (mL/2-g)	25	27	NM

[a] Approximated via measured fiber bundle size and assuming cylindrical fiber bundles.
NM = not measured

2.2 *Hydration Solutions*

A summary of the five solutions used in this study is in Table 2. Solutions used for hydraulic conductivity testing included DIW, TW, a synthetic soil pore water reflective of worst case subgrade hydration and porewater percolation (control water, or CW, as described in Scalia and Benson 2010, and included in ASTM D5084), a synthetic gold mine process solution (Au-PS), and a synthetic bauxite mine process solution (B-PS). The synthetic mining solutions were developed to represent actual mine process liquids (Ghazi Zadeh et al. 2017). The Au-PS represented a typical gold and silver mine pregnant leach solution, and had a neutral pH, $I = 0.049$ M, and RMD = 0.47. The B-PS represented typical bauxite solution characteristics, and had pH > 12, $I = 0.067$ M, and RMD = 1.2. pH and electrical conductivity (EC) were measured using an Orion Versa Star pH/Conductivity meter (Thermo Scientific, Waltham, MA).

Table 2. Characteristics of solutions used in hydraulic conductivity and shear strength testing.

	Solutions				
Parameter	DIW	TW	CW	Au-PS	B-PS
Ionic Strength, I (mM)	-	-	6.0	49	67
Ratio monovalent-to-divalent cations, RMD ($mM^{1/2}$)	-	-	0.19	10	26
Electrical Conductivity, EC (S/m)	4.2×10^{-4}	1.3×10^{-2}	5.1×10^{-2}	0.34	0.70
pH	7.0	6.7	5.7	5.1	12.0

2.3 *Hydraulic Conductivity Testing*

Hydraulic conductivity of GCLs with permeant solutions was measured in chemically resistant permeameters following ASTM D6766, expect backpressure saturation and permeant interface devices were not included. A volumetrically-graded burette was used as the falling headwater reservoir, and a constant outflow pressure was achieved by connecting the outflow tube to a flexible, fluorinated ethylene propylene bag. Backpressure was not applied to allow for the convenient collection of outflow for chemical analysis and to preclude chemical alterations in the solution. Detailed description of the test method is in Conzelmann and Scalia (2016).

For hydraulic conductivity specimens that exhibited high k (> 10^{-10} m/s) after the termination criteria listed in ASTM D6766 had been achieved, rhodamine WT dye (5 mg/L) was added to the influent liquid at the conclusion of testing to determine if sidewall leakage was occurring. Tests were permeated with rhodamine WT dye bearing influent solution until dye became visible in the effluent. No indication of sidewall leakage was found in any tests. However, dye testing did reveal preferential flow paths as described subsequently.

2.4 *Direct Shear Testing*

2.4.1 *Displacement-Controlled Direct Shear*

Displacement-controlled direct shear tests were conducted in a unique shear machine designed with capabilities to test GCLs under non-standard chemical solutions (e.g., pH ≤ 1 or pH ≥ 12), high normal stresses (σ_n up to 2000 kPa), and elevated temperatures (up to 80 °C). Details of the shear machine are described in Soleimanian and Bareither (2016). Internal shear strength testing of GCLs was conducted in accordance with ASTM 6243/6243M. Minor deviations were adopted as needed to adhere to the developed testing equipment and recommendations in literature (e.g., Fox and Stark 2015). All experiments reported herein were conducted on 150-mm-square GCL specimens at a horizontal displacement rate of 0.1 mm/min.

Geosynthetic clay liner specimens were hydrated following a 2-stage procedure. Stage 1 included $\sigma_n = 20$ kPa and hydration for at least 2 d or longer times in select non-standard chemical solutions to evaluate effects of longer-duration exposure. Stage 2 involved hydration for 1 d under the target σ_n for shear testing. The required σ_n for shear testing was applied incrementally to minimize bentonite extrusion. An initial $\sigma_n = 20$ kPa was applied and σ_n was increased via a load-increment-ratio of one such that σ_n on the GCL was doubled every 3 to 4 h.

2.4.2 *Stress-Controlled Direct Shear*

Stress-controlled direct shear tests were conducted in unique shear boxes developed with the following capabilities: dead-weight loading on a 150-mm-square GCL specimen; hydration and testing of GCLs in non-standard chemical solutions; and mitigation of shear platen rotation using a vertical load reaction frame (Ghazi Zadeh and Bareither 2017). Experiments discussed herein were conducted following a rapid-loading shear (RLS) test procedure, whereby shear loads were successively increased until a specimen failed or until a desired shear stress was attained. All test specimens were conditioned via application of an initial normal stress (σ_{ni}) = 20 kPa followed by bentonite hydration for at least 4 d. Longer hydration times of 15 d were applied to select specimens to assess the effect of extended hydration and exposure to non-standard solutions on shear behavior.

Shear deformation was initiated following specimen conditioning via stepwise increase in the shear load at the rate of 20 kg applied every 15 min. This loading sequence approximated an average horizontal displacement rate of 0.1 mm/min. The RLS tests were conducted in a manner such that the shear load was systematically increased until a specimen failed or the test was terminated. Following each RLS test, GCL specimens were investigated for any signs of non-standard internal failure (e.g., geotextile elongation, stress localization, etc.).

3 RESULTS

3.1 *Hydraulic Conductivity*

Hydraulic conductivity for experiments on GCL-1 and GCL-2 permeated with DIW, TW, CW, Au-PS, or B-PS are shown in Fig. 1. Specimens of GCL-1 were permeated with DIW, TW, CW, and Au-PS. All tests permeated with DIW (n = 1) and TW (n = 3) exhibited low k ranging from 8.3×10^{-12} m/s to 2.6×10^{-11} m/s. Tests permeated with CW (n = 4) exhibited low k for one of four tests, 3.5×10^{-11} m/s, and higher k for three of four tests ranging from 9.2×10^{-10} m/s to 4.5×10^{-9} m/s. Tests permeated with Au-PS (n = 3) exhibited higher k for all tests ranging from 4.9×10^{-10} m/s to 2.6×10^{-8} m/s. Specimens of GCL-2 were permeated with CW, Au-PS, and B-PS. All GCL-2 specimens exhibited low k ranging between 5.4×10^{-11} m/s and 2.7×10^{-11} m/s.

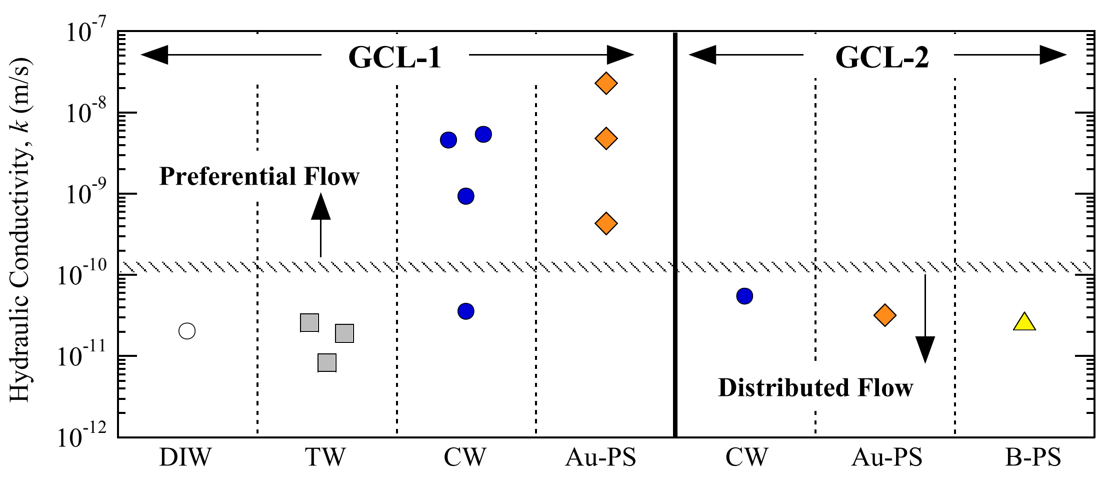

Figure 1. Hydraulic conductivity for GCL-1 and GCL-2 permeated with DIW, TW, CW, Au-PS, or B-PS.

Rhodamine-WT dye was added to the influent liquid, and permeated through specimens that exhibited $k > 10^{-10}$ m/s. The dyed specimens revealed preferential flow along some, but not all, needle-punched fiber bundles. Exemplary photographs of the stained fiber bundles are shown in Fig. 2. The peel strength of GCL-1 was approximately three times larger than GCL-2 (Table 1), which resulted in a larger percentage of the GCL surface that was covered by fiber bundles. The dyed fiber bundles of GCL-1 (Fig. 2) are believed to have functioned as preferential flow paths, resulting in the 50- to 100-times greater k observed in GCL-1 compared to GCL-2.

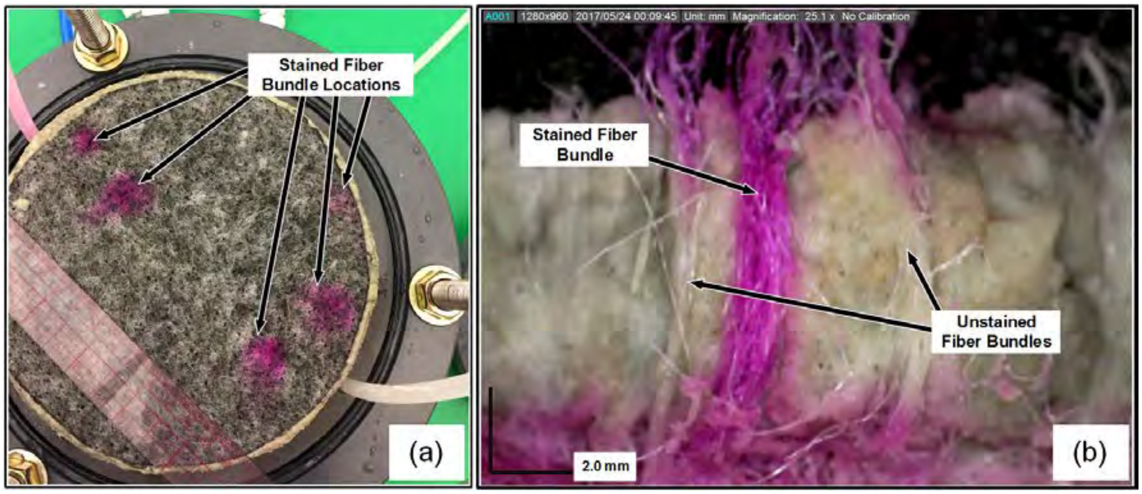

Figure 2. (a) View of dyed carrier (black) textile of GCL-1 during disassembly. (b) Fiber bundles and surrounding bentonite stained with rhodamine-WT dye after termination.

A plot of k versus manufacturer reported peel strength is shown in Fig. 3. The higher peel strength and percent area covered by fiber bundles of GCL-1 compared to GCL-2 is the likely cause of GCL-1 exhibiting higher k due to preferential flow compared to GCL-2 exhibiting low k. These results suggest that GCLs with higher degrees of needle punching may result in preferential flow through fiber bundles under low effective stress conditions (e.g., 27.6 kPa, 4 psi).

All specimens of GCL-1 and GCL-2 permeated with CW, Au-PS, and B-PS that did not exhibit preferential flow resulted in low k similar to that of DIW and TW, which demonstrates that all tests liquids were compatible with both GCLs if preferential flow was not present. The low k of the test permeated with B-PS demonstrates that GCL-2 can maintain low k at high pH.

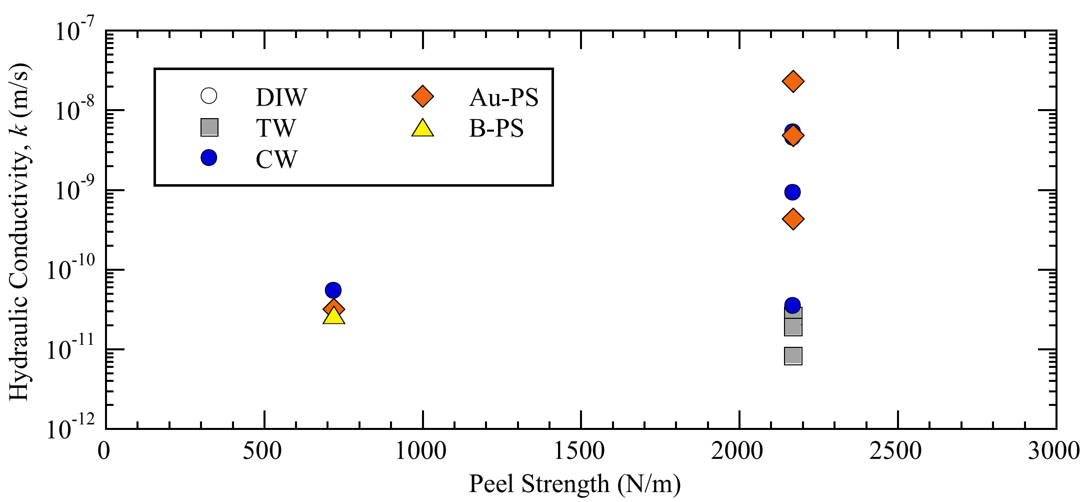

Figure 3. Relationship of hydraulic conductivity versus average peel strength for GCL-1 and GCL-2 specimens permeated with different solutions.

3.2 *Internal Shear Strength*

3.2.1 *Displacement-Controlled Direct Shea*

Relationships of shear stress (τ) versus horizontal deformation (δ_h) for 150-mm GCL shear tests on GCL-2 to assess the influence of hydration time in B-PS on internal shear behavior and shear strength are shown in Fig. 4. These experiments include 4-d hydration in DIW and 15, 30, and 60 d hydration in B-PS that were sheared under σ_n = 500 kPa. Similar shear behavior was observed for all four specimens, which exhibit an increase in τ to peak shear strength (τ_p) and subsequent decrease in τ to large-displacement shear strength (τ_{ld}). The three experiments conducted in B-PS displayed a decrease in both τ_p and τ_{ld} relative to the specimen hydrated in DIW.

Swell index tests (ASTM D 5890) were conducted on bentonite extracted from specimens hydrated in B-PS for different durations (i.e., 15, 30, and 60 d). The swell index for all experiments in B-PS ranged between 22 and 24 mL/2-g, which was modestly lower than the swell index of the non-pre-hydrated bentonite (27 mL/2-g, Table 1). This observation suggests that limited cation exchange occurred during hydration and the shear strength of bentonite should be more or less unaffected. The reduction in τ_p and τ_{ld} with increasing hydration time was hypothesized to develop from changes in frictional resistance that occurred at locations of reinforcement fiber entanglement in the carrier geotextile. Additional research is ongoing to evaluate this hypothesis and elucidate mechanisms that contributed to strength loss following hydration in B-PS.

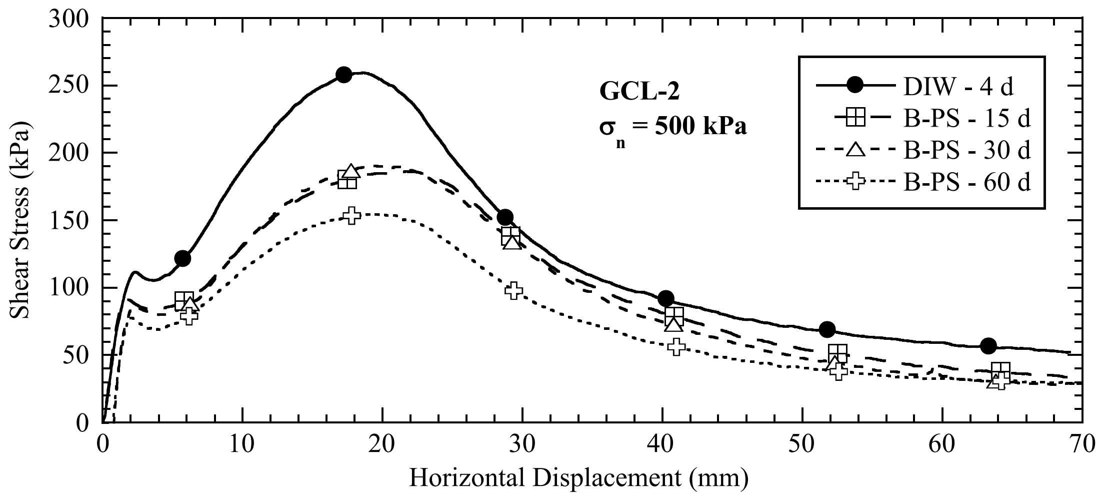

Figure 4. Relationships between shear stress and horizontal displacement for GCL-2 tested under a normal stress (σ_n) of 500 kPa following hydration in de-ionized water (DIW) for 4 d and hydration in synthetic bauxite mine process solution (B-PS) for 15, 30, and 60 d.

3.2.2 *Stress-Controlled Direct Shear*

Rapid-loading, stress-controlled shear tests were conducted on GCL-2 and GCL-3 to evaluate the effects of hydration in non-standard chemical solutions. All GCL specimens were sheared under σ_{ni} = 20 kPa at room temperature (20 °C). Experiments conducted on GCL-2 and GCL-3 included 4-d hydration in DIW as well as 15-d hydration in DIW, Au-PS, and B-PS. The hydration time of 15 d was arbitrarily selected to provide a longer hydration time, but was also reasonable to conduct multiple experiments. The two GCLs (i.e., GCL-2 and GCL-3) had comparable peel strength (Table 1), but GCL-2 was a non-heat-treated needle-punched GCL, whereas GCL-3 was a heat-treated needle-punched GCL.

Relationships of δ_h versus elapsed time are shown in Fig. 5 for the stress-controlled direct shear tests on GCL-2 and GCL-3. In general, the shear behavior measured for specimens hydrated in DIW for 4 d and 15 d were similar for both GCLs. This observation suggests that shear behavior and shear strength should not change with increasing hydration time when all other experimental variables are held constant. Similar findings regarding the negligible effect of hydration time on GCL shear strength have been reported by McCartney et al. (2009) and Soleimanian and Bareither (2016). In contrast, GCL-2 and GCL-3 specimens hydrated in B-PS experienced higher shear deformation and reached failure under lower τ relative to shear tests in DIW. GCL-2 and GCL-3 specimens hydrated in Au-PS experienced lower shear deformation and did not fail under the maximum applied τ of 92.9 kPa. The maximum applied τ was limited to 92.9 kPa for comparison with experiments in DIW.

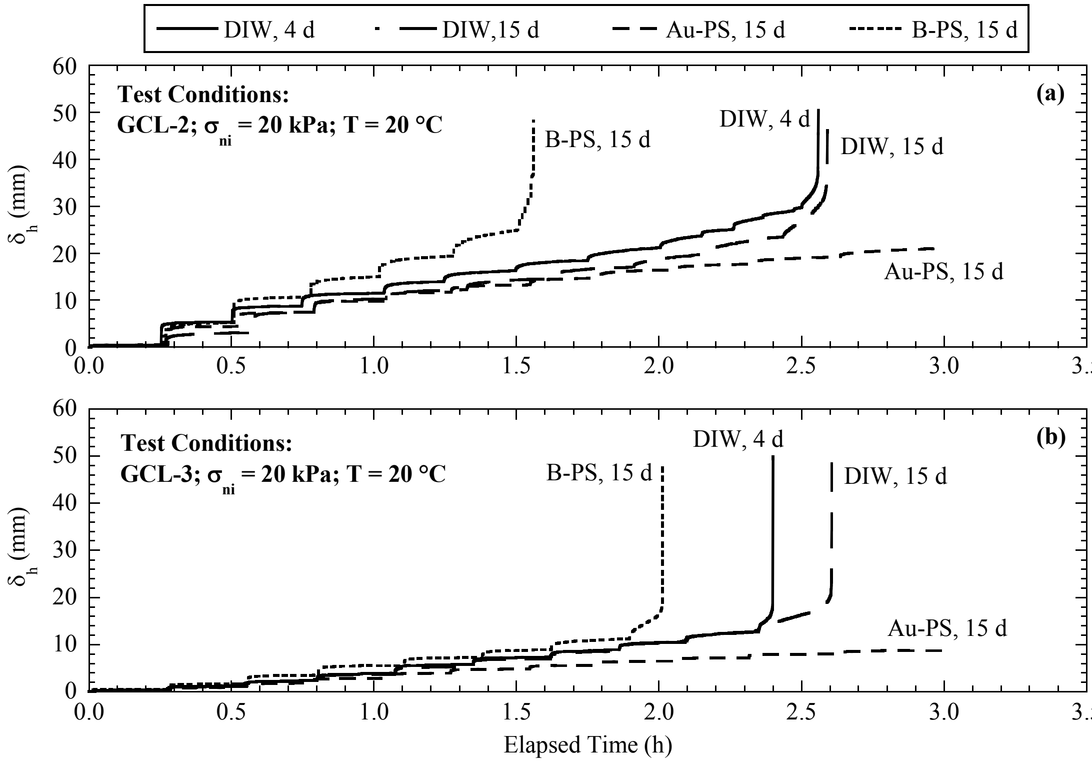

Figure 5. Relationship between horizontal deformation (δ_h) and elapsed time for (a) GCL-2 and (b) GCL-3 specimens hydrated for 4 d in de-ionized water (DIW) and 15 d in DIW, synthetic gold solution (Au-PS), and synthetic bauxite solution (B-PS). Experiments were performed with an initial normal stress (σ_{ni}) = 20 kPa at a temperature (T) of 20 °C.

The increase in the internal shear strength of GCL specimens hydrated with Au-PS was hypothesized to result from cation exchange of divalent cations for monovalent cations in the bentonite. Cation exchange of Ca^{2+} for Na^+ has been reported to increase the friction angle of hydrated bentonite (Mesri and Olson 1970). The hypothesis of cation exchange was evaluated via swell index tests (ASTM D5890) on bentonite extracted from GCL-2 specimens following the experiments. The swell index of bentonite decreased from 27 mL/2-g (non-exposed bentonite) to 13 mL/2-g after 15-d hydration in Au-PS. The lower swell index is evidence of cation exchange of divalent for monovalent cations that can lead to higher bentonite friction angles. Thus, the absence of failure of GCL-2 and GCL-3 hydrated in Au-PS for a comparable level of shear stress that generated failure in DIW can be attributed to cation exchange and potentially higher bentonite friction angles that increased internal shear resistance of the GCLs.

Swell index tests were also conducted on bentonite extracted from GCL specimens hydrated in B-PS following shear testing. The swell index of the B-PS specimens decreased from 27 mL/2-g (non-exposed bentonite) to 24 mL/2-g. This negligible change in swell index is supported by the chemistry of the B-PS, which had a high Na^+ concentration relative to divalent cations of Ca^{2+} and Mg^{2+}. This high Na^+ concentration likely prevented cation exchange in the bentonite during hydration in B-PS. The reduced internal shear strength of GCLs hydrated in B-PS was consistent with the displacement-controlled shear tests (Fig. 4). Additional research is ongoing to understand the mechanisms of strength reduction of NP GCLs following exposure to B-PS.

Relationships between the ratio of shear to total normal stress (τ/σ_{nt}) and horizontal deformation at the end of each shear load ($\delta_{h\text{-}EOL}$) for the RLS tests on GCL-2 and GCL-3 are shown in Fig. 6. The magnitude of $\delta_{h\text{-}EOL}$ for a given τ increment was larger in GCL-2 compared to GCL-3. This difference in $\delta_{h\text{-}EOL}$ was attributed to GCL heat treatment, whereby larger shear deformation developed from a combination of tensile elongation and disentanglement of reinforcement fibers in the non-heat-treated GCL (GCL-2) compared to tensile elongation of the reinforcement fibers in the heat-treated GCL (GCL-3). The shear behavior of GCL-2 and GCL-3 specimens hydrated for 4 d and 15 d in DIW were similar, which further supports the negligible influence of longer hydration times in DIW on internal shear strength of GCLs. As discussed previously, specimens hydrated with B-PS yielded reduced shear resistance, which is shown by the lower τ/σ_{nt} for GCL-2 and GCL-3. Furthermore, the specimens hydrated in Au-PS yielded the highest τ/σ_{nt} ratios and did not experience internal failure. Although numerous questions remain to be answered regarding the effects of non-standard solution hydration on internal GCL shear strength, similar shear behavior was observed in two GCLs that had different heat treatments, as well as in two different direct shear experiments on B-PS (i.e., displacement-controlled and stress-controlled). Thus, the observed effects of hydration in Au-PS and B-PS on internal shear strength indicate the importance of additional research to understand mechanisms contributing to shear behavior of GCLs in non-standard hydration solutions.

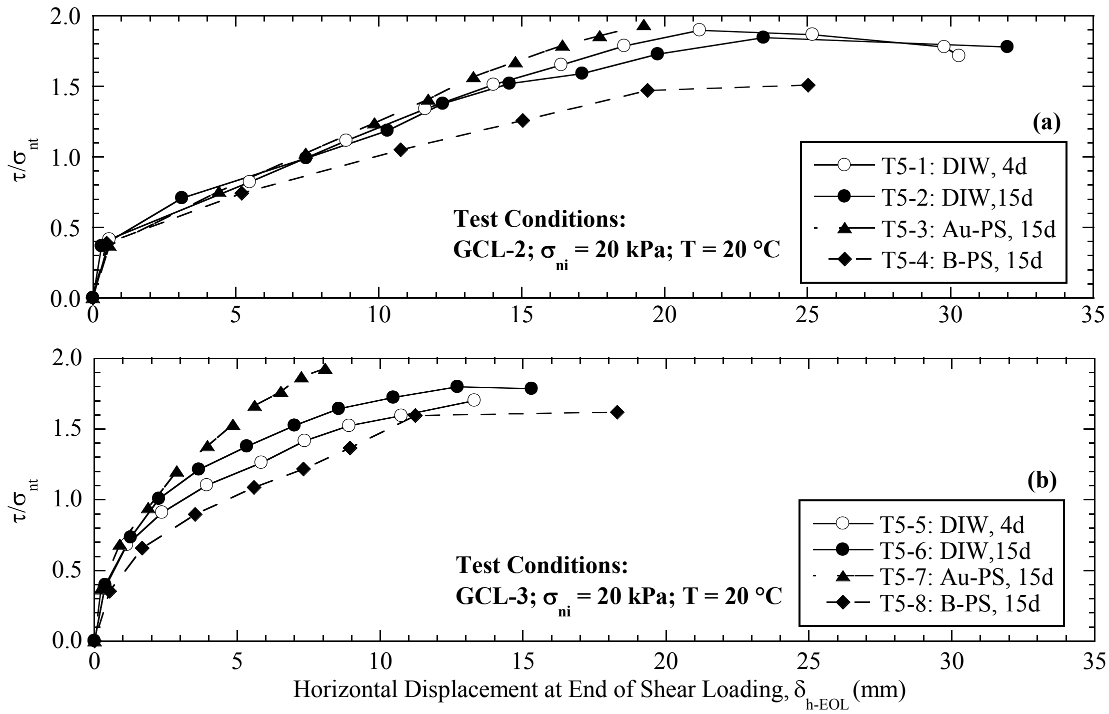

Figure 6. Relationship between the ratio of shear to normal stress (τ/σ_{nt}) and horizontal deformation at the end of shear loading ($\delta_{h\text{-}EOL}$) (a) GCL-2 and (b) GCL-3 specimens hydrated for 4 d in de-ionized water (DIW) and 15 d in DIW, synthetic gold solution (Au-PS), and synthetic bauxite solution (B-PS). Experiments were performed with an initial normal stress (σ_{ni}) = 20 kPa at a temperature (T) of 20 °C.

4 CONCLUSIONS

The use of GCLs in mining applications can expose GCLs to aggressive, non-standard chemical solutions. Exposure to these non-standard solutions can alter the chemical and mechanical properties of both the bentonite and geotextiles comprising a GCL. The hydraulic conductivity and internal shear strength of needle-punched GCLs were evaluated with varying hydration solutions. The following observations and conclusions were drawn from this study.

- Preferential flow through fiber bundles can occur in GCLs with higher degrees of needle punching under low effective stress conditions (e.g., 27.6 kPa, 4 psi). However, all high peel strength (GCL-1) and low peel strength (GCL-2) specimens that did not exhibit preferential flow yielded the same low *k* in the three non-standard chemical solutions (CW, Au-PS, and B-PS) as the low *k* measured with DIW and TW. Thus, all tests liquids were compatible with both GCLs if preferential flow was not present.
- Non-heat-treated NP GCLs hydrated in a highly-alkaline (pH > 12) synthetic bauxite mine process solution (B-PS) for 15, 30, and 60 d exhibited a decrease in shear strength compared to specimens hydrated in de-ionized water (DIW) for 4 d. The reduction in shear

strength for GCLs hydrated in B-PS was attributed to reduced frictional resistance at entanglement locations of the reinforcement fibers and bentonite-geotextile interface.

- Stress-controlled direct shear tests conducted on a non-heat-treated (GCL-2) and heat-treated (GCL-3) GCL following 15-d hydration in Au-PS showed increased shear resistance and did not fail, whereas those hydrated for 15-d in B-PS showed reduced strength relative to DIW. The increase in shear resistance for GCLs hydrated in Au-PS was attributed to cation in the bentonite, which was supported via swell index tests.
- The reduction in shear strength following hydration in B-PS was observed in displacement-controlled and stress-controlled direct shear experiments on GCL-2 as well as in both GCL-2 and GCL-3 in stress-controlled direct shear. Additional research is needed to evaluate mechanisms of internal shear strength reduction following hydration in a highly alkaline, synthetic bauxite mine process solution.

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Mine Closure

As Closure Approaches, Are you Preparing as Well as you Think You Are?

Andrew Baisley, Steven Pearce & Mike O'Kane
O'Kane Consultants Inc., Calgary, AB, Canada

ABSTRACT: To what extent will you encounter unrecognized and unfunded liability during closure, or when undertaking detailed closure planning leading up to closure? Unrecognized / unfunded liability generally results from a social license to operate and environmental issues surrounding water treatment costs in perpetuity. These community and environment issues result in the most substantial financial risk to the mining industry today, expected only to increase with time.

There is a general feeling community and environmental risks are being addressed because there has been a ground swell of discussion and action regarding material stewardship of waste material physical stability. Consequences of physical stability are perceived "…in my lifetime…" and are therefore heightened. However, materials risk also includes chemical stability during the entirety of the wastes life cycle which includes associated adverse impacts to water quality. This is not typically considered because these risks are perceived as a "…Not in my Lifetime…" issue. These risks are marginalized and poorly quantified due to a lack of appreciation for complexities of holistic mine closure and the timeframes associated with water quality issues. Although advancements waste characterization have occurred, there are a number of factors known to control ML-ARD and water quality risks; geochemistry forms only one of these risk factors.

Study outputs and operational requirements are disconnected, because a direct assessment of placement method such that risk and cost can be quantitatively assessed does not exist. Quantifying risk/cost trade-off from differing waste management approaches requires a broader technical assessment approach. O'Kane Consultants have developed an assessment process based on risk that captures multifaceted inputs and employs an analytical model to provide quantitative analysis and outputs. Now, these "…not in our lifetime…" risks can be addressed "…in our lifetime…" allowing cash flow and an operation to be optimized economically.

1 INTRODUCTION

1.1 *Asking a Question*

This paper asks the question: "As closure approaches, are you preparing as well as you think you are?" The question should result in a follow-up along the lines of: "What is the context and what is the metric against which we are measuring ourselves?" The focus of this paper is technical; however, to address the question posed in the title, as well as the proposed follow-up question, context is needed. In short, whether or not one is "preparing well enough", comes down to a question of risk acceptance. That is to say, to what extent will you encounter unrecognized and unfunded liability during closure, or when undertaking detailed closure planning leading up to closure? Typically this liability manifests as water quality issues a result of geochemical processes occurring within waste storage facilities.

MEND (2013) conducted a survey within the mining industry of mine drainage treatment and sludge management practices. The majority of sites surveyed reported the expectation to treat in perpetuity. As such, the mine's choice of treatment is critical not only for economic but also for environmental reasons. That is not to say that these sites are not economic, or weren't economic; rather, that because of unfunded and unrecognized liability during mine planning, which arose during closure planning, that these projects are not economically optimized. Clearly then, there is an economic impetus to addressing this potentially unfunded and unrecognized liability and economically optimize projects.

1.2 *An Economic Impetus for Improvement*

The premise in regards to an economic impetus for improvement is presented from the perspective of investment in the metals and mining industry. For the purposes of discussion within this paper, Morgan Stanley (2016) state that safety standards are improving but continue to focus on improvements in Total Recordable Injury Frequency (TRIF) with less account given to the severity of each incident. Furthermore, geographic location plays a role in safety statistics. However, there is a very telling adage provided by Morgan Stanley as: "…safe operations are usually productive operations…".

It is very appropriate, and highly encouraging, that Morgan Stanley's 2016 ESG also includes management of water as one of their four focusses on helping identify companies who are good stewards of existing mining assets. Morgan Stanley state that water availability is one of the two most important constraining factors for mining companies (power being the other). However, the authors of this paper herein contend that Morgan Stanley's approach for inclusion of water only, while being a substantial step forward, addresses only one facet of "water in the context of mine closure". In short, there is an important, and clearly fundamental focus within Morgan and Stanley's ESG approach in regards to water availability, consumption, recycling, etc.; however, there is no discussion whatsoever provided by Morgan Stanley in regards to water quality. In the context of mine closure planning, water quality is equally important as water quantity, and in many cases one could argue more important, as evidenced by the MEND mine effluent and water treatment study summarized above.

Although many mines provide attractive equity value, the authors of this paper herein would argue that this value is rarely optimized, due to an under appreciation for cash flows required for closure liability revolving around water quality. In short, it is the "Community and Environment" component of Health, Safety, Environment and Community that very often results in unfunded and unrecognized liability to a mine operation. Furthermore, communities and stakeholders are very aware of this facet of mining, and this can lead to lengthy delays in receiving a mine permit, higher closure bonding, lengthier times for receiving a closure bond back such that it can be distributed to shareholders, or quite possibly not receiving a permit at all.

The path forward then is to quantify this water quality risk appropriately, with the reality of "real closure" at-hand, and indeed quantify the benefits of reducing this water quality liability during operations, which is a time when the mining industry can quantify risk with much greater certainty.

The framework for moving down this path is grounded in understanding the need for alignment of mine planning and closure planning; early, and often.

2 A NEW PATH FORWARD FOR THE MINING INDUSTRY

2.1 *Alignment of Mine Planning and Closure Planning*

To address the above (i.e. encountering unfunded and unrecognized liability at closure), the mining industry is moving towards an appreciation that mine planning and closure planning should be aligned. And indeed, in order to optimize economics of a project, that this alignment must occur early, and often, during mine planning so that there is an appreciation of the need for community and stakeholder engagement, establishing closure objectives, to communicate using risk as a communication tool, and a wide variety of other closure planning facets, such as closure time frames, which should be part of mine planning.

Mine closure planning practitioners often struggle communicating with convincing mine planners that there might be value, or economic benefit, in, for example, building a waste rock dump (WRD) differently; this is because the mine planner is very often answering a "different question". For example, a mine planner is typically tasked with optimizing ore grade, extraction, blending, etc. for the lowest cost possible; and time frames are typically life-of-mine (LOM). In contrast, a mine closure planning practitioner does not function on a LOM time scale; rather, it is a mine-life cycle time frame, where the realities of closure planning are often brought forth by communities and stakeholders that are highly knowledgeable and possessing an understanding for longer term closure liability
(i.e. longer that LOM).

The key then is to change the question such that mine planners and closure planners are working within the same time frame, such that it is not a question of, for example: "optimize development of the open pit to build the lowest cost LOM waste rock dump…OR…management of mine waste water effluent"; but rather: "optimize construction of the LOM waste rock dump…AND…minimize risk of long-term water quality resulting in adverse risk to the environment". The "OR" question drives us to be compliant and efficient; however, the "AND" question drives us to be effective and sustainable. It is the latter "question" that communities and stakeholders are looking for from the mining industry such that the site can be considered to be a good steward of the mine asset.

Appropriately addressing this "New Question", from a technical perspective, is not a trivial challenge. However, the following presents a quantitative assessment approach to addressing this new question. The assessment approach has been developed by the authors of this paper herein, such that the benefits of aligning mine planning and closure planning when designing a WRD can be understood, quantitatively.

2.2 *A Quantitative and Integrative Approach to Linking Waste Rock Dump Design to Seepage Geochemistry*

The significant "message" of the methodology presented herein is not so much that the method for placing mine waste rock has a significant control on metal leaching and acid rock drainage (ML-ARD) risk. This is known and accepted in the mining industry; rather, the key message of the methodology is that this risk can now be assessed on a quantitative basis. The fact that the relative difference can be quantified is fundamentally important to appreciate because this means that the risk/cost trade-off assessment of how WRDs are constructed can be completed at the mine planning stage, and mine closure risk and cost estimation can be improved significantly. In this way, the objective of achieving alignment of mine planning with closure planning, early on within the mine-life-cycle, and indeed during LOM, can be realized. And most importantly, realized in a quantitative manner.

The secondary implication of this finding is that management of reactive waste during construction as a result of placement technique is likely a key risk driver for future ML-ARD release, due to challenges with controlling gas flux in periods when WRDs are be constructed and "open to the atmosphere". This conclusion is perhaps contrary to widely held views that ML-ARD risks can be largely managed at closure. It is clear that an assessment of WRD construction requires consideration when final closure solutions, such as cover systems and mine effluent collection and treatment, are being selected and relied on as the main closure mitigation solution for ML-ARD management.

2.3 *Mine Waste Rock Management Assessment Tool*

Characterization and assessment of waste rock forms the initial stage of waste management strategy planning. Prior to decision making, waste management strategies should evaluate both costs of waste management and risks associated with exposure, stockpiling and placement of waste materials, such as spontaneous combustion, toxic gas production, and ML-ARD. These risks are complex, are all interrelated, and are associated with air and water entry into the waste material where subsequent oxidation reactions occur (Lottermoser, 2010).

It has become common practice in the industry as part of waste characterization to classify material in a deterministic manner on the basis of primarily geochemical risk factors and to define material types, for example, as potentially acid generating (PAG). Material that is determined to pose significant risks of ML-ARD such as PAG is then prescribed a specific management method such as "encapsulation" as part of a placement strategy to reduce potential ML-ARD risks. However this method of assessment is prescriptive and polarized as materials are categorized into a few catch all categories, such as PAG, which in turn results in polarized decisions, such as all PAG must be managed in a set manner.

In the field, ML-ARD risks are known to be complex and interrelated and are strongly related to the structure of the WRD and how this influences oxygen ingress and water flow into the WRD, where subsequent oxidation reactions can occur. The influence of airflow, water flow and storage, and the site specific diurnal or seasonal variations in these are likely to be key risk drivers.

General factors that are known to control ML-ARD risks are detailed in Pearce et al (2015a) and include:

- Geometry of WRD, including footprint, height, slope area;
- Sulfide and carbon content of material;
- Physical properties of material (grain size and distribution, saturation, weathering rate);
- Geochemical properties of material (intrinsic oxidation rates, carbonate dissolution rates, kinetically controlled metal/nonmetal release rate)
- Timing of waste placement and any closure mitigation engineering solution (such as a cover system and/or mine effluent collection and treatment);
- Structure of WRD due to placement (pathways for air and water movement); and
- Climate (temperature and rainfall).

Geochemistry forms only one of these risk factors, however it is important to note that typical industry approach to ML-ARD assessments is for the study to be based for the most part on laboratory testing related to geochemical properties only. A simple summary of this observation is to state that although the characterization of materials is important, the method and timing of placement, and the site environment in which they are placed are perhaps more important variables; and are often disregarded in the typical industry approach for assessing ML-ARD risk.

To evaluate risk based on multifaceted variables requires application of semi quantitative analysis. The authors of this paper herein have developed an assessment process based around a risk matrix that captures these multifaceted inputs and employs an analytical approach to provide semi quantitative analysis and outputs. This method of assessment allows risk to be assessed on the basis of placement technique and not just on material geochemical properties in isolation. A full description of this method is outside the scope of this paper and is described in detail in Pearce et al (2015a). In summary the assessment process is based around a quantitative risk assessment tool that utilizes a series of complex algorithms to "model" how waste materials will react to placement in a given scenario. Outputs from the assessment tool are then collated into a risk matrix that captures these multifaceted inputs. This method of assessment allows risk to be assessed on the basis of placement technique, and incorporation of closure mitigation solutions, and not just on material properties in isolation.

The assessment tool developed evaluates convective gas transport, intrinsic oxidation rate (pyrite and carbon) spontaneous combustion, seepage, carbonate dissolution rate and acidity generation.

2.4 *Waste Rock Dump Construction and Links to ML-ARD Seepage Risk*

Construction of WRDs generally includes one, or some combination of the following methods: end-tipping, paddock dumping, push-dumping, or encapsulation. The specific method used on a given site to construct the WRD is generally based on availability of equipment, cost and the scale of construction; hence, construction methods are far from uniform across all sites.

Aspects of WRD construction that relate to ML-ARD risk, which unfortunately are typically overlooked when developing an understanding for these risks, include the internal structure created as a consequence of the prevalence of end tipping material, and the resulting hydrogeological characteristics which control air (oxygen) and water flux throughout the waste material. Given that oxygen and water flux are major controls in the production and release of ML-ARD to the receiving environment, WRD construction is clearly a very significant variable to factor into ML-ARD risk.

The waste rock placement strategy to address ML-ARD risk focuses on minimizing oxidation of sulfide minerals during waste placement. This results in minimizing stored oxidation products (or more appropriately, stored acidity), and thus long-term reliance on a cover system and/or mine water effluent collection and treatment as the "sole" means of managing seepage from a WRD. Minimizing oxidation of sulfide minerals involves strategic placement of run-of-mine (ROM) waste such that advective gas transport within the WRD (i.e. oxygen transport) is limited because airflow capacity (air permeability) is controlled. The primary airflow mechanism being addressed by utilizing this strategy is convection, which results from a temperature differential within, and external, to the WRD.

Diffusion of gas (oxygen), as well as any 'residual' oxygen availability to sulfide minerals due to advection, can then be used to determine sulfide oxidation rates during waste placement, and following cover system construction.

This management strategy also allows operators to manage highly reactive materials being placed in a WRD and the potential for elevated temperatures (i.e. spontaneous combustion).

3 CASE STUDY

3.1 *Waste Rock Management*

A gold and silver mine in the Province of North Sumatra includes a primary WRSF area that is integrated into and being developed as the main downstream containment structure of the TSF. The WRSF will include all mine waste, including potentially acid forming (PAF) waste rock, from open pits. There are currently two active pits, and over time additional pits are expected to be brought on line as exploration and resource development activities are progressed.

A detailed waste rock management plan has been developed by OKC and mine technical teams (mine geology, exploration, mine planning, TSF) for the site. The plan provides technical guidance for specific aspects of waste rock management during the development and operational phases, and an overall framework for the management of waste materials during the construction of the TSF.

The operator has developed a significant materials characterisation database through geochemical characterisation of the waste rock. Several sources of geochemical data was available to develop the risk-based waste rock classification process flow methodology for operational use in characterising blocks of waste rock.

Waste classification and subsequent modelling into discrete class system in the reserve model and schedule have been designed to take into account the broad characteristics of the deposit and translate this into a means of identifying material based on predicted AMD risk, potential utility for use in construction (soft materials are more amenable to compaction for example), and potential acidity buffering potential (presence of carbonates such as calcite).

A detailed mine waste schedule has been developed for the site based on the waste classification system and identifies the sources of materials over LOM to ensure that the build plan can meet the design specification. It is therefore important that the mine schedule and LOM build plan reference each other to ensure the successful management of waste requiring management as it exits the pit. Scheduling is important to be carried out over LOM because design specification indicates that construction of sealing layers require a source of low risk (with respect to AMD) finer textured lower risk materials, and criteria for compaction is based on particle sizing.

3.2 *The Progressive Encapsulation Method and Oxygen Ingress Assessment*

The conceptual model for assessing AMD risk assumes that oxygen availability to PAF waste rock within the WRSF is dominated by diffusion rather than advection; a result of the site's high rainfall and associated infiltration rates, and the placement of fine textured waste on the em bankment outer slopes. This creates and maintains an "engineered tension saturated layer" across the surface of the embankment which limits oxygen ingress and thus AMD production and release.

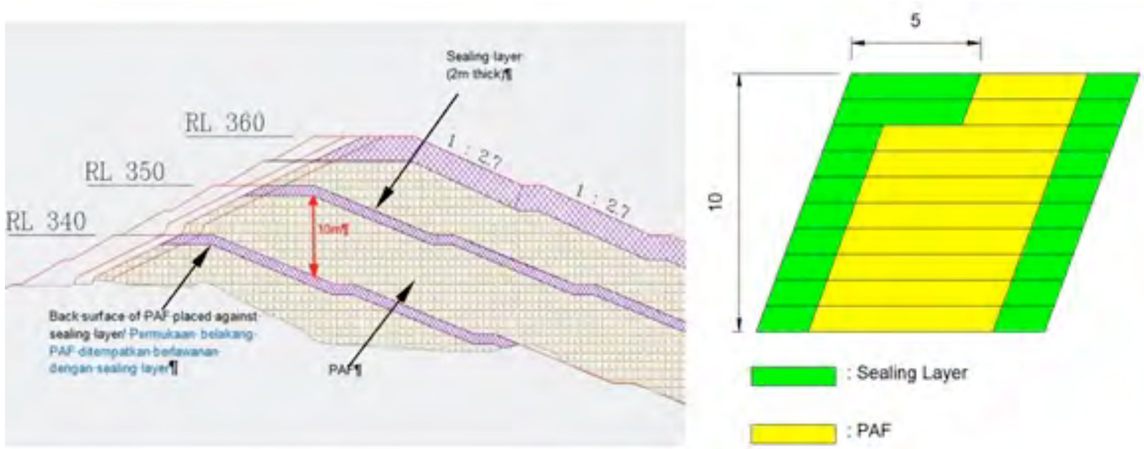

Figure 1 PEM design used as basis for modelling assessment

Figure 1 shows the engineering concept, with PAF material being progressively encapsulated during construction on a lift by lift basis, where lifts constructed as part of the embankment raise are 10m in height, and waste is placed in 1m thick compacted layers.

OKC completed a detailed modelling study of advective airflow within the WRSF LOM design using numerical modelling tools coupled within the GeoStudio (Geo-Slope International, 2012) software suite: TEMP/W, AIR/W, and SEEP/W (Pearce 2016). Surface infiltration seepage rates were calculated using VADOSE/W. The objective of the numerical modelling program was to develop guidelines for waste placement.

Key conclusions from the airflow modelling work were:

- Advective airflow rates are substantially lower than diffusion rates as a result of the WRSF "wetting up". As long as the material maintains sufficient saturation, advection will not contribute a significant source of oxygen for oxidation.
- Oxygen ingress due to thermal convection cells is anticipated to be low, even with elevated internal WRSF temperature and low degree of saturation conditions (worst case scenario).
- The placement of high grade sulfide sulfur near the outlying slopes of the landform should be minimised.
- Oxygen ingress was shown to be substantially decreased by the presence of the sealing layers. Oxygen ingress varied greatly for the material depending on the texture of the material, its water retention characteristics and the assumed in-place dry density.
- With increased as-placed waste density a decrease in oxygen ingress results. Additional compactive effort produces increased density leading to decreased porosity, increased air entry value and water retention, and a decreased hydraulic conductivity. All leading to an increase in the degree of saturation of the encapsulation system materials and decreased oxygen ingress rates.

Results from the numerical modelling program were used to inform waste material placement guidelines encompassing the range of potential waste and operational cover system materials for the interior of the WRSF. Material envelopes were developed based on particle size, and included the geotechnical specifications required for the as-constructed sealing layer to reach the specific targets for oxygen ingress.

3.3 *WRSF Concept Validation*

PT Agincourt initiated a program to validate the WRSF engineering design concept in 2015, with the objective of confirming that sulfide oxidation within the WRSF embankment is being reduced due to the implementation of the sealing layer concept. Validation work to date has consisted of OKC designing and installing monitoring systems within the sealing layer profile at two locations monitoring temperature gradients, oxygen concentration, pore-water pressure, volumetric water content, and matric potential.

Preliminary monitoring data indicates that the sealing layer is performing akin to the conceptual model and as per modelling predictions. VWC and suction data are indicative of material that is maintaining a high degree of saturation. Calculated saturations are high in that they remain above approximately 80%. Oxygen concentrations within and below the sealing layer are reduced to near zero (Figure 2).

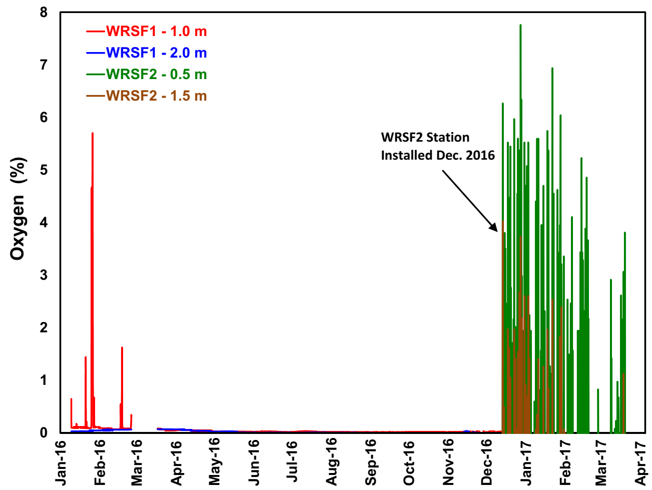

Figure 2 Oxygen concentration within WRSF sealing layer.

Material characteristics and monitoring data collected during the 2016 monitoring period, were utilised to develop estimates of a range of air permeability (k_{air}) for the field for the various material envelopes at the Martabe site under a range of dry density values (Fig. 3), which is an important facet in understanding the potential air permeability. In situ dry density significantly affects the achieved field k_{air}, as illustrated in Figure 5. The estimates in k_{air} shows that material can have up to an order of magnitude decrease with increases in compaction effort.

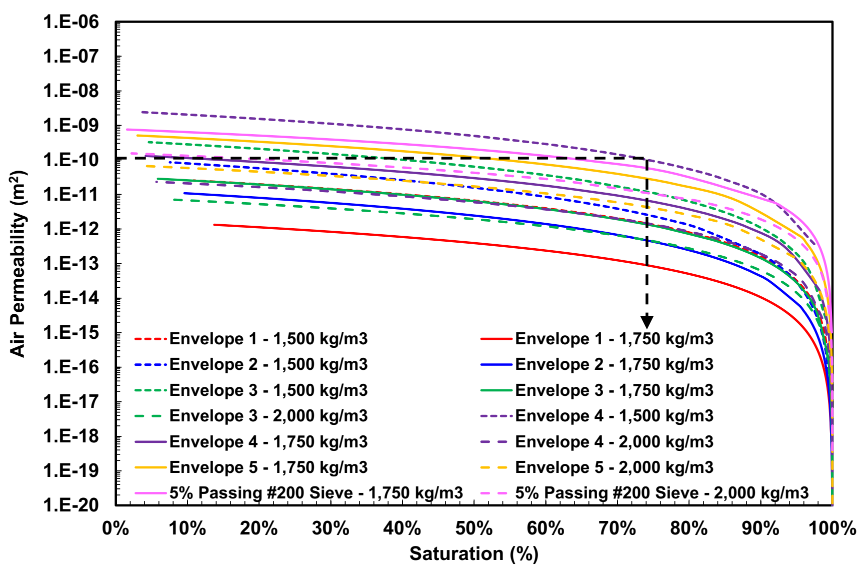

Figure 3 Air permeability of Martabe material at compaction specification envelopes.

When saturation levels are maintained above 75%, air permeability is lower than 1 × 10-10 m2 which limits the dominant oxygen ingress mechanism to diffusion irrespective of the envelope the material falls in. Figure 3 demonstrates there is inherent flexibility in the construction of the WRSF sealing layers given that if sufficient fine grained material is not available, oxygen ingress targets can still be achieved through application of increased compaction effort.

4 SUMMARY

This paper presents a technical framework, for LOM development of a WRD whereby the waste placement methodology is viewed as being as important, if not more important, to closure performance, as compared to a final (closure) cover system and mine effluent collection and treatment. Creating a physical environment within the WRD that addresses the risks presented by the reactive waste material, represents a fundamental shift in the typical approach to managing reactive mine waste. This framework was successfully utilized for the case study site to mitigate AMD risk. It is clear that an assessment of WRD construction requires consideration when final closure solutions such as cover systems or mine effluent collection and treatment are being selected and relied upon as the main closure mitigation solution for ML-ARD management.

It is notable that to date the strategy has proven to be compatible with the Martabe mine plan, logistically feasible and cost effective. The strategy has been validated to date by detailed in situ monitoring studies, with close alignment between modelled and measured performance.

With the technical framework presented here, the industry can now quantify the relative difference in waste placement techniques. Our technical framework represents a substantial advancement in mine waste rock management. The advancement our framework produces is due to the risk/cost trade-off assessment of how WRDs are constructed, how that can be completed at the mine planning stage, and the improvement to mine closure risk and cost estimation. In summary, this paper presents an economic discussion, as well as a quantitative technical framework that allows mine operators to be good stewards of mining assets by closely aligning mine planning and closure planning early on within the mine-life-cycle, and during LOM.

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Constructing a Cover Performance Test Section on a Uranium Mill Tailings Management Cell

Melanie M. Davis
Stantec Consulting Services Inc., Fort Collins, CO, USA

Craig H. Benson
School of Engineering, University of Virginia, VA, USA

Harold R. Roberts
Energy Fuels Resources (USA) Inc.

ABSTRACT: Reclamation of an inactive tailings management cell was initiated in 2016 at the White Mesa Uranium Mill facility located in Utah, USA. Reclamation activities included design and construction of a cover performance test section within the cover for the cell. The cover system is a monolithic water balance cover designed to minimize percolation, meet the radon emanation standard, minimize maintenance over the short and long term, and promote sustainability. The test section was based on the design developed by United States Environmental Protection Agency's Alternative Cover Assessment Program (ACAP) with a large-scale drainage lysimeter to evaluate all components of the water balance. The test section is evaluating the entire cover profile proposed for reclamation. This paper summarizes design and construction of the test section, provides a brief background on reclamation cover design development and agency interaction, and presents data collected through calendar year 2016. Test section construction included building the test section, installing instrumentation, and seeding the top surface. Field samples were collected for laboratory testing to compare with cover design properties. Monitoring equipment was calibrated in the field and laboratory. Monitoring data include water balance and meteorological data collected remotely. Acceptable cover performance will be demonstrated via performance monitoring over approximately 7 years.

1 INTRODUCTION

A test section was constructed on an inactive tailings management cell at the Energy Fuels Resources (USA) Inc. White Mesa Uranium Mill located in southeastern Utah, USA. EFRI facilities at the site consist of a uranium processing mill and five lined tailings/process solution evaporation cells located south of the Mill within an approximately 156-ha restricted area.

The test section was constructed as a design-build project during reclamation cover construction using procedures adopted from the test section installation instructions developed by the United States Environmental Protection Agency (EPA) Alternative Cover Assessment Program (ACAP) (Benson et al., 1999). The test section will be monitored to assess the performance of the reclamation cover system for the tailings management cells.

This paper presents an overview of the design and construction of the test section, background on the site history and regulatory framework, a summary of the reclamation cover design, and discussion of the current monitoring.

2 BACKGROUND

The Mill was developed in the late 1970s and has a capacity of approximately 1,800 dry metric tonnes per day. The Mill has been operating since 1980 on a campaign basis, with operations placed on standby during various periods over the last 37 years as market conditions warrant.

Commencing in the early 1990s through today, the Mill has processed alternate feed materials when the Mill has not been processing conventional ores. Alternate feed materials are uranium-bearing materials other than conventionally mined uranium ores. EFRI installed an alternate feed circuit in 2009 that allows the Mill to process certain alternate feed materials simultaneously with conventional ores. The Mill has processed over five million metric tonnes of ore and alternate feeds since inception.

Construction of the five tailings management/process solution cells began in 1978 with the most recent cell constructed in 2010. Two cells are being used for evaporation of process solutions (Cell 1 and 4B), one cell is inactive (Cell 2), one cell has been used for tailings storage, but is currently only receiving mill waste and is partially reclaimed (Cell 3), and one cell is currently receiving tailings and process solutions (Cell 4A). The top surface area for the cells range from approximately 16 to 28 hectares.

The site is regulated by the Utah Department of Environmental Quality (UDEQ). The cover for the tailings management cells has been designed in accordance with applicable Utah regulations and is included in the reclamation plan EFRI submitted to UDEQ in 2016 (EFRI, 2016). EFRI and UDEQ developed an agreement in 2016 to define the commitments and schedule for completing placement of reclamation cover on Cell 2 and performance of the cover system in accordance with the 2016 reclamation plan. Both parties agreed that the Cell 2 cover would be placed in two stages (Phase 1 and Phase 2), with the first stage constructed in 2016 along with the test section. Performance of the cover will be evaluated between Phases 1 and 2 via a 7-year performance monitoring program that follows principles in NUREG/CR-7028 (Benson et al. 2011).

Placement of cover on Cell 2 for Phase 1 commenced in April 2016 and was completed in April 2017. The test section was constructed within Cell 2 in August-September 2016 concurrently with placement of reclamation cover. Phase 2 construction will commence after a seven-year monitoring period and approval of the test section monitoring results by UDEQ.

3 COVER DESIGN

The cover system for reclamation of the tailings cells at the site is designed as a monolithic evapotranspiration (ET) cover. A monolithic ET cover is the preferred design in this semi-arid climate to minimize percolation, meet the radon emanation standard, minimize maintenance, and promote sustainability. The proposed ET cover has been designed with sufficient thickness to protect against frost penetration, attenuate radon flux, minimize plant root and burrowing animal intrusion, and provide adequate water storage capacity to control percolation into the underlying tailings. The cover is designed to be stable under both static and seismic conditions, and to provide tailings isolation under long-term wind and water erosion conditions.

Thickness of the reclamation cover ranges from 2.9 to 3.2 meters (m) depending on location. The minimum design cover thickness of 2.9 m was used for the lysimeter area of the test section to assess the lower bound thickness. Cover placed outside the lysimeter and within the perimeter of the test section was constructed to the full-depth cover profile for Cell 2 (3.2 m). The cover profile within the lysimeter is shown on Figure 1. The cover layers within the lysimeter are listed below from top to bottom:

Layer 4 – 0.15 m thick Erosion Protection Layer (topsoil-gravel admixture)
Layer 3 – 1.07 m thick Growth Medium Layer (loam to sandy clay)
Layer 2 – 0.91 m thick Compacted Cover (highly compacted loam to sandy clay)
Layer 1 – 0.76 m thick (minimum) Interim Fill Layer (loam to sandy clay)

Layer 1 was placed in stages as interim cover on Cell 2 from 1991 through 2008. Starting in April 2016, Phase 1 cover construction included placement of: (1) additional interim cover to achieve design grades prior to placement of cover Layer 2 and (2) the entirety of Layer 2. Layers 3 and 4 will be constructed during Phase 2.

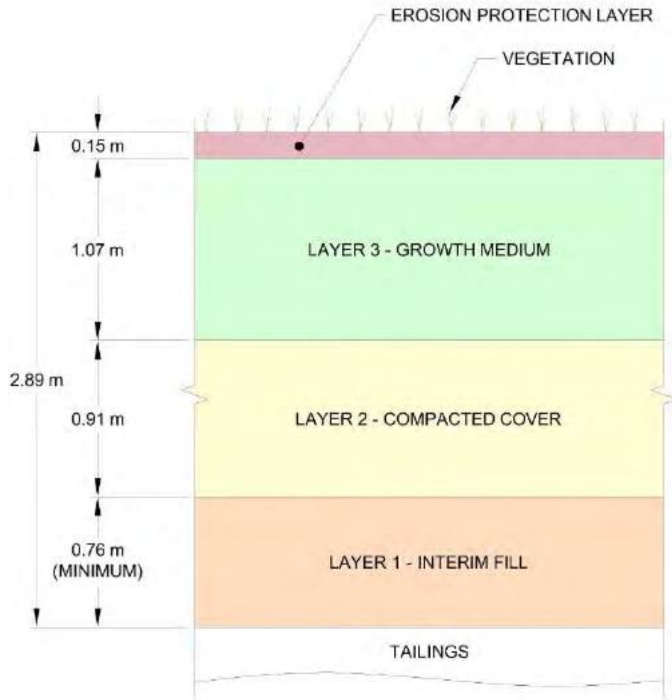


Figure 1. Cover profile within test section lysimeter

4 TEST SECTION DESIGN

The design of the test section is adopted from the ACAP installation instructions described in Benson et al., (1999) and incorporates performance monitoring recommendations provided in NUREG/CR-7028 (Benson et al., 2011). The test section includes a large ACAP-style drainage lysimeter that provides direct measurement of all water balance components except evapotranspiration (ET), which is computed by closing the water balance (Apiwantragoon et al. 2014).

The lysimeter includes the following components:

- Geomembrane-lined (LLDPE) base and vertical side slopes
- Geocomposite drainage layer draining percolation to a collection sump above the LLDPE base
- Geosynthetic root barrier layer above Layer 1 (interim fill)
- Collection sump and pipes for lateral flow at the interface of Layer 3 (growth medium layer) and Layer 2 (compacted layer)
- Surface run-off collection berm that diverts surface run-on and directs run-off to a single collection point
- PVC drainage pipes for percolation, lateral flow, and surface run-off that drain to separate measurement stations

The test section is approximately 30 m x 30 m, with the 10 m x 20 m drainage lysimeter centered within the test section (Figures 2, 3). The longer side of the lysimeter is oriented parallel to the cover slope. The buffer area outside the lysimeter minimizes hydrological and thermal boundary effects and will be used for periodic destructive sampling of soils and biota as needed. These attributes are the same as those recommended in NUREG/CR-7028 (Benson et al., 2011).

The lysimeter collects percolation from the base of the cover, surface runoff, and interflow from the textural interface between the interim fill (Layer 1) and compacted cover (Layer 2). Sensors monitor hydrologic state variables (temperature and water content) within the cover. Percolation rate, lateral drainage, runoff, internal state conditions, and meteorological data are recorded continuously using a datalogger located near the southern edge of the test section.

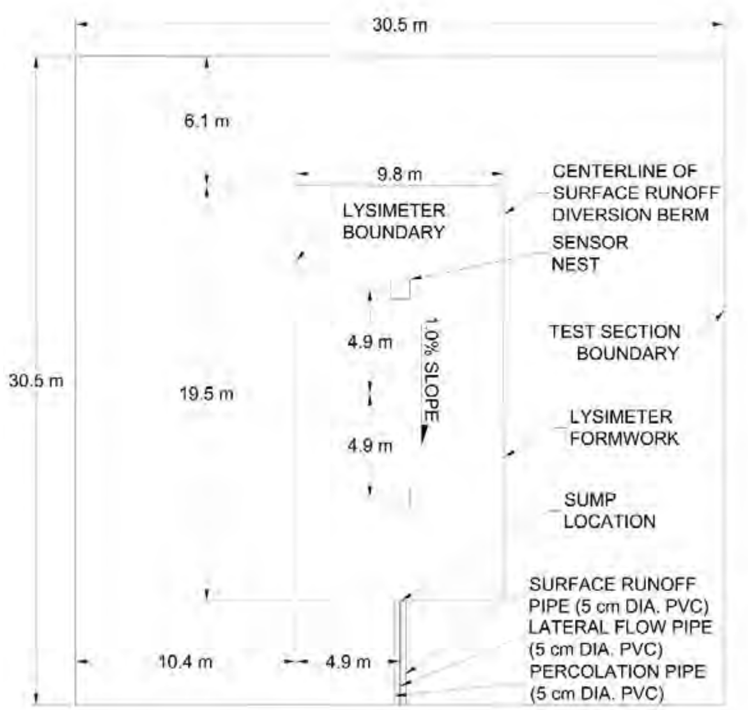
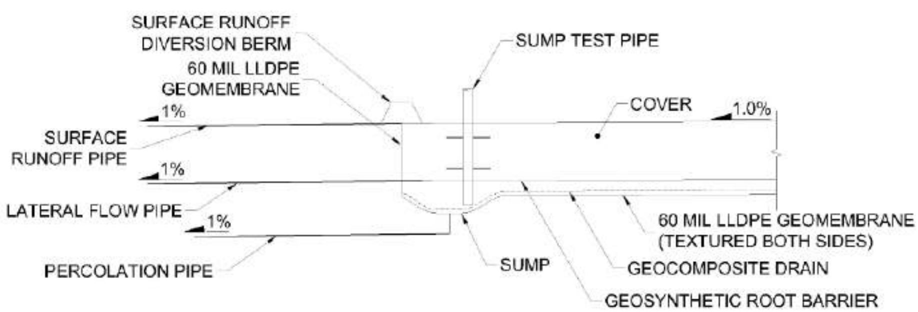


Figure 2. Plan view of the test section.



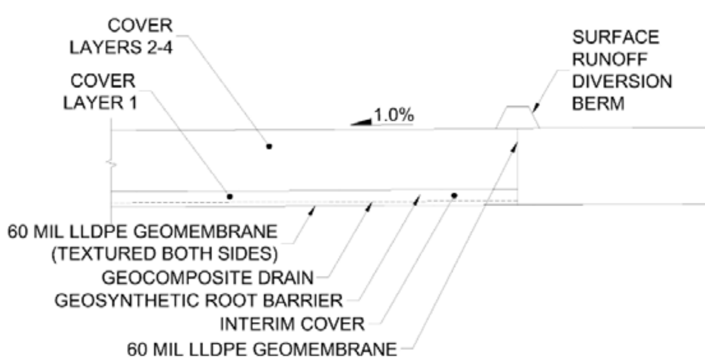


Figure 3. Cross sections for downslope (upper) and upslope (lower) ends of lysimeter.

5 CONSTRUCTION SUMMARY

The test section was constructed in August and September 2016. Preparation of the subgrade for the test section consisted of regrading the existing interim cover (Layer 1) to meet design grades and then compacting the top surface. Adjustments were made as needed so that the surface met the smoothness requirements for installation of the lysimeter geomembrane.

Formwork for the lysimeter was placed in two stages, both 1.2 m high. Formwork for each stage was constructed on site using 13-mm-thick plywood panels (1.2 m x 2.4 m) stiffened with 50 x 100 mm studs (Figure 4). The second stage of formwork was installed when the cover was within 300 mm of the top of the first stage of the lysimeter construction. The downslope end wall and both side walls were installed at the start of construction and before soil placement began. The upslope end wall was installed after the soil and geosynthetic layers were placed in the lysimeter.

Figure 4. Lysimeter formwork installed for first stage and prior to deploying the geomembrane.

The geomembrane to form the lysimeter (1.5-mm linear low-density polyethylene, LLDPE, geomembrane) was deployed in two stages. For the first stage, two panels were placed, with the overlap oriented parallel to the slope. The overlap, as well as the seams in the corners and at the sump were welded with an extrusion welder and checked using a vacuum box per ASTM D5641 (ASTM, 2011). The sump for collecting percolation was fabricated off-site, installed during construction, and welded to the lysimeter geomembrane with an extrusion weld. The sump pipe was installed in a narrow trench in the subgrade with a pipe extending to a collection basin. A sump test pipe and riser were installed to provide a means to test the sump, if needed.

The geocomposite drainage layer (GDL) in the base of the lysimeter (5 mm geonet with 340 g/m^2 nonwoven geotextiles heat-bonded to both sides) was deployed from the top of slope directly over the geomembrane in three panels, with the middle panel overlying both exterior panels (Figure 5). The upper geotextile was heat-bonded at the overlaps and around the periphery. Interim fill was placed over the GDL in the sump area immediately after placement to provide a surcharge to prevent thermally-induced movement.

Soil placement extended a minimum of 3 m upslope from the upper end wall during the first stage of construction following the same specifications employed for construction of the full-scale reclamation cover. Before placement of soil began, protective plywood panels were placed over the geomembrane extending upward from the end of the lysimeter (Figure 5). This section of geomembrane was later folded up to create the upslope end wall after the soil and geosynthetic layers were placed in the lysimeter.

After the cover profile was constructed within the lysimeter, the upslope end of the lysimeter was closed (see Figure 6). The corners were welded with an extrusion welder (Figure 6) and checked using a vacuum box per ASTM D5641 (ASTM, 2011).


Figure 5. Geosynthetic drainage layer after placement during first stage of construction. Plywood panels were placed to protect the geomembrane upslope of the lysimeter during placement of the cover soils.


Figure 6. Folding up and welding geomembrane on the upslope end of the lysimeter after placement of the cover soils during the first stage. A technician is extrusion welding the northwest corner (right side of photograph).

The interim cover soil (Layer 1, 0.76-m thick) was placed on top of the GDL as a working platform so that the construction equipment would not contact or displace the geosynthetics. The interim cover soil was placed and compacted using methods expected for full-scale construction, except low pressure equipment was used for the first lift.

A root barrier layer (Biobarrier, Fiberweb, Inc.) to prevent root penetration into the GDL and other components of the monitoring system was placed directly on top of the interim cover following the manufacturer's instructions (Figure 7). The polyethylene nodules on the root barrier were oriented upward and there was no contact between the root barrier and the GDL or bottom geomembrane in the lysimeter.

Figure 7. Deploying root barrier (yellow layer) over the interim layer.

The remaining cover layers (Layers 2-4) were placed and compacted following the technical specifications both inside and outside the lysimeter. After compaction, a fillet of granular bentonite was placed around the periphery of each layer to prevent sidewall flow between the sidewall geomembrane and the cover soil. Composted biosolids were mixed into the upper 150-mm of Layer 3 (growth medium layer) and the surface was seeded with the seed mix planned for the full-scale reclamation cover.

Construction quality assurance testing included nuclear density and water content testing for each cover lift. Standard Proctor compaction and gradation testing were conducted on the borrow stockpiles prior to construction. Undisturbed block and disturbed samples were also collected from each cover lift within the lysimeter to measure index and hydraulic properties. Collection of a block sample is shown in Figure 8.

Figure 8. Collecting hand-carved block sample from interim cover layer (left) and block sample in wooden shipping frame for transport to the laboratory (right).

Interflow along the contact between the interim fill (Layer 1) and compacted cover (Layer 2) is collected using a sump constructed across the breadth of the downslope endwall. Surface runoff diversion and collection berms were constructed along the periphery of the lysimeter (Figure 9). A 460-mm long strip drain to collect runoff was placed along the centerline of the test section, 0.5-m downslope of the end wall, and adjacent to the diversion berm on the downslope end of the lysimeter. Flows from the interflow sump and the strip drain are routed in separate PVC pipes to collection basins.

Figure 9. Final surface of test section (looking north) with diversion and collection berms for runoff.

6 INSTRUMENTATION MONITORING

Instruments were installed to meter flows from the test section (runoff, lateral flow, percolation), state variables within the cover profile (water content and temperature), and meteorological conditions at the surface.

Flows from the lateral flow and percolation sumps and the surface runoff collection points are metered in basins (Figure 10) located in a concrete vault (Figure 11) placed below grade and downslope of the lysimeter. The vault contains three basins, and each basin contains a flout that drains the basin at the high-water point. PVC piping was used to connect the percolation and lateral flow collection sumps and the surface runoff collection points to the basins. Each basin is equipped with a tipping bucket and a pressure transducer.

Figure 10. Cylindrical collection basins inside vault for monitoring flows.

Figure 11. Excavation and placement of vault for monitoring basins.

Water content reflectometers (WCRs, CS 616, Campbell Scientific Inc.) were installed for measuring soil water content and thermocouples (Type T, Omega, Inc.) were installed for measuring soil temperature. The thermocouples were soldered in the field, coated with a thin film of flexible polyurethane, and taped to the head of the WCR as suggested in Benson and Wang (2006) (Figure 12). The WCR and thermocouple were then installed together.

Figure 12. Water content reflectometer with thermocouple taped to head and ready to be deployed.

WCRs with thermocouples were installed in two nests with seven monitoring points per nest. The sensors were installed after the layer had been placed in a small hole excavated by hand that was slightly larger than the WCR. The sensor was then covered with soil and the hole backfilled and compacted by hand. Cables from the WCRs were routed along with the thermocouple wires in 25-mm PVC electrical conduit to a riser near the edge of the lysimeter.

A meteorological station was installed south of the lysimeter adjacent to the vault (Figure 13). The weather station consists of a Model 05103 Wind Monitor (RM Young Inc.) for wind speed and direction, an SP-100 pyranometer (Apogee Inc.) for solar radiation, a PTB110 barometer (Vaisala Inc.) for barometric pressure, a HMP60 Temperature and RH sensor with radiation shield (Vaisala Inc.) for air temperature and relative humidity, and a T-200B precipitation gauge with shield (Geonor Inc.) to measure unfrozen and frozen precipitation.

Data from the sensors on site is collected by a CR1000 datalogger (Campbell Scientific Inc.). The datalogger is powered by a 12V charging regulator (Campbell Scientific Inc.) connected to a 115-V electrical service. A 12-V gel-cell battery provides back up power. The battery is continuously charged by the charging regulator. A 50-W solar panel provides backup charging capabilities for the battery should the 115 V electrical service be unavailable for an extended period. Data stored by the datalogger are transmitted via cellular modem to a server off site.

Figure 13. Weather station located adjacent to vault and south of lysimeter.

7 INSTRUMENTATION MONITORING

7.1 Water Balance Data

The monitoring system became operational on October 1, 2016. Water balance data collected during fourth quarter of 2016 are shown on Figure 14. All water balance quantities on Figure 14 are cumulative, except for soil water storage. ET was computed by closing the water balance as described in Apiwantragoon et al. (2014).

Most precipitation received at the test section was released as evapotranspiration, as illustrated by the coincidence of the precipitation and evapotranspiration curves. Percolation, lateral flow, and runoff were nil. Soil water storage began to increase slightly in mid-December 2017 in response to heavier precipitation.

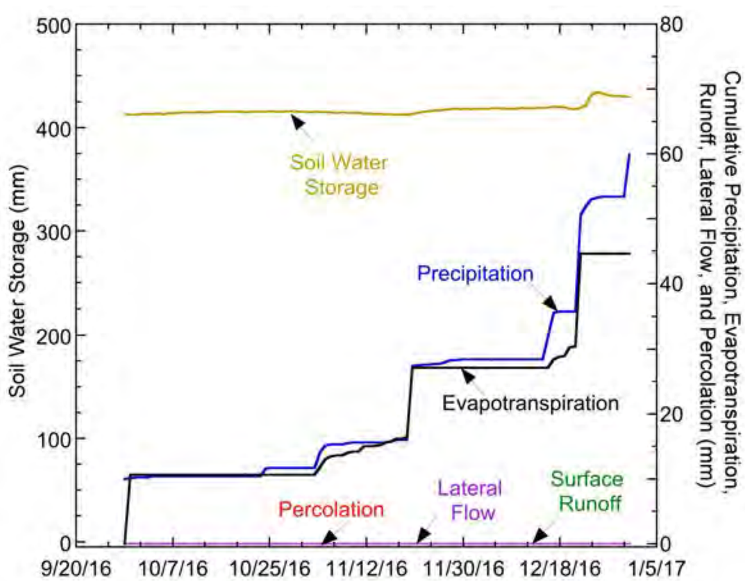


Figure 14. Water balance data collected at test section from October 1- December 31, 2016.

7.2 Meteorological Data

Air temperature and solar radiation gradually decreased and the relative humidity increased during fourth quarter 2016, which is expected during the transition from fall to winter. Precipitation was scant during the first half of the fourth quarter 2016, but became more frequent beginning in mid-November and through December. Several larger precipitation events occurred in December, which resulted in infiltration and a slight increase in soil water storage evident in December. Nearly all of the precipitation was released at ET.

7.3 Subsurface State Variables

Hydrologic conditions within the test section were quiescent during the fourth quarter 2016 due to sparse precipitation and relatively uniform water content of the soils at the time of construction. Water contents at all depths except the upper portion of the cover Layer 3 (growth medium) were virtually unchanged during the fourth quarter of 2016. The upper portion of the growth medium is an exception. Wetting of the growth medium began in late November in response to more frequent and larger precipitation events. Wetting of the upper portion of the growth medium continued through the end of 2016, but did not propagate to greater depths.

Soil temperatures within the test section gradually dropped during the fourth quarter 2016 in response to decreasing air temperatures. Greater reductions in temperature occurred closer to the surface. Temperatures at shallower depths also exhibited greater variability in temperature, reflecting less thermal damping at shallower depths (Benson et al. 1995).

7.4 Vegetation

Moist conditions in the spring promoted emergence of the vegetation on the test section (Figure 15). A protocol to evaluate establishment of the vegetation is included in the monitoring program.

Figure 15. Vegetation emerging from test section in May 2017.

8 SUMMARY CONCLUSIONS

A cover performance test section was constructed in 2016 as part of reclamation cover placement on the inactive tailings Cell 2 at the White Mesa Uranium Mill site. The test section was designed and constructed in accordance with the State of Utah regulations, an approved reclamation plan, and applicable NRC guidance. Initial percolation monitoring shows no percolation from the base of the cover. Vegetation plant cover is progressing well. Placement of the Phase 2 cover on Cell 2 is expected to occur after performance monitoring is complete (approximately 7 years) and UDEQ acceptance that the performance criteria has been met is received.

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Jerritt Canyon Tailings Storage Facility 1 Operational History and Closure Case Study

Dave Wanner
SRK Consulting, (U.S.), Inc., Elko, NV, USA

Dave Bentel
SRK Consulting, (U.S.), Inc., Reno, NV, USA

Mirinda Jones
Jerritt Canyon Gold LLC, Elko, NV, USA

1 INTRODUCTION AND BACKGROUND

In 2010, the Nevada GeoEnvironmental Department of SRK Consulting (U.S), Inc. (SRK), located in both Reno and Elko, Nevada, completed a Final Plan for Permanent Closure (FPPC) for the Jerritt Canyon Mine Tailings Storage Facility 1 (TSF 1). This paper describes:

- Background information pertinent to development of closure plans for TSF 1
- Technical factors affecting development of a FPPC of TSF 1
- Development of FPPC for TSF 1
- FPPC evolution between 2011 and 2017

TSF 1 is located at the Jerritt Canyon Mine facilities approximately 50 miles north of Elko, in Elko County, Nevada, USA (Figure 1). The Jerritt Canyon Mine began in 1980 as an open pit gold mining and milling operation. In 1994, underground mining began to supplement ore production and feed to the Jerritt Canyon Mill.

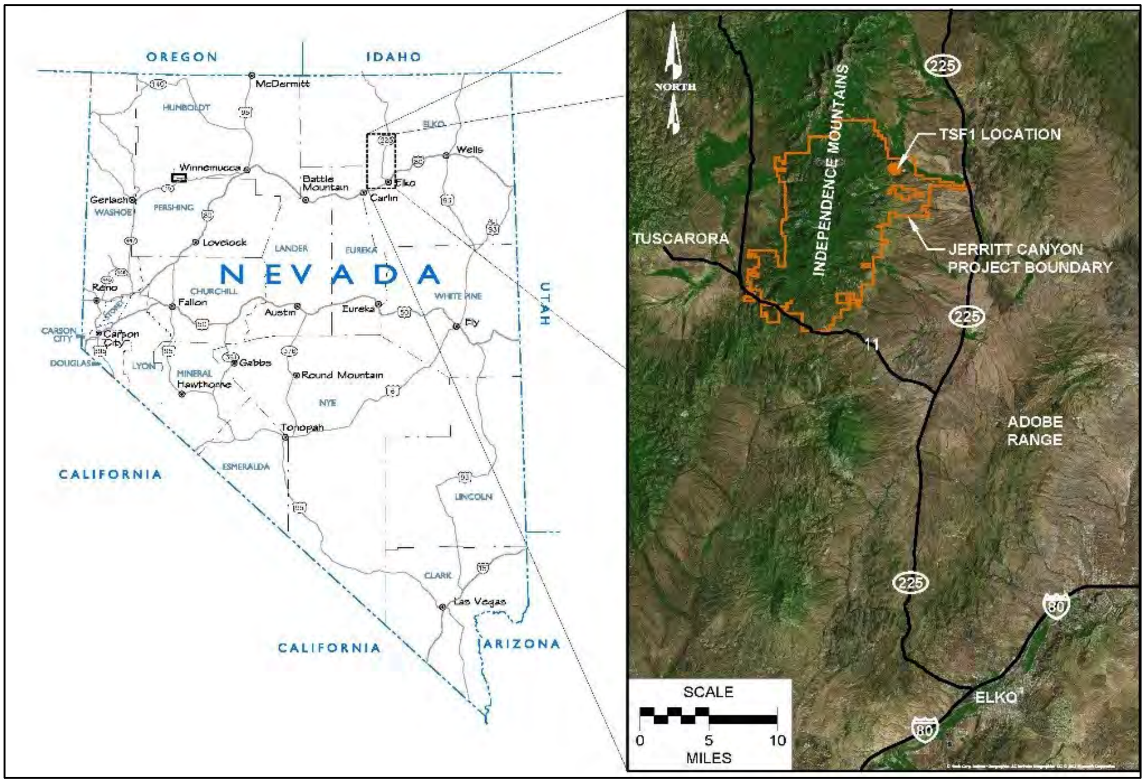

Figure 1. Jerritt Canyon Mine location map.

1.1 *Ownership History*

The Jerritt Canyon TSF 1 has been under the management and ownership of nine different companies or joint ventures over the 35 years of its operation. Table 1 below provides a summary of the Jerritt Canyon Mine's owner and operator history since 1980, when Jerritt Canyon was first owned and operated under a joint venture between FMC Minerals and Freeport Minerals Company.

Table 1: Summary of the Jerritt Canyon Mine's owner and operator history since 1980.

Year	Owner	Operator
1980-1985	FMC Minerals / Freeport Minerals Company	FMC Gold
1985-1990	Freeport-McMoRan Gold Company	FMC Gold
1990-1996	Minorco USA	Independence Mining Company
1996-1998	Minorco / Meridian	Minorco
1998-2003	AngloGold / Meridian	AngloGold
2003-2007	Queenstake Resources USA, Inc.	Queenstake
2007-2012	Yukon Nevada Gold Corporation	Queenstake
2012-2015	Veris Gold Corporation	Veris Gold USA Inc.
2015-Present	Sprott Mining Inc. / Whitebox Asset Management	Jerritt Canyon Gold, LLC

1.2 *Physiographic and Topographic Setting*

The Jerritt Canyon mill site and 360-acre TSF 1 (Figure 2) are located west of the North Fork of the Humboldt River on an eastern-flanking alluvial fan of the Independence Mountains, within the Great Basin section of the Basin and Range Physiographic Province. Elevations at the Jerritt Canyon mill site and TSF 1 area range from approximately 6,300 feet above mean sea level (ft amsl) to approximately 6,400 feet ft amsl. The area is drained by numerous ephemeral and some perennial drainages which originate in the mountain blocks and flow onto alluvial fans, and thence to the east into the North Fork of the Humboldt River. Surface soils include alluvial terrace and valley deposits.

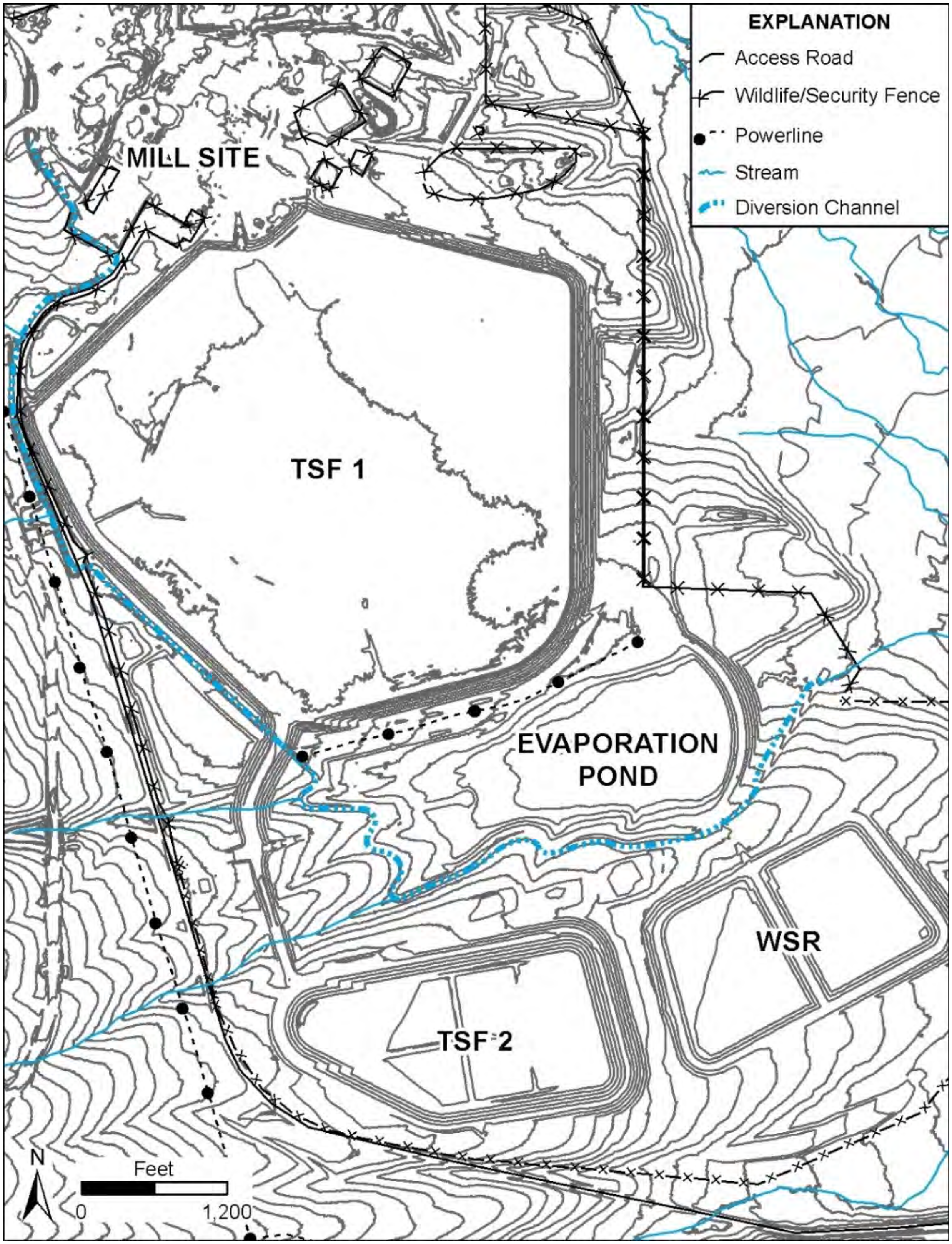

Figure 2. Jerritt Canyon Mill Site and the TSF 1 layout
Note: Figure 2 also shows the TSF 1 Evaporation Pond, TSF 2 (commissioned in 2011) and the TSF 2 Water Storage Reservoir.

1.3 *Site Geology and Hydrogeology*

Lithology of the Tertiary and Quaternary alluvial fan deposits underlying the TSF 1 display considerable spatial variability consistent with mountain front alluvial fans. The underlying alluvial fan deposits consist of interbedded lenses of clayey silts, sands, and gravels with discontinuous clay layers. The depth to bedrock in the vicinity of TSF 1 is estimated to be at least 1000 feet based on drilling results from the deep water supply wells adjacent to TSF 1 and the mill site.

The hydrogeologic characteristics of the groundwater system within the alluvial fan underlying TSF 1 reflect the lithological variability of the alluvial fan. The overall gradient of the alluvial aquifer is west-northwest to east-southeast (SRK 2009).

Local groundwater recharge is a result of infiltration from California and Foreman Creeks, both of which are perennial streams flowing into the North Fork of the Humboldt River. Additional up-gradient recharge occurs periodically through snowmelt and runoff along the eastern front of the Independence Mountains (SRK 2009).

Initial hydrogeologic investigations of the alluvial aquifer at the Jerritt Canyon site, prior to construction of TSF 1, reported a region of compartmentalized zones of saturation between ground surface and 100 feet (30 meters) in depth. These perched zones appeared in silty sand to gravel lenses surrounded by clayey soils. The regional groundwater system within the alluvial fan was reported to start at a depth of approximately 100 feet below ground surface (bgs). Testing completed within the regional aquifer, during these initial investigations, resulted in a transmissivity of 5,450 gallons per day per foot (gpd/ft), and a storativity of 0.0059 (Sergeant, Hauskins & Beckwith 1984).

Natural groundwater flow from the west through the area of the TSF 1 has been previously estimated at 32 gallons per minute (gpm) per 1000 ft width of flow path at a gradient of 2% (Water Management Consultants 2003). Assuming an area of influence on the TSF 1 having a width of 7,500 feet along the range front, a total through-flow of 250 gpm would result (SRK 2009).

Previous seepage recovery investigations were focused on the south and east perimeters of the TSF 1. Based on testing of Jerritt Canyon monitoring wells (Hydro-Engineering 1991), transmissivity values ranged between approximately 63 gpd/ft and 35,000 gpd/ft, and storage coefficients ranged between 0.0004 and 0.021. The wide range of values may be due to the combination of geologic heterogeneity prevalent in the aquifer, and variations in well screen lengths and intervals. The transmissivity results equate to horizontal hydraulic conductivities of approximately 10^{-3} to 10^{-6} cm/sec. Vertical hydraulic conductivity within the regional aquifer was estimated at between 10^{-6} and 10^{-7} cm/sec (Water Management Consultants 2003).

1.4 *Climatology*

Site climate data (i.e. statistically summarized precipitation, temperature, and evaporation records) were obtained from the Jerritt Canyon Mine meteorological station. Average annual precipitation at Jerritt Canyon measured between 1997 and 2010 was 12.7 inches (32.2 cm), which occurred mostly as snowfall during the winter.

Evaporation data from the mine site pan evaporation station were collected in March through October, from 2003 through 2008. No data are available in the months December through February due to snow cover and freezing conditions, and evaporation/sublimation losses during November through February are assumed to be zero. Average annual evaporation for the years 2003−2008 was 51.5 inches.

Rainfall intensity data were obtained from the National Oceanic and Atmospheric Administration (NOAA). Estimates of 24-hour duration storm events at latitude 41.40°N and longitude 115.89°W with recurrence intervals of 10 years, 25 years, and 100 years as well as the probable maximum precipitation event (PMP) are summarized in Table 2. 24-hour duration storm event precipitation**.**

Table 2. 24-hour duration storm event precipitation**.**

Recurrence interval	Precipitation depth (inches)
10-year	1.6
25-year	1.9
100-year	2.3
PMP	8.2–9.5[1]

[1] Data from NOAA's National Weather Service Precipitation Frequency Data Server website.

Average daily temperature variations range from 39°F to 19°F (4°C to -7°C) in the winter months (December–February) and 82°F to 56°F (28°C to 13°C) in the summer (June–August).

2 TECHNICAL FACTORS AFFECTING FPPC FOR TSF 1

Technical factors affecting development of the TSF 1 FPPC include:

- Design and construction history for deposition
- Methods of tailings deposition
- Supernatant water inventories
- TSF 1 seepage water solution inventories
- Seepage remediation system operation

2.1 *Design and Construction History*

Sergeant, Hauskins & Beckwith designed the initial phases (Phase 1 and Phase II expansion) of TSF 1. Initial Phase 1 construction and subsequent operation of TSF 1 began in 1980. Knight Piésold designed five subsequent raises, including the current final raise (Phase VII) that was constructed in 1998 (Veris 2014). A 1985 aerial photo of TSF 1 between the Phase II and Phase III expansions is shown in Figure 3 below.

Figure 3. Aerial Photo of TSF 1 looking north (circa 1985). Source: Jerritt Canyon Mine.

TSF 1 is a ring-dike impoundment constructed over native soils which were graded and partially compacted prior to depositing tailings. The raise methods for these expansions consisted of combinations of downstream, upstream, and centerline raises, as shown in Table 3 below. The final Phase VII raise construction resulted in an impoundment area of 300 acres, a total facility footprint (with embankment) of 360 acres, and a maximum embankment height of 142 feet (at the east side of TSF 1).

During Phase IV construction, a water reclaim slot was installed to facilitate the control of and access to the tailings supernatant pond and reclaim barges. The slot was comprised of an excavated trench and bulkhead that controls the water intake level. Raising and extension of the slot have been conducted during subsequent facility expansions as necessary.

Table 3. Summary of TSF 1 embankment raises.

Raise	Raise height (ft / m)	Embankment elevation (ft amsl / m amsl)	Raise method
Phase II (1982)	34 / 10	6,340 / 1,932	Downstream Raise
Phase III (1988)	10 / 3	6,350 / 1,935	Downstream
Phase IV (1991)	12 / 3.7	6,362 / 1,939	Upstream
Phase V (1993)	12 / 3.7	6,374 / 1,943	Centerline
Phase VI (1995)	11 / 3.3	6,385 / 1,946	Upstream (East) Centerline (West)
Phase VII (1998)	7 / 2.1	6,392 / 1,948	Centerline

(Veris 2014)

2.2 *Methods of Tailings Deposition*

During operations, tailings were typically deposited from the outside perimeter, creating a beach sloping towards the center of the impoundment at an approximate 0.7% slope. This deposition practice developed a large conical depression in the center of the TSF 1 impoundment. Subaqueous tailings deposition into TSF 1 was practiced until 1987, when deposition was changed to subaerial in an attempt to 1) reduce the volume of tailings supernatant solution containing high concentrations of chloride entering TSF 1 as described in the following section; and 2) increase compaction of the tails. In the late 1990s, a combination of both subaqueous and subaerial deposition techniques were used in an attempt to fill the depression in the center of TSF 1. In 2001, the tailings deposition was reverted to exclusively subaerial which has continued to date.

2.3 *TSF 1 Seepage*

Seepage from TSF 1 was detected in the alluvium as early as 1981, when a recharge mound and migration of tailings decant solution were first observed. This seepage from TSF 1 was found to be moving in the underlying vadose zone alluvium predominately to the south-southwest and at a rate of approximately 450 gpm. Groundwater within a few hundred feet of the tailings facility was impacted with high concentrations of chloride and total dissolved solids.

The chloride concentrations of seepage in the vadose zone and groundwater resulted from the high chloride concentrations in the TSF 1 supernatant, which originated from the operation of the now-closed wet mill chlorination circuit. The high chloride concentrations substantially hindered the recycling of TSF 1 supernatant back to the mill operations for reuse. Therefore, large volumes of supernatant began to accumulate in TSF 1, which in turn increased a driving hydraulic head of seepage into the vadose zone and groundwater underlying TSF 1. The highest chloride concentrations in the groundwater have typically been found in the upper levels of the aquifer at shallow depths.

2.4 *Solution Inventories*

As previously discussed, due to the poor quality of the TSF 1 supernatant water, reuse as make-up water has been limited since the beginning of milling operations. The resulting buildup of supernatant inventories in TSF 1 has been the principal driver of Jerritt Canyon water management for most of the project life. Earliest records from bathymetric surveys indicate a surplus of more than 800 million gallons of supernatant within TSF 1, with depths exceeding 30 feet (9 meters). TSF 1 supernatant volumes from 2002 to 2008 are provided in Table 4.

Table 4. TSF 1 supernatant volumes (2002–2008).

Year	Supernatant volume (million gallons)
2002	>800
2003	668
2004	672
2005	694
2006	583
2007	469
2008	507

Google Earth© images provided in Figures Figure 4 and Figure 5 illustrate the relative consistency in supernatant water ponding on TSF 1 for circa 1994 through 1999. Figures Figure 6 and Figure 7 represent 2006 and 2010 conditions (i.e. before and after construction of the 456-million-gallon Evaporation Pond constructed in 2006/2007 for disposal of surplus process solutions). Detailed bathymetry surveys in TSF 1 were not conducted after 2008; however, temporary shutdown of milling operations in 2008 and 2009 combined with enhanced evaporation techniques used on TSF 1 and supernatant transfer to the Evaporation Pond resulted in noticeable reductions in supernatant inventories.

Figure 4. TSF 1 circa 1994.

Figure 5. TSF 1 circa 1999.

Figure 6. TSF 1 circa 2006.

Figure 7. TSF 1 circa 2010.

2.5 *Seepage Remediation System*

As previously mentioned, groundwater quality down-gradient of TSF 1 began to show signs of seepage as early as 1981 shortly after tailings deposition began. In 1985, the first down-gradient seepage collection wells were installed. The change in groundwater quality seen in the initial TSF 1 monitor wells prompted several studies and subsequent installation of collection, recharge, and monitor wells over the following years. These well installations and their operation became known as the Jerritt Canyon Mine Seepage Remediation System or SRS.

The Jerritt Canyon SRS was designed and implemented in cooperation with the Nevada Division of Environmental Protection (NDEP) to minimize the volume and rate of seepage from TSF 1. Currently, the SRS consists of a ring of 90 collection wells (i.e. pumpback wells) surrounding TSF 1 to collect seepage to create localized reversal of the groundwater gradient (Figure 8). Fourteen freshwater recharge wells are installed down-gradient of the seepage collection wells to re-inject clean groundwater in an attempt to accentuate the reversing of the groundwater gradient in the vadose zone. In addition, there are 82 monitoring wells located predominantly down-gradient of the collection wells and various piezometers installed within TSF 1.

Collected seepage from TSF 1 is pumped to the mill as process makeup water or to the Evaporation Pond, where it is actively evaporated. Until 2011, the SRS was collecting up to about 900 gpm from the outside perimeter of TSF 1.

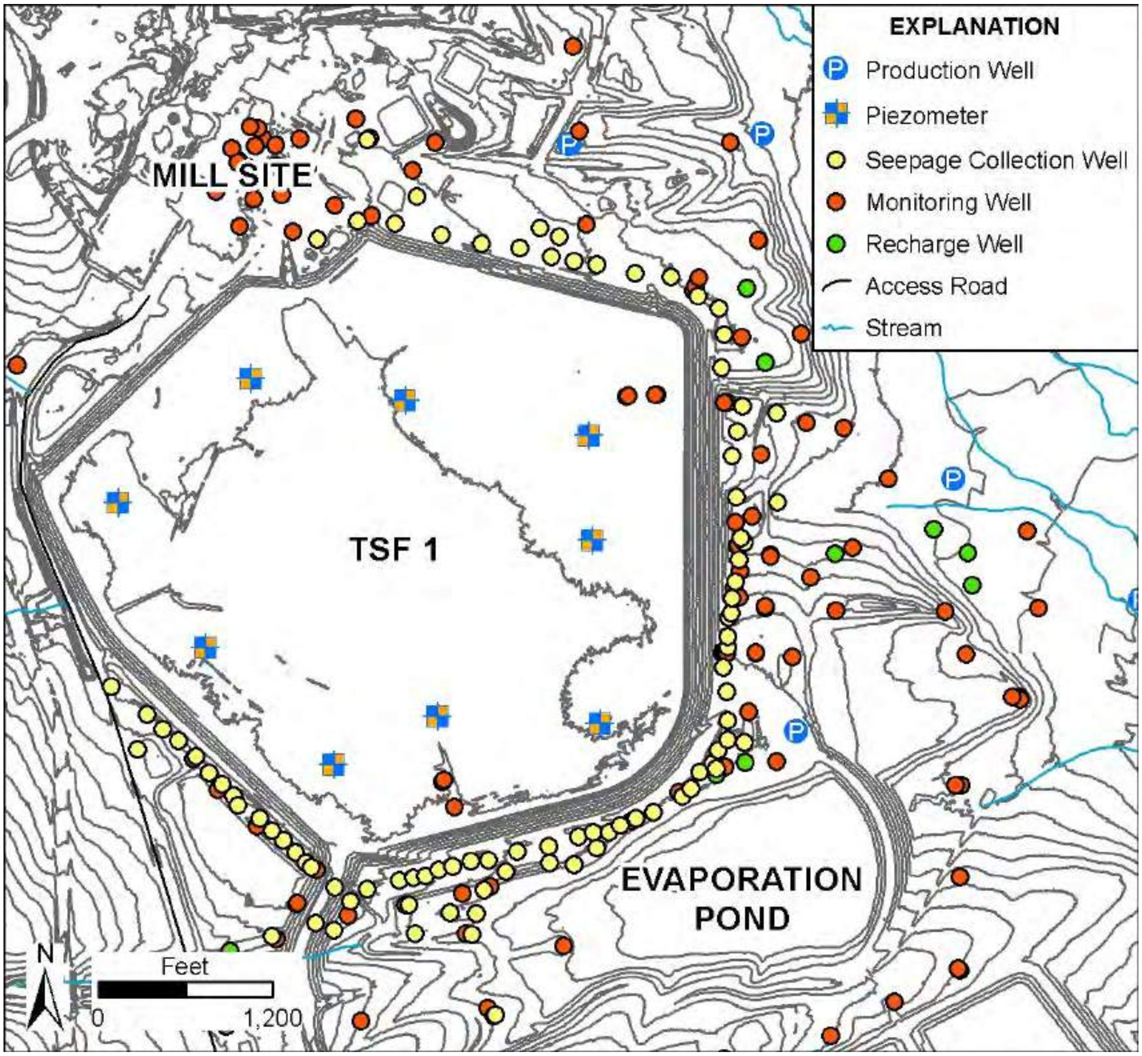

Figure 8. TSF 1 SRS collection, recharge, and monitor wells and piezometers

3 TSF 1 FINAL PLAN FOR PERMANENT CLOSURE

SRK was commissioned by Jerritt Canyon Gold LLC (JCG) to complete a FPPC for TSF 1 in accordance with Nevada Administrative Code (NAC) 445A.447, which requires an FPPC for a process component to be submitted to the NDEP at least two years before its anticipated permanent closure. Submittal of a FPPC at least two years prior to permanent closure is intended to allow the operator time to identify data gaps and conduct necessary characterization of the process component (i.e. TSF 1) in order to refine the FPPC and develop a detailed and workable plan to implement the closure of the facility.

During the preliminary development of the FPPC, SRK evaluated store-and-release covers versus synthetic-lined covers for TSF 1. Store-and-release covers primarily rely on evapotranspiration from plants to limit infiltration. However, these types of covers were considered inadequate in meeting the NDEP design criteria, since most precipitation at Jerritt Canyon occurs in the form of winter snow and subsequent snowmelt and infiltration occurs while plants remain dormant in the early to mid-spring.

Therefore, a synthetically-lined geomembrane was incorporated into TSF 1's cover design in the ultimate FPPC submittal during 2010. The cover design, subsequently approved by the NDEP in 2011, was selected to 1) preclude any meteoric infiltration into the underlying tailings and 2) eliminate the driving hydraulic head causing continued migration of contaminates into underlying vadose zone and groundwater.

The FPPC for TSF 1 included a closure cover consisting (from bottom to top) of an interim working platform (IWP) layer, 40-mil HDPE geomembrane, and a 3-foot-thick alluvial-type growth media cover. The final cover is designed to slope towards the southern end of the impoundment where stormwater runoff is allowed to drain off the facility in a controlled manner through an outflow spillway and into an existing stormwater diversion channel.

4 FPPC EVOLUTION BETWEEN 2011 AND 2017

In an effort to completely evacuate supernatant inventories from TSF 1 and prepare for closure, Jerritt Canyon began transferring remaining TSF 1 supernatant in 2011 to the newly-constructed Water Storage Reservoir southeast of the Evaporation Pond (Figure 9). By the third quarter of 2013, the new TSF 2 had become operational and tailings deposition ceased in TSF 1. By the middle of 2014, the TSF 1 supernatant pool was completely removed (Figure 10) by either pumping to other facilities (i.e. Evaporation Pond or the TSF 2 Water Storage Reservoir), enhanced evaporation from the tailings surface, or infiltration into the existing tailings.

As a direct result of the above actions, the current necessity for seepage collection and removal has reduced from 900 to approximately 600 gpm. This is primarily due to the removal of the TSF 1 supernatant pool, resulting in reduction of the recharge mound and lowering of the groundwater table beneath TSF 1. This, in turn, has caused numerous collection wells to dry up.

Figure 9. TSF 1 circa 2012.

Figure 10. TSF 1 circa 2014

Relevant closure design elements either partially or fully completed during the time period 2011 through 2017 include the IWP, geomembrane liner installation, growth media cover, and the outflow spillway. Each of these elements is discussed below.

4.1 *Interim Working Platform*

Since acquisition of the Jerritt Canyon Mine, the current owner, Sprott Mining Inc., has made aggressive strides in initiating the closure of TSF 1. Since 2015, over half of the TSF 1 impoundment area has been covered with IWP material and almost one-third has been lined with geomembrane. Cover construction is ongoing and is expected to be completed within the next three years.

JCG began placement of the IWP layer in 2014 over tailings beach areas adequately dried to allow safe equipment access. The IWP consists of varying thicknesses of alluvial, waste rock, and spent heap materials. These IWP materials were installed over the tailings to provide a stable working area to facilitate the installation of the remaining cover elements (i.e. geomembrane liner and growth media). The IWP establishes a base grade over which a uniform three feet of growth media would be installed. The base grade on the IWP and the matching surface of overlying growth media layer are designed to promote stormwater drainage off the final cover. Typical base grades over the TSF 1 closure cover vary from 0.5% to 4%.

The majority of IWP material placed to date has originated from 750,000 cubic yards (CY) of spent ore offloaded from the Jerritt Canyon heap leach pad. Remaining IWP material is currently being excavated from alluvium material underlying the heap leach pad. To date, an estimated 830,000 CY of this underlying alluvium and a small amount of waste rock material (approximately 36,000 CY) from the Jerritt Canyon underground mining operations has been placed as IWP material onto TSF 1. Remaining IWP material for the TSF 1 closure is expected to be obtained from the alluvium material below the offloaded heap leach pad. Advancement of the IWP cover in 2015 is shown in Figure 11.

Figure 11. 2015 Advancement of IWP. Aerial Photo Source: Jerritt Canyon Mine, 2015.

During the 2016 closure cover construction, IWP surfaces were leveled and smoothed to the extent practical with a motor grader and smooth-drum roller. To cover protruding rock and provide additional liner cushion, the IWP surface was capped with a thin layer (~1−4 inches) of dried tailings (Figure 12). Dried tailings were excavated and hauled from areas within the TSF 1 impoundment yet to be covered with IWP materials (Figure 13).

Figure 12. Capping IWP with dried tailings.

Figure 13. Excavating and hauling dried tailings.

4.2 *Filling of the Topographical Low Depression*

In an effort to fill the depression (i.e. low point) in the center of the TSF 1 impoundment and create positive drainage from this area and towards the outflow spillway, JCG received approval from the NDEP to deposit additional tailings in this area during the construction of the final closure cover (as was originally anticipated in the 2010 FPPC). This was accomplished by separating the tailings deposition from the areas being closed (i.e. by constructing an interior contour berm around the perimeter of this depression during the 2016 construction season (Figures Figure 14, Figure 15, and Figure 16)).

This 4-foot-high berm is designed to collect and convey non-contact stormwater from the “outer ring” closure cover towards the future outflow spillway constructed as part of the final closure cover. This berm design prevents non-contact stormwater from comingling with the temporary tailings deposition in the middle of TSF 1. Construction of this interior contour berm is shown on Figure 17.

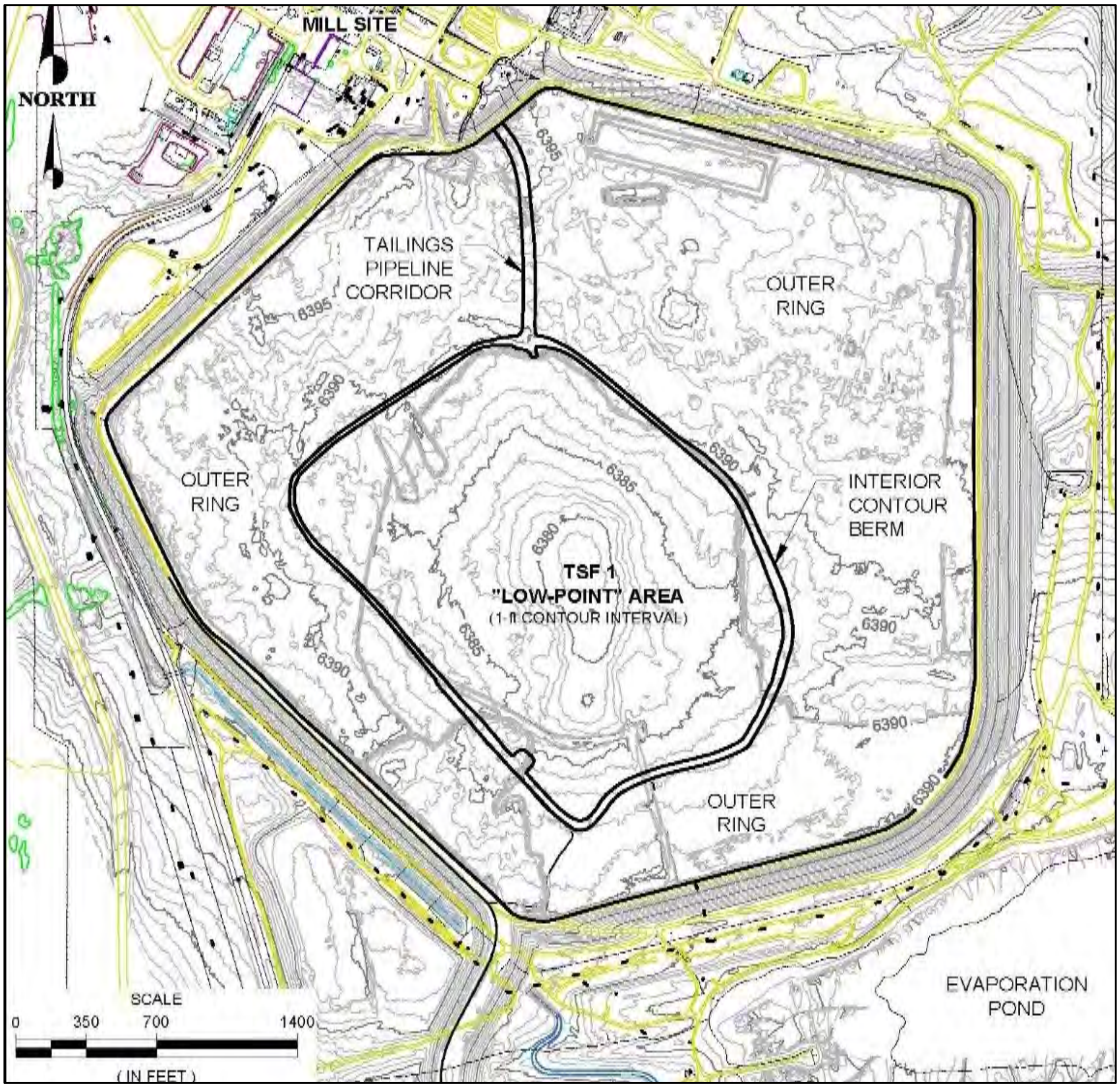

Figure 14. TSF 1 interior contour berm layout.

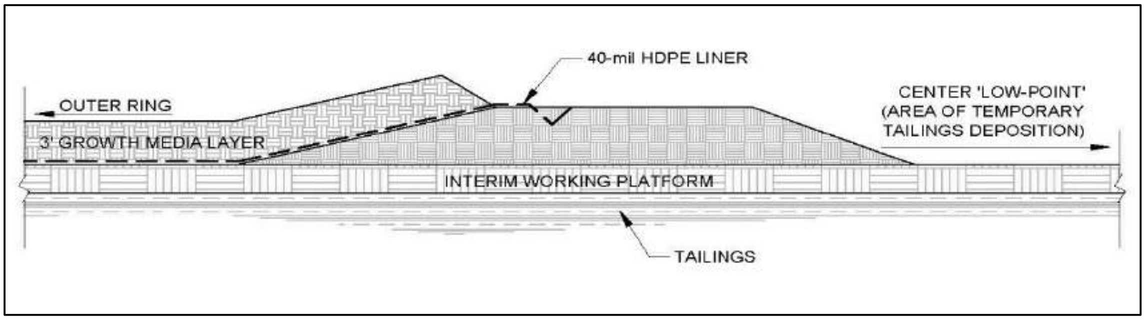

Figure 15. TSF 1 interior contour berm section.

Figure 16. TSF 1 interior contour berm construction (looking west).

Figure 17. TSF 1 interior contour berm construction.

In December 2016, JCG began depositing tailings inside this interior contour berm. This operation is planned to continue throughout 2017 until the center depression is filled with tailings, and the desired final tailings beach slopes are established. At the same time, the remaining closure cover (i.e. IWP, liner, and growth media) will continue to be constructed outside this berm. Ultimately, the center depression will be covered with IWP, liner, and growth media materials, and its surface will be graded to drain runoff to the south TSF 1 perimeter where the future outflow spillway will be constructed. The final IWP grading design is provided in Figure 18.

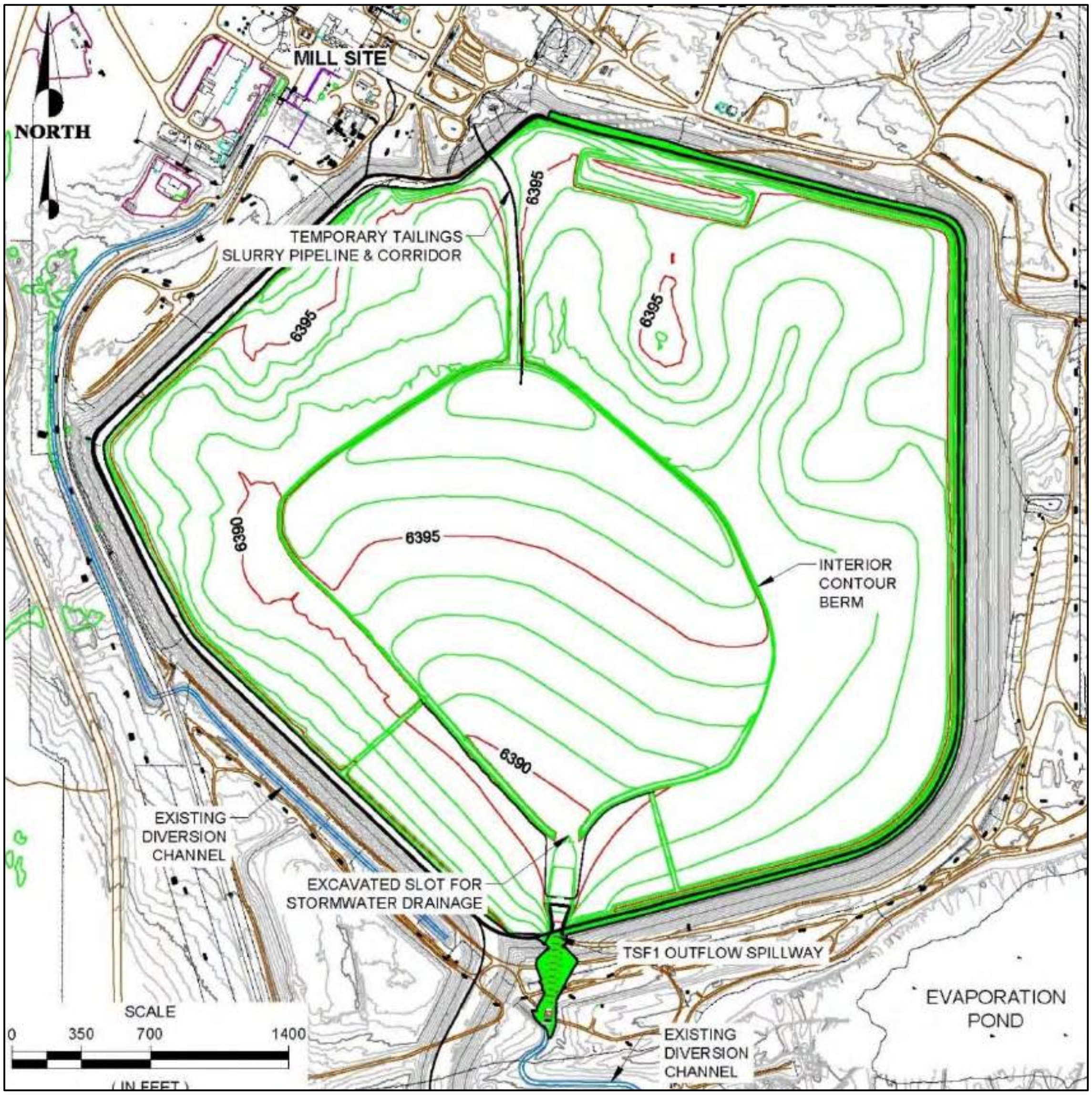

Figure 18. TSF 1 IWP Final Grading & Spillway Design.

4.3 *Geomembrane Liner Installation*

The final cover design includes the installation of a 40 mil HDPE smooth geomembrane liner placed over the entire 300-acre IWP surface followed by 3 feet of alluvial growth media. JCG began installing this liner in August 2016, and by November 2016, approximately 3.82 million square feet of liner were installed over the "outer ring" of the TSF 1 cover area (Figure 19).

During liner installation, approximately 0.5 CY of alluvium material was placed for ballast every 100 feet along each 23-foot-wide liner panel (Figure 20). Material used as ballast originated from alluvial borrow sources planned for use as final growth media cover.

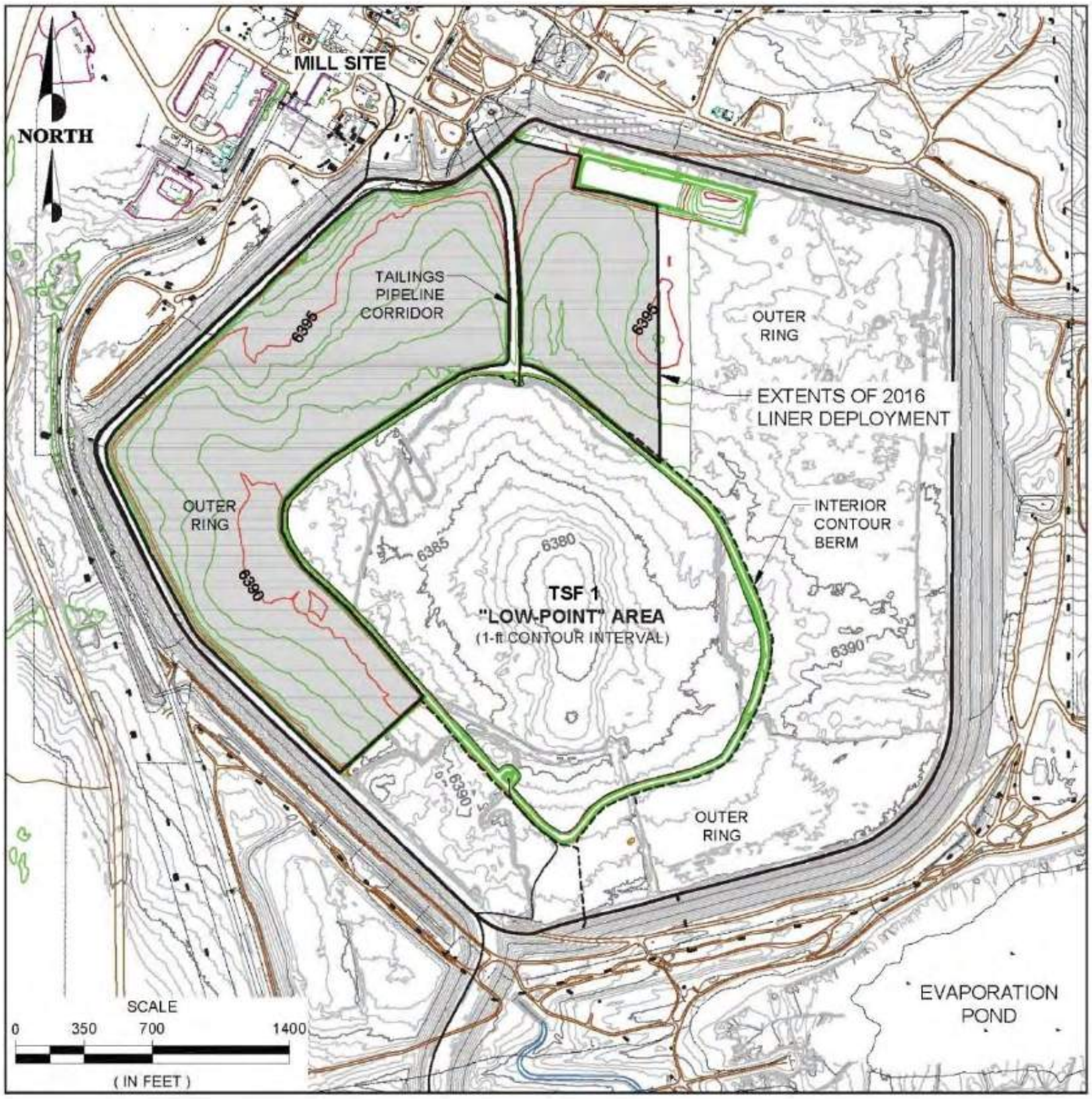

Figure 19. 2016 liner installation progress.

Figure 20. Ballast placement during liner installation

4.4 *Growth Media Cover*

The growth media thickness for the TSF 1 cover was determined from minimum required rooting depths of vegetative species planned for use in the final Jerritt Canyon reclamation seed mix, Table 5. As shown, the deepest minimum rooting depth in this list is 24 inches (~60 cm). From negotiations with the NDEP regarding the TSF 1 FPPC, the final growth media cover thickness was increased to 36 inches (~90 cm) to accommodate additional rooting depth and allow for additional vadose storage of precipitation.

Table 5: Final cover vegetation minimum rooting depths.

Common name	Scientific name	Minimum rooting depth (in.)	Source/comment
GRASSES			
Idaho fescue	*Festuca idahoensis*	14	http://plants.usda.gov/java/charProfile?symbol=FEID
Slender wheatgrass	*Elymus trachycaulus*	16	http://plants.usda.gov/java/charProfile?symbol=ELTR7
Sandberg bluegrass	*Poa secunda*	10	http://plants.usda.gov/java/charProfile?symbol=POSE
Bluebunch wheatgrass	*Pseudoroegneria spicata*	10	http://plants.usda.gov/java/charProfile?symbol=PSSPS
Beardless wildrye	*Elymus triticoides*	10	http://plants.usda.gov/java/charProfile?symbol=LETR5
Great Basin wildrye	*Elymus cinerus*	20	Used basin wildrye (*Leymus cinereus*)
Mountain brome	*Bromus marginatus*	20	http://plants.usda.gov/java/charProfile?symbol=BRMA4
FORBS			
Ladak alfalfa	*Medicago sativa*	24	http://plants.usda.gov/java/charProfile?symbol=MESA
Small burnet	*Sanguisorbia minor*	12	http://plants.usda.gov/java/charProfile?symbol=SAMI3
Lewis blue flax	*Linum lewisii*	14	http://plants.usda.gov/java/charProfile?symbol=LILE3
Rydberg's penstemon	*Penstemon rydbergii*	14	http://plants.usda.gov/java/charProfile?symbol=PERY
Western yarrow	*Achillea millefolium*	8	http://plants.usda.gov/java/charProfile?symbol=ACMIO
SHRUBS			
Mountain big sage	*Artemesia tridentata vaseyana*	16	http://plants.usda.gov/java/charProfile?symbol=ARTRV
White stem rubber rabbitbrush	*Chrysothamnus nauseosus*	18	Rooting depth not available in USDA database
Antelope bitterbrush	*Purshia tridentata*	20	http://plants.usda.gov/java/charProfile?symbol=PUTR2
Average minimum rooting depth		15.1	

Growth media cover material for the TSF 1 closure is expected to be sourced from the alluvium material in and around the Jerritt Canyon mill site and offloaded heap leach pad. As of August 2017, about 50,000 CY of growth media material have been placed over lined areas. When complete, approximately 1.45 million CY of growth media will be used to cover TSF 1. The final growth media cover design is shown on Figure 21.

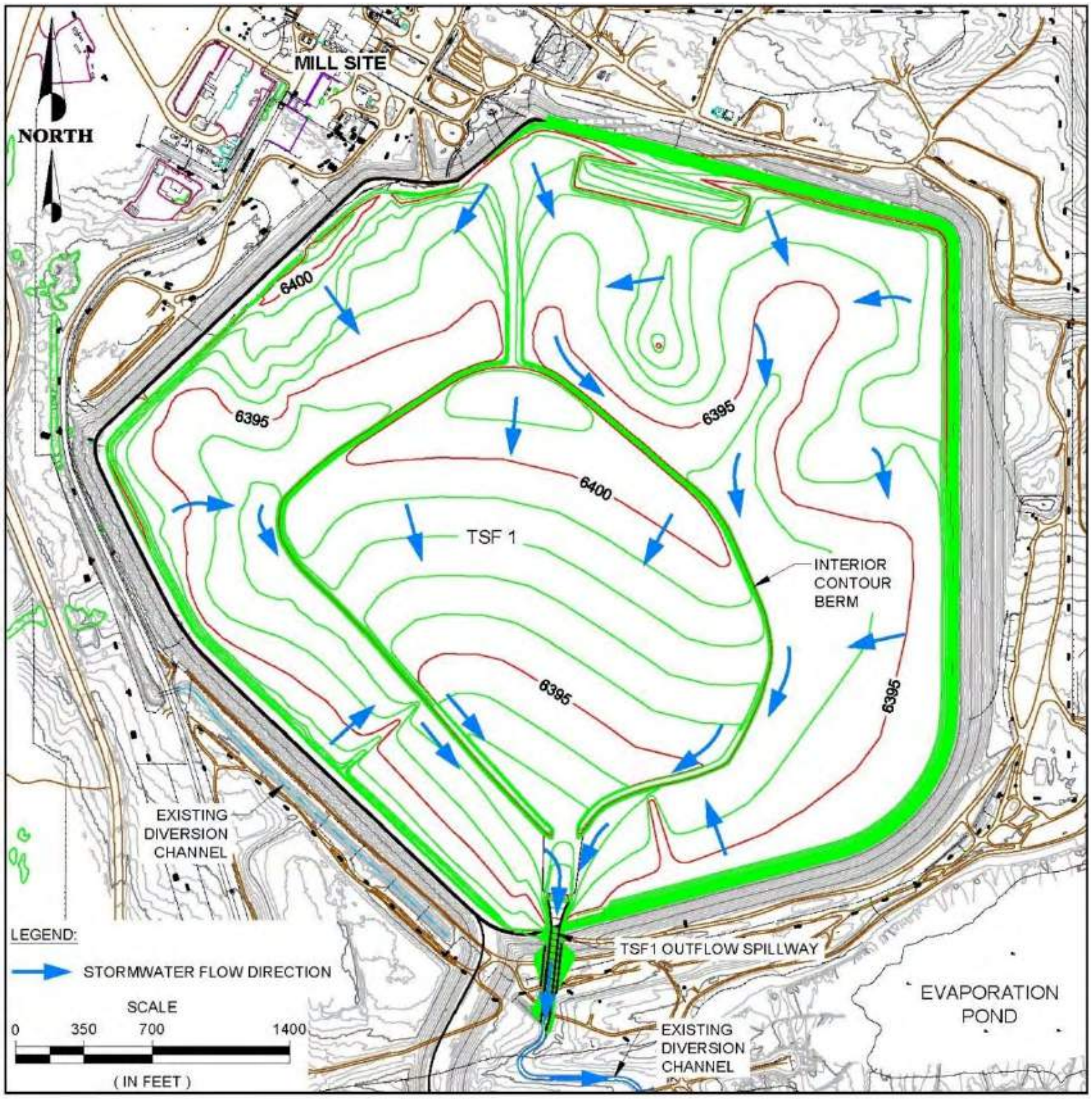

Figure 21. TSF 1 growth media cover design.

To provide as much protection of the liner as possible, growth media is placed in one 3-foot-thick lift over the liner using a bulldozer and 40-ton articulated haul trucks. During placement, the growth media is not allowed to be dumped directly from the trucks onto the liner, but only in windrows on top of previously placed growth media material and short of the growth media cover advancement. With bulldozers, these windrows are then pushed out over the liner to the specified 3-foot depth to advance the cover, as shown on Figure 22.

Figure 22. Growth media placement over geomembrane liner

4.5 *Outflow Spillway*

The outflow spillway and conveyance is designed to drain stormwater off TSF 1 into the existing stormwater diversion channel at the south TSF 1 embankment. This spillway channel is designed to manage peak flows generated by PMP storm conditions (9.5 inches in 24 hours) and route these PMP flows off the final TSF 1 surface. The spillway will be cut through the south TSF 1 embankment and constructed as part of the final soil cover installation. Tailings excavated to construct the channel will be placed to fill remaining low points in the IWP and middle low-point area or used to cap the IWP to prepare an amenable surface for liner installation.

At the south end of the impoundment, collected stormwater runoff will be routed to a shallow 100-foot-wide trapezoidal channel connecting the interior contour berm to the throat of the 40-foot-wide spillway, which finally discharges into an existing 15-foot-wide diversion channel south of TSF 1. The steeper portions of the outflow spillway will be armored with HydroTurf™. A riprap energy dissipation structure will be constructed at the discharge end of the spillway.

ACKNOWLEDGEMENTS

I would like to thank JCG and the Jerritt Canyon Mine Environmental Department, headed by Mirinda Jones, for their assistance with the preparation of this paper. I would also like to recognize the Jerritt Canyon Mine Operations and Reclamation Services for their team efforts in implementing the TSF 1 closure and reclamation, especially: Chad Eklund, Surface Operations Superintendent; Bronco Garton, General Foreman; Cecil Pranke; Process Superintendent; and John Vipham, Mine Planner. Various contractors and vendors who have made the TSF 1 closure

a success up to this point include: Ross Wines and Wines Peak Construction of Elko, Nevada; High Mark Construction, LLC of Elko, Nevada; Clark West and Agru America, Inc. of Fernley, Nevada; and EC Applications of Sparks, Nevada.

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Reclamation

The Re Using of old Tin tailings to increase the life of San Rafael Mine in Puno, Peru

Josue Huere Iparraguirre
Minsur S.A. Lima, Perú

ABSTRACT: San Rafael is an old tin mine located in the south Andean in Peru and started its operations in 1977, but now, its ore reserves and production has been decreased. However, geology investigations have confirmed that there is an old tin tailings storage facility (TSF) called B2, where it is possible to extract the mineral contained in those tailings with the opportunity to increase in five years the life of San Rafael and maintaining the production of concentrated of tin. This project will be developed with the optimization of the current facilities, using and recycling the water operation and installing a new tailing re use plant and a new TSF called, B4.

1 GENERAL PROJECT DESCRIPTION

San Rafael is a Minsur tin mining operation located in the District of Antauta, Province of Melgar, Department of Puno, Peru. The mine is close to Lake Chognacota, and is geographically located in the upper part of Chognacota ravine, south of the Eastern mountain range of the Peruvian Andes, at an altitude ranging between 4,500 and 5,200 meters above sea level. The mine main activity is tin (Sn) ore extraction by underground operation and subsequent treatment at a metallurgical plant to obtain Sn concentrate as final product that is then sent to Minsur's Smelter in Pisco to produce tin bars to be sold to the international market. San Rafael's current concentrate plant capacity is 2,900 tons per day (tpd). Tailings are sent to TSF B3.

The Tailings B2 Reuse Project will be developed at this mine, with the purpose of reusing Sn tailings stored at old TSF B2 and extracting concentrate at a new tailing reuse plant. The tailings from such a process will be sent to a new storage facility, TSF B4, with a capacity of 7,670 million tons (Mt). The final concentrate will also be transported on closed trucks currently used at San Rafael and taken to the Smelter in Pisco.

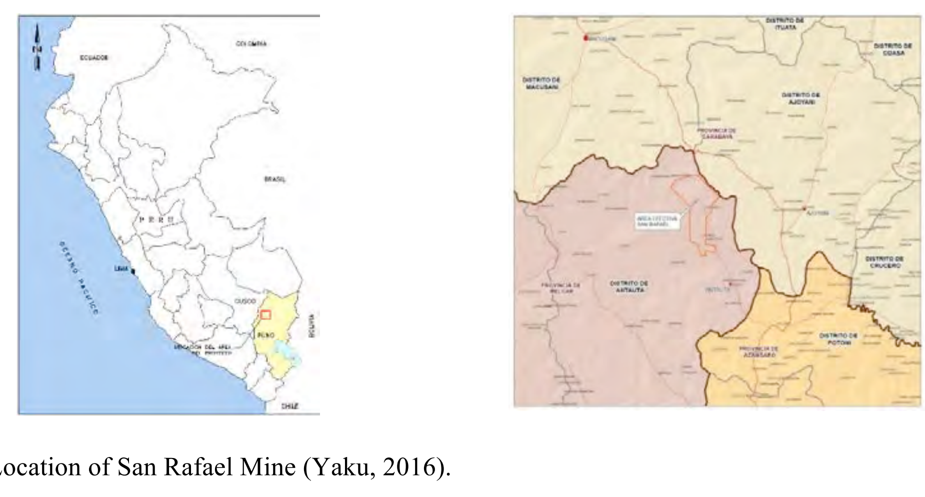

Figure 1. Location of San Rafael Mine (Yaku, 2016).

Photograph 1. B2 and B3 tailing reservoirs (Yaku, 2016).

The main components of the B2 Reuse Project are:

- Tailings Storage Facility B2, extraction process, and access ramp;
- Tailings Reuse Plant;
- Tailings Storage Facility B4 and related facilities;
- 10 KV distribution line and electrical substation expansion;
- Use of quarries for loan material;
- Expansion of camps and domestic sewage plant;
- The project's operation phase life will be 9 years, that is, from 2019 to 2027, and the construction phase will be of 2 years.

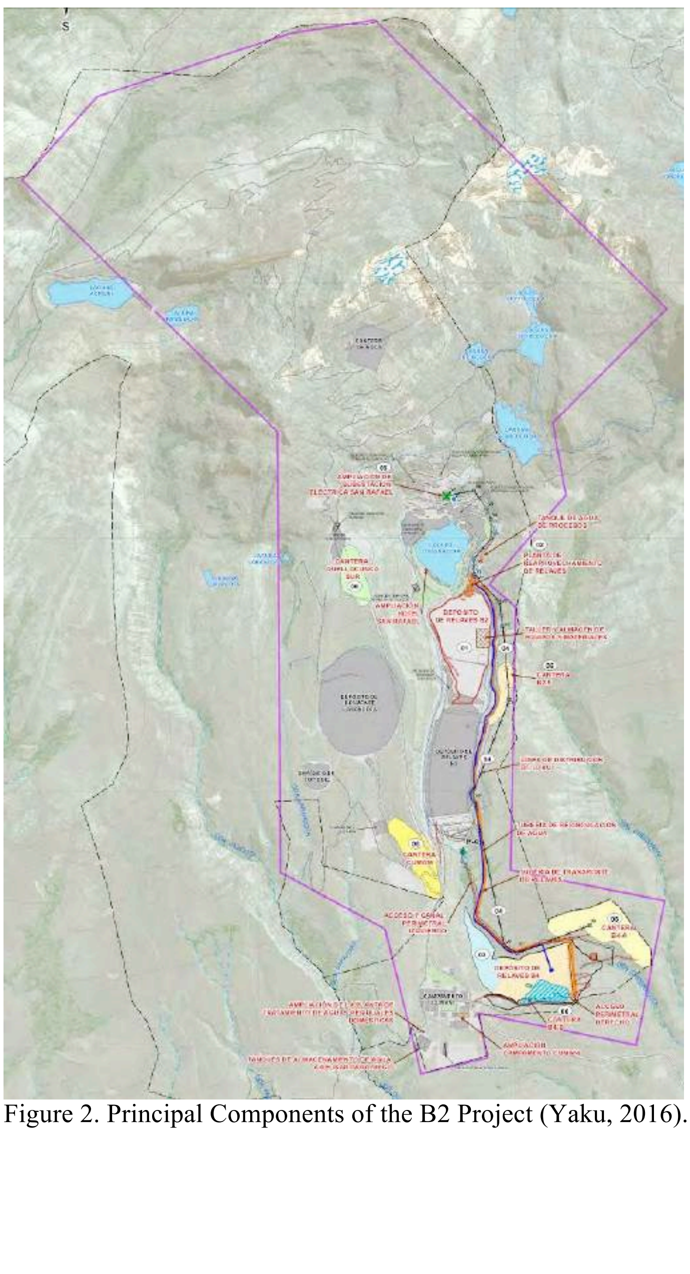
Figure 2. Principal Components of the B2 Project (Yaku, 2016).

2 TAILINGS EXTRACTION FROM OLD TSF B2

The project includes the extraction of tailings stored in old tailings storage facility B2 of 27.53 ha (480m, 825m long and 50m deep) through the conventional open pit method, and its processing at a new reuse plant to extract tin concentrate. The early activities will be: 1) a 10 percent slope access ramp 500 m long, 6.2 m wide to be implemented as tailings storage facility B2 is mined, 2) dewatering of tailings storage facility B2 through wells and 3) a drainage system.

A total of 7,619 million tons of tailings are estimated to be extracted and fed to B2 reuse plant for a 9-year period from 2019 to 2027.

Table 1 shows the amount of B2 tailings extracted during the 9-year operation to be reprocessed at the reuse plant.

Table 1. The number of officially reported plaque cases in the world.

Period	Stockpile 1			Stockpile 2			Total (kT)
	Thousands of tons (kT)	Sn (%)	Ratio	Thousands of tons (kT)	Sn (%)	Ratio	
Year 1 Month M1	45.87	0.94	0.29	25.63	0.82	0.49	71.50
Year 1 Month M2	34.51	0.67	0.30	36.99	0.70	0.43	71.50
Year 1 Month M3	25.72	0.74	0.29	45.78	0.64	0.41	71.50
Year 1 Month M4	35.19	0.75	0.30	36.31	0.68	0.40	71.50
Year 1 Month M5	47.69	0.82	0.28	23.81	0.72	0.56	71.50
Year 1 Month M6	34.13	0.82	0.28	37.37	0.75	0.63	71.50
Year 1 Month M7	29.77	0.78	0.30	41.73	0.72	0.58	71.50
Year 1 Month M8	16.45	0.94	0.24	55.05	0.71	0.57	71.50
Year 1 Month M9	28.88	0.90	0.24	42.62	0.73	0.55	71.50
Year 1 Month 10	34.06	1.09	0.19	37.44	0.75	0.53	71.50
Year 1 Month M11	42.51	0.96	0.21	28.99	0.77	0.51	71.50
Year 1 Month M12	13.39	0.94	0.27	58.11	0.80	0.50	71.50
Year 2 Quarter 1	85.36	1.07	0.23	129.64	0.78	0.49	215.00
Year 2 Quarter 2	110.99	1.02	0.24	104.01	0.81	0.64	215.00
Year 2 Quarter 3	97.32	1.17	0.18	117.68	0.93	0.61	215.00
Year 2 Quarter 4	97.11	1.19	0.16	117.89	0.93	0.62	215.00
Year 3	204.42	1.07	0.18	653.08	0.83	0.66	857.50
Year 4	371.63	1.33	0.18	485.87	0.92	0.66	857.50
Year 5	363.87	1.55	0.21	493.63	1.12	0.62	857.50
Year 6	28.29	1.21	0.32	829.21	1.06	0.63	857.50
Year 7	416.55	1.34	0.28	440.95	1.19	0.54	857.50
Year 8	294.45	0.97	0.20	563.05	1.16	0.60	857.50
Year 9	237.76	0.86	0.24	518.37	1.05	0.60	756.13
Total	2695.93	1.16	0.22	4923.20	0.99	0.61	7619.13

Source: Amec Foster Wheeler, 2016.

3 TAILING REUSE PLANT

The new tailing reuse plant will cover an estimated total area of 1.02 ha (0.445 ha overlaps the tailings storage facility B2 area), which includes the access area, and the fresh water and process tanks area. The tailings reuse plant treatment capacity will be 2,500 tons per day (tpd) and its operation phase will go from 2019 to 2027, in parallel with San Rafael’s process plant. Tailings resulting from the reuse plant process will be disposed of at tailings storage facility B4.

The tailings reuse plant will include a re pulping zone, a primary milling area (IsaMill), a gravimetry building, a flotation building and reactant zone, and a concentrate thickening, filtration and storage area. The water for the tailings reuse plant operation phase will come mainly from the recirculation of water recovered in TSF B4 and for the flotation phase, approximately 25 l/s of water will be required and will come from Lake Estancococha, for which Minsur already has a permit to use a maximum of 70 l/s.

4 TAILINGS STORAGE FACILITY B4

TSF B4 will be southeast and downstream of San Rafael operation's existing TSF B3, partially over Chognacota stream bed, covering an area of approximately 36.15 ha. TSF B4 will have a storage capacity of 7.67 Mt of tailings from the tailings storage facility B2 tailings reprocessing.

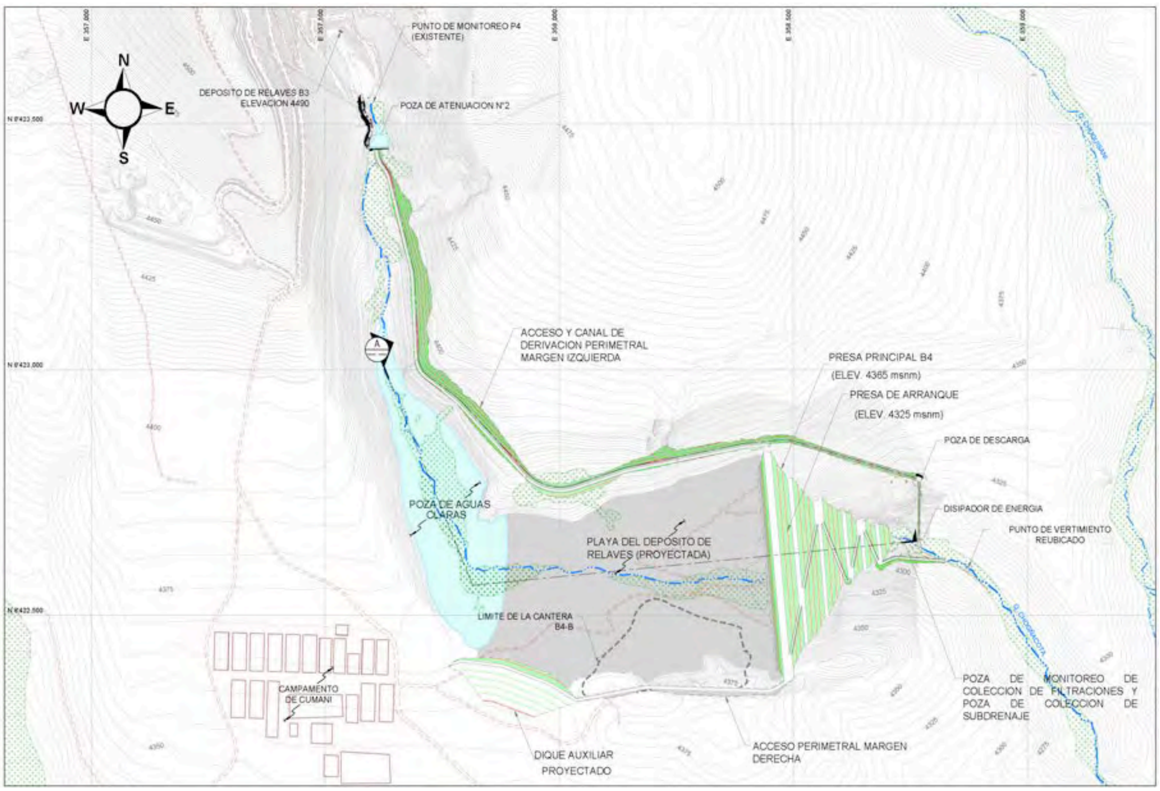

Figure 3. Tailing Storage Facility B4 (Knight Piésold, 2016).

TSF B4 will include a main dam or dike, made up of ordinary filler material, an auxiliary dam at the final stage, a tailings transport system and a recovered water recirculation system. This dam is designed as a dam capable of containing water, with a 35m high starter dam and a low permeability material central core. The starter stage considers the HDPE geo-membrane lining and temporary geomembrane lined shunting canals.

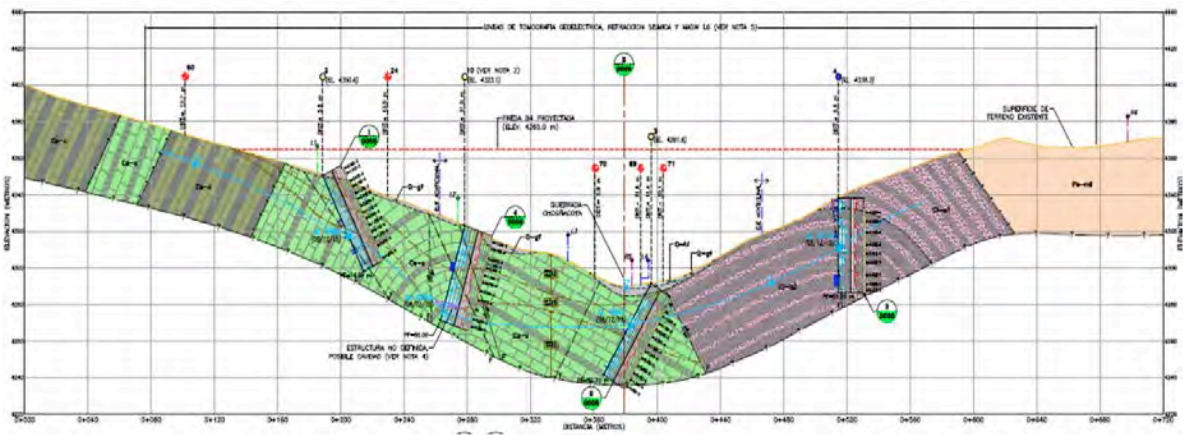

Figure 4. Tailing Storage Facility B4 – Cross Section (Knight Piésold, 2016).

The dam section will have a compacted ordinary filler main body, a low permeability material central core anchored to the foundation, as well as layers of filter / drainage and transition material. The vertical filter / drainage layers on the dam foundation will connect to a drainage mantle that will capture the leaks through the dam body and take them to a collection and monitoring pond at the slope toe downstream of the dam.

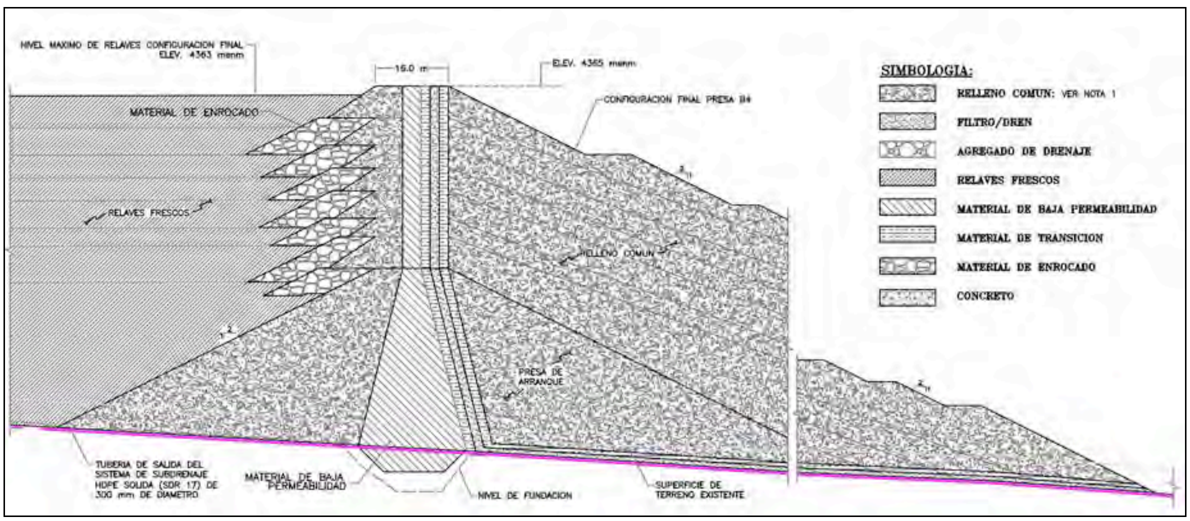

Figure 5. B4 Dam – Cross Section (Knight Piésold, 2016).

B4 dam will include a water recirculation system that will allow to pump water recovered in TSF B4 to the future tailings reuse plant for its reuse in the tailings reprocessing. Table 2 shows the geometrical characteristics of B4 dam:

Table 2. Geometric characteristics of dam TSF B4.

Characteristic	Starter Dam	B4 Dam
Crest width (m)	16	16
Maximum height (m)	35	75
Upstream slope	2H:1V	2H:1V
Downstream slope	2H:1V	2.7 H:1V
Crest elevation (masl)	4325	4365
Cumulative fill volume (m3)	455000	2255000

Source: Report the Project's Tailings Storage Facility Feasibility Study Knight Piésold, 2016.

5 THE WATER BALANCE OF THE B2 PROJECT AND SAN RAFAEL MINE

A water balance was developed to simulate the water management during the operation of the B2 Project and San Rafael mine in several scenarios. The Figure above shows the water balance considering both, the B2 Project and San Rafael mine operating at the same time.

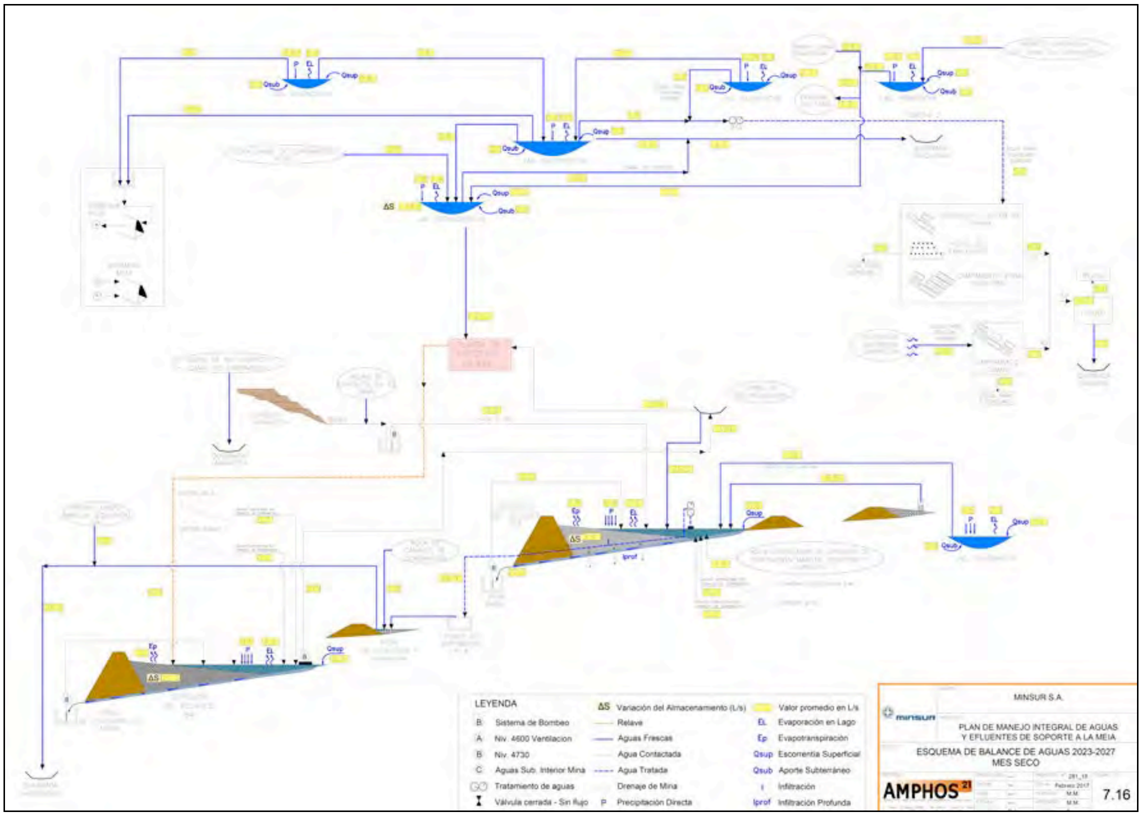

Figure 6. Water Balance – B2 Project and San Rafael operating at the same time scenario (AMPHOS 21, 2016).

6 WATER MANAGEMENT SYSTEM FOR TSF OPERATION

Recycled water from tailings storage facility B4 will be used for the tailings reuse process. Water from Lake Estancococha will be used for flotation only. Drainage water from TSF B2 mining will be sent to TSF B3 and water from TSF B4 dam will be pumped to TSF B3; then to the tailings reuse plant and San Rafael's process plant. Through such water management previously described, TSF B4 will make "ZERO" discharge to the environment.

Water surplus from TSF B4 that is not recycled to the reuse plant will be sent to TSF B3 operating system, which has discharge point (P4) to the environment and complies with The new tailings reuse plant will cover an estimated total area of 1.02 ha (0.445 ha overlaps the tailings storage facility B2 area), which includes the access area, and the fresh water and process tanks area. The tailings reuse plant treatment capacity will be 2,500 tons per day (tpd) and its operation phase will go from 2019 to 2027, in parallel with San Rafael's process plant. Tailings resulting from the reuse plant process will be disposed of at tailings storage facility B4.

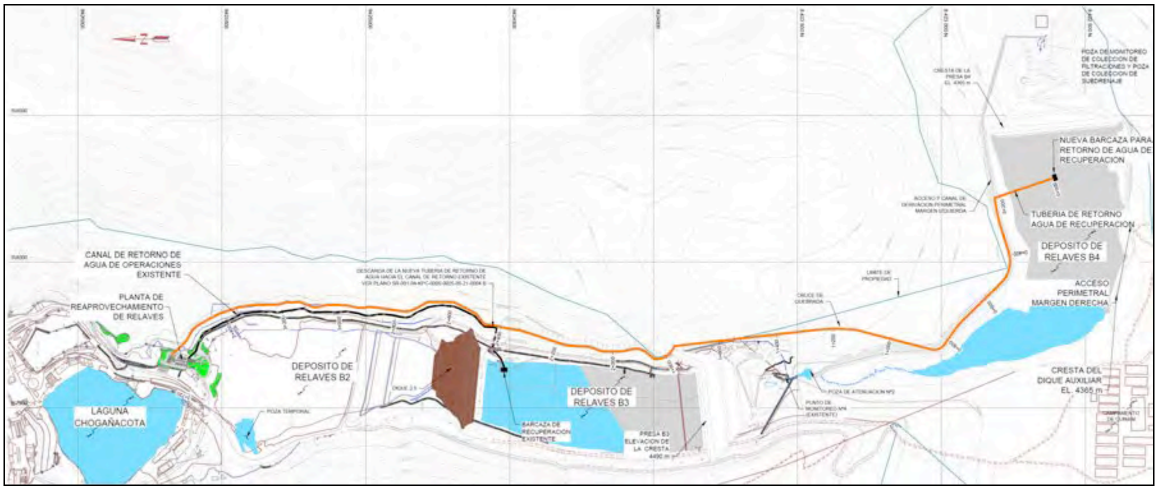

Figure 7. Water Management System of the TSF Operation (Yaku, 2016).

7 THE CONCENTRATE TIN PRODUCTION WITH THE B2 PROJECT IN SAN RAFEL

A water balance was developed to simulate the water management during the operation of the B2 Project and San Rafael mine in several scenarios. The Figure above shows the water balance considering both, the B2 Project and San Rafael mine operating at the same time.

Table 3. Production of concentrate

Plant	Capacity (tpd)	Law Sn (%) average	Production of concentrate (tpd)			Total production (tpd)
			2018	2019-2025	2026-2027	
San Rafael	3480	1.59	55.5	55.5	--	162060
B2	2500	1.05	--	51.0	51.0	167535

8 ENVIRONMENTAL IMPACTS IDENTIFIED IN B2 PROJECT

Table 4. Environmental Impact Classification of the B2 Project

Environmental Component	Impact	Construction Phase	Operation Phase	Closure Phase
Physical Environment				
Physiography	Change in local relief	Moderate	Moderate	Not applicable
Landscape	Change in landscape visual quality	Not significant	Moderate	Not significant
	Recovery of landscape visual quality	Not applicable	Not applicable	Moderate
Air	Change in air quality due to particulate material generation	Not significant	Not significant	Not significant
	Change in air quality due to gas emission generation	Not significant	Not significant	Not significant
Noise and vibrations	Increase of noise levels	Not significant	Not significant	Not significant
	Increase of vibration	Not significant	Not applicable	Not applicable
Surface water resources	Change in water quality due to increased sediments	Not significant	Not applicable	Not applicable

Environmental Component	Impact	Construction Phase	Operation Phase	Closure Phase
Underground water resources	Change in water course flow	Moderate	Moderate	Not applicable
	Change in water table and loss of springs	Moderate	Moderate	Not applicable
Soils	Soil erosion	Not significant	Not applicable	Not applicable
	Change of land use	Moderate	Not applicable	Moderate
Biological Environment				
Land flora	Loss of vegetation cover	Moderate	Not applicable	Not applicable
	Change in flora due to particulate material presence	Not significant	Not significant	Not significant
	Habitat fragmentation	Moderate	Not applicable	Not applicable
	Recovery of vegetation cover	Not applicable	Not applicable	Moderate
Fauna	Disturbance of wildlife	Not significant	Moderate	Not significant
	Loss of fauna habitat	Moderate	Not applicable	Not applicable
	Recovery of fauna habitat	Not applicable	Not applicable	Moderate
Hydrobiology	Change in aquatic flora and fauna communities	Not significant	Not applicable	Not applicable
	Loss of aquatic habitat	Moderate	Not applicable	Not applicable
Ecosystem	Loss of scrubland	Not significant	Not applicable	Not applicable
Fragile ecosystem	Loss of wetlands	Moderate	Not applicable	Not applicable
Social Environment				
Social	Potential increase of dust and noise due to vehicular traffic	Not significant	Not significant	Not significant
	Local employment generation	Moderate	Not applicable	Not significant
	Local market stimulation	Moderate	Moderate	Not applicable
	Transfer of canon (concession fee) and royalties to national governments	Not applicable	Moderate	Moderate
	Continuity of social investment	Moderate	Moderate	Not significant
	Change of habits and customs	Moderate	Not applicable	Not applicable
	Increase of expectations for more social benefits	Moderate	Not significant	Not applicable
	Reactivation of demands from direct social influence areas	Moderate	Moderate	Not applicable

Legend:

Serious positive importance	Moderate positive importance	Irrelevant positive importance or Not significant	Critical negative importance	Serious negative importance	Moderate negative importance	Irrelevant negative importance or Not significant

9 CONCLUSIONS

- During the construction and operation phases of the B2 Project, the social environmental impacts to be generated will be mostly moderate and not significant.
- The B2 Project will increase the production of tin concentrates and the life of the San Rafael mine in five more years, without extracting more ore but, will be reused the old tailings.
- The B2 Project will optimize the current mine facilities of San Rafael and recycling the water from the operation and without using more water than the authorized.
- The B2 Project will transform a potential environmental negative impact generator to a production positive impact for the mine.
- The B2 Project will eliminate a potential generator of negative impacts to the environment, allowing to improve the criteria for the closure of the San Rafael mine.
- The B2 Project is a Peruvian sustainable mining model that will improve the environmental and productive conditions of San Rafael and the local employment for surrounding villages.

REFERENCES

Amec Foster Wheeler, 2016. B2 Reuse Project of San Rafael Selection Study

AMPHOS 21, 2016. Integral Water Management Plan and MEIA Effluent Support.

Knight Piésold, 2016. Project's Tailings Storage Facility Feasibility Study.

Yaku Consultores, 2016. Modification of the Environmental Impact Assessment for the B2 Reuse Project.

Vegetation Establishment in Rock-Soil Mixtures for Evapotranspirative Covers

Jason Cumbers
Stantec, Fort Collins, CO, USA

Jesse Dillon
Cedar Creek Associates, Inc., Fort Collins, CO, USA

Stephen Dwyer
Dwyer Engineering, LLC, Albuquerque, NM, USA

ABSTRACT: Evapotranspirative covers or rock-armored covers are common solutions to stabilize tailings piles and mine materials in the arid southwestern United States. However, traditional covers for tailings or waste, were constructed with either rock-armored surfaces for erosion protection or vegetated surfaces when gentle slopes were part of the design. This case study will present vegetation results for a cover designed with a soil/rock mixture to protect a stabilized former uranium mine site in New Mexico, United States. The project included cover soils mixed with rock at 33% and 50% by volume that was subsequently seeded to provide a vegetated final surface. The mixtures were based on the coarse fragment size (33% by volume D_{50} = 2-inch rock and 50% by volume D_{50} = 3-inch rock) necessary to provide erosion protection. The case study will describe construction details of the cover, revegetation techniques used, and focus on vegetation performance in the soil and rock mixtures over the four years since the project was constructed. The objective of the case study is to provide an example of how rock/soil mixtures that contain a significant portion of rock can lead to successful vegetation establishment and describe the benefits that rock content can provide for vegetation establishment and sustainability on evapotranspirative covers for mine wastes in unpredictable precipitation regimes.

1 INTRODUCTION

The site is a former uranium mine located in a historic uranium mining district in New Mexico, USA. Past mining operations left behind mine waste, including soils, waste rock, and structural debris at the ground surface. The design and construction for cleanup and isolation of the materials from the surrounding environment beneath an evapotranspirative (ET) cover, was performed in accordance with a non-time critical removal action under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA). The project was required by the United States Forest Service (USFS) who also administered regulatory oversight during the design and construction of the project. Vegetation establishment of the ET cover is a key component of reviewing agency approval for the project and serves to aid in stabilization of the site from an erosional stability standpoint.

2 COVER DESIGN

The overall objectives of the cover design were to accommodate the external environmental processes while protecting the disposed materials without long-term maintenance. The ET cover profile was designed to minimize downward flux of meteoric water, limit radon emanation and minimize soil loss due to surficial erosion. The performance standards specified for the repository cover were 1) a minimum design life of 200 years, 2) percolation of no more than 3 mm per year for the wettest year on record 3) erosion of the cover surface limited to 2 tons/acre/year (445 mton/km^2/year), and 4) revegetation of the cover such that the revegetated area emulates the native plant communities and has a minimum ground cover of 80 percent of a natural vegetation analog area by the end of the 5-year maintenance period. The erosion protection and the site diversion channels were designed for the 100-year, 24-hour storm event.

The ET portion of the cover included the top slope of the cover designed with slopes of 5% or less. The cover was seeded with native vegetation designed to stabilize the soil against erosion loss and maximize moisture removal from the cover profile via transpiration. The resulting ET cover required to meet these requirements is thinner and therefore less expensive than a prescribed multi-layered cover system designed to meet the same requirements. The use of an ET cover allowed the flexibility to design the thickness of the cover layers to meet the overall project objectives for storage and transpiration while maximizing the use of near site soils.

The cover was designed with a 3.5 feet (1.1 m) thick profile, consisting of 2 feet (0.6m) of moderately compacted soil overlain by 1.5 feet (0.5m) of rock/soil admixture. The cover profiles are shown in Figures 1a and 1b. The surface rock/soil admixture was designed to minimize erosion, prevent rill and gully formation, provide a rooting medium for native vegetation, and provide storage capacity (in addition to the underlying cover soil) for infiltrated precipitation. This top layer is designed to emulate the aesthetic and protective qualities of a "desert pavement" layer typical of this arid environment.

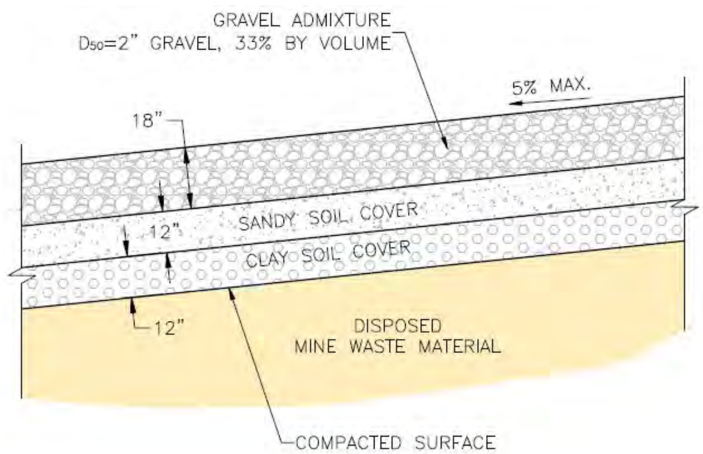

Figure 1a. ET cover details - top slope

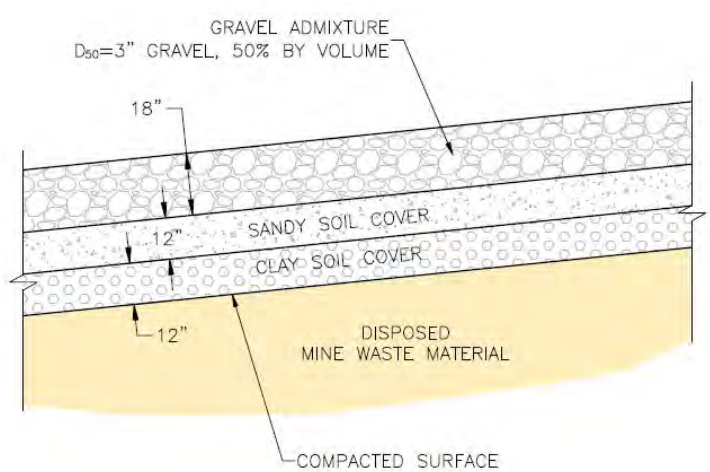

Figure 1b. Cover details - side slopes

One key benefit of an ET cover is the ability to use on-site soils for cover construction, thus limiting imported material and significantly reducing both environmental impacts from construction and construction costs. The layers of the cover system were designed to provide storage for infiltrated water until that water could be removed via ET. However, because the sandy soil had a relatively high saturated hydraulic conductivity, infiltrated water was able to rapidly advance through it. Therefore, a clay soil layer with a lower saturated hydraulic conductivity was included to slow the advancement of a wetting front to allow ET to remove the infiltrated water. The overall profile provides adequate storage capacity to minimize flux from moving below the cover and attenuates radon gas from the covered mine spoils.

3 CONSTRUCTION DESCRIPTION

Two different borrow sources were used for construction, one a sandy material and the other with about 20 to 30% more fines. The clayey layer of the cover consisted of approximately 43,000 cy (32,880 m^3) of soil sourced from an adjoining property, with a haul distance of approximately 3.5 miles (5.6 km) roundtrip. Approximately 75,000 cy (57,350 m^3) of sandy soil was available at the site for construction of the sandy and gravel admixture layers, but the material was located below soils with elevated concentrations of Ra-226. Therefore, construction of the sand and admixture layers for the cover required excavation of the overlying impacted soils and radiological confirmation that the area was cleaned up prior to any excavation for cover borrow could begin.

3.1 *Grading plan*

Following soil cleanup of the site, the materials were consolidated in one location for grading and burial beneath the cover. The overall grading concept is shown in Figure 2. The plan concept was intended to fit naturally with the surrounding topography, as it was understood to exist prior to mining, and includes upstream diversion channels to direct water run-on away from the repository. The grading includes a flatter top slope and steeper side slopes which resulted in variations in the erosion protection requirements for the two slopes.

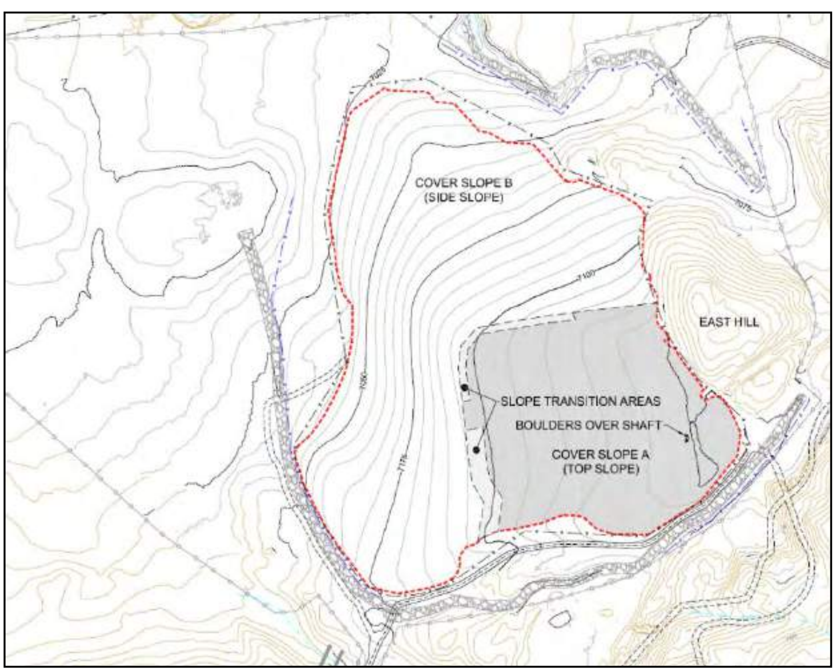

Figure 2. Cover surface grading plan

3.2 *Material descriptions*

Gradations for the two types of soil used in the cover layers are shown in Figures 3 and 4. Due to the variations in grades and the slope lengths, the erosion protection rock required for the top slope (median rock size (D_{50}) of 2 inches (50.8 mm)) was smaller than the rock required on the side slopes (D_{50} of 3 inches (76.2 mm)). The rock on both the top and side slopes was mixed with soil for vegetation. The specifications required 50 percent rock by volume on the side slopes and 33 percent rock by volume on the top slope, each mixed into the 18-inch (0.45 m) gravel admixture layer. An organic soil amendment for vegetation establishment on the cover was also to be mixed into the soil component of the cover.

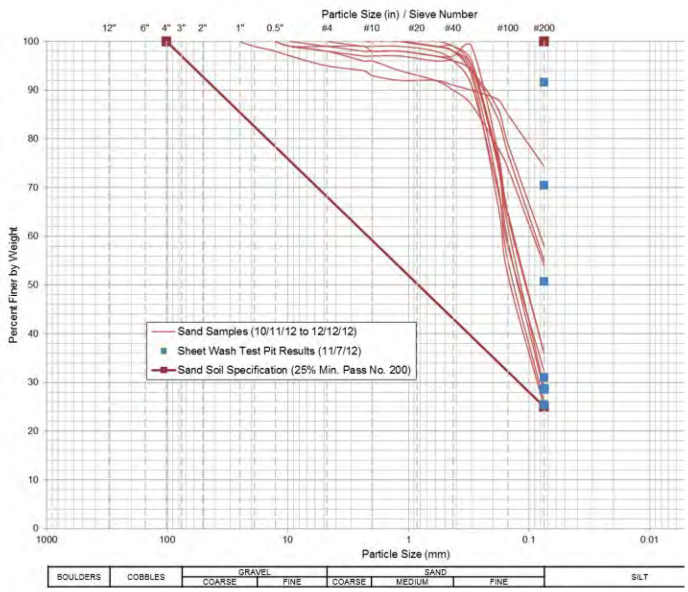

Figure 3. Sand Layer Gradations and Specification

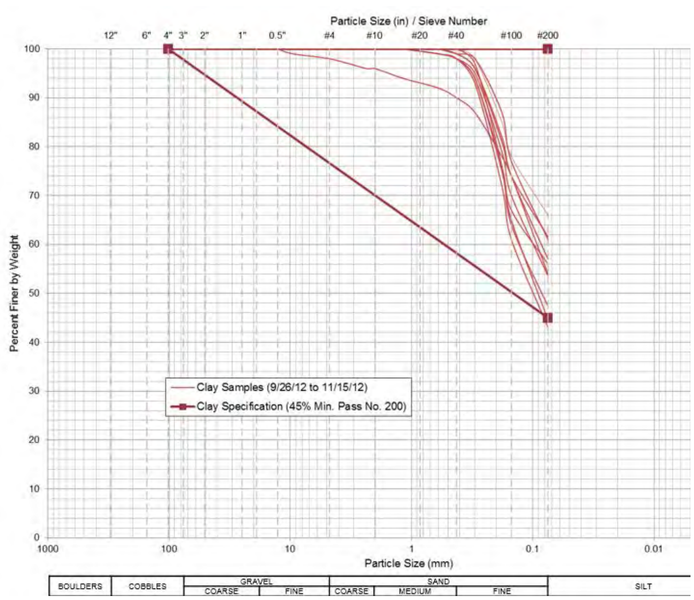

Figure 4. Clay Layer Gradations and Specification

3.3 *Soil layer thickness and compaction*

The cover construction was completed in layers. The two types of fill soil were placed in subsequent 12-inch (304.8 mm) layers. The thicknesses for the rock-soil admixture layers were calculated based on the volume requirements for the rock. The soil layer was placed initially and then covered by the rock layer. The two materials were then mixed together in place on the cover. The soil layers were placed in two separate lifts of 12-inches each and were required to be compacted to between 85% 92% of the standard Proctor density for the materials. The rock-soil admixture, once mixed, was track-walked in place. Although leaving the mixture too loose was a concern for erosion, the admixture layer was not compacted due to concerns that vibratory densification would inhibit vegetation establishment.

3.4 *Rock incorporation*

One of the unique aspects of the project was the method used for construction of the mixture layer of the ET cover. The selected contractor proposed to mix the layers of rock and soil in-place using a rotary mixer. The rotary mixer is equipment specifically designed for asphalt pulverization or soil stabilization by mixing. In this case, the mixer was equipped with a drum and carbide-tipped teeth specifically designed for mixing soil. The contractor placed the layers of soil and rock to thicknesses that would result in the correct volumetric components in the final mixture. The mixer would then pass over the layered materials and mix them in-place. The resulting gradations for the 2-inch admixture (soil and rock) are included in Figure 5.

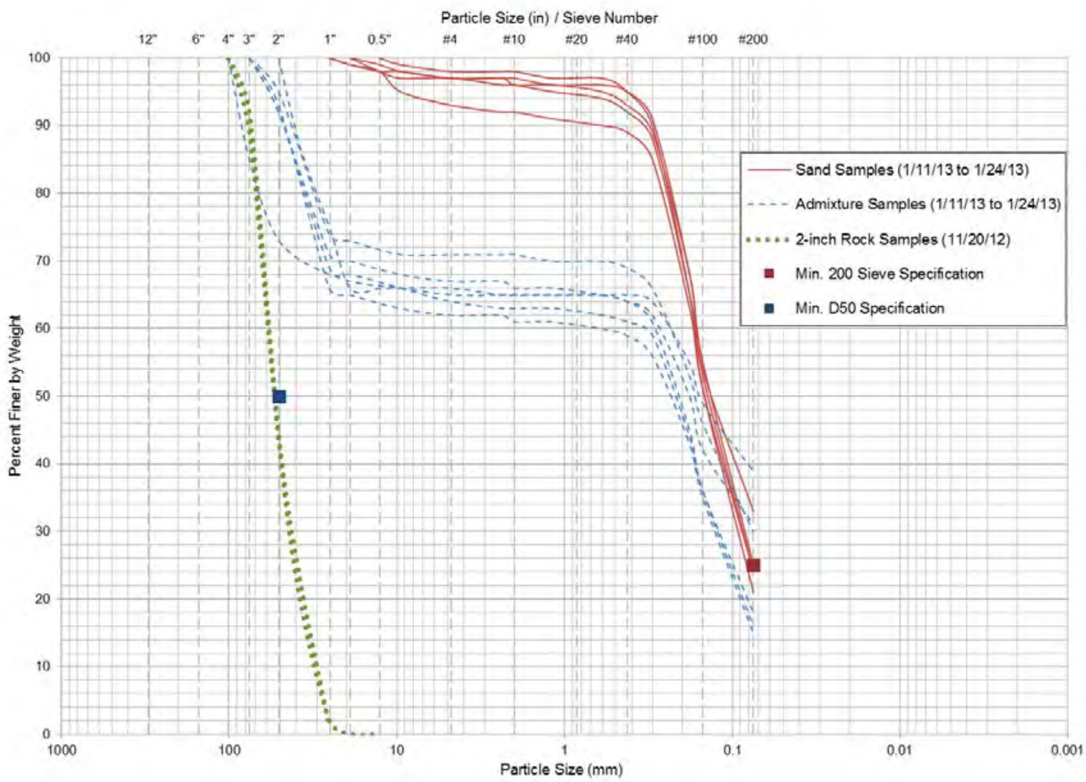

Figure 5. 2-inch Rock/Soil Admixture Layer Gradations and Specification

4 REVEGETATION DESCRIPTION

4.1 *Surface preparation*

Once the ET cover was graded to the approximate final configuration cap material and growth media on flatter slopes (less than3H:1V) was placed and compacted. Compaction on the final surface was limited to wheel-rolling only to enhance vegetation establishment. However, the upper 3 to 6 inches (8 to 15 cm) was prepared for seeding through fragmentation into a fine-grained unconsolidated material by use of a heavy disc, or heavy harrow, pulled by low-ground pressure equipment. This process oxygenates the media and opens up inter-grain poor spaces, facilitating moisture retention. Final passes were oriented parallel to the contour to preclude creation of preferential erosion pathways.

4.2 *Amendments*

Chemical analysis indicated that soils from the borrow areas displayed modest nutrient deficiencies which needed to be addressed to ensure plant germination and establishment on the ET cover. The primary nutrient of concern was phosphorus, although nitrogen and organic carbon levels were also low. To correct these deficiencies, the sand portion of the gravel admixture of the ET cover received 16 cubic yards per acre of sterile organic mulch (compost) on the top slope and 12 cubic yards per acre of sterile organic mulch (compost) on the side slopes. This incorporation was performed during mixing, and prior to final seeding. The incorporation of sterile organic mulch was intended to provide improved nutrient availability, nutrient cycling, and soil moisture retention (Brady and Weil, 1999).

4.3 *Seed mix*

Seeding on the ET cover consisted of native species, which were adapted to local climatic and edaphic conditions. The seed mix for the ET cover consisted of only grasses and forbs (Table 1), which were applied via drilling and broadcast methods.

Table 1. Evapotranspirative Cover Seed Mix

	On Site	Common Name	Scientific Nomenclature	PLS / lb.	Recommd. PLS lbs/ac	PLS / ft^2
1		Western Wheatgrass	*Agropyron smithii*	110,000	2.00	5.1
2		Sideoats Grama	*Bouteloua curtipendula*	191,000	1.00	4.4
3	XX	Blue Grama	*Bouteloua gracilis*	825,000	1.00	18.9
4	XX	Galleta	*Hiliaria jamesii*	159,000	2.00	7.3
5	XX	Indian Ricegrass	*Oryzopsis hymenoides*	141,000	2.00	6.5
6		Bottlebrush Squirreltail	*Sitanion hystrix*	192,000	1.00	4.4
7		Alkali Sacaton	*Sporobolus airoides*	1,758,000	0.25	10.1
8	XX	Sand Dropseed	*Sporobolus cryptandrus*	5,298,000	0.10	12.2
			Grass Subtotal		9.35	68.9
9		Purple Prairie Clover	*Dalea purpurea*	300,000	0.50	3.4
10		Lewis Flax	*Linum lewisii*	293,000	0.50	3.4
11		Firecracker Penstemon	*Penstemon eatonii*	900,000	0.25	5.2
12		Rocky Mtn. Penstemon	*Penstemon strictus*	592,000	0.25	3.4
13		Prairie Coneflower	*Ratibida columnifera*	737,104	0.50	8.5
14	XX	Scarlet Globemallow	*Sphaeralcea coccinea*	500,000	0.25	2.9
			Forb Subtotal		2.25	26.8
			Total		11.60	95.7

PLS = pure live seed

5 MONITORING RESULTS

The project was observed full-time for quality assurance during construction. Following completion of construction in February 2013, a period of observation and maintenance began. For the first three years (2013-2015) the site was inspected quarterly, followed by semi-annual inspections for the next two years (2016-2107) as the vegetation began to establish. A native vegetation analog area is located in an undisturbed area adjacent to the project site. This area represents a suitable comparison target for revegetation performance.

The point-intercept technique is used to measure vegetation establishment. This technique uses 10 meter transects made up of 100 points. At each point, a hit on vegetation (by species), litter, rock, or bare soil is recorded. Results for each transect yield a percent ground cover for each plant species, litter, rock, or bare soil encountered. Transects are distributed across the sampling units in a systematic grid.

5.1 *Erosion inspections*

Minor erosion damage was stabilized in years one and two using rock check dams, straw bales, and minor earthwork to repair rills and gullies. However, most of this repair work was done along transitions in slope, near the edges of the cover. Erosional impacts observed to date on the ET cover itself have been minor and have decreased as vegetation has established.

5.2 *Rock cover results*

Rock cover was quantified during vegetation monitoring. Rock cover is defined as the percent of the soil surface which is composed of rock. It is derived from the proportional ratio of rock to bare soil. Ground cover monitoring occurred in the 2nd, 3rd, and 4th growing seasons after the ET cover was completed. Rock cover on the slopes has increased each monitoring effort from 8% in 2014, 11% in 2015, to 14% in 2016.

5.3 *Vegetation results*

Vegetation monitoring has been conducted on the ET cover annually in September, following monsoonal rains, from 2014-2016 (years 2-4) to evaluate progress of the developing revegetation. Desirable perennial cover has increased each monitoring effort on both the ET top slope and the side slopes of the cover (Fig. 6). Very favorable growing season precipitation (195% of average) is responsible for the significant vegetation cover increase observed in 2015 (Year 3). As expected, the revegetation on the top ET surface of the cover is outperforming the slopes. This is a result of elevated plant available water on the top surface. A portion of the precipitation on steeper slopes becomes overland flow and is transported offsite, whereas most precipitation on gentler slopes becomes soil water which is either evaporated or used by plants.

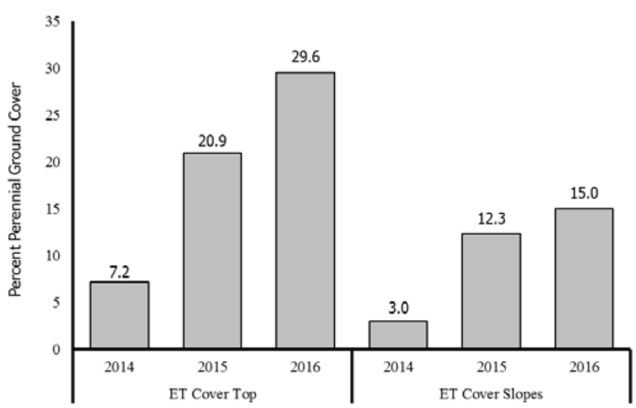

Figure 6. Perennial Cover Progression

Vegetation monitoring has also revealed revegetation lifeforms, which were comprised primarily of grasses and shrubs (Table 2), even though shrubs were not seeded. The vegetation composition and structure are similar to the native analog area in terms of percentages of perennial grasses and shrubs. The plant diversity is generally better with more species identified on the ET cover than in the analog area. Revegetation on the cover top surface is already exceeding the performance criteria (80% of the perennial vegetation cover native analog area in Year 5) prior to the end of the maintenance period. The side slopes of the ET cover are on a trajectory to exceed performance criteria in Year 5. Overall, revegetation establishment and development on the ET cover has been successful in less than a five-year monitoring period.

Table 2. 2016 (Year 4) Average Cover Summary

	ET Cover Top	Side Slope	Analog
Summary by Lifeform (% Ground Cover):			
Perennial Grasses	17.47	13.00	16.66
Annual Grasses	-	-	-
Perennial Forbs	0.20	-	-
Annual & Biennial Forbs	10.00	1.47	-
Noxious / Aggressive Weeds	-	-	-
Shrubs, Sub-shrubs, Cacti & Trees	1.93	0.40	3.13
Diversity (Number of Species with ≥1% Average Cover):			
Number of Important Species =	7	5	3

6 CONCLUSIONS

ET covers provide an alternative to monolithic rock covers for erosional stabilization. Rock covers have traditionally been used for erosion protection on mine tailings impoundments. However, cost implications and environmental impacts related to the need for large volumes of rock, in regions with limited access to durable rock sources, lead to the application of ET covers in a broader range of situations. The flexibility of the ET cover design allows for optimization of cover thicknesses to achieve the overall objectives for storage and transpiration while maximizing the use of near site borrow sources. In most cases, however infiltration and the vegetation component of the transpiration model can be a concern. The data presented for the ET cover case study in this paper helps to alleviate concerns that vegetation can be successfully established both in rock-soil mixtures with up to, and greater than, 33% rock by volume and in specific arid climates.

6.1 *Implications of rock incorporation in erosion control and vegetation performance for covers*

As soil consolidation and soil erosion occurs on the ET cover, surface rock cover increases as more of the incorporated rock within the mixture becomes exposed. The increased surface rock cover provides increased erosion protection through rain splash protection and surface stabilization. Essentially, incorporation of rock in the ET cover creates self-armoring and self-healing slopes; as soil erosion progresses materials are often deposited in lower rills and gullies between rocks.

The incorporated rock has also helped to establish a diverse revegetation community (Stark and Redente, 1985). Fine textured soils keep plant available water near the surface, where herbaceous vegetation has a competitive advantage. Mixing rock into the rooting profile enables a portion of the meteoric water to percolate deeper, where shrubs have a competitive advantage. As a result of the incorporated rock, the revegetation community on the ET cover is a stable and natural ecosystem. The species and lifeform composition is very similar to the native analog site which suggests that revegetation communities on the ET cover are sustainable in the climatic conditions of New Mexico, USA.

6.2 *Considerations for arid climates*

Incorporated rock in the ET cover increases plant available water in the rooting zone by providing rock mulch at the surface which limits potential evaporation, and creates preferential flow pathways which allow water to percolate deeper in the profile, where evaporation via capillary action is constrained. Both of these circumstances create opportunities for plants to utilize additionally available precipitation which can be of vital importance to vegetative sustainability in drought periods where competition for limited resources can lead to plant mortality.

6.3 *Considerations for episodic precipitation regimes*

Episodic precipitation regimes can lead to high intensity and short duration storm events which can be particularly destructive from an erosional standpoint. Since there is generally low average precipitation and high evaporation in this arid climate, vegetation will unlikely ever be robust enough to preclude erosion under high intensity precipitation events. Therefore, the incorporation of some rock in the cover system is not only beneficial for vegetation establishment, but necessary for overall erosion protection.

ACKNOWLEDGEMENTS

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Revegetation of tailings soil using local and native grasses, shrubs and tree species

B.T. Abela & R.C. Favila
Coral Bay Nickel Corporation, Palawan, Philippines

ABSTRACT: The 71-hectare reddish-brown tailings soil of the year 2010 decommissioned Tailings Storage Facility No. 1 (TSF-1) required immediate and fast revegetation. Heavy siltation on rainy days and its ugly image need immediate solution. Three species of local grasses were grown in potting bags prior to planting and four species of shrubs were directly planted. The soft tailings soil can't carry heavy equipment, so works were done manually with the indigenous people as the main labor force. After two years of planting, the whole area was totally covered with grasses and shrubs. Siltation is controlled, green landscape recreated and much needed biomass produced. In 2013, trees were next planted where the TSF-1 area in December 2016 has 94,338 local and native tree species that are growing sustainably at a survival rate of at least 90%. Continuous plant care and maintenance will be done for at least five years more.

1 INTRODUCTION

Coral Bay Nickel Corporation (CBNC) in Palawan, Philippines operates a hydrometallurgical processing plant (HPP) that applies the high pressure acid leach technology (HPAL) starting in April 2005. The HPAL process utilizes sulfuric acid to leach out nickel and cobalt from laterite nickel ore and the dissolved nickel and cobalt is then subsequently precipitated as mixed sulfide with the use of hydrogen sulfide. Final neutralization is the last step of the process to produce neutral tailings and to precipitate the dissolved metals as oxides. This is done by the use of limestone and slaked lime. The neutral tailings that is a thin slurry with about 70% water is pumped into the TSF-1 which is bounded by a 2.5km long dam in the west, south and east perimeters. The TSF-1 north perimeter is a naturally high ground. Dam height ranges from 25 – 32m. The tailings storage location is a mined-out area where the raw ore was actually mined.

TSF-1 was filled to its maximum capacity and eventually decommissioned in June 30, 2010. The 71 hectare (ha.) surface area of tailings soil that is reddish-brown due to its high iron content does not look good and produces heavy siltation during rains. The wide TSF-1 ugly looking and bare landscape is a favorite google image to use by anti-mining advocates to support their clamor to stop mine operations in Palawan. Company management decided to do rehabilitation in order to address these serious concerns. Rehabilitation works was started in March 2011, 8 months after decommissioning when the soil surface is already relatively dense due to moisture evaporation and hard enough to step on by the workers.

1.1 *TSF-1 rehabilitation objectives*

The revegetation of tailings soil is in accordance to the overall TSF-1 rehabilitation objectives which are as follows:

- *Physical stability.* The dam must be stable and the tailings soil must not cause siltation during rainy days nor dust emissions during sunny days.
- *Visual acceptability.* The negative aesthetic impact of the reddish-brown tailings soil must be eliminated.
- *Productivity.* The rehabilitated area must be in a condition where its former ecosystems services are restored and other forest and agricultural products are produced.
- *Self-sustainability.* The end goal of the rehabilitation of TSF-1 is to restore a functional ecosystem. The rehabilitated works will be finish only when the rehabilitated area is found self-sustainable.

2 METHODS

2.1 *Tailings soil condition*

The HPAL process which is at high temperature eliminated all beneficial microorganisms in the tailings which is made of fine soil particles and tend to form hardened clay when dried. Surface covering with top soil as usually practice in mined-out area rehabilitation site adjacent to TSF-1 could not be applied since the soil volume requirement is too big and the tailings soil with high water content as discharge is very soft and creates big cracks as its water naturally evaporates by sun drying. This condition makes it difficult to use heavy equipment within the rehabilitation site so manual labor is used in practically all works, except in hauling of materials from the source to the rehabilitation site perimeter. Manpower is readily available though as many indigenous peoples (IP's) in the nearby villages need employment. Soil sampling and analysis was conducted prior to planting trials to determine appropriate species for surface covering. Table 1 shows the plant nutrients and other elements present in the tailings soil.

Table 1. Average nutrients and other elements* composition of tailings soil

No.	Parameter	Unit	Date of sampling 02-09-2011	10-20-2012	10-05-2013	02-18-2017
1	Moisture, H_2O	%	42.23	41.20	38.50	38.80
2	Nitrogen, N	%	0.02	0.14	0.00	0.05
3	Phosphorous, P	%	0.03	0.01	0.02	0.02
4	Potassium, K	%	0.00	0.03	0.00	0.00
5	Carbon, C	%	0.31	0.75	0.82	0.81
6	Sulfur, S	%	5.44	5.88	7.70	5.72
7	Calcium, Ca	%	6.67	8.51	7.79	8.65
8	pH		8.72	8.66	8.77	7.76

* Most elements are present in the soil as oxides. Ca and S are present as gypsum.

Other elements like iron, magnesium and manganese which are in the form of oxides and zinc in sulfide form are present in the tailings soil in varying quantities sufficient to supply the secondary nutrients requirement of the plants.

The primary nutrients, Nitrogen, Potassium and Phosphorous (NPK) were observed to be deficient. To supply the deficient NPK, triple-14 inorganic fertilizer is applied while vermicompost is used to re-introduce beneficial microorganisms to the soil. To enhance growth of beneficial microorganisms and to keep the soil loose, carbonized rice hull (CRH) is mix with the soil media.

2.2 *Vermicompost and carbonized rice hull production*

Vermicompost is produced at the perimeter of the rehabilitation site as part of the rehabilitation project. 15 vermicomposting beds with a size of 0.5m high x 1.5m wide x 3m long each were used that is capable of producing 10 cubic meters of vermicompost in a month. Composting materials are rice hay and cow dung which is stack layer by layer in a way that the composting bed contains about 70% by volume rice hay and 30% by volume cow dung. 3 kg of composting earthworm, that is African night crawlers is being introduced in each composting bed. Within a period of 2 months, the hay and cow dung mixture is decomposed into an organic fertilizer with abundant beneficial microorganisms.

CRH is produce by means of controlled burning of rice hull. CRH burner is made of a half metal drum with 10-mm holes, and chimney attached to the base and to be use upside down or inverted. Burned firewood is put under the CRH burner and the raw rice hull is put on top and surroundings of the burner. Carbonization is complete when the yellow color of the rice hull turned black. The blackened rice hull is separated from the burner and poured with water so as not burn into ash.

2.3 *Grass seedlings production*

For growing seedlings of grasses and vines as well as seedlings of trees, the soil media being used is composed of the following mixing ratio: 70%v top soil : 10%v vermicompost : 20%v CRH. Top soil is bought from farm owners at the farmlands surrounding the rehabilitation site. About 1-liter volume of the fertile soil media is put into 15cm x 20cm size potting bags where the grass seedlings are grown for 3 – 4 weeks prior to transplanting. Grasses and vines seedlings grown are from seeds or small grass shoots taken from the wild.

2.4 *Planting grasses and shrubs as tailings soil surface cover*

Planting of the locally available grasses, vines and shrubs were tried and found growing fast and sustainable. The purpose of planting grasses is to have a surface cover fast and to improve the nutrients for plants in the tailings soil. Table 2 is the list of tailings soil surface cover plants.

Table 2. Species of grasses, vines and shrubs planted as surface cover

No.	Common name	Scientific name	Description and comments
1	Humidicola	*Brachiara humidicola*	A strong creeping perennial grass which forms a dense matted sward that grows well in infertile soils and full sunlight.
2	Stylo	*Stylosanthes sp.*	An erect, branching, green, bushy, 40-70cm tall perennial legume with woody base. It is tolerant to prolonged dry season. It is suitable in clay soils.
3	Centro	*Centrosema pubescens*	A perennial, twining and trailing legume which will climb associated grasses and plants.
4	Napier grass	*Pennisetum purpureum*	Tall, leafy type used in cut-and-carry systems. It grows in wet areas. This grass, humidicola, stylo and centro are forage plants for potential livestock farming in the future.
5	Banana	*Musa sp.*	Its wide leaves can enhance fast greening of the area and its fruit will attract fauna later.
6	Elephant ear	*Xanthosema robustum*	Like banana, the wide leaves can enhance fast greening.
7	Pandan	*Pandanus sp.*	It is a palm that usually grows near creeks. These was planted in low areas that have water lagging during rainy season.
8	Diverse other plants	-	The top soil that was taken from farm lots in nearby villages naturally contains slips and seeds that also grew and enhance biodiversity.

A small hole is dug using a pointed wooden pole with a size similar to that of the seedling pot. The seedling pot is then shot into this hole. NPK input of about 10g was added on the planting hole during transplanting.

Figures 1-2 show the planting lay-out. The 10m x 10m plot is established where the area is first planted with grasses and within the grasses the shrubs are inter-planted. This 100 square meter plot was replicated all over the tailings soil area.

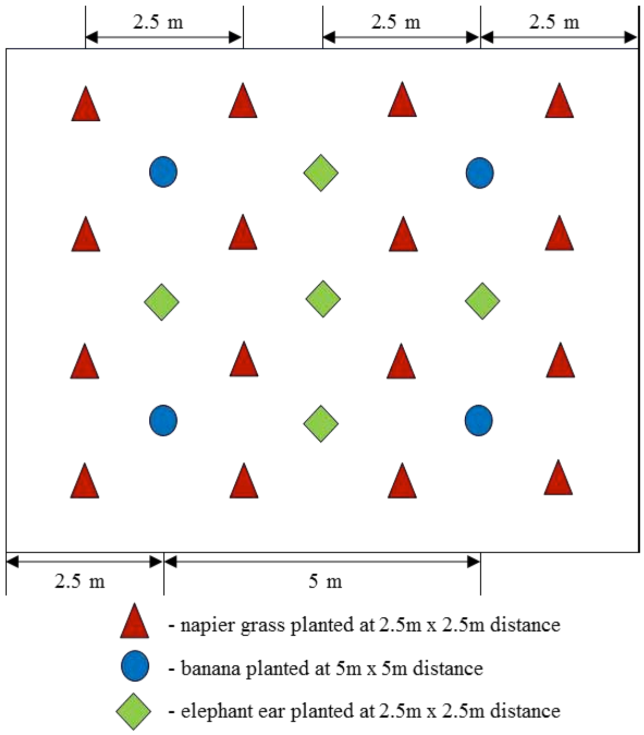

Figure 1. Planting plot for shrubs in every 10 m^2 area (illustration not to scale)

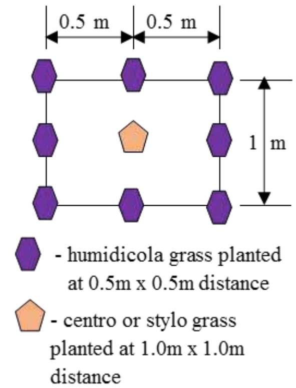

Figure 2. Planting plot for grasses (illustration not to scale)

Figure 3 below shows the grasses and shrubs as initially planted.

Figure 3. Planting lay-out with newly planted shrubs and grasses

Banana suckers, napier grass cuttings and elephant ear wildlings are directly planted while humidicola, stylo and centrosema are reared in pots prior to transplanting. Fertile soil input to a banana planting hole is about 5 liters while fertile soil input for napier grass and elephant ear is about 1 liter. The yellowish material that filled the cracks as shown in Figure 3 is raw rice hull.

Rice hull used was acquired from rice mills located within about 15km away from the rehabilitation site. Rice mill owners gladly gave the waste rice hull for free because this helped them in disposing the waste rice hull that accumulated at the back yard of the rice mills. The supply of raw rice hull from the rice mills was not enough for the whole rehabilitation area so this was only used for a limited number of planting plots. The raw rice hull as additional input was observed to enhanced faster plant growth.

The tailings soil where grasses are planted always contains moisture and never runs dry even during prolonged sunny days. When the tailings soil surface water content is reduced by evaporation, the soil surface becomes highly dense and hard serving as an insulator that prevents subsurface soil water evaporation. The insulating characteristic of the surface soil however resulted to cracks as the moisture from the deeper part of the tailings scape to the atmosphere. During prolonged sunny days, water is still available for plants which roots that already reach about 15cm deep from the surface. The wet condition of the soil helps much in sustaining continuous growth of plants all throughout the year, even during the 6 months dry season which is usually from December to May of each year.

3 GRASS PLANTING RESULTS

The planted grasses and shrubs grew sustainably that in a period of twenty eight (28) months, from March 2011 to July 2013, the whole 71-hectare tailings soil surface area was already covered. Silted water run-off during rainy season was eliminated and the previous reddish-brown color already turned green. Additionally, these initial grass plants produced the much needed bio-mass that enhanced soil fertility. Banana, napier grass, humidicola and centro are the dominant shrubs and grasses that grew sustainably. The elephant ear and stylo in most areas were choked by the fast creeping humidicola. After observing the performance of the plants for one year where about 40% of the area had been planted already, elephant ear and banana were not planted anymore. Banana planting was discontinued because this required more inputs and care as compared to the other plants. Napier grass was found to be growing fast without additional input of fertilizer. When Napier grass grow to about 2 meters tall, these are cut at the base and the cut grasses are allowed to decompose on the ground for the purpose of biomass production. After 3 months from cutting, the cut napier grass will again grow to at least 1m tall.

In about ten hectares of low-lying areas, particularly the pond area during the TSF operations, the plants died due to water lag during rainy season of July – September of 2013. Pandan plant (Pandanus sp.) was planted as replacement since this usually grows near waterbodies like swamps and creeks. On the succeeding dry season more surface soil cracks were created during tailings soil moisture evaporation resulting to no more water lagging when the next rainy season came. The pandan still grew well since the water still stays in the cracks for two to three months during rainy season creating some kind of small creeks in the area which favors the sustainability of pandan.

The established grass land created a potential livelihood. Napier grass, humidicola and centrosema are fodders for livestock while pandan leaves are being used as material for making mats, baskets, bags, fans and hats. A small cottage industry for making mats and bags from pandan leaves started in 2016 by the IP's who were also the workers planted this palm.

4 TREE PLANTING

Beginning June 2013, tree planting on TSF-1 tailings soil commenced. Tree species chosen for planting are local and native trees that were observed to grow well in the tailings soil in the conducted trial plots. Major consideration for the choice of pioneer tree species is fast growth, able to enhance soil fertility and potential for economic benefit later.

Trees planted from June 2013 to December 2016 are listed in Table 3.

Table 3. Species and quantity of trees planted at TSF-1 rehabilitation area

No.	Common name*	Scientific Name	Number of trees
1	Ipil-ipil	*Leucaena leucocephala*	42,281
2	Narra	*Pterocarpus indicus*	26,371
3	Palawan agoho	*Gymnostone nobile*	11,046
4	Kakawate	*Gliricidia sepium*	5,724
5	Udling	*Eugenia sp.*	1,606
6	Syzygium	*Syzygium sp.*	1,120
7	Aripa	(to be determined)	651
8	Batino	*Alstonia macrophylla*	570
9	Palawan mangkono	*Xanthostemon palawanensis*	549
10	Ipil	*Intsia bijuga*	524
11	Palawan amayan	*Angelesia palawanensis*	508
12	Bitanghol or Palumarya	*Calophyllum blancoi*	449
13	Aratilis	*Muntingia calabura*	350
14	Duhat	*Syszygium cumini*	300
15	Cashew	*Anacardium occidentale*	256
16	Guava	*Psidium guajava*	238
17	Star apple	*Chrysophyllum cainito*	210
18	Palawan cherry	*Prunus javanica*	177
19	Bangkal	*Nauclea orientalis*	165
20	Alupag	*Litchi sp.*	143
21	Rambutan	*Nephelium lappaceum*	140
22	Guyabano	*Annona muricata*	107
23	Durian	*Durio zibethinus*	101
24	Langka	*Artocarpus heterophyllus*	81
25	Yemane	*Gmelina arborea*	78
26	Malugay	*Pometia pinnata*	68
27	Kalamansi	*Citrus microcarpa*	64
28	Bignay	*Antidesma bunius*	51
29	Mango	*Mangifera indica*	42
30	Kape	*Coffea Arabica*	40
31	Bunog	*Garcenia benthami*	39
32	Mahogany	*Swietenia macrophylla*	34
33	Marang	*Artocarpus odorassitimos*	28
34	Katmon bugtong	*Dillenia luzoniensis*	22
35	Kamansi	*Artocarpus camansi*	22
36	Dita	*Alstonia scholaris*	21
37	Malakape	*Canthium dicocum*	15
38	Manggis	*Koompassia excels*	15
39	Kandis	*Garcinia binucao*	14
40	Magparay	*Timonius sp.*	14
41	Palawan Dita	*Alstonia iwahigensis*	14
42	Linatog	*Eurycoma longifolia*	13
43	Bakaw-bakaw	*Psychotria sp.*	12
44	Marangub	*Protium connarifolium*	12
45	Dukloy	*Polyscias aherniana*	11
46	Amugis	*Koordersiodendron pinnatum*	10
47	Maglandak	*Magnolia sp.*	10
48	Kapok	*Ceiba pentandra*	10
49	Buga	(to be determined)	9
		Total	94,338

* Common names as given by locals and IP's are adapted.

4.1 *Tree planting strategy*

Tree species chosen for planting are local and native trees that were observed to grow well in the tailings soil in the conducted trial plots. Wildlings and germinated seeds are nurtured in the nursery in 15cm x 20cm potting bags for 4 – 8 months prior to transplanting. Planting distance for most trees is 2.5m x 2.5m. The largest number of trees planted is Ipil-ipil (*Leucaena*) since it is fast growing, has the potential to renew soil fertility and will serve as a shade plant for native trees that will be inter-planted later with the Ipil-ipil to enhance the rehabilitation area biodiversity. Second largest number of trees planted is Narra (*Pterocarpus*) which is a biological nitrogen fixer (BNF) like Ipil-ipil. Third largest number of trees planted is Palawan agoho (*Gymnostone nobile*) which is one of the dominant tree species in the nearby forest and in the actual rehabilitation area prior to mine development. Ipil-ipil, Narra and Agoho are inter-planted to enhance biodiversity. Kakawate (*Gliricidia sepium*) is the fourth most number of trees planted. Kakawate poles where actually used as markers for planting lay-out but these eventually sprouted and survived the soil condition since it is also a BNF. Most of the listed tree species in smaller numbers are the native trees, including fruit trees found in the nearby forest. Fourteen fruit tree species were already planted as of December 2016 and five of these started bearing fruit after three years of planting. Fruit trees planting is one of the major strategies in order to attract birds and bats, and other faunal species to the re-created forest. Continues collection of native tree wildlings is being done with the target of planting all native tree species found in the forest adjacent to the mined-out area. If no wildlings are found and no seeds are available, seedlings propagation is done in the clonal laboratory.

Every planting hole of 15-cm deep, 20-cm wide and 20-cm long is poured with about 5 liters fertile soil media and about 30g of triple-14 NPK as basal fertilizer. Subsequent fertilization were later done two times, every 3 months adding 30g of NPK each time. Plant growth is being observed and trees that grow slow as compared to others are again fertilize with 30g of NPK one or two more times. Planted trees care and maintenance is usually for a two-year period after which the trees are observed to grow sustainably without additional fertilization. Periodically, soil cultivation and ring weeding of the trees is done and the cut grasses are used for mulching.

In all planting activities, only manual labor was utilized. Trucks nor heavy equipment is not suitable to use because of soft soil and/or soil cracks created during soil moisture evaporation on sunny days. IP's and local indigents are employed as the main work force. This is a very good strategy since these IP's have the knowledge of the nearby forest where wildlings for seedlings production are source out.

For the purpose of additional productivity, coconut palm planting was tried and found to grow well with the input of sodium chloride in addition to NPK as fertilizer. As of December 2016, 2,000 coconut palm trees are sustainably growing.

4.2 *Tree planting results – recreated forest land*

Survival rate of planted trees is estimated at 90%. Trees planted in the year 2013 already grow to a height of 5 – 6m, occupying an 8ha. close canopy forest. Trees planted in 2014 to 2016 ranges from 1m – 3m in height in about 60ha. area. The Ipil-ipil continues to increase population because the seeds that fell to the ground germinated naturally and started to grow. The Ipil-ipil leaves that fall to the ground also continue to enrich the soil. Eventually in the coming years, the Ipil-ipil will dominate some of the rehabilitated areas but this will be a welcome development because the timber from these trees will be later converted into charcoal fuel for the purpose of replacing the charcoal produced from mangroves in the plant site's neighboring villages. Native trees which were already listed in Table 3 above can be observed growing even if these were not intentionally planted. These native trees could have germinated from the top soil taken from nearby farmlands. In the year 2012, when about 28ha. of the TSF-1 tailings surface area are already covered with grasses and shrubs, presence of several species of avifauna and mammals were observed in the vegetated area. Initial fauna monitoring was conducted in September 2012 so as to determine the positive impact of revegetation accomplishment.

The list of tree species planted at TSF-1 rehabilitation area will still increase as more native and endemic tree species are discovered in the nearby forest. The next challenge is on how to correctly identify the newly discovered tree species that are yet not familiar among foresters.

Figure 4 is a representative picture of the trees planted in 2013. Figure 5 and Figure 6 is the comparison of the TSF-1 image before rehabilitation in April 2010 and the current image taken in March 2017.

Figure 4. Trees planted in 2013

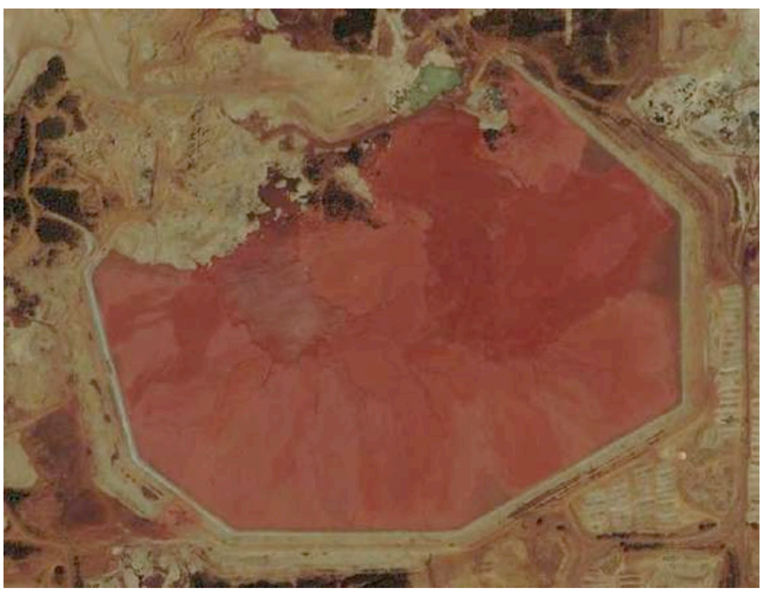

Figure 5. TSF-1 Image taken in April 2010

Figure 6. TSF-1 Image taken in March 2017.

4.3 *Impact of tailings soil revegetation to faunal biodiversity*

To evaluate the impact of the revegetated TSF-1 tailings soil to faunal biodiversity, a third-party expert from the University of the Philippines, Los Baños was commissioned to conduct fauna monitoring in the TSF-1 rehabilitation site. The summary of monitoring results is presented in Table 4.

Table 4. Summary of Fauna Monitoring at TSF-1 Rehabilitation Site*

	Number of Species			
Monitoring Date	Birds	Mammals	Herps	Total
September 7, 2012	20	2	(not monitored)	
June 6, 2016	21	4	3	28

* Source of data – CBNC Terrestrial Wildlife (Vertebrate) Monitoring Report, 2012 & 2016.

The fauna monitoring data shown in Table 4 show quite a number of species such as about 20 kinds of birds and a few mammals, and herps (amphibians and reptiles). It is a decent start from the viewpoint of biodiversity because before the rehabilitation works started in 2011, not a single faunal species was observed going to the bare TSF-1 tailings soil landscape. In the coming years as the trees continue to grow into a forest and mature to fruit-bearing age, and provide a greater area for wildlife habitat, faunal biodiversity is expected to improve. Monitoring will be continuously done in order to have an updated data on faunal biodiversity.

5 CONCLUSION AND FUTURE PLANS

The objectives of the TSF-1 rehabilitation project to recreate a forest land will be totally attained in a period of at least 5 more years. By then, forest productivity will increase and the ecosystems services provided by the recreated forest will be greatly enhanced.

Planted trees which are two years or older were observe to grow sustainably. The strategy to plant grass first as surface cover is effective in producing biomass and in controlling siltation and dust emissions. The use of organic inputs like vermicompost, carbonized rice hull and top soil from farms in the communities near the plant site proved to be the reason behind sustainability. The grasses and tree leaves litters are observed to be being decomposed in a 6-month to 1-year period. This indicated that nutrient cycling is continuous. The top soil which was presumed to contain native seeds, shoots and slips proved to be true. Plants that were not planted can be observed growing and these sprouted from the top soil.

Five species of the planted fruit trees started to bear fruits in 2016 which attracted birds and bats to frequent the reforested site. This birds' colonization of the site will eventually enhance biodiversity in the future as bird droppings may contain seeds from the nearby forests.

The humidicola which is a strong creeping grass is expected to be overgrown by the trees and eventually eliminated in the reforested sites. A plan is that the humidicola grass, the napier grass and the centrosema will be maintained in some areas where the trees are planted at 10m x 10m spacing in order to start a livestock farming if desired by those who will be stewards of the rehabilitated site later.

To evaluate objectively the success of the rehabilitation project, a landscape function analysis (LFA) shall be conducted in 2 – 3 years from now. Findings and recommendations that will be the output of LFA shall be considered by then.

A very important positive impact of the conducted rehabilitation works is the employment generated for the IP's and local indigents. During the peak of grass planting in 2011 – 2013, more than 100 workers are employed. The present plant care and maintenance activities at TSF-1 still generate a workforce of more or less 45 individuals daily.

The revegetation of tailings soil as described above serves as a "poster boy" that relays a strong message to anti-mining advocates that a sustainable and productive mine rehabilitation can be done. As of this writing, the TSF-1 revegetated landscape had been host to thousands of visitors coming from different regions in the Philippines and from other countries in Asia and Europe.

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A Waste Fines Cell with a Multitude of Concurrent Activities

Johan Boshoff
Golder Associates Pty Ltd, Perth, Western Australia

ABSTRACT: This iron ore mine site, located in the Pilbara region of Western Australia, commenced operations in 1998, with ore derived by mining of open-cut pits in Channel Iron Deposits (CIDs). A portion of the ore extracted at the mine is processed at wet processing plants that produce both ore saleable product and waste fines (i.e. tailings) slurry. The waste fines produced at the wet plants are presently stored within Waste Fines Cells (WFCs).

Waste fines cell A (WFCA) is located within an Autonomous Haul System (AHS) area and consists of two main embankments (northern and southern) that span the width of the pit in east-west alignments, and an additional embankment along the eastern edge of the pit (the eastern embankment). Technical studies and design analyses were carried out to support embankment raises and ongoing development of the WFC. During operation of the WFC, a multitude of simultaneous activities took place to the immediate north and south of the WFC respectively. These activities included the construction of the embankment raises using the AHS fleet, mining of the CID to not only mine the ore but to also create future deposition space, blasting next to the WFC embankments as part of the mining activities, seepage collection and dewatering of the pit in the vicinity of the WFC.

This paper discusses the material characteristics, design approach, construction staging and equipment requirements, blasting near embankments, water management, facility consequence category and closure concept.

1 INTRODUCTION

At this mine, Saleable Ore Product is derived from Channel Iron Deposits (CIDs) with the WFCs located at the bottom of the pit excavations, thus on the exposed upper surface of the Limonite-Goethite Channel (LGC) succession. The CID successions are completely excavated and the pit sidewalls are then characterised by alluvium successions. These excavations are dry and dewatering abstractions lower the water table to elevations compatible with the exposed upper surface of the LGC.

The original design for Waste Fines Cell A (WFCA) allowed for the construction of the embankments to RL 480 m (approx. 30 m high) with embankment fill placed by end-tipping. Waste fines deposition commenced in June 2015.

After completion of a design review and subsequent investigations, Golder was engaged to design short term stability remedial measures and raises to the existing WFCA embankments as well as a future waste fines cell (WFCB) in order to grant additional waste fines storage capacity. WFCB will eventually consist of two embankments i.e. an interim embankment and a final embankment.

2 BACKGROUND

The layout of WFCA is shown in Figure 1. The different stages of development of the WFCA embankments are depicted in Figure 2. WFCA consists of two main embankments that span the width of the pit in east-west alignments (the Northern and Southern embankments). The Northern and Southern embankments are approximately 40 m high. The Northern embankment will eventually be constructed to a higher elevation to form a future Autonomous Haul System (AHS) haul road.

Figure 1. WFCA layout

The different stages of development of WFCA can be summarised as follows:

- Stage 1: Buttresses were constructed to both the Northern and Southern embankments during April 2016 for stability and risk mitigation purposes.
- Stage 2: The northern buttress was increased in size in accordance with future embankment geometry and stability requirements.
- Stage 3: Final raising of WFCA northern embankment beyond Stage 2 to accommodate a future haul road.

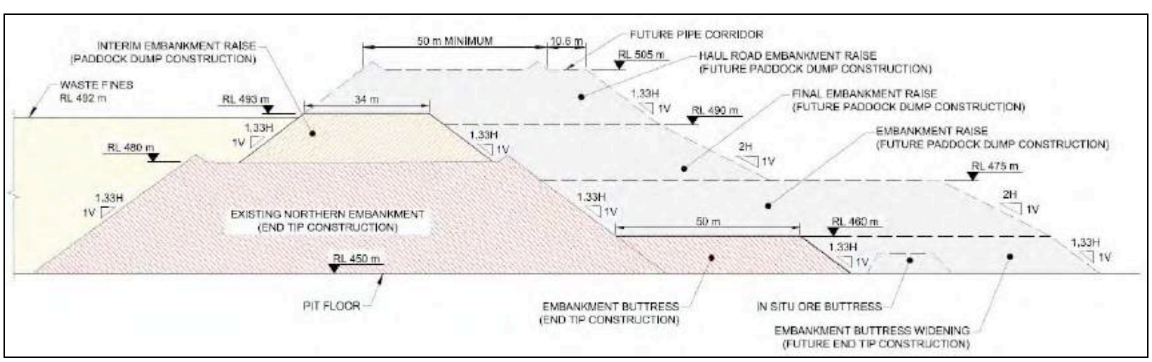

Figure 2. WFCA embankment stages

WFCB is located to the immediate north of WFCA and is also located within the same pit as WFCA (Figure 3). WFCB comprises an interim cell (commissioned in March 2017), situated directly north of WFCA, and a proposed final cell formed by an embankment to the north of the WFCB interim cell. It must be noted that the exact locations of this embankment was not fixed as the final location was dictated by the rate of mining achieved within the. The interim cell embankment will eventually be submerged by waste fines stored in the final cell.

Figure 3. WFCB layout showing future deposition and embankments

Waste fines are currently being discharged into WFCB from a single discharge point located on the western side of the WFC. Multiple discharge points will be installed along the WFCA northern embankment in due course.

3 SITE CHARACTERISTICS

3.1 *Regional setting*

The Mine is situated on the eastern edge of the Hammersley Plateau, adjacent to the Fortescue Plains biogeographic region. The area is typical of the Hamersley plateau, with low lying hills and mesas shaped by the alluvial processes of streams. Rainfall is erratic, often occurring in large downpours as a result of summer thunderstorms or cyclonic activity to the north.

3.2 *Geology and groundwater*

The WFC is located within a pit which is part of a Paleochannel system. This particular Paleo channel system is a Channel Iron Deposit (CID) made up of pisolite that were deposited, then re-worked in a braided stream environment during the Tertiary Period approximately 65 million years ago.

The CID is approximately 300 to 500 m wide and bounded at by the lower permeability Weeli Wolli Formation. The CID materials have high porosity, which enables them to drain freely. More than 80 % of the CID is below the water table and forms an aquifer. A Basal Clayey Conglomerate (BCC) layer lies beneath the CID, which is overlain by overburden comprised of alluvial gravels (ALL).

Groundwater moves principally within two aquifer systems at the Mine site, namely through the CID (historical drainage) and alluvial (current drainage) aquifers (Figure 4). The CID and alluvial aquifers are inter-connected and are recharged from infiltration generated by water flowing along the creeks.

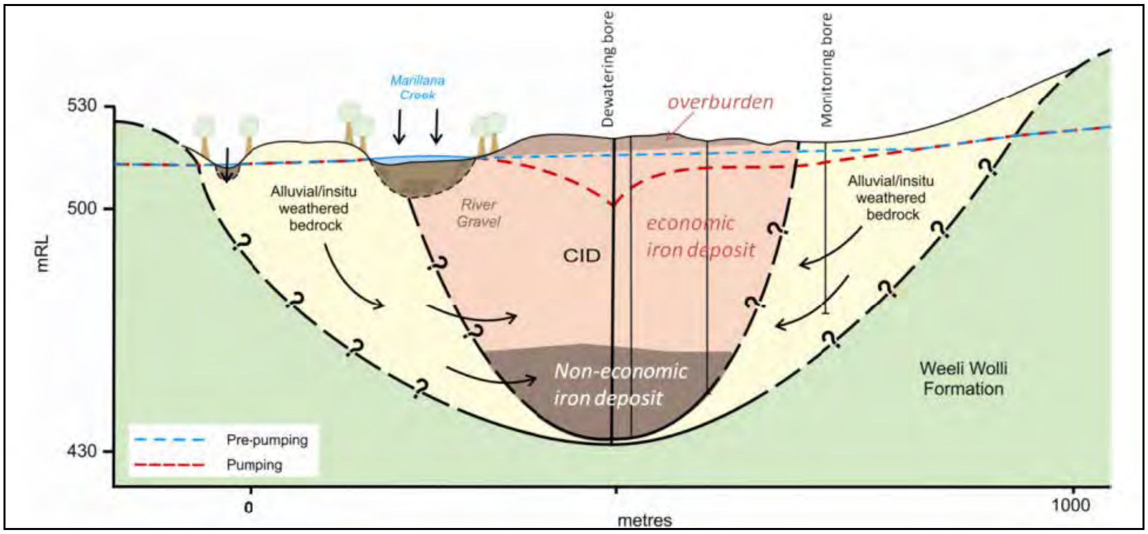

Figure 4. Schematic presentation of groundwater aquifers at the mine site

4 MINING AND DEWATERING

The pit is actively dewatered to maintain dry working conditions as mining progresses. Dewatering is undertaken using "sacrificial" in-pit bores, in-pit sumps and ex-pit bores located to the south and north of the pit.

Once mining is completed, and waste fines are deposited in the WFC, pumped volumes from the pit are expected to decline. It is however important to estimate the rate of groundwater recharge and to assess the impact of the rising water level as dewatering bores are decommissioned as it has an impact on the WFC embankment stability.

5 CONSEQUENCE CATEGORY

In accordance with the Australian National Committee on Large Dams 2012 (ANCOLD 2012) guidelines on tailings dams, WFCA is classified as a 'High C' facility. This classification is assigned to the facility due to a combination of Minor severity with a population at risk (PAR) in the range of 1 to 10, acknowledging the potential of one or more human lives being lost in the unlikely event of failure.

WFCA is considered Minor Risk to damage of the surrounding environment due to failure (Dam Failure Consequence Category) and Minor Risk to environmental damage due to overtopping (Environmental Spill Consequence Category).

6 WASTE FINES CHARACTERISTICS

Waste fines are pumped to the WFCs from the wet processing plant. The waste fines are classified as a clayey Silt with a particle SG of 3.8 and a P68 of 63 μm.

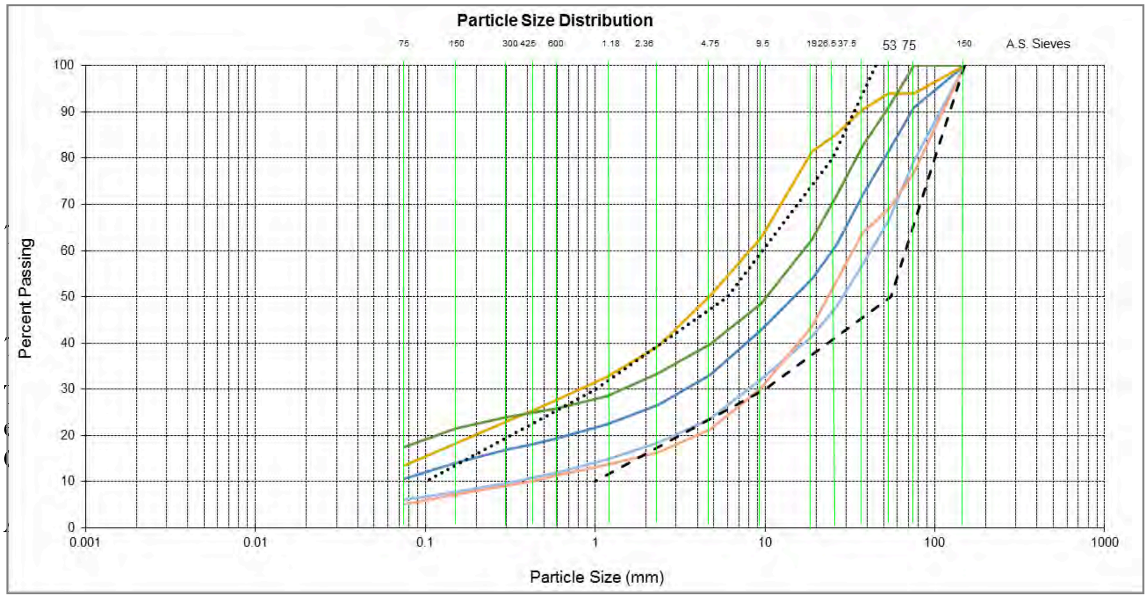

Figure 5. Waste material particle size distribution

6.1 Potential for contractive behaviour

In the geotechnical assessment and analysis of waste stockpiles consisting of primarily coarse rock, with negligible sand or fine-grained particles, it is typical to assume that such materials will be dilative and/or fully drained during shear – i.e. that contractive behaviour and undrained strengths are not relevant. However, it has been well established that materials made up primarily of loose sand can exhibit contractive behaviour, characterised by a peak or "yield" shear strength, followed by strain softening referred to as static liquefaction. Similarly, silts and clays are recognised as exhibiting contractive behaviour and hence present undrained strengths lower than drained strengths.

To assess the potential for the existing WFC embankment materials to exhibit contractive behaviour, Golder reviewed the literature to identify examples of static liquefaction of waste materials. These are summarised in Table 1 in comparison with material samples tested by Golder during the WFC geotechnical investigations. This comparison suggests that the WFC materials are within the range of waste materials that have previously been observed to undergo static liquefaction. In particular, the fines content of the WFC materials are higher than the referenced failure case histories, further indicating the likely susceptibility of WFC materials to static liquefaction should it become saturated, remain in a loose state, and exposed to sufficient shear stress ratios. The inferred material state is consistent with cracks that have been observed on portions of the embankment crest. Golder subsequently recommended to modify the end-tipping method to paddock-dumping construction.

Table 1. Examples of Static Liquefaction

Example	D50 (mm)	% < 75 µm	Reference
Coal waste – laboratory model flow-slides	2-6	5-10	Eckersley (1990)
Coal waste stockpiles in north Queensland	2-5	< 5	Eckersley (1985)
Waste stockpiles in British Columbia (1)	5-10	Trace	Dawson et al. (1998)
Waste stockpiles in British Columbia (2)	7-10	Trace	Dawson et al. (1998)
Waste stockpiles in British Columbia (3)	4-10	Trace	Dawson et al. (1998)

Aberfan failure	1-15	Unknown	Bishop et al. (1969)
Literature review	1-12	< 5	Hunter and Fell (2002)
WFCA existing embankments	2-9	14-23	

6.2 Shear strength for saturated contractive zones

As indicated above, contractive behaviour is likely for much of the saturated portions of the embankment below RL 480 m. As such, the relevant shear strengths available for this material under undrained loading are likely to be significantly lower than drained strengths. However, assessing the peak yield strength for such a material is difficult. The coarse-grained particles make undertaking laboratory tests difficult in most conventional test devices. Further, even if it was possible to test some of the samples recovered in this manner, the *in situ* variability would mean that such laboratory test results may have limited value/applicability. Also, the yield strength ratio is likely to vary somewhat depending on loading direction (Sadrekarimi 2014). Typically, loading in the triaxial compression direction gives the highest yield strengths, followed by simple shear and then triaxial extension. Owing to the geometry and phreatic conditions within the WFC embankments, failure surfaces through the saturated zones are likely to consist primarily in the simple shear loading direction, with a small portion in triaxial compression.

Owing to the above considerations, it is unlikely that laboratory strength testing would provide useful assessment of the yield strength ratios of the existing WFC material. Therefore, examination of previous studies and liquefaction case histories provide another potential estimate method, particularly where such case histories have been empirically correlated to yield strength (for example, Olson and Stark, 2003). The summarised results indicate a range of values from 0.20 to 0.50. Taking consideration of the potential for coarser zones within the fill, and in cognisance of the potential conservatism implicit to assuming that the majority of the saturated zones of WFC3 are contractive, a yield strength ratio of 0.30 has been selected by Golder to characterise saturated zones of the WFC embankments.

7 PIPING AND EROSION RISKS

On the basis of previous investigations conducted by Golder, and initial interpretation thereof, Golder has identified the potential for piping and internal erosion of the material within the original end-tipped WFCA embankments. This risk was previously identified by Golder in an initial third party review of the original WFCA design.

The internal stability of the embankment fill materials at WFCA had been assessed based on the method proposed by Wan and Fell (2003). The results of the assessment indicated that a significant quantity of the in situ materials at WFCA were likely internally unstable, based on comparison to the method proposed by Wan and Fell. The density of the in situ materials was assessed by Golder as part of geotechnical investigations undertaken at WFCA. These indicated that the original end-tipped WFCA embankments were generally loose and consistent with the end-tipped method of construction.

The potential for internal erosion to commence and propagate within an embankment is also affected by the hydraulic gradient relevant to the *in situ* seepage profile in relation to the material's critical hydraulic gradient. The results of the assessment indicated that hydraulic gradients were increasing such that the likelihood of internal erosion would continue to increase with a rising waste fines level. Further, the hydraulic gradients were at a magnitude that was at, or approaching a level which had been demonstrated to initiate internal erosion processes.

As the potential for piping and internal erosion likely already existed the only practicable option to reduce risk in the short term was monitoring of the embankment to provide timely warning of such a potential occurrence. Other remedial measures considered included filter zone construction and additional buttressing.

8 EMBANKMENT DESIGN CONSIDERATIONS

8.1 *WFCA buttressing*

Buttresses (50 m wide) to the northern and southern embankments of WFCA were end-tipped and dozed from the RL 480 m level to a minimum elevation of RL 460 m and RL 465 m respectively to improve the short term stability of the WFCA end-tipped embankments. While end-tipped material is generally unsuitable for water or tailings retaining earth structures, in the specific context of buttressing of the embankments, end-tipping was still likely to produce improved stability conditions for the existing WFC embankments at the time.

8.2 *Embankment staging*

The WFCA embankment raises above RL 480 m were constructed by paddock-dumping and flat-topping in 1.5-2 m lifts, until the embankments were raised to RL 491 m. This method is favourable to end-tipping as it reduces segregation of the embankment fill, and it improves the *in situ* bulk density. The embankments were constructed using waste material sourced from the pit, and placed with the AHS haul trucks which requires a minimum access way width of 55 m for two way traffic. Nominal compaction of each lift was achieved by the bulldozer while additional compaction was provided by the haul truck traffic.

The location of the WFCB interim embankment was dependent on the remaining storage capacity in WFCA, the rate of ore removal within the pit and the completion of the WFCA Northern Embankment Buttress raise to RL 475 m. The resources used for the buttress construction were the same as required for the interim embankment and, therefore, the construction of the interim embankment could only start upon completion of the buttress. The initial waste fines discharge into WFCB also occurred from the crest of the northern embankment buttress at RL 475 m.

Steeper side slopes were proposed for the WFCB interim embankment due to its temporary use, as waste fines will eventually be deposited upstream and downstream of the interim embankment (Figure 6). The construction method of the WFCB final embankment will be similar to that adopted for the interim embankment. The proposed location of the final embankment coincides with the location of a remaining in situ waste block that had yet to be mined. This waste block was subsequently incorporated into the final embankment design as per Figure 7.

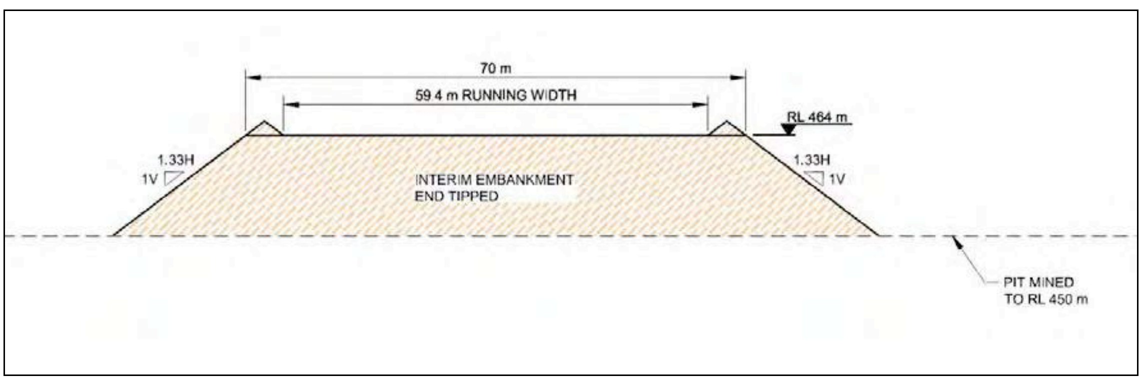

Figure 6. WFCB interim embankment section

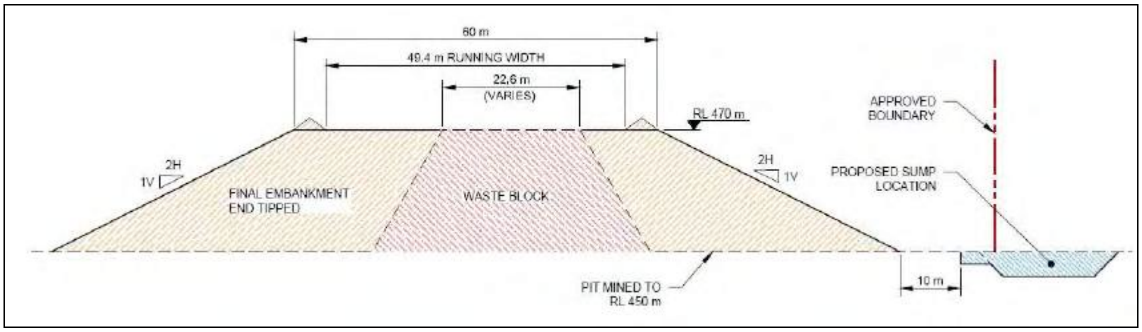

Figure 7. WFCB final embankment section indicating waste block

9 RATE OF RISE

The elevation of waste fines has been tracked by regular survey since December 2015, which to date has indicated an average rate of rise of approximately 0.07 m/day (or ~26 m/year). Consolidation modelling indicated an average t50 and t90 of 1.2 and 3.1 years, respectively, representing the time taken to achieve 50 % and 90 % consolidation. The results also indicated the potential for significant post-deposition settlement to take place. Up to 8.7 m of consolidation settlement may occur over the years following cessation of deposition, which represents increasing freeboard with time and can provide additional waste fines storage capacity if needed. The closure plan allows for the pit to be backfilled with waste material above the waste fines level.

10 SEEPAGE COLLECTION

Seepage water from the waste fines deposited in the WFCs flux through the base of the embankments and the embankment foundations, and accumulates in sumps located immediately downstream of the WFC embankments. The stability analyses indicated that the water level in the sumps had to be maintained below estimated target levels at all times in order to not compromise the stability of the WFC embankments during mining in the pit. Closure allows for the dewatering bores and pits to be abandoned and the pit to be backfilled with waste above the waste fines level.

11 BLAST ASSESSMENT

Blasting occurred immediately downstream of the WFCA and WFCB embankment as part of the mining activities.

The intensity of ground vibrations, which is an elastic effect measured in units of Peak Particle Velocity (PPV), is defined as the speed of excitation of particles within the ground resulting from vibratory motion. The PPV is the most commonly used measure of the intensity of the ground vibration due to blasts.

Appropriate limits for blast-induced vibrations at earth dams and embankments have been discussed in numerous publications, including Charlie et al. (1987, 2001), Oriard (2002) and Pfeifer (2010). According to Pfeifer (2010), the amount of damage from blasting correlates best to the PPV (longitudinal), while peak ground acceleration (PGA) is more appropriate when evaluating damage from earthquakes. Charlie et al. (1987) suggested the following criteria for blasting near dams (Table 2), based on liquefaction potential and susceptibility to pore pressure increases.

Table 2. General guidelines to vibration damage thresholds for blasting near dams

Dam Construction	PPV Limit (mm/s)
Dams constructed of or having foundation materials consisting of loose sand or silts that are sensitive to vibration.	25
Dams having medium dense sand or silts within the dam or foundation materials	50
Dams having materials insensitive to vibrations in the dam or foundation materials	100

The information presented in the above tables can be used as general guidelines for assessing the potential for blast vibration damage to structures. Zones within the historic WFCA embankments (below RL 480 m) are in a loose, saturated state, and may be susceptible to liquefaction. Therefore, a PPV limit of 25 mm/s was recommended for the WFCA embankments. The PPV limit of 25 mm/s yielded offset distances between the WFCA northern embankment and mining activities.

However, prior to commissioning of WFCB interim cell and during construction of the interim embankment, the interim embankment was dry and not prone to liquefaction. Therefore, a PPV limit of 100 mm/s was recommended for the WFCB interim embankment, prior to deposition of waste fines into WFCB.

Geophones (velocity transducers) were installed at various locations within and around the WFC embankments to monitor vibrations from a given blast and to refine the blast model. Based on the results of the ground vibration monitoring to date, a relationship has been developed between the maximum explosive charge weight and a given separation distance from a blast to maintain a PPV within the limit 25 mm/s.

12 STABILITY

12.1 *Method*

The geotechnical stability of the WFCs has been assessed using the method recommended by ANCOLD 2012 guidelines, consisting of:

- Assessment of static equilibrium using effective (drained) and/or undrained strength parameters.
- Assessment of the liquefaction potential of the waste fines, embankment and foundation materials.
- Assessment of post-seismic stability and estimation of the embankment deformations using simplified methods such as Swaisgood, 2003.

12.2 *Model set up*

The geotechnical stability of the WFC embankments was carried out using the 2D limit equilibrium slope stability analysis software SLIDE version 6.0 (Rocscience 2010). Model sections as presented in Figures 2, 6 and 7 were analysed using the Morgenstern-Price method.

12.3 *Material properties*

The key material parameters used in the stability analyses are summarised in Table 3.

Table 3. Stability analysis parameter summary

Material	Unit Weight (kN/m^3)	Drained Strengths	Undrained Strengths	Post-seismic strengths
Saturated contractive embankment fill	25	N/A	s_{yield}/σ'_v – 0.30	s_r/σ'_v – 0.15
Unsaturated and/or coarse embankment fill	25	Φ' – 50 @ 0-10 kPa Φ' – 45 @ 20 kPa Φ' – 40 @ >50 kPa c' – 5 kPa	N/A	Drained Strengths
Coarse foundation layer with clay lenses	20	N/A	Strength function	Strength function
Waste Fines	20	No Strength	s_{yield}/σ'_v – 0.05	No Strength

12.4 *Loading conditions and minimum factor of safety*

The minimum factor of safety (FoS) used to assess the outcome of the stability analyses for the WFC embankments was based on the requirements of the ANCOLD 2012 guidelines and are summarised in Table 4.

Table 4. Loading conditions and recommended minimum FoS

Loading Condition	Waste Fines Strength Model	Recommended Minimum FoS
Long-term	Drained/undrained strength	1.5
Post-seismic	Post-seismic shear strength	1.0

12.5 *Stability results*

The results of stability analyses indicate that the global minimum FoS exceeds the recommended values proposed by ANCOLD documents, under static and post-seismic conditions. The critical slip surfaces produced by the models intersect the sacrificial downstream fill zone of the embankments, and if mobilised are not likely to result in significant loss of retention.

13 SEEPAGE

A seepage assessment was undertaken to:

- Estimate the seepage expected from the storage of waste fines within WFCA; and
- Estimate the location and shape of the phreatic surface after deposition has ceased on WFCA, once WFCB has been commissioned and the northern embankment has been raised to RL 505 m.

The seepage assessment for WFCA and WFCB was undertaken using SEEP/W (Geo-slope International) finite element numerical modelling software.

Previous studies conducted by Golder have indicated that the phreatic level within the embankments is controlled by the downstream water level in the associated sumps. This is consistent with a relatively permeable embankment, where the tailings and upstream water provide a source flux to the downstream embankment water level. The predicted phreatic surfaces are generally consistent with these observations. However, the potential presence of preferential flow paths through the embankment (in particular the lower, end-tipped zone) may result in localised seepage expressions in downstream areas.

The seepage assessment also indicated that when waste fines deposition shifts to WFCB, the phreatic surface in WFCA will start drawing down with the rate of drawdown based on the consolidation of the waste fines, and permeability of the embankment waste rock.

14 CLOSURE

The pit is to be backfilled with waste fines (WFCA and WFCB), such that no in-pit lake will form following closure. The established waste fines cells in the pit will be rehabilitated to integrate into the proposed in-pit landform.

15 CONCLUSIONS

During this project, a multitude of simultaneous technical studies and analyses were carried out to support embankment raises and ongoing development of the WFCs. The designs had to take cognisance of equipment constraints, ongoing blasting and mining, dewatering and forward planning to create ongoing fines storage while at the same time maintaining a safe and stable structure.

WFCA has since been successfully decommissioned while WFCB is currently in operation. Raising of the WFCB interim embankment and mining to the north of the interim embankment are currently ongoing while planning for the construction of the WFCB final embankment has been completed.

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Waste Remediation

The Development of Remedial Design Options for the Questa Mine Waste Rock Piles using a Collaborative Approach

James R. Kuipers
Kuipers & Associates, LLC, Wisdom, MT, United States

Debora J. Miller
Miller Geotechnical Consultants, Inc., Fort Collins, CO, United States

Daniel D. Overton
Engineering Analytics, Inc., Fort Collins, CO, United States

Richard Dawson
Norwest Corporation, Vancouver, BC, Canada

Greg Fischer
Shannon & Wilson, Inc., Denver, CO, United States

ABSTRACT: This paper describes and summarizes the results of the conceptual design and Technical Working Group (TWG) review process conducted for the identification of remedial actions for nine waste rock piles at the Questa Mine operated by Chevron Mining Inc. (CMI) This paper describes the facilitated multi-stakeholder TWG process, including the initial options developed for three "angle-of-repose" waste rock piles that sit adjacent to a state highway and the Red River (referred to as the Roadside Piles). The Roadside Piles design options were followed by development and TWG evaluation of "option suites" and integrated design concepts for all of the waste rock piles. This paper explains the development and screening process for options that would meet the remedial requirements. The key design criteria and constraints, potential failure modes and other considerations are identified and discussed, and the comparative analysis and conclusions are presented.

1 INTRODUCTION

The Questa Mine is owned by Chevron Mining Inc. (CMI) and is located approximately 4 miles east of the Village of Questa in Taos County, New Mexico. Underground mining began in 1919 and large-scale open pit mining began in 1965 and continued until 1983. Large-scale underground mining using block-caving methods began after the open pit was closed. In 2014 CMI permanently closed the mine.

Over decades of operation, waste rock from the open pit and a limited amount from the underground mine was placed in waste rock piles around the mine. An aerial photograph of the waste rock piles is shown on Figure 1 and a summary of their characteristics is provided in Table 1. The site contains ten waste rock piles totaling 163 million cubic yards in volume and covering an area of approximately 780 acres. The mine is located in steep, mountainous terrain along the Red River Valley with natural canyon slopes that underlie the waste rock piles varying between 1.3H:1V (horizontal to vertical) and 2H:1V. The existing slopes of the waste rock piles vary from 1.4H:1V to 1.8H:1V – generally at angle of repose from truck end dumping.

The mine and tailing facility were proposed by the Environmental Protection Agency (EPA) as a National Priority List (NPL) Superfund site in 2000 and the facility was formally listed on the NPL in 2011. A Record of Decision (ROD) was issued in December 2010 describing the requirements for remediation of the site (EPA 2010). The EPA issued an Administrative Order on Consent (AOC) for Early Design Actions in 2012 (EPA 2012). The AOC contains a Statement of Work (SOW) which details the Early Design Actions that CMI will implement.

Specific to the waste rock piles, the SOW stated that CMI would convene a Technical Working Group (TWG) to evaluate reclamation of 9 of the 10 waste rock piles. The purpose of the TWG, as defined in the SOW, was to provide technical expertise in the development and evaluation of conceptual design options for the waste rock piles.

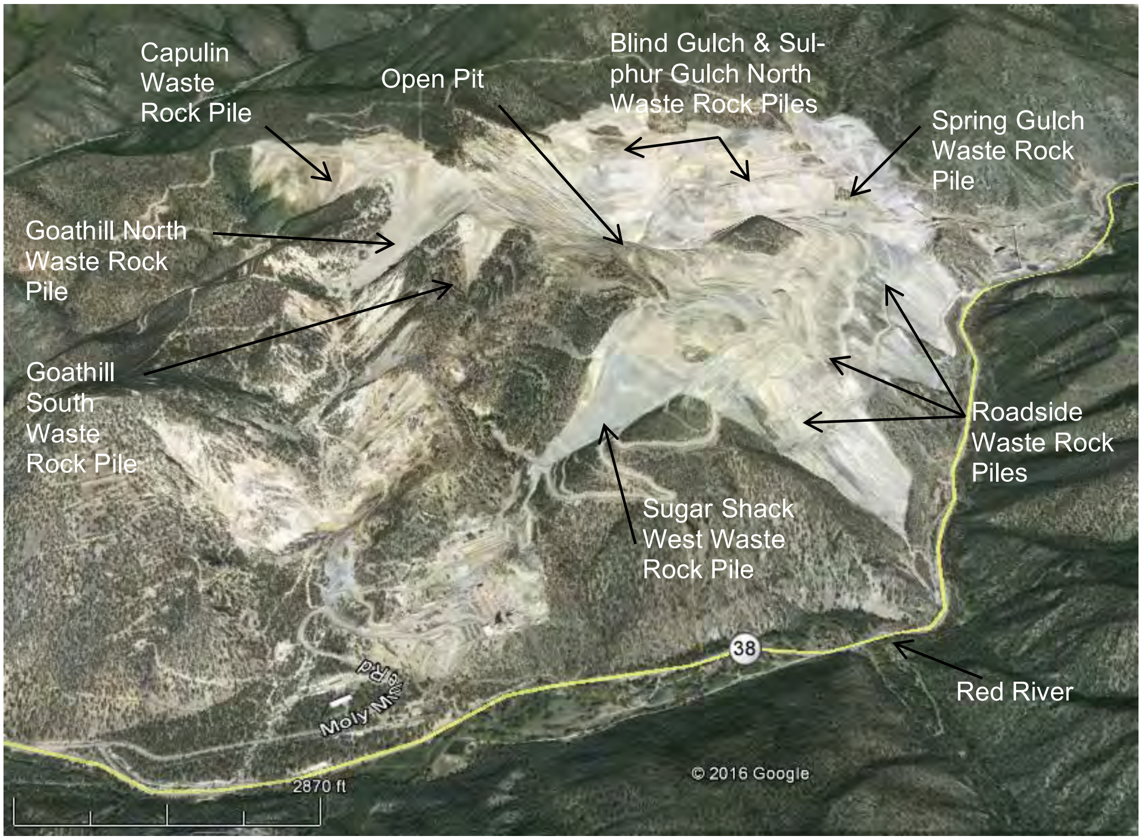

Figure 1. Questa Mine Site Waste Rock Piles

Table 1. Summary of Characteristics of Existing Waste Rock Piles

Waste Rock Pile[1]	Max. Height (ft.)	Max. Waste Rock Thickness (ft.)	Area (acres)	Existing Inter-Bench Slopes[2] (H:V)	Avg. Slope Angle[3] (H:V)	Est. Volume[4] (Mcy)	% of Underlying Slopes Steeper than[5]:	
							3H:1V	2H:1V
Sulphur Gulch South	1,140	395	143.8	1.4 to 1.3:1	1.8:1	39.6	90	82
Middle	1,500	490	147.4	1.4 to 1.3:1	1.7:1	34.5	96	81
Sugar Shack South	1,650	420	88.6	1.4 to 1.3:1	1.6:1	19.5	88	88
Capulin	450	265	59.1	1.5 to 1.3:1	1.4:1	8.1	93	48
Goathill South	520	95	10	1.7 to 1.3:1	1.4:1	0.6	84	76
Sugar Shack West	1,200	200	50.8	1.9 to 1.6:1	1.7:1	6.2	96	82
Spring Gulch	795	395	93.4	0 to 1.4:1	N/A	17.2	97	82
Blind Gulch / Sulphur Gulch North	1,130	375	187.1	3.0 to 1.3:1	3.0:1	31.8	98	88

N/A – Not applicable.
[1]The Goathill North Waste Rock Pile is addressed under another requirement of the SOW.
[2]Existing Inter-Bench slopes represent the range of slope angles that currently exist on the waste rock pile.
[3]The average slope is of the waste rock pile is taken from toe to crest).
[4]The estimated volume of the Questa Mine waste rock piles (excluding Goathill North waste rock pile) is ~ 158 Million Cubic Yards (Mcy.)
[5]Native slopes underneath waste rock piles.

The TWG consisted of six remediation experts, listed in Table 2, each individually nominated to the TWG by key parties that included CMI; EPA; New Mexico Environment Department (NMED); Mining and Minerals Division of the New Mexico Energy, Minerals and Natural Resources Department (MMD); the Village of Questa, and Amigos Bravos. Each TWG expert was to both: (i) participate so as to reflect the interests of their nominating party but also to (ii) exercise their independent professional judgment. The TWG was convened in late 2012, and the remedial design process for the waste rock piles involving the TWG continued over the next three years.

The remedial design process for the waste rock piles and interaction between CMI and the TWG is described by Bolis et al. (2016). In summary, the process involved three general phases, as follows:

1. Phase I –Develop general design guidelines and parameters for all nine waste rock piles.
2. Phase II – Develop preliminary design options at a conceptual level for the waste rock piles located next to Highway 38 (Roadside Waste Rock Piles). Compare and contrast those options using ten key design factors that were listed in the SOW.
3. Phase III – Develop integrated design options at a conceptual level for all waste rock piles and any necessary waste rock pile repositories.

This paper further expands on the overview of the process that was provided in Bolis et al. (2016) by describing in more detail the methods used by the TWG for identifying and screening of options that would meet the remedial requirements. Key design criteria and constraints, potential failure modes and other considerations are discussed, and the comparative analysis and conclusions are presented.

Table 2. TWG Members and Stakeholders (Sorted By Company Name)

	Name	**Stakeholder**	**Company**
Technical Working Group	Dan Overton, PE	USEPA	Engineering Analytics, Inc.
	Jim Kuipers, PE	Amigos Bravos	Kuipers & Associates
	Debora Miller, Ph.D., PE	Village of Questa	Miller Geotechnical Consultants
	Richard Dawson, Ph.D., P.E.	CMI	Norwest Corporation
	Greg Fischer, Ph.D., PE	NMED/MMD	Shannon & Wilson, Inc.
	Will Hultman, PE	NMED/MMD	Shannon & Wilson, Inc.

2 PHASE I – CONCEPTUAL DESIGN GUIDELINES AND PRELIMINARY OPTIONS

The first phase of the SOW included development and assessment of general design guidelines and preliminary design options (at a conceptual level) for the waste rock piles.

2.1 *Conceptual Design Guidelines*

Site-specific, as well as general, design criteria and constraints were identified and quantified, to serve as guidelines in the development of conceptual designs. The key design criteria and constraints identified by the TWG are discussed in this section.

Slope Stability Criteria: For critical structures (defined in the ROD as slopes where there is an immediate danger to human health and safety, or severe consequences of failure), the ROD requires a minimum calculated static factor of safety of 1.5. These criteria applied to the Roadside Waste Rock Piles. A minimum calculated static factor of safety of 1.3 was agreed to by the regulatory agencies for other piles, where the consequences of failure were less severe.

Phreatic Conditions: For stability analyses, phreatic conditions in the waste rock piles were agreed to be based on the highest values recorded by instrumentation over the period of record plus an additional increment of head to account for the relatively short monitoring history (approximately 10 years), which may not yet have captured maximum heads. Sensitivity analysis were performed with varying phreatic surface elevations to assist in determining this additional increment.

Surface Water Management: There are significant challenges to surface water management in the remediation approach because of the height and steep slopes of the waste rock piles and underlying and surrounding natural slopes, the erosive nature of any exposed underlying natural geothermal alteration "scar" areas, and the extreme nature of site-specific storm-events that are typified by late-summer monsoon thunderstorms. The options assumed that runoff from ground above a remediated waste rock pile would be diverted around the piles directed to the toes of the Roadside Waste Rock Piles. Primary water conveyance alignments were developed for runoff estimates based on a 24-hour, 100-year return-period storm event.

Footprint Constraints: The geographic/geomorphic features of the mine property and surrounding topography, in particular the proximity of the Red River and Highway 38 immediately adjacent to and below the Roadside Waste Rock Piles (Figure 1), limited the remediation approaches. Regrade concepts for the three Roadside Waste Rock Piles were limited by the stakeholders to: no change in the highway alignment and the Red River corridor, an access/service corridor at the toe of each waste rock pile, and placing waste material inside the current property boundary.

Existing Mine Closure and Remediation Operations: The Questa Mine operations ceased in June 2014. The conceptual designs that were developed accounted for the requirements of ongoing mine closure and remediation operations, particularly with respect to potential waste rock piles footprints, access, and water management. Therefore, the regrading options did not encroach upon areas required for ongoing closure and remediation operations, such as the headframe/warehouse area, plant site, mill, and water treatment facility. The operating mine plan had intended to use the pit for tailing management so it would not have been available as a waste rock repository. However, the mine closure allowed the TWG to consider the pit as the primary repository for excess waste rock generated from waste rock piles regrading.

Construction: While the designs considered were conceptual, there was a need to demonstrate that they could be safely and efficiently constructed. The size and nature of the waste rock piles would result in large-scale remediation and require the use of large construction/mining equipment to complete the construction efficiently. Adequate access and working areas would be required for the appropriate equipment set. Work during remedial design and construction would need to be performed in a manner to minimize impacts to the environment.

Public and Worker Safety: Worker and public safety were recognized as being the highest priority. All design concepts were required to be consistent with CMI's corporate safety requirements, as well as local, state and federal guidelines.

Compliance with ARARs: All options were developed to comply with the Performance Standards, Applicable or Relevant and Appropriate Requirements (ARARs) in the ROD,

Underlying Bedrock: The topography and bedrock geology beneath the waste rock piles material is an integral component of each of the design option suites and was considered in the development of the suites.

Public Impact. Public impacts through road closures or other access restrictions were identified and minimized where practical.

2.2 *Preliminary Remedial Measures*

At the Phase I level, seven viable remedial measures were identified for potential use in various combinations to develop remedial options for stabilizing the various waste rock piles. These seven remedial measures are described below.

Grading: Large-scale mass movement of waste rock pile materials using heavy mining or construction equipment would be required for remediation. Regrading was the base component of all of the options considered.

Buttresses (mass berm): Buttresses were considered for some options as they would provide increased slope stability through a combination of mass at the toe of the slopes and increased material strength.

Retaining Walls: Engineered systems, tieback walls, mechanically stabilized earth (MSE) walls, and gravity retaining walls, were considered for some options.

Mechanical Reinforcement: Soil nails and mesh, tiebacks and anchor blocks (tension elements); or drilled shafts or soil-mix piles (shear elements) were considered to provide below-grade support to enhance stability.

Soil Improvement: Measures to improve the geotechnical properties of the bulk material, including grouting and densification were considered.

Drains: Drainage systems to collect and route groundwater to discharge locations, including wells (passive or active), horizontal drains, and granular subdrains, were considered.

Surface Stabilization: Vegetation, low-permeability covers, cellular covers and durable rock covers were considered to reduce erosion.

2.3 *Preliminary Remedial Options*

Seven preliminary options were developed by combining the various measures described above. The preliminary options are described below.

2H:1V Regrade with Benches: This option consists of the waste rock piles regraded at a uniform 2H:1V inter-bench slope with overall slopes being shallower (~2.2H:1V). Fifty-foot wide benches would be constructed at intervals to provide an allowance for surface water management.

3H:1V Regrade with Benches: This option is similar to the 2H:1V regrade, but with uniform 3H:1V inter-bench slope with overall slopes being shallower (~3.2H:1V).

Landform Regrade: This option incorporates smooth curvilinear slope faces rather than uniform, linear features in an attempt to mimic the surrounding topography.

Toe Buttresses: This option consists of construction of a buttress at the toe of the waste rock piles. The buttress could be constructed with roller compacted concrete (RCC), reinforced soil, soil cement, compacted earth or compacted rock fill.

Toe Retaining Walls: This option consists of construction of a near-vertical gravity wall (e.g., MSE) or a cantilever wall that could incorporate ground anchors.

Multiple Retaining Walls. This option consists of construction of multiple retaining walls at the toe and along the face of the waste rock piles using similar methods as discussed above.

Tiebacks and Anchor Blocks. This option consists of construction large precast concrete anchor blocks with ground anchors on the lower parts of the waste rock piles slope.

3 PHASE II - ROADSIDE WASTE ROCK PILE OPTIONS AND EVALUATION

3.1 *Final Option Suite Development.*

During the second phase of the process, initial conceptual remedial design options for the Roadside Waste Rock Piles (Sugar Shack South, Middle, and Sulphur Gulch South; see Figure 1) were identified. The initial list of options identified during Phase I was simplified to reduce the number of viable options carried forward for evaluation. It was agreed that landform grading, for example, would be a final design consideration that could be overlain on any of the mass re-grade or buttressing scenarios considered; this technique was not considered as a separate option. Multiple retaining walls and tie-backs also were eliminated for technical and constructability reasons.

As initial option suites were developed, the TWG reviewed available geologic and geotechnical data on the waste rock piles materials and foundation conditions. As necessary, external technical expertise was provided during meetings and included topics such as site-specific geology, geotechnical and geochemical information, construction approach, and cover design. Several visits were made to the site to view the waste rock piles, as well as to various locations along the Red River to view local geological features such as the alluvium, colluvium, debris fan, and scars.

The TWG requested a supplemental field and laboratory investigation to fill in data gaps on geotechnical characteristics of rock pile materials and other material characteristics at the Roadside Waste Rock Piles. The investigation was performed by CMI from February through August 2014.

During the Phase II TWG design review process it was concluded that the three roadside waste rock piles should be evaluated as an integrated system, not as distinct waste rock pile units. Once this was recognized, CMI was able to develop rational preliminary “mine plans” for each remedial option “suite”, including the phasing of earthwork, requirements for temporary and permanent haul routes, preliminary storm water handling, and other key construction sequencing requirements. Ultimately, four distinct option suites were retained for evaluation. These options are described below.

3H:1V Regrade Option Suite: Regrade to a nominal 3H:1V inter-bench slope, with 50-foot wide benches located at approximately every 200 feet of slope length for overall slopes being shallower (~3.2H:1V) to allow for water management.

2H:1V Regrade Option Suite: Similar to the 3H:1V option suite, except inter-bench slopes would be re-graded to a nominal 2H:1V slope with overall slopes being shallower (~2.2H:1V) to allow for water management.

Continuous Gravity Wall Option Suite: Construct a continuous gravity wall approximately 100-feet high positioned at the toes of all three Roadside Waste Rock Piles, with nominal 2H:1V slopes above the wall.

Toe Buttress Option Suite: This option suite included a 450-foot high toe buttress on the Sugar Shack South Waste Rock Pile, in combination with 2H:1V regrade at Middle and Sulphur Gulch South Waste Rock Piles. The placement of the buttress at the toe of Sugar Shack South would allow additional waste rock volume to be left in place in a stable configuration.

3.2 *Option Evaluation and Comparison*

The TWG considered the information and designs, provided feedback to CMI to refine the options, and then identified the potential failure modes and design factors that distinguished the various options with respect to the potential failure modes.

The TWG utilized Potential Failure Mode Analysis (PFMA) as part of the option screening process. PFMA is a systematic, proactive method for evaluating a structure or system to identify where and how it might fail (potential failure mode) and assessing the likelihood of the failure mode occurring and the effects (consequences) of such failure. The goal of the PFMA was to identify stability-related potential failure modes (PFMs) for the Roadside Waste Rock Piles in their final regraded configurations. The identified PFMs were then used in conjunction with other distinguishing design factors (as discussed in the following section) to evaluate and compare the remediation options. The potential failure modes were categorized into the following major types:

- Shallow failure modes that may occur in the top 30 feet of the waste rock piles and tend to be dominated by the effects of extreme events, and construction and maintenance activities.
- Intermediate failure modes that may cover over a range of depths from shallow to deep and are governed almost entirely by the strength of the waste rock pile material (i.e. the failure surface is contained entirely within the waste rock pile material).
- Deep failure modes that extend to, or through, the base of the waste rock pile and involve failure at the interface of the waste rock pile and underlying materials, or through the underlying materials.
- Toe failure modes that may occur at the toes of the waste rock piles and involve global instability of toe retaining structures, structural failure of toe retaining structures, or failure of the surrounding waste rock.

A total of 20 PFMs were identified and evaluated. These PFMs were then categorized as Shallow (9), Intermediate (2), Deep (5), and Toe (4) failure modes. Screening categories presented on Table 3 (Category I through IV) were assigned for each PFM. Two PFMs were assigned a dual Category I/II (Category I Highlighted/Category II Considered, but not Highlighted); both were shallow failure modes associated with mass erosion triggered by storm events or failure of storm water conveyance systems. Twelve PFMs were assigned to Category II (Considered, but not Highlighted). These PFMs were considered credible, but due to one or more considerations were not highlighted. One PFM related to shallow slip-off of the cover leading to exposure of the underlying waste rock was assigned to Category III (Insufficient Information) because more information was needed on the cover, which has not yet been designed. The remaining five candidate PFMs were assigned to Category IV (Ruled Out) after further analysis and discussion.

Table 3. PFMA Screening Categories

Category	Description	Definition
I	Highlighted	Physical possibility is evident, fundamental flaw or weakness is identified and conditions and events leading to failure seem reasonable and credible.
II	Considered, but not Highlighted	Category II PFMs are also considered credible, in that they are physically possible, but are not highlighted because these are judged to be of lesser significance and likelihood than Category I potential failure modes.
III	Insufficient Information	These PFMs to some degree lacked information to allow a confident judgment of significance. Additional investigation or analyses is recommended.
IV	Ruled Out	Candidate PFMs may be ruled out because the physical possibility does not exist, information came to light which eliminated the concern that had generated the development of the PFM, or the PFM is clearly so remote a possibility as to be non-credible or not reasonable to postulate.

3.3 *Distinguishing Design Factors.*

For purposes of the TWG process, evaluation of options for the waste rock piles was focused on civil engineering analyses and design considerations involving geotechnical characterization, slope stability, earthwork design, and constructability issues. The TWG process did not explicitly include evaluation of the potential impacts the various alternatives might have on water quality. In all cases it was assumed that other existing or future mechanisms, such as waste rock seepage capture, underground mine water capture, storm water management, and water treatment as required to meet discharge standards, would be in place to address potential impacts to water quality. Engineered covers were considered and discussed principally in the context of geotechnical stability issues, and not in relation to their performance as source controls to address water quality or growth material to support vegetation.

Five key factors were identified that distinguish between the four remediation options. These five factors were determined based on the comparison between the key design factors and reclamation options as presented above. These five key distinguishing factors are described below.

Stability and Erosion: For regraded slopes, deep seated stability was evaluated and the preliminary results suggested that the factors of safety comply with the ARARs and the differences between options are not significant. For surficial stability of the reclaimed waste rock piles, the 3H:1V options have the highest factors of safety.

For natural slopes exposed by the remediation, the key distinguishing factors among the four remediation options were massive erosion, debris flows and failure of natural slope areas. These failures modes could consist of erosion and mobilization of debris flows caused by concentrated runoff from the unvegetated steep slopes. The presence of unvegetated natural slopes and scars and exposure of those features as a result of waste rock piles reshaping requires additional attention to storm water run-on management than is typical for most waste rock piles. The volume of debris and failed material generated from the exposed natural slopes and scars is anticipated to be greater for the 3H:1V option as compared to the other options.

Toe structures are unique risks that do not exist for the regrade-only options. The consequences of a toe failure on the continuous gravity wall and buttress options are more significant than a localized toe failure for the regrade-only options. The structures may present lower reliability than the regrade-only options depending on the choice of materials used to construct the toe structures and the anticipated design life of those materials.

Engineered Systems: Storm water management is anticipated to be different for the 3H:1V option versus the other options. Compared to the other options, the 3H:1V option will likely need larger flow collection and routing systems to manage both run-on to, and run-off from, the reclaimed and covered waste rock piles. This larger collection and routing system could result in large conveyance structures and licensed storm water impoundment dams that present unique risks and could impact the highway and river if they were to fail. Conversely, it is anticipated that the 3H:1V option will have a less extensive storm water control system on the surface of the regraded waste rock pile than the other options, simply due to the smaller surface area of reclaimed pile.

There are additional considerations with regard to the toe structures required for the buttress and continuous gravity wall options in comparison to the 2H:1V and 3H:1V options. The design and construction of the toe structures require specialized expertise compared to the regrade only options.

Construction Period: All of the options are large multi-year earthwork projects. The period of construction is approximately twice as long for the 3H:1V option. While this results in longer employment opportunities for the local work force, it may result in larger worker risks because of the exposure period, more impacts on traffic and tourism, longer period of interim water management, and potentially more environmental impacts such as greenhouse gas and dust emissions.

Community Impacts: Representatives of the local community indicated an interest in the aesthetics of the proposed reclamation as viewed from both the Village of Questa and from Highway 38 along the Red River. The preference expressed is that the reclamation tie into the native area with appropriate geomorphic design that mimics natural contours and uses vegetation that is similar to the adjacent natural ecosystem.

Costs: The preliminary costs were estimated at $433M, $237M, $462M, and $379M for the 3H:1V, 2H:1V, continuous gravity wall, and buttress options, respectively. The 2H:1V option had the lowest overall estimated cost of all options.

4 MAJOR FINDINGS AND UNDERSTANDINGS

During the course of the TWG process, a number of major findings and understandings that affected the analysis and outcome were identified, as summarized below:

- In the past there have been strong perceptions that reclamation of the Roadside Waste Rock Piles was technically infeasible because it would be too costly, take too long (25 to 75 years), be too complicated and unsafe, result in extended highway shutdowns and road relocation, significantly impact the river and recreational opportunities, and prove to be unnecessary to protect public health and safety. The efforts of the TWG process addressed these perceptions and showed that reclamation is feasible and can be done at a reasonable cost. Construction can be completed in a reasonable time frame, will not be unnecessarily complicated, can be done safely, and will not require road relocation or result in extended closure, and will not result in significant impacts to the health of the river or restrictions on public use
- Closure of the mine in 2014 opened up the pit to be used as the permanent repository. This affected the TWG evaluation process because the previously excavated waste rock had to be hauled a long distance and deposited at the Capulin Canyon waste rock pile. This long haul was a significant cost item, especially for the Roadside Waste Rock Piles, which will generate the largest volume of regrade waste rock. In addition, the use of the pit repository may provide a significant benefit in terms of buttressing/stabilizing slope instability on the west pit slope. The pit repository also provides an opportunity for storm-water detention.
- The 3H:1V option for the Roadside Waste Rock Piles stood out as distinct from the other options in a number of important ways. Construction duration for the 3H:1V option would be nearly twice as long as for other options. The final appearance and long-term environmental conditions also would be very different compared to the other options as large areas of exposed, steep slopes in bedrock would remain after removal of most of the waste rock and underlying natural soil deposits. Much of those exposed slope areas will not support a cover and vegetation; including the exposed scar areas. These conditions will result in substantially higher volumes and rates of storm-water runoff that likely would require capture and treatment over the long-term.
- All options for regrading of the waste rock piles provide opportunities to apply natural landform grading principles and possibly innovative cover/re-vegetation designs. It was determined that landform grading would be difficult to do after the piles were regraded, and hence should be implemented concurrent with the mass regrading construction to the degree practicable.

- Storm water management will be a key final design consideration for the waste rock piles:
  - The only Category I (Highlighted) potential failure modes identified by the TWG for the reclaimed Roadside Waste Rock Piles were those associated with massive erosion events that could result from failure or exceedance of the design capacity of storm water conveyance systems.
  - Storm water detention at the toe was a significant design consideration for regrading of the Roadside and Capulin Waste Rock Piles. For the Roadside Waste Rock Piles, in particular, the 3H:1V option had a 1 to 2-year period of vulnerability to storm water issues during construction when water cannot be routed to the pit. This was not an issue for other options for the Roadside Waste Rock Piles.
- Geotechnical data and information was lacking for the alluvial and colluvial foundation materials at the toes of the Roadside Waste Rock Piles. This was recognized as an important data gap that was addressed by a substantial supplemental subsurface investigation program, which was recommended by the TWG and carried out in 2014.
- Large scars are present beneath several of the rock piles. Although the scar conditions and potential exposure of large scar areas by some remediation options were discussed at length, the presence of scars beneath several of the rock piles was not a primary driver for selection of an option. However, there was a preference to not unnecessarily expose scar areas because of concerns about increased runoff and acid generation from steep, fresh exposures of these mineralized zones and uncertainty about the condition of the scar areas once exposed. Erosion rates from the scars and post-mining areas are similar, but both are ten times greater than erosion from natural slopes outside of scar areas.
- Long-term degradation of shear strength of the waste rock pile materials due to geochemical alteration and formation of clay minerals is not believed to be a significant issue for the waste rock pile materials over the design life based on findings of site-specific studies that were presented and discussed during the course of several TWG meetings.
- Creep behavior was discussed at various times throughout the process. Settlement creep is ongoing within the existing, angle-of-repose rock piles; as evidenced by slope inclinometer (SI) movements and localized shear zones that have been detected in several SI graphs. Based on behavior of SI instrumentation in the re-graded Goathill North Waste Rock Pile, creep settlement in the freshly re-graded material is substantially greater in terms of both magnitude and rate compared to creep settlement in the waste rock pile material that was not disturbed by re-grading. This observation was discussed, and was a factor in the decision to eliminate remediation concepts that used multiple retaining structures built at various elevations on top of the waste rock pile materials.

5 SUMMARY OF INTEGRATED PLAN (PHASE III)

A schematic of the overall integrated conceptual design concept for the integrated conceptual design is shown on Figure 2. The concept results in movement of a total of 63 Mcy of material, including material placed in the Pit Repository (36.1 Mcy), as cover (5 Mcy), and as fill within the various waste rock piles (19 Mcy). This amounts to approximately 37 percent of the total volume of material in the waste rock piles. A total of 743 acres of waste rock pile would be covered, of which approximately 47 percent would be on slopes shallower than 2.2H:1V overall. Approximately 175 acres of natural ground currently covered by waste rock piles would be exposed during the regrading process. Of this area, approximately 15 percent is at slopes of 2.2H:1V or less overall and may be covered.

It was estimated that the integrated regrade plan would be carried out over about 14 years depending on climatic conditions and other external factors, with a work force varying from approximately 50 workers at the start and end of the project and rising to a peak of approximately 200 workers during the middle of the project. The overall preliminary cost of the project was estimated to be approximately $400 million with mass earthworks (43 percent) and cover placement (26 percent) accounting for the bulk of the cost.

The TWG process and the resultant reports provide a good base for future design work for the Questa Mine waste rock piles with no major concerns of fatal flaws identified by the TWG or CMI. In addition, the TWG process resulted in many benefits to the entire stakeholder group.

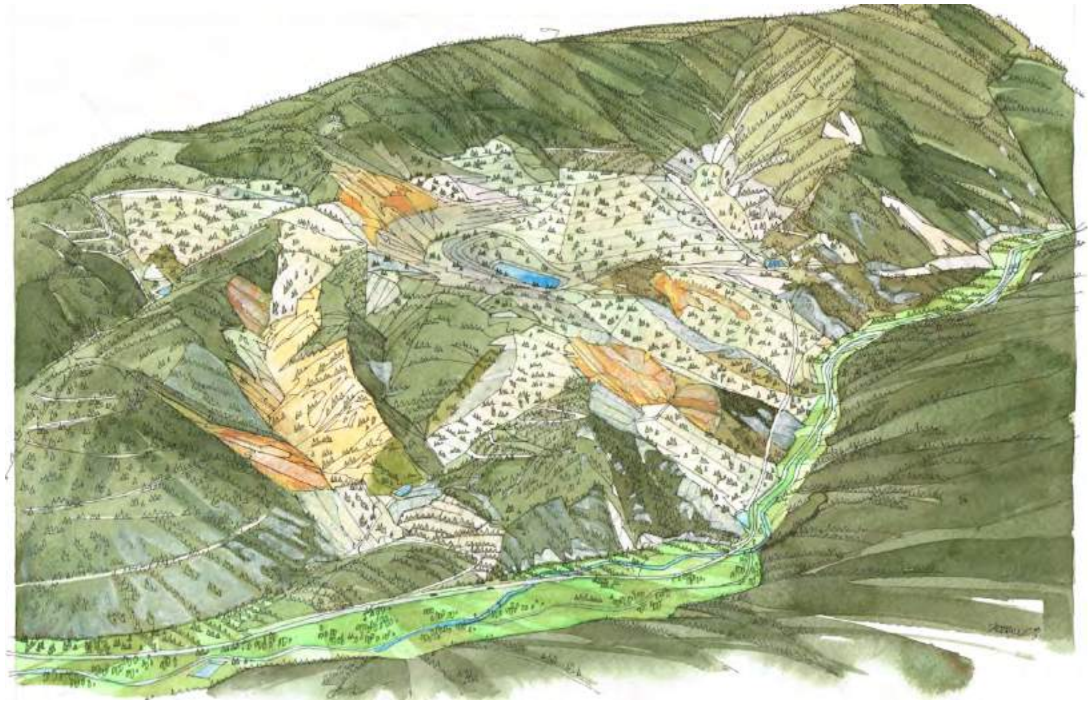

Figure 2. Overall Integrated Design Concept

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Passive Treatment Concepts for Mine Closure at Mount Polley Mine

Luke Moger
Imperial Metals Corporation, Vancouver, BC, Canada

Katie McMahen
Mount Polley Mining Corporation, Likely, BC, Canada

Björn Weeks & Thalita da Silva Sympovsky
Golder Associates Ltd., Vancouver, BC, Canada

Jerry Vandenberg
Golder Associates Ltd., Kelowna, BC, Canada

ABSTRACT: Mount Polley Mining Corporation has recently completed a Long-term Water Management Plan and a Reclamation and Closure Plan Update that outline water management considerations at the Mount Polley Mine during operations and closure, respectively. The Mine is initiating a number of modelling and pilot-scale studies in order to progressively transition from the active water treatment technology currently implemented to an integrated system of passive water treatment and distributed discharge in closure. The advancement of the closure water management planning contemplates a five-pronged approach that is customized for various water sources around the Mine site, employing a combination of proven and experimental technologies. The approach includes: (1) diversion of clean water; (2) evaluation of low-flux covers to reduce infiltration and contact with mine waste; (3) in-situ treatment in pit lakes for particulate metals, nitrate and selenium treatment; (4) wetlands for water that is slightly above guidelines; and (5) biochemical reactors for waters that are significantly above copper, selenium, nitrate and sulphate guidelines.

1 INTRODUCTION

1.1 *Project History*

The Mount Polley Mine (the "Mine"), owned and operated by Mount Polley Mining Corporation (MPMC; a wholly owned subsidiary of Imperial Metals Corporation), is an open pit copper/gold mine with an underground component, and is authorized to process an average of 22,450 tonnes per day of ore on an annual basis. The Mine is located 8 kilometres (km) southwest of Likely, British Columbia (BC), and 56 km (100 km by road) northeast of Williams Lake, BC. Mine concentrates are trucked to facilities at the Port of Vancouver and then shipped to overseas smelters or transported by rail to smelters in North America.

The Mine property consists of 53 mineral tenures covering 20,113 hectares (ha), which consists of seven mining leases totaling 2,007 ha and 46 mineral claims encompassing 18,106 ha. The current disturbed footprint of the Mine site is 1,216 ha, with an anticipated closure footprint of 1,301 ha (neither footprint includes areas having undergone progressive reclamation or areas constituting pit lakes in closure).

Clearing of the Mine site and construction of the facility began in 1995, with the mill being commissioned in June 1997. The first full year of mining and milling at the Mine took place in 1998. Operations were suspended in October 2001 due to low metal prices; however, the Mine reopened in December 2004, with mill production recommencing in March 2005.

1.2 *Tailings Dam Embankment Breach*

On 4 August 2014, a foundational failure occurred along the Perimeter Embankment of the Mine's Tailings Storage Facility (TSF). The foundational failure of the TSF resulted in a breach of the TSF, with water, tailings and construction materials being released or displaced.

The breach of the TSF caused physical impact to the downstream environment, notably: erosion of the embankment separating the TSF from Polley Lake, as well as along Hazeltine Creek; deposition of trees and woody debris in Polley Lake, along the sides of the erosional path associated with Hazeltine Creek, and into Quesnel Lake at the mouth of Hazeltine Creek; and, deposition of tailings and eroded earth in Polley Lake, Hazeltine Creek and Quesnel Lake.

Following the TSF embankment foundational failure, mining and milling operations at the Mine were immediately suspended. The focus of Mine operations shifted to monitoring and remediation work at the TSF, around Polley Lake, down the Hazeltine Creek corridor to Quesnel Lake, and in Quesnel Lake. On 5 August 2014, under the provisions of the *Environmental Management Act*, the Ministry of Environment issued Pollution Abatement Order 10746 to MPMC, ordering MPMC to attend to the environmental impacts of the TSF embankment breach.

The short-term emergency response to the TSF embankment foundational failure and resulting breach included: addressing health and safety concerns; containment to prevent further release of materials from the TSF; water quality monitoring, cleanup of woody debris on Quesnel Lake; lowering of the Polley Lake water level (which had been increased by the event); assessment of the physical stability of the sediment deposited at the outlet of Polley Lake; and implementation of sediment and erosion control measures in the Hazeltine Creek corridor. These efforts have are described more fully elsewhere (McMahen & Hughes 2016; Nikl et al. 2016; Kennedy et al. 2016).

1.3 *Present Status and Planned Mine Closure*

In 2015, the Mine received authorization for a Short-term Water Management Plan to manage surplus water accumulating on the site following the 2014 foundational failure of the TSF. The authorization period for the Short-term Water Management Plan (two years) also allowed for the development of a Long-term Water Management Plan. Following a return to modified operations in 2015 (utilizing an open pit for deposition of site contact water and tailings), the Mine received authorization for a return to normalized operations in 2016 (based on a four-year mine plan and including resumed deposition of tailings in the repaired TSF).

In April 2017, the Mine received authorization for the Long-term Water Management Plan, which formalizes water management planning during the operational phase of the Mine. This operational water management plan entails water storage in the reconstructed TSF (at maximum volumes significantly less than those existing pre-breach) and active water treatment provided by a centralized collection system and a Veolia Actiflo® water treatment plant. The Actiflo® clarifier system uses sand-ballasted settling to remove suspended solids present in the inflow water.

In addition to operational considerations, the Long-term Water Management Plan also introduced closure water management strategies, which were further refined in the 2017 Reclamation and Closure Plan Update. In keeping with the Mine's original commitments (including the Mine's 1995 Environmental Assessment Certificate), and considering comment from the public, regulators and First Nations, the Mine is pursuing a closure water management strategy that entails distributed discharges through passive water treatment systems.

It is envisioned that the mine plan may be extended to 10 years (cumulative), depending on, among other considerations, future commodity prices; however, no formal application for this mine life extension has been made at this time.

As described in the following sections, the closure water management strategies for the Mine are envisioned to be implemented before mine closure if bench-, pilot- and demonstration-scale projects are successful.

2 APPROACH TO CLOSURE WATER MANAGEMENT

2.1 *Overview*

The overarching goal of the closure water management plan is to develop a water management system that is:

- Distributed – to the extent practical, all site contact waters will be directed back to the watersheds that existed before mining.

– Passive – water quality will be attained by passive systems that do not require mechanical, electrical or chemical inputs. Recognizing that even passive systems require periodic inputs, hybrid or semi-passive systems will be evaluated where overall efficiencies can be gained.

The approach to passive water treatment includes five tactics, described in more detail in the following sub-sections: (1) diversion of clean water; (2) evaluation of low-flux covers to reduce infiltration and contact with Mine waste; (3) in-situ treatment in pit lakes for particulate metals, nitrate and selenium; (4)treatment wetlands for water that is slightly above guidelines ; and (5) biochemical reactor followed by sulphide polishing cell and constructed wetlands for waters that are significantly above copper, selenium, nitrate and sulphate guidelines.

2.2 *Diversion of Clean Water*

To protect natural watercourses from potential adverse impacts of Mine site contact water, a series of collection ditches and water containment structures have been established outside (upstream and downstream of) the footprints of disturbed areas. Where possible, non-contact water diversion ditches are in place upstream of Mine site components to divert non-contact runoff, preventing unnecessary collection of water in the Mine site contact water collection system, and maintaining as much water as possible in the natural watersheds.

Similar to the operational water management system, the closure water management system will seek to maximize the diversion of clean water directly off site (with sedimentation ponds, if necessary) and to the pre-development watersheds. The first steps in this process were to delineate the pre-development, operational and closure watersheds, then to estimate future flow rates at all anticipated points of discharge (accounting for changes associated with land reclamation at closure). In parallel, water quality screening was completed on all site waters to determine which of these flows could be directly discharged, versus those that would require one of the treatment methods described below. The pre-development and closure watersheds are shown in Figure 1.

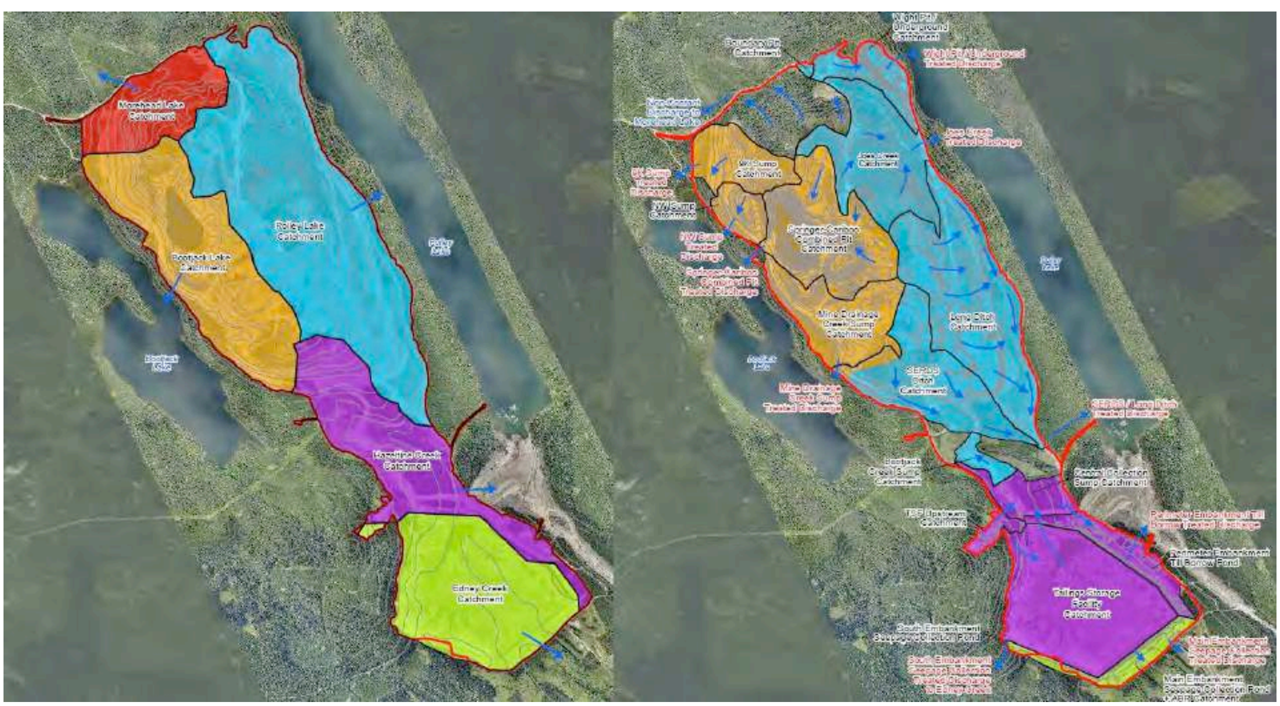

Figure 1. Pre-development (left) and Closure (right) Drainage regimes

2.3 *Low-flux Covers*

Infiltration of precipitation through mine wastes generates contact waters, which are influenced in their quality by the time spent in contact with the mine waste. The amount of infiltration through mine waste is generally at a maximum prior to the implementation of reclamation measures. Most conventional forms of reclamation will tend to significantly reduce the amount of precipitation that will infiltrate through any underlying waste material. The reclamation measures currently considered for waste rock dumps at the Mine site, including placement of fine-grained soil or organic material over rock, will tend to increase the moisture-retaining capacity of the surface, providing storage for precipitation and promoting evaporation, which will reduce the net infiltration into the underlying rock. The establishment of vegetation will further reduce infiltration, as evapotranspiration will increase the return of water to the atmosphere, as will interception of precipitation on plant leaves. Infiltration through reclamation soil covers has been shown to be reduced by improved quality and density of vegetation on the reclaimed surface (Tratch 1994).

Where there is a reclamation need to minimize the total volume of infiltration, reclamation covers may be modified to further reduce the total flux of water through the covers. These modifications can include a number of approaches, such as:

- placement of thicker fine-grained soil layers to maximize the moisture storage capacity;
- placement of lower permeability soils or waste materials under the revegetation layer to impede infiltration;
- construction of covers with capillary barrier effects (which use contrasting moisture retention properties to impede infiltration);or
- any of a variety of other approaches.

In all cases, the selection of an appropriate cover design is tied to climate and consideration of the short- and long-term impacts in the receiving environment (INAP 2009).

While infiltration of precipitation through rock and tailings is not anticipated to contribute to the generation of acid rock drainage at the Mine, infiltration seepage is anticipated to be affected by contact with the Mine waste. Specifically, the contact water accumulates loads of major ions such as sulphate and nitrate, and metals and metalloids that are soluble in neutral to alkaline mine drainage such as molybdenum and selenium. To support the site-wide water quality model for the Mine, geochemical source terms were developed for the waste rock stockpiles and tailings based on conceptual geochemical models, interpretation of rock geochemical characteristics, and seepage data.

MPMC developed a five-part study plan to evaluate infiltration reduction approaches for the waste rock and ore stockpiles. This plan consists of the following steps:

- As part of the site-wide water balance and water quality model update, conduct a sensitivity analysis for cover alternatives to identify (at a high level) the magnitude of improvement that could be obtained in mass loadings to the receiving environment for constituents of interest, considering the range of reductions in infiltration that might reasonably be obtained with typical infiltration reduction covers. These results are used to direct and prioritize future studies.
- Better characterize the infiltration reduction anticipated from the current reclamation prescriptions. This step could include a program of laboratory testing of cover soil samples to characterize moisture retaining properties, and numerical modelling incorporating site-specific climate data
- Following the laboratory testing, evaluate through numerical modelling a range of cases for modification to the current reclamation prescriptions designed to further reduce infiltration
- Depending on the results of this modelling, conduct additional runs of the site-wide water balance and water quality model to quantify the benefits of possible changes to the cover prescriptions (linked to water treatment studies discussed herein).
- If the modelling results show significant potential benefits to one or more of the approaches evaluated, field-testing of test plots (lysimeters) could be conducted to confirm predictions and calibrate models.

The first step in this five-step program has been completed. A water quality model was used to evaluate the impact of cover placement on the predicted seepage water quality. For this sensitivity analysis, the impact of a low-flux cover placed on the entirety of the East Rock Disposal Site (RDS) was compared to the results of closing the dump with a conventional reclamation prescription, aligned with the current reclamation prescription for the East RDS (soil substrate and revegetation). The effect of the low-flux cover over the East RDS was first evaluated in the closure/post-closure water balance scenarios. The results of this water balance assessment were then used as an input in the water quality model, and the effect on the full range of parameters calculated. The results of the water quality model analysis indicated that the placement of a low-flux cover over the East RDS was predicted to have a minimal impact on the water concentrations at the selected discharge point in the scenario modeled; however, for that scenario, the significant benefit of the cover was shown to be in the overall mass loading of the constituents under consideration. By reducing the volume of water that comes into contact with waste rock, the overall mass of the constituent that would be released is reduced significantly. Using the example of dissolved copper, the reduction in mass loading from the East RDS would be approximately 50%, even though the change in concentrations was negligible.

A reduction in overall loading is potentially of interest in the closure design due to the interaction between the overall loading and the passive water treatment alternatives that are currently under evaluation. Reduced loadings (and less seasonal variability in contact water flows) at the points of water treatment may promote success of smaller treatment systems.

2.4 *In-situ Treatment in Pit Lakes*

In-situ pit lake treatment is achieved through biological and sedimentation processes.

The biological treatment process involves amending surface water with the required nutrients, including organic carbon and phosphorus, to promote anaerobic conditions. The goal of in-situ treatment is to create and maintain reducing conditions in the deeper portions of the water column to sequester constituents of concern, such as selenium, within the lake sediments. The biological process relies on the same anaerobic microbial selenate and selenite-reduction processes as active and passive biological treatment. Nitrate is removed by the process of denitrification, wherein microbes facilitate the reduction of nitrate to nitrogen gas. The nitrate is reduced prior to selenium reduction due to the thermodynamics of the reactions. Therefore, the effectiveness and cost of selenium treatment depends on the presence of nitrate. Similarly, sulphate reduction may also occur with pit lake treatment, which would lead to dissolved metal removal via metal sulphide precipitation. Divalent metals including copper can be removed via this mechanism.

The sedimentation process allows constituents associated with settleable solids to be removed by gravity – this process has been observed to effectively remove particulate metals during 2016 when the Springer Pit at the Mine was temporarily used as the site's primary water storage facility (Vandenberg and Litke, in press).

2.5 *Treatment Wetlands*

MPMC has retained Contango Strategies Ltd. (Contango) to conduct a complementary preliminary assessment of passive water treatment potential at the Mine site and advance potential conceptual designs. With a focus on treatment wetlands, Contango considers a phased approach most conducive to passive treatment system development, and, in November 2016, conducted a site assessment ("Phase 1"), which included sampling of water, soils, vegetation and microbial communities in the context of information gathering and gap-filling to inform development of site-specific passive treatment system design for the Mine site. The data collected will help determine the range of parameters that the wetland plant species natural to the Mine site are currently living in, and the types of natural processes that could facilitate treatment; Contango will use the results of the site assessment to support technology selection (or re-evaluation) prior to making recommendations for bench/laboratory-scale testing and off-site pilot-scale (controlled environment) testing and optimization (Haakensen et al., in prep). Based on the findings of the bench- and pilot-scale studies, demonstration- and full-scale wetlands will be designed, constructed and monitored.

2.6 *Biochemical Reactors*

Previous pilot-scale biochemical reactor work was initiated at the Mine site in 2009, in partnership with the Genome BC/NSERC project (Baldwin et al. 2015). Following this work, Golder advanced conceptual design for a biochemical reactor, which was followed by a sulphide polishing cell and constructed wetlands to treat the most highly concentrated streams of Mine contact water. As identified in the closure water treatment best available technology assessment, a passive or semi-passive system is the preferred option for water treatment during closure/post-closure; however, optimization through bench- and pilot-scale testing is required to address uncertainties and to optimize the design of a full-scale system.

The contemplated passive system would consist of a sedimentation pond, biochemical reactor (filled with substrate consisting of wood chips, manure, lime and hay), sulphide polishing cell, subsurface constructed wetland, free-water surface constructed wetland, and aeration cascade. Additionally, semi-passive process units, which would use active components, are being considered, with the purpose of improving the performance and decreasing the footprint of the passive system. The proposed semi-passive components consist of a biochemical reactor (filled with inert material such as gravel) with active addition of labile carbon and nutrients; a sulphide reactor (to which ferric or ferrous chloride would be actively added); and a mechanically aerated pond. This treatment "train" is anticipated to be followed by a settling pond. The conceptual design for the pilot system is shown schematically in Figure 2.

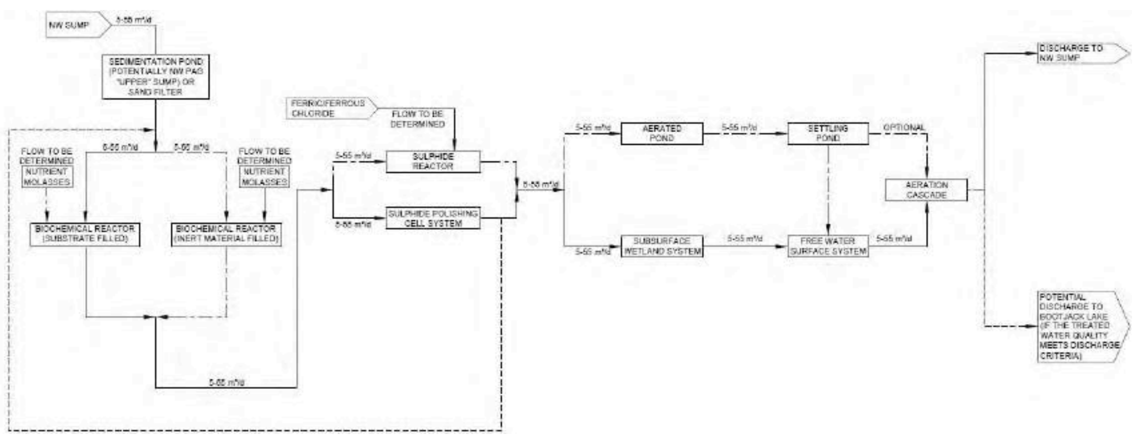

Figure 2. Conceptual Design for Passive System

Wherever possible, materials to be used in a passive treatment system would be sourced at the Mine site, and in some cases, the materials can be sourced from by-products of Mine waste. If a pilot-scale system demonstrates this proposed technology to be feasible, then decentralized full-scale, passive/semi-passive systems would be proposed for the Mine site. The process units proposed for the full-scale systems would consist of the process units used in the pilot-scale system, as the primary objectives of pilot-scale testing are to demonstrate feasibility and performance of the processes.

3 CONCLUSIONS

Planning for long-term site water management is an integrated process that benefits from research, piloting and monitoring across a wide breadth of Mine components and their associated activities. Gathering and analyzing key data early and throughout the mine life cycle (even as early as baseline) supports a holistic approach to integrated long-term planning across multiple disciplines, and creates opportunity to design, implement and refine closure and post-closure strategies during operational phases.

Planning for (and where possible, transitioning into) closure and post-closure water management strategies during active operations is advantageous in that it allows for additional resources and lead time in preparing, implementing and refining strategies, understanding that each site represents a unique set of geochemical, climatic and hydrologic conditions, as well as unique requirements that are dictated by the assimilative capacity of the receiving environment.

Specific to water management planning, consideration of aspects such as pre-mining watersheds and reclamation soil properties, identification of land bases and pits that will be available for passive/semi-passive systems, and implementation of source control strategies present opportunities to integrate mine life activities and realize tangible economic and environmental benefits.

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Applying ORE to Balangero Asbestos Mine Dumps Environmental Rehabilitation Risk Informed Decision Making

Franco Oboni & Cesar Henri Oboni
Oboni Riskope Associates Inc., Vancouver, BC, Canada

ABSTRACT: Twenty years ago the Balangero Asbestos Mine Dumps Environmental Rehabilitation (BAMDER) competitive bid was won by an engineering group supported by what was then called Risk Based Decision Making (RBDM). Thus it was demonstrated that including risk assessment through the design of a project, from cradle to delivery and including risk driven maintenance concepts brought value and a leading edge to the proponents. Recently NASA and the US Nuclear Regulatory Commission (USNRC) have brought forward a process called Risk Informed Decision Making (RIDM). Both RBDM and RIDM use probabilistic risk assessment (PRA) as a tool within the process. In this paper we will show the step by step RBDM procedure used for Balangero and highlight the subtle differences with RIDM. The differences are necessary to make the process accessible and economically sustainable for any civilian project, including, of course mining ones. 20 years post remediation lesson learned at BAMDER will be described, including remote monitoring made possible by drones and data treatment. RBDM/RIDM are presently deployed for a similar project in North America.

1 INTRODUCTION

Despite its small throughput, the Balangero asbestos open pit mine, located 35km N-W of Torino (Torino), was the largest operation of this kind in Western Europe. After bankruptcy the site was abandoned, including the tailings storage facility (dump located at 45°17' 40.51N, 7°31' 17.40E, Fig. 1).

There are similar examples of orphan mining sites around the world where tailings are responsible of long term public health issues. However, Balangero remains to date and to the best of our knowledge, the only example in the world where a rehabilitation has been designed using risk based decision making (RBDM), built and monitored for almost two decades.

The Balangero open pit was cut into the ridge of an elongated hill. The mill was located on one side of the hill and the tailings dumps on the other. In 1918, it was foreseen that the mine would extract 26,000 m^3 rock per year, but in 1961 the mine extracted 1.3 Mm^3 rock. In 1966 a new mill with a capacity of 25,000 metric tonnes of fiber per annum was installed.

2 TAILINGS HANDLING AND STORAGE

The dry tailings were lifted from the mill, located at the foot of the hill, by a conveyor belt (which partially ran underground) to a location near the ridge. From there they were conveyed to the opposite side of the hill, and then dumped over a natural slope with an approximate angle of 25 degrees from the altitude of about 830 m a.s.l. to the bottom of the valley at 580 m a.s.l. for final storage (Fig. 1, 2).

Fig. 1 ©Google 2017 (link) the link displays a site map with various system's macro-elements

As the dumping proceeded, a total surface of about 250.000 m^2 was progressively covered by the storage with tailings thicknesses going from a few meters to an estimated maximum of 60m–80m, resulting in an estimated 60 Mm^3 dry asbestos tailings storage (Fig. 1). This dump, as well as all the production facilities, was abandoned when the mining company abruptly stopped its activities in the early '80s for economic reasons.

Fig. 2: Looking down slope: nearby houses, the Fandaglia Creek and the decant basin

3 REHABILITATION PROJECT REQUIREMENT AND MEASURES

In 1992 a public company formed by the Province of Torino, the Mountain Community of the Lanzo valleys, neighboring communities and other public stakeholders named RSA was mandated by the regional government of Piedmont to organize an international design competitive bid for the Balangero Asbestos Mine Dumps Environmental Rehabilitation (BAMDER). RSA became the "project owners" and contract holders. The goals of the competition and the proposed measures can be summarized as follows:

- select the best possible alternative to increase the stability of the slopes (over-steepened, critically eroded and prone to mudflows). Gravity and water are the main combined external agents posing a threat to the stability of the over-steepened slopes of the dump. Thus, it was necessary to act:
  - against gravity to enhance the stability of the slope and
  - against water to eliminate surface erosion, gullies formation and increase of saturation triggering frequent mudflows along the slope.
- reduce the dispersion of fibers (long term hazard to the neighboring population). The entire restoration process had to include dust minimizing procedures which ended up driving the choice of:
  - excavating, hauling and
  - disposing equipment on the steep slopes.
- re-vegetate the slopes for aesthetic and environmental reasons as the storage is in a densely inhabited area at the Alps foothills (Fig. 2). As the dump material is highly sterile and generally too steep to retain humus, a special program of tree and shrubs planting was designed including the plantation of grasses, 45'000 shrubs and trees:
  - their root system was treated with selected fungi helping the rooting/vegetation process in the sterile slope.
  - A general hydro-seeding of the full area was undertaken step by step, operating remotely, from a helicopter, again to reduce disturbance to the steep slope.

4 USING RISK TO DEFINE THE PROJECT ROADMAP

One of the major challenges faced by the BAMDER was related to the amount of fiber contaminated material to be excavated and disposed of within the mine area in order to unload the over- steepened head of the dump slope. Between the top and the bottom of the slopes 4.5 km of dirt track were present. The preliminary design demanded for the removal of about 280,000 m^3 of residues (mainly sand and gravel) with mixed random asbestos fibers. It was obvious that beside technical hazards and resulting risks there was a strong social, public health component and neighboring communities had to be part of the development in addition to regulators and related authorities.

It was also clear to the design team that risk and uncertainty analyses can and will always be challenged by opposing stakeholders not satisfied with probability assessments based on their subjective, modeled or even "pseudo-statistical" approaches. Under the excuse of limited or poor available knowledge of the problem at stake some stakeholder may invoke the "unrepresentative" character of expert analyses, "mistrust" in the results (as their gut feeling is self-assessed better than a science based approach) to avoid making a sensible decision.

During the implementation of decisions, it is common practice for decision-makers to seek for further protection by adding conservatisms and using traditional engineering frameworks of "defense-in-depth." This is typical of deterministic approaches to hazard and risk management, where layers of protection are added, without explicitly evaluating their effectiveness, to bound known uncertainties to, for example, "credible thresholds." However, these approaches have:

- limited effect in reducing the "unknown unknowns", i.e. reducing the completeness uncertainty.
- They can lead to unsustainable mitigations or to misallocate mitigative budgets.

In particular, these approaches may lead to censored results as they:

- identify a group of failure events sequences leading to credible worst-case accident scenarios called design-basis accidents;
- predict their consequences “with a single magic-number”;
- design appropriate safety barriers which prevent such scenarios and protect from, and mitigate, their associated consequences only

As a matter of fact, accidents in all sorts of industries have shown that the “credible” scenario established in this way oftentimes represent a strong censure of the possible and actually credible ones; codes are generally not covering the full breath of situations that should be covered; mitigations result severely under estimated or misallocated.

The underlying principle has been that if a system is designed to withstand all the individual worst-case credible accidents, then it is “by definition” protected against any credible accident. That does not cover for uncertainties, interdependencies and common cause failures (CCF) which unfortunately do characterize accidents in our complex systems.

Given the complexity and the intricacies of the Balangero project, the design team abandoned the common “engineering approach” described above and accepted to be supported by the systematic application of Risk Based Decision Making based on the deployment of a set of probabilistic techniques which were later consolidated into one platform called ORE (Optimum Risk Estimates, ©Oboni Riskope Associates Inc.) (Fig. 3).

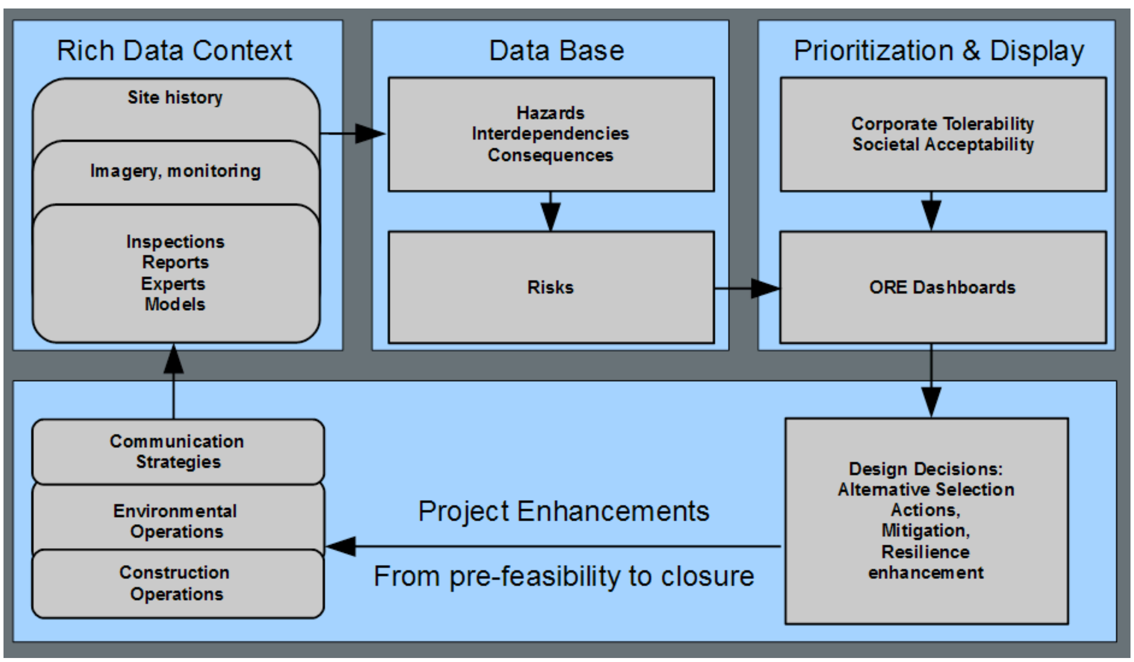

Fig. 3 ORE (Optimum Risk Estimates, ©Oboni Riskope Associates Inc.) scheme/ flow chart

Recently large organizations such as NASA and USNRC have published documents referring to Risk Informed Decision Making (RIDM) a technique that bears astonishing similarities to what was used at Balangero. It is comforting to see that, once in a while, what miners do is not only “moving rocks and dirt” but can be used to “go to the stars”.

RIDM requires to acknowledge uncertainties, multiple hazards, multiple stakeholders and compliance needs, integrating stakeholders' concerns and perceptions.

ORE and the RBDM approach used at Balangero:

- included the uncertainties by working with ranges. As a matter of fact the key point is to guarantee that uncertainties are taken into account in each step of the risk assessment. Uncertainties have to be:
    - systematically identified and classified;
    - represented and described by rigorous mathematical approaches;
    - propagated through the steps of the risk assessment procedure onto the risk measures until the decisions.

- allowed to integrate
    - multiple concerns (multiple stakeholder expectations) and
    - information sources as well as
    - multiple hazard types (natural, man-made, technological, etc.).

This lead to greater stakeholder participation in decision-making, in which technocratic decision processes, driven purely by rational technical considerations, were modified to integrate the concerns and the perceptions of stakeholders.

Thus, the match between ORE RBDM and modern RIDM requirements is perfect.

Modern RIDM requires the classic engineering/technical approach to risk assessment which includes:

- safety margins (Factor of Safety),
- engineered redundancy and diversity to prevent and reduce the impact of failures and stochastic analysts based analyses such as:
- Interdependencies
- Common Cause Failure
- Scenario building and
- Extreme event analysis

to be integrated in the decision-making process while recognizing that all the aspects of a given system under consideration may not be known or ready for analysis simultaneously at different stages of a project/operation development/life.

ORE builds a hazard and risk register that will accompany the project/operation thought its existence. The register is:

- Scalable, meaning it is built in such a way that different levels of knowledge can co-exist in different areas.
- Drillable, meaning that it is always possible to perform queries on the threats-from, threats-to, per process thread, etc.
- Concise, meaning it avoids double counting and fuzzy statements.
- Economical, meaning information is never wasted, there is never the need to "start- over", even if considerable amount of new information becomes available.
- Updatable, if necessary and if data available, up to real-time.

All the argumentation behind the analysis itself, including the assumptions, hypotheses, parameters and their uncertainties can be transparently laid out for disclosure points (however ORE register structure and numerical techniques remain proprietary).

Thus ORE RBDM was already fulfilling RIDM requirements and provided decision-makers with a clearly informed picture of the problem upon which they can confidently reason and deliberate.

Building trust with Stakeholders not satisfied with probability assessments was possible by deploying ORE within a RBDM/RIDM process. The approach helped achieving success by:

- supporting the selection of decision alternatives,
- ensuring that decisions between competing alternatives were taken with proper awareness of associated risks all along the life cycle of the operation/project.

When applied to projects ORE RBDM/RIDM helped avoiding usual project management hiccups like: late design changes, which can be relevant sources of risk, cost overshoot, schedule delays.

5 MAIN RESULTS BROUGHT IN BY USING ORE RIDM

The systematic application of ORE RBDM/RIDM at Balangero brought to develop a series of unusual solutions.

The use of hauling trucks was quickly discarded due to:

- environmental risks,
- air pollution from exhaust fumes and
- fiber dispersion from the excavated material together with the need to upgrade the tracks to roads.

A far better overall multi-hazard risk profile was obtained by installing a temporary aerial tramway (Fig. 4). This device was designed with a single span of 960 m between the two terminal stations to avoid building a tower foundation at mid-slope.

The aerial tramway had a bucket that allowed unloading at ground level as it was possible to lower it from “travel position” to “unload position” anywhere along the tramway path. The layout was selected to minimize lateral movements and allow building a buttress (slope stability enhancement) directly under the tramway.

The cable car was removed at the end of the earth movement works. The excavated material was wetted at excavation time and remained wet during the full trip from the source to the final resting position to reduce fiber dispersion.

Fig. 4: Aerial Tramway for hauling down material. Braking of the tramway produces electricity which is sold to the grid.

Fig. 5: Planted and seeded “pathway” after completion.

The process proved very efficient and only a couple of times, with very strong winds, the dust monitoring instrumentation displayed critical concentrations of aero-dispersed fibers in the surrounding area environment.

The aerial tramway produced electricity which was sold to the grid further reducing the carbon footprint of the project.

6 A REVIEW OF THE OVERALL PROCEDURE APPLIED AT BALANGERO

The overall rehabilitation procedure can be summarized as follows:

- Unload of the upper part of the slope by digging three 10m high berms (Fig. 6) and by storing the excavated material at the bottom of the slope on an artificial earth fill 8 m high using the cable car. The engineered fill is geared towards protecting from possible residual mudflows originated in the steeper eastern part of the slope (up to 42 degrees) the lower part of the slope, the Fandaglia creek etc. (Fig. 1, 2).
- Cut a series of 8 "path-ways", i.e. small berms 2.5 m wide, along the slope at regular height intervals (Fig. 5). The "pathways" are each about 600 m long and were designed to minimize the volume of material to be evacuated. Indeed, the material excavated upslope is deposited down-slope in the same cross section. This procedure dramatically reduces the hauling needs down the slope in the steepest part of it, thus the minimization of fibers dispersion in the atmosphere during works. Furthermore the "pathways" create an access to the slope for present and future works/observation. The "pathways" are reinforced with small palisades built with wood logs (20 cm diameter on the average) increasing the use of natural materials and reducing the need for concrete and steel. Earth totally covers the downhill-side palisades, whereas the uphill palisade remains visible. It is complemented by a geogrid and densely planted to obtain, once vegetation is mature, a "green retaining structure".

Fig. 6: Runoff collection channel on a top berm. Grass has already covered the remodeled sterile fills.

- Build whenever deemed necessary composite wood-earth structures to retain the steepest parts of the slope, or create necessary platforms.

From the hydraulic/water control point of view, surface erosion created deep (up to 3 m) gullies on the slope in the past. The remedial measures undertaken are the following:

- General control of all the surface water falling on the area in form of rain or snow via a net of small wooden channels (on the average 0.5 m to 1.0 m wide). These channels collect surface runoff on the slope thanks to the access created via the top berms and intermediate "pathways". The small dimensions of the channels have been designed to limit the use of heavy equipment on the slope and the need for large excavations for their construction.
- The collecting system is relayed by secondary segments of channels located running on top of the berms and on the "pathways" (Fig. 5).

- The collected runoff is concentrated into four main channels located on the slope along the steepest gradient: these channels – called “water chutes” – are built with wood logs and stones (Fig. 1,7).
- The four “water chutes” finally converge into a unique main canal – built again just with logs and stones – that allows the outflow to reach the Fandaglia creek at the foot of the slope. Before release to the external environment the collected runoff water flows through a decant basin where the fine material and the fibers can be retained (Fig. 2).

Finally, sub-horizontal drains were drilled on the slope to control underground water.

RSA, the owner of the BAMDER project, produced in 2015 an interesting short movie of the works geared toward main-stream public information.

7 CLOSING REMARKS

The BAMDER international competitive bid was won by an engineering group supported by Risk Based Decision Making (RBDM). Today a similar approach is used by large entities under the slight different name of Risk Informed Decision Making.

By winning the international competitive bid it was demonstrated that including quantitative risk assessment through all the phases of the design and construction of a project, from cradle to delivery and including risk driven maintenance concepts brought value and a leading edge to the proponents.

Fig.7: Aerial view of the site. Horizontal pathways and one water chute are visible, as well as pre-existing deep erosion features.

Examples of decisions that were risk-analysis driven and delivered a winning hand to the proponents:

- replacing classic truck-hauling with an aerial tramway,
- reducing the project carbon footprint by using local material and producing electricity (when braking the aerial tramway)
- reducing the volume of displaced materials by creating an earth buttress
- fostering plant growth by carefully selecting species, using selected fungi and
- minimizing future erosion potential (regular maintenance plan).

Since completion the site has endured the passage of climate-change related events (Medicanes, or Mediterranean Hurricanes), and has been monitored by various means, including drones and classic geotechnical instruments.

By using the ORE (Optimum Risk Estimates, ©Oboni Riskope Associates Inc.) quantitative risk assessment platform it is not only possible to develop all the required preliminary risk evaluations, but also to include new information (as from monitoring, for example) as it becomes available, producing regular multi-hazard risk landscape dashboards. These allow decision-makers to understand the evolution of the site and proactively generate road-maps for the maintenance.

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Mechanical Stability of Waste

Hydro-geotechnical analysis of a thickened tailings deposit in Northern Canada

Shunchao Qi, Wiliam Thomas & Paul Simms
Carleton University, Ottawa, ON, Canada

Nuri Hmidi
GoldCorp, Toronto, ON, Canada

ABSTRACT: Rate of rise limitations for thickened tailings technology may be constrained by both geotechnical (stability) and seepage and / or acid generation concerns, depending on site specific conditions. This paper reports an experimental and numerical analysis of hydro-geotechnical characteristics of a thickened tailings site in Northern Canada. The site employs upstream dike construction on the thickened tailings. Measurements of consolidation and unsaturated properties of the tailings are used as input to a consolidation-unsaturated coupled model. Output from the model, including depth profiles of density, strength, and degree of saturation, are compared to field observations. The model is used to explore the effects of changing rate or rise, the influence of deposition time, and increasing the cycling between deposition points.

1 INTRODUCTION

1.1 *Outline of paper*

Thickened tailings technology has been employed successfully at several sites in a variety of climates (Simms 2017). Wider adoption of the technology is probably held up by technical uncertainties related to footprint management and to rate of rise impacts on strength development and acid generation. This paper presents a case study on using experimental data to calibrate and coupled large-strain consolidation – unsaturated flow model, which is then used to analyze field performance of a field site. The analysis provides insight into how rate of rise or deposition cycling may affect dewatering, strength gain, and susceptibility to acid generation.

1.2 *Musselwhite Mine*

Musselwhite mine is a gold mine in Northwestern Ontario that switched from conventional to thickened tailings deposition in 2010 to improve storage capacity of the existing footprint. Tailings deposition occurs at a rate of 3,000-5,500 tpd. Thickener underflow falls mostly between 65% and 68% solids (Kam et al. 2014). The thickened tailings are deposited from a 1.5 km long horseshoe shaped dyke, which is progressively raised on top of the tailings. Deposition cycles from one end of the dyke to the other over the course of the year, and deposition point spacing is 30 m and is changed every two weeks (Kam et al. 2014, 2011). Between 2010 and 2013 the tailings height near the deposition point increased by about 6 m, but by only about 2.5 m on average over the full beach of thickened tailings (about 500 m from the deposition point). The climate is very cold for much of the year. though lake evaporation is about 400 mm per year, while average precipitation is 700 mm. More details on the mine can be found in Kam et al. (2011, 2014).

1.3 *Hydro-geotechnical study of tailings*

Carleton university received tailings from Musselwhite mine collected from a deposition point during active tailings disposal. Tests performed on the tailings included determination of the soil-water characteristic curve, using both axis-translation for low suction and a Dewpoint Hygrometer for high suction, column deposition tests instrumented with pore-water pressure sensors and volumetric water content tests, as well as basic geotechnical characterization tests such as Atterberg limits, specific gravity, and grain size distribution. The function of the column tests, which were done under saturation conditions, was to determine the consolidation parameters (hydraulic conductivity and compressibility as a function of void ratio).

1.4 *Coupled large strain consolidation-unsaturated flow analysis using UNSATCON-ML software*

UNSATCON-ML is a software recently developed at Carleton with the support of the Canadian Oil Sands Innovation Alliance (COSIA) and the Natural Science and Engineering research /council and Canada (NSERC). The software couples large strain consolidation and unsaturated flow and can simulate permanent volume change and wet/dry hysteresis, using a number of different theories, including a state surface approach using constitutive surfaces proposed by Vu and Fredlund (2003), as well more high coupled theories such as the Glasgow coupled model (Wheeler et al. 2003). The theoretical development of the model is described in Qi et al. (2017 a b, c).

The software here is calibrated to the laboratory columns tests and then used to predict the evolution of pore-water pressure, density, and degree of saturation for different deposition scenarios, including the actual deposition practice in the field. The model simulations are compared with actual field behaviour, and provide some insight into the possible influences of deposition control.

2 MATERIAL AND METHODS

2.1 *Materials*

The measured grain-size distribution of the tailings in the laboratory has grain size characteristics of D10, D50, and D90 of 6, 50, and 150 microns, Atterberg limits of LL and PL of 21.5 and 20 (by fall cone method), and a specific gravity of 3.3. By comparison, measurements of the field tailings in Kam et al. (2014) were 10, 50, 150 microns for D10, D50, and D90, with a specific gravity of 3.2.

2.2 *Experimental Methods*

The SWCC was measured using axis-translation with volume change measurement (Fredlund Cell from GCTS in Tempe Arizona) and using a Decagon Model WP4T Dewpoint Hydrometer for suctions larger than 500 kPa. More details on this method as applied to a range of tailings is reported in Simms et al (2017).

Deposition simulations were conducted in 0.50 m tall 0.15 m diameter columns, instrumented with pore-water pressure sensors (Model T5 from MS instruments) and volumetric water content sensors at elevations of 0.05 m, 0.13 m, and 0.33 m. Height of water was recorded using an non-contact ultrasonic displacement sensor, while the tailings-water interface was tracked using a webcam and with the aid of elevations marked on the column. The column was closed at the top, but water could drain through a porous stone at its base. First line of text or heading

2.3 *Numerical analysis*

The UNSATCON-ML software requires constitutive surface for void ratio and water content as functions of total stress and matric suction, as well as hydraulic conductivity function with varies with void ratio as well as degree of saturation. Values of these parameters determined from analysis of a multilayer deposition experiment on another gold tailings (Daliri et al. 2016, Qi et al. 2016), were used as initial estimates, calibration was then performed to fit the column test by adjusting the saturated hydraulic conductivity function only.

3 RESULTS

3.1 *Soil water characteristic curve (SWCC)*

The SWCC is presented in Figure 1. Notably, the air entry value, the point at which the degree of saturation begins to substantially decrease, si about 50-60 kPa, which is somewhat lower than for other hard rock tailings (70-100k Pa, Simms 2017 Bussiere 2004). From the same data the shrinkage limit was 20%, and the void ratio after drying was 0.65.

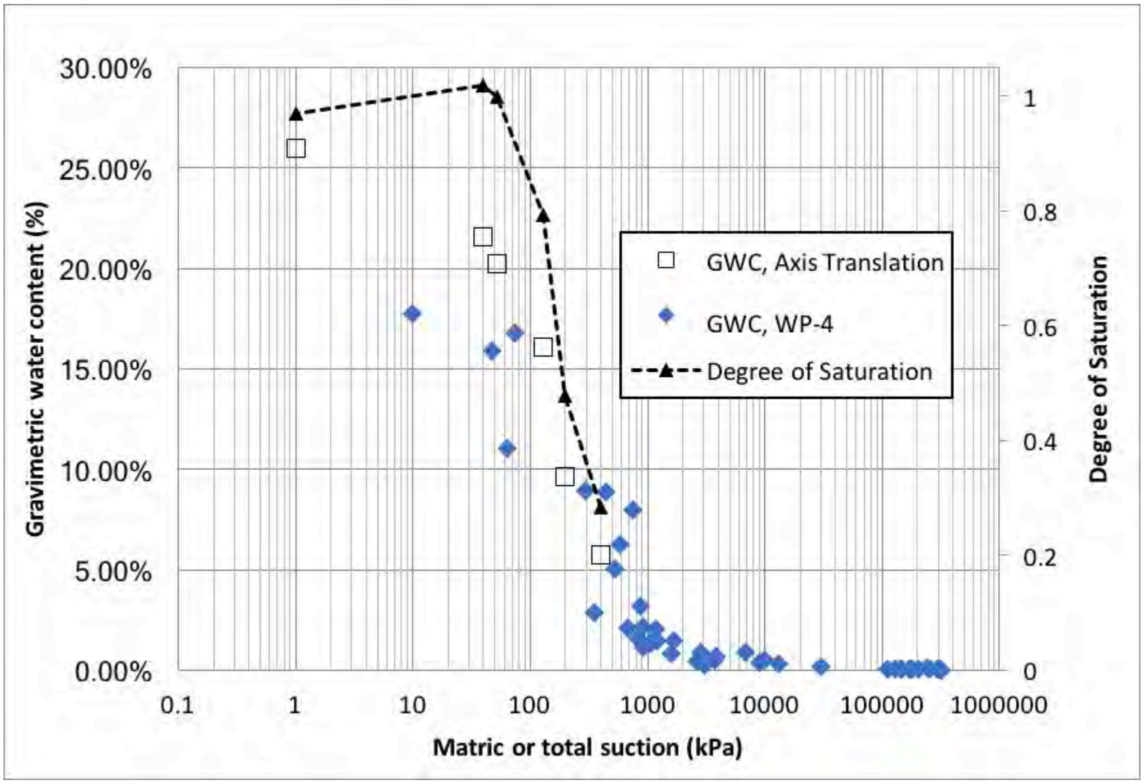

Figure 1. Soil-water characteristic curve of Musselwhite tailings

3.2 *Column tests*

Column tests were performed on 0.50 m high tailings samples at two initial solids contents, 65% and 70%. Duplicate tests were performed for each initial density. Examples of typical data such as Tailings-water interface height and water height, average void ratio, and pore-water pressure variation are presented in Figures 2 through 4. Interestingly, the difference between the final void ratio for tailings at initially 65% or 70% solids is small.

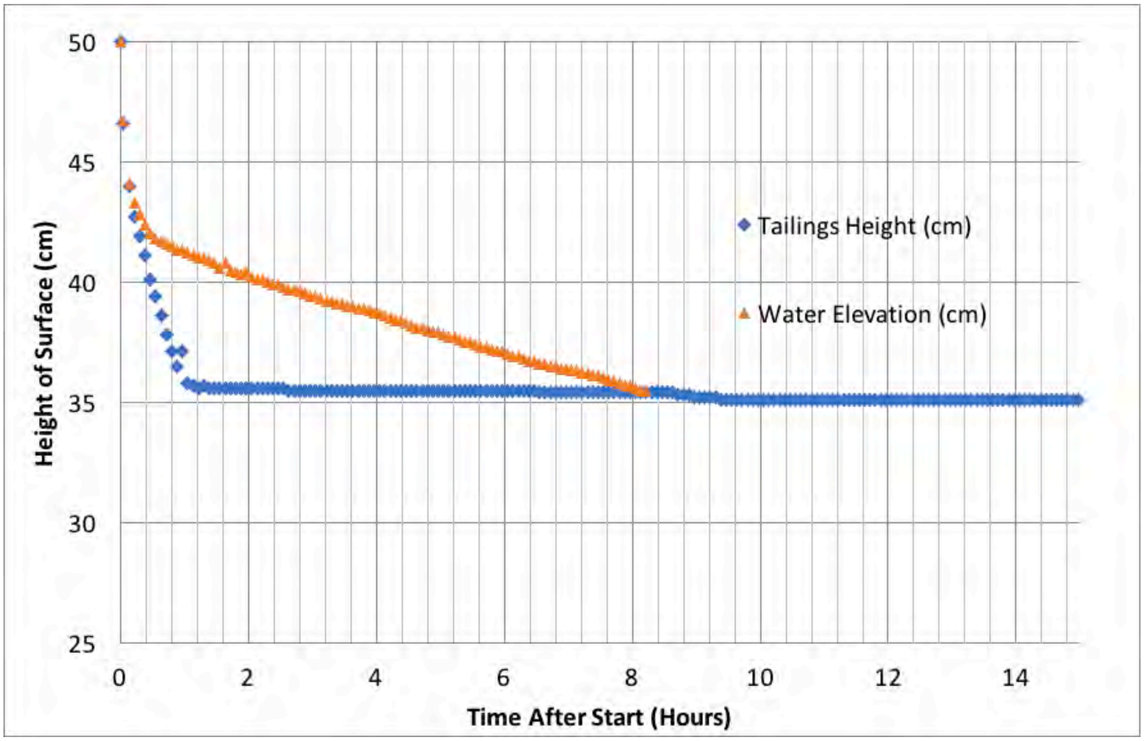

Figure 2. Heights of water and tailings-water interface in column with 70% solids tailings

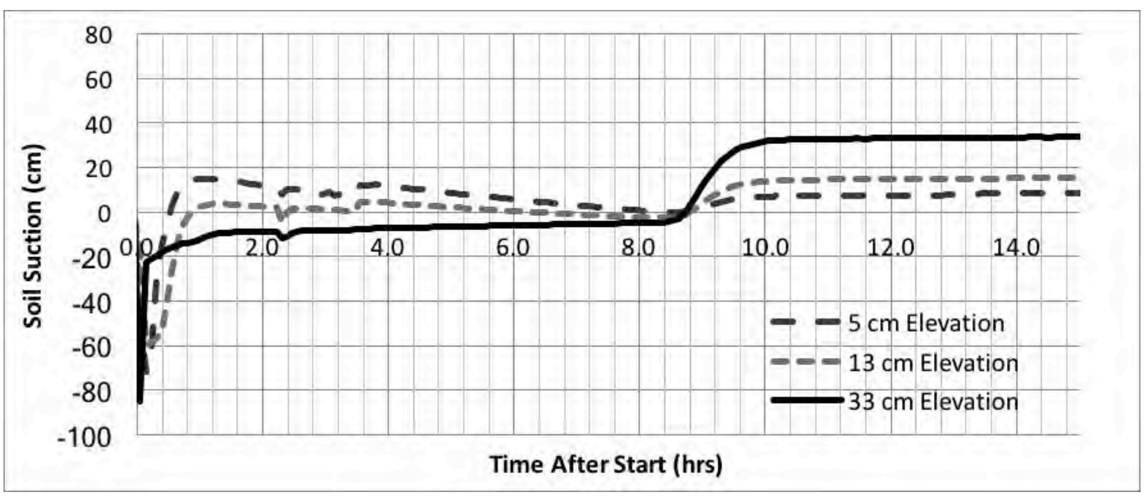

Figure 3. Pore-water pressure in column with 70% solids tailings

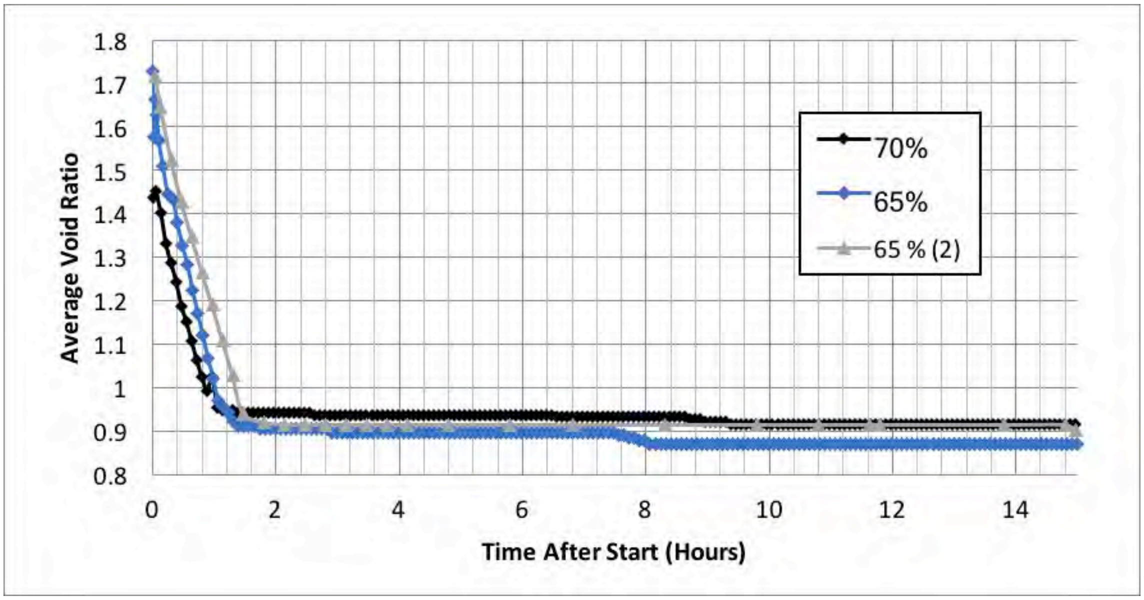

Figure 4. Average void ratio in a number of columns

3.3 *Model calibration to laboratory column tests*

A good calibration could be achieved for a saturated hydraulic conductivity function of 5e-4 x e^{-4} cm/s. The model predicts slightly lower void ratios than in the measurements, though the rate of initial settling of the tailings-water interface and the development of pore-water pressures are well predicted. This level of calibration was judged sufficient for the subsequent field analysis.

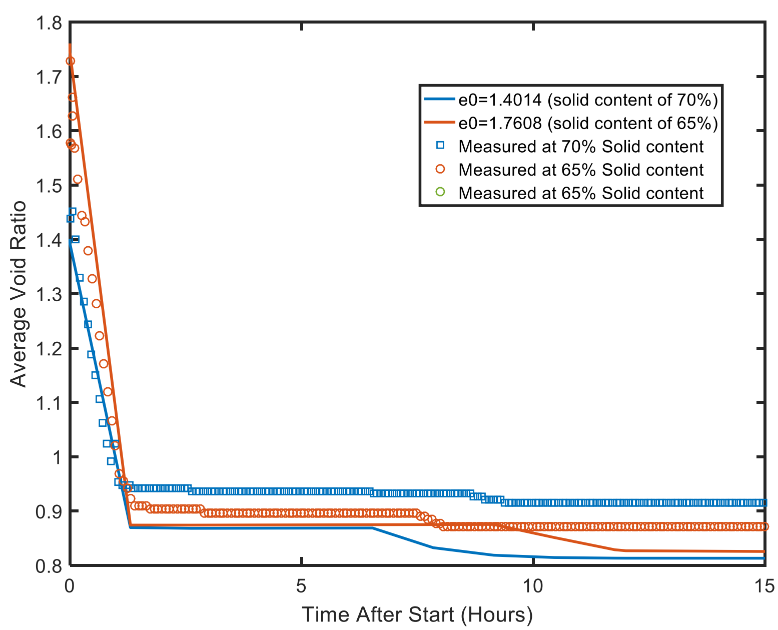

Figure 5. Modelled and measured average void ratios in the column tests

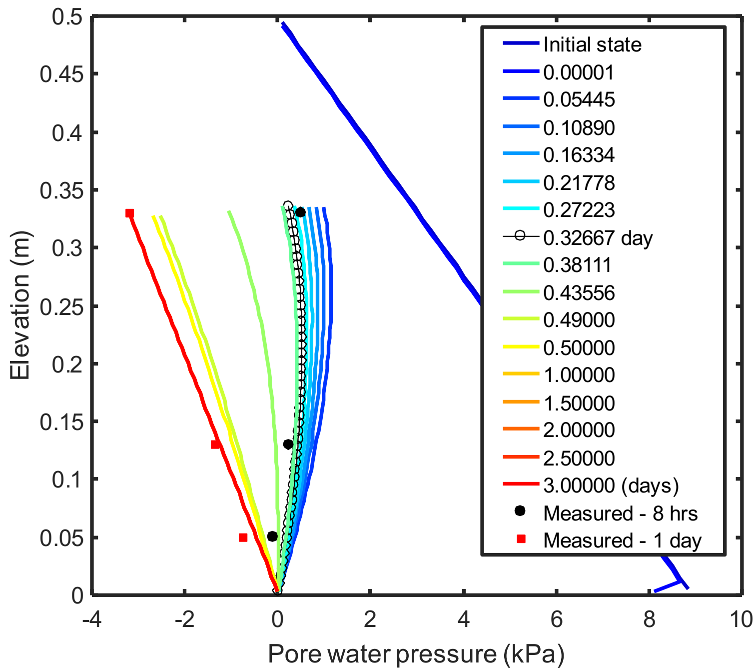

Figure 6. Modelled and measured pore-water pressure in column test with intial solids at 70%

3.4 *Analysis of field deposition*

For investigation of the current and hypothetical deposition scenarios, a number of simulations were conducted, including yearly deposition of 1.5, 2, and 3 m initial thickness, and 1 m deposition twice a year. Deposition time was varied between the start of summer and the start of winter. Summer (May 15th through September 30th) is characterized by an evaporation rate of 2.9 mm / day to give the average yearly lake evaporation. Precipitation is assumed to mostly runoff to the base of the pond. Infiltration of precipitation would tend to increase degree of saturation in the tailings but would not impede volume change.

Results are presented in terms of normalized height, average void ratio (across all layers), and average gravimetric water content in Figures 7, 8, and 9 respectively. Select profiles of void ratio, stress, pore-water pressure and degree of saturation are presented in Figure 10.

Figure 7 is presented in terms of height normalized to the deposition rate. For example, the final height of the deposit with deposition rate of 3 m / year after 3 years is 9 m x 0.67 = 6 m. Why the normalized heights are almost the same for all deposition schemes can be explained by considering the sequence of the different dewatering phenomena. The initial self-weight consolidation phase happens very quickly (within 2 days) for all heights considered, due the relatively high hydraulic conductivity of these tailings. While there is some difference in the magnitude of self-weight consolidation due to the difference in layer thickness and therefore total stress, this become eventually eclipsed by the influence of evaporation and suction, which drives all the tailings to the shrinkage limit. Though total stress does affect the shrinkage limit (the compressibility function is dependent on total stress and suction), for all these heights and stress levels, the variance is relatively small. Therefore, the final void ratio for all these deposition scenarios approach very similar, but not identical, values. This is shown in Figure 8, where the average void ratio in each of the subsequently deposited layers is shown. The difference in void ratio between the scenarios at the end of the simulation, while still small, is most visible in the first layer, due to the differences in total stress because of different heights of the overlying tailings.

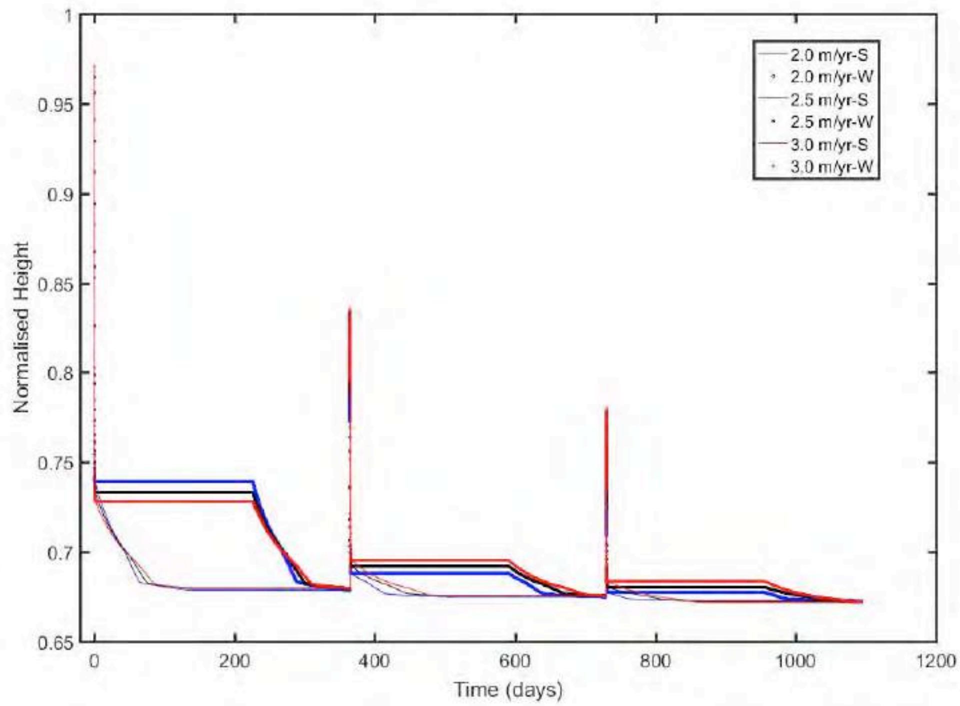

Figure 7. Effect of deposition rate and timing on height.

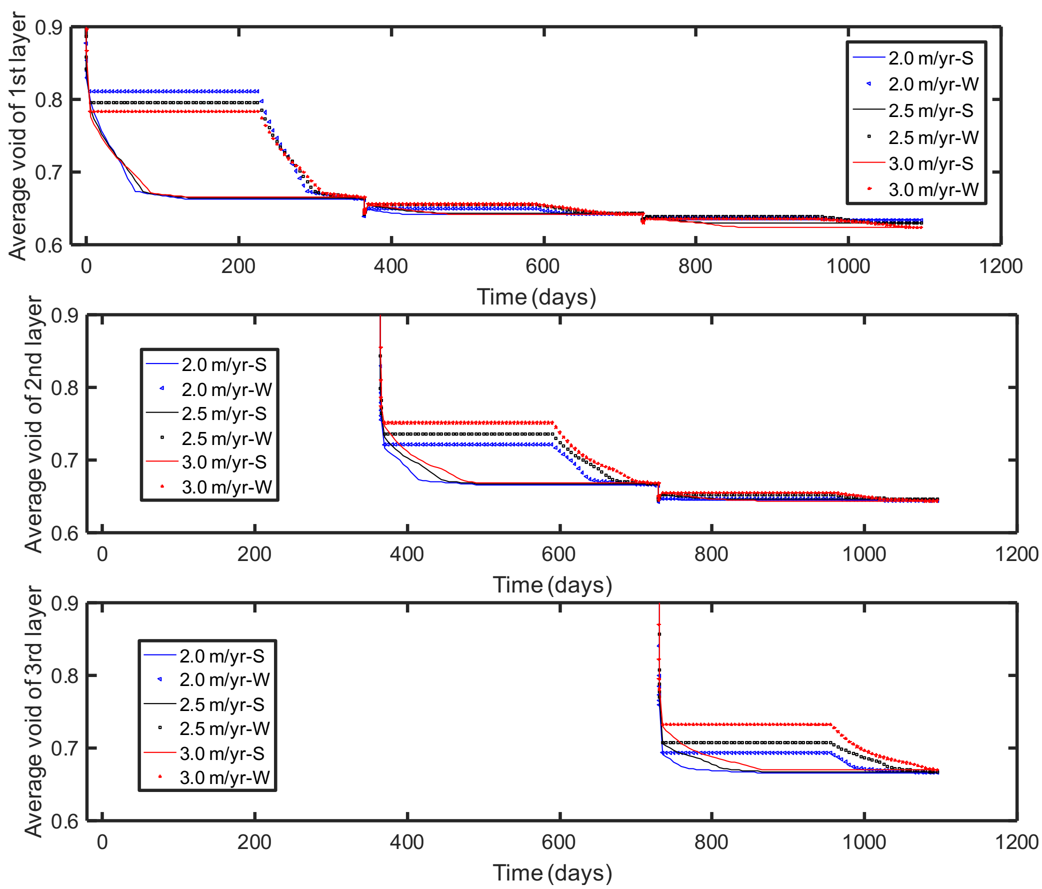

Figure 8. Average void ratios in each of the layers

Winter deposition is characterized by horizontal lines following initial self-weight consolidation (the tailings are not losing water by drainage or evaporation) until the beginning of summer, while summer deposition has a similar but reverse pattern. This is also reflected in the overall gravimetric water content predictions shown in Figure 9. Here, the faster rate of rise results in higher water content and higher degree of saturation, as water removed by consolidation is not very different between scenarios on a per dry mass of tailings basis, whereas the water removed by evaporation is constant and therefore has less of an effect of the thickener deposit.

Of interest is to compare these simulated water contents with those measured in Kam et al. (2014). In that study, water contents during wet periods varied around 27%, while during dry periods the water contents decreased to between 15 and 20% after 40 days. These seem to conform to the “winter” and summer” simulations, respectively, excepting the degree of initial self-weight consolidation, which, as with the laboratory results, is slightly greater in the model (25% compared to 27% GWC).

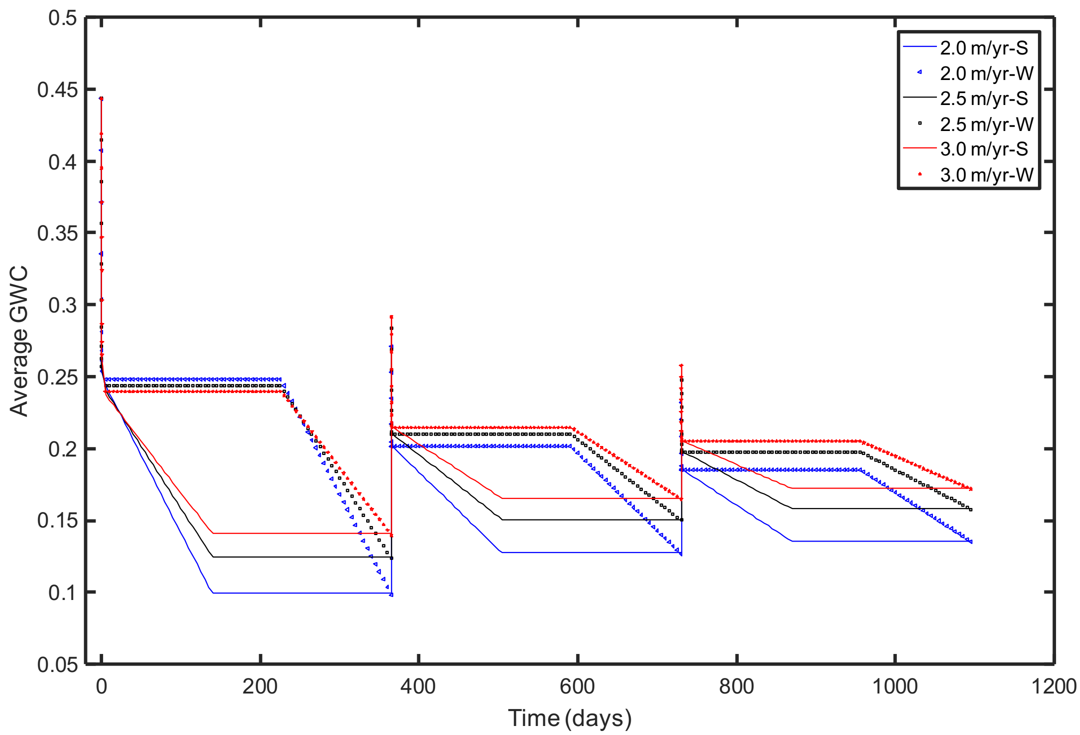

Figure 9. Overall gravimetric water content

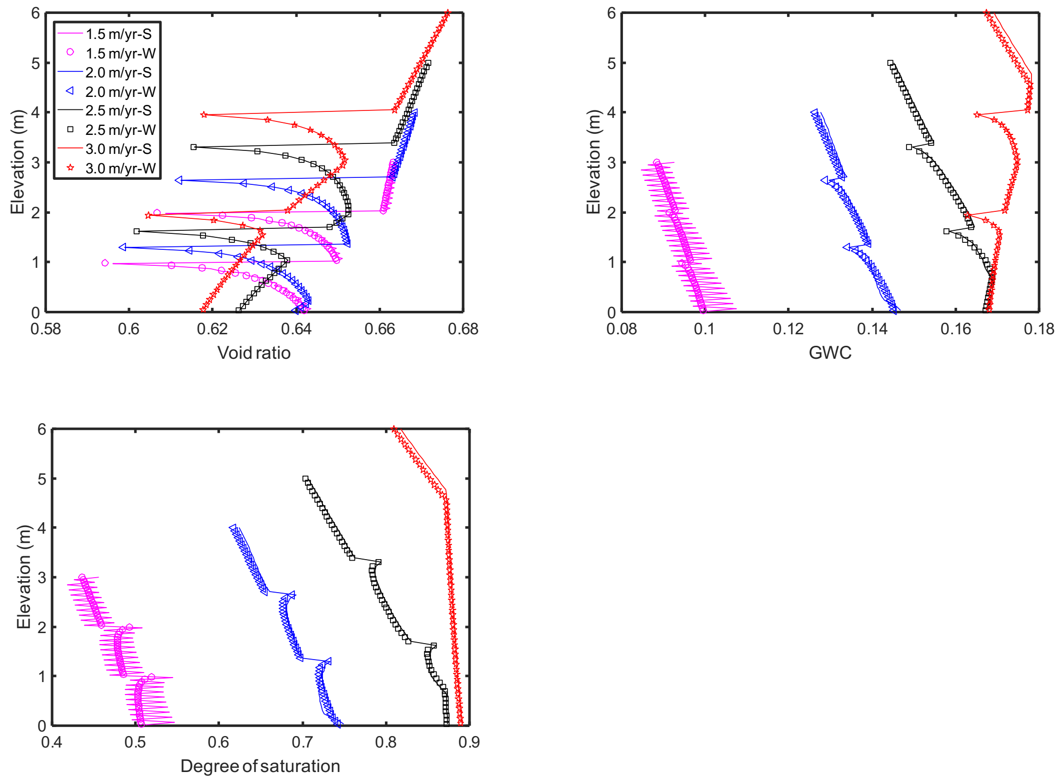

Figure 10. Depth profiles after 3 years of void ratio, GWC, and degree of saturation

While there are slight but probably insignificant changes in void ratio between the simulations, the differences in water content and therefore degree of saturation are significant. Musselwhite tailings do not have acid generating capacity, but at other sites, the low degree of saturation for the case of 2 or 1.5 m deposition per year would facilitate oxygen ingress and might contribute to oxidation. On the other hand, the 17 to 18% GC of the highest rate of deposition might be getting close to the upper bond of deposition rate with respect to strength, and some degree of desiccation in hard rock tailings appear to contribute substantially to strength (Daliri et al. 2016).

Another option is to increase the frequency of lift deposition. Figure 11 shows the difference in average water content in a two layer per year deposition versus a single lift deposition, for the same content (and therefore degree of saturation) are higher for the case of two layer deposited per year. This also limits the exposure time of fresh tailings near to the surface to oxygen ingress. Again, while this strategy might not be useful at Musselwhite, at other sites with high acid generating potential this option could have net value for the mining operation.

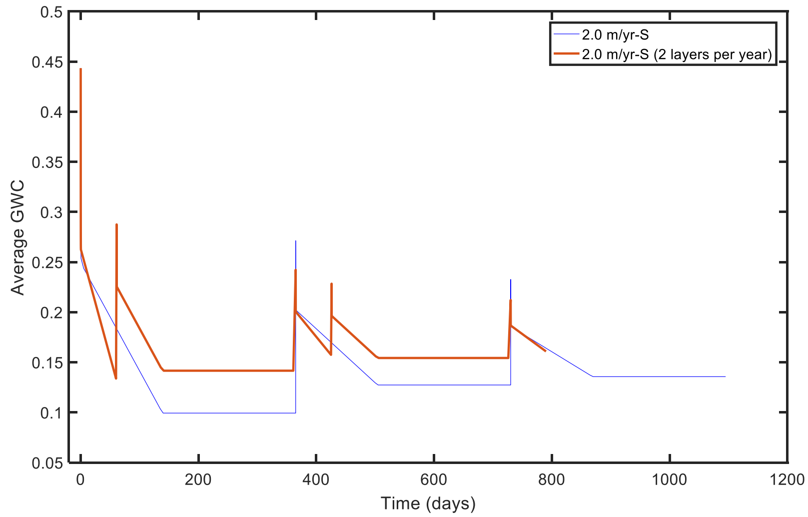

Figure 11 Average gravimetric water content in tailings profile, once a year and twice a year deposition in summer, rate of rise 2 m /year.

4 SUMMARY AND CONCLUSIONS

With input from a laboratory study on the hydro-geotechnical properties of the tailings, field deposition at a thickened tailings site is analyzed using a coupled large strain consolidation – unsaturated flow model. Given the available field data, the model appears to replicate the observed dewatering behaviour. The fast consolidation behaviour of these tailings, and the relatively low rate of rise, permit maximization of volume change for all the rates of rise analyzed (1.5 to 3 m a year), despite the relatively cold climate. Slower rates of rise do induce substantial desaturation of tailings. While not an issue at Musselwhite, this might be a concern at other sites with high acid generating potential. Increasing cycle time and therefore depositing multiple lifts in a year, is shown to provide some value in limiting exposure of fresh tailings to oxidation.

ACKNOWLEDGEMENTS

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Back analyses of the August 2016 Luoyang red mud tailings facility failure

David Reid
Golder Associates Pty Ltd, Perth, WA, Australia

Andy Fourie
University of Western Australia, Crawley, WA, Australia

ABSTRACT: Post-failure analyses were carried out to assess the likely cause of the August 2016 Luoyang red mud tailings storage facility (TSF) failure. The TSF was originally upstream-raised, then converted to a "piggyback" arrangement wherein filtered tailings were placed across the TSF surface. The geometry and staging of the TSF for failure analyses were primarily based on Google Earth images, with material parameters for the tailings assessed parametrically across a range of typical values for red mud tailings. Evaluation of aerial imagery suggests that failure was initiated when filtered material approached the TSF crest. Limit equilibrium analyses carried out with input from analytical consolidation analyses suggest that filtered tailings placement triggered an undrained failure of underlying contractive tailings. Excess pore pressure from incomplete tailings self-weight consolidation, rapid filter material placement, and contractive undrained shearing of tailings are all likely to have played a role in the failure.

1 INTRODUCTION

On 8 August 2016 a slope failure occurred at a red mud tailings storage facility (TSF) at the Xiangjiang Wanji Aluminum plant in Luoyang, Henan Province (AZ-China 2016). Warning time for the failure was apparently sufficient to enable the evacuation of approximately 300 residents from a village immediately downstream of the TSF (Aluminium Insider 2016). The TSF appears to have been an upstream-raised TSF, constructed across an existing valley, later converted to a "piggy back" arrangement (Fourie, 2017) where filtered tailings were stacked above the slurry-placed tailings. The failed tailings submerged most of the downstream village, then flowed approximately 2 km down a valley, where the majority of the flow was stopped by a small embankment. An image of the failure aftermath and extent of flow is shown in Figure 1.

Figure 1. TSF failure and runout material, September 2016

Owing to the importance of awareness of the causes of TSF slope failures, the authors attempted to follow post-failure developments such as rehabilitation works and post-failure forensic investigations. However, negligible information could be found. Owing to lack of further reports, in early 2017 we emailed Luoyang Wanji Aluminium Processing Company to ask if a forensic investigation of the failure would be made public and if not, whether any information could be shared to enable analyses to be carried out. Receiving no reply, we decided to carry out analyses to assess the likely cause and mechanism of failure that lead to the development of this paper. Although we had no first-hand information about the site, as we hope to show subsequently, the high quality aerial photography available for the site was sufficient to enable a reasonable reconstruction of events and TSF geometry.

2 INFORMATION SOURCES AND DATA SYNTHESIS

2.1 *Aerial Imagery*

Petley (2016) was able to identify the location of the TSF and its location by comparing aerial photography to images of the failure published by AZ-China (34.696° N, 112.057° E). We then obtained images from throughout the life of the TSF up to failure, using Google Earth's historical imagery for the site, and the purchase of one commercial satellite image from Harris Mapmart.

While providing lateral distances, and approximate pre-construction elevations, Google Earth is typically unable to provide elevation information for TSFs as the elevation data is not updated to reflect the constructed features. However, as the TSF examined herein was located across a valley, we were able to estimate the elevation of TSF features by checking the elevation where they intersected natural ground on either side of the valley. This process enabled elevation data of the TSF to be extracted along with horizontal distances (for features, such a embankment crests, likely to have limited grade). Geometry development is discussed further in Section 3.4.

2.2 *TSF Development*

Examination of the available data suggests the following TSF development timeline:

- <u>*March 2010*</u>: No evidence of the TSF, or preparation for its construction. The centre of the valley where the TSF embankment will be constructed is at an elevation of about 455 m.
- <u>*January 2012:*</u> The upstream-raised development of the TSF is nearing completion, with a final raise being constructed with imported fill. Owing to the soft nature of surficial tailings, the raise appears to be being pioneered as a thick layer commencing at the north end of the embankment (**Figure 2**).
- <u>*March 2013:*</u> Deposition continues, with the crest elevation at 492 m and remaining freeboard of approximately 5 m.

- *April 2014:* The filter plant has commenced operation, with trucked filtered tailings being used to raise the embankment, possibly to increase freeboard for bypass deposition. It is noted that the location of the filter plant is not visible in the April 2014 imagery. However, its presence is clear based on the route of trucks, and subsequent images (**Figure 3**).
- *October 2014:* Filter material deposited from stackers has begun advancing onto the tailings beach. A significant amount of bypass slurry deposition appears to be occurring during this early stage of filter plant operation. Bleed water from the bypass slurry is ponding in the northern corner of the embankment (**Figure 4**).
- *February 2015*: Stacking of filtered material continues further onto the TSF beach. There is evidence of bow waves ahead of the stacks, and sloughing of some stack material (**Figure 5**).
- *June 2016*: Stacking of filtered material is continuing across beach, with significant sloughing evident near TSF crest. This is final image we could obtain prior to failure (**Figure 6**).
- *September 2016*: Failure has occurred, and appears to have breached the perimeter embankment across a wide extent, based on apparent curvature to embankment crests outside tailings flow area (**Figure 7**).

The lack of any evidence of the TSF in March 2010, and its condition in 2012/2013 suggests a high rate or rise (10 – 15 m/year) at the center of the valley. This is a far higher RoR than typical for upstream-raised TSFs, particularly for similar red mud storage facilities. However, based on the geometry of the TSF and images of wall raising in 2012-2013, it is clear that a significant portion of the embankment was raised using upstream methods.

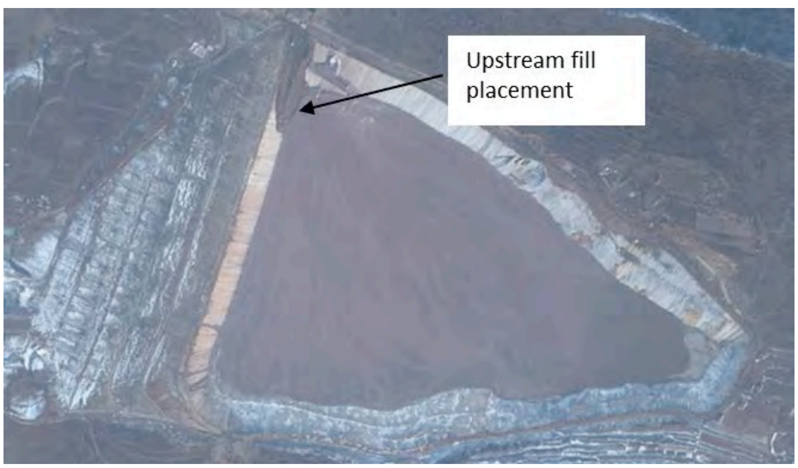

Figure 2. January 2012 – Upstream raising at north end of embankment with thick lift of fill

Figure 3. April 2014 – Wall raise construction with using filtered material

Figure 4. October 2014 – Significant bypass deposition and standing water in north west corner

Figure 5. February 2015 – Filter stacks advancing onto beach, with evidence of bow waves

Figure 6. June 2016 – Filter stack advancing north, near highest point in upstream-raised embankment

Figure 7. September 2016 – TSF approximately one month after failure

2.3 *Likely cause of failure*

Examination of available aerial imagery and other information suggests the following:

- The volume of failed material is too large to have been solely from an overtopping caused by a small slough where the stacked filter material approached the crest
- Supernatant water from bypass deposition was decanted through a trench to a sump at the northern corner of the embankment, which could pose a risk of overtopping. However, the location of the ponded water and collection trench compared to the location of the failure argues against overtopping as the cause of failure.
- No seismic events were reported around the period of the failure.
- Failure was concentrated on the highest portion of the embankment, and there is evidence of deformation of some benches where they are visible after the failure (refer Figure 7).

On the basis of the above, we suggest failure was likely caused by undrained slope instability through the tailings, triggered by placement of filtered material near the crest. This hypothesized failure mode is analyzed further for the remainder of the paper.

3 ANALYSES

3.1 *General*

To assess the potential for slope instability to have caused TSF failure, analyses were carried out as follows:

1) Simplified consolidation calculations were carried out, to assess a potential range of self - weight consolidation-based excess pore pressures from high RoR and filter material placement.
2) Limit equilibrium analyses were then carried out to assess the following stages of the TSF's history:
    a. End of upstream-raising and subsequent raising with filtered material (i.e. 2012 to 2015), to ensure that the rate of consolidation estimated and strength parameters assumed for the analysis were consistent with what had actually occurred – i.e. that the parameters selected did not indicate the TSF would have failed during raising.
    b. August 2016 failure following placement of filtered material upstream of the highest portion of the perimeter embankment.

Stability analyses were carried out using the limit equilibrium code Slide v7.0, using the Morgenstern-Price method of slices. Optimised failure surface geometry was used to identify critical failure surfaces, rather than circular or block failure shapes.

For a stability scenario of the type modelled herein, where there is rapid loading of tailings and subsequent fill, it would generally be more appropriate to use a finite element or finite different stress deformation code to model these processes together. However, such modelling requires far more information on material properties than could be obtained or reasonably assumed.

The slope failure inferred from aerial images was approximately 220 m wide, 40m high, and 160 m in length. For such slope geometry, in a valley setting, underestimation of FoS by perhaps 20% seems reasonable based on studies of three dimensional effects for different slope angle and width/height ratios (for example, Stark 2003). While three dimensional analyses were not carried out as part of this study, cognizance of the likely underestimation of the two dimensional method used is taken. This would suggest a 2D limit equilibrium factor of safety (FoS) result as low as 0.8 to 0.9 might be equivalent to unity in actual 3D geometry of the failure.

3.2 *Material parameters*

3.2.1 *General*

No information on the tailings properties was available, other than that they are red mud. To develop a range of potential inputs, a literature review of publically-available data on red mud tailings undrained shear strength and coefficient of consolidation (c_v) was carried out. These parameters largely control stability of an upstream-raised TSF of saturated, contractive tailings with a high RoR. While there is evidence of red mud tailings exhibiting dilative behavior when sheared from low stresses (Gore 2015, Cooling and Beveridge 2015), this typically only occurs at sites where significant drying occurs, often promoted by amphirolling (a form of mud-farming). Given the high RoR of the Luoyang TSF, negligible air drying was likely.

3.2.2 *Consolidation*

A number of authors have published data on the c_v of red mud tailings (Poulos et al. 1985, Vick 1990). These generally indicated values ranging from 25 to 200 m^2/year.

3.2.3 *Undrained strength ratio*

The relevant strength parameters for the type of failure observed are the undrained shear strength ratio (s_u/σ'_v) in the triaxial compression ($s_u/\sigma'_{v(TC)}$) and simple shear ($s_u/\sigma'_{v(DSS)}$) loading directions. These would ideally be obtained from K_0 triaxial and direct simple shear (DSS) tests. However, there is limited data available in the literature. For reasons that are unclear, most testing on fine-grained tailings to obtain undrained shear strength still seems to consist of isotropically consolidated (CIU) triaxial tests, despite the inferiority of this test method having been made clear long ago (for example, Ladd 1991).

The following undrained strength testing of red mud tailings were identified in the literature:

- Newson et al. (2006) carried out CIU triaxial tests on borehole-recovered samples, which indicated dilative behaviour. However, the sampling procedure described appears likely to have led to significant disturbance, and the samples were tested from stresses that may in some cases have been below the in situ stress.
- Schnaid et al (2007) reported s_u/σ'_v values of 0.3 to 0.4. However, this was with an assumed N_{kt} factor – i.e. no site-specific calibration to a specific loading direction was carried out.
- Gore (2015) carried out CIU triaxial tests on surficial samples, which indicated dilative conditions when sheared from low effective stresses.
- Reid et al. (2015) carried out monotonic direct simple shear (DSS) tests, indicating $s_u/\sigma'_{v(DSS)}$ of 0.25. K_0 Triaxial compression on the same material gave $s_u/\sigma'_{v(TC)}$ of 0.35.

In the basis of the above, an undrained strength ratio of 0.30 was used in the analyses. Compared to c_v, we suggest there is likely much less range in conceivable undrained strength ratio. Therefore, our iterative analyses have focused on varying the magnitude of excess pore pressure that resulted from self-weight consolidation.

3.3 *Rate of consolidation*

Excess pore pressure resulting from the high tailings RoR was accounted for using the "B-Bar" input method. This method allows limit equilibrium codes to account for excess pore pressure generated from rapid placement of fill. In such an application, the "new" fill material is identified to the code, as are layers below the fill likely to exhibit excess pore pressure from loading in such a scenario. The ratio of excess pore pressure within the underlying layers to total stress application from new fill is also applied (i.e., B-Bar value, from 0 to 1). This functionality can also be used, in a simplified manner, to account for excess pore pressure in a slurry from rapid RoR. In this case, the deposited slurry tailings was selected as both the new fill, and the underlying material for which excess pore pressure from loading is relevant. Excess pore pressure from embankment fill placement was also activated.

Table 1 summarises simplified consolidation analyses that were carried out based on the method of Gibson (1958), which allows progress of consolidation to be analysed for an accreting mass of tailings, albeit based on infinitesimal strain assumptions. While clearly finite strain consolidation methods are more appropriate for the Luoyang TSF, such methods require both a void ratio – vertical effective stress profile, and a permeability – void ratio profile. Developing assumed parameters for these profiles is unlikely to be realistic or sensible with available information. Therefore, simplified methods based on inputs of c_v alone were applied.

Table 1. Estimated B-Bar values from range of potential c_v values and drainage conditions

c_v	End of deposition B-Bar		Estimated Reduction in B-Bar,
m^2/year	Top only*	Top and Btm.**	from post-deposition consolidation
50	0.7	0.4	20%
100	0.6	0.3	45%
200	0.4	0.2	90%

*At base of tailings

** At point of highest excess pressure

3.4 *Geometry*

As noted previously, the TSF geometry was estimated based on lateral distances and elevation data from Google Earth. Heights of upstream embankment raises were assumed to align with visible benches on the downstream embankment slope. This assumption typically suggests raises of 10 – 15 m, consistent with the observed raising works in January 2012. An average embankment slope of 1V:4H was indicated.

The location of the section developed in this manner is indicated on Figure 8, along with the approximate extent of the embankment failure. The geometry development outlined with stability analysis results in Sections 3.5 and 3.6.

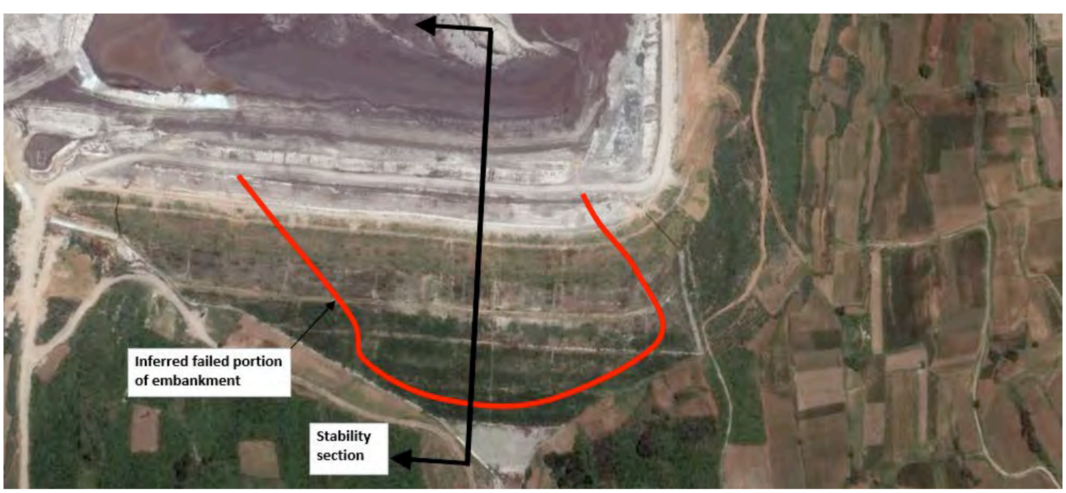

Figure 8. June 2016 geometry, with inferred failure extent and stability section location

3.5 *Upstream raising analyses (prior to main Filter Stack approaching crest)*

Analyses of the upstream-raised TSF and with additional raising by means of filtered material placement with a range of potentially relevant B-Bar values are summarized in Figure 9. A typical failure surface for the TSF with the additional raises of filter material is presented in Figure 10. Note that as no information on foundation soil was available, the base of the TSF was regarded as a rigid interface. This is considered appropriate as the under-consolidated red mud is likely to have been much weaker than the underlying soil.

The results suggest that it would have been possible to construct the TSF upstream at the RoR inferred, assuming reasonable magnitudes of excess self-weight consolidation pore pressure. For example, assuming a B-Bar of 0.2 at end of conventional deposition, or 0.1 during filtered raising (after some dissipation from the previous high RoR during conventional deposition), the resulting 2D FoS for both cases analysed is within 20% of unity – thus likely to have been stable when including 3D considerations. However, the analysis suggests the FoS would clearly have been far below typically acceptable levels for static undrained conditions of a TSF where loss of containment was possible.

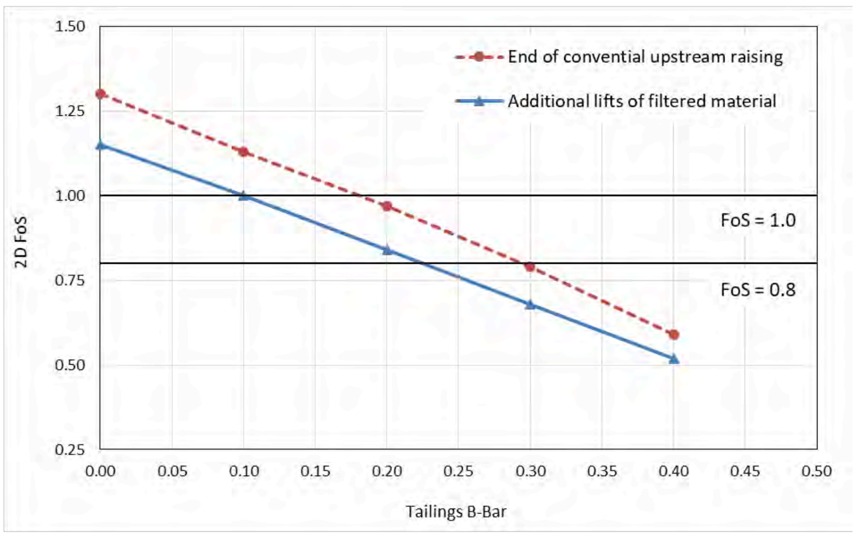

Figure 9. FoS vs. B-Bar for tailings, end of upstream raising and after additional lifts of filter material

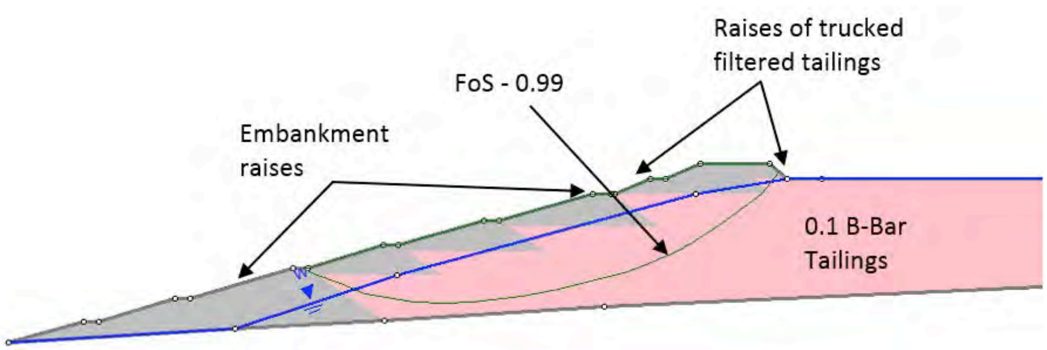

Figure 10. Stability analysis, with additional truck-placed filter lifts, B-Bar of 0.1 for tailings

3.6 *Failure analyses*

Taking cognizance of the range of realistic B-Bar values suggested by the previous analyses (based on self-weight consolidation processes), the inferred geometry of the filtered stack was added to the analyses. The analyses iteratively examined the following variables:

- Different B-Bar magnitudes induced by the placement of the filtered stack itself, i.e. separate to excess pore pressures developed from the high tailings RoR

- Varying the offset of the filtered stack from the crest. As the last image of the TSF available was approximately 2 months before failure, it is unclear to what extent the stack had advanced towards the crest prior to failure.

It was found that when the stack was offset from the tailings crest, application of any significant magnitude of B-Bar within the tailings resulted in superficial failures of the stack through the adjacent tailings beach (separate to its effect on overall perimeter embankment stability). While this is consistent with the behaviour of the stack observed in aerial photographs, it is not the key focus of the failure assessment. Failures of the advancing filtered material would first manifest as slumps, which reduce the slope of the filter stack, and hence may locally increase the FoS of the stack face at a given location. It is clear from the aerial photograph that such sloughing, followed by continued placement of material either from the stacker directly or pushed out by a bulldozer occurred often. Analyzing this process is not carried out herein, rather the potential of the stack material to induce perimeter embankment instability is the primary focus.

The results of the analyses are summarized in Figure 11, with an example failure surface outlined in Figure 12. Figure 12 provides annotation as to the two zones of differing B-Bar within the tailings. Loading due to filtered tailings approaching the embankment crest sugested undrained slope failure was likely under such conditions. Further, the shape of the critical failure surfaces identified indicate large-scale slope instability, consistent both with the quantity of tailings discharged and the observed deformation of lower benches of the perimeter embankment.

Importantly, it is noted that exclusion of B-Bar excess pore pressures within the model resulting from the high tailings RoR, even with the use of undrained shear strengths to represent the slurry-deposited tailings, indicates failure was unlikely. This highlights the importance of multiple forms of excess pore pressure generation (high RoR, contractive shear) on the stability of TSFs, and the different means to account for both in an analysis of contractive tailings undergoing high RoR and/or loading.

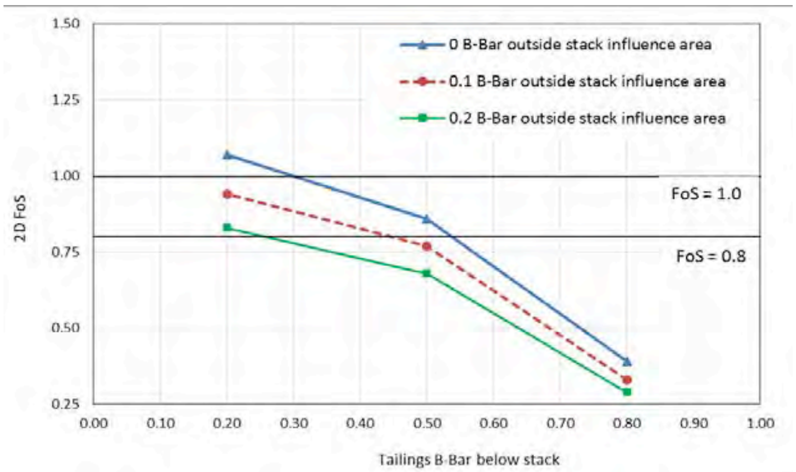

Figure 11. FoS vs. B-Bar for scenarios with filter stack adjacent to embankment crest.

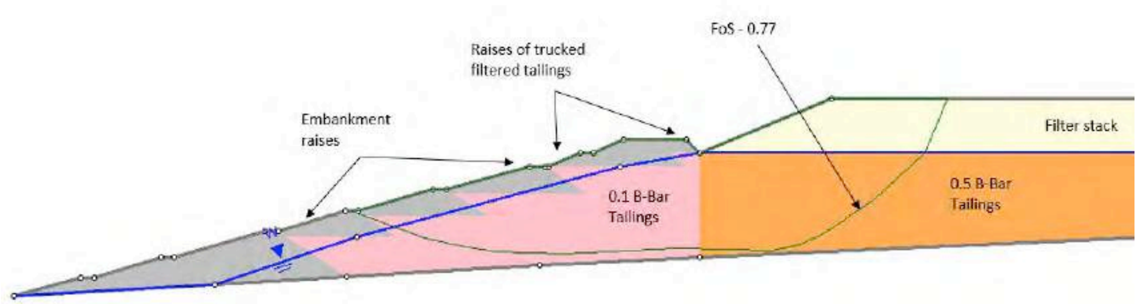

Figure 12. Stability analysis, with B-Bar of 0.1 for tailings under perimeter and 0.5 below filter stack

4 CONCLUSIONS

A red mud TSF failure occurred in Luoyang, China in August 2016. The TSF appears to have been an upstream-raised facility, with filter stacking later carried out in a "piggy back" manner. A review was carried out of available aerial imagery leading up to and after the failure, which suggested that slope instability from filter stack material loading was the likely cause of failure. Based on geometry developed from aerial imagery, and a range of assumed material parameters for red mud tailings, slope stability analyses were carried out to assess the likely cause of failure. These suggested that slope instability likely occurred in an undrained manner, triggered by rapid placement of filtered tailings adjacent to the TSF crest. Excess pore pressure from incomplete tailings self-weight consolidation, rapid filter material placement, and contractive shearing of tailings are all likely to have played a role in the failure.

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Best Available Technologies to Stabilize a Historical Tailings Impoundment

Amy Adams & Ken Brouwer
Knight Piésold Ltd, North Bay, ON, Canada

Scott Davidson
New Gold Inc., Kamloops, BC, Canada

ABSTRACT: Historical tailings impoundments may contain saturated, semi-fluid materials at depth long after tailings deposition has ceased and after surface reclamation has been completed. These saturated materials can liquefy and flow if the impoundment is compromised. A historical tailings pile can also present a risk to an underground mine development if there is potential for the mine development to generate a propagating zone of cracking and/or surface subsidence that ultimately interacts with the tailings impoundment. The risk of a sudden mud-rush breach can be mitigated by using ground improvement techniques to reduce the potential for the tailings to flow.

This paper presents a case study for the New Afton Mine located in British Columbia, Canada. The opportunities to use best available technologies to reduce the risks associated with operating and closed tailings facilities are presented and discussed. The emphasis is on the novel application of simple and proven technologies to efficiently and effectively stabilize the tailings to achieve the ultimate objectives for dam and mine safety.

1 INTRODUCTION

Tailings are variable in nature and the disposal techniques vary significantly depending on the mining methods/rates, the ore characteristics, the site conditions and also the environmental, social and economic considerations that prevailed during the permitting, construction and operational stages of the mine. Tailings impoundments often continue to represent an ongoing liability, long after mine operations cease and the surface facilities have been closed and reclaimed. Dams for decommissioned or inactive tailings impoundments need ongoing care and maintenance for as long as these dams are required to securely retain ponded water and/or potentially liquefiable tailings solids within the impoundment.

The dams will no longer need to function as dams, and the tailings pile can be considered to be a stable landform, only after:

- the impoundment surface is suitably dewatered, shaped and capped so that it can no longer retain a surface water pond
- the tailings mass at depth is suitably consolidated and/or drained such that there are no longer 'flowable' materials that could generate a mudflow capable of leaving the site boundary, and
- the facility can be considered to have a risk profile similar to the surrounding environment (MEM, 2008)

The surfaces of many tailings impoundments have been reclaimed by shaping, capping, and revegetation, but there are fewer examples where the tailings pile can be shown to be suitably stabilized with no potentially flowable materials. Best Available Technologies (BAT's) can be used to reduce the risk associated with operating and closed tailings facilities. The stabilization of the entire tailings deposit is possible when filtered tailings are produced and compacted during placement to form a suitably dense dilatant soil material that would not be prone to liquefaction, even if the tailings landform became re-saturated at any time in the future. Similarly, dense drained tailings deposits have been successfully developed when controlled, thin layer sub-aerial tailings deposition in suitably arid climates is coupled with underdrainage to develop dense, non-liquefiable tailings deposits that meet both of the above criteria (Knight & Haile, 1983; Haile & East, 1986; Ulrich et al, 2000).

However, many historical tailings impoundments were progressively developed using relatively simple hydraulic slurry placement within a flooded or partially flooded facility. These 'conventional' hydraulically emplaced tailings deposits form a relatively loose and somewhat segregated mass of interlayered sandy and silty materials, with the finest grained silty and clay-sized (slimes) particles typically deposited farthest from the discharge points. These hydraulically emplaced tailings deposits are typically comprised of materials that are contractive and prone to liquefaction, particularly in the upper 20 to 40 meters of the deposit where they are less consolidated than at greater depths. The geotechnical characteristics of tailings sand and slimes deposits are very different. Operational factors, such as changing ore characteristics, grind size, or changing the tailings facility filling rate or discharge locations (spigots vs. single point discharge), can also significantly influence the geotechnical properties of the sands and slimes within the heterogeneous tailings pile that is developed during mine operations. The 2015 Samarco tailings failure (Morgenstern et al, 2016) provides an example where loose, saturated tailings liquefied and created a catastrophic mudflow that rapidly migrated downstream. The resulting failure resulted in the inundation of a village and caused 19 deaths.

The geotechnical characteristics of a tailings impoundment can also become a critical factor in the success of a project when underground mining activities extend laterally and mine-induced deformations result in cracking or surface subsidence features that may interact with the tailings impoundment. Water and/or fluidized tailings materials can represent significant risks to an underground mine development due to the potential for a catastrophic mudrush. A mudrush event occurred at the Mufulira mine in 1970, in which 89 underground miners lost their lives when ponded water and liquefied tailings created a highly fluid slurry that rapidly flowed into the underground workings through mine-induced cracks. Post disaster forensic investigations led to the development of remedial drainage measures within the remaining surface tailings pile in order to stabilize the materials and allow safe underground mining operations to be resumed (Sandy et al, 1976). Mufulira was a relatively shallow mine compared to the New Afton block cave, but serves as a relevant case history nonetheless.

This paper presents a case study for the New Afton Mine located in British Columbia, Canada. The opportunities to use best available technologies to reduce the risks associated with both operating and closed tailings facilities, are presented and discussed. The emphasis is on novel application of simple and proven technologies to efficiently and effectively stabilize the tailings to achieve the ultimate objectives for dam and mine safety. A case study focusing on the site investigations and tailings characterization related to the design and progressive implementation of selected best available ground improvement technologies is further discussed in our companion paper "Characterizing and Stabilizing a Historical Tailings Facility: The Rheology to Soil Mechanics Continuum" (Adams et al, 2017a).

2 PROJECT OVERVIEW

The New Afton Mine is a copper gold mine located approximately 10 km west of Kamloops in British Columbia, Canada (Figure 1). The New Afton Mine occupies the site of the former Afton Mine that was historically developed from 1978 to 1997 using open pit mining methods. Conventional flotation processes produced a tailings slurry that was hydraulically discharged at approximately 35% solids content via multiple spigots into a nearby tailings storage facility (TSF). The historical mine site was closed and the surface facilities were partially reclaimed. In 2005 New Gold Inc. acquired a portion of the overall property and continues to develop an underground block caving mine operation to exploit deeper mineralized zones. Slurry tailings from the current mill are disposed of in the active New Afton TSF, as illustrated on Figure 1.

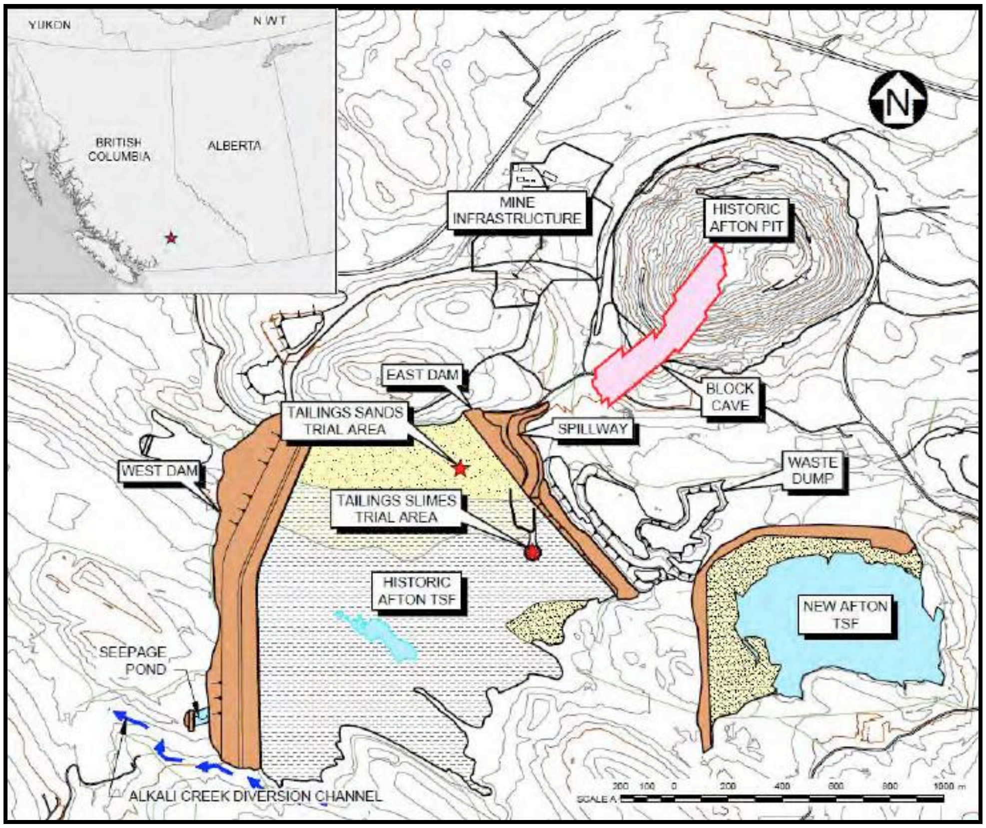

Figure 1 Project Location and Site Layout

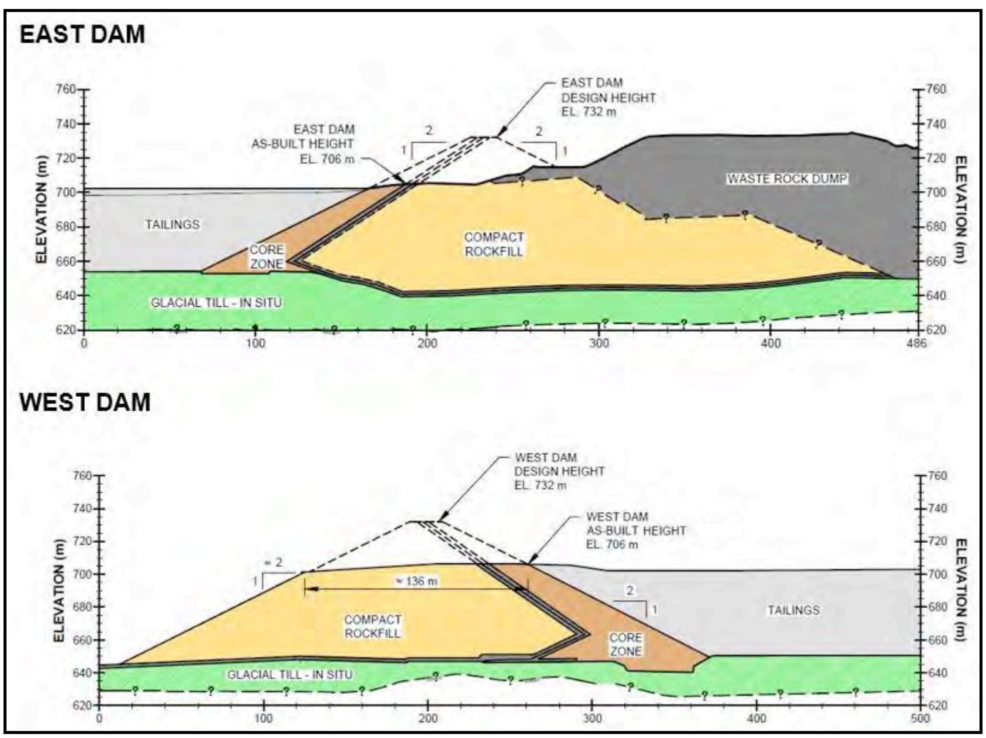

Figure 2 Typical West and East Dam Sections for the Historical Tailings Impoundment

The Historic Afton TSF (Historic TSF) includes two zoned earthfill/rockfill dams (Figure 2) that were constructed in stages during the previous mine operations. The West Dam is approximately 75 m high and has a crest width greater than 100 m. The East Dam was constructed to a height of approximately 65 m (Elevation 706 masl). The current East Dam crest is greater than 100 m wide and was intended to be raised an additional 26 m to the ultimate impoundment elevation of 732 masl (Figure 2). Both dams were constructed using compacted rockfill with an upstream core zone of compacted glacial till. The East Dam is buttressed by an extensive waste rock dump that is higher than the dam itself and is immediately downstream of the facility. The underground mine has the potential to induce deformations and cause cracking and surface subsidence that may impact the East Dam.

Water management for the Historic TSF features include an upstream diversion channel to safely route surface runoff from up-gradient catchments around the West Dam, as well as an emergency spillway constructed along the left abutment of the East Dam. The TSF spillway was sized to safely route runoff from extreme storm events into the historical open pit east of the TSF. The TSF water pond was previously removed. The semi-arid climate provides greater annual evaporation than precipitation and the TSF is thus in a negative water balance condition; it is expected to stay dry with no surface ponding in the long-term.

3 HISTORIC TSF BREACH CONSIDERATIONS

3.1 *Background*

A surface breach of a tailings retaining structure could result in the release of the entire supernatant pond volume and full or partial discharge of the impounded tailings (Fontaine & Martin 2015). Two discharge mechanisms are typically observed during a TSF dam breach including: (1) an initial flood wave of supernatant water mixed with tailings and embankment materials that may travel 10's to 100's of kilometres downstream, and (2) slumping or flow of liquefied tailings that may result in a smaller inundation footprint to the downstream environment. The outflow volume of the breach can be estimated based on the volume of the supernatant pond and the characteristics of the deposited tailings including their density, moisture content, and saturation. Fontaine & Martin (2015) demonstrated that reducing the size of the supernatant pond yields a linear reduction in the size of the breach outflow volume, whereas increasing the dry density of the tailings yields an exponential decrease in the volume of the breach outflow.

3.2 *Dam Breach Analyses*

A detailed dam breach and inundation assessment for the Historic TSF was completed in 2015 (Akkerman & Martin, 2015). The East Dam was classified as a HIGH consequence structure based on the Canadian Dam Association Dam Safety Guidelines (CDA 2007, revised 2013) as a hypothetical dam breach could outflow into the Historic Afton Pit (see Figure 1).

Two failure conditions were considered during the dam breach assessment: (a) *Sunny Day* failure, or failure during normal operating conditions that would be caused by a dam collapse due to any circumstance; and (b) *Rainy Day* failure, or flood-induced failure that would be caused due to dam overtopping. It is difficult to imagine any of these failure modes as plausible given the following characteristics:

- the arid local climate,
- the large capacity of the spillway,
- the apparent robustness of the embankment design and extensive crest width,
- the negative water balance resulting in a typically very small or non-existent pond, and
- the large storage capacity between the tailings surface and the dam crest.

Full or partial blockage of the spillway would be required to cause overtopping of the facility during a Probable Maximum Flood event. It is unlikely that this would develop under the continued monitoring and maintenance resulting from the ongoing mining activities at the site. It is important to recognize that the surface pond was removed by pumping to the New Afton TSF in early-2015 and the remaining surface water subsequently evaporated throughout the summer of 2015. Seasonal accumulations of small volumes of water in depressions on the surface of the facility do exist but these are very shallow and limited in size.

Nevertheless, a dam breach scenario cannot be completely discounted given the semi-fluid nature of the stored tailings and the potential for a pond to form after an extreme storm. Removal of the supernatant water pond in 2015 and construction of the Alkali Creek diversion channel have substantially reduced the potential for an initial flood wave to develop during the unlikely event of a dam breach event. This can be further reduced by selective surface grading to eliminate the potential for ponding during a storm event. Furthermore, densification and dewatering of the tailings using appropriate ground improvement technologies will reduce the volume of interstitial water, increase the solids content, and preclude liquefaction. This will in turn eliminate the potential for the outflow of liquefied tailings during a breach event, reducing the consequences to localized slumping. Combined with the very low probability of a dam breach occurring given the robust design and climatic setting, credible failure modes are eliminated once the tailings are suitably stabilized and capped to prevent water ponding during extreme precipitation events.

3.3 *Interaction with the Underground Mine*

The New Afton underground mining method commenced in 2012 and will result in surface cracking and subsidence that is conservatively postulated to potentially interact with the overlying Historic TSF as the mine development becomes progressively larger and deeper (Figure 3). Worker safety is a primary and fundamental requirement for ongoing mining operations. Sophisticated monitoring systems are coupled with extensive numerical modelling to track deformations and to enable accurate prediction of the influence of future mine developments on surface facilities. The proximity of the Historic TSF to the underground mine has been identified as a potential risk factor and represents a potential mudrush hazard, unless appropriate tailings stabilization techniques are implemented as mitigation measures.

Densification, dewatering, and reduction of the potential for liquefaction were identified as critical objectives for the ground improvement program. The tailings in the Historic TSF are segregated with Tailings Sands (silty sand, less than 70% fines) in the north and silty Tailings Slimes (low to high plasticity clays) in the southern portion of the facility as illustrated on Figure 1. High in-situ moisture contents (5 to 45% for the Tailings Sands and 30 to 90% for the Tailings Slimes) suggest that some of the tailings could behave more like a fluid than a soil when disturbed/liquefied.

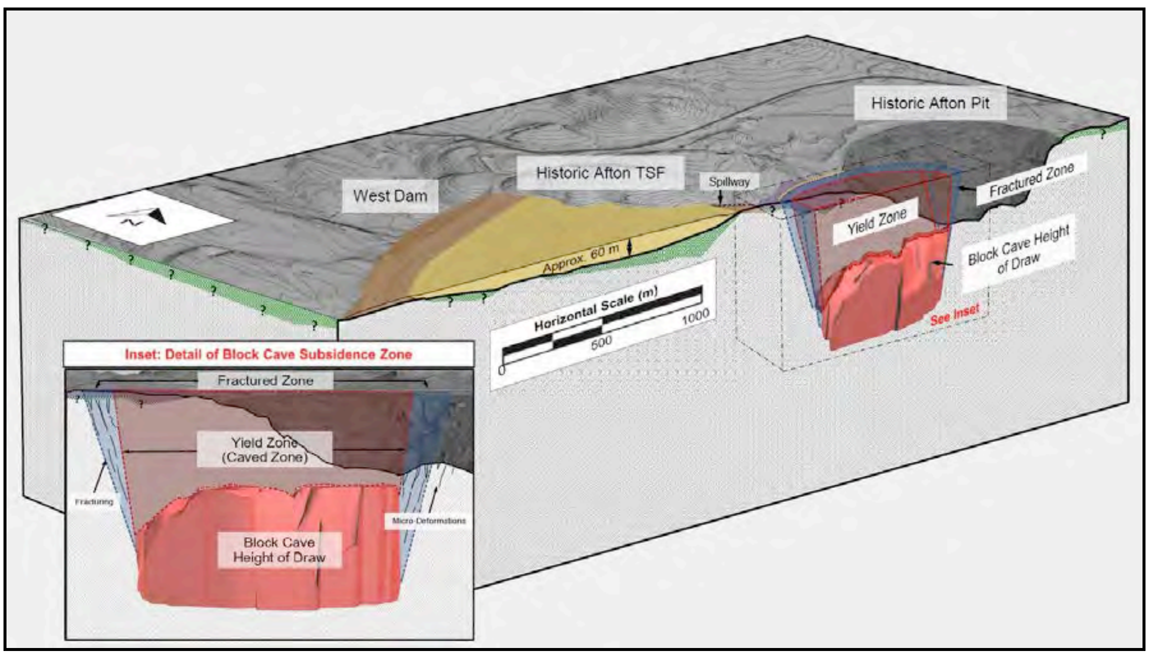

Figure 3 Potential Interaction between Underground Mine and Historical Afton TSF

4 GROUND IMPROVEMENT TECHNOLOGIES

Ground improvement technologies can be used to improve soil properties, including increasing the density, reducing the water content, increasing the shear strength, reducing the permeability, or causing changes that may alter the way the soil deforms. Such modifications can be used to improve the soil bearing capacity or slope stability, or to reduce the liquefaction potential.

Ground Improvement technologies that are commonly used to stabilize weak, liquefiable soils include the following (Hayward Baker, 2017):

- Soil mixing (wet or dry) with cement or other additives
- Wick drains, earthquake drains or de-watering wells
- Jet grouting or compaction grouting
- Stone columns or vibro compaction
- Ground freezing
- Consolidation loading
- Dynamic compaction or explosive compaction

Reducing or eliminating the risk of liquefied tailings flowing from a hypothetical TSF breach or a mudrush of liquefied tailings flowing from the Historic TSF into the underground mine workings requires removal of the carrier fluid (water). The first and simplest objective is to remove the surface water pond, and the second objective is related to dewatering the historical tailings in order to increase the yield stress and reduce the flowability of the heterogeneous deposit.

Two tailings stabilization methods were selected for field scale evaluation. Dewatering with pumping wells combined with wick drains was selected to reduce the moisture content and lower the piezometric surface in the coarser tailings (Tailings Sands, Figure 4). Compressive loading combined with wick drains was selected to consolidate, densify, and reduce the moisture content of the finer tailings (Tailings Slimes, Figure 5). Wick drains decrease the drainage path and promote more rapid consolidation.

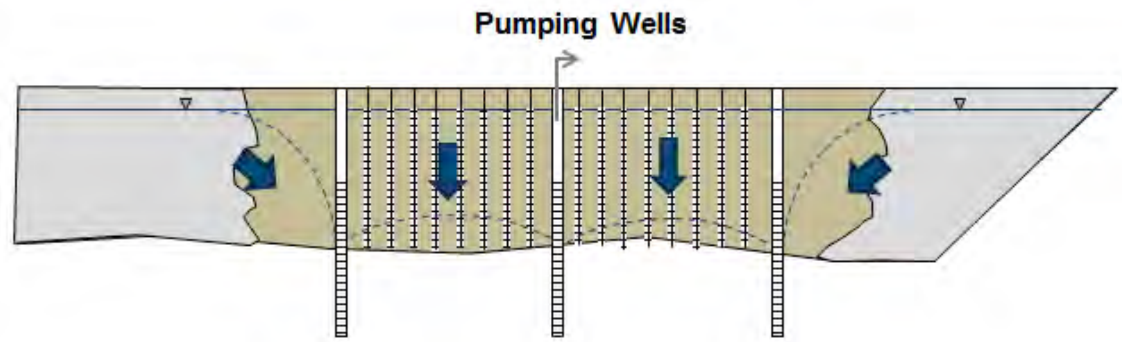

Figure 4 Schematic ground improvement technology for the Tailings Sands consisting of pumping wells and wick drains to reduce the moisture content and lower the piezometric surface.

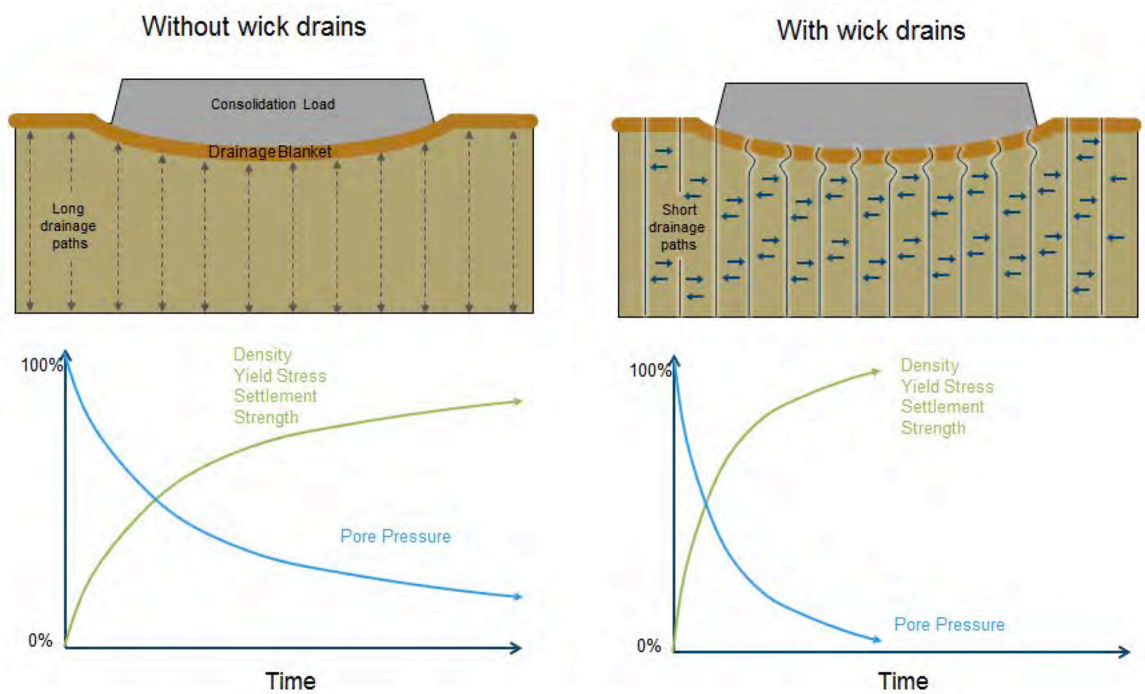

Figure 5 Schematic ground improvement technology for the Tailings Slimes consisting of a compressive load constructed over tailings amended with wick drains.

Explosive compaction was also considered as a potentially applicable ground improvement technology (Figure 6) Explosive compaction causes a rapid increase in pore pressure at depth through detonation of explosive charges. Dissipation of the excess pore pressure induces consolidation and densification of the tailings mass. Wick drains can be installed to promote consolidation. Explosive compaction was not evaluated in the field trials but was considered to be a potentially applicable ground improvement technology based on precedent at other sites. It could be implemented as a contingency measure for targeted zones within the stabilized tailings mass if needed.

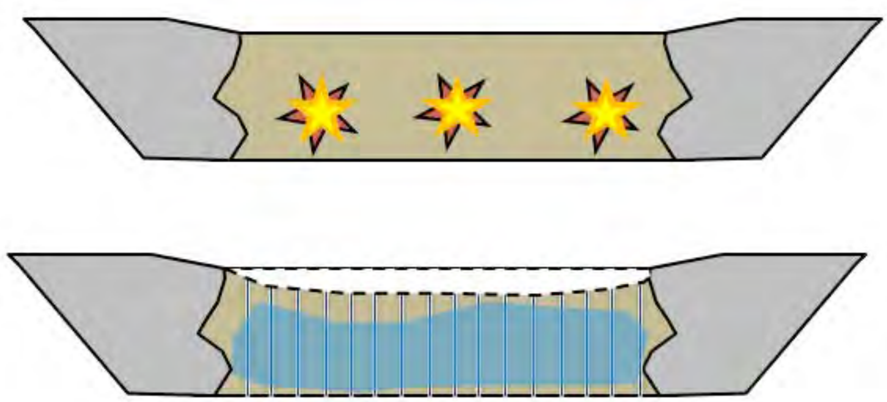

Figure 6 Schematic explosive compaction ground improvement technology consisting of the detonation of explosive charges to induce a rapid pore pressure increase with subsequent consolidation and densification.

5 TRIAL PROGRAM

5.1 *Trial Program Description*

A field scale trial program was developed to evaluate the effectiveness of the selected stabilization methods for the Tailings Sands and Tailings Slimes and to collect field scale data to support the full scale design (Adams et al, 2017b).

Specific details concerning the tailings characterization including site investigations and laboratory testing to support the field trial design, as well as detailed analysis are provided in our companion paper entitled "Characterizing and Stabilizing a Historical Tailings Facility: The Rheology to Soil Mechanics Continuum" (Adams et al, 2017a). The following sections summarize the field trial programs for the Tailings Sands and Tailings Slimes.

5.1.1 *Tailings Sands*

An area approximately 100 m long and 50 m wide was selected for study in the Tailings Sands are of the impoundment. Pump tests were completed using monitoring wells and Vibrating Wire Piezometers (VWP's) to monitor pore pressures and drawdown. Wick drains were installed in a 5 m triangular pattern to 33 m depth.

The pumping tests demonstrated that conventional pumping wells screened through the Tailings Sands can effectively dewater the Tailings Sands, thereby increasing the yield stress and reducing the flowability. This in turn mitigates the risk of the tailings flowing from the TSF during a hypothetical breach scenario as well as the mudrush risk for the underground mine. Wick drains may also be installed in the portion of the stabilization zone where the tailings sand is interlayered with finer silts and clays (Tailings Slimes) to increase the vertical hydraulic conductivity of the Tailings Sands and enhance dewatering. This will help drain groundwater perched above low permeability horizons and reduce the potential for groundwater compartmentalization.

5.1.2 *Tailings Slimes*

An 11 m high and 50 m diameter conical fill load with access ramp (Test Pad), shown on Figure 7, was constructed over the approximately 25 m thick Tailings Slimes using staged construction techniques over a two month period. Vertical wick drains were installed in a 2 m triangular grid pattern to increase the rate of tailings densification, moisture reduction, and strength gain. VWP's and survey monuments were installed for pore pressure and settlement monitoring. The ultimate as-constructed thickness of the Test Pad was approximately 10 to 11 m after accounting for settlement during construction.

The tailings below the centre of the Test Pad on the slimes area compressed approximately 2.2 m vertically as a result of the applied fill loading. The tailings along the edge of the Test Pad settled approximately 0.25 to 0.50 m vertically. No signs of major displacements, either vertical or lateral, or slope instabilities in the test pile were observed.

A - Installation of Wick Drains

B - Fill Placement

Figure 7 Test Pad Construction

Follow-up site investigations including SCPT probings and hollow stem auger drilling. Undisturbed samples were collected from the auger drilling. The moisture content, density, and yield stress of the tailings were evaluated. The results demonstrated an increase in the CPT tip resistance (q_t) implying an increase in density, a decrease in the moisture content, and an increase in the estimated yield stress for the majority of the underlying Tailings Slimes. This showed that consolidation loading, combined with wick drains, is an effective ground improvement technology to densify, dewater, and reduce the liquefaction potential of the Tailings Slimes.

6 SUMMARY AND CONCLUSIONS

The New Afton Mine occupies the site of the former Afton Mine. The Historic TSF is currently inactive and was last operated by the Afton Mine using hydraulic emplacement of slurry tailings from 1978 to 1997. It contains approximately 37 million tonnes of saturated tailings solids, which were naturally segregated during multiple spigot tailings deposition at approximately 35% solids to form sandy tailings beaches along the north side of the impoundment and finer grained slimes tailings deposits along the south. The impoundment is constrained by the 75 m high West Dam and the 65 m high East dam.

A stabilization program is planned to improve the in situ characteristics of the historical tailings in order to achieve two fundamental objectives:

- the development of a stable landform comprised of densified and/or dewatered tailings that are no longer capable of generating a mudflow that can migrate off site during a hypothetical dam breach scenario.
- to stabilize the tailings mass so that it does not represent a potential mudrush risk to the underground mining operations.

Complete removal of the surface pond is the simplest and most significant mitigation measure, as the absence of a water pond eliminates many of the credible failure modes that would typically be considered for a dam breach assessment. Surface shaping of the impoundment, along with appropriate storm water routing can also be implemented to preclude the potential for pond development in the future. Once the release of a surface water pond is precluded, the only remaining failure mechanism would be liquefaction and subsequent flow of saturated tailings solids. If these saturated tailings materials can be dewatered and densified to the extent required to render them non-flowable, then the tailings impoundment can be considered to be a stable landform with no credible failure modes.

Ground improvement technologies can be used to improve soil properties, including increasing the density, reducing the water content, increasing the shear strength, reducing the permeability, or causing changes that may alter the way the soil deforms. Such modifications can be used to improve the soil bearing capacity or slope stability, or to reduce the liquefaction potential. Ground improvement techniques can be used to reduce the moisture content, increase the in situ density and thus reduce the fluidity of remoulded tailings solids to such an extent that they would no longer readily flow in the event of any catastrophic breach of the impoundment. A review of available ground improvement technologies was completed and two technologies were selected for field trial evaluation as follows:

- Pumping wells were supplemented with vertical wick drains to enhance the dewatering of the Tailings Sands. The trial pumping tests indicated that dewatering using multiple pumping wells is viable.
- Compressive loading combined with vertical wick drains to increase density, reduce moisture content and increase the yield stress of the Tailings Slimes so that remoulded materials will be non-flowable. An 11 m high conical surcharge load was constructed in stages above the Tailings Slimes over 2 months. Approximately 2.2 m of consolidation settlement was observed at the centre of the Test Pad. Follow-up site investigations showed an increase in the tailings density, a decrease in the in situ moisture content, and corresponding increase in yield stress due to the consolidation that resulted from the trial loading.

The combination of pumped dewatering wells for the more permeable sandy tailings zones, and surcharge loading to promote consolidation dewatering and densification has been shown to be a viable ground improvement strategy for the Historic TSF adjacent to the New Afton mine.

These ground improvement methods can be enhanced by including a suitable spacing of vertical wick drains to enhance fluid migration during the dewatering processes. Explosive compaction is also considered to be a potentially viable ground improvement technology and would be implemented as a surgically targeted contingency measure. The stabilized tailings mass will no longer be susceptible to uncontrolled rapid flow out of a hypothetical dam breach and would not represent a potential mudrush hazard to the underground mining operations. These stabilization measures are considered to represent BAT's and will promote drainage, consolidation, and insitu soil stabilization of the Historic TSF. These technologies may also represent practical options to consider during the design of new tailings impoundments, and for the expansion and closure of pre-existing TSF's.

ACKNOWLEDGEMENTS

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Characterizing and Stabilizing a Historical Tailings Facility: The Rheology to Soil Mechanics Continuum

Amy Adams & Daniel Friedman
Knight Piésold Ltd, North Bay, ON, Canada

Scott Davidson
New Gold Inc., Kamloops, BC, Canada

ABSTRACT: Historical tailings facilities often contains materials that have a consistency that ranges from fluid to solid, depending on a number of factors such as particle size, depth, drainage, and depositional history. Historical impoundments may contain saturated semi-fluid materials at depth, long after tailings deposition has ceased and after surface reclamation has been completed.

This paper presents a case study of the investigations and testing relating to the design and progressive implementation of remedial stabilization measures for the historical tailings facility at the New Afton Mine located in British Columbia, Canada. It was necessary to evaluate both the geotechnical conditions (soil characteristics) of the tailings mass, as well as the potential rheological behaviour (fluid flow characteristics) of loose saturated zones that could be susceptible to liquefaction and migration into the cave zone or underground workings. Therefore, this study relies on integration of the principles of advanced soil mechanics in combination with fluid mechanics and rheology, particularly in relation to slurry viscosity and flow behaviour of contractive potentially liquefiable tailings materials.

A rheological model was developed to characterize the yield stress and flowability of the historical tailings deposit. In-situ and laboratory testing was completed to understand the variability of the tailings in the facility. Simple index properties including moisture content and clay-sized particle fraction were used to characterize the tailings rheology. A field-scale trial program was implemented to demonstrate that the tailings could be quickly and effectively stabilized by densification and dewatering using wick drains, consolidation loading, and dewatering wells.

1 INTRODUCTION

Tailings are variable in nature and the disposal techniques vary significantly depending on the mining methods/rates, the ore characteristics, the site conditions and also the environmental, social and economic considerations that prevailed during the permitting, construction and operational stages of the mine. Tailings impoundments can continue to represent an ongoing liability, long after mine operations cease and the surface facilities have been closed and reclaimed. The surfaces of many tailings impoundments have been reclaimed by shaping, capping, and revegetation, but there are fewer examples where the tailings pile can be shown to be suitably stabilized with no potentially flowable materials.

Many historical tailings impoundments were progressively developed using relatively simple hydraulic slurry placement within a flooded or partially flooded facility. These 'conventional' hydraulically emplaced tailings deposits form a relatively loose and somewhat segregated mass of interlayered sandy and silty materials, with the finest grained silty and clay-sized (slimes) particles typically deposited farthest from the discharge points. These hydraulically emplaced tailings deposits are typically comprised of materials that are contractive and prone to liquefaction, particularly in the upper 20 to 40 meters of the deposit where they are less consolidated than at greater depths.

The geotechnical characteristics of a tailings impoundment can become a critical factor in the success of a project when underground mining activities extend laterally and mine-induced deformations result in cracking or surface subsidence features that may interact with the tailings impoundment. Water and/or fluidized tailings materials can represent significant risks to an underground mine development due to the potential for a catastrophic mudrush. A mudrush event occurred at the Mufulira mine in 1970, in which 89 underground miners lost their lives when ponded water and liquefied tailings created a highly fluid slurry that rapidly flowed into the underground workings through mine-induced cracks. Post disaster forensic investigations led to the development of remedial drainage measures within the remaining surface tailings pile in order to stabilize the materials and allow safe underground mining operations to be resumed (Sandy et al, 1976). Mufulira was a relatively shallow mine compared to the New Afton block cave, but serves as a relevant case history nonetheless. The 2015 Samarco tailings failure (Morgenstern et al. 2016) provides a more recent example of the liquefaction and subsequent mudflow in loose, saturated tailings. A catastrophic mudflow rapidly migrated downstream, inundating a village and causing 19 deaths.

This paper presents the case study of the investigations and testing relating to the design and progressive implementation of remedial stabilization measures for the historical tailings facility at the New Afton Mine. The methods that have been investigated and are proposed for full scale implementation are considered best available technologies to develop a geotechnically stable landform. The opportunities to use best available technologies to stabilize tailings are discussed in our companion paper “Best Available Technologies to Stabilize a Historical Tailings Impoundment” (Adams et al, 2017a). It was necessary to evaluate both the geotechnical conditions (soil characteristics) of the tailings mass, as well as the potential rheological behaviour (fluid flow characteristics) of any loose saturated zones that could be susceptible to liquefaction and migration at surface in the case of a dam breach and at depth into the cave zone or underground workings. This study relies on integration of the principles of soil mechanics along with fluid mechanics and rheology to describe contractive liquefiable tailings materials that may become fluid in nature and flow.

2 PROJECT OVERVIEW

The New Afton Mine is a copper gold mine located approximately 10 km west of Kamloops in British Columbia, Canada (Figure 1). The New Afton Mine occupies the site of the former Afton Mine that was historically developed from 1978 to 1997 using open pit mining methods. Conventional flotation processes produced a tailings slurry that was hydraulically discharged at approximately 35% solids content via multiple spigots into a nearby facility. The historical mine site was closed and the surface facilities were partially reclaimed. In 2005 New Gold Inc. acquired a portion of the overall property and continues to develop an underground block caving mine operation to exploit deeper mineralized zones. Slurry tailings from the current mill are disposed in the active New Afton TSF, as illustrated on Figure 1.

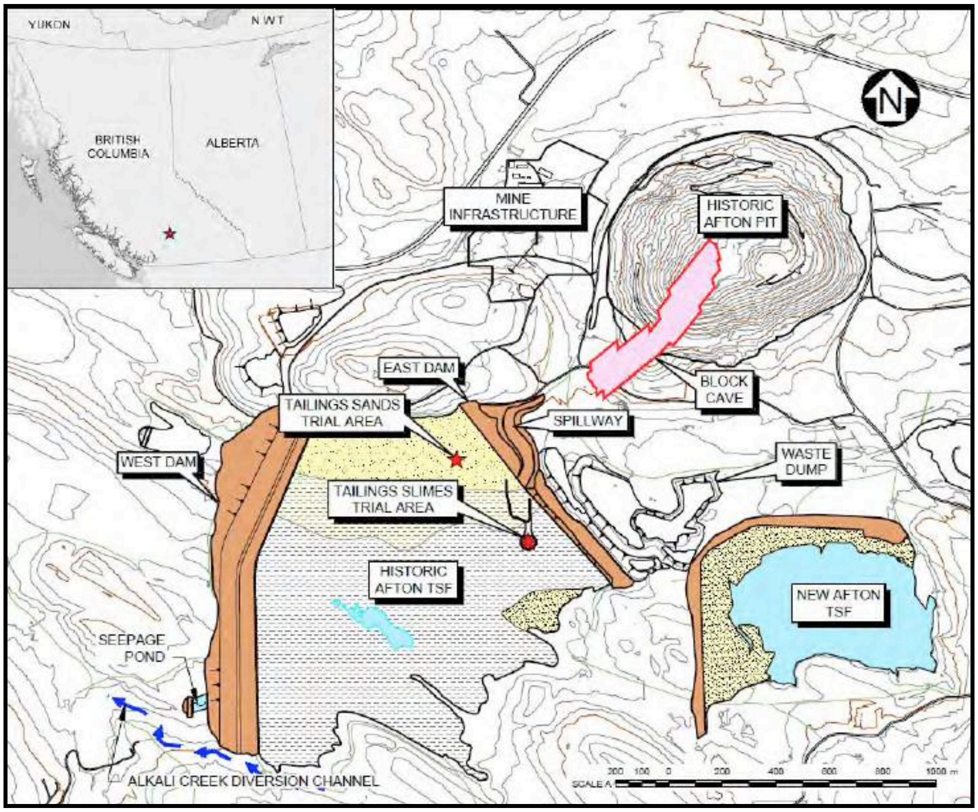

Figure 1. Project Location and Site Layout

Water is routed around the Historic Afton TSF (Historic TSF) via a diversion channel. The small water pond, shown on Figure 1, has been removed and the Historic TSF is in a negative water balance condition; it is expected to stay dry in the long-term. Thus a key objective for removal and elimination of any free water pond on the tailings surface has been readily accomplished.

The New Afton underground mining method commenced in 2012 and will result in surface cracking and subsidence that is conservatively postulated to potentially interact with the overlying Historic TSF as the mine development becomes progressively larger and deeper (Figure 2). Worker safety is a primary and fundamental requirement for ongoing mining operations. The proximity of the Historic TSF to the underground mine has been recognized as a potential risk factor relating to a potential mudrush hazard, unless appropriate tailings stabilization techniques are implemented as mitigation measures.

Given that surface ponding has been eliminated, the presence of free water as carrier fluid is no longer a potential factor in a mudflow risk assessment. Thus, the residual mudflow risk is only related to the flowability of any portion of the tailings solids contained within the Historic TSF. Densification, dewatering, and reduction of the potential for liquefaction were thus identified as critical objectives to stabilize the saturated tailings. Detailed in-situ geotechnical investigations were completed using seismic cone penetration testing, specialized sampling methods and instrumentation arrays. Hydrogeological testing incorporated pump testing and detailed pore pressure monitoring. Laboratory testing methods included conventional soil mechanics test methods, as well as specialized rheological test work to characterize the full continuum of tailings properties within the facility – which ranged from loose, saturated flowable materials to dense, non-flowable soils.

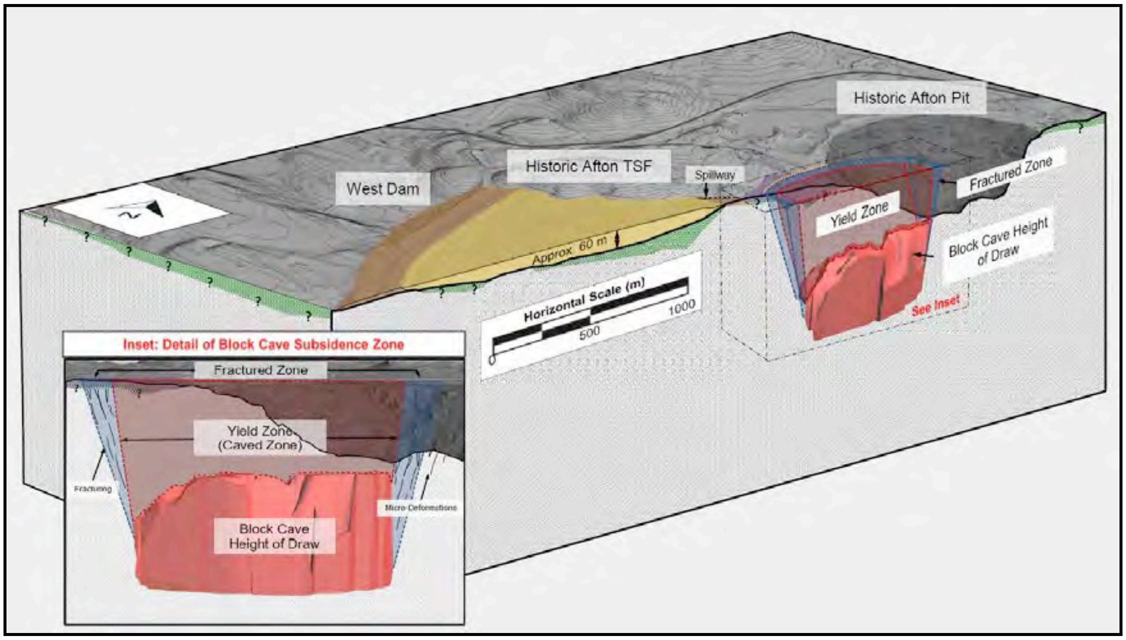

Figure 2. Potential Interaction between Underground Mine and Historical Afton TSF

These investigations were used to evaluate ground improvement technologies, including the use of dewatering wells, wick drains, and compressive loading, to densify and dewater the historical tailings over the full height of the tailings column. The use of ground improvement technologies are discussed in our companion paper entitled "Best Available Technologies to Stabilize a Historical Tailings Impoundment" (Adams et al, 2017a). A field scale trial was undertaken to evaluate the effectiveness of the selected ground improvement technologies with the results indicating successful tailings stabilization at surface and at depth. The following sections describe the tailings characterization and field program developed to evaluate the selected ground improvement technologies.

3 TAILINGS CHARACTERIZATION

The Historic TSF contains approximately 37 million tonnes of saturated tailings solids which were naturally segregated during multiple spigot tailings deposition at approximately 35% solids. Sandy Tailings beaches formed along the north side of the impoundment and finer grained silty Tailings Slimes tailings deposits formed towards the south as shown on Figure 1 and Figure 3. High in-situ moisture contents (5 to 45% for the Tailings Sands and 30 to 90% for the Tailings Slimes) suggest that some for the tailings could behave more like a fluid than a soil when disturbed/liquefied.

3.1 *Site Investigations*

The physical state of the tailings within the Historical TSF at the New Afton site prior to the field trials was characterized by three phased site investigations carried out between April and October, 2014. The site investigation locations are shown on Figure 3 and included:

- 21 sonic drillholes
- 45 land-based Seismic Cone Penetration Test (SCPT) and 7 amphibious CPT probings
- 10 test pits
- Installation of 1 observation well, 1 pumping well and 3 monitoring wells, including hydraulic response testing in the sandy and silty tailings at 2 locations
- Installation of 25 Vibrating Wire Piezometers at 8 locations

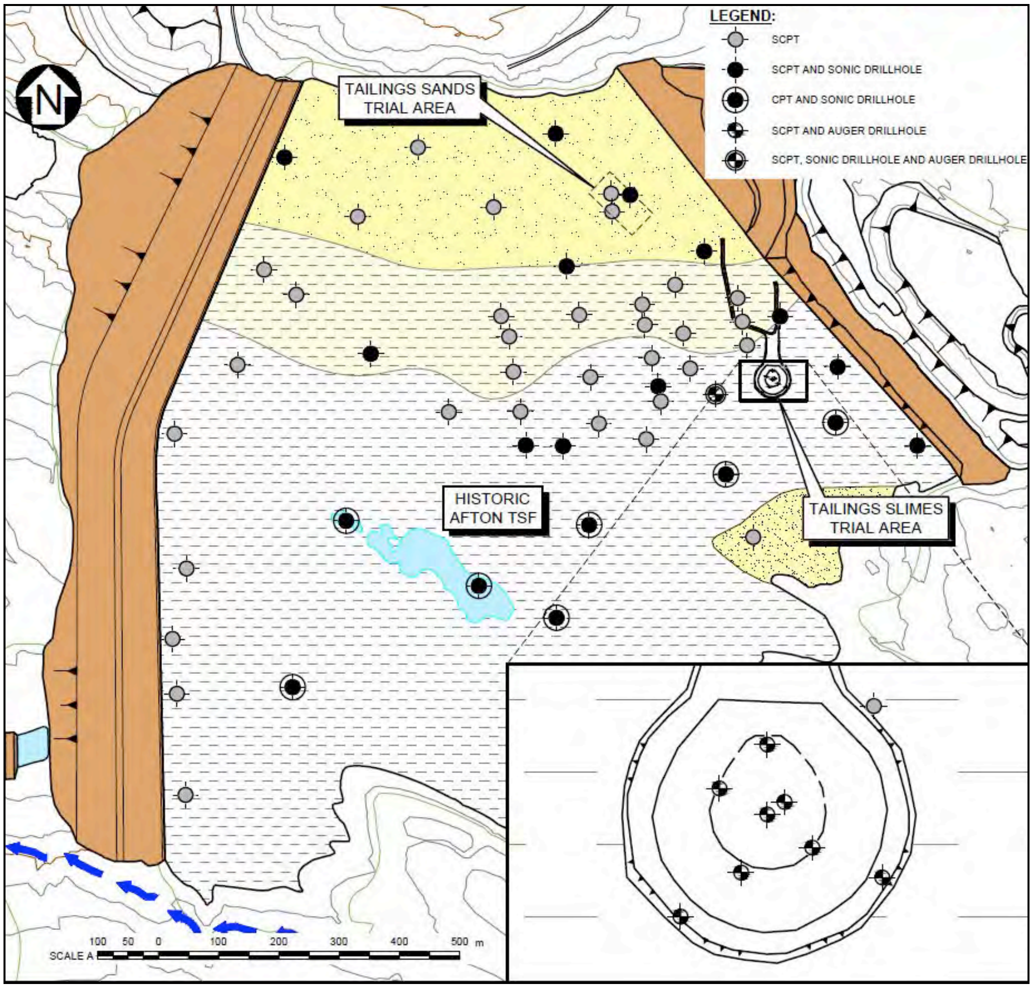

Figure 3. Site Investigation Locations

A total of 99 tailings samples (29 undisturbed) and 8 overburden samples were collected during the Phase 1 through 3 site investigations. The collection of undisturbed samples was challenging due to the high moisture content and fluid like nature of some of the tailings materials. Moderate success was achieved using a Parky Piston Sampler with passive suction.

3.2 *Geotechnical Testing*

Laboratory testing was conducted to characterize the moisture content, particle size distribution (including hydrometer to measure the clay-sized fraction), plasticity, and density of the tailings. Slurry consolidometer testing was completed on select samples to evaluate the compressibility and permeability, and X-ray diffraction was completed to evaluate the mineralogy.

The tailings in the Historical TSF are segregated with Tailings Sands (silty sand, less than 70% fines) in the north and silty Tailings Slimes (low to high plasticity clays) in the southern portion of the facility as illustrated on Figure 1. High in-situ moisture contents (5 to 45% for the Tailings Sands and 30 to 90% for the Tailings Slimes) suggest that some of the tailings could behave more like a fluid than a soil when disturbed/liquefied. The tailings contain up to 40% clay minerals including clinochlore and illite / muscovite.

3.3 *Rheological Testing*

Laboratory rheological testing was completed on composite mixtures of tailings samples to visually and quantitatively measure the variation in yield stress with moisture content and with clay sized fraction. The following rheological testing was conducted:

- **Vane Yield Test:** This consists of applying torque to a vane inserted into the sample using a 2-inch vertical tube viscometer. The peak torque is recorded as the yield stress required to mobilize the sample.
- **Boger Slump Test:** This 3-inch cylinder slump test is a fast and simple method that can be used to estimate the yield stress of thickened slurries and pastes. The measured slump is related to the yield stress using analytical methods.
- **Crack Simulation Test:** This qualitative test was developed specifically for this program to illustrate the potential for tailings samples at various water contents and yield stresses to flow into a crack, such as those that could hypothetically develop below the Historical TSF during future mine operations. This test apparatus consisted of a flat surface with an adjustable gap (crack) that was slowly opened.

The results of the rheological testing are illustrated to represent the soil to slurry continuum on Figure 4. As the yield stress increases, the tailings transition from a slurry to a paste and then a soil. The photographs show the results of Boger Slump testing and Crack Simulation testing to illustrate the behaviour of the tailings as the moisture content increases. Four ‘flowability zones’ were developed based on observations and measurements made during the rheological testing.

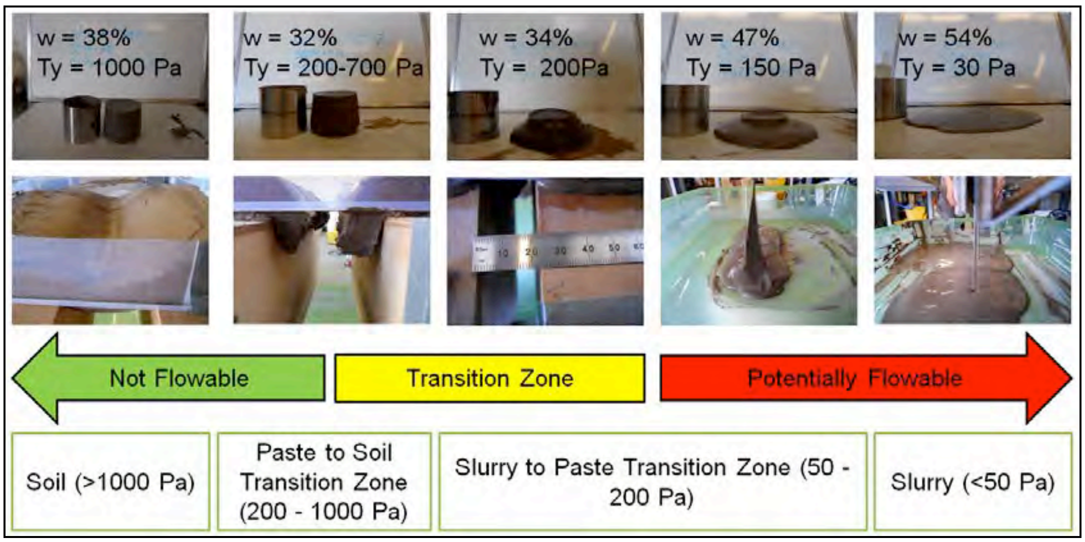

Figure 4. Tailings Rheology: Soil to Slurry Continuum

Vane yield stress tests were carried out on the fluidized samples to develop a rheological model (Figure 5). This model was used to estimate the tailings yield stress knowing the approximate clay-sized fraction and the in-situ moisture content. These parameters were obtained through drilling, the collection of undisturbed samples (passive suction piston sampling), and laboratory index testing.

A “hockey stick” relationship between yield stress and moisture content is observed in Figure 5. The blade (bottom) of the hockey stick represents the slurry or paste-like behaviour where the tailings flow and are best described using rheological parameters. Large changes in moisture content are required to cause small changes in yield stress. The handle (upper part) of the hockey stick represents tailings materials demonstrating soil-like behaviour, with small changes in moisture content corresponding to large changes in yield stress. The heel (hinge) of the hockey stick represents the transition between a fluid and a semi-solid. The transition is abrupt for the sandy tailings (see 5% clay, red line) and gentler for the Tailings Slimes (see 40% clay, orange line).

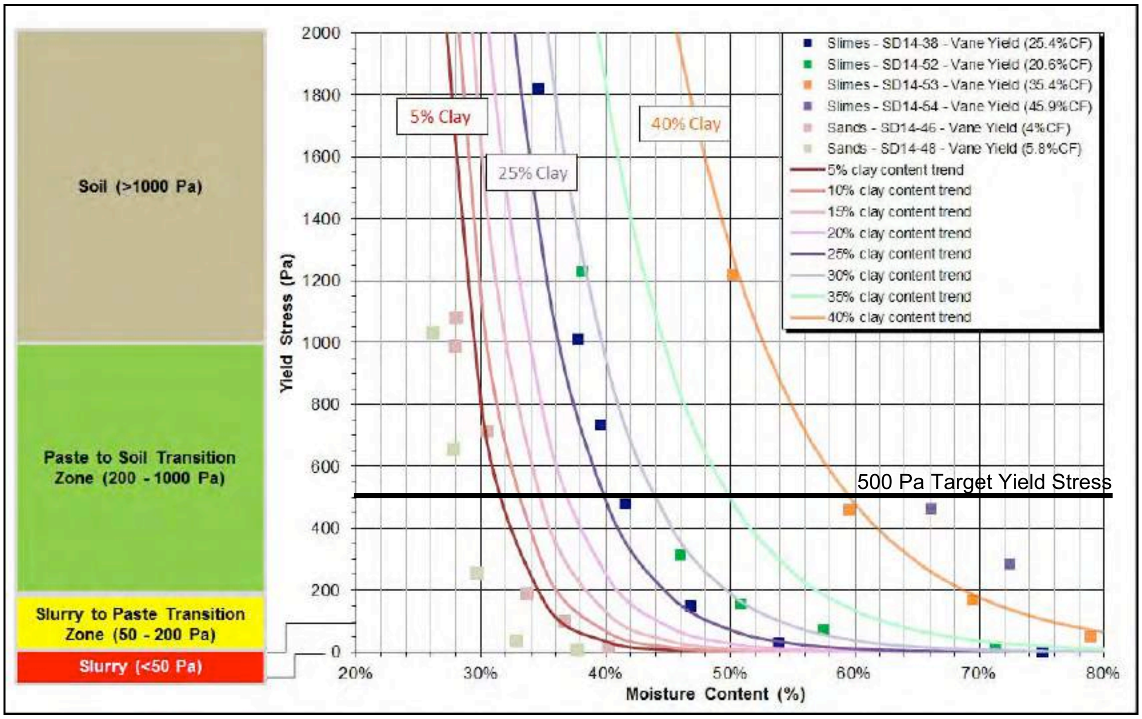

Figure 5. Tailings Rheology Measurements and Model

3.4 *Estimating Flow Through a Crack*

A simplistic mathematical model was developed to describe the potential flow of tailings through a hypothetical crack intersecting the TSF (Figure 6) and to support the development of a target yield stress. The model assumes an idealized singular crack is formed by two parallel plates separated by a gap, t. The width, w, and length, L, of the crack are much greater than the gap, t, making sidewall and inlet effects negligible. The crack slopes downwards at an angle θ from the horizontal, giving a vertical intrusion depth equal to Lsinθ. The flow of an assumed infinite volume of homogeneous tailings with yield stress τ_y through the crack is modelled as a force balance between the driving forces of the tailings stored in the impoundment applying a pressure P, and the shear resisting forces of the tailings along the upper and lower crack walls. The tailings will always be in contact with the lower crack wall under the force of gravity, with the possible exception of a vertical crack, but may not be in contact with the upper wall depending on the angle of repose and flow rate.

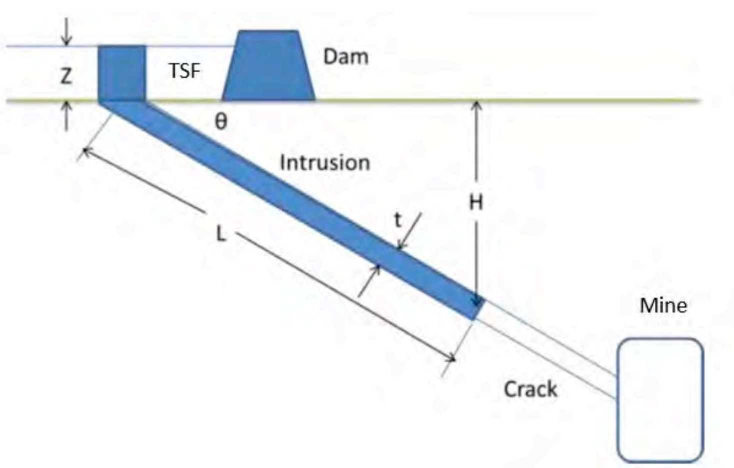

Figure 6. Tailings Flow Through Hypothetical Crack

If there is no surface pressure on the crack, the paste can not move through the crack if the gap is too small,

$$t \leq \frac{2t_y}{\rho g sin\theta}$$

If the crack is pressurized, with the static pressure of the tailings above the crack idealized as a fluid pressure, and the tailings within the crack and the impoundment are assumed to be homogeneous (i.e. have the same yield stress), the gap size at which flow will initiate is described as follows:

$$t = \frac{2t_y}{\rho g sin\theta}\left\{\frac{1+\frac{H}{Z}sin^2\theta}{1+\frac{H}{Z}}\right\}$$

For unpressurized tailings with density ranging from 1.7 to 2.0 t/m^3, the crack width required to initiate flow varies from 2 to 2.5 cm at 200 Pa to 10.5 to 12.5 cm at 1000 Pa if a conservative 70 degree angle is assumed between the underground mine and the eastern extent of the Historic TSF (Figure 2). Under these conditions, tailings with a yield stress of approximately 500 Pa will begin to flow towards the underground workings once the crack aperture exceeds approximately 5 to 6 cm angled at 70 degrees or steeper. Variations in the surface pressure were found to have limited effect (one to two cm) on the crack width required to initiate flow.

Any cracks that do develop are expected to vary in aperture, asperity, and tortuosity. Based on site observations and bedrock crack prediction modelling, a 5 to 6 cm wide crack width was judged to be a conservative upper bound estimate. A target yield stress of 500 Pa, representative of the paste to soil transition, was thus selected for the design of stabilization field trials.

3.5 *Summary of Geotechnical and Rheological Characteristics*

The tailings behaviour and transition from slurry to paste and then to soil is strongly influenced by the in-situ moisture content and clay-sized fraction. These properties can be measured in the laboratory on undisturbed samples obtained through a site investigation program. The test work shows a consistent increase in the yield stress and reduction in the tailings flowability with decreasing moisture content of the tailings. The sandy tailings experience a sharp transition and rapidly increasing yield stresses once unsaturated conditions develop. The Tailings Slimes experience a more gradual increase in yield stress with decreasing moisture content. This behaviour is observed because coarser tailings rely on particle to particle contact and negative pore pressures (or the "sand castle" effect) to develop strength, while finer tailings with clay minerals are influenced by electro-chemical forces between the fine particles. The sandy tailings are therefore more sensitive to increasing moisture content beyond the point where the shear stresses are transferred to the carrier fluid from the coarse particles, similar to how a sand castle will quickly collapse once enough water is added to the mixture. A lower fines content also results in reduced carrier fluid density and yield stress, which compounds this effect.

A target yield stress of 500 Pa was selected for the design of stabilization field trials based on crack flow modelling. This yield stress is judged to be within the paste to soil transition zone where small changes in moisture content cause large changes in yield stress.

4 FIELD TRIAL PROGRAM

4.1 *Trial Program Description*

Reducing or eliminating the risk of liquefied tailings flowing from a hypothetical TSF breach or a mudrush of liquefied tailings flowing from the Historical TSF into the underground mine workings requires removal of the carrier fluid (water). Given that the surface pond has already been removed, the objective is to dewater the historical tailings in order to increase the yield stress and reduce the flowability of the heterogeneous deposit.

Two ground improvement technologies were selected for a field trial program (Adams et al, 2017b):

- Dewatering with pumping wells was selected to reduce the moisture content and reduce the piezometric surface in the coarser tailings (Tailings Sands).
- Compressive loading combined with wick drains was selected to consolidate, densify, and reduce the moisture content of the finer tailings (Tailings Slimes).

The objective of the trial program was to evaluate the effectiveness of the selected stabilization methods and to collect field scale data to support the full scale stabilization program. The trial program incorporated confirmatory site investigations (Phase 4) to collect additional data for analysis of the trial program results. The following sections briefly describe the field trials and results.

4.1.1 *Tailings Sands*

An area approximately 100 m long and 50 m wide was selected for study in the Tailings Sands. Two 60 m deep pumping wells were screened for the full interval within the Tailings Sands. Three existing wells ranging in depth from 27 to 52 m were used as observation wells. Wick drains were installed in a 5 m triangular pattern to 33 m depth. Ten (10) drive point VWP's were installed along three cross sections to monitor pore pressures and drawdown. The wick drains and VWP's were installed by Hayward Baker, a specialist ground improvement contractor. Survey monuments were installed on the surface of the tailings and monitored multiple times daily.

The water level fluctuation was measured during 6 hour, 24 hour, and 7-day pumping tests with pumping rates varying from 0.3 L/s (4.5 gpm) to 6.6 L/s (100 gpm). The pumping tests demonstrated that conventional pumping wells screened through the Tailings Sands can effectively remove water from the aquifer. Comparison of pump test results completed prior to and following wick drain installation demonstrated that wick drains increased the aquifer storativity (i.e. the total amount of water available for pumping) by approximately one order of magnitude. A very small vertical displacement of the tailings surface was observed in the survey results during the long term (7 day) pumping test that likely indicates some volume reduction due to consolidation.

The trial confirmed that pumping is a viable method to dewater the Tailings Sands, thereby increasing the yield stress and reducing the flowability. It is expected that the degree of saturation can be sufficiently reduced in the Tailings Sands to achieve the target yield strength of 500 Pa needed to mitigate the risk of liquefaction of the tailings. This in turn mitigates the risk of the tailings flowing from the TSF during a hypothetical dam breach scenario as well as eliminating the mudrush risk for the underground mine. Wick drains may also be installed in the portion of the stabilization zone where the tailings sand is interlayered with finer silts and clays (tailings slimes) to increase the vertical hydraulic conductivity of the aquifer and enhance dewatering. This will promote drainage of groundwater perched above low permeability horizons and reduce the potential for groundwater compartmentalization.

4.1.2 *Tailings Slimes*

A 50 m diameter conical consolidation fill load with access ramp (Test Pad, Figure 7) was constructed over the approximately 25 m thick Tailings Slimes using staged construction over a two month period. The fill was underlain with Mirafi HP570 geotextile placed on the tailings surface to provide safe access for wick drain installation. Vertical wick drains were installed in a 2 m triangular grid pattern to depth 25 ft to help increase the rate of tailings densification, moisture reduction, and strength gain. VWP's were installed at five locations, including two locations at the centre and three locations along the circumference of the Test Pad, to monitor pore pressures during construction. Survey monuments were installed on and around the test pad to monitor settlements. The Test Pad was developed using staged construction in 1 to 2 m lifts over a two month period. The ultimate as-constructed thickness of the Test Pad was approximately 10 to 11 m after accounting for settlement during construction.

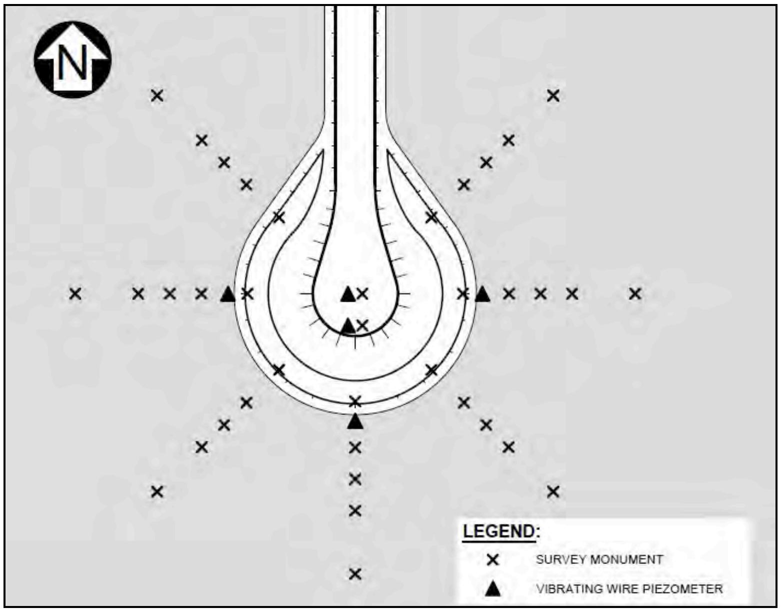

Figure 7. Schematic plan view of Test Pad Constructed over Tailings Slimes

The measured settlement and pore pressures below the test pad are provided in Figure 8. The tailings compressed approximately 2.2 m vertically at the centre of the test pad and between 0.25 and 0.5 m vertically at the edge of the test pad as a result of the applied 11 m of fill loading. The pore water pressures increased during fill placement and dissipated rapidly suggesting that consolidation of the tailings was occurring with minimal horizontal displacement (Figure 7). No signs of major displacements, either vertical or lateral, or slope instabilities were observed. All VWP's remained intact and functional at the end of the construction program, and were still functioning one year later.

The Phase 4 site investigation program was conducted following completion of primary consolidation to evaluate the degree of soil improvement that resulted due to placement of the test fill. Eight Hollow Stem Auger drillholes and nine SCPT's were completed, and undisturbed (Shelby Tube) samples were collected. Index testing was completed on select specimens from the Shelby tube samples to determine the moisture content, grain size, plasticity, and specific gravity at various depth intervals.

Auger drilling methods (without drilling fluids) were used to avoid influencing the moisture content of the tailings materials during drilling and sampling and to provide confidence that the moisture content values from the Shelby tube samples were representative of in-situ conditions. A mechanically actuated stationary piston sampler was used to maximize sample recovery and minimize disturbance in the difficult to sample Tailings Slimes. The use of a mechanically actuated sampler as opposed to one which is hydraulically actuated also eliminated the potential to influence in-situ moisture content as no water is added down the hole to actuate the sampler.

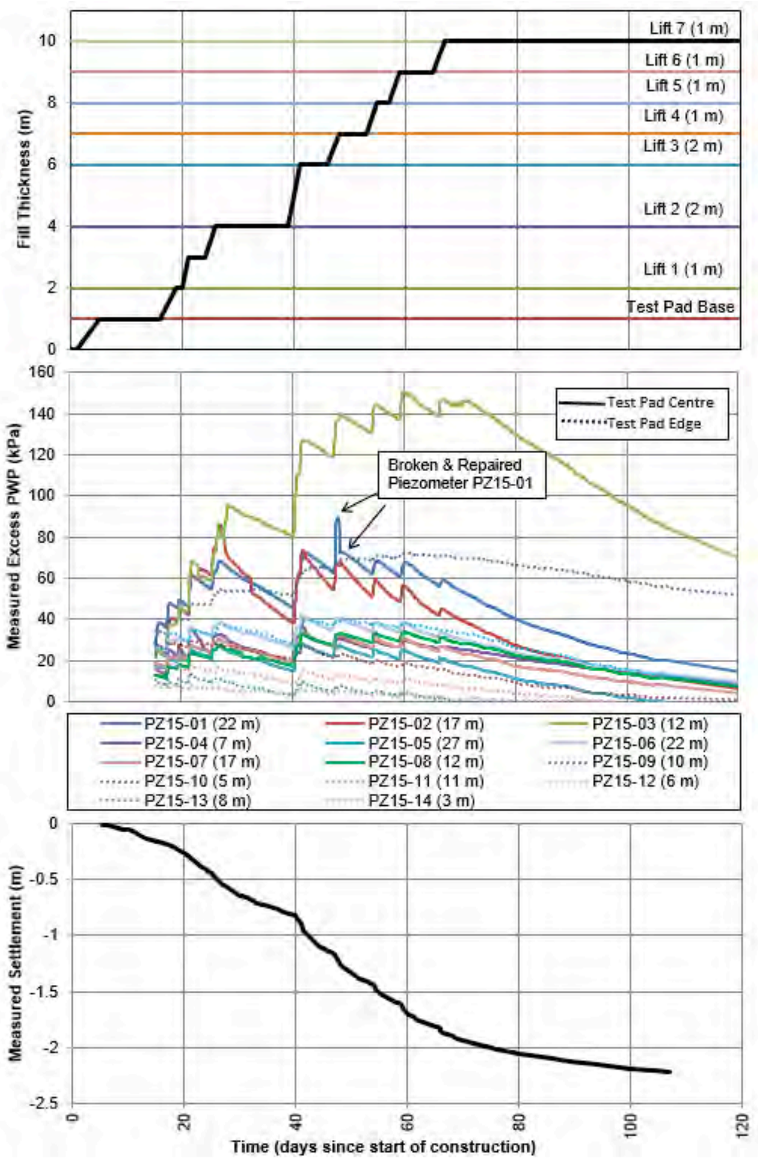

Figure 8. Construction Schedule, Measured Excess Pore Pressures and Measured Vertical Settlement at the Centre of the Test Pad

One CPT and one auger drillhole were completed at the same location as a previous CPT and Sonic drillhole to confirm the quality of the data collected using both sonic and auger drilling methods. The results showed comparable moisture content and particle size distributions suggesting that both auger and sonic drilling methods provided similar sample quality.

Construction of the Test Pad resulted in an increase in the CPT tip resistance (q_t), a decrease in the moisture content as the Tailings Slimes were densified, and an increase in the estimated yield stress (Figure 9) for the majority of the underlying tailings slimes. The yield stress was estimated using the rheology model shown in Figure 5. Based on the observed results, more consolidation time and/or higher loads will be required to achieve the target yield stress of 500 Pa throughout the tailings column, although the target yield stress was achieved in the trial for the majority of the tailings below a depth of 14 m and overlying any potential cracks.

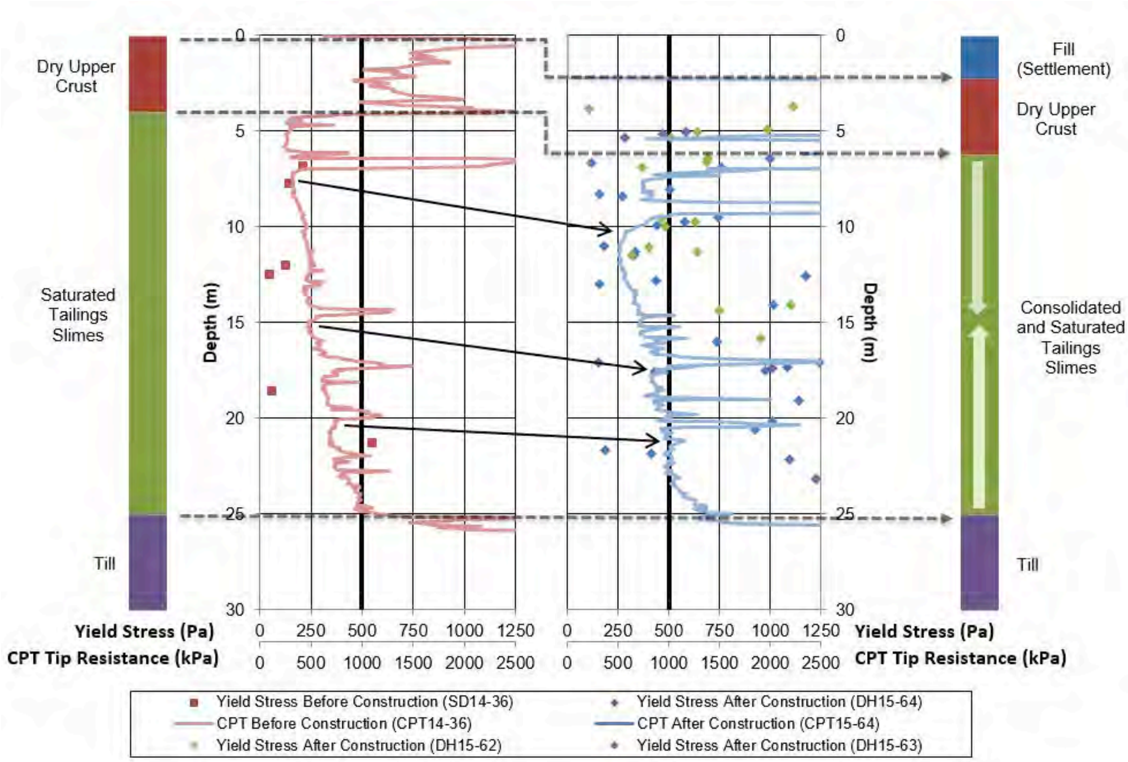

Figure 9. CPT Tip Resistance and Yield Stress - Before and After Consolidation Loading

5 SUMMARY AND CONCLUSIONS

The New Afton Mine occupies the site of the former Afton Mine. The Historic TSF is currently inactive and was last operated by the Afton Mine using hydraulic emplacement of slurry tailings from 1978 to 1997. It contains approximately 37 million tonnes of saturated tailings solids, which were naturally segregated during multiple spigot tailings deposition at approximately 35% solids. The tailings deposition formed sandy tailings beaches along the north side of the impoundment and finer grained slimes tailings deposits along the south. The Historical TSF represents a potential risk to the nearby underground mine development, which will cause surface cracking and subsidence that could potentially interact with the Historical TSF. A catastrophic mudrush could develop if fluid, or semi-fluid materials, contained within the Historical TSF are able to migrate via connected pathways into the underground workings.

Conventional soil mechanics analyses were coupled with rheological assessments to define and evaluate appropriate ground improvement technologies to transform the tailings to a stable landform and to mitigate the risk of tailings mudrush through any hypothetical cracks that could interconnect into the progressively expanding underground block cave mine. The tailings were characterized through geotechnical and rheological testing. A site-specific rheological model was developed based on the results of vane yield rheological tests completed at different moisture contents and on samples with varying clay-sized particle fractions. This rheological model will be used to estimate the tailings yield stress using simple index properties that can quickly and easily be measured via a direct sampling drilling program.

Field scale trial programs were used to demonstrate that the following two ground stabilization methods could be utilized to dewater, densify, and increase the yield stress of the historical tailings:

- Sandy Tailings: Dewatering via pumping wells to reduce moisture content, and increase the yield stress of remoulded tailings. Pumping wells can be supplemented with vertical wick drains to enhance the dewatering of the Tailings Sands.

- Tailings Slimes: Consolidation loading with wick drains to increase density, reduce moisture content and increase the yield stress so that remoulded materials will be less susceptible to liquefaction and thus non-flowable.

These stabilization measures are considered to represent best available technologies and will promote drainage, consolidation, and in-situ soil stabilization of the Historical TSF. These technologies may also represent practical options to consider during the design of new tailings impoundments, and for the expansion and closure of pre-existing TSF's.

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Multi-Plane Slope Stability Analysis of Tailings Dams

M.D. Fredlund & Z. Ivkovic
SoilVision Systems Ltd., Saskatoon, SK, Canada

H. Lu
SoilVision Systems Ltd., Edmonton, AB, Canada

G. Gitirana
Universidade Federal de Goias, Goiana, Brazil

ABSTRACT: A number of recent tailings dams failures have brought the slope stability analysis of tailings dams to the forefront. The geotechnical practice has focused on the stability of individual structures through the use of 2D limit equilibrium analysis. Such analysis have typically been applied to unique profiles of geotechnically designed structures. However, such analysis has not typically been applied over large areas. It is often too costly to analyze multiple cross-sections in a longer tailings retention structure considering varying subsurface stratigraphy. The recent failures show us that increased rigor of analysis is one aspect that requires further consideration in the geotechnical analysis of such structures.

This paper presents a methodology of combining the exacting calculations of the 2D and 3D limit equilibrium method with the long structures for a tailings dam analysis. Such analysis has been termed a "Multi-plane analysis" (MPA) approach. The new method could be applied to the analysis of long engineered structures such as tailings dams and allows the geotechnical engineer access to analysis information of greater detail. The method can be based on 2D or 3D calculations and also allows the analysis of corners, abutments and other geotechnical design features that are not capable of being analyzed in 2D planes.

1 INTRODUCTION

Slope stability of tailings dams has received increased attention in recent years due to recent tailings dam failures such as Mt. Polley, Canada and Fundao, Brazil. These failures have resulted in increased review assessments of the stability of existing tailings dams worldwide by geotechnical consultants. The difficulty with such stability assessments is related to:

i) Is the potential failure mechanism properly identified?
ii) Is the correct constitutive model used to represent the relevant materials
iii) Is there a change in the computed factor of safety spatially?
iv) Is the difference between a 2D and a 3D factor of safety significant?

Tailings dam analysis has typically focused on a representative cross-section of the tailings dam. It can be noted, however, that the foundational geo-strata may vary under different sections of the tailings dam. Analysis of multiple 2D cross-sections can be time-consuming and costly and is therefore avoided or limited to a select few cross-sections. This issue is compounded by the fact that many tailings dams may be quite long.

There is also the issue of how to adequately handle the 3D intersections of different sections of tailings dams. Are the 3D aspects adequately analyzed?

Factors of safety of 1.3 or 1.5 are typically utilized for design purposes. The process typically involves a geotechnical engineer selecting a critical 2D slice to analyze. The geotechnical engineer designs the slope to a given factor of safety based on the 2D analysis.

There are two difficulties with this approach.

Firstly, the engineer must make an assumption regarding the difference between the 2D and the 3D factor of safety. Secondly, the engineer is assuming that the slice they have selected at the site is the critical slice. This traditionally approach has been accepted because i) the view is that the 2D analysis Factor of Safety (FOS) is conservative, that is, it is lower than a more accurate 3D analysis FOS. The engineers "eyeball" judgement as far as WHERE the critical slip may occur has been relegated to the department of professional judgement.

This paper demonstrates how a slope stability analysis can be performed in an improved manner by taking into account both the 3D aspects of the analysis as well as determining the potential location of the critical slip surface.

1.1 *Extruding 2D to 3D – Slip Shape Differences*

It is of importance to understand the difference between the slip shape analyzed in a 2D analysis as opposed to a 3D analysis. In a 2D analysis the slip shape is ultimately extended infinitely in the 3rd dimension and shear on the end surfaces is not considered. An example 2D analysis is presented in Figure 1. The equivalent 3D analysis is shown in Figure 2 where there is no shear strength applied to the vertical end surfaces.

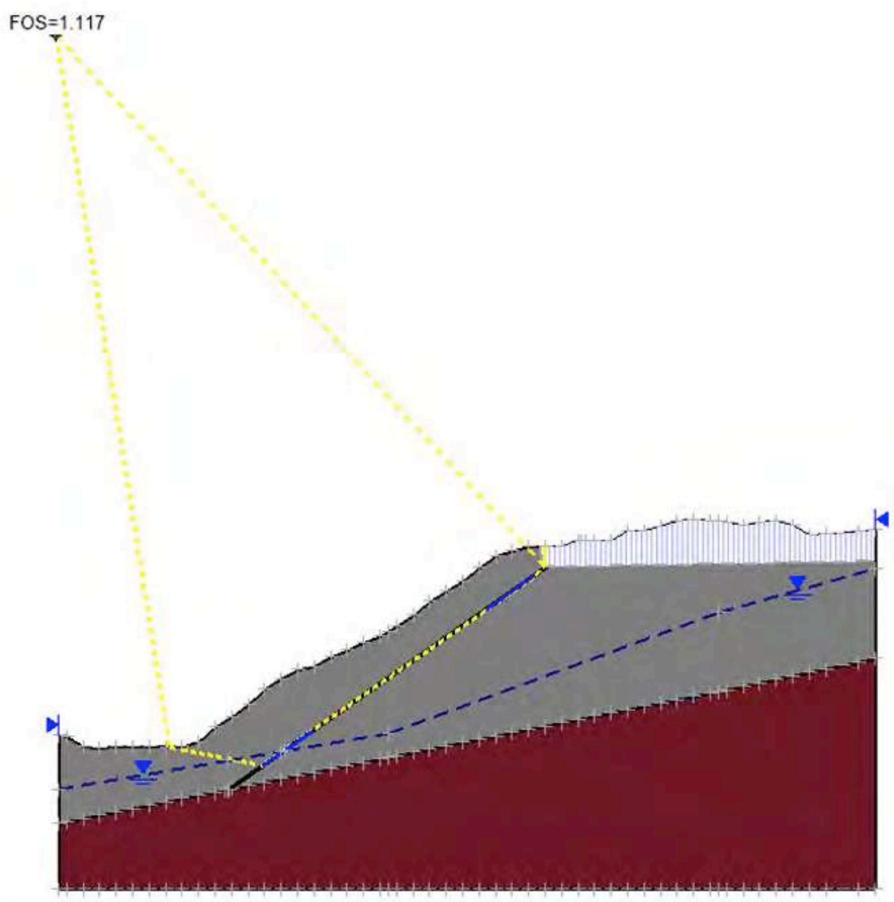

Figure 1 Example 2D stability analysis

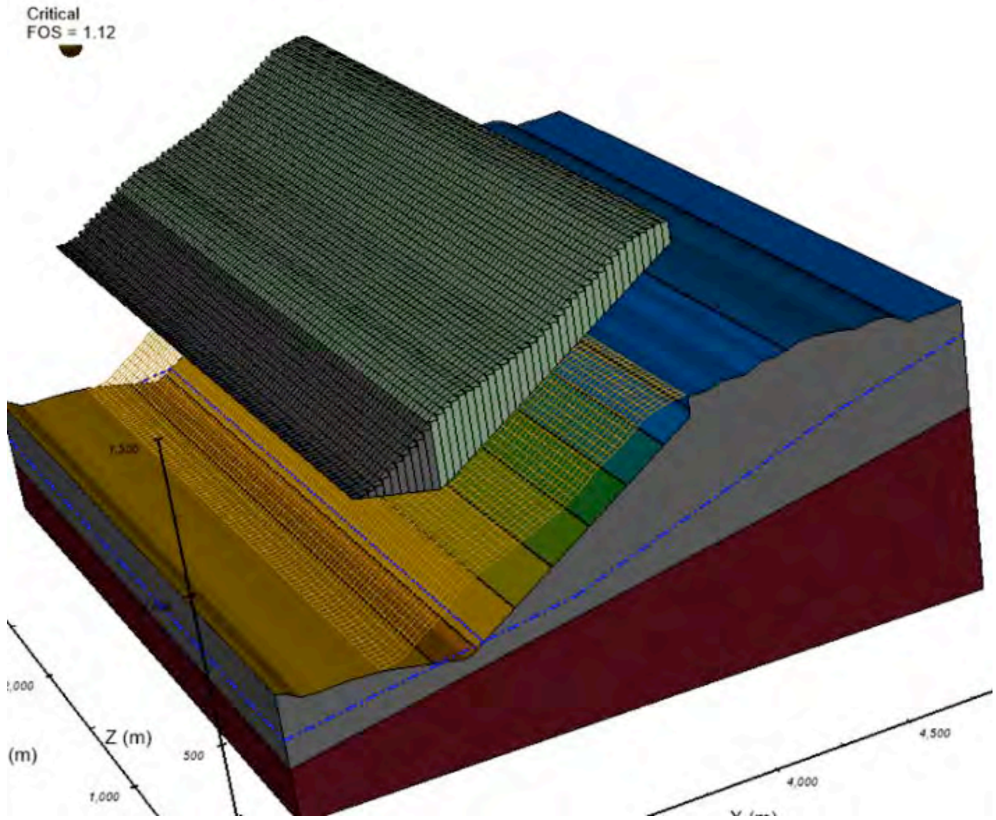

Figure 2 Equivalent 3D analysis (slip surface exploded out of slope)

It should be noted that a simple proof of a 3D analysis can be done by creating a 3D model of a 2D extruded slip surface and applying zero shear strength to the end-walls. This is an easy way to prove the 3D equivalent scenario of any 2D analysis. It also highlights the fundamental limitation of a 2D analysis which i) considers the slip to be of infinite length in the 3rd dimension and ii) does not consider the influence of shear strength on the end surfaces.

2 SLIP SURFACE SHAPE EFFECT

If an engineer wants to strictly determine the difference that slip surface shape makes then it is easiest to extrude a 2D profile model out to 3D space to determine the factor of safety of a slip surface assuming an ellipsoidal slip surface. It is recognized that not all 3D slip surfaces are of a perfect ellipsoidal shape but a majority of slips are an ellipsoidal shape. Therefore it was considered a reasonable starting assumption by the authors.

The benefit of this type of analysis is it allows the engineer to determine the effect on the factor of safety (FOS) of moving from a 2D plane-strain analysis to a 3D analysis with an ellipsoidal slip surface. When the slope is extruded to 3D then an aspect ratio for the ellipsoid must be found through a trial-and-error process. For the present paper a number of example models were selected as benchmark 2D models and a few are presented in Figure 3 and Figure 4. These models were selected out of list of classic slope stability benchmarks as compiled by SoilVision Systems Ltd.

A common question related to 3D analysis is: “Won’t a 3D analysis of an extruded profile tend to an ever wider and wider 3D elliptical slip surface as it tries to approach the 2D result (with its lower FOS)?”. One of the key aspects to answering this question is related to the assumption of the slip surface shape. If the shape is cylindrical with small beveled or rounded ends then as the cylinder becomes longer and longer the effects of the shear on the ends theoretically becomes a smaller and smaller percentage contribution. It should be noted, however that the “extended cylinder” described in the above argument is not a common slip surface failure shape in the field. If we assume an elliptical slip surface failure then the results illustrate that the failure does not tend to be infinitely wide as previously assumed.

Figure 5 shows the results of extruding the VS_24 model to an equivalent 3D numerical model. It can be seen from this extruded model that a wide range of aspect ratios for the ellipsoid have been tried (the trial slip surfaces have been plotted) and the ellipsoid with the minimum FOS corresponds to an aspect ratio of 16.8 with a minimum FOS of 1.72 which is a 19.6% increase over the 2D FOS of 1.438. The results of extruding three such 2D models to 3D are shown in Table 1. It can also be seen from these results that the difference for simple and low-angle slopes between the 2D and 3D FOS ranges between 4.3% to 20%.

Table 1 Results of 2D profiles extruded to 3D

	2-D		3-D			
Model	FOS	Max Depth (m)	FOS	Aspect	Max Depth (m)	Diff
VS_1	0.988		1.031	4.8	3.013	4.35%
VS_3	1.375	4.76	1.45	8.6	4.56	5.45%
VS_24	1.438	6.889	1.72	16.8	6.814	19.61%

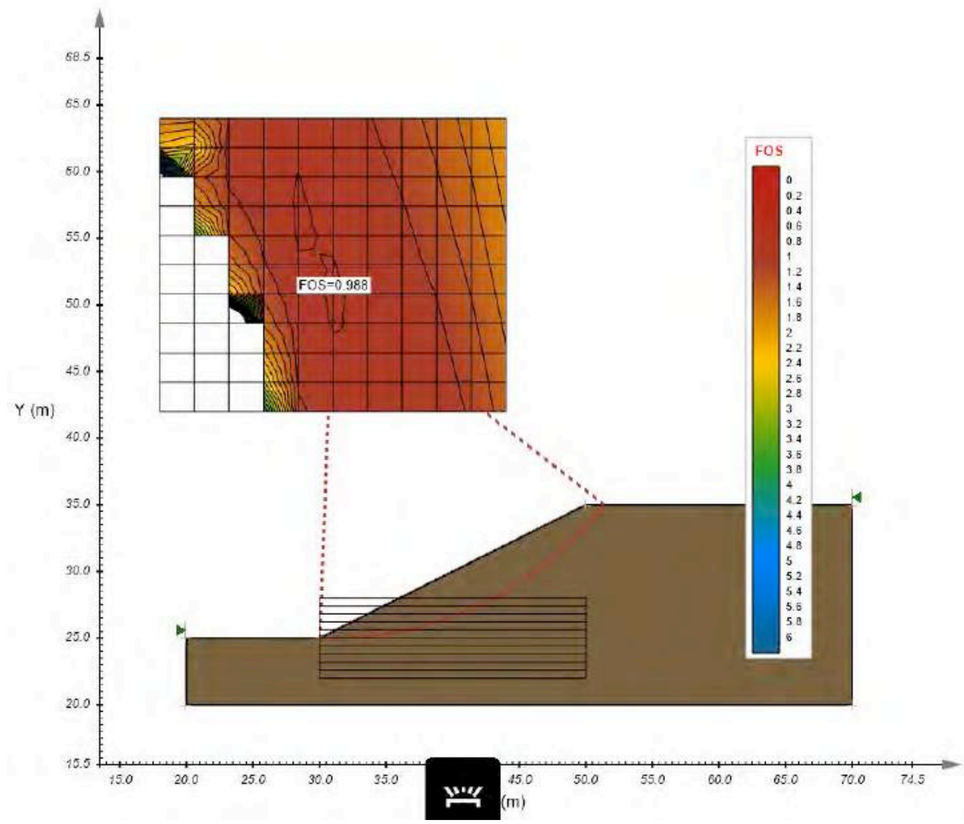

Figure 3 VS_1 2D profile model

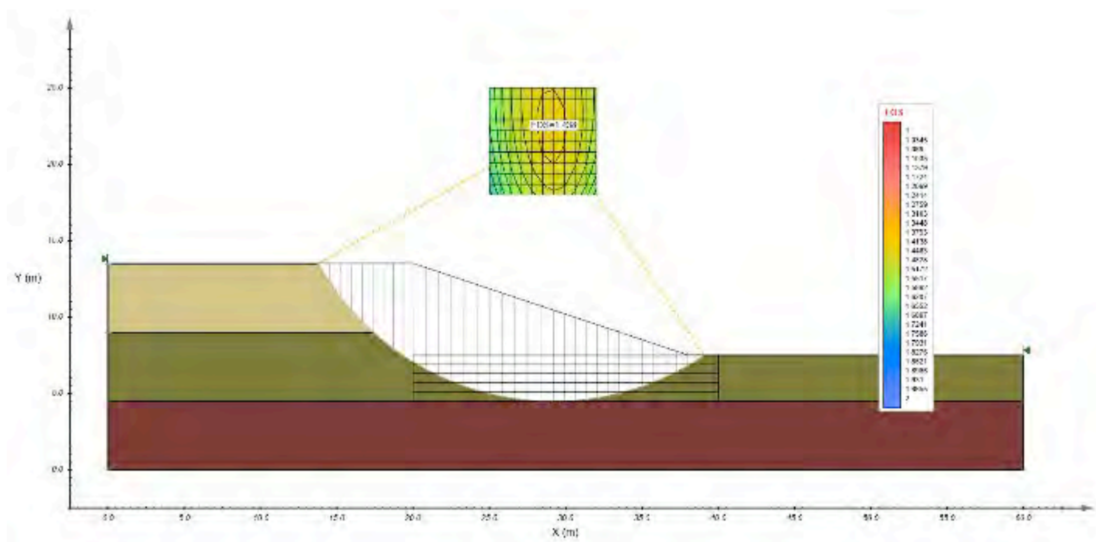

Figure 4 VS_24 2D profile model

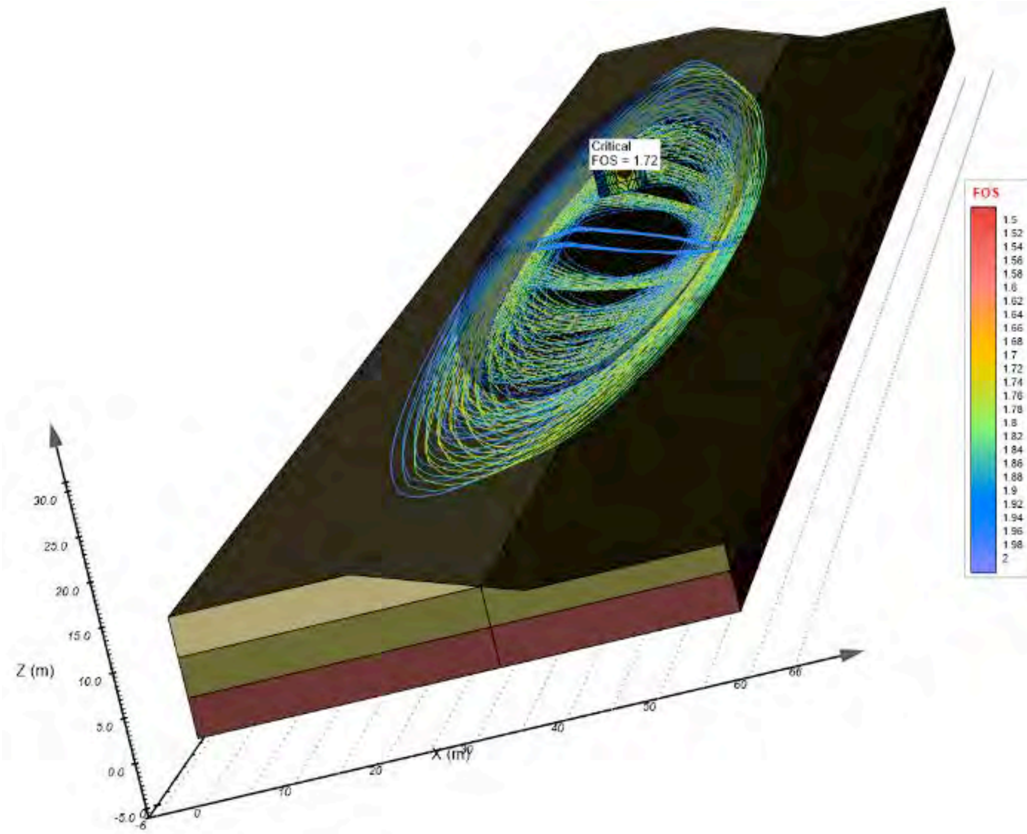

Figure 5 VS_24 Extruded 3D model with critical slip surface

3 GROUND SURFACE TOPOGRAPHY VARIATIONS

The second primary reason why 3D analysis differs from 2D analysis is that there may be variation of the surface topology Such variation can produce differences in the computed FOS. Such variations can largely be grouped into surface shapes which are convex or concave. While it is recognized that typical topography may be highly variable, the general trends can be easily quantified with simple concave and convex shapes. Such an analysis has been performed (Zhang, 2015; Domingos, 2014) and the geometry for the comparison can be seen in Figure 6.

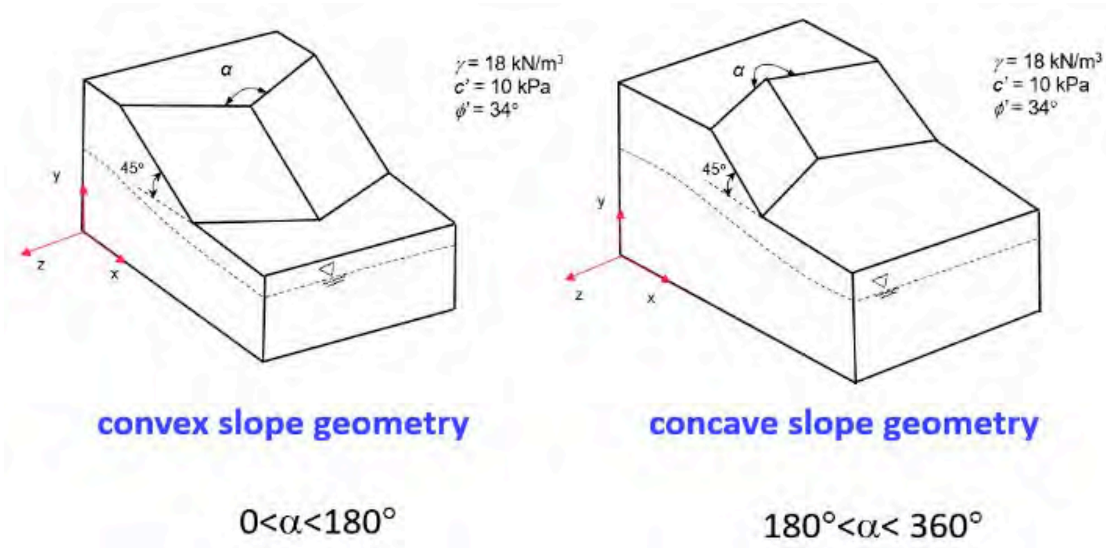

Figure 6 Convex and concave geometries used in 3D simulations

From analyzing the geometry it can be seen that i) the consideration of such geometries results in an increase in the computed FOS over a 2D scenario and ii) the consideration of unsaturated shear strengths further increases the computed FOS in 3D. Thus it can also be surmised that the consideration of 3D geometries in a 3D numerical model can often result in computed FOS values which are higher than 2D analysis and have the potential to save on construction costs in the design of slopes.

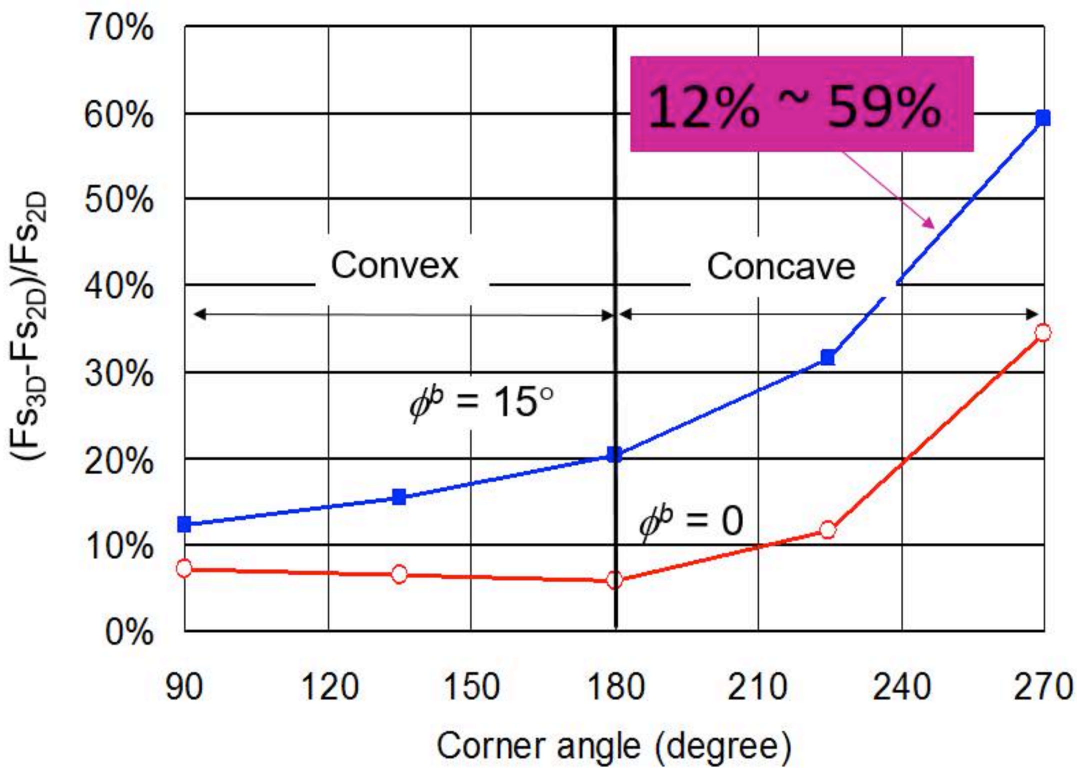

Figure 7 Differences between 2D and 3D computed FOS for convex and concave saturated/unsaturated slopes (Zhang, 2014)

The results of the influence of surface topology suggest that the consideration of such influence may not be trivial. The consideration of 3D effects of topology may therefore be useful in the practical application of 3D slope stability.

4 2D MULTI-PLANE ANALYSIS

One of the secondary issues with a standard 2D geotechnical analysis is that the geotechnical engineer may not know the correct location of the 2D plane which produces a critical factor of safety. A classic example of such a problem was presented by Jian (2003). The example presents a simple slope to illustrate two problems with conventional 2D plane-strain stability analysis; namely that the location of the slip as well as the correct factor of safety can be difficult to determine from this relatively straightforward example.

Multi-plane analysis (MPA) allows the engineer to quickly perform hundreds or thousands of analyses around a slope quickly using parallel computations. In the 2D MPA analysis method the slope is sliced into 2D profiles each of which is a full 2D LEM analysis. Each slice is analyzed and the results are then plotted over top of the original 3D slope therefore giving a spatial perspective to the many 2D slices. Each slice is oriented such that the primary slip direction is the steepest slope. Since the slip may not happen on the steepest slope the MPA analysis method can be specified such that multiple orientations, such as +/- 10 degrees can be analyzed. In this way the most likely slip orientation can also be determined.

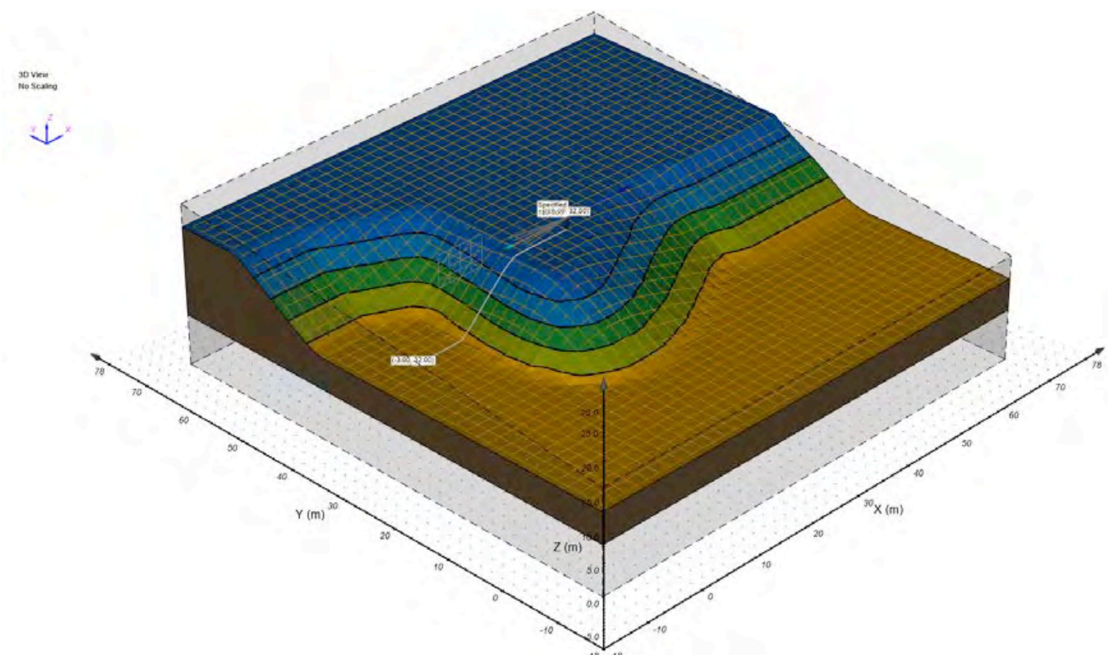

Figure 8 Example of slope difficult to analyze in 2D plane strain (Jiang, 2003)

The analysis of slope in Figure 8 can be seen in Figure 9. From the analysis we can see the critical zone clearly outlined as to the left of the nose of the hill. This is counter-intuitive from the sense that most engineers would feel that the point of the nose of this model would yield the lowest FOS value. Therefore this illustrates the value of using analytical procedures to determine the correct location of a probable failure zone in an irregular topology. This result must also be noted that it is only the result of 2D slices through differing spatial locations in the 3D model. There is no relation implied or computed between the slices so the 3D lateral effects on the slope are not considered. Therefore the model must be considered as an indication of a probable failure zone but not a definitive analysis which considers 3D effects.

It can be seen in **Figure 10** that the methodology can be applied to real-world designs with complex geometries in order to provide a comprehensive view of where critical slip surfaces may occur.

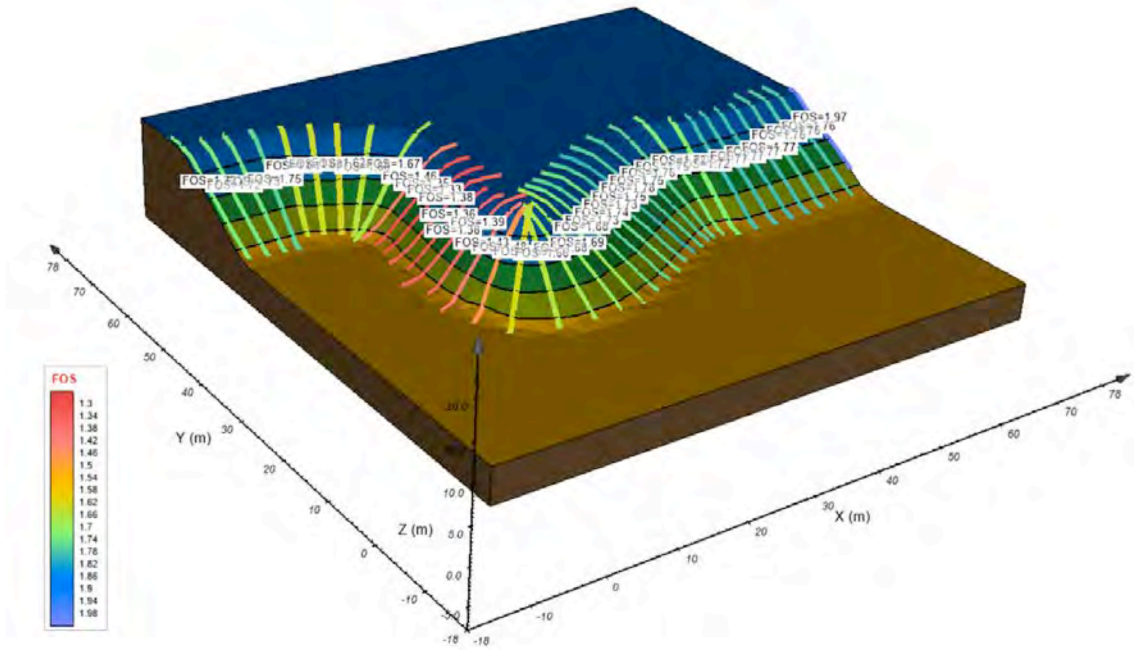

Figure 9 Example of a 2D MPA analysis

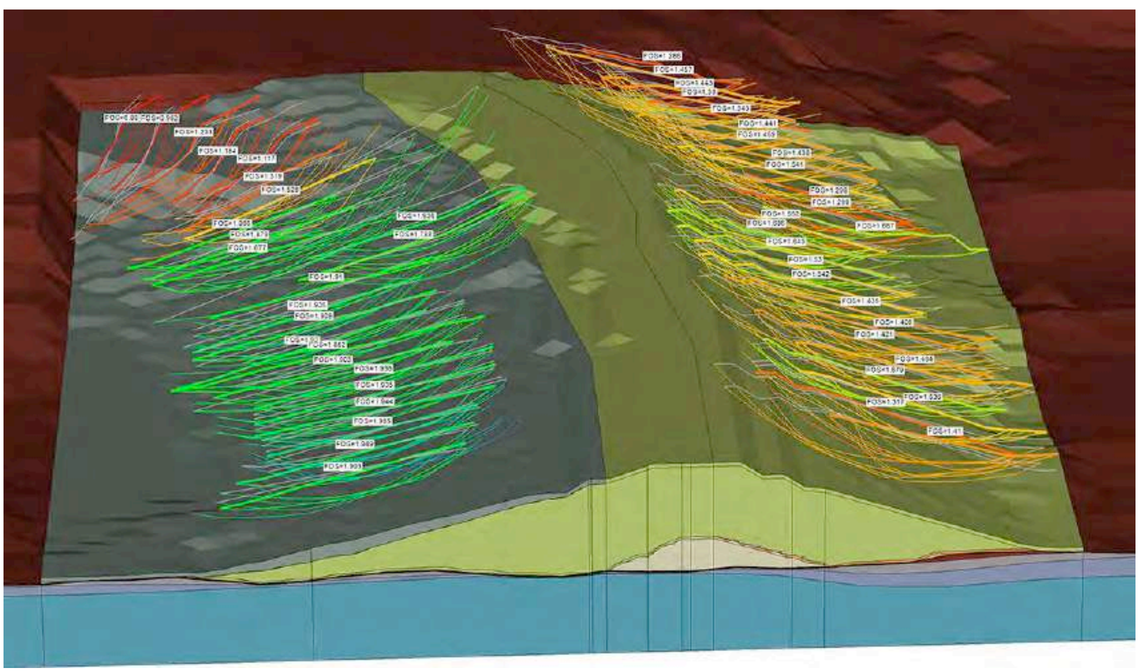

Figure 10 Example of tailings dam analysis by MPA 2D method

5 3D MULTI-PLANE ANALYSIS

The benefit of the multi-plane analysis is that it can easily be applied in 3D space using the same slicing planes established for the 2D analysis only with the analysis of a 3D ellipsoid at each slicing plane location. Similar searching techniques such as Entry & Exit, Grid & Tangent, Slope Search, or Auto Refine methods can be applied to the searching technique. The aspect ratio of the ellipsoid as well as faults and fractures can be considered as well in the analysis.

An enormous amount of information is generated with each slice being considered as a full 3D analysis. The 3D results can therefore be presented as a series of individual critical slip surfaces or contoured to produce a contoured map of the relative factor of safety over an area. The results of such an analysis for an open pit are shown in Figure 10. It can be seen that MPA is a useful methodology for i) understanding the relative factor of safety of a large spatial area as well as ii) locating potential zones of instability which may exist. The 3D results also provide a higher and computationally more realistic analysis of the true 3D FOS.

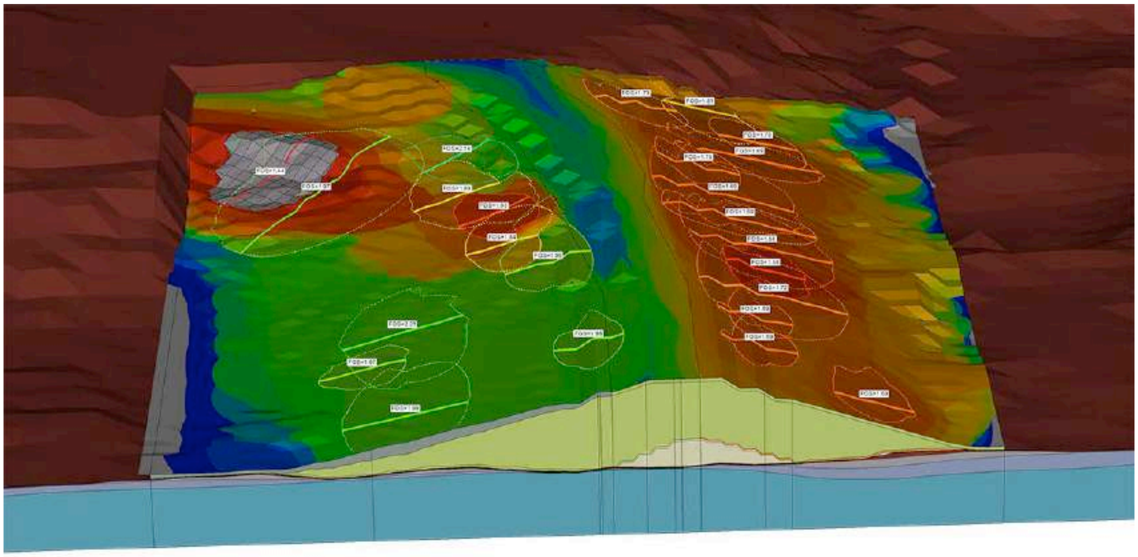

Figure 11 Example of 3D MPA applied to the analysis of an open pit

6 SUMMARY

A methodology for moving geotechnical engineers forward from the traditional 2D LEM analysis to a more comprehensive 3D LEM analysis is presented. The method of understanding the logical transition to a 3D numerical model in consulting practice is defined and the methods of simplifying a 3D analysis back to a comparable 2D analysis are presented. The specific differences between a 2D and a 3D slope stability analysis are reduced to their components of i) slip surface shape, ii) topography differences, and iii) geo-strata differences.

The difficulty of knowing where a critical slip surface may exist is addressed through the use of multi-plane analysis (MPA). Through the MPA method the geotechnical engineer can make use of hundreds or thousands of analysis specified over a large 3D numerical modeling space such that the relative values of FOS can be computed and contoured over large spatial areas. Such analysis greatly enhances the ability of the geotechnical engineer to determine stability values over large spatial areas and has application in the area of municipal design, roadways/railways, open pits, tailings facilities and other large earth structures.

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Sloughing of a tailings excavation slope due to rapid drawdown

D. Johnson, M. Tyldesley & B. Riddell
Golder Associates Ltd., Mississauga, ON, Canada

ABSTRACT: This paper describes the sloughing of a temporary tailings excavation slope during construction of a soil cover for rehabilitation of a historic tailings management area. Tailings were being excavated with a 10 horizontal to 1 vertical slope prior to placement of the soil cover. Slope stability analyses were carried out to back-calculate groundwater conditions required to replicate the observed sloughing of the excavated tailings slope. Stability analyses confirmed that the observed sloughing was caused by rapid drawdown. A groundwater model was used to predict potential long-term high groundwater levels in the tailings beneath the cover and ensure the long-term stability of the tailings cover. Based on the slope stability analysis results described in this paper, the tailings soil cover design was modified to improve the constructability and long-term stability of the channel and soil cover.

1 INTRODUCTION

1.1 *Background*

This paper discusses design and construction of a multi-layer soil cover for rehabilitation of two Tailings Management Areas (TMAs) at a decommissioned uranium mine in Canada (the Site). Placement of a soil cover over the tailings is being carried out to reduce the emission of radon gas into the air from the tailings and also reduce infiltration of precipitation into the tailings.

Junqueira et al. (2016) describe the cover field trial program and option evaluation that was carried out to select a preferred soil cover for TMA rehabilitation at the Site. The selected multi-layer soil cover consists of a 0.4 m thick sand/bentonite layer (10% bentonite by weight) beneath a 1 m thick layer of granular material. The primary objective of the soil cover is to reduce radon emissions from the tailings to an acceptable level. In addition, the final TMA landform must be physically stable and assimilate into the surrounding environment; including vegetation growth at the surface that does not affect the performance of the cover. The soil cover should require limited maintenance and monitoring requirements after construction when exposed to a range of expected long-term post-closure climatic conditions (e.g., freeze / thaw cycles, wet and dry seasons). Finally, the cover will minimize infiltration of precipitation resulting in reduced groundwater discharge.

To meet the physical stability objective of the cover and to achieve the desired final topography, the tailings surface required excavation and grading of drainage channels prior to soil cover placement. The final cover surface incorporates a main drainage channel and several swales that will prevent any ponding while limiting non-channelized overland flow path lengths to 200 m. The cover has a maximum slope of 10% to reduce the potential for erosion over the long-term. The drainage system is designed in accordance with the Canadian Dam Association (CDA) recommended design flood for dam closure (CDA, 2014). The drainage system design includes sufficient erosion protection measures, such as rip-rap, to safely convey the design storm event while controlling erosion and limiting flooding. Rip-channel sizing and bedding filter specifications were designed to resist erosion in the main channel and swales during the design storm. After completion of the TMA rehabilitation and soil cover construction, the TMA perimeter dams will no longer be required to function as water retaining structures.

Rehabilitation activities involved with construction of the TMA soil cover include:

1. Clearing and grubbing of the existing TMA surface;
2. Excavating tailings to the cut grades required for the soil cover drainage design;
3. Hauling, spreading and compacting excavated tailings to the fill grades required for the soil cover drainage design;
4. Excavating on-site granular borrow material and blending with bentonite to create a Sand-Gravel-Bentonite (SGB) material for the soil cover system;
5. Construction the multi-layer SGB soil cover system over the graded tailings surface;
6. Placement of erosion protection materials in the swales and channel; and
7. Establishment of vegetation on the soil cover surface.

This paper describes sloughing or slumping that occurred at the base of the tailings excavation prior to placement of the multi-layer soil cover over the graded tailings surface and how the observed sloughing influenced the ultimate TMA rehabilitation design.

1.2 *Observed sloughing*

Sloughing of a tailings slope was observed in an excavated swale prior to placement of the overlying soil cover materials. The side slope of the tailings excavation that experienced sloughing was 10 horizontal to 1 vertical (10H:1V) and the tailings slope was covered with a temporary non-woven geotextile to control erosion during the construction period. Figure 1 is a photograph of the excavated swale slope where sloughing of the tailings was observed.

Although erosion of exposed tailings slopes had occurred around the site; the large volume of tailings that moved to the base of the swale indicate that sloughing or slope instability was the likely mechanism of the tailings movement. Furthermore, the excavated tailings surface was covered in a non-woven geotextile reducing the potential for extensive surface erosion.

The soil cover design and specifications required that the excavation be dewatered prior to placement of cover materials over the tailings. The tailings excavation had not been fully dewatered when the sloughing occurred. During the tailings excavation, groundwater seepage was draining into the excavation and precipitation events would cause temporary ponding of water at the base of the excavation, which was then pumped down as part of the active dewatering effort. The rapid drawdown of the ponded water in the excavation and groundwater seepage into the excavation were inferred to be contributing factors to the observed sloughing.

2 MATERIAL PROPERTIES

2.1 *Tailings grain size distribution*

As part of the site characterization and rehabilitation option evaluation, a number of field investigation programs were carried out in the tailings management area. A total of sixty seven grain size distribution tests were completed on tailings samples, which are presented in Figure 2. Utilizing the Unified Soil Classification System (ASTM D2487-11), the tailings can be characterized as varying from a silty fine sand (SM) to a silt (ML). The grain size distributions of the tailings vary spatially (both horizontally and vertically) throughout the tailings management area due segregation related to the sequencing of historic tailings deposition.

Figure 1. Photograph of the observed tailings sloughing in the lower swale.

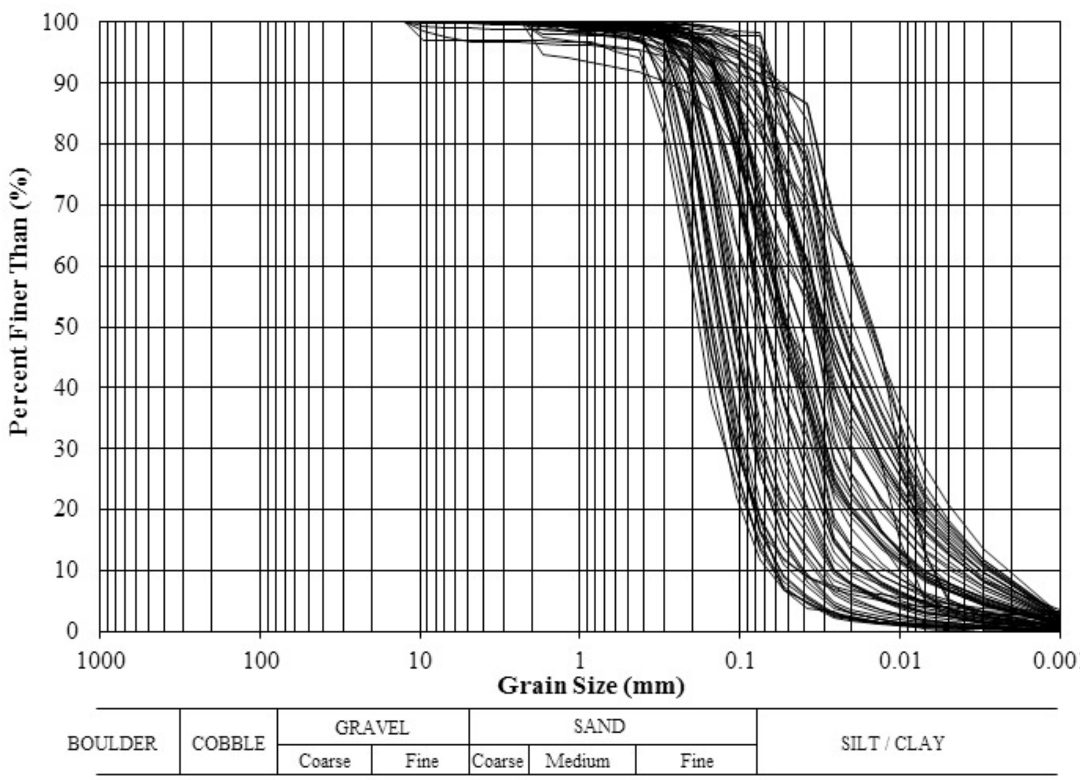

Figure 2. Tailings grain size distribution test results.

2.2 *Hydraulic conductivity*

Hydraulic conductivity of the tailings, underlying native granular materials and bedrock was measured in-situ with packer testing of open bedrock boreholes and single-well response tests of monitoring wells. The testing consisting of a combination of falling, rising and constant head test conducted at varying depths within each respective strata. The hydraulic conductivity at each tested interval was estimated using the Hvorslev method (Hvorslev 1951).

A total of five well response tests were completed in monitoring wells screened within the tailings. The results of the well response tests conducted in the tailings indicate the intervals tested have hydraulic conductivities ranging from 1.0 x 10^{-5} m/s to 4.0 x 10^{-7} m/s, with a geometric mean hydraulic conductivity of 3.2 x 10^{-6} m/s. The large variability in measured hydraulic conductivities within the tailings is consistent with the wide range of gradations present.

A total of 23 well response tests were completed in monitoring wells screened within the native granular strata underlying the tailings. At the time the monitoring wells were installed, the native materials were visually classified as ranging from sand and gravel to silty sand. The results of the well response tests conducted in the native granular material indicate hydraulic conductivities ranging from 9.3 x 10^{-4} m/s to 1.1 x 10^{-5} m/s, with a geometric mean hydraulic conductivity of 1.6 x 10^{-4} m/s.

3 ANALYSES

3.1 *Slope stability analysis methodology*

A two-dimensional (2D) limit equilibrium approach was used to analyze the slope stability of the excavated tailings slope and the tailings cover. Slope stability analyses were carried out using the commercially available software GeoStudio Slope/W 2007 Version 7.17 and the limit equilibrium method based on Morgenstern & Price (1965).

3.2 *Slope stability analysis material properties*

Effective stress parameters were used for the stability analyses. The material properties used in the slope stability analyses are listed in Table 1.

Table 1. Slope Stability Analysis Material Properties.

Material Zone	Unit Weight (kN / m^3)	Internal Angle of Friction φ' (deg)	Cohesion c' (kPa)
Zone 1 – Tailings	18	28	0
Zone 2 – Sand Gravel Bentonite (SGB)	18	30	0
Zone 3 – Sand and Gravel	19.5	30	0
Zone 4 – Rip Rap Bedding	21	30	0
Zone 6 – Rip Rap	22	35	0
Zone 9 – Waste Rock Subgrade	22	35	0

3.3 *Back calculation of observed tailings sloughing*

The geometry of the slope and material properties have a direct influence on slope stability. The tailings excavation side slopes where sloughing was observed had a 10% grade (i.e., 10H:1V). Slope angle alone cannot explain the sloughing that was observed. In the absence of pore water pressure, the Factor of Safety (FoS) of a slope is equal to tan φ' / tan β. This would result in a FoS of 5.3 for a 10H:1V tailings slope with a φ' equal to 28 degrees, indicating a stable slope.

Excess pore water pressure in slope materials caused by rapid drawdown can have a negative influence on slope stability. Rapid drawdown occurs when the water level outside a slope drops and the stabilizing influence of the water pressure on the slope is lost. If the water level subsides so rapidly that the pore water pressure within the slope does not have time to equilibrate to the drop in external water level, the slope is made less stable (Duncan et al. 2014). Rapid drawdown can be simulated in slope stability analyses using the limit equilibrium method with an elevated phreatic surface.

Slope stability analyses were carried out to back-calculate groundwater conditions required to replicate the observed sloughing of the excavated tailings slope. The assessed cross-section is located perpendicular to the lower swale excavation slope identified as Section A-A' in plan on Figure 3. Several scenarios were modelled to determine the lowest FoS and to replicate the observed sloughing conditions.

Figure 4a shows the case with a horizontal phreatic level equal to the elevation of the swale bottom. The lowest FoS for this case is 4.0 indicating the slope is stable.

Figure 4b shows the case with the groundwater table (i.e., phreatic surface) following the excavated tailings surface across the swale. This resulted in a minimum FoS of 2.4, indicating that the slope is stable. This result indicates that, in order for the slope to have slumped, an elevated pore water pressure condition (i.e., above ground surface) would have to be present.

Figure 4c shows the results of the slope stability analysis with the phreatic level elevated 0.5 m above the channel bottom. This resulted in a shallow failure surface at the base of the channel (FoS of 0.7) similar to the observed sloughing. Larger slip surfaces that extend above the assumed phreatic surface had a FoS greater than 2.0, as shown in Figure 4d, indicating that any slumping would be limited to the zone of pore water pressure elevation in the tailings (i.e., the ponded water level prior to rapid drawdown).

Stability analyses confirmed that an elevated pore water pressure in the excavation could cause slope instability. Based on observations of the sloughing, it is likely the elevated pore water pressure at the base of the excavation was caused by rapid drawdown of the water level adjacent to the slope when tailings were excavated below the water table.

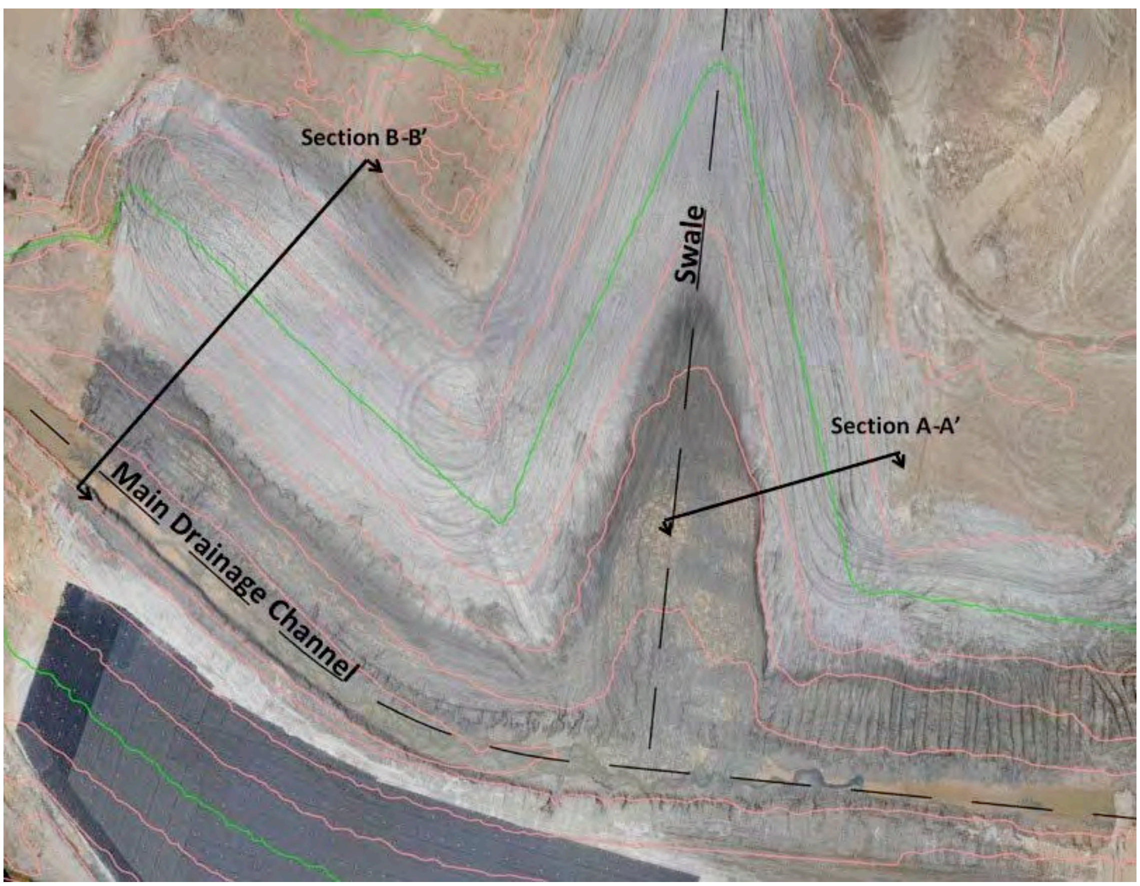

Figure 3. Plan view illustrating slope stability cross-section locations.

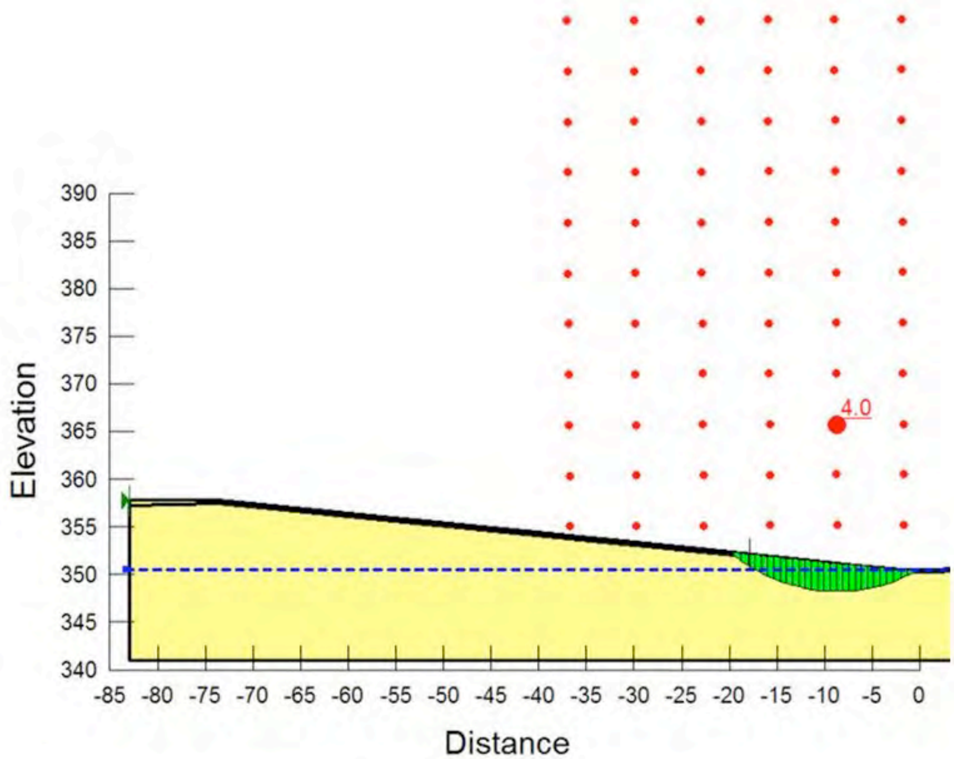

Figure 4a. Slope stability model with phreatic surface at channel bottom (FoS=4.0).

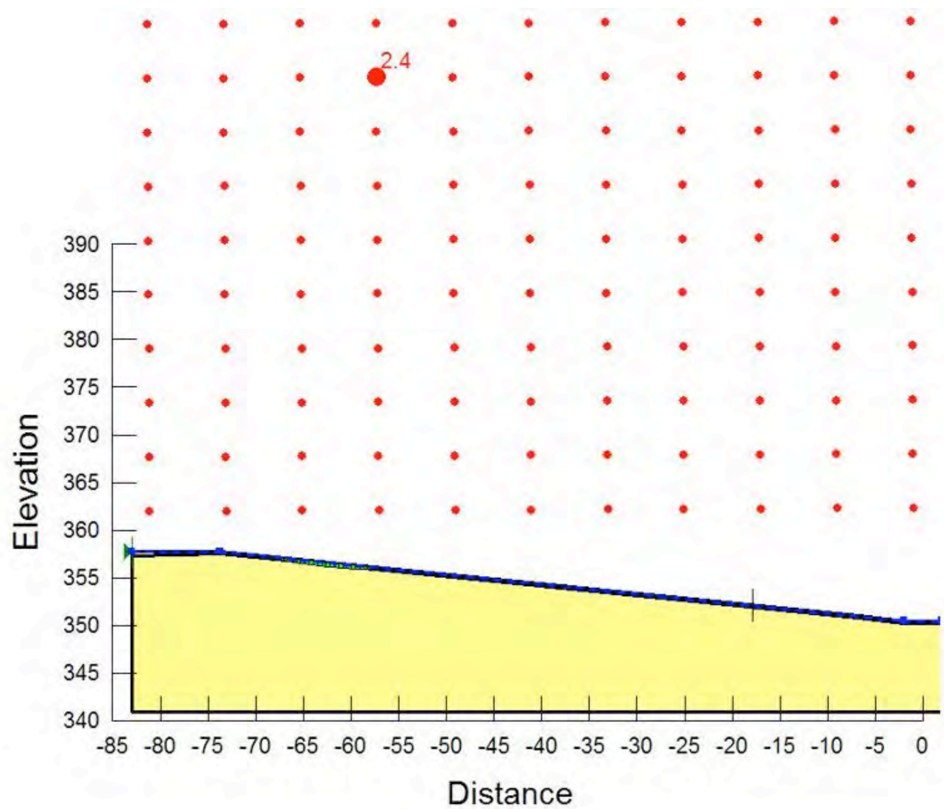

Figure 4b. Slope stability model with phreatic surface along ground surface (FoS=2.4).

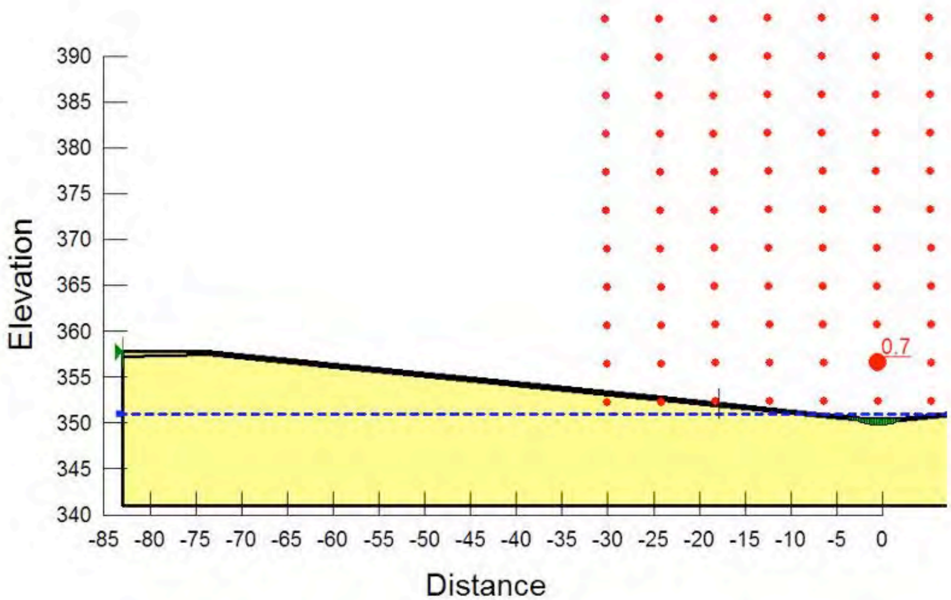

Figure 4c. Slope stability model with phreatic surface elevated 0.5 m above the channel bottom indicating shallow slumping below the water table (FoS = 0.7).

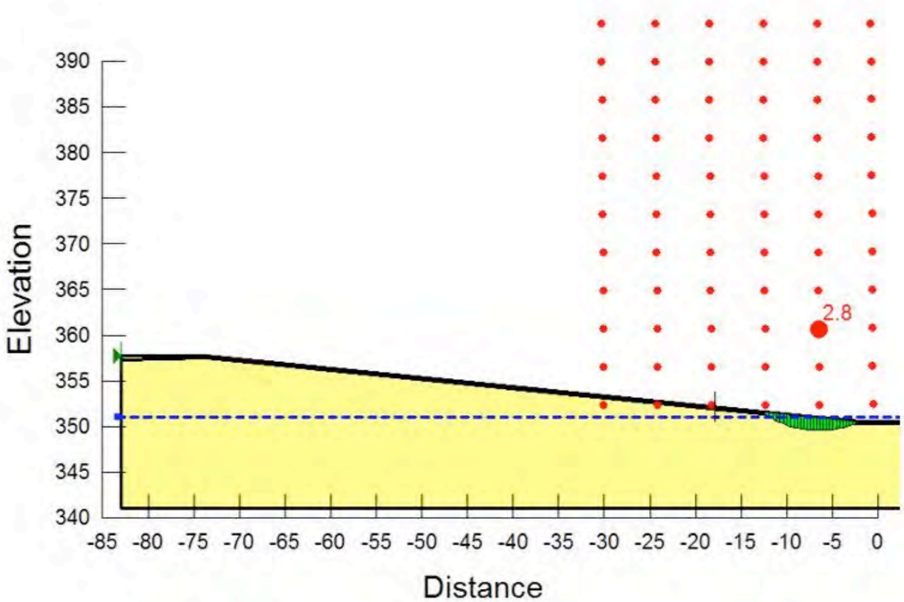

Figure 4d. Slope stability model with phreatic surface elevated 0.5 m above the channel bottom indicating stable slopes above water table (FoS = 2.8).

3.4 *Prediction of potential long-term high groundwater levels*

The TMA rehabilitation must be stable over long-term post-closure conditions. A 3-Dimensional (3D) groundwater model was developed to predict potential long-term high pore-water pressures in the tailings beneath the cover (i.e., due to the design storm event and potential rapid drawdown). The predicted maximum groundwater pressures were used in the slope stability analyses for the tailings cover.

3.4.1 *Long-term stability of tailings cover*

Stability analyses were carried out to check the long-term stability of the soil cover with elevated pore water pressure in the tailings beneath the cover calculated from the 3D groundwater model (i.e., due to design storm event and subsequent rapid drawdown). The assessed cross-section is located perpendicular to the main drainage channel identified as Section B-B' in plan on Figure 3.

During a rapid drawdown condition, tailings underlying the containment by the low permeability SGB layer could maintain elevated pore water pressure after water drains off the overlying cover; resulting in loss of the resisting force of the overlying weight of water, while elevated pore water pressure remains in the underlying tailings. This condition was modeled by simulating two phreatic surfaces within different materials. The first piezometric surface follows the top of Zone 2 – SGB and represents the drained channel. This phreatic surface was applied to the cover materials. The second phreatic surface is assumed to be horizontal at a fixed elevation, representing the high water level before rapid drawdown. This phreatic surface was applied to the tailings underlying the Zone 2 – SGB. A sensitivity analysis was carried out for various phreatic surface elevations. Figure 5 illustrates the slope stability model cross-section with an elevated phreatic surface of 353.2 m in the tailings that resulted in a FoS of 1.0 (i.e., state of instability). The minimum FoS for various groundwater elevations in the tailings are summarized in Table 2.

Based on the 3D groundwater modeling results, the predicted maximum groundwater elevation within the tailings underlying the cover is 352.0 m following the 100 year return period rainfall event. This corresponds to a FoS equal to 2.0 indicating a stable slope following the 100 year return period rainfall event.

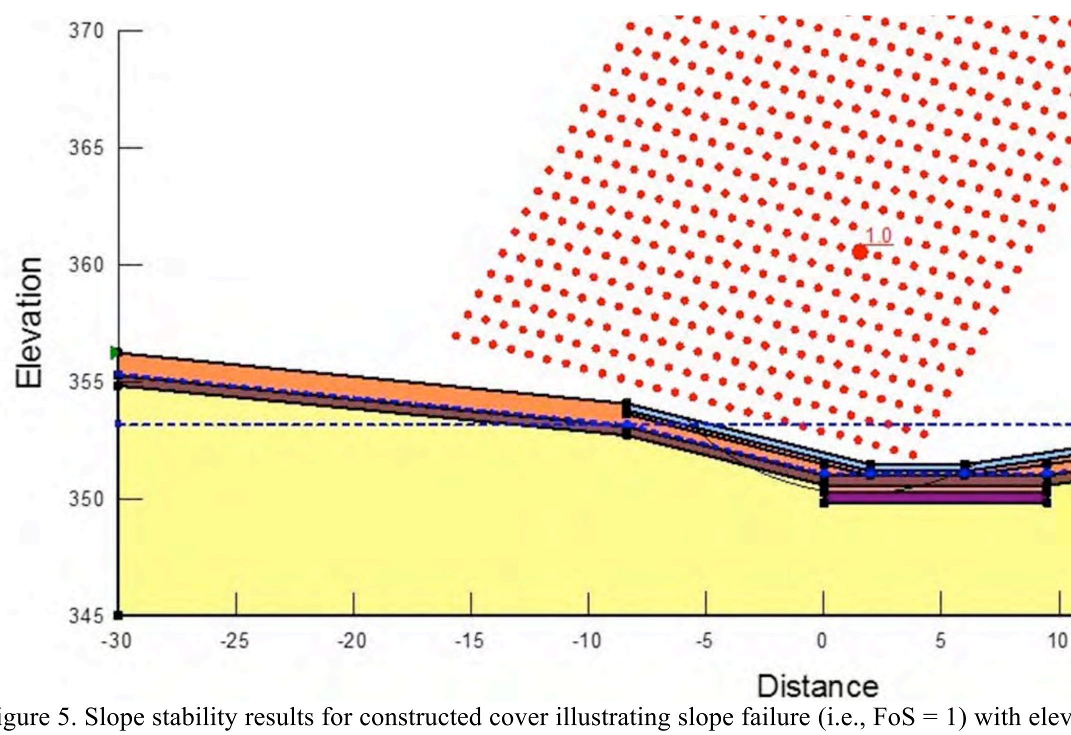

Figure 5. Slope stability results for constructed cover illustrating slope failure (i.e., FoS = 1) with elevated phreatic surface in underlying tailings at 353.2 m (i.e., simulating rapid drawdown).

Table 2. Long-Term Covered Slope Stability Analysis Results for Various Groundwater Elevations

Tailings Groundwater Elevation (m)	Minimum Calculated Factor of Safety
352	2.0
352.5	1.6
353	1.2
353.2	1.0

4 SOIL COVER DESIGN MODIFICATIONS

Based on the observed sloughing and subsequent slope stability analysis results described in this paper, the tailings soil cover design was modified to improve the constructability and long-term stability of the channel and soil cover. Groundwater modeling results determined that there was potential for a temporary elevated phreatic surface within the underlying tailings following the design rainfall event. The outlet detail for the main drainage channel was modified to include granular filter compatible layers which can safely drain excess pore water pressure from the underlying tailings following the design rainfall event.

To improve temporary construction conditions and to allow for the placement and compaction of the soil cover profile over soft tailings in the base of the drainage channel, a 0.5 m thick zone of coarse rockfill, overlain with filter compatible bedding layers was integrated into the soil cover profile design.

5 CONCLUSION

Slope stability analysis back calculations determined that sloughing of an excavated tailings slope was caused by elevated pore water conditions likely caused by flooding in the channel followed by rapid drawdown. During rapid drawdown, tailings underlying the low permeability sand-gravel-bentonite cover layer could maintain an elevated pore water pressure after the overlying cover drains; potentially resulting in slope instability. A 3D groundwater model was developed to predict potential long-term maximum pore water pressures in the tailings beneath the cover following the design storm event. Slope stability analyses were carried out with the maximum groundwater pressures predicted by the 3D groundwater model to check the long-term stability of the tailings soil cover. Slope stability results indicate that the channel and cover slopes will remain stable following the design rainfall event. Based on the observations of the sloughing of the tailings excavation during construction and the subsequent slope stability analysis results, the tailings soil cover design was modified to improve the constructability and long-term stability of the channel and soil cover.

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Sulphide oxidation in TSFs and its influence on long term dam stability

Jennifer Durocher & Lindsay Robertson
Klohn Crippen Berger Ltd., Sudbury, ON, Canada

Brent Usher
Klohn Crippen Berger Ltd., Brisbane, QLD, Australia

ABSTRACT: Geochemical processes that occur at the mineral-water interface, within tailings storage facilities, vary widely depending the surrounding physical and geochemical properties of the tailings and surrounding surface, pore and seepage water. The formation of secondary minerals in tailings storage facilities, including within the tailings stratigraphy, filter materials, drains, and embankment dam fill, can alter the hydrologic and physical properties and subsequently, the design intent, of these materials. Geochemical reactions, including mineral dissolution and hydrolysis, solution neutralization, solid-solution-substitution, surface sorption processes and solution precipitation reactions, alter the mineralogical composition, structure, strength, porosity, and permeability of TSF materials. As the long-term management of TSFs are reviewed by the engineering community (including regulators, owners, and consulting engineers), geochemical reactions have been shown to influence dam stability and should be considered in TSF design, monitoring and management. This paper presents secondary iron mineral formation mechanisms that potentially influence the physical properties of TSFs.

1 INTRODUCTION

The oxidation and subsequent dissolution of sulfide minerals in tailings storage facilities (TSFs) results in the release and mobilization of acidity, metals and metaloid(s) such as Fe, Ni, Cu, sulphate and various trace elements, to the tailings surface and pore waters. These processes are commonly known as acid rock drainage (ARD) and metal leaching (ML). Reviews of sulphide oxidation, the formation of acid mine drainage and the mobilization of metal(oid)s are given by Lowson (1982), Evangelou (1995), Nordstrom (1997), Alpers (1999), Jambor (2003), Blowes (2003) and Lottermoser (2007); among many others.

Acidic surface and porewaters, containing elevated metal(oid)s, subsequently react with surrounding gangue minerals of the TSF, in a complex set of thermodynamic and kinetic reactions that include: mineral dissolution and hydrolysis, solution neutralization, solid-solution-substitution, surface sorption processes, and solution precipitation reactions. These reactions alter primary minerals, change the composition of tailings surface and porewaters, and lead to the formation of secondary minerals (i.e. formed post tailings deposition). Secondary minerals that form from the oxidation of sulphide-bearing tailings include various M-oxides, M-oxyhydroxides, M-sulphates (including M-hydroxy-sulphates) (where M includes Al, Fe, Ca, Cu, Pb or Ba), depending on the chemical composition of the surrounding water, pH, Eh, temperature, pressure and the presence of micro-organisms.

In TSFs, where the tailings contain primary iron sulphide minerals (such as pyrite and pyrrhotite), secondary Fe-bearing minerals are common. Secondary iron minerals occur throughout the tailings stratigraphy and dam sections, as hardpan layers in the oxidized surficial zone (or previous surficial zones), amorphous sludges settled in ponds and as a combination of crystalline and amorphous precipitates at seepage emergence points. Secondary minerals differ from primary minerals in terms of their chemical composition, density, grain size, and, therefore strength and porosity (Table 1).

Table 1. Properties of Primary and Secondary Minerals Common to Base Metal TSF's

Primary Mineral	Sym	Chemical Composition	Molecular Weight (g/mol)	Specific Gravity	Hardness
Pyrrhotite	Po	$Fe_{(1-x)}S$	85.12	4.61 (avg)	3.5-4.5
Pyrite	Py	FeS_2	119.98	5.01	6.5
Galena	Gl	PbS	239.27	7.4	2.5
Chalcopyrite	Cpy	$CuFeS_2$	183.53	4.1-4.3	3.5-4.5
Alkali Feldspar	Fp	$(K,Na)[Ali_3O_8]$	278.33	2.55-2.63	6
Plagioclase	Pg	$Na[AlSi_3O_8]$-$Ca[Al_2Si_2O_8]$	270.77	2.62-2.76	6-6.5
Muscovite	Mv	$K_2Al_4[Si_6Al_2O_{20}](OH,F)_4$	398.71	2.77-2.88	2-2.5
Secondary Mineral	**Sym**	**Chemical Composition**	**Molecular Weight (g/mol)**	**Specific Gravity**	**Hardness**
Jarosite	Jar	$KFe_3(SO_4)_2(OH)_6$	500.81	2.9-3.3	2.5-3.5
Schwertmannite	Sch	$Fe_{16}O_{16}(OH)_{12}(SO_4)_2$	1545.76	3.77-3.99	2.5-3.5
Ferrihydrite	Fhy	Fe_2O_3•0.5(H_2O)	168.70	3.8	
Melaterite	Mel	$Fe^{2+}(SO_4)$•7(H_2O)	278.02	1.89-1.9	2
Rozenite	Roz	$Fe^{2+}(SO_4)$•4(H_2O)	223.97	2.2	2.3
Copiapite	Cop	$Fe^{2+}Fe^{3+}{}_4(SO_4)_6(OH)_2$•20($H_2O$)	1249.94	2.1	2.5
Halotrichite	Hal	$Fe^{2+}Al_2(SO_4)_4$•22(H_2O)	890.40	1.78-1.9	1.5-2
Goethite	Gt	FeOOH	88.85	4.3	5-5.5
Anhydrite	Ah	CaSO4	136.1	2.97	3.5
Gypsum	Gp	$Ca(SO_4)_2$2H2O	172.2	2.3	2
Cerrusite	Cer	$PbSO_4$	267.21	6.58	3-3.5
Kaolinite	Kln	$Al_4[Si_4O_{10}](OH)_8$	258.16	2.61-2.68	2-2.5
Smectite	Smc	$(1/2Ca,Na)_{0.7}(Al,Mg,Fe)_4$ $[(Si,Al)_8O_{20}](OH)_4$. nH_2O		2-2.7	1.5-2
Illite	Il	$K_{1.5-1.0}AL_4[Si_{6.5-7.0}Al_{1.5-1.0}O_{20}]$ $(OH)_4$	389.34	2.6-2.9	1-2

The development of secondary crystalline minerals and amorphous solid phases can change the physical and hydrologic properties of the tailings and fill materials, altering their design intent. If left unmitigated, these changes can result in structural deterioration of the dam material over time. The understanding of secondary mineral formation mechanisms and the identification of key processes along the pore-water-seepage pathway allows for successful management and mitigation of these processes.

2 SECONDARY IRON MINERAL FORMATION MECHANISMS

Primary sulphide minerals alter to more stable secondary -sulphate and -oxide forms depending on the Eh, pH and ions in solutions, which can include rain water, pore water or seepage water. As a result, primary sulphide minerals oxidize, and/or dissolve, resulting in a change in mineralogy and leads to increased oxidation products in the surrounding water. These geochemical reactions occur at the mineral-fluid interface throughout a TSF; as an example, pyrite FeS_2, oxidizes to form Fe^{3+}, H^+ ions and SO_4^{2-} ions in solution (1). The overall (simplified) oxidation of pyrite can be described as (Lawson, 1982 and Evangelou, 1995):

$$FeS_{2\,(s)} + 3.5\,O_{2\,(g)} + H_2O_{\,(l)} \leftrightarrow Fe^{2+}_{\,(aq)} + 2SO_4^{2-}{}_{(aq)} + 2H^+_{\,(aq)} \qquad (1)$$

However, the oxidation of pyrite is a series of reactions facilitated by both O_2 (2) and Fe^{3+} (3) (Figure 1) and often proceeds to various degrees of completion depending on the surrounding physio-chemical conditions (temperature, pressure, Eh, pH, etc.).

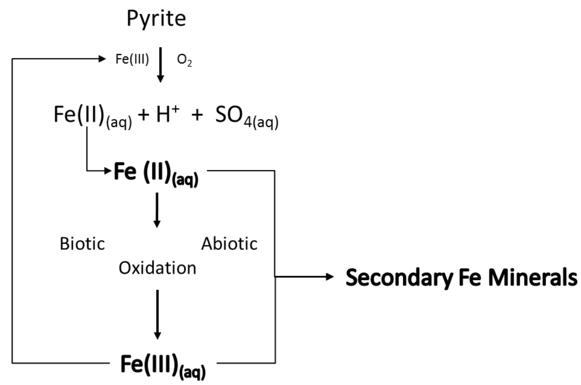

Figure 1. Simplified Schematic of Pyrite Oxidation Pathways

Partial oxidation results in the release of Fe^{2+} into solution (2). The fate of aqueous Fe^{2+} depends on the surrounding physio-chemical conditions, which vary throughout the different regions of a TSF. Aqueous Fe^{2+} is more soluble under less oxidizing conditions (low Eh) and decreasing pH conditions which allows for higher concentrations of Fe^{2+} in solution. This is typical in the less oxidized, mildly acidic, deep saturated zone of a TSF, where elevated concentrations of Fe^{2+} are mobilized in porewater. Fe^{2+} and mixed valent iron phases can form upon supersaturation.

Under more oxic conditions, aqueous Fe^{2+} oxidizes to form Fe^{3+} (3). Aqueous Fe^{3+} then acts as an oxidant to the remaining pyrite, releasing more Fe^{2+} and subsequently Fe^{3+} into solution (i.e. a positive oxidation feedback loop as shown on Figure 1).

$$FeS_{2(s)} + 3O_{2\,(g)} + 4H^+_{\,(aq)} \leftrightarrow Fe^{\,2+}_{\,(aq)} + 2SO_4^{2}{}_{(aq)} + 2H_2O_{\,(aq)} \qquad (2)$$
$$FeS_{2(s)} + 14Fe^{3+}_{(aq)}\ 8H_2O_{(l)} \leftrightarrow 15\ Fe^{2+}_{(aq)} + 16H^+_{(aq)} + 2SO_4^{2-}{}_{(aq)} \qquad (3)$$

As solutions become more oxidized (increasing Eh), and pH decreases (as oxidation increases), aqueous Fe^{2+} completely reacts with O_2 and Fe^{3+} becomes the dominant species (4); Fe^{3+} is insoluble under mildly acid to alkaline pH and will precipitate as secondary ferric-iron oxyhydroxide (FeOOH) (5).

$$4Fe^{2+}_{\,(aq)} + O_{2\,(g)} + 4\,H^+_{\,(aq)} \leftrightarrow 4Fe^{3+}_{\,(aq)} + 2H_2O_{\,(l)} \qquad (4)$$
$$FeS_{2\,(s)} + Fe^{3+}_{(aq)} + 3O_{2\,(g)} + 6H_2O_{\,(l)} \leftrightarrow 2FeOOH_{\,(s)} + 2SO_4^{2-}{}_{(aq)} + 12H^+_{\,(aq)} \qquad (5)$$

Similarly, sulphur oxidizes to form sulphate ions and acid, depending on the physiochemical conditions of the solution. In mildly acidic to neutral pH solutions in TSF's, SO_4 is the dominate species. Sulphate concentrations vary depending on the degree of oxidation and weathering, however, once sulphate becomes supersaturated in solution, secondary metal sulphates and gypsum, often precipitate.

The oxidation rate of pyrrhotite and pyrite by Fe^{3+} at low pH is generally faster than the rate of oxidation by O_2. Oxidation reactions are also catalyzed by the presence of bacteria such as *Acidithiobacillus ferrooxidans and acidithiobacillus thioxidans* which can substantially increase rates and have been confirmed in many TSF's (McGregor *et al*. (1994), Boorman and Watson (1976), Blowes *et al*. (1995), McGregor et al. (1998b)).

The Fe^{3+} produced through reaction (4), is less soluble under oxidizing conditions and precipitates as many types of secondary iron phases including secondary ferric oxy-hydroxides and ferric oxy-sulphates depending on the pore water composition and pH. Three possible reaction paths for the precipitation of Fe^{3+}, as oxy hydroxides or sulphates are; (i) the formation of amorphous ferric hydroxide ($Fe(OH)_3$) (6), (ii) goethite ($\alpha FeOOH$) and/or lepidocrocite ($\gamma FeOOH$) (7), and (iii) jarosite [$KFe_3(SO_4)_2(OH)_6$] (8) as secondary minerals (McGregor et al. 1998b). The oxidation of primary Fe-sulphides pyrite and pyrrhotite and the formation of common secondary Fe-minerals is presented schematically in Figure 2 (Hammerstrom et al., 2005).

$$Fe^{3+}_{(aq)} + 3H_2O_{(l)} \leftrightarrow Fe(OH)_{3\,(s)} + 3H^+_{(aq)} \quad (6)$$
$$Fe^{3+}_{(aq)} + 2H_2O_{(l)} \leftrightarrow \alpha FeOOH_{(s)} + 3H^+_{(aq)} \quad (7)$$
$$K^+_{(aq)} + 3Fe^{3+}_{(aq)} + 2SO_{4\,(l)} + 6H_2O \leftrightarrow KFe_3(SO_4)_2(OH)_{6\,(s)} + 6H^+_{(aq)} \quad (8)$$

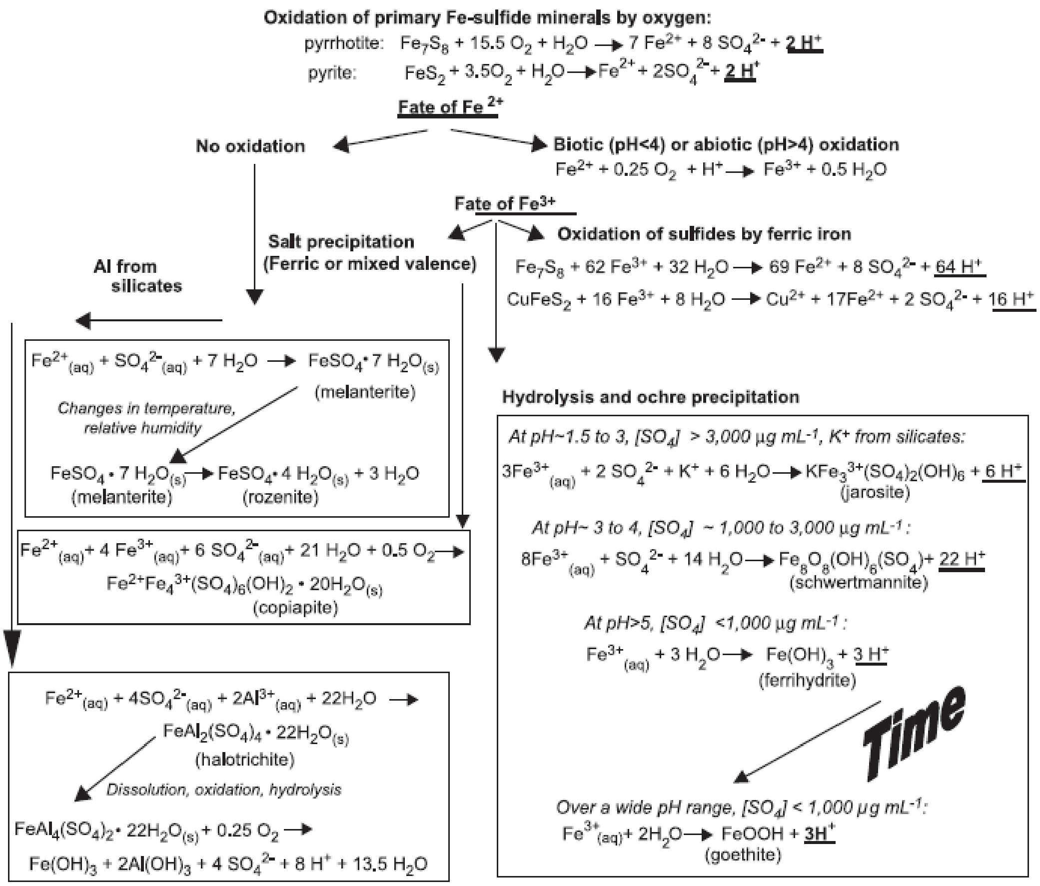

Figure 2. Processes that lead to secondary iron mineral formation from primary Fe-sulphide minerals. (From Hammarstrom et al. 2005)

3 PROCESSES AND LOCATIONS OF SECONDARY IRON MINERALS IN THE TSF

Primary sulphide minerals alter to secondary -sulphate and -oxide forms because their mineral composition is not in equilibrium with the atmosphere and/or the ions in solutions they are in contact with; whether, rain water, porewater, or seepage water. Secondary iron minerals form as reactions between minerals, gases and concentrated waters move towards a new chemical equilibrium.

Secondary iron mineral formation occurs when the concentrations of aqueous Fe ions, as a result of primary sulphide oxidation and dissolution processes, lead to supersaturation. Supersaturation depends on the new chemical equilibrium conditions which are governed by the composition of the minerals and the pH, Eh, temperature, pressure and the chemical composition of the gases and solutions of the continually evolving system. The degree of mineral saturation, is given by the saturation index (SI) which provides the ratio between ion activity product (i.e.. reaction quotient) and the thermodynamic reaction or equilibrium constant. This indicates whether a mineral is supersaturated (SI>1) and should precipitate or whether it is undersaturated (SI<1) allowing further increase in the dissolved ionic concentrations. Supersaturation with respect to secondary iron minerals can result from any of the following (while the converse conditions will encourage mineral dissolution):

- Elevated concentrations beyond equilibrium promoting precipitation (i.e., naturally supersaturated seepage water from upstream sulphide oxidation reactions).
- Precipitation promoted by a change in pH.
- Precipitation promoted by a change in redox potential (Eh) (e.g., altering the relative proportion of Fe^{2+}/Fe^{3+} and S^{2-}/S^{6+}).
- Precipitation promoted by a change in temperature or pressure.
- Precipitation from the mixing of two or more solutions, resulting in increased ions in solution or a change in pH, Eh, temperature in the mixed solution.

Many of these processes can occur simultaneously and/or as a result of one another. For example, precipitation caused by a change in redox potential will often be paired with a change in pH (e.g., when Fe^{2+} oxidizes to Fe^{3+}, hydrogen protons are released increasing the acidity and subsequently lowering the pH).

The accumulation of secondary minerals in tailings stratigraphy, dam fill, filter zones, and other structural and fill materials can be the result from one or many of the above listed mechanisms and each can vary spatially within the TSF or dam.

Conventional TSFs are generally comprised of one or more embankments (dam), which physically contain conventional slurried or thickened tailings. The design and operation of these facilities depends largely on the site characteristics (mountainous vs. flat terrain), foundation conditions, climate (net positive or net negative water balance), and tailings physical properties or geochemistry (split or blended tailings streams). The discussion herein presents a generalized configuration of a TSF and embankment under an active or standing pond with the perimeter embankment designed to convey seepage (a leaky dam).

Secondary iron minerals, from the oxidation of nearly limitless supply of primary sulphides in the tailings of most base metal mines, and other secondary reaction products, progressively form reaction zones along a flow path. These processes result in several geochemical zones, including five (5) general "zones of interest" presented here with respect to Fe sulphide oxidation and secondary mineral formation in TSFs (Figure 3).

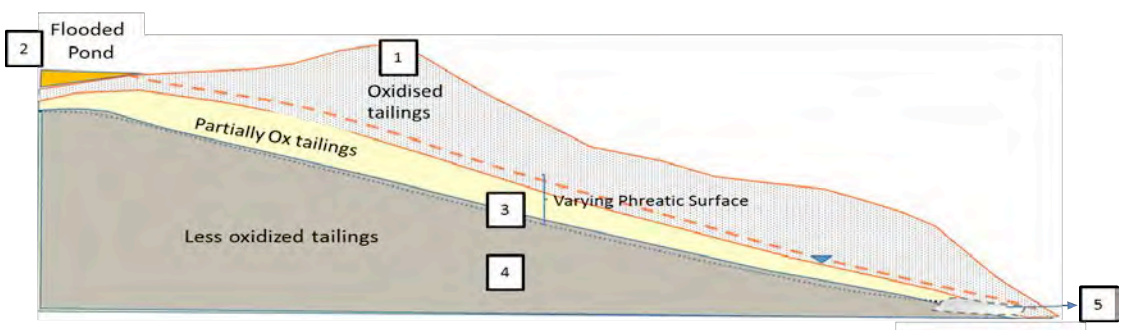

Figure 3. Conceptual TSF and Dam Section Representing Geochemical Zones of Interest (refer to text for details)

3.1 *Zone 1 - Oxidized Surficial Tailings*

Secondary iron minerals form throughout dry exposed surficial tailings, along dam crests and slopes within the visibly oxidized (orange coloured) and transition zone (orange to green-grey) tailings. The downward oxidation front often results in secondary precipitates that form thick coatings, and hardpan layers, and are largely composed of iron-oxyhydroxides and iron-sulphates (depending on the pH at the time of formation). Secondary minerals that form throughout the surficial tailings tend to be amorphous or crystalline and their mineralogy evolves as the oxidation front progresses downward. Hardpan layers often mark the boundary between oxidized and less oxidized tailings and have been shown to limit infiltration.

3.2 *Zone 2 - Flooded Pond Zone*

Pond water, either process water, slurry water or collected contact runoff, partially achieves equilibrium with the atmosphere however, the underlying tailings can be far from equilibrium with the pond water and the atmosphere (Figure 4). Sulphides oxidize beneath the shallow waters and along beaches, and, over time, to a depth of several feet below the pond surface depending on sulphide content and the age of the facility. Sulphide oxidation releases Fe^{3+} and protons (H^+) to the pond water resulting in the pond solutions reaching saturation with respect to Fe^{3+} oxyhydroxides and/or sulphates minerals (depending on the pH of the pond).

Slightly below the base of the pond, in the wet saturated tailings oxygen is less available and the dominant iron species in solution becomes Fe^{2+}. Aqueous Fe^{2+} can act as a further oxidant to the underlying unoxidized sulphides, releasing Fe^{2+} to the pore water and resulting in the oxidative feedback loop. As more Fe^{2+} oxidizes additional secondary Fe^{2+} and Fe^{3+} oxy-hydroxide and sulphate minerals precipitate in pore spaces. The extent of this process will depend on pH and redox conditions which develop in this zone.

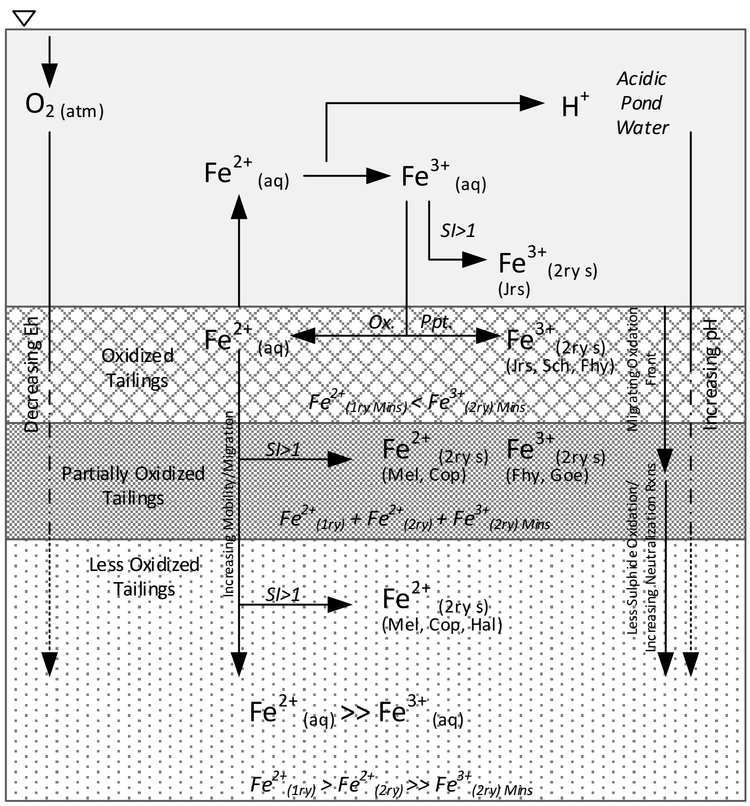

Figure 4. Schematic of the Iron Reaction Cycle in Tailings Ponds and Shallow Pore Water

3.3 *Zone 3 - Shallow Pore Water Zone*

Tailings located near the level of the phreatic surface tend to exist under various stages of oxidation. In this zone, the presence of secondary minerals in pore spaces varies depending on the physiochemical conditions of the shallow pore waters (Figure 5). Shallow pore waters tend to be less acidic and more reducing than pond waters. The shallow pore water is often slightly more acidic and less reducing than deeper pore waters.

Oxygen is less available; however, Fe oxidants and sulphate can be abundant. The dominant iron species in solution ranges between Fe^{2+} and Fe^{3+} and a variety of secondary Fe oxy-hydroxide and sulphate minerals can precipitate in pore spaces. The physiochemical conditions control the species in solution and the type of precipitates which can include both Fe^{2+} and Fe^{3+} phases.

In addition, when shallow pore waters encounter the exposed surficial zone, additional precipitates can form due to changing physiochemical conditions surrounding the pore water, including an increase in oxidant availability (in the form of Fe^{3+} from existing precipitates).

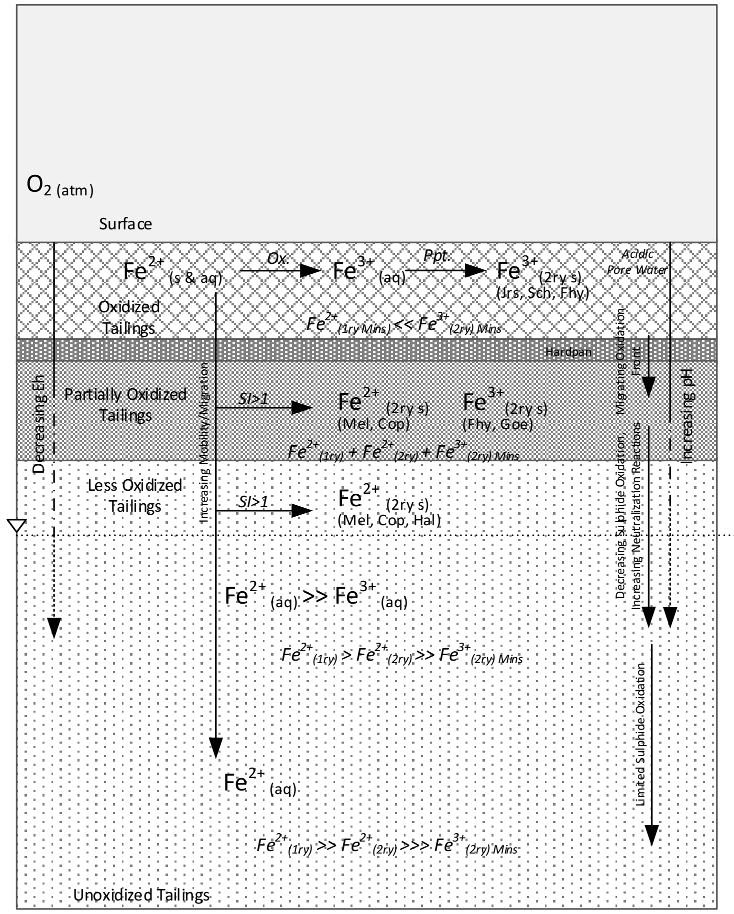

Figure 5. Schematic of Iron Cycle in the Shallow and Deep Pore Water

3.4 *Zone 4 - Deep Pore Water Zone*

Tailings located well below the level of the phreatic surface, pond, and deep below the crest and slopes, tend to be unoxidized and have often undergone less reaction than the shallow zones, under the physiochemical conditions of the deep pore water (Figure 5 and Figure 6). Deep pore waters can be mildly acidic to alkaline and are characterized by less oxidizing/more reducing conditions (low Eh); therefore, aqueous Fe^{2+} becomes the dominant species and oxidant availability is relatively limited.

Secondary Fe^{2+} minerals may form below the phreatic surface from partial sulphide oxidation depending on the surrounding physiochemical conditions however, the presence of secondary minerals is often less abundant well below the phreatic surface.

The increased pH, relative to shallower pore water and pond waters is facilitated by less sulphide oxidation, less Fe oxidation products and acidity, and subsequent mineral buffering.

3.5 *Zone 5 - Emerging Seep Zone*

As seepage reaches the dam toes, or drains in a TSF, the physiochemical conditions change. The increased availability of oxidants (both O_2 and Fe^{3+}) and increased Eh cause aqueous Fe^{2+} in porewater to oxidize and form secondary Fe^{3+} oxy-hydroxides and/or sulphates along the emerging seepage pathway (Figure 6):

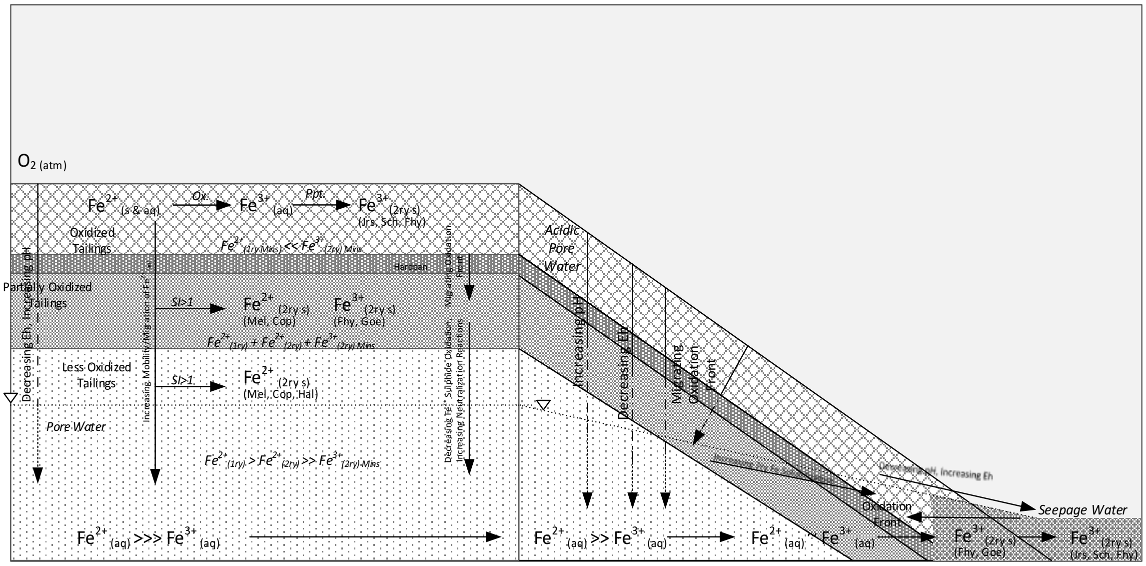

Figure 6. Schematic of Iron Cycle along the Seepage Flow Pathway

The formation of secondary precipitates in pore spaces at the toe of the dam is governed by the change in the physiochemical conditions and the shift in thermodynamic equilibrium. Emerging seepage that migrated from shallow pore waters is characterized by mildly acidic pH, moderate to low oxidation-reduction potential (Eh) and high metal(oid) content. Secondary minerals form within the pore spaces along the seepage emergence point. The formation of secondary mineral precipitates clogs seepage flow paths and reduces the average flow velocity (or increases flow path lengths) and consequently increases time for reactions to occur.

Emerging seepage that migrated from below the phreatic surface, on the other hand, is generally characterized by near-neutral pH, low oxidation-reduction potential, and higher metal(loid) content however, the formation of secondary minerals tends to occur further downstream when oxidation reactions resume upon seepage emergence. The location where secondary minerals precipitate is therefore governed by the source of the seepage, its respective composition, the flow rate and pathway as well as time (both time since original placement and subject to factors such as changing hydrology and seasonality). Secondary mineral formation along the seepage emergence points can partially or completely blind pore spaces altering the permeability of the material.

3.6 *Summary*

The geochemical interactions occurring in the shallow pore water (Zone 3) and emerging seep zones (Zone 5) are important when considering the physical and chemical changes that are occurring to the materials. Geochemical reactions can alter the composition and partially or completely blind pore spaces subsequently increasing water levels in TSFs.

The configuration of dam underdrains and toe drains (if any) are critical to the rate and mitigation of secondary precipitates in Zone 5. The key factors controlling precipitate accumulation in the critical zones are:

- The composition of the seepage and concentrations of key parameters such as iron and sulphate
- The rate of change in equilibrium from along the porewater seepage flow path.

- The residence time of porewater and seepage flow rates, to allow reactions to proceed.
- The presence of oxygen, Fe or other oxidants and microorganisms.
- Changes in temperature, pressure, pH, and Eh.

4 SUMMARY AND CONCLUSIONS

The weathering of primary minerals and the formation of secondary minerals is common in tailings storage facilities; however, the changing chemical and mineralogical properties of the materials is often not explicitly considered by design engineers during TSF design and management. Mineral alteration occurs at the mineral-fluid interface in materials used in as dam embankment fill, drains, and other structural components as well as within the stored tailings in the TSF. Physical and chemical changes occur from geochemical processes that, if left unmitigated, can result in structural deterioration of the dam materials over time. Primary sulphide oxidation, dissolution and precipitation, influences dam stability as secondary minerals vary in their chemical composition, density, grain size, and, therefore strength and porosity. Changes in the strength and porosity from these geochemical reactions are important factors that should be considered in dam design.

An understanding of the primary and secondary minerals and contacting waters should be included to assess the geochemical interactions that may be occurring throughout the TSF currently and in the future. Field monitoring and laboratory testing is imperative to evaluate the changes in material properties over time and to monitor the potential chemical indicators of further weathering or secondary mineral formation. Geochemical modelling can be used as a tool to model current and potential future conditions as mitigation strategies are assessed.

The evaluation of the interaction of dam construction materials with tailings porewater and seepage is not currently evaluated in today's state of practice, however, the long-term nature of tailings management has resulted in deterioration of tailings water drainage systems. The importance of evaluating these geochemical systems at the design stage as well as throughout the monitoring stages of ageing facilities will become increasing important as greater scrutiny is placed on the integrity of tailings facilities.

These systems are dynamic and need to be understood to be able to plan for changes in the TSF, such as at closure, with changing pond elevations, or facility expansion and stabilization (such as dam raises and buttresses).

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Post-failure runout analysis from tailings dams using viscosity bifurcation rheology

Komeil Kazemi & Paul Simms
Carleton University, Ottawa, ON, Canada

ABSTRACT: Determining failure consequences of tailings dam breaches is of interest to the mining community in the wake of recent high profile failures, and the consequent efforts in different jurisdictions to improve tailings dam design and regulation. Determining failure consequence is important as it gives direct incentives to mine operators to implement safer tailings management practices. Therefore, appropriate methods for predicting runout must consider i) the state of the tailings in the impoundment at the moment of failure, and ii) use a realistic constitutive relationship for the rheology. This paper addresses the second of these points. Numerical simulations of tilings flows are presented, using a viscosity bifurcation rheology model. Viscosity bifurcation models track how shear and ageing can modify rheological parameters over short time scales. This is shown to be very pertinent to dam collapse problems, where the very high rate of shear will substantially degrade the rheology, leading to much longer runouts and therefore greater consequences of failure than predicted using conventional rheology.

1 INTRODUCTION

1.1 *Tailings dam failures*

Tailings dams have failed for multiple reasons, including dam foundation problems, poor water control leading to dam overtopping or excess pore water pressure development, or earthquake loading. These failures and their consequences are documented in a number of sources, including the 2001 report of the International Commission On Large Dams (ICOLD 2001), a review by Rico et al. (2008) that also attempts to correlate runout lengths to tailings impoundment geometry, and online resources (e.g. www.tailingsinfo.com). Cases documented in the research literature include the Merriespruit failure (Fourie and Tshabalala 2005), various failures in Chile (Villavicencio et al. 2014), and the 1978 Isu-Ohshima earthquake (Marcuson et al. 1979, Bryne and Said-Kharbasi 2013). Jeyapalan et al. (1983) reviewed a number of cases in terms of run-out characteristics. Failures are more common for upstream tailings structures, where the tailings themselves serve as the foundation for the upper portion of the dam (Martin and McRoberts (2002).

1.2 *Factors affecting run-out*

Factors identified in the literature contributing to run-out include: Impoundment height and volume (Rico et al. 2008), topography, rheology, the type of tailings, and the volume of water stored in the tailings impoundment: Jeyapalan et al. (1983) proposed that tailings flows could be either modeled as a Bingham plastic, or as a turbulent sediment-entrained water flow for finer grained tailings: subsequent authors have proposed either conventional yield stress measurements or to use residual or steady state strength for the Bingham yield stress (Seddon 2010). Many practitioners believe that water on top of the tailings acts as a driving or eroding agent contributing to very long runouts; however, suspension of tailings in entrained water flow is not seen at all sites with long run-outs. For example, a height gauges in a rivers inundated with flow from the los Frailes dam 11 km from the site showed a double peak: the first due to the released pond water, and a later peak due to the tailings flow itself (Rico et al. 2008), which suggests the pond water drained off and ran ahead of the tailings themselves.

1.3 *Modelling of runouts*

Modelling of tailings flows after dam break have usually involved two separate calculations: i) determination of the volume of tailings fluidized, and ii) assuming a certain rheology, calculating the subsequent flow of tailings using a flow model. As these analyses are typically done to examine failure, i) is usually known. The tailings are then propagated using an assumed rheology, usually Bingham, most commonly using finite volume (Jeyapalan et al. 1983), or meshless methods such as SPH (Pastor et al. 2002, 2014). Depth-averaged analyses are often used for tractability of problems with complex 3D topography. Most analyses back calculate the assumed rheology (often the yield stress and a viscosity) in order to force a fit to the field data, which itself is often limited. Jeyapalan et al. (1983), for instance, verified their model only using small flume tests on idealized viscous fluids (not tailings). Other methods or recent innovations in the larger research domain of geohazards or solid–fluid transitions that are likely relevant to tailings runout analysis, include constitutive relations that can describe the transition from elasto-plastic to rheological behaviour (Prime et al. 2014), the inclusion if pore-water pressure dissipation in run-out calculations (Pastor et al. 2014).

Analysis of post-failure runout, however, would be most useful, not only to better characterize risk, but to help quantity advantages of various tailings deposition strategies. To do this, runout predictive methods must incorporate i) the role of state of the tailings prior to failure, ii) use appropriate rheology constitutive models. While i) might be addressed using something like the approaches of Prime et al. (2014) or Pastor et al. (2014), the objective of this paper is to consider the effects of using a more realistic rheology constitutive model.

1.4 *Viscosity bifurcation models and their applicability to tailings rheology*

While short timescale and small volume tests, in the order of seconds, can be characterized using Bingham rheology, the same is not necessarily true for flows that undergo a large degree of shearing. Recent investigations of the rheology of both hard rock and oil sands tailings (Mizani and Simms 2016, Mizani et al. 2017), have shown that multiple tailings types exhibit shearing and ageing effects on their rheology. These effects have been observed for pure clay suspensions; and have been explained using viscosity bifurcation models (Coussot et al. 2002a,b; Hewitt and Balmforth 2013). These models simulate viscosity variation by correlating viscosity to a structure parameter, λ, which is affected by both destruction due to shear, and re-structuring due to ageing. Depending on the initial state of structure, the viscosity will either increase or decrease at a given shear rate or shear stress. Yield stress behaviour is manifested when shear stress drops below a critical value, such that the viscosity rapidly increases as ageing dominates over shearing. Variable yield stress, hysteresis, jamming and avalanche behaviours of various non-Newtonian fluids have been explained using these viscosity bifurcation models (Alexandrou et. al 2009; Bonn et. al 2004; Moller et.al 2009; Hewitt and Balmforth 2013).

This paper proceeds by implementing a particular viscosity model, the Cousot model, into two numerical methods – a smooth particle hydrodynamics code (SPH) and an implicit finite element (FEM) code with particle tracking. A number of 2D and 3D simulations are performed to illustrate the consequences of the a viscosity bifurcation model on tailings flow rheology.

2 THEORY

2.1 *The Coussot model*

Coussot's model (Coussot et al. 2002) describes competition between aging and jamming by a state variable λ evolves in time and governed by the following differential equation:

$$\frac{d\lambda}{dt} = \frac{1}{T} - \propto \dot{\gamma}\lambda \qquad (1)$$

where α and T are material constants and $\dot{\gamma}$ is the strain rate. T is the characteristic time of evolution of the structure. Coussot then considers the instantaneous viscosity to be a function of the instantaneous state of the material, and suggests $\mu = \mu_0 (1+\lambda^n)$, where μ is the viscosity, μ_0 is the fully sheared viscosity where structure is broken down (λ=0), and n is a parameter.

Therefore, the structure and viscosity evolve as a function of ageing and shearing. Yield stress behaviour manifests when ageing is dominant over shearing. Figure 1 shows different predictions of Equation 1 with identical material parameters but different constant shear rates. The full sheared viscosity is 2 PaS, the residual value of viscosity reached for the higher shear rates. Below a particular shear rate or shear stress, ageing dominates and the material comes to rest. The graph is also dependent on the initial value of λ.

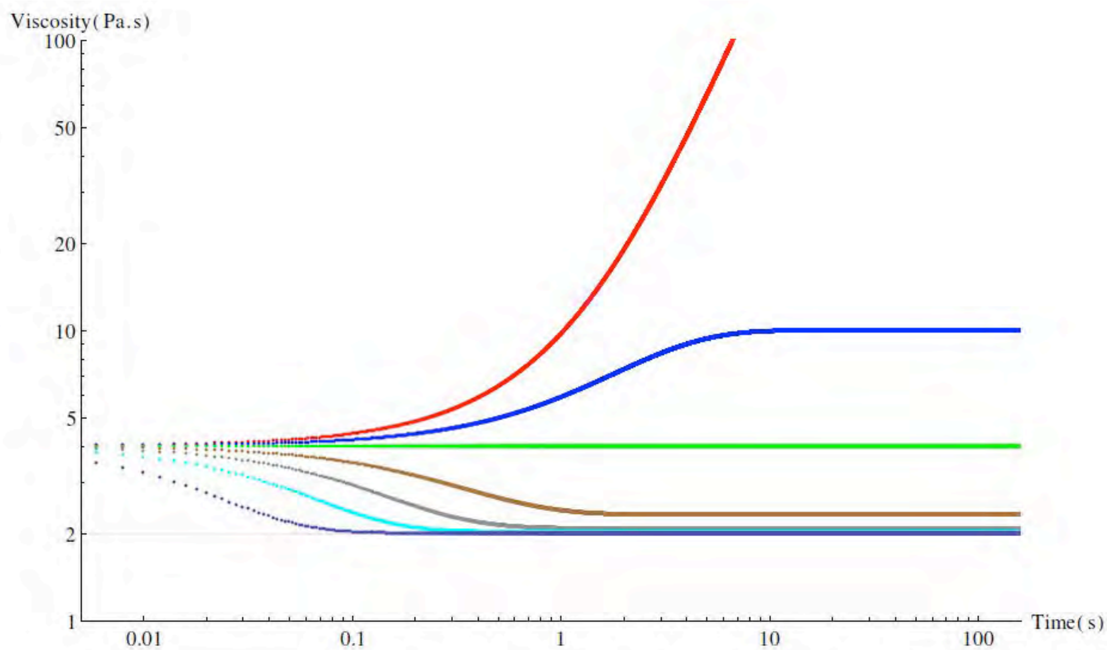


Figure 1. Viscosity as a function of time and strain rate. Strain rate levels are 0.005, 0.1, 0.2, 0.5, 1, 2, and 5. T=1, α=5, n=2 , and μ_0=2 PaS. Initial λ is 1.

The manifestation of this behaviour is that a the final shape or extent of a tailings flow will depend on time and initial state.

2.2 *Numerical Simulations*

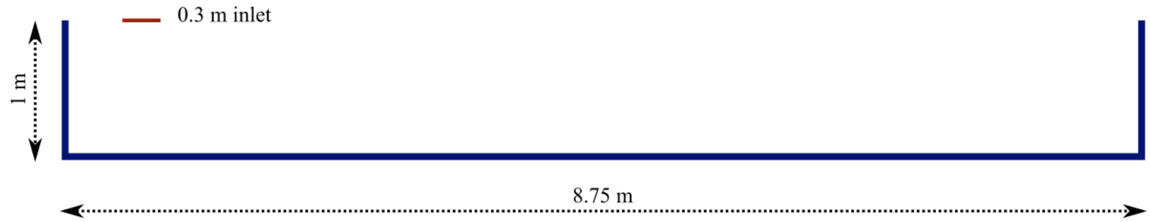
All the results shown in this paper, with the exception of the large 3D tailings dam simulations, are performed using SPH. In the SPH method a domain is divided into a finite number of particles. These particles carry properties such as mass, physical properties and variables such as velocity, pressure and density.

One aspect of the conventional SPH method is that it is slightly compressible. An explicit scheme is usually used for the time integration which dictates small time step sizes. Time steps size is usually governed by an articial speed of sound. To overcome this weakness, different numerical techniques can be used to solve the linear system of equations implicitly as well as make the continuum divergence free. In this research, a parallel computing implementation supported by a GPU is used to speed up the SPH solver.

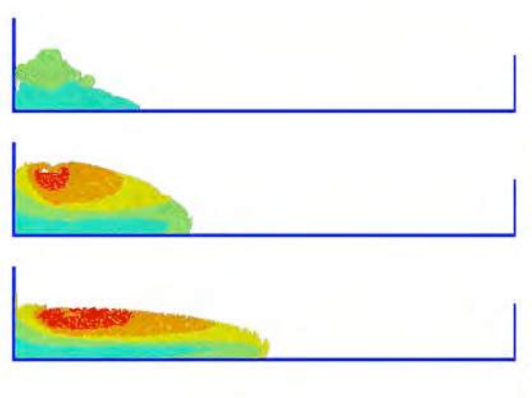
The large 3D tailings dam simulations are generated using an implicit FEM solver with particle tracking. This particular FEM technique is more suited to complex geometries where the flow is relatively slow. The SPH technique is more efficient for simulations involvingrelatively higerh velocities, such as near the opeing of a pipe during tailings deposition. Both codes are developed by the research group of the first author who work on non-Newonian flow of weld materials.

3 RESULTS

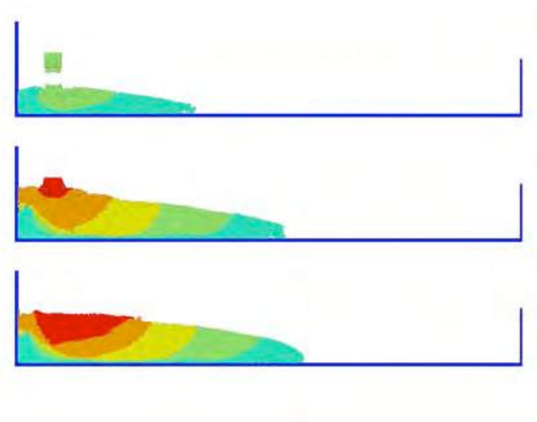
3.1 *Simulation of 2D tailings flows deposited from an inlet*



(a)



(b)



(c)

Figure 2. Simulations of deposition in a 2.75 m long flume with (b) 0.2 m/s inlet speed and c) 0.4 m/s. Screenshots taken for equal volume of tailings in the flume. Colour denotes time of deposition.

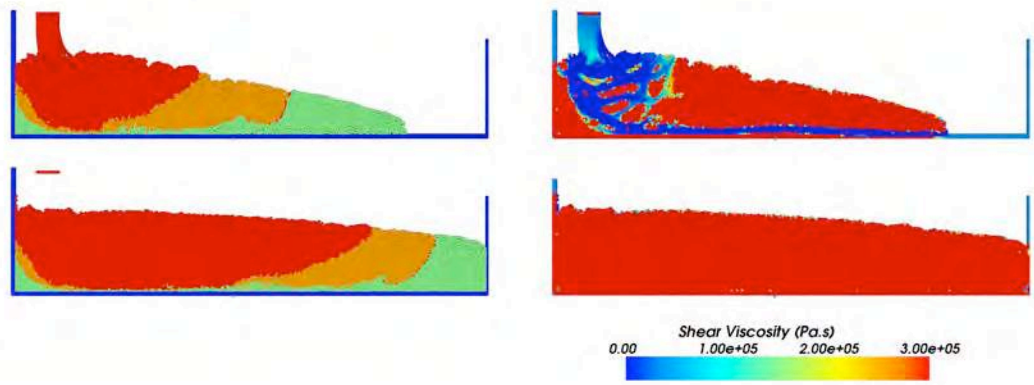


Figure 3. Continuous deposition in a 2.5 m long flume, two different times (30s and 50s). Viscosity is shown on the right

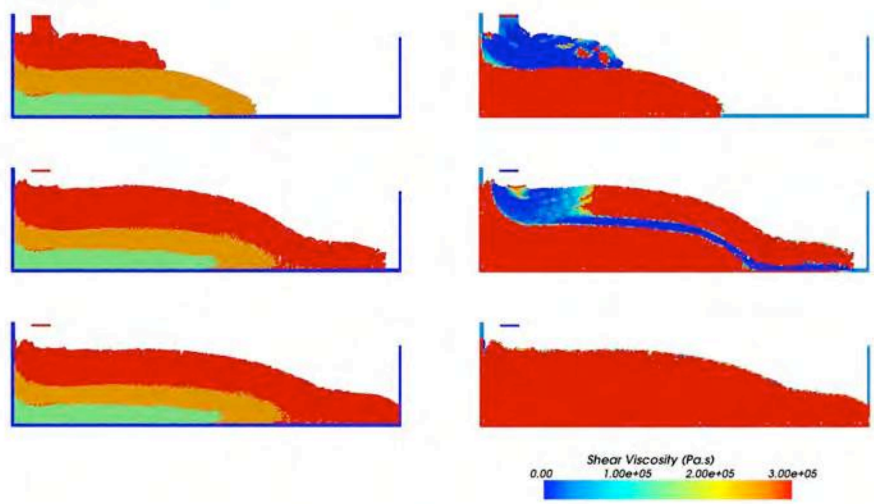


Figure 4. Deoposition is paused for 6 s twice, tailings deposited up to the first pause are coloured green, then orange up to the second pause. Screenshots are after the last pause. Viscosity on the right.

The influence of time can be seen on the tailings geometry. In Figure 2, screenshots shown a change in the arrangement of the tailings. In the fast deposition, the older tailings are pushed by the fresh tailings to the right. In the slow depositions, the older tailings accumulate higher viscosity through ageing, and fresher tailings tend to flow over the older tailings. This is more dramatically seen in Figures 3 and 4, where identical volumes of tailings with identical rheology are deposited contuninuously (Figure 3) or by inserting two 6 second pauses (Figure 4). In Figure 3, the older tailings are pushed to the right, and a shear band of low viscosity is maintained on the bottom. In Figure 4, the pause allows for fast ageing of the previous deposit under low or absent shear, and therefore the developed high visocosity prevents remobilization of the older tailings. The band of low viscosity develops between the fresh tailings and the older tailings, resulting in a very different shape than Figure 3.

3.2 *Dambreaks at bench scale and field scale*

The first result is an analysis of dam break experiments conducted on a thixotropic fluid by Chanson et al. (2006). The rest time of the flud is varied before a dam is raised in a tilted flume (Figure 5). This results in a different initial structure, λ_0, at the time of raising of the gate. For this scale of test, the initial state has a strong influence on the degree of runout (Figure 6). For low initial λ, most of the material is mobilized, the viscosity is degraded quickly to near the fully sheared value, and a substantial runout occurs. For high initial values of λ, a narrow shear band develops, and only a slump failure occurs.

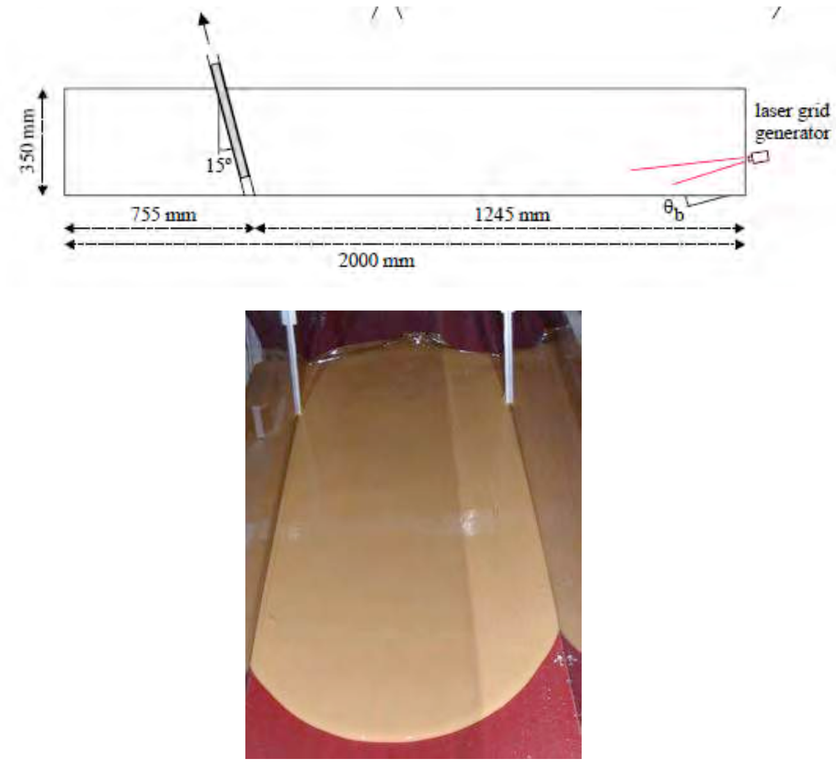
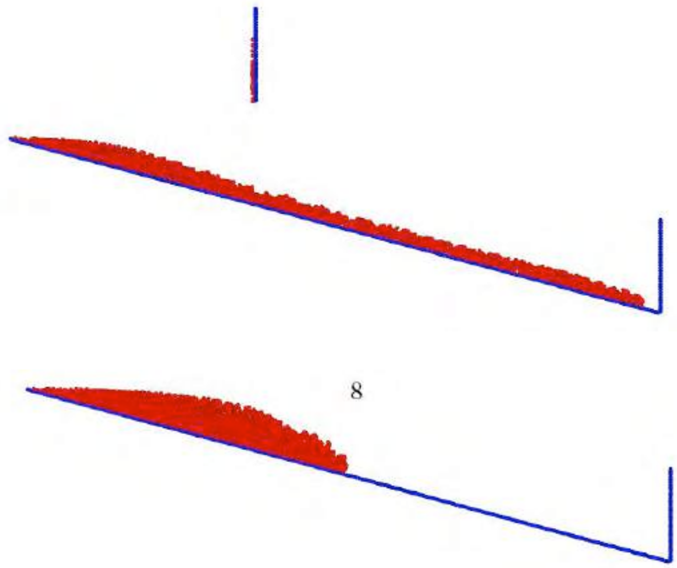


Figure 5. Experimental setup of bench scale dam break tests of a thixotropic fluid (Chanson et al. 2006).



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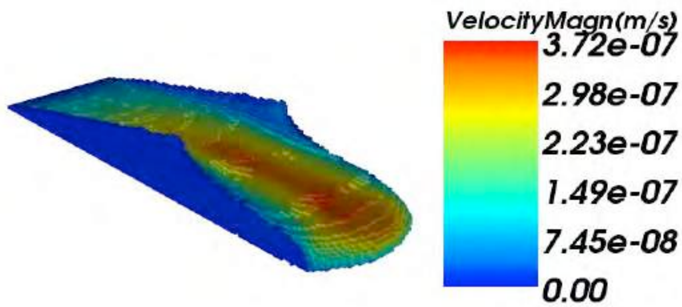

Figure 6. 2D and 3D simulations of benchscale dambreak experiment of Chanson et al. (2006); Flow Bead shape geometry at stoppage, t=30 s for $\lambda_0 = 1, 5$. Other parameters, $\mu_0 = 2$Pa s, T= 1, $\alpha = 1$, n = 5, in both simulations. Bottom two figures for $\lambda_0 = 5$.

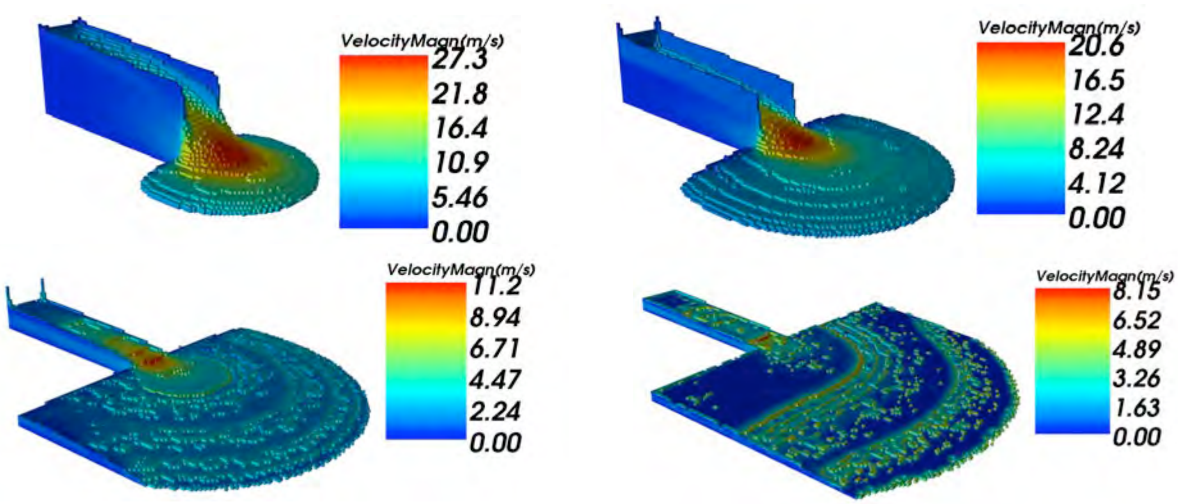

Figure 7. FEM simulation of dam break at 50 m wide by 100 m high by 200 m long impoundment, using same parameters as in Figure 6, except that no change is observed for a wide range (up to 1000) of λ_0. Last snapshot shows flows at about 500 m from the break after 60 s.

The next result is a simulation of a large dam break (100 m high 50 m wide, with a reservoir length of 200m) onto a horizontal plance, using the same parameters as in Figure 6. In the case shown λ_0 is increased up to 1000, but with no effect on the deposition, as all the simulated material is quickly reduced to λ=0 by the high shear rates induced by the high energy failure. The simulation is shown only up to 1 minute, the material has already propogated 500 m on a flat surface and the material is still moving in places at greater than 1 m/s. The high energy of the dam failure has reduced the viscosity of almost all the simulated tailings to the fully-sheared value, and the shearing mechanism dominates over ageing at least until this time.

4 DISCUSSION

The lessons of the comparison between the results at the benchscale and the field scale simulation are several. First, if one were to attempt to deduce a conventional rheology from interpretation of the bench scale flume tests, one sees that this become meaningless if the material is substantially thixotropic, as any back calculated Bingam rheology would change with the rate of deposition, or the resting time of the material before the dambreak. Second, the the application of such conventional rheology would be limited in the field application. In the field application, the tailings continue to flow until the shearing term decreases to below the ageing term in Equation 1. Though one might be tempted to see a critical stress in the term $\lambda\,\gamma$ (effective viscosity x shear rate = a stress), it does not work the same way as a yield stress, due to the varying viscosity in the approach to that value. Mizani and Simms (2016) showed that the manifested yield stress in a gold tailings depends on the rate of approach to that value.

The λ parameter in the Coussot model is somewhat problematic as it is unbounded, and there is yet no physical interpretation of what it means in terms of real tailings parameters, such as density or shear strength. Other viscosity bifurcation models, such as the Heweitt model (used to interpret Oil ands tailings rheology by Mizani et a. 2017), bound the structure parameter, and link it to measurable yield stress values measured using specific imposed stress histories. Of course, all viscosity bifurcation models are only flow models, and do not say anything about the initial failure of the tailings. Linking rheology models with conventional soil mechanics models, as stated in the introduction, is another important aspect of developing increasing useful runout analysis tools.

5 SUMMARY AND CONCLUSIONS

1. Viscosity bifurcation models have been previously shown to explain rheology of both a gold tailings and an oil sands tailings.
2. A viscosity bifurcation model (the Coussot model) is implemented into two non-Newtonian flow codes, one SPH, the other an implicit FEM solver wth particle tracking.
3. At the benchscale, the Coussot models exhibits time-dependent behaviour observed in many experiments, which would not be capturated by conventional rheooligcal constitutive models.
4. In the simulation of the large dambreak, the rheology of the simulated materials is reduced to the fuly sheared state, and the material essentially flows like water, regardless of relatively large values of the initial structure parameter.
5. Future work should focus on relating the structure parameter to measurable characteristics of tailings. Future work should also focus on integrating conventional elasto-plastic failure models with rheometry, so as to better quantify the influence of the actual state of tailings in an impoundment on post-failure runout risks

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Yield Shear Strength Ratio for Liquefaction Triggering Analysis of Tailings Dams

I.A. Contreras & B.E. Zwissler
Barr Engineering Co., Minneapolis, MN, USA

ABSTRACT: Liquefaction is one of the most common causes of catastrophic failure of tailings dams, challenging engineers to assess liquefaction potential and develop designs to account for both static and dynamic triggers. Liquefaction analysis includes the evaluation of liquefaction triggering, which typically incorporates yield shear strength ratio, $s_{u\ (\mathrm{yield})}/\sigma'_{\mathrm{vo}}$. A common procedure for estimating $s_{u\ (\mathrm{yield})}/\sigma'_{\mathrm{vo}}$ uses cone penetration test (CPT) data in the form of normalized cone penetration resistance, $q_{c1,}$ or standard penetration test (SPT) data in the form of normalized blow-count, $(N_1)_{60}$. This paper presents a preliminary relationship for the estimation of $s_{u\ (\mathrm{yield})}/\sigma'_{\mathrm{vo}}$ from CPT as a function of the equivalent clean sand cone penetration resistance, $Q_{tn,cs}$. This is believed to be a better parameter than q_{c1} when assessing the strength of fine-grained materials (i.e., fine tailings) because it accounts for tip resistance, sleeve friction, and pore-water pressure. The relationship was developed by combining $s_{u\ (\mathrm{yield})}/\sigma'_{\mathrm{vo}}$ from the case history database developed by Olson and Stark (2003) and the corresponding $Q_{tn,cs}$ reported by Robertson (2010). The relationship also includes data collected by the authors at a tailings basin using CPT (for $Q_{tn,cs}$) and vane shear tests (FVT) (for $s_{u\ (\mathrm{yield})}/\sigma'_{\mathrm{vo}}$) conducted in adjacent soundings at the same depths. The preliminary relationship shows relatively good agreement between the $s_{u\ (\mathrm{yield})}/\sigma'_{\mathrm{vo}}$ from the case history database and the authors' fine tailings dataset, but requires a more robust dataset covering a wider range of $Q_{tn,cs}$ and soil behavior types (SBT) to enhance the relationship and better establish the trend between $Q_{tn,cs}$ and $s_{u\ (\mathrm{yield})}/\sigma'_{\mathrm{vo}}$.

1 INTRODUCTION

Because many mine tailings impoundments involve structures constructed with or on top of saturated soils deposited in a loose condition, soil liquefaction is a major design concern. Liquefaction occurs in undrained conditions and is induced by static or dynamic loading. Liquefaction is characterized by a sudden decrease in shear strength from the yield strength to the steady-state strength, which can be substantially lower. The loss of shear strength during liquefaction is so significant that the soil temporarily acts like a thick liquid (Terzaghi et al. 1996). At mine tailings impoundments, the consequences of liquefaction can include flow slides of sloping ground, lateral displacement of dams and retaining structures, ground rupture, formation of sand boils, and catastrophic failure of tailings dams.

Even though fine tailings contain significant amounts of silt- and clay-size particles, they are often susceptible to liquefaction because they typically contain low plasticity or non-plastic solids. The potential for fine tailings to liquefy in response to triggering events is related to the fact that these materials are often hydraulically deposited, come to equilibrium under loose conditions, and tend to remain continually saturated. Furthermore, liquefaction is typically observed in young, natural soil deposits like fine tailings, which are a waste material from mining processing operations (i.e., deposits of a fairly young geologic age). The loose condition resulting from the deposition method and the young age of fine tailings generally results in contractive behavior during undrained shearing.

Liquefaction analysis of slopes, embankments, and sloping foundations represents a challenge for engineers because of the complex nature of this assessment. In general, the analysis involves three steps: (1) a liquefaction susceptibility analysis, (2) a liquefaction triggering analysis, and (3) a post-triggering/flow failure stability analysis.

In the first step (liquefaction susceptibility analysis), engineers determine whether the material is contractive or dilative. Contractive material is susceptible to liquefaction and strain softening, while dilative material is not. Researchers have developed several relationships using CPT and SPT data with laboratory test results that distinguish between contractive and dilative sandy materials.

If the material is found to be contractive, the second step (liquefaction triggering analysis) is performed to determine if liquefaction will be triggered. This is done by determining whether the anticipated static shear stress or seismic stresses will exceed the yield shear strength of the contractive soils. In the case of seismic triggering, this step includes a site-response analysis that allows calculation of the factor of safety against triggering. This analysis requires the yield shear strength ratio $s_{u\ (yield)}/\sigma'_{vo}$.

If it is determined that liquefaction will be triggered, the third step (post-triggering/flow failure stability analysis) is conducted to determine if the static shear forces are greater than the available shear resistance. If the factor of safety against flow failure, FS_{FLOW}, is less than or equal to unity, flow failure is predicted to occur once triggered. This analysis requires the liquefied shear strength ratio $s_{u\ (liq)}/\sigma'_{vo}$.

This paper presents a relationship to estimate the yield shear strength ratio $s_{u\ (yield)}/\sigma'_{vo}$ for use in liquefaction triggering analysis. The proposed relationship uses CPT data ($Q_{tn,cs}$) to estimate the yield shear strength ratio. The relationship was developed using two data sources: (a) the case history database published by Olson and Stark (2003) and augmented by Robertson (2010) and (b) data from adjacent CPT and FVT soundings in fine tailings from basins where the authors have worked. For the database of case histories, yield shear strengths were back-calculated from post-failure analyses and $Q_{tn,cs}$ were measured from post-failure CPT soundings. For the fine tailings data collected by the authors, the yield shear strengths were measured from FVT soundings; $Q_{tn,cs}$ were measured from adjacent CPT soundings at the same depths.

2 TRIGGERING LIQUEFACATION IN TAILINGS BASINS

Figure 1 illustrates the behavior of saturated, contractive, sandy soils during undrained shearing or loading (Olson and Stark 2003), applicable to fine tailings. Yield shear strength, $s_{u\ (yield)}$, is the peak shear strength available to the soil during undrained loading (Terzaghi et al. 1996) and is illustrated as point B in Figure 1. In contrast, steady-state (or liquefied) shear strength, $s_{u\ (liq)}$, illustrated as point C in Figure 1, becomes the shear strength available to the soil during and after undrained strain softening (or liquefaction) has occurred. Liquefaction can be triggered by both static and dynamic loads, as well as by deformation under static shear stress that exceeds the liquefied shear strength.

Liquefaction triggered by static or monotonic loading (illustrated by the yield shear strength envelope connecting Point A and Point B to Point C in Fig. 1) can occur at tailings impoundments during activities such as tailings deposition on a slope or upstream dam construction. For static liquefaction triggering to occur, the shear stress in the saturated fine tailings must exceed Point B during undrained conditions. Once triggered, the loose structure of the soil particles yield and collapse, causing the loss of shear strength with additional shear strain, illustrated as the curve between Point B and C. If enough shear strain is experienced by the soil to reach Point C, liquefied shear strength (Point C) governs the fine tailings.

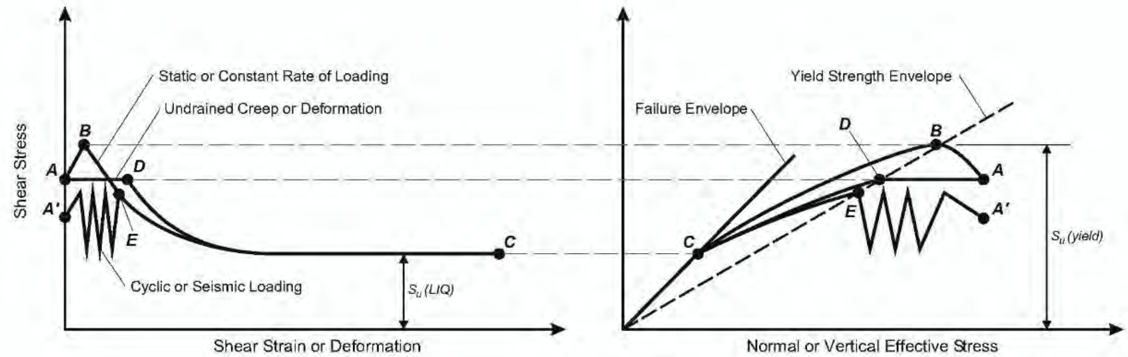

Figure 1. Undrained response of saturated contractive sandy soils, including fine tailings (after Olson and Stark 2003)

Liquefaction triggered by deformation under static shear loading (illustrated by the yield shear strength envelope connecting Point A and Point D to Point C in Fig. 1) can also occur at tailings impoundments during events such as dam foundation deformation or erosion at the toe of a slope. For liquefaction to be triggered by static shear deformation, the static shear strain must exceed that at Point D and the effective stress must drop below that at Point D during undrained conditions. Once triggered, just as with liquefaction triggered by monotonic loading, the loosely packed soil particles yield and collapse, causing the loss of shear strength with additional shear strain, illustrated as the curve between Point D and C. If enough shear strain is experienced by the soil to reach Point C, liquefied shear strength (Point C) then governs the fine tailings.

Liquefaction triggered by seismic or dynamic loading (illustrated by the yield shear strength envelope connecting Point A and Point E to Point C in Fig. 1) can occur at tailings impoundments during earthquakes or be caused by vibrations from construction activities. For dynamic liquefaction to occur, the duration and intensity of the seismic/dynamic load must cause enough excess pore-water pressure within the fine tailings that the effective stress drops below that at Point E during undrained conditions. Once triggered, just as with static and deformation-induced liquefaction, the loosely packed soil particles yield and collapse, causing the loss of shear strength with additional shear strain, illustrated as the curve between Point E and C. If enough shear strain is experienced by the fine tailings to reach Point C, liquefied shear strength (Point C) governs.

Most liquefaction analysis discussions in literature are associated with liquefaction of sandy deposits during seismic events. However, mine tailings, which are typically fine-grained with low plasticity, have undergone more static than seismic liquefaction events (Davies et al. 1998). Tailings basins should be operated and constructed such that changes in load within the fine tailings are slow enough to prevent significant generation of excess pore-water pressures. In this case, the tailings are sheared under drained conditions during normal operation, avoiding static liquefaction. However, circumstances can occur at tailings basins that involve rapid changes in load or stress changes that can lead to localized undrained loading conditions, triggering static liquefaction. These shear stress changes can be caused by factors such as foundation deformation, erosion at the toe of a slope, change in piezometric head, or seismic shaking. Table 1 summarizes some typical triggering mechanisms associated with upstream tailings dams. The first four mechanisms are associated with monotonic (static) loading or deformation-induced liquefaction triggering. The fifth mechanism, acceleration and/or vibration, is associated with seismically or dynamically induced liquefaction triggering.

Liquefaction triggering analysis is a particularly important step in the assessment of tailings basins that uses the upstream construction method which has portions of the embankment dam founded on top of liquefiable materials (Contreras et al. 2016). As a result, there is a static shear stress applied to the fine tailings prior to a liquefaction triggering event. Table 1 also shows that some triggering events can be very small. This is why some research (Silvis and de Groot 1995, Robertson 2010) suggests that triggering should always be assumed if the soils are susceptible to strength loss. Details on conducting a liquefaction triggering analysis are discussed in the following section.

Table 1. Triggering Mechanisms for Liquefaction Failures of Upstream Tailings Dams (after Martin and McRoberts 1999)

Triggering Mechanism	Potential Cause
Over-steepening at toe	- Erosion (intense stormwater runoff, pipeline break causing wash-out)
	- Construction activities or "housekeeping" (excavation)
Overloading of slope/foundation	- Rapid rate of impoundment raising
	- Steepening of slope near crest
	- Construction activities at crest
Changes in pore pressures	- Seepage breakout on face of dam
	- Deterioration in performance of under drainage measures
	- Concentrated tailings discharge at one location for extended period
	- Accelerated rate of construction
	- Foundation or embankment movement
	- Intense rainstorms
	- Increased pond levels
Overtopping of dam	- Severe stormwater runoff
	- Failure of diversion dams/ditches
	- Blockage and failure of spillways/decants
	- Embankment settlement/deformation and loss of freeboard
Acceleration/vibration	- Earthquakes
	- Construction traffic
	- Blasting

3 LIQUEFACATION TRIGGERING ANALYSIS

There are well-established procedures for liquefaction triggering analysis of level ground, including Seed et al. (1985) and Youd et al. (2001). However, there are few procedures to evaluate the triggering of liquefaction in sloping ground, including Poulos et al. (1985a), Seed and Harder (1990), and most recently Olson and Stark (2003). In the authors' opinion, Olson and Stark (2003) provide the most appropriate procedure for liquefaction triggering analysis of fine tailings on sloping ground, which is the basis of the liquefaction analysis discussion in this paper.

The Olson and Stark (2003) liquefaction triggering analysis procedure uses the yield undrained shear strength ratio, $s_{u\ (yield)}/\sigma'_{vo}$, to evaluate the triggering of liquefaction in sloping ground subjected to static shear stress. The analysis procedure determines whether the combined static, seismic, and/or other shear stresses exceed the yield shear strength of the contractive material. This allows for the calculation of the factor of safety against liquefaction triggering. In this procedure, the yield shear strength ratio, $s_{u\ (yield)}/\sigma'_{vo}$, is an important parameter required to assess liquefaction triggering. Olson and Stark (2003) proposed relationships to estimate this ratio, developed by back-analysis of case histories with SPT and CPT data available.

4 YIELD UNDRAINED SHEAR STRENGTH RATIO, $S_{U\ (YIELD)}/\sigma'_{VO}$

The relationship proposed by Olson and Stark (2003) to estimate the yield shear strength ratio, $s_{u\ (liq)}/\sigma'_{vo}$, uses the normalized cone tip resistance, q_{c1}. This relationship was developed by back-analyzing 33 case histories of liquefaction flow failures and correlating the yield shear strength ratio with the normalized cone tip resistance. Normalized cone tip resistance, q_{c1}, corrects CPT tip resistance for effective overburden stress, but does not use any correction for soil type, fines content, sleeve friction, or pore-water pressure. Olson and Stark (2003) advised that the relationship using CPT should be corrected for unequal end area effects through use of the corrected cone tip resistance, q_t.

Using the 33 case histories presented by Olson and Stark (2003) and adding three new cases where reliable CPT was available, Robertson (2010) introduced a CPT-based relationship to evaluate the susceptibility to strength loss and to predict the liquefied shear strength ratio, $s_{u\ (liq)}/\sigma'_{vo}$. To use normalized CPT data to estimate liquefied shear strength ratio, $s_{u\ (liq)}/\sigma'_{vo}$, Roberson (2010) introduced the normalized equivalent clean sand cone resistance value, $Q_{tn,cs}$. This corrects for effective overburden stress, soil type, and pore-water pressure. The normalized cone parameters are given by the following equations:

$$Q_{tn} = \left[\frac{q_t - \sigma_{vo}}{p_a}\right]\left(\frac{p_a}{\sigma'_{vo}}\right)^n \tag{1}$$

$$F_r = [f_s/(q_t - \sigma_{v0})] \times 100\% \tag{2}$$

where q_t is the corrected cone resistance, p_a is the atmospheric pressure, and σ_{vo} and σ'_{vo} are total stress and effective stress, respectively. The exponent n is a function of the SBT index, I_c, which is defined by Equation 3:

$$I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)2]^{0.5} \tag{3}$$

The stress exponent n in Equation 1 varies with both SBT index I_c (soil type) and stress level given by Equation 4:

$$n = 0.38(I_c) + 0.05\,(\sigma'_{v0} / p_a) - 0.15 \qquad \text{where } n \leq 1.0 \tag{4}$$

Finally, the normalized equivalent clean sand ($Q_{tn,cs}$) is given by Equation 5:

$$Q_{tn,cs} = K_c\,(Q_{tn}) \tag{5}$$

where K_c is a correction factor that is a function of the soils characteristics as follows:

$$K_c = 1.0 \qquad \text{if } I_c \leq 1.64$$

$$K_c = 5.581 I_c^3 - 0.403\, I_c^4 - 21.63\, I_c^2 + 33.75\, I_c - 17.88 \qquad \text{if } I_c > 1.64 \tag{6}$$

Robertson (2010) uses normalized equivalent clean sand cone resistance value, $Q_{tn,cs}$, to propose a boundary that separates contractive and dilative soil response and indicates that the contours of the equivalent clean sand cone resistance, $Q_{tn,cs}$, are essentially contours of the state parameter, ψ. Based on the work developed by Jefferies and Been (2006), who postulated that the boundary between contractive and dilative soil is related to a state parameter, ψ, of -0.05, Robertson proposed that a contour line of normalized equivalent clean sand cone resistance, $Q_{tn,cs}$, equal to 70 separates contractive and dilative soil response. Robertson (2010) also introduced a relationship that represents a lower bound estimate of the liquefied shear strength ratio, $s_{u\,(\text{liq})}/\sigma'_{vo}$, based on the normalized equivalent clean sand cone resistance, $Q_{tn,cs}$.

Robertson (2010) reports the normalized equivalent clean sand cone resistance, $Q_{tn,cs,}$ for 33 case histories, and Olson and Stark (2003) report the yield strength ratio, $s_{u\,(\text{yield})}/\sigma'_{vo}$, for 29 case histories. Therefore, it is possible to combine the normalized equivalent clean sand cone resistance, $Q_{tn,cs}$, from Robertson (2010) with the corresponding yield strength ratio, $s_{u\,(\text{yield})}/\sigma'_{vo}$, from Olson and Stark (2003) to develop a relationship to estimate yield strength ratio, $s_{u\,(\text{yield})}/\sigma'_{vo}$, as a function of the normalized equivalent clean sand cone resistance, $Q_{tn,cs}$.

Such a relationship would be valuable for use in liquefaction triggering analysis for fine tailings. Many fine tailings are intermediate materials that cannot be classified simply as sand or silt or clay. This makes it difficult to obtain representative samples (Kramer and Wang 2015, Shuttle and Cunning 2007) for characterization with typical laboratory testing (Poulos et al. 1985a, Poulos et al. 1985b, Poulos 1988), which makes in-situ characterization appealing. Many existing in-situ correlations do not accurately represent the behavior of intermediate materials like fine tailings, but their behavior can be more accurately characterized by $Q_{tn,cs}$. This is because $Q_{tn,cs}$ accounts for tip resistance, sleeve friction, soil type, and pore-water pressure, which are all significant for fine tailings. Developing this relationship to estimate $s_{u\,(\text{yield})}/\sigma'_{vo}$ as a function of $Q_{tn,cs}$ using the case history database was the starting point for this paper.

In addition to the case history database, the authors have accumulated extensive experience through work at tailings basins. A valuable data set has been developed by the authors in which adjacent CPT and FVT soundings have been conducted within fine tailings at a tailings impoundment. The FVT allows for direct measurement of the yield shear strength, and the CPT data allows for the calculation of the normalized equivalent clean sand cone resistance, $Q_{tn,cs}$. The following details the case history database and fine tailings dataset, which were used to develop the proposed correlation between $s_{u\,(\text{yield})}/\sigma'_{vo}$ and $Q_{tn,cs}$.

4.1 *Case histories data used in proposed relationship for yield shear strength ratio*

Table 2 contains a summary of the case histories presented by Olson and Stark (2003) and augmented by Robertson (2010) that were used in this paper. For consistency, the case history numbers and structure names identified in Table 2 are the same used by Olson and Stark (2003) and Robertson (2010).

The classes reported in Table 2 are based on Robertson's (2010) classifications and serve as a way to represent the reliability of the data for each case history. Class A cases had reliable CPT measurements that included both tip resistance and sleeve friction values. Class B cases had less reliable CPT measurements and included mechanical or electric tip resistance values without sleeve friction values. Case histories where CPT values were estimated from either SPT, relative density, or best estimates were identified as Class C, Class D, or Class E, respectively, and were considered the least reliable. Similar to Robertson (2010), this paper only considered case histories with Class A and B data, since the data comes from actual CPT results and is considered reliable.

Approximate D_{50} and fines content (FC) for each case were reported by Olson and Stark (2003). Normalized cone resistance, Q_{tn}; normalized friction ratio, F_r; soil behavior type index, I_c; and normalized equivalent clean sand cone resistance, $Q_{tn,cs}$, were reported for each case by Robertson (2010). These values are all summarized in Table 2.

Undrained yield shear strength ratio, $s_{u\ (yield)}/\sigma'_{vo}$, was reported by Olson and Stark (2003) for many of the case histories shown in Table 2. However, for three cases, other sources were used to estimate the undrained yield shear strength ratio. For case 15, the undrained yield shear strength ratio was obtained from Olson (2001). For cases 9 and 27, the normalized cone resistance, q_{c1}, was obtained from Olson and Stark (2003). This was used to calculate brittleness index, I_B, and estimate yield undrained shear strength ratio, $s_{u\ (yield)}/\sigma'_{vo}$, using methods from Sadrekarimi (2014).

In total, 10 case histories (categorized as class A or B with available $Q_{tn,cs}$ data and either reported yield shear strength ratio or supporting data to calculate it) were selected for use in the development of a relationship between $s_{u\ (yield)}/\sigma'_{vo}$ and $Q_{tn,cs}$, as summarized in Table 2.

Table 2. Case Histories of Flow Liquefaction Failures with Measured Penetration Resistance

Case history number[1]	Structure[1]	Class[2]	Approx. D_{50}[1] (mm)	Approx. FC[1] (%)	Q_{tn}[2]	F_r[2] (%)	I_c[2]	$Q_{tn,cs}$[2]	Yield strength ratio[3]
1	Zeeland	B	0.12	3 to 11	30	0.25	2.1	43	0.265
9	Kawagishi-Cho Bldg.	B	0.35	<5	31	0.5	2.2	50	0.283
14	Hokkaido Tails Dam	B	0.074	50	4	1.50	3.2	36	0.195
15	LSFD	A	0.074	50 (5-90)	5	3.5	3.3	52	0.282
17	Mochi-Koshi Tailings 1	B	0.04	85	5	2.5	3.3	48	0.27
18	Mochi-Koshi Tailings 2	B	0.04	85	5	2.5	3.3	48	0.22
19, 20, 21	Nerlerk Slide 1, 2, 3	A	0.22	2 to 12	40	0.4	2.0	55	0.21
22	Hachiro-Gata Road	B	0.2	10 to 20	30	0.5	2.2	50	0.16
27	Fraser River Delta	A	0.25	0 to 5	15	1.5	2.6	58	0.368
31	Soviet Tajik	B	0.012	100	19	1.0	2.6	53	0.30

(1) From Olson and Stark (2003)
(2) From Robertson (2010)
(3) Yield strength ratio for case 15 from Olson (2001) and for cases 9 and 27 calculated using Olson and Stark (2003) and Sadrekarimi (2014). All other yield strength ratios from Olson and Stark (2003).

4.2 *Fine tailings data used in proposed relationship for yield shear strength ratio*

Table 3 presents the fine tailings data collected by the authors where CPT soundings were performed adjacent to FVT soundings at 10 locations across a tailings basin. The testing was conducted in fine tailings with low plasticity (plasticity index typically less than 12 percent). The fine tailings tested at these 10 locations were found to have a soil behavior type (SBT) distribution as follows: 58% SBT 3, 19% SBT 4, 21% SBT 5, and 2% SBT 6. According to the classification provided by Robertson (1990), these SBT values correspond to "clays – clay to silty clay" (SBT 3), "silt mixtures – clayey silt to silty clay" (SBT 4), "sand mixtures – silty sand to sandy silt" (SBT 5), and "sands – clean sands to silty sands" (SBT 6). The predominant materials in these deposits were characterized as clays, silt mixtures, and sand mixtures (SBT 3, 4, and 5), with clays and silt mixtures (SBT 3 and 4) accounting for 77% of the fine tailings.

FVT allows for direct in-situ measurement of the peak shear strength during undrained loading, which represents the yield shear strength, $s_{u\ (yield)}$. FVT is the only in-situ testing method that provides a direct measurement of undrained shear strength. For the authors' tailings dataset (Table 3), FVT was conducted using electronic equipment that measures the torque down the hole at a rotation rate based on Blight (1968) to maintain undrained conditions during testing (Castro 2003).

Figure 2 shows that the FVT results compare well to the CPT interpretation of the undrained shear strength. Because CPT does not measure undrained shear strength directly, a site-specific bearing capacity factor, N_{kt}, of 16 was used to interpret the CPT shown in Figure 2.

Table 3. Tailings Locations with Adjacent CPT and Undrained Shear Strength (FVT) Measurements

Tailings location	Class [1]	SBT[2]	Q_{tn} [3]	Fr [3] (%)	I_c [3]	$Q_{tn,cs}$ [3]	Yield strength ratio[4]
A	A	3,4,5	5.62	1.06	2.99	37.3	0.339
B	A	3,4	4.39	1.44	3.14	37.1	0.258
C	A	3,4,5	4.75	1.50	3.12	38.7	0.329
D	A	3	3.66	2.07	3.29	38.5	0.287
E	A	3	3.91	1.23	3.16	34.0	0.215
F	A	3	3.68	1.11	3.17	32.3	0.200
G	A	3	5.18	1.44	3.08	39.7	0.252
H	A	3,4	6.58	1.26	2.95	40.6	0.240
I	A	3	3.35	1.70	3.28	34.9	0.254
J	A	3	3.12	1.99	3.60	50.2	0.229

(1) Classification based on Robertson (2010)
(2) Classification based on Robertson (1990)
(3) Calculated based on author's tailings CPT data and Equations 1 through 6
(4) Calculated based on authors' tailings FVT data

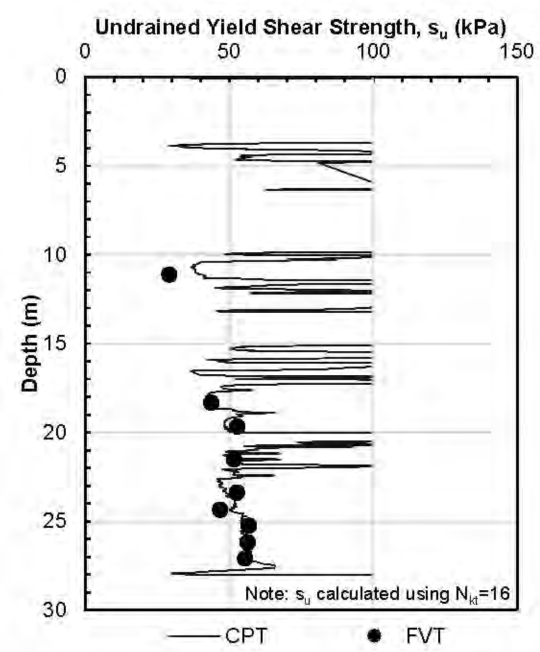

Figure 2. Tailings CPT and FVT profiles (case G from Table 3)

4.3 *Susceptibility to liquefaction*

Figure 3 presents the CPT data from the case history database (summarized in Table 2) and the CPT data from the authors' tailings dataset (summarized in Table 3) in terms of the normalized cone results with respect to the normalized CPT-based SBT chart. The normalized CPT-based SBT chart has lines delineating each SBT, as well as the contour line that represents a clean sand equivalent penetration resistance ($Q_{tn,cs}$) of 70, proposed by Robertson (2010) to separates materials with contractive and dilative behavior in undrained shear. The mean value of the data presented in Figure 3 is represented by a square for the case histories and a circle for the authors' tailings dataset. The data from the author's tailings dataset include error bars to illustrate the variability (showing one standard deviation) of the normalized friction ratio, F_r, and normalized tip resistance, Q_{tn}, at each location. It can be seen in Figure 3 that the mean values from the case histories are classified as SBT 3, 4, 5, and 6, and the mean values from the authors' tailings dataset are classified as SBT 3 with error bars extending into SBT 4. It can also be seen from Figure 3 that all of the data plot below Robertson's (2010) $Q_{tn,cs}$ = 70 contour line, indicating contractive or strain softening behavior for the case histories and the authors' tailings dataset.

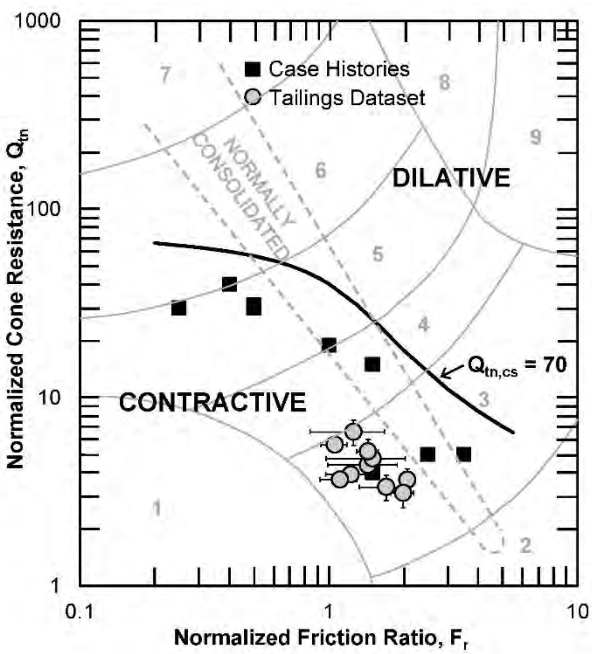

Figure 3. Soil behavior type (SBT) chart based on normalized CPT parameters (after Robertson 2010), showing Olson and Stark (2003) case histories and the author's tailings dataset

5 PRELIMINARY RELATIONSHIP BETWEEN $Q_{tn,cs}$ AND $S_{U\ (\mathrm{YIELD})}/\sigma'_{\mathrm{VO}}$

Based on the case histories and the authors' tailings dataset, the proposed relationship between $Q_{tn,cs}$ and $s_{u\ (\mathrm{yield})}/\sigma'_{\mathrm{vo}}$ to use in liquefaction triggering analysis is shown in Figure 4. Figure 4 includes the mean value for each case history/location represented by squares for the case histories (Table 2) and circles for the authors' tailings dataset (Table 3). Figure 4 includes a linear regression trend line through the data points and also the linear trend line ±1 standard deviation. This preliminary relationship between $Q_{tn,cs}$ and $s_{u\ (\mathrm{yield})}/\sigma'_{\mathrm{vo}}$ can be represented by Equation 7:

$$s_{u\ (\mathrm{yield})}/\sigma'_{\mathrm{vo}} = 0.0015\ (Q_{tn,cs}) + 0.1915\ (\pm\ 0.043) \tag{7}$$

Equation 7 was developed using materials with contractive behavior. Soils with a $Q_{tn,cs}$ greater than 70 are expected to be dilative and not susceptible to liquefaction, so the relationship between $Q_{tn,cs}$ and $s_{u\ (\mathrm{yield})}/\sigma'_{\mathrm{vo}}$ should not be used for $Q_{tn,cs}$ greater than 70.

It should be noted that not all data points fall within one standard deviation of the linear regression trend line represented by Equation 7. The three data points that fall above one standard deviation are not concerning; even though using the relationship would lead to under-prediction of the undrained yield shear strength ratio, this is conservative for use in liquefaction triggering analyses. The three data points that fall below one standard deviation are more concerning because using the relationship would over-predict undrained yield shear strength ratio.

To improve the preliminary relationship presented in Equation 7, additional data from a robust number of sites with variable tailings characteristics needs to be collected and analyzed. Additional data with a wider range of $Q_{tn,cs}$, $s_{u\ (yield)}/\sigma'_{vo}$, and SBT should be collected and analyzed. Once the data set is expanded, the preliminary relationship can be revisited to assess if the trend between $Q_{tn,cs}$ and $s_{u\ (yield)}/\sigma'_{vo}$ can be refined and if it can be better represented by something other than a linear trend line. Lastly, the data presented in this paper can be further assessed to determine if there are reasons that can explain the scatter in the data.

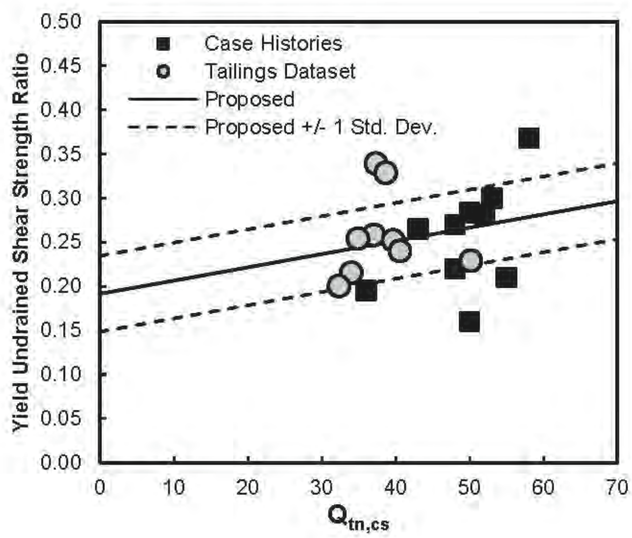

Figure 4. Relationship to predict yield undrained shear strength ratio based on $Q_{tn,cs}$ (linear regression ± 1 standard deviation)

6 SUMMARY AND CONCLUSIONS

A preliminary relationship for estimating the yield shear strength ratio, s_u (yield)/σ'_{vo}, as a function of the equivalent clean sand cone penetration resistance, $Q_{tn,cs}$, has been developed and presented. First, the CPT data from the case histories and the authors' tailings dataset were plotted on the normalized CPT-based SBT chart, which shows that all data used to develop the relationship plots below the $Q_{tn,cs} = 70$ contour line. This indicates contractive or strain softening behavior. Then, the new relationship was developed by pairing $Q_{tn,cs}$ from the case histories reported by Robertson (2010) with the corresponding yield strength ratio $s_{u\ (yield)}/\sigma'_{vo}$ from Olson and Stark (2003). Additional data from the authors, consisting of CPT and FVT soundings conducted in fine tailings, were included in the relationship development. The FVT allows for direct measurement of the yield shear strength, and the CPT data allows for the calculation of the normalized equivalent clean sand cone resistance, $Q_{tn,cs}$. Equation 7 is proposed on a preliminary basis as a relationship to estimate the yield shear strength ratio from $Q_{tn,cs}$, determined from CPT data for use in liquefaction triggering analysis. The authors intend to collect and analyze additional data to expand the dataset. The objective is to include a robust number of sites with variable tailings characteristics including $Q_{tn,cs}$, $s_{u\ (yield)}/\sigma'_{vo}$, and SBT to enhance the relationship and assess if a linear trend is the most appropriate for the data. The additional data, with an updated relationship for use over a broader range of conditions, will be published in the future.

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Characterization of Unsaturated Tailings & its Effects on Liquefaction

Bryan F. Ulrich
Knight Piésold and Company, Denver, CO, USA

Jeffrey G. Coffin
Knight Piésold Ghana Limited, Accra, Ghana

ABSTRACT: In the mining industry, it is becoming relatively common for filtered mine tailings to be stacked to a significant height (>100m) with little to no compaction. The resulting deposit can be loose and potentially unsaturated. Characterization of these filtered tailings facilities for slope stability and deformation under earthquake loading conditions can be complex due to the potentially unsaturated state. This paper presents a summary of test results from the assessment of a filtered tailings facility which is located in Latin America. Cone penetration tests (CPTs) and seismic velocity measurements (SCPTu) were carried out along with selected drilling, sampling and laboratory testing. The results of laboratory testing utilizing bender element testing indicate that the CPT-based liquefaction analysis could not be relied upon due to suction-hardening of the unsaturated tailings, and that the seismic wave tests were the more applicable approach to assessing the potential for liquefaction in this case. The paper presents the results of the characterization and the interpretation of the test data.

1 BACKGROUND

1.1 *Basic Project Information*

The project under discussion is an inactive filtered tailings facility located in an arid and high-altitude region of Latin America. To gain an understanding of the liquefaction potential of the deposited tailings, a series of in-situ tests was carried out during a large-scale site investigation completed to characterize the deposited tailings. Due to the high seismicity project setting and nature of the tailings, seismic cone penetration tests with pore pressure measurements (SCPTu) were utilized with the intention of characterizing the liquefaction susceptibility of the tailings. Additionally, due to the arid project setting and elapsed time from previous operations, seismic wave measurements were obtained during probing to provide insight into the saturation of the tailings.

The CPT-based liquefaction analysis, following the Robertson approach (e.g., Robertson and Wride, 1997, Robertson, 2015), indicated that the tailings were likely contractive and susceptible to liquefaction during the design earthquake event, even at significant depth. The shear-wave-based liquefaction approach, following the methodology described by Kayen et al. (2013), indicated that the tailings were dilative and therefore not susceptible to liquefaction during the design earthquake event.

Given this vast disparity in results, an additional step was made to improve the understanding of those test results. Included in the additional work was a series of triaxial shear strength tests, with bender element excitation during consolidation and during shear, to confirm that testing was conducted under in-situ conditions, which will be described in a later section of this paper.

1.2 *Filtered Tailings and the Tailings Continuum*

Filtered tailings reside at the higher solids content end of the tailings thickening continuum. The thickened tailings continuum has been presented numerous times elsewhere (e.g., Ulrich and Coffin, 2013, Kerr and Ulrich, 2011, Ulrich and Kerr 2011)). The continuum simply represents the nature and behavior of tailings at various degrees of thickening, as follows:

- Conventional slurry tailings
- Conventionally thickened tailings
- High density tailings
- Paste tailings
- Filtered tailings

Yield stress is often employed as a distinct boundary to help define these transitions within the continuum. According to Ulrich and Kerr (2011), the yield stress separating conventional slurry tailings and thickened tailings tends to range between 5 and 20 Pascals (Pa) [1 lb/ft^2 is approximately 48 Pa]. The boundary between thickened and paste tailings is approximately 100 Pa, and 800 Pa is the boundary between paste tailings and filtered tailings. Some practitioners prefer to think of the thickened tailings continuum in terms of solids content rather than using yield stress as the reference condition. While convenient, it may not be entirely meaningful in understanding material behavior. For example, consider two tailings slurry samples. One has a high magnetite content. The other sample is "exactly the same", but without the magnetite component (the magnetite is "replaced" instead with the ordinary host rock, thus the volume of solids is identical for both samples, as is the volume of water). The difference in average particle specific gravity between the two samples is the only difference, but the two samples would have different solids contents. It isn't altogether satisfying to place these two materials at different positions on the thickened tailings continuum based simply on their different magnetite content (or more generally, specific gravity). A similar comparison could be made for otherwise identical samples containing different amounts of clay minerals. In this case, they may indeed have quite different yield stresses, but identical solids content. So, although conceptually less intuitive for many people, yield stress is the author's preferred reference for the thickened tailings continuum rather than solids content, due to the direct evaluation of material behavior.

Ulrich and Kerr (2011) indicate that some of the key drivers for selecting thickened, paste or filtered tailings pertain to social acceptance, ease of permitting, and water savings. Especially potential fresh water consumption savings. In very arid regions, water can come at a considerable cost and even more so if population centers are situated near the mining area who also rely on the resource. Accordingly, finding a means of reducing fresh water consumption can come to the forefront in tailings management planning. Social acceptance and ease of permitting fall right in line with the water savings considerations. Other drivers include:

- Water recovery prior to deposition
- Avoidance of evaporative losses
- Topographic constraints
- Land constraints
- Regulatory restrictions
- Closure implications
- Draindown considerations
- Corporate decision making

One advantage of reducing the moisture content of the tailings before sending it to the disposal area is that there is a potential to reduce seepage that may be released into the environment. Another advantage is the reduced amount of consolidation the tailings will experience (including post-closure consolidation) should be reduced in most cases. This is also an advantage of subaerial tailings (refer to Knight and Haile, 1983 and Ulrich, East and Gorman, 2000), although this achievement is accomplished differently with the two methodologies. The first process (filtered tailings) recovers water in the mill and the latter process (subaerial tailings) recovers water via an underdrain and decant pond, but also water lost to evaporation is promoted to increase desiccation of the tailings. Compared to conventional slurry, either process helps to reduce the magnitude of consolidation by removing water from the tailings. The possible disadvantage of subaerial tailings is that some of the moisture may be lost to evaporation (although if water does not come at a premium price, this may not be a disadvantage).

1.3 *Filtered Tailings*

While the use of the filtering process in tailings management has been growing rapidly over the past several years, there are a disproportionately meager number of publications regarding this topic compared to other tailings preparation methodologies. Davies et al. (2010) indicate that there are approximately triple the number of filtered tailings facilities as there are surface paste tailings facilities, but the number of publications for surface paste facilities far outweigh the number of publications for filtered tailings facilities. This disparity is primarily due to the perceived relative ease of the design and operation of a filtered tailings facility compared to a surface paste facility, but also the uniqueness, complexity and idiosyncrasy of surface paste facilities. In fact, the industry 'go-to' handbook on the topic, 'Paste and Thickened Tailings – A Guide' (Jewell and Fourie, 2006) discusses the topic of surface filtered tailings facilities only in passing, although there is a good discussion on the filtering process itself.

Filtered tailings facilities may be perceived as being very ordinary and somewhat undeserving of extensive coverage in the literature. This perception is somewhat true as there are many ordinary filtered tailings facilities which have been constructed and operated; however, as the filtering technologies continue to develop in the efficiency of filter plants, very significant facilities (i.e., high throughput) are now being designed that will bring this application to ever greater importance. In fact, USFS (2010) reports on an operation that will be producing filtered tailings at a rate of approximately 70,000 tonnes per day with a facility that will ultimately store 300 million tonnes of tailings to a height of 150 meters (Ulrich and Coffin, 2013).

It has been said elsewhere (e.g., Davies and Rice, 2001, Ulrich and Coffin, 2013), but it bears repeating here. There is no one-size-fit-all tailings panacea, and no single tailings preparation method should be considered a cure-all. This includes filtered tailings. For example, Wilson and Robertson (2015) wrote:

> "The writer's note that in the specific case of 'dry stacking of filtered tailings' this method should not be considered a panacea for the elimination of failure potential. One of the writers is currently reviewing two such dry stacks where the design, operating and site conditions have [led] to an urgent need for remedial modifications to avoid failure conditions".

Filtered tailings facilities are often referred to as 'dry stacks' (e.g. Davies and Rice, 2001; USFS, 2010; Davies et al., 2010; Lupo and Hall, 2010), while contradictory terms are also used for certain portions or aspects of the deposit such as 'wet cake' (Davies and Rice, 2001; Davies et al., 2010; Newman et al., 2010), and zones of the dry stack that may be 'overly wet' (Lupo and Hall, 2010). Davies and Rice (2001) indicate that the term dry stack has been adopted by designers and regulatory authorities for filtered tailings facilities, admitting that the facilities are not truly dry, but that if practitioners bear in mind that this is a somewhat misused term, it is acceptable to continue using this terminology. This understanding was a focus of the research presented herein as liquefaction is a phenomenon associated with saturated, or nearly saturated, materials and not generally considered a risk for 'dry' materials.

Regarding terminology, the rather misleading term dry stack is generally not a good engineering term since the target moisture content coming from the filter plant is typically desired to be somewhere around the optimum moisture content based on the Proctor compaction procedure (either the standard or modified test, as determined by the design engineer). Geotechnical engineers associate the optimum moisture content with moisture levels just below full saturation after compaction, thus terming such a facility as a dry stack is a misnomer. The present authors would encourage practitioners to abandon the use of the term dry stacking in favor of the more straightforward term, 'filtered tailings'. It is not desirable to unintentionally mislead the public at large with an industry term that is noticeably misused (Ulrich and Coffin 2013).

1.4 *Liquefaction and In Situ Testing*

A good discussion of liquefaction is provided by Robertson and Wride (1998). They indicate that during cyclic undrained loading, almost all saturated, frictional soils develop positive pore pressures due to the contractive response of the soil at small shear strains. If there is shear stress reversal, the effective stress state can advance to the point of momentary zero effective stress.

When frictional soil reaches the condition of essentially zero effective stress, the soil has very little stiffness/shear strength and large deformations can occur during, or immediately after, cyclic loading. When cyclic shaking stops, those deformations essentially stop, except for deformation due to pore-pressure redistribution, which may still be significant.

Significant developments have taken place in recent decades in evaluating the liquefaction potential of soils. The cone penetration test is now commonly used to evaluate liquefaction potential due to the ability to conduct the testing on in-situ materials at in-situ states of stress, thus, removing the complication of undisturbed soil sampling or sample remolding in the laboratory (Robertson et al., 1992; Robertson and Wride, 1997; Robertson, 2015). Additionally, liquefaction assessments using the CPT have the advantage of producing nearly continuous, repeatable measurements that provide a detailed profile of the soil.

There have also been significant developments in evaluating liquefaction potential based on in-situ shear wave velocity (V_s) measurements (Kayen et al., 2013). Potential liquefaction assessment methods based on shear wave velocity have the advantage that they are essentially independent of intrinsic soil characteristics, such as fines content, as they only measure the bulk response of a soil mass. However, this approach usually lacks the stratigraphic information provided by the CPT (Robertson, 2015) due to the practical resolution of the testing interval in-situ.

2 THE PROJECT

2.1 *Facility Description*

The filtered tailings facility that is the subject of this paper is located in a very arid, high-seismicity, and high-altitude site in Latin America. The facility is composed of filtered tailings that were placed via conveyor and radial arm stacker. The material was placed in lifts of approximately 20 to 30m thickness during the mining operations with minimal compaction. Lifts were placed at individual slopes ranging from approximately 1.5H:1V (horizontal to vertical) to 2.5H:1V with each lift offset by either 40 to 60m to create benches, resulting in an approximate overall downstream slope of 4H:1V. At the time of the investigation, the maximum height from toe to crest was about 200m with a maximum thickness of tailings of approximately 125m. The total surface area of the facility is about 30 hectares with an estimated storage of 100 million tonnes of filtered tailings.

One of the main design objectives of the assessment was the stability of the tailings facility under static and seismic loading conditions; therefore, detailed characterization of the shearing behavior of the tailings under both static and seismic loading conditions was important. This included establishing whether the tailings could experience strength loss and related instability (Robertson et al., 2017) due to the occurrence of the design earthquake event.

Given the dry climate of the region and the placement of the tailings in a moist, loose state, the tailings were expected to be predominately unsaturated. In unsaturated soils the voids are filled with a mixture of fluid and air, which can result in the development of suction forces. In general, unsaturated soils have a higher resistance to cyclic loading but may experience some strength loss if the degree of saturation is relatively high and the soils very loose (Grozic et al., 2000).

A site-specific seismic hazard assessment was completed to characterize the potential seismic loading conditions to be used in the design and operation of the filtered tailings facility. The maximum design earthquake (MDE) was defined as the event with a recurrence interval of 2,475 years considering the Significant Hazard Class (Canadian Dam Association, 2014) assigned to the facility and the owner's corporate design requirements. This event corresponds to a subduction zone type earthquake event of moment magnitude 9.0 producing a peak horizontal ground acceleration (PHGA) of 0.45g.

2.2 *Site Investigation*

Since the onset of mine operations in the 1990s there have been several geotechnical studies to characterize the tailings deposit. This current paper focuses on the most recent study that began in 2014 and included SCPTu probes, drilling, sampling (both disturbed and undisturbed sample recovery) and the installation of piezometers. Laboratory testing was carried on selected and representative reconstituted samples, tested in either a saturated or unsaturated state.

Based on grain size distribution curves, the tailings are classified as predominately silty sand to sandy silt with a mean grain size (D_{50}) of about 0.065mm. The grain size distribution curve for the tested tailings is similar to other poorly graded silty tailings reported by Jefferies and Been (2016) based on classification under the Unified Soil Classification System (USCS) and visual comparison. The average specific gravity of the tailings is 2.73 and the tailings are predominately non-plastic.

The SCPTu program was carried out in accordance with ASTM D5778 using a portable hydraulic ram mounted on to a drill rig. A total of 12 SCPTu probes were advanced in the tailings facility. Compression wave (V_p) and V_s measurements were made using a geophone and the down-hole method (e.g., Robertson et al, 1986). The intention was to use the V_p data to identify the depth at which materials become saturated (generally, an estimated V_p equal to 1,500 meters per second [m/s] indicates a material is saturated) and the V_s data as both a liquefaction potential indicator (generally, a V_s less than 200 meters per second indicates potentially liquefiable materials [Robertson et al., 1992]) and to estimate small strain behavior of the tailings for seismic deformation analyses. Pore pressures were measured behind the cone tip continuously during probe advancement. Additionally, pore pressure dissipation (PPD) testing was conducted by halting advancement at various depths to allow the penetration induced pore pressure to equilibrate to the in-situ ambient condition. PPDs that did not stabilize sufficiently during the test were extrapolated using the method discussed by Scheremeta (2014). Numerous vibrating wire piezometers were also installed at six locations. At each location, several piezometers were installed at different depths in a nested array to monitor the piezometric pressures over time.

The measured V_s data varied between 200 and 500 m/s (with an average normalized shear wave velocity [V_{s1}] of approximately 225 m/s) over the depth of tailings tested. The measured V_p data was consistently less than 1,500 m/s with an average of approximately 800 m/s. Over significant continuous intervals, the penetration induced pore pressure measured at or near zero (regularly slightly negative), indicating that the materials are sufficiently unsaturated that both volumetric and shear induced volumetric strain were insufficient to cause saturation and positive pore pressure development. The results of the PPD and piezometer readings estimated zero, or slightly negative, ambient pore pressures, with the exception of the bottom several meters of the deposit which indicated a nominal positive pore pressure in both the PPD results and piezometer readings.

2.3 *Liquefaction Analyses*

While the CPT data suggests that the materials are likely unsaturated, potentially precluding the occurrence of liquefaction, sufficiently undisturbed samples could not be obtained to estimate the in-situ degree of saturation. Due to this uncertainty, the extreme seismicity of the region, and the hazard class of the facility, it was deemed prudent to conduct a thorough liquefaction assessment. Liquefaction analyses were completed using the CPT-based approach following the Robertson methodology (e.g., Robertson and Wride, 1997, Robertson, 2015), and the shear-wave-based approach, following the methodology described by Kayen et al. (2013).

The two methodologies indicated different results, where liquefaction potential was predicted using the CPT approach, but it was not predicted by the shear wave velocity approach. Refer to Figure 1 for a typical example of contradictory results from field data. Given this contradiction, additional laboratory testing was conducted to attempt to resolve this apparent discrepancy.

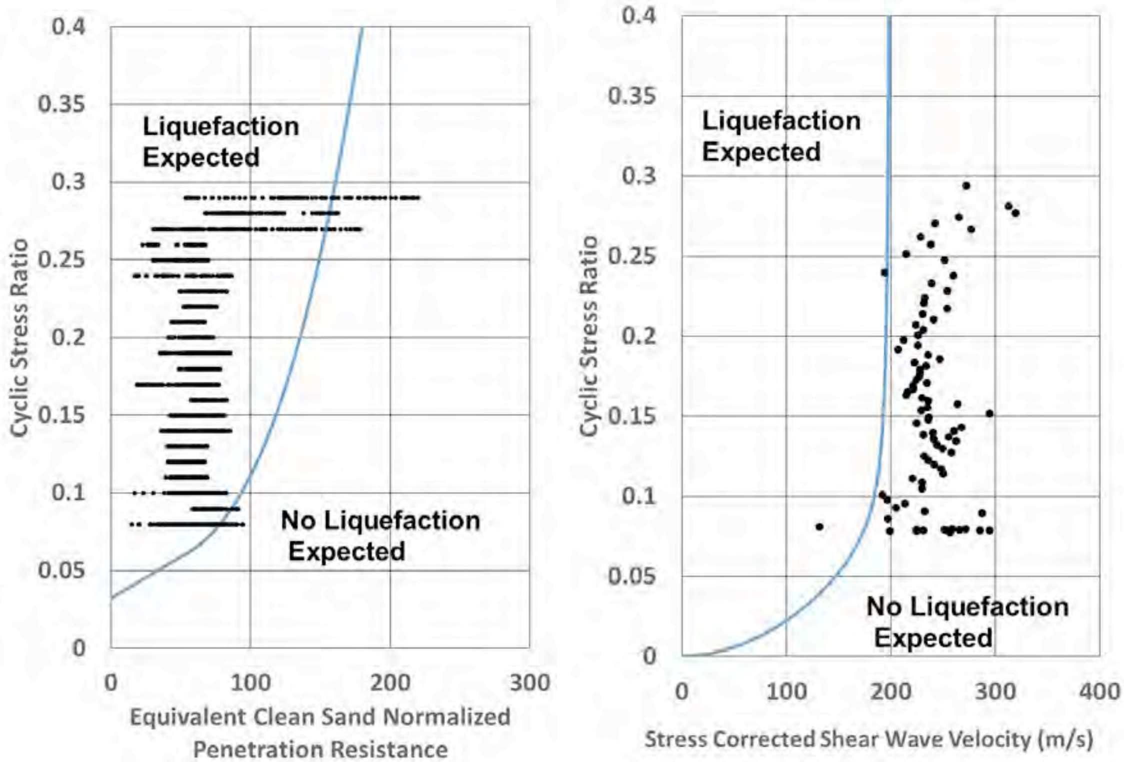

Figure 1. Typical Example of Contradicting Results from Field Data

The laboratory testing program consisted of a suite of triaxial shear strength testing on saturated and unsaturated samples under drained and undrained conditions. Additionally, bender element testing was conducted during the consolidation phase and during the shearing phase of the triaxial testing to gain additional insight and for comparison with the in-situ test data.

3 BENDER ELEMENT AND TRIAXIAL SHEAR STRENGTH TESTING

3.1 *General*

Bender elements provide a simple means of determining the elastic (very small-strain) shear stiffness of a soil sample. Bender elements are shear wave transducers for instrumenting soil cells due to soil–transducer coupling and compatible operating frequency (Lee & Santamarina, 2005). Bender elements, in which an elastic modulus is derived based on the wave propagation theory, enhance the capabilities of triaxial testing devices such that one can measure both dynamic and static parameters of soils subjected to axisymmetric stress conditions. Because the bender elements induce very small strains which keep the specimen intact during loading, measurement of elastic and elastic-plastic responses may be made simultaneously during monotonic loading (Leong, 2009). Kim et al. (2015) provide a good description of bender element testing:

> "Bender elements are thin piezoceramic electro-mechanical transducers capable of transmitting and receiving signals; when installed on both ends [of a test apparatus], they can be used to measure wave velocities in a sample. Bender elements consist of two piezoceramic plates bonded together in series or parallel with an electrode plate in between. They convert electrical energy into mechanical movement and vice versa. Bender elements are typically mounted in the base pedestal and top cap of a triaxial cell. When excited by an input voltage, the source element bends, emitting a horizontally polarized wave that travels through the soil sample. In response, the wave motion causes the receiver element to mechanically vibrate, which is captured by a high-speed digital data acquisition system. The shear wave velocity is calculated by determining the travel distance and the travel time".

For this project, bender element tests were carried out on reconstituted, representative saturated and unsaturated samples of filtered tailings collected at the project site. The reason that reconstituted samples were tested was due to the difficulty of obtaining high-quality, undisturbed samples during the site investigation. It also allowed for variance in the degree of saturation and initial sample density to readily represent the range of conditions anticipated in-situ.

Samples were prepared by remolding using the moist tamping technique (Vaid et al., 1999) with an initial moisture content of 11.5 percent that resulted in loose samples (i.e., samples prepared at as low an initial dry density as practical) prior to consolidation and shearing. A series of triaxial shear tests was performed on samples saturated using percolation and backpressure, according to the procedure proposed by Viana da Fonseca et al (2015). Upon achieving saturation, the samples typically collapsed slightly resulting in slightly lower void ratios at the onset of the consolidation phase. Seismic velocities (both V_s and V_p) were measured using the bender elements during consolidation and the shearing phase of the testing. Triaxial shearing (drained and undrained) was carried out to determine the critical state conditions of the samples and the Critical State Line (CSL) of the material (Jefferies and Been, 2006). Additionally, a similar series of tests was conducted on unsaturated samples to measure the behavior of the material more representative of the in-situ conditions. Those samples were subject to suction measurement using a suction probe, and volume change was tracked using radial calipers. Seismic wave velocity measurements on unsaturated samples with known suction provide an opportunity to determine the relationship between V_{s1} and void ratio based on effective stress values that incorporate suction.

3.2 *Results*

The results of the saturated suite of tests are consistent with previous published work for similar materials (Cunning et al, 1995; Jefferies and Been, 2014) in terms of the shape and location of the CSL and V_s contours at the end of isotropic consolidation. The in-situ and laboratory testing results at a mean effective reference stress of 100 kPa indicate that a V_s between 115 and 135 m/s would represent the contractive-dilative boundary; while Cunning et al. (1995), indicated a V_s as high as 150 m/s for young, uncemented sands under these conditions. Based on these results and the in-situ measured V_s of 225 m/s, the project tailings would be expected to dilate under shear if the materials were saturated.

The results of the unsaturated suite of tests (refer to Figure 2) indicated that V_{s1} values significantly increase with suction; this appears to be primarily due to suction hardening rather than from the change in effective stress due to suction (Robertson et al., 2017). In addition, the CSL is located at higher values of void ratio (for similar confining stress) compared to the saturated samples. Also shown on Figure 2 are approximate contours for shear wave velocity (V_s). These contours were established based on the project-specific relationship:

$$V_{s1} = 220 - 145e$$

Where e is the void ratio at the end of consolidation for saturated samples, and V_{s1} are from the Bender testing. The normalized shear wave velocity was reverted to shear wave velocity assuming a K_o=0.50 condition.

The shift in the unsaturated CLS line is as expected and previously described by Leroueil and Hight (2003). The reason for this shift is due to the development of matric suction within the unsaturated matrix which generally increases the effectives stress and causes a stronger and stiffer response to loading than identical materials under saturated conditions. The position of the CSL for the unsaturated samples appears to be independent of the magnitude of suction, at least within the range of suction carried out in these tests. Leroueil and Hight (2003) proposed that this suction hardening can be described by the following:

- Suction increases the size of the yield surface for soils, such that unsaturated soils tend to behave more like over-consolidation soil (i.e., respond inside the yield surface)
- The size of the yield surface is a function of the amount of suction (e.g., higher suction values produce larger yield surface and a more dilatant response)
- Suction appears to move the CSL and is also a function of the magnitude of suction

– The movement of the CSL produces an apparent ‘cohesion’ in terms of strength (due to the higher yield surface and CSL)

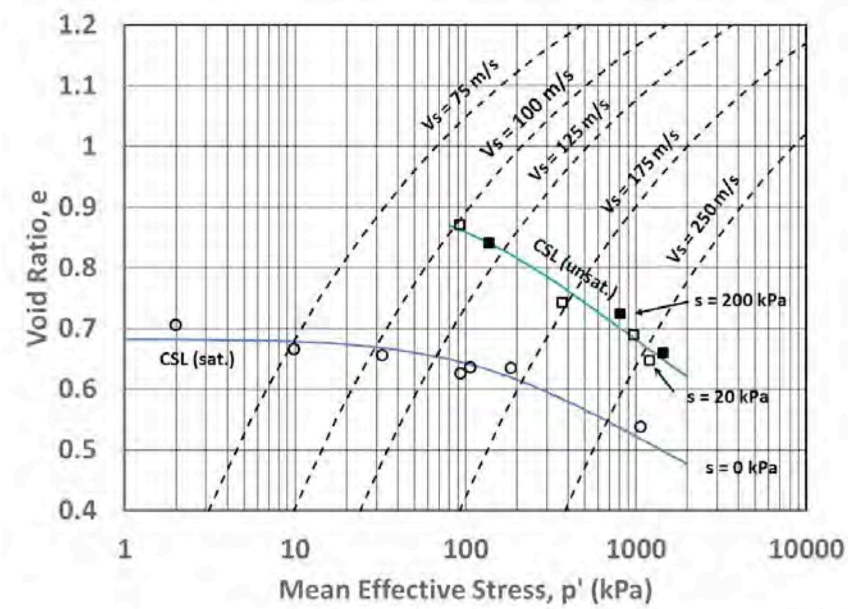

Figure 2. Summary of Bender Element Test Results

The unsaturated triaxial testing of the project tailings was conducted at V_s and V_p values (180 to 330 and 460 to 740 m/s, respectively) generally within the range of those measured on site (200 to 550 and 350 to 1150 m/s, respectively) to confirm that testing would be representative of the in-situ conditions. While the suctions anticipated under these in-situ conditions (likely between 50 and 100 kiloPascal (kPa) based on the project specific soil water characteristic curves) are relatively small, the effect of suction hardening is significant in terms of soil behavior under shear.

Based on the results of both the saturated and unsaturated triaxial testing, it was concluded that the tailings are expected to be dilative under large strain, even if the materials were to become saturated in the future. This is consistent with the shear wave liquefaction assessment methodology, but at odds with the CPT based assessment. The unsaturated nature of the tailings would lead to a higher compressibility due to the entrained air within the voids which would appear as a lower measured cone resistance during CPT probing. This lower penetration resistance will increase the estimated liquefaction potential of the materials, and in this instance, the CPT-based liquefaction assessment provided an inaccurate indication that the materials would likely liquefy due to the occurrence of the design earthquake.

4 CONCLUSIONS

The filtered tailings facility that is the subject of this paper is an example of a well-operated facility. This is not always the case for filtered tailings facilities. When designing a filtered tailings facility, the designer should duly consider the site topography, climate, operational practice and the intended geotechnical characteristics of the filtered tailings. The inclusion of drainage features should be considered, as should cold/wet season operation and/or storage areas within the facility. The designer should consider the potential of upset conditions to occur as well as variation of the geotechnical characteristics of the tailings over time. There is no panacea for tailings deposition, and filtered tailings is but one option that designers have in their toolbox.

The results of the project site investigation provided conflicting liquefaction potential indicators. The degree of saturation and V_s were consistent with materials which are not expected to liquefy under shear. However, the industry standard practice considering the CPT cone resistance methodology indicated that the materials were likely to liquefy if saturated, or nearly saturated. Upon additional investigation using drained and undrained triaxial shear strength testing, with bender element testing to confirm samples are prepared at the measured in-situ seismic wave velocities, it was determined that the materials present within the project facility are expected to dilate under large strain. The “false positive” indicated by the CPT results is likely due to the higher compressibility of air in the voids of the unsaturated tailings artificially reducing the cone resistance during probing and decreasing the beneficial effects of suction hardening which is present in unsaturated soils.

The results of the triaxial shear strength testing were consistent with previous research and provided a suitable alternative to understanding the tailings behavior under shear. When industry standard approaches seemed to provide insufficient understanding of the materials the more sophisticated, less common approach provided suitable reconciliation of the entire data set for use in the design and engineering analyses.

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Risk Management

Tailings Facility Risk, Resilience, and Robustness

Andrew Robertson & Jack A. Caldwell
Robertson GeoConsultants, Vancouver, BC, Canada

ABSTRACT: This paper re-examines the failures of the Mt Polly and the Samarco tailings facilities through the perspective of risk, resilience, and robustness. The paper defines these terms and the many other terms including factor of safety and reliability that may be used to evaluate and quantify the soundness, suitability, and performance of mine tailings facilities. These terms have been used, some consistently, some irregularly, some for a long time, some just recently. It is therefore worthwhile re-examining them in the light of recent tailings facility failures and new guidelines and regulations for tailings facility design, construction, operation, and closure.

1 INTRODUCTION

The factor of safety has conventionally been used as the measure of stability or satisfactory performance of a tailings facility. A factor of safety of at least 1.5 for static conditions came to be considered reasonable, although some groups accepted a factor of safety of 1.3. With the advent of computer slope stability analyses, it was easy to calculate a factor of safety, although it was often difficult to say exactly what was implied by a specific factor of safety.

With the acceptance by the geotechnical community of statistical theory and methods, the idea of the probability of failure of the tailings facility came to be used by some, for example see Steffen (1978). In the early 1980s, the senior author of this paper begun applying the concept of the risk of failure to uranium mill tailings facilities, for example see Van Zyl and Robertson (1987).

Recent failures of major tailings facilities including Mount Polley (2014) and Samarco (2015) have prompted us to adopt and apply other concepts and measures to describe and quantify the performance of tailings facilities. Accordingly in this paper we consider terms including reliability, redundancy, resilience, and robustness as they may be used in judging the safety and performance of a tailings facility.

2 ENGINEERING JUDGMENT

Long before the advent of sophisticated soil sampling, precise laboratory testing, and the power of computers to calculate, Terzaghi and Peck put engineering judgment as the premier art to be practiced by the geotechnical engineer. Ralph B. Peck (1969) noted as follows regarding engineering judgment:

- The successful practice of engineering requires a high degree of engineering judgment.
- Judgment involves and requires a "sense of proportion."

- Theory and calculations are not substitutes for judgment---but are the basis for sounder judgment.

Thus the terms we discuss in this paper are ultimately intended to provide a basis for sounder judgment. The terms will never be substitutes for judgment.

Morgenstern (1995) suggests that the most powerful tool to exercise judgment regarding risk and robustness is the observational method. He writes:

> "Risk in geotechnical engineering is best managed by diligent application of the observational method. The observational method is a form of consequential risk analysis that emphasizes robustness, the capacity for intervention and adaptability. These are important consideration to enhance the reliability of engineering system. Where circumstances preclude the application of the observational method in its strictest form, consequential risk analysis still provides the most effective guidance for risk management."

3 DEFINITIONS

The following are definitions and a brief discussion of the basic ideas associated with some of some of the terms use in this paper as applicable to tailings facilities. All these terms are in common use in many other fields besides the engineering of tailings facilities, and accordingly there may be overlaps, or gaps, and certainly differences of meaning as applied say in medicine as compared to tailings facility engineering.

3.1 *Risk*

The product of the probability of a hazard occurring multiplied by the consequences. This definition has been stated many different ways by many different users, and there are many simple and complex methods for describing or quantifying the basic aspects, namely hazard, probability, and consequence.

There is however little general consensus on precisely how to undertake a risk assessment of a tailings facility and there is less general agreement as to what constitutes an acceptable risk for the common hazards that may arise in the context of tailings facility engineering. (See Caldwell and Crystal, 2017).

3.2 *Reliability*

The ability of a system to function under stated conditions for a specified period of time. Reliability is quantified as:

Reliability = 1 – Probability of Failure. (Wikipedia.)

3.3 *Redundancy*

Redundancy has been defined as "approaching analysis, design and construction in such a way that if one element, whether physical or human fails to function in the way intended, other elements take over in such a way that the structure will still function essentially as intended." (Osterberg, 1989).

3.4 *Resilience*

There are many definitions that have been updated as thinking on the topic develops. Here are some:

> The intrinsic ability of a system to maintain or regain a dynamically stable state which allows it to continue operations after a major mishap and/or in the presence of continuous stress. (Hollnagel et al. 2006).

> The ability of a system to absorb or avoid damage without suffering complete failure. (Wikipedia)

> A system is resilient if it can adjust its functioning prior to, during, or following events (changes, disturbances, and opportunities), and thereby sustain required operation under both expected and unexpected conditions. (Hollnagel. 2016)

3.5 *Robustness.*

Robustness in the context of tailings facilities has been defined as "reducing the likelihood and severity of failure through a combination of Factor of Safety and redundancy." (Crystal and Smith. 2016)

Risk and reliability can both be quantified (or at least numerically specified). There are no agreed measures or quantification of resilience or robustness. Only engineering judgment enables a professional engineer to gauge or assess resilience or robustness.

4 A RESILIENT SYSTEM

Last year, in Vancouver, at the conference Risk and Resilience these concepts as applied in mining were broadly discussed. The following are some of the ideas stemming from that conference on the application of the idea of resilience to tailings facilities.

The simplest definition of resilience useful when considering a tailings facility is this:

> The ability of a system to maintain a stable state which allows it to continue operations in the presence of continuous stress.

While we cannot quantify resilience in modern terminology and application to tailings systems, it is reasonable to postulate the following guidelines for establishing difference categories or measures of resilience:

- High resilience: The perimeter embankment is low, the sideslopes flat, the crest wide, the beach broad, and the pool far from the embankment. Judgment would conclude that such a system is robust and reliable.
- Moderate resilience. The perimeter embankment is getting higher, the crest is being narrowed, the sideslope being made steeper, and the pool is getting closer to the embankment. Judgment would conclude that this system is beginning to become less robust and certainly its reliability is decreasing---or put another way, its probability of failure is increasing.
- Low resilience. The perimeter embankment is high, the crest narrow, the sideslope very steep, the pool is lapping against the upstream side of the embankment. Judgment must lead to the conclusion that this system is not robust, not reliable, and probably has a high probability of failure regardless of the calculated factor of safety.

5 PROFESSIONAL RESPONSE TO REDUCING RESILIENCE

Prudent professional engineering practice and the exercise of professional judgment would surely lead a reasonable engineer to exercise greater vigilance in the light of reducing resilience.

As Morgenstern notes, this could include implementation of the observational method—essentially increasing the number of observations of the state of the tailings facility, analyzing the data, and comparing the result to predetermined acceptable or unacceptable performance.
Thus the issue arises: at what point does failure to increase vigilance of a system undergoing reducing resilience, constitute professional negligence?

6 CASE HISTORIES

The following discussion of some case histories, provides an opportunity to consider the application of the ideas of this paper in tailings facility engineering. Nothing said below is intended to contradict or negate the many fine investigations that detail the causes of the failures. All that it intended is to set out additional tools for thinking about tailings facilities, and applying good judgment in their design, construction, operation, and closure.

6.1 *Bafokeng*

In 1974 in South Africa, the Bafokeng tailings facility failed when the pool approached the perimeter. The precise cause of failure is still debated. But as is relevant to this paper, we note that the technicians operating the dam, and the engineer who inspected the dam some two weeks before its failure, had no concerns about stability. They were operating on the basis of judgment established by the past successful operation of literally hundreds of upstream tailings facilities in the preceding hundred years in South Africa.

They had no samples of the foundation clay, they had no measure of its strength, and they had no calculated factor of safety. Certainly they had not done a formal risk assessment, nor were they even remotely aware of the potential hazard involved in a rising pool of water directly adjacent to the upstream crest over varved tailings, i.e., with interlayering of sand and slimes.

Recall that the official cause of failure was determined to be piping of the sand between adjacent slime (clay) layers. Some writers (Blight. 2010) have postulated that the failure was static liquefaction –see below regarding the failure of Samarco.

In the four years following the Bafokeng failure, the junior author dealt with slope failure at three identical platinum tailings facilities. The only difference was that there was no water on these three dams. He and fellow engineers established that the low strength of the weather norite clay (ten degree with a plasticity index of over eighty) that constituted the foundation, was the primary cause of slope instability. They were left wondering if plain old-fashioned, weak foundation sliding was not the cause of the Bafokeng failure.

These alternative causes of failure were commonly discussed amongst tailings engineers in South Africa at that time. The sad lesson learnt is that this case history never affected international standards of practice or influenced the judgment of most North American engineers. Had this happened, we believe that neither Mount Polly nor Samarco would have failed.

Looking back from the advantages of today, we now know that at failure Bafokeng was not robust, resilient, or safe. It had no redundant features, although the new, currently operated No. 4 dam does have an important redundant perimeter buttress embankment.

6.2 *Ekati*

About eighteen years ago, the design of the Ekati Mine, Northwest Territories, Canada Long Lake Containment Facility was prepared. The facility is still in operation, although getting full. It is the most robust facility we know of. As described in McKenzie et al (2011), four rockfill embankments form five separate cells. Only the upper three cells are used for deposition of fine processed kimberlite, although the forth cell is permitted to receive fine processed kimberlite. The upstream face of the upper three embankments include filters that allow supernatant to pass through the embankment but retain the fine processed kimberlite.

The point is this facility is: robust—big throughflow embankments; redundant—downgradient cells permitted to receive errant tailings; and resilient—the embankments move to accommodate foundation loading and tailings loading. These never were the design criteria. We postulate they are the result of good judgment in the absence of accurate topography, no soil samples, no laboratory testing, and no computers. The lesson learnt is that today we are lucky to have foundation soil samples, laboratory test data, and computer codes—but they are no substitute for judgment founded on conservative practice.

6.3 *Mount Polley*

There is little more to say about the failure of the Mount Polley tailings facility. Nevertheless, we note below some events or conditions that may be said to have reduced the robustness and resilience of the facility. Maybe the lesson learnt is that if these conditions had stimulated the judgment of the engineers, the failure would not have occurred.

The original design of the Mt Polley facility included underdrains upstream of the embankment; the underdrains were intended to preclude the presence of saturated tailings behind the embankment. This good original design feature appears to have been forgotten or ignored, and the pool was allowed to lap up against the upstream crest of the embankment.

The sideslope of the embankment was steepened to almost 1.4:1 to about 1.3:1. It is one of the simple, well-known facts of slope stability that waste rock, dumped at its angle of repose, will form a slope of about 1.3:1. If this dump is on a strong foundation soil of bedrock, the factor of safety of the slope is 1.0 -- in spite of this low factor of safety, the probability of failure is low. Mines all over the world happily accept this condition.

When, however, the foundation conditions are not well established or may include relatively low strength soils, a rock slope at 1.3:1 is essentially on the verge of failure. One must ask: did this not alert the engineers to a need for greater conservatism?

6.4 *Samarco*

The simplest explanation of the failure of the Samarco tailings facility is this: (a) slimes layers near the perimeter of the upstream embankment moved sideways as a result of the increasing tailings load; (b) the tailings sand between the slimes layers also moved sideways along with the slime; (c) the density of the tailings sand decreased as it deformed; (d) the sand skeleton collapsed on itself, thus causing sand liquefaction; and (e) the liquefied sand and the pool of water flowed down the valley to the sea.

This is called deformation-induced static liquefaction. As noted above, some believe this may have been the cause of failure at Bafokeng.

Regardless, the design of the Samarco dam must surely qualify as one of the least conservative we know of. The design epitomizes optimism over judgment; an astounding lack of interest in the history of failure of tailings facilities; disregard for over fifty years of lessons learnt in other jurisdictions; and a failure to provide for proper operation of an admittedly dicey, dangerous facility. That they left the village beneath an upstream tailings facility only proves the magnitude of design incompetence and veniality. Had they never heard of Merriespruit?

The same attitudes that characterized the design, characterized the operation. They did not build as designed; they made changes without consulting experts or the original designer; they ignored peer reviewers—who seem to have been silent during the many years of poor operation. That twenty-seven people face criminal charges of homicide simply reinforces what we say here.

The most astounding fact is that even the failure of the Mount Polley tailings facility seemed not to have alerted anybody to change their ways at Samarco. For just a little engineering awareness and judgment would have alerted even a casual observer to the need for most conservatism, robustness, resilience, and a higher factor of safety.

6.5 *Some Observations*

At the time of the Mt Polley and Samarco failures, the concept of resilience was not common in the industry. In hindsight the concept of resilience serves well to capture the many factors that went into the negligence of some of the engineers involved.

But at the time of failure the concepts of robustness, reliability, risk and appropriate factors of safety were well known and had long been applied in tailings engineering. The need for judgment based on doing all the measurements, interpreting the results, and understanding the implications was as old as geotechnical engineering itself.

So even if some of the engineers were ignorant of the concept of resilience, they should not have been ignorant of the many other similar and related concepts and practices. But they appear to have been ignorant of more than the great philosophical concepts that underlie the art of geotechnical engineering. They appear to have been ignorant of the fundamental principles of geotechnical engineering, including elementary slope stability, seepage in porous media, and the fluidity of wet tailings.

7 CONCLUSIONS

A tailings facility is ultimately a work of art. It may be a great, successful work of art. Or it may flow down the river in disgrace to obscurity. Like the great works of art of the past built of earth, the best will endure for a very long time. Here are some that set the challenge for future tailings facilities (see links to relevant websites in references):

- The Cahokia Mounds of Illinois that were constructed of compacted soils in about 1,000 AD and which still stand stable today.
- The Silbury Hill Mounds in England that were constructed over 3,500 years ago and which are still stable and a tourist attraction.
- Chavin de Huantar, Peru which was constructed between 2,500 BC and 500 BC. The rock mounds and rock structures are still stable.
- The terraces of Carmen Salcedo-Andamarca in Peru that have been used for farming for over 2,000 years.
- The 24 piles of the United States Uranium Mill Tailings Remedial Action (UMTRA) Project which involved stabilization of 24 piles in 10 states in accordance with the criterion that they be stable for 1,000 years to the extent reasonable and at any rate for 200 years. Work was done from the mid-1980s to the mid-1990s. (US Academy of Science. 2007).

The point is the art of creating great tailings facilities has been done, can be done, and must be done. Key to future success is the exercise of professional judgment founded on the many modern engineering philosophies, methods, and tools now at our command.

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Mine closure risk assessment – A living process during operation

Cristián Marambio
Golder Associates S.A., Santiago, Chile

ABSTRACT: Mine closure increasingly important, as mining companies recognize not only the needs for regulatory compliance, but also the importance of effective closure in maintaining social license for the development of new operations. To demonstrate to communities and other stakeholders that the risks inherent in mine closure are well-dimensioned and controlled is critical. The risk assessment process is therefore tremendously important, nevertheless, it can be hampered by a lack the information needed to measure and evaluate correctly the risk scenarios.

The following sections provide an overview about how mine closure risk assessment involves understanding facilities design criteria, and how they were constructed and operated, as these set the basis of how it will behave in the long term. This paper will then highlight the importance of gathering and managing operational data, as it will determine how the closure risk assessment team is going to understand long term behaviors.

1 INTRODUCTION

Understanding mine behavior after closure is a key factor in determining the actions and closure measures required to achieve a “walk away” closure. Without good data and a good understanding of site conditions and past performance, the understanding of closure risks and the needed closure measures is likely to be inadequate.

Closure risk assessment starts with the identification of significant failure modes and their associated consequences in terms of safety, environmental, or financial risk. Then, potential controls to mitigate those risks are also identified. During this process, the existence of reliable data and the understanding of the potential failure modes and how they affect the environment, can lead to an efficient and successful closure.

For the best outcomes, ‘closure’ should be taken into consideration throughout the mine life, i.e. mine planning, design and operation. Techniques and considerations can be included in the design of mine facilities to significantly simplify the actions required at closure. Closure considerations should be part of the design criteria of mine facilities.

In the same way, there are several kinds of data that can be gathered and managed considering closure aspects, helping to understand and predict mine behavior after closure.

The following sections of this paper provide an overview of how mine closure behavior uncertainty may be reduced by incorporating mine closure risk assessment processes into the mine operation, particularly in terms of collecting data that will support risk assessments. It will include examples of operational data that may be recorded during operation and how this contributes to the mine closure assessment. Finally, the consequences of uncertainty in site data for the closure assessment will be identified.

2 CLOSURE RISK ASSESSMENT

In general terms, the closure risk assessment is the process where potential failure modes are identified (in this case in the context of mine closure), with their significance determined based on the combination of their probability of occurrence and their consequences.

Closure risks are formed by three components: the event or potential failure mode, its related probability of occurrence and the level of the impact generated.

For mine closure, the events typically are unusual situations that can be triggered by the occurrence of a natural event (i.e. rainfall, earthquake, wind, or flood) or by the nature of the facility (i.e. static failures, uncontrolled increase in settlements, or over-stressing). The event, however, has to be understand not only as the event itself, but as the combination of the triggering situation (i.e. natural event), its impact (in terms of the magnitude), and the probability of occurrence of their combination (as a certain event with a certain magnitude has a specific probability of occurrence).

For example, if an event is defined as the failure of a tailings dam due to an earthquake, which reaches a village 10 miles away from the toe of the dam; to determine the probability of occurrence of this failure mode, some considerations has to be taken into account:

- The earthquake has be of sufficient magnitude to trigger a dam failure, which should mean that the event exceeds the design criteria (or the resistance of the structure as constructed, independent of the original design); and
- The failure has to be sufficient to result in the consequence of concern. In this case, the released tailings have to be able to be spread 10 miles away (implying a certain fluidity of the tailings).

Both of these aspects interact in the estimation of the event probability. First, the earthquake has to be more severe than the one considered in the design (or effectively resisted by the dam as constructed). This severity has an associated return period and therefore probability of occurrence. Also, that the tailings in closure retain the water content required for them to be spread 10 miles away has its own probability of occurrence. Therefore, the probability of occurrence of the whole event becomes the combination of both the probability of occurrence of the earthquake and the probability of having certain level of saturation in the tailings. The closure risks assessment therefore has to consider the probabilities that link an event to a certain level of consequences, taking into account how both are related.

Generally, closure risk assessment methodologies define the failure mode as a combination of its probability of occurrence and the severity of its consequences. However, the methodology to quantify both factors is not necessarily defined, leaving quantification to the criteria of the evaluation team. This quantification will be determined largely by the quality of the information available or managed by the team. In this regard, the understanding of the facilities under evaluation, the environmental components that are involved (wind, surface water, underground water, materials involved, materials conditions, etc.), and how they behave after closure is fundamental for a correct and realistic closure risk evaluation. Closure risk assessment has to be considered as an analysis with context and history. What if the evaluation team does not have information about how the facility was designed or operated? Probably they will have to make conservative assumptions, which may lead to closure measures that are over dimensioned or otherwise incorrect, and excessively costly. If the evaluation team knows about the design criteria, but does not realized that those criteria were never met during construction or operation, they may significantly underestimate closure risks and, therefore, underestimate the needs for closure measures and their costs.

Part of understanding closure behavior and the real significance of closure risks is to have the information and the data required to project different events to a post closure scenario.

3 OPERATIONAL DATA RELATED TO CLOSURE

When closure risks need to be determined, being able to identify the triggering events, their probability of occurrence and how the events impact on sensitive receptors (environmental components, people health and safety; and stakeholders) is a critical factor.

In order to achieve the required level of understanding, the following must be taken into account:

- Design criteria: knowing how the facilities were designed, will put the evaluator into a solid starting point. In recent years, the concept of 'Design for closure' has gained strength, and has helped to reduce closure costs and optimize operations. Knowing, for example, that the maximum credible earthquake (or MCE) has been considered in the design of a facility helps to understand that the probability of occurrence of a failure related to dynamic conditions is much lower than for a facility where only operational-magnitude events have been considered in design. In the same way, if during the design phase of a project the groundwater water model has considered post closure conditions, the scenarios related to groundwater water will be better dimensioned, with fewer unknowns related to their management. With fewer unknowns, it will be easier to estimate credibly the probability of occurrences and the dimensioning of level of impacts for associated risks.
- Operational data: understanding facility design is the starting point in a closure risk analyses. However, knowing how facilities were operated, constructed and monitored is even more important as it indicates the real state and the real behavior of the facility. Although planning and design set expectations for how the facilities will behave under different conditions, is not until they are constructed and operated when real responses can be measured (monitoring) and extrapolated forward potentially more extreme scenarios. It is much more relevant, for example, to have a numerical model fed with measurements of the quantity and quality of the infiltration coming from a tailings impoundment after a measured rainfall event, rather than the best simulation done without any site data. Therefore, while the design criteria sets the basis for closure risk analyses, the operational and monitoring data gathered during the operation of the facilities plays a fundamental role into the risk assessment.

A great deal of the information measured and stored during the operation of mines can be useful for closure risk assessment. Some of the relevant information for different types of facilities is listed below as an example:

- Pits:
    - Dewatering flow and how they evolve during the pit excavation;
    - Pit walls monitoring, considering water appearances, cracking, deformations, local failures, among others;
    - Existence of units with ARD potential;
    - Water quality in any pit lakes that form during mine life, and its evolution over time;
- Underground mines:
    - Level (flux) and quality of water infiltration;
    - Monitoring of subsidence during mine life;
- Waste dumps:
    - Geology of the material deposited on the surface and slopes;
    - ARD or any other type of impacted seepage;
    - Seepage quantity produced during/following different rain events;
    - Monitoring of deformation, settlements or cracking;
    - Grain size distribution of the material deposited in the dump;
    - Downstream monitoring of water quality;
- Tailings deposits:
    - Geochemical characterization of the tailings (total tailings, slimes or sands if applicable);
    - Tailings dam geotechnical monitoring: deformations, compaction level if applicable, water table, drainage water quality and flux during operation and during/following heavy rainfall, seismic response (deformations, settlements);
    - Tailings beach slope monitoring;
    - Air quality monitoring during operation;
    - Downstream groundwater quality monitoring.

The information indicated here are examples of operational measurements that can support a closure risk assessment, and provide an evaluation team with solid data to support projections over the long term.

Data management is a key challenge. Operational information may exist in servers, but be distributed over many folders and years of operations, and operators may change or leave the mine (especially as it approaches closure), resulting in a loss of continuity and access to the information. Or the information may simply have never been collected or properly stored in the first place.

When this happens, the evaluation team will not have the same basis for projections, and maybe be forced to assume (unduly) conservative scenarios and considerations for the risk assessment, and also for the closure measures definition. This may lead to important misunderstandings of the real problems, and the risk of making the wrong decisions.

4 CONSEQUENCES OF UNCERTAINTY AT CLOSURE

We have recently see the Mount Polley tailings dam failure on 2014. The independent expert engineering investigation and review panel concluded that the dominant contribution of the failure resided in the design. The design did not take into account the complexity of the sub-glacial and pre-glacial environment associated with the perimeter embankment foundation, and, as a result, foundation investigation and associated site characterization failed into identify a continuous GLU layer in the vicinity of the breach and to recognize that it was susceptible to undrained failure when subject to stress associated with the embankment (Government of British Columbia, 2015).

In 2015, the Fundao Tailings dam in Brazil also failed, spreading tailings many miles away from the tailings deposit. In this case, the original design concept of the Fundao Dam employed an unsaturated sand zone to support the weak slimes zone. Unsaturated sand is not amenable to liquefaction and hence the original design was robust in this regards. However, difficulties were encountered in executing the design and a modified design was put forward and adopted. As part of this modification, a change in the design concept was adopted and saturated conditions were permitted to develop in the sand. As the sand tailings were deposited by hydraulic means, it resulted in loose conditions and a growth in saturation, generating the conditions needed to develop liquefaction, resulting in the flow slide (Cleary Gottlieb Steen & Hamilton LLP et. al. 2016).

Even though these cases were not closure scenarios, it was the misunderstanding of the facilities, its designs, and in the case of Mount Polley the foundation; that generated the conditions needed to produce the failures. These are examples of scenarios where only considering how facilities were originally designed would produce a misunderstanding of the real risks (i.e. Mount Polley design did not considered the glacial and pre-glacial environment, and Fundao dam design changed during operation).

Misunderstanding of the potential risks can lead to making wrong decisions based on over- or under-valuing the consequences and probabilities. Moreover, misunderstanding variables involved in closure could lead to not identifying and evaluating relevant risk scenarios, with potential serious consequences.

Minimizing uncertainty to the degree possible is closure goal for proper data collection and management during operation. This can help control not only risks, but also costs as closure provisions can be better dimensioned. Uncertainty can produce significant cost overruns during closure, as new closure measures need to be designed or the consequences of unwanted events rehabilitated.

In some jurisdictions, sites are not transferred to the regulatory authority until a complete rehabilitation is achieved, which involves years of post-closure monitoring (environmental and social), and development of confirmatory studies. In this regard, adequate dimensioning of the post-closure period is needed to develop an accurate estimate of costs for post-closure. Again, good quality operational information can provide the needed data for better estimates of post-closure requirements.

Reduced closure uncertainty also provides benefits for the relationship with communities and stakeholders. Increasing involvement and empowerment of community stakeholders has resulted in an increased expectation to be well informed about site closure and post-closure. Therefore, reducing closure uncertainty and demonstrating that the risks inherent to mine closure are controlled, well-dimensioned, and well-managed, is very important for gaining community support and acceptance of both mining projects and their eventual closure.

Managing closure as a living process during the mine operation provides benefits for reduced uncertainty in closure and long term planning, as well as the for the relationship between mining companies and their stakeholders.

5 CONCLUSIONS

As mentioned in the introduction of this paper, understanding mine behavior after closure is a key factor in determining the actions and closure measures required to achieve effective closure solutions. Closure risk assessment is based on the identification of failure modes, their related probability of occurrence, and the level of the impact generated by them (consequences). Being able to determine those three variables is fundamental for developing a successful assessment. And a successful assessment will lead to well dimensioned closure measures, post closure actions and a clearer understanding of the required timeframes. In order to do so, uncertainty needs to be controlled and minimized, with analyses based on the most detailed information practical, including both design criteria and information gathered during operation.

Design criteria are often available, but they should be just the starting point for closure risk analysis, as they only represent how facilities have been planned – not how they were constructed or operated, which is what really controls their behavior in the long term. The construction and operation of mine facilities typically bridges many years, meaning that the information generated can be amongst different groups and persons. This can generate a disaggregated source of relevant data. In this regard, knowing which types of information are relevant for closure needs, and keeping organized records can be critical for the closure risk evaluation team, helping them to generate a full understanding of how the different facilities behaved under operating conditions, and make better projections of long-term performance under closure conditions. This is a key element in achieving confidence in the closure risk assessment and closure designs.

Successful closure has to be understood as not solely a low-cost closure. A successful closure is also an activity that was developed in close alignment with how it was planned – with relatively few surprises. This may not always mean the lowest costs, but does mean lower risk for the owner. In summary, a successful closure can be understood as:

- A process that starts with the facilities design and continues through its construction and operation;
- A process that is planned and understood as a continuation of the operational life;
- A process that is typically going to be defined and carried out by professionals who were not throughout the full life of the facility. In this regard, how information is gathered and managed will determine how the closure risk assessment team is going to understand the facilities and project their long term behavior;
- A process that needs to achieve a certain level of accuracy, as it is involved in the company's finances (closure provisions) and legacy (stakeholder's opinion), as well as continued social license for operation.

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The role of the "Responsible Person" as a key control for mitigating risks related to management of Tailings Facilities

C.A. Iturralde
Norwest Corporation, Vancouver, BC, Canada

ABSTRACT: This paper highlights the importance of the "Responsible Person" as one of the most important "controls" that mining operations have for mitigating risks related to the management of their Tailings Storage Facilities (TSF's). Some other key "risk controls" recommended by the Mining Association of Canada (MAC) in their 2017 edition of the "Guide to Management of Tailings Facilities" include: Engagement of the Engineer-of-Record (EoR) and Independent Reviewers; implementation of a risk-based tailings management system; development of a quality OMS manual. These controls are all considered to be Best Available Practices (BAP's), and their proper implementation and integration into any site's tailings management system is highly dependent on the "Responsible Person".

More specifically, this paper presents details on the relationship and interaction between the "Responsible Person" for the tailings management system at a site and some of the other risk controls recommended by MAC. Also, the typical role of the "Responsible Person" through the various phases of a mining operation is discussed briefly in the last section. This paper aims to improve the general understanding of the role and benefits of engaging a qualified "Responsible Person(s)" as an important risk mitigation tool to support with the safe and responsible management of TSF's.

1 INTRODUCTION

1.1 *General*

Each Tailings Storage Facility (TSF) presents unique characteristics and challenges. Costs for implementation and operations of a TSF depend on the size and technical complexity of the project, and on the regional social and environmental framework in which the mine is operating. In general, development of a tailings facility can require early stage investments ranging between $50 Million to $100 Million dollars or higher. These are significant investments that mining companies progressively makes over the life of the mine with the goal to manage mine wastes being generated in their operations safely and responsibly.

The mining industry, however, has frequently lagged in the implementation of proper management systems to manage their tailings related projects and investments. For a long time, highly qualified professionals have been engaged to properly execute projects of similar magnitude in other key areas of the mine, including the process plant and ore mining operations. Projects related to tailings management, however, were not getting that the same attention, leading into some well-known cases of poor performance in recent years.

In recognition of the issue, and considering the increasing complexity of mining projects and their regulatory environments in the various jurisdictions around Canada and the world, the industry has been working towards further improving their performance with regards to tailings management. The Mining Association of Canada (MAC), in their third edition of the "Guide to Management of Tailings Facilities" recommends having a "Responsible Person" (RP) in each mine operation to adequately manage its tailings facility. In a broad context and not being prescriptive, MAC refers to the RP as a person working for the mine with the appropriate qualifications and experience, and capable of taking responsibility over the various phases of a TSF, including the construction, operations, expansion and closure phases of the facility. As such, the RP will identify scope of works and budget requirements (subject to final approval) for all aspects of tailings management systems (MAC 2017). MAC's tailings management guidelines also allude to some mining companies having RP's participate at a corporate level. Under such a scenario, the RP will delegate specific tasks and responsibilities to other qualified personnel at each site under his/her responsibility.

1.2 *The "Responsible Person" as a risk control*

From a risk management perspective, the RP is an important control to reduce risks associated with a TSF. In his/her capacity, the RP can identify, evaluate and prioritize risks affecting the various aspects of the tailings management system, and develop and implement mitigation strategies to control these risks. By following best management practices, and delegating specific tasks and responsibilities to qualified personnel, the RP can allow for timely implementation and execution of the various designs, construction, operational, maintenance, and surveillance activities required for proper operation of the TSF. For that, the RP will work and train site personnel in the TSF area, support the EoR, and engage specialized contractors and consultants required to complete a quality job.

Depending on the sophistication of the mining operator and its existing management systems, the RP will also support, and in some cases guide, the development and implementation of the tailings management systems for an operation. This may include supporting the development and implementation of some of the following aspects for the TSF area:

- A risk-based approach management system;
- Critical controls and operational controls, such as the Operation, Maintenance, and Surveillance (OMS) manual and Emergency Response Plan (ERP);
- Continuous improvement and assurance processes, including engagement and definition of scope for the EoR, and
- Set-up and completion of regular independent review processes and management systems.

Application of these risk controls and Best Available Practices (BAP) for the TSF area are recommended by MAC (2017), and these should be applied through the life cycle of the facility.

2 THE RESPONSIBLE PERSON AND OTHER RISK CONTROLS & BEST PRACTICES

2.1 *The "Responsible Person", the Accountable Executive Officer, a risk-based management approach*

As per MAC (2017), the overall accountability of the tailings management systems at a mine relies with the Accountable Executive Officer for the TSF area and the site. The Accountable Executive Officer is typically designated by the Board of Directors of a mining company. A manager representing the Accountable Executive Officer at a site is responsible for engaging a qualified RP for management of the site's tailings management systems and the TSF area. The RP is responsible for the adequate implementation and monitoring of the tailings management systems during all phases of the TSF area to allow for a safe and responsible operation.

MAC, and several other institutions in major mining jurisdictions around the globe, recommend implementation of a risk-based tailings management system. A risk-based approach requires of completion of risk assessments and identification, development and implementation of operational controls and critical controls to adequately mitigate and manage occurrence of potential unwanted events and associated consequences. The RP for a TSF is typically responsible for proper implementation and monitoring of the field components of a risk-based tailings management system, including implementation and development of field-based operational controls and critical controls. To accomplish this task, the RP works in coordination with the EoR, mine staff supporting TSF operations, and other specialized consultants, including the IR's and contractors, to identify and evaluate potential risks and critical controls related to the TSF performance. Once risks are identified and properly evaluated, the RP becomes responsible for assigning tasks and responsibilities to staff and contractors working in construction, operations, maintenance and surveillance activities for the TSF area. These tasks are aligned with the timely implementation of the identified critical and operational controls.

RP's in a corporate role, or working for smaller mining companies with a less-established tailings management framework, might be tasked by the Accountable Executive Officer with the responsibility of the initial development, in addition to its implementation and monitoring, of a risk-based management system(s) to serve one or various of the mining company's operations. This role requires engagement of a highly qualified professional with several years of extensive tailings engineering and management experience.

2.2 *The "Responsible Person" and other site personnel and contractors*

The RP is responsible for having the required site personnel and contractors engaged for proper implementation and operations of the TSF area. All personnel should be trained accordingly to perform their tasks in a safe and responsible manner. This training should include overall site safety training, and general and specific concepts related to their activities in the TSF area, including aspects of the TSF's design, operations, construction, maintenance, and surveillance of the TSF area, including some closure and post-closure concepts. Resources available to the RP for training of his/her staff are: the OMS manual, ERP and related protocols developed by the mine site; operating and maintenance instructions provided by vendors for the various equipment; and designs reports and construction drawings and specifications generated by the design engineer and other specialized consultants.

The RP relies frequently on personnel and contractors working for other departments of the mine, such as (not an all-inclusive list) process plant supervisors and operators, environmental supervisors and technicians, mine surveying personnel, and mine and maintenance technicians, equipment, and operators. It is important for the RP to have these personnel trained accordingly to perform their assigned tasks in the TSF area. The RP should retain the authority over the TSF area and be able to stop staff from working and entering the TSF area if he/she considers that some of these personnel do not have an adequate level of understanding and training on the proper operations and execution of the tasks in the TSF area. Thus, the RP should clearly identify to other department's managers and supervisory personnel, the skills and training requirements for staff working in the TSF area. Also, the RP should have the authority to review the work performed by these support staff in the TSF area, and in coordination with the other department managers and supervisors, request timely completion of activities meeting required construction and operational specifications and quality criteria.

2.3 *The "Responsible Person" and the EoR*

Together with the RP, the Engineer-of-Record (EoR) is an effective control to reduce risks related to the management of TSF's. The relationship between the RP and the EoR of a TSF is arguably one of the most important ones to allow for an adequate management of the TSF area at a site. These two entities working in close collaboration can provide the mining company with technically sound and pragmatic proposals and solutions for best management of their TSF's, taking into consideration the mine's operational and regulatory requirements, design criteria, construction and financial aspects related to the project.

Engagement of an EoR is recognized as one of the best management practices for a TSF. The EoR should be a registered professional engineer with proper qualification and ample experience in the field of tailings management systems. This qualified professional should confirm their availability and capacity to support the client with the required services. Typically, the design engineering firm, awarded to complete the designs for a TSF area, acts as the EoR for the TSF once construction begins and then into operations. In some less frequent cases, the EoR is engaged at a later stage in the operation by following a procurement process, typically led by the RP for the TSF, or by direct order provided by the Accountable Executive Officer. Following this approach, however, carries some additional risks that the Accountable Executive Officer should properly evaluate and weigh with input from the RP and in consultation with the Independent Reviewers and site managers.

The RP will frequently act as the Owner's representative to the EoR. The EoR role is to provide technical direction to the Owner. As such, the RP will work with the EoR on annual scopes of works budgets, and schedules for deliverables. Depending on budget approval by the mine's Accountable Executive Officers, the RP will work with and support the EoR to allow all technical input to feed into the site's risk-based tailings management systems. The goal is to always focus on the safe performance of the facility. At the same time, the EoR should allow for sufficient flexibility in their designs and technical recommendations to allow for the client to accommodate unforeseen changes and variations that are typical of mining operations. Hence, engaging an experienced and knowledgeable EoR is a must for any TSF, and should be properly sought by the RP with support of the Accountable Executive Officer.

A few larger mining companies, such as some of the largest operators in oil sands, have opted for internalizing the EoR role by directly hiring qualified registered professional engineers to perform the EoR's role and activities. This alternative simplifies the role of the RP by having the EoR form part of the same company team managing the TSF. This approach, however, might only be applicable in large operations with very large or multiple TSF's and for the lesser risk structures. Consideration should also be given to the local regulations since this might not be an option in some jurisdictions.

2.4 *The "Responsible Person" and the development of a quality OMS Manual*

Development of a quality OMS Manual, and its regular updating, is the responsibility of the RP for a TSF. The main purpose of an OMS Manual is to provide staff and contractors working in the TSF area with the most important operating criteria and documentation for safe and responsible operations of the various infrastructure and systems located in the TSF area. At the same time, the OMS Manual serves as a reference document for site managers, safety and environmental personnel, and other supervisory staff having some accountability or responsibility in the TSF area. A quality OMS manual will identify and present the operational and critical controls for the TSF area. These controls are intended to allow for a safe and responsible operation by clearly communicating regular operating practices and allowing for proper evaluation and continuous management of some of the high consequence risks for the TSF area.

The OMS Manual for the TSF should be owned by the RP. The RP can refer to MAC's guide for Development of an OMS Manual published in 2011 (MAC, 2011) for development and updating of a site-specific OMS Manual. Alternatively, the RP can rely on additional professional and operational staff familiar with the TSF area for preparation of the OMS Manual and regular updates to this document. External consultants can support by providing an initial general framework for the OMS Manual or performing an overall upgrade to the document following most up-to-date BAP's. However, the RP and the operational personnel working in the TSF area remain responsible for completing and finalizing this site-specific document. The final annual draft of the OMS Manual should be approved by site managers and supervisory personnel with accountability and/or responsibility over the operations, maintenance, and surveillance of the TSF area.

The OMS Manual for a TSF area should be reviewed and updated at a minimum once a year to verify that the information presented within it is still accurate and considers all most up-to-date information on the facility and best management practices. The RP might consider updating the OMS Manual more frequently if the facility experiences significant changes within the operating year, such as expansions or additions of new monitoring systems or infrastructure; changes from operating phase to care and maintenance or closure phases; significant increases or reductions in the mine's production rates, changes in legislation significantly affecting regular operations of the TSF area and related infrastructure, or changes in resources availability, including staff, equipment, and financial resources. The most up-to-date version of the OMS Manual should be circulated for feedback and input with the EoR during his/her annual inspection of the TSF. The EoR will verify that operating, maintenance, and surveillance procedures presented in the OMS Manual allow for a safe and responsible operation, and are aligned with the design of the TSF and its performance criteria.

2.5 *The Responsible Person and the Independent Reviewer(s)*

The Independent Reviewers (IR) are typically engaged by order of the Accountable Executive Officer and in coordination with the Company's Board of Directors. The purpose of the IR is to perform an independent technical evaluation and provide an "un-biased" opinion to the Accountable Executive Officer on the overall technical and management performance of the mine's tailings management systems. The Accountable Executive Officer might rely on information provided by others Executive Officers and qualified technical professionals such as the EoR and the RP to decide who is to form part of the IR.

The RP supports the work of the Independent Reviewers (IR) by providing key and up-to-date design, technical, construction, and operational documentation generated for the tailings management area This can include results of latest risk assessments completed for the site, annual EoR inspections reports and any dam safety reviews reports that might have been completed, recent and historical design and as-built documentation, latest versions of OMS manual, site surveillance and maintenance reports and instrumentation monitoring reports, etc.

Depending on the role and the experience of the RP in charge of the tailings management systems of a mining company, the RP, in coordination with the Accountable Executive Officer and input from the EoR, might be tasked to complete the scope of work, estimate budgets and schedule for deliverables, and engage the IR's (pending approval by the Accountable Executive Officer).

3 ROLE OF THE RESPONSIBLE PERSON DURING THE VARIOUS MINE PHASES

The RP has a critical role during all phases of a TSF. He/she should be knowledgeable of the TSF's implementation, operational, and closure schedules and budgets. During all phases of the facility, the RP supports the estimating of preliminary budgets and schedules for completion of works in the TSF area. The RP will use this information to guide mine staff, consultants and contractors working in the TSF area, focusing their efforts in controlling high-consequence risks to allow for a safe and responsible operation that meets with the site operational requirements.

3.1 *Initial Studies and Designs*

During the initial development stages of a TSF, and after selection of a qualified engineering firm for the TSF design, the RP participates in completion of the TSF's designs and studies for the area. The role includes reinforcing a risk-based approach to all engineering work completed for the area, and the adequate incorporation of tailings BAT and BAP, and continual improvements and review processes. For example, the RP might participate in Multiple Account Analysis (MAA) exercises led by the design engineer and other specialized consultants, such as MAA completed for selection of the location for the TSF area and technologies.

In addition, the RP, as the owner's representative, provides site-based operational input and feedback to the designer, and should verify that the designer of the TSF area considers related mine corporate policies, regional regulatory requirements and permits, and other agreements the mine might have with the different stakeholders.

3.2 *Construction and Expansions*

Depending on the RP's role within the mining company, and on the size and complexity of the TSF area, the RP might also act as the project manager responsible for coordinating the implementation of the TSF area in some cases. In most cases, however, it is recommended that the responsibility for construction of a TSF expansion be given to a different professional to act as the project manager for the TSF project and expansions. Under such a scenario, the RP is still responsible for integrating construction activities for the TSF within the tailings management framework, and should be able to identify most significant risks and critical controls for these projects. The RP should follow closely the progress, budgets and schedules for execution of these projects.

3.3 *Operations*

As the designs, construction and/or expansions of the TSF area are completed, the RP is responsible for the development and implementation of operational and training programs to support with the future commissioning and operations of the TSF area. These programs should be developed as a component of the company's tailings management systems and following a risk-based approach and BAP and BAT. Implementation of operational programs require, among other things, completion of risk assessments and mitigation strategies for the TSF area; and the development of an OMS Manual, EPP/ERP, and inspection forms and procedures, covering operational and critical controls. A well-defined and established surveillance and review process is the cornerstone for a safe TSF operations, and allows for proper maintenance and operational activities, including risk controls, to be implemented in a timely manner.

3.4 *Closure and Post-Closure*

Finally, the RP shall be available to support with transition of a TSF area into care and maintenance and ultimately into the closure phase. TSF areas are frequently one of the largest reclamation and closure liabilities of a mine once the mine ceases operations. The RP responsibilities for a TSF under care and maintenance or closure will require similar participation in design, studies, construction, and operations as for the pre-operational and operational phases of the facility. Specially the initial closure period of a TSF area can be a very active period. Thus, it is BAP to have actions to support proper closure and post-closure scenarios to be taken progressively in previous stages of the TSF.

4 CONCLUSIONS

The RP, as the responsible person for the proper operations of a TSF is one of the most important controls that an operation has in terms of managing the risks associated with the TSF area. MAC recognizes the value of the RP in the 2017 edition of its tailings management guidelines. An experienced and qualified RP allows for the adequate implementation of the various components of a tailings management system, including BAP's and selection of BAT's. It is important for mining companies, small or large, to identify the RP within their operations and clearly define its role and responsibility. The RP for a TSF acts as a hub for the application of all other operational controls and critical controls in the field. Mining companies should recognize the value of application of this BAP to allow for further risk-reduction and management improvements to their current operations.

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Tailings Facility Risk Assessment and Risk Management

Jack A Caldwell
Robertson GeoConsultants, Vancouver, BC, Canada

Colleen A Crystal
Geo-Logic Consultants, Irvine, CA, USA

ABSTRACT: This paper describes a case history of a new tailings facility in South America for which a Risk Assessment and Risk Management Plan have been prepared as part of facility detailed design. The risk assessment was undertaken in accordance with the principles set out in performance standards issued by the World Bank/International Finance Corporation (IFC), and other generally accepted guidelines that comprise international standards of good practice, as defined by the IFC guidelines. This involved establishing the system or aspects of the mine to be included in the risk assessment; the hazardous and their probabilities that could affect facility operation and closure; the consequences of events; and finally a consideration of the tolerability of remaining identified risks. On the basis of the risk assessment the Risk Management Plan and accompanying operational Risk Registers have been prepared and the design and operations formalized to mitigate or avoid intolerable risks. The result is a planned Tailings Storage facility that is robust, resilient, and reliable.

1 INTRODUCTION

This paper summarizes a risk assessment and risk management plan prepared for the Buritica Mine dry-stack tailings storage facility (TSF) to be constructed in Colombia, South America.

In the context of the case study presented, the authors introduce some basic risk assessment tools and concepts including a discussion of system definition and risk identification, hazards evaluating, tolerability, robustness and reliability in design, and operational risk management approaches, including the use of risk registers as operational check-lists.

TSF failures may involve structural failures, operational failures, equipment failures, unforeseen conditions, and/or unanticipated loading conditions. The risk assessment and risk management plan were based on evaluation and management of three broad categories of risk: i) preventable risks that can be monitored and controlled through risk registers, rules, values, standards of practice, standard compliance tools and best management practices and guidelines to reduce uncertainties; ii) strategic (or calculated) risks undertaken for strategic gain often with imperfect knowledge guided by judgment and experience to make informed decisions based on appropriate margins of safety and consideration of both economic factors and consequences of failure; and iii) external risks (including natural hazards such as seismic and flood hazards) whose probability of occurrence are generally outside the control of owners, designers and stakeholders, but for which consequences can be mitigated by design or operational controls.

Identification and management of preventable risks for the Buritica dry-stack were evaluated in accordance with the principles set out in performance standards issued by the World Bank/International Finance Corporation (IFC), and other generally accepted guidelines and rules-based approaches that comprise international standards of good practice.

Successfully managing strategic risks is a key driver in capturing potential financial gains from any endeavor (across a broad range of industries). Because rules-based risk management approaches will not reduce the likelihood or consequences of failure, Strategic risk management requires an approach based on frank, transparent and explicit risk identification and discussions of scenarios, criticality analyses, peer review, engaged corporate leadership and adequate dialogue, including definition and communication of risk tolerances both corporate and societal. In terms of the case study presented, the risk management plan included of a series of explicit risk registers, one each for construction, operation, performance, and closure.

Evaluation and mitigation of external risks (including natural hazards such as seismic and flood hazards) was undertaken in accordance with conventional probabilistic and deterministic engineering evaluations of occurrence risk, Factors of safety against failure, consequences of failure (including magnitude of potential deformation) and cost of mitigation by design versus tolerability of remaining risk not mitigatable by design or operations. In this context, it is important to note that there is no such thing as zero risk.

2 BACKGROUND

The risk assessment undertaken for the Buritica TSF is based on experience gained in the design, construction and operation of the tailings facility at the Escobal Mine in Guatemala (Caldwell 2015). Both involve placement of waste rock and filter pressed tailings (with production rates of between 4,500 and 5,000 tpd in a valley in a relatively wet climates, with mean annual precipitation on the order of 1400 to 1700 mm.

The site of the TSF is a long, narrow valley surrounded by steep, high hills (Figure 1). The valley fill is up to twenty-five meters of mixed colluvium and landslide gravel, sands, and silt. The fill is permeable and the groundwater is generally on the order of 30 meters below existing grades.

Figure 1. The valley in which the Buritica TSF will be constructed.

The tailings will be filter pressed and placed and compacted in a lined dry-stack TSF up to 40 meters in height (Figure 2). Optimum moisture contents for the tailings are anticipated to be on the order of 13 to 15 percent, with dry unit weights on the order of 1.8 kg/m^3. Potentially acid generating mined waste rock will be used to construct the perimeter berms within the lined footprint. Excavated native alluvial and colluvial deposits will be used to great outer perimeter buttresses, reinforced rock fills, access roads and channels. On either side of the TSF is an access road and then a surface water channel cut into the valley fill that separates the TSF from the native steep hillside terrain by distances of between 10 and 20 meters.

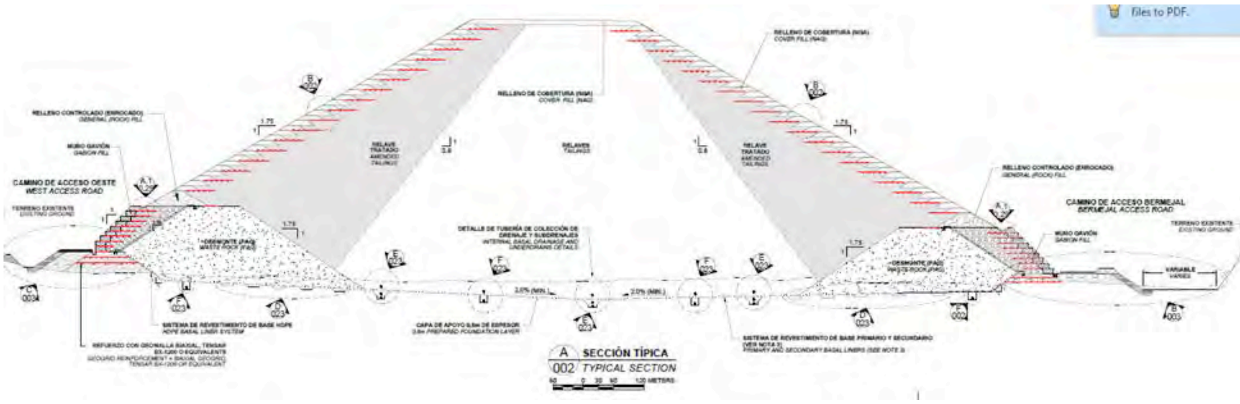

Figure 2. Cross section of the Buritica TSF.

3 OBJECTIVES

We undertook the risk assessment during preparation of the conceptual design to comply with developing standards of practice. Subsequently the risk assessment was expanded as part of preparation of the detailed design undertaken for project permitting and construction.

The resulting risk assessment and risk management plan enabled us to incorporate conservative features to reduce the likelihood that the defined hazards would affect the facility and to mitigate the consequences should the hazards transpire. The risk registers serve to remind those managing the facility of what to look out for, what to change is certain conditions are observed, and how to respond should potential hazards become immanent.

We recognized that there is no risk free tailings facility. But use of risk methods enabled us to describe conditions that could be judged as tolerable or intolerable by the mine developers, and hence to implement details that allowed for the occurrence of tolerable risk outcomes only.

4 THE SYSTEM

A project may be thought of as a system, and as such, the techniques of systems analysis can be employed when evaluating the risk of failure for the project. The failure probability or consequences of failure can only be estimated or reduced by appropriately identifying and understanding the vulnerabilities of the system. System definition should be relatively broad, lest the team hone in on individual hazards too quickly and neglect to recognize the risks associated with a series of unrelated circumstances, events or minor failures culminating in overall system failure. The system defined for the risk assessment is the TSF and its immediately surrounding area, including:

- TSF foundation
- The TSF, itself, including earthwork buttresses, cover, and geosynthetic liner components, drainage/seepage controls and leak detections systems
- Associated proposed surface and ground water control channels
- Adjacent native hillside terrain
- Access/haul roads

5 HAZARDS

A hazard may be identified as any factor (condition, event, or force) that could damage the TSF or result in performance not in accordance with design intent or standards of practice. The hazards to which the TSF may be subjected are divided into two categories

- Natural hazards including: extreme precipitation; earthquakes; flood flows in the channels; and surface water and groundwater hydrology (including flooding/overtopping of channels, erosion and a potential seasonal rise of the groundwater table).
- Operational/Mining Related Hazards arising from mining practices including: a shortage of waste rock to construct perimeter berms and buttresses; and filter pressed tailings too wet to compact in accordance with specifications; inadequate drainage.
- Site Condition geologic hazards (foundation conditions; native hillsides; faulting)
- TSF Performance Risks – including Physical and chemical properties of the tailings and waste rock as they relate to structural stability, leachability and leachate quality
- Human Factor hazards (management, communication, peer review, Engineer of Record, etc., including lack of appreciation of mechanisms that could trigger failure and communication thereof.)

These identified hazards include, but are not limited to those most often associated with TSF failures as outlined in ICOLD Bulletin 121: inadequate management, lack of control of water balance, unsatisfactory foundation conditions, inadequate drainage, lack of care, lack of appreciation of mechanisms that trigger failure. We recognize that these primary hazards may come to be because of many preceding factors. For example floods in the channel may result from one or more of the factors noted in the Fish Bone Diagram of Figure 3. Similar plots were compiled for each primary hazard. It was not considered possible or reasonable to attempt to quantify each hazard or preceding subhazard. So for flooding we simply set the one-hundred year recurrence interval as the design flood. But in addition we calculated the flow and established the consequences of the occurrence of the 1,000- year and Probably Maximum Flood (PMF) in the channel.

6 RISKS

In the context of this paper, risks are categorized as preventable, strategic and external. Key Preventable risks identified in the risk management plan included, but were not limited to:

- Slope failure risks
- Cover erosion risks
- Foundation deformation risks
- Mass wasting risks
- Basal seepage risks
- Channel flooding/overtopping.

These risks may be the result of the preceding hazards:

- Unsatisfactory foundation conditions
- Control of the water balance (including control of run-on/run-off and decant of contact or impounded water on operational decks)
- Inadequate drainage
- Inadequate management (engagement, oversight, peer review, monitoring)
- Lack of appreciation of mechanisms that trigger failure.
- Control of filter pressing operations
- Control of storage, handling, moisture condition, placement, compaction and daily finish grading during tailings placement; particularly deposition and management of the tailings in a manner that they will not be subject to static or seismic liquefaction at placement or in the future (due to continued loading or potential re-saturation due to high precipitation)
- Control of rate of rise and monitoring of o re pressures during operations (and closure)
- Control of earthworks related to stability buttresses and surface water controls
- Plan for dealing with off-spec materials, including role of chemical additives/tailings amendment
- Chemical properties of tailings and waste rock (including PAG materials) disposition and management

- Contact water management and water treatment requirements, including ponding and storage of surcharge waters during storm events
- Human dynamics; including leadership, recruiting and training of skilled operational staff, assignment of an in-house tailings engineer/construction manager, identification of EOR and succession plan, and provision for peer review and real time line management addressing of identified risks/changing conditions.

Strategic risks identified include:

- Vulnerabilities of native slopes to hydro-dynamic forces (steep terrain subject to potential landsliding and debris flow events)
- Vulnerabilities associated with stormwater control channels (including potential flooding, debris flows and erosion) in extreme storm events (1,000 year, 10,000 year and PMF events);

External risks identified include:

- Potential for inundation/flooding/damage to TSF and/or downstream areas in event of extreme storm events and associated hydro-dynamic forces;
- Seismic hazards (including design ground motions)

7 RISK IDENTIFICATION AND ASSESSMENT TOOLS

Risk assessment tools for studying facilities, or systems, with many components and different potential modes of failure include event trees, fault trees, fish-bone diagrams, bow-tie diagrams and failure modes and effects analyses (FMEA) tables and charts, among others. The tools used in the current analyses included:

- Fault trees: a plot showing potential combinations of failures that may cause the overall system to fail. Fault trees start with an assumed potential failure condition and work backwards to evaluate the causes under the hazard categories of natural, mining, performance and human factor hazard categories that may act independently or in concert to cause TSF malfunction or failure.;
- Fish bone diagrams: that shows risk as the result of other primary causes (hazards).
- Event trees: an attempt to model the system's response to potential initiating or "trigger" events.
- Bow-Ties – is a way to illustrate the relationship between hazards and consequences.
- Failure modes and effects analyses (FMEA): a qualitative or semi-quantitative risk screening tool used to systematically postulate component failures and identify resultant effects on system operations, provided the correct "system" or conceptual model is appropriately defined.

Event trees, fault trees and FMEA's do not, however, readily lend themselves to capturing the "human factors" that are often identified as the root causes of many tailings facility failures. These human factors arise from: communication problems; lack of engaged corporate leadership; and lack of stakeholder engagement (who may have different risk tolerability levels). In addition the absence of clear assigned roles and responsibilities and continuity are human factors that may lead to tailings facility failure.

8 TOLERABILITY

Risk reduction can be achieved by lowering the probability of failure, or by lowering the severity or consequence of failure, or both. Tolerability includes consideration of local conditions, local tolerance for risk, influence of standards of practice and risk tolerance as defined by practice elsewhere in the world (including for other industries).

In preparing the design and operating plans for a tailings facility, the risk assessment is useful in establishing tolerable consequences. For example, detailed 2D hydraulic modeling showed that the 1,000-year recurrence interval and PMF in the east surface water channel could result in overtopping and erosion of the toe access road between the TSF slopes and the channel. Such erosion can be repaired during mine operation and in the surveillance and maintenance phase of post-closure, and is therefore tolerable.

9 RISK MANAGEMENT

The risk management plan is based on the risk assessment. For Buritica, the risk management plan includes of a series of risk registers, one each for construction, operation, performance, and closure. The risk registers include identification of risks and management requirements associated with:

- Tailings management (including filter pressing, handling, placement and compaction) and physical characteristics (including grind, clay fraction, optimum moisture content, potential for re-saturation, trafficability, strength, potential for liquefaction, construction quality assurance and deformation and pore pressure monitoring requirements);
- Water management (including surface and groundwater controls, contact and non-contact water management)
- Geochemical risks (including ARD potential)

Monitoring of the performance of the TSF during operation will be done on a daily basis by those immediately involved in operations. Some observation guidelines:

- Dust: This is immediately apparent; may be monitored and quantified by air quality monitoring equipment; and may be mitigated immediately by wetting, application of dust suppressants, or further compacting dust-prone areas.
- Erosion: This will be immediately apparent. Gulleys, rills, or sheet movement of cover materials or tailings may be observed on a daily basis by field staff. Accumulation of eroded materials at the toe of the TSF slope, in diversion channels, or on the top of the tailings placement surface are indicators of erosion. Erosion may be repaired by infilling, regrading, enhanced vegetation, or placement of additional rip rap.
- Stability: Piezometers and inclinometer will be installed. They will be read regularly by qualified staff and the results interpreted by engineers. Thus the current factor of safety of the slopes may be established. If there is excess pore pressure, wick drains or drain pipes maybe installed.
- Basal Seepage: There are two basal drainage systems: one above the liner and one below the liner. Seepage from both will be monitored and seepage quantities and quality measured. Little if any seepage is anticipated from the upper drainage system. This is because the tailings will be of low permeability, and there is no standing water on the top surface of the tailings. Seepage from the upper drains will be directed to the water treatment plant. There may be some seepage from the lower drainage system. Water entering this system is most likely to come from a rise in the groundwater table. Such water may be clean and suitable for immediate discharge.

A key objective of the risk registers is to provide a framework (series of check lists) to regulator monitoring and reporting (by site operations, reviewers, risk managers, construction personnel, corporate oversight and EOR) such that early warning signs and risk information are available to a least one person in line-management that if acted upon will trigger actions to prevent component (or system) failure.

The second objective or the risk registers, is to avoid behavioral or institutional biases (as FMEA's and PIG's more readily susceptible to) that prevent potential risks or seemingly unrelated events or decision from being recognized or acted upon.

The risk registers do not remove uncertainty from the system, nor do they alleviate the need for judgment. They do however, provide a means for explicitly identifying and subjectively categorizing (within reason) potential key risks and providing for regular review, training and identification of on-site qualified personnel in responsible charge of tailings operations and ongoing updates and peer review of the risk registers (by EOR, peer reviewers, owners and various stakeholders) so that potential normalization of deviance pitfalls can be minimized.

10 POST CLOSURE

The risk assessment demonstrates that the TSF is robust, resilient, reliable, and safe. During construction and operation any malfunctioning may readily be repaired. Post closure and in the long term after cessation of surveillance and maintenance, the following long-term risks are acknowledged and provided for:

- Landslides. Landslides may occur in the hills of the valley. It is conceivable that piles of rock may slide down into the corridor between the facility and the hillside. This could reduce the flow capacity of the corridor in the event of extreme events. To mitigate this, the two corridors are large enough to pass the PMF. And the probability of the PMF occurring simultaneously with or soon after a landslide is very small.
- Extreme Earthquake. In the event of an extreme earthquake, some deformation of the facility may occur. To mitigate this, large perimeter rock berms are provided at the toe of slopes and smaller berms provided a thick cover to the tailings. Thus in the event of even significant deformation, the tailings are not likely to be exposed or subjected to erosion.
- Geomorphic Change. In the very long term, it may be anticipated that the hillsides will continue to wear down via a series of landslides. This has been happening for at least ten thousand years as the valley fill has accumulated. To the extent that in the very long term, geomorphic change occurs as a result of landslides and ongoing deposition of valley fill, it is feasible the closed tailings facility will be completely covered by and encapsulated in future, rising valley fill materials.
- Climate Change. It is not fully established how climate change may affect future climate and the Buritica mine site. It may get wetter or it may get hotter and drier. To the extent that the TFS and associated components are designed for the Probable Maximum Precipitation and the Probable Maximum Flood, it is unlikely that climate change will result in currently quantifiable changes to these two events.

11 DISCUSSION

The TSF is designed and will be constructed, operated, and closed to comply with Colombian and international standards of practice, including the guidelines prepared by the Mining Association of Canada (MAC), and the provisions of the Canadian Dam Association Technical Bulletin (2014).

Tailings dams are generally categorized in accordance with the Canadian Dam Association criteria (CDA, 2014). While it is questionable if these categories are applicable to a filter pressed tailings facility, to the extent the criteria apply, the Buritica TSF may classified as a Low Hazard structure. This is because—as noted by CDA, 2014 and using their wording:

- The population at risk is zero.
- The area downstream of the TSF contains limited infrastructure and services.
- Failure (by any conceivable means) would not cause loss of life.
- The environmental and cultural impact of failure would be minimal short-term loss and no long-term loss.

The TSF incorporates the following Best Available Technology (BAT) for tailings management:

- Eliminate surface water from the impoundment.
- Promote unsaturated condition in the tailings with drainage provisions.
- Achieve dilatant (density) conditions throughout the tailings deposit by adequate moisture control, placement, and compaction.

In particular, for BAT as regards physical stability (BAT-PS), the TSF includes compacted rockfill embankments, compacted filter-pressed tailings, and strong foundation soils that are not liquefiable. As regard BAT for chemical stability (BAT-CS) the TSF includes basal liners, drains, and a cover that will minimize infiltration.

Over the projected mine life of some thirteen years, the TSF will be continuously constructed, operated, and reclaimed in a series on nominal cells. Current Best Management Practices (BMPs) that include the following will be implemented:

- A designated Engineer of Record.
- Regular independent technical peer review.
- Implementation of risk management principles and practice.

Ongoing reclamation of the dry facility will be undertaken. As successive lifts of filter-pressed tailings are placed behind the perimeter buttresses and berms, the exterior surfaces of the outer cover will be re-vegetated. Thus at all stages of operation, the outer slope of the dry facility will be, in essence, a vegetated slope that replicates the natural slopes at and in the immediate vicinity of the mine.

At mine closure, the tailings facility will be essentially similar to the natural slopes at and in the vicinity of the mine: a natural landform, of dry, dense soils and rockfill, that supports a natural climax stand of vegetation. Thus a new geomorphological form is created that will, in the long term, respond to natural geomorphic forces as do the surrounding slopes and hillsides.

12 CONCLUSIONS

Risk assessment is a valuable aide to the exercise of judgment by the tailings facility designer and the managers, engineers, and technical folk charged with facility construction, operation, and closure. Formal risk management is mandated in British Columbia, for as is demonstrated by the failure of Mount Polley: tailings facility folk err; the mine manager who is responsible for the facility cannot always rely on them; but the use of risk principles and practice can identify areas to focus management and action to lead to a safe facility.

Risk management is no substitute for or savior of a bad design. It is hard to envisage how risk management would have precluded the failure of the Samarco facility. We are entering a new era of tailings facility management and it will inevitably, we believe, be an era of increased use of risk assessment and management. It is hoped that this paper helps by setting out just one, initial attempt.

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What does it mean to be the Engineer of Record (EoR) for a Tailings Storage Facility (TSF)?

K.F. Morrison
Morrison Solutions, Inc., Lakewood, CO, USA

R.E. Snow
D'Appolonia Engineering Division of Ground Technology, Inc., Pittsburgh, PA, USA

P.W. Ridlen
Knight Piésold, Inc., Denver, CO, USA

C.N. Hatton
Haley & Aldrich, Inc., Denver, CO, USA

ABSTRACT: One would think the term "Engineer of Record (EoR)" is an easy concept to grasp. For most public infrastructure projects (e.g., roads, bridges, buildings, water dams, etc.), the Design Engineer and the EoR are one and the same. This concept has also been applied, to some degree and in a similar manner, to tailings storage facilities (TSFs). However, TSFs do not apply a "conventional" construction process, nor do they adhere to a typical construction schedule. Instead, they typically apply the observational method described by Peck (1969), with a construction life that covers decades. The usual application of the EoR concept does not necessarily translate well in these instances. The Mount Polley (Canada) tailings dam reportedly had five named individuals serve as EoR during a four-year period prior to the failure that occurred in August 2014. That incident served as the catalyst for review of the EoR concept by the mining industry and those that regulate it. This paper provides results of a survey conducted to obtain information on the current state of practice for TSF EoR services, and identify concerns within the engineering community who perform such work.

1 INTRODUCTION

The Geoprofessional Business Association (GBA) sponsored a workshop on January 26, 2017, addressing the subject of the Engineer of Record (EoR) for tailings storage facilities (TSFs). This workshop, held in Denver, Colorado, was supported by the United States Society on Dams (USSD) and the Association of State Dam Safety Officials (ASDSO), and included participation of more than fifty tailings dam practitioners from the United States, Canada, and Chile. While most of attendees were consulting engineers, seven of the participants were employed by state regulatory agencies and one was employed by a mine operator. Many of the participants are currently serving as EoR on TSFs around the world with distinguished careers as tailings dam designers, and have first-hand knowledge of the issues associated with this responsibility.

A survey was prepared in advance of the January 2017 workshop to obtain information on the current state of practice. Two breakout sessions were held during the workshop to further characterize concerns held by the participants and identify possible solutions and approaches to improving the state of practice with the ultimate goal of preventing future failures. GBA is currently preparing a practice guideline for the TSF EoR similar to the guideline GBA (formerly ASFE, 2010) developed for the Geotechnical Engineer of Record (GER), which is applicable to conventional design/bid/build construction. Although GBA is preparing this guideline, the workshop organizers believed the industry as a whole would value from distributing the findings of the workshop, leading to the development of this paper.

2 SURVEY RESULTS

A web-based survey was conducted in advance of the workshop using SurveyGizmo to gauge the attitudes and concerns of the group regarding ongoing efforts to better define the roles and responsibilities of the owners, engineers, third-party reviewers and regulators involved in maintaining the safety of tailings dams around the world. The results of the survey are presented herein. For the purposes of the survey, a TSF was considered a mine or mineral processing tailings dam and impoundment, or a coal combustion residuals (CCR) or coal refuse impoundment. EoR services were defined as formal designation as the EoR, as well as situations where an engineer's endorsement of the design and/or construction is required. The survey was used to compile information on the participant roles (e.g., EoR, owner) and usual work products (e.g., construction plans, specifications, etc.) of TSF projects, and to identify concerns among the engineering community about providing EoR services for TSFs.

2.1 *Demographics*

A series of demographics questions were asked to assess the backgrounds and experience levels of the survey respondents. Fifty-one responses were tabulated (although not all respondents answered each question). The survey was distributed to workshop attendees and to a wider audience via LinkedIn. Accordingly, the response rate cannot be determined.

The average years of experience of the respondents was 25, with a general range from 8 years to 45 years of experience (though one respondent had zero years of experience). The average amount of tailings dam experience was 18 years, indicating that most of the respondents had dedicated a significant portion of their careers to the tailings practice. Ninety percent (90%) of respondents were registered professionals, with 65% registered in the United States, 29% registered in Canada, and 10% registered in other countries (some respondents registered in multiple countries).

2.2 *Scope of Practice*

The survey respondents and their organizations exhibited experience with effectively all types of tailings dams, ranging from TSFs at metal mines, to coal processing facilities and coal combustion plants, and other tailings dams (e.g., oil sands, phosphates, etc.). Notably, 90% of the respondents indicated that their organizations had experience with more than 20 tailings dams in the prior five years, while individually more than 20 respondents had experience with more than 10 tailings dams in the prior five years, further highlighting the level of tailings experience of the respondents. Approximately 84% of respondents indicated that they, or their respective organizations, were involved with TSFs for metal mines, with 53% involved with tailings dams other than those at metal mines, coal processing facilities, or coal combustion residual (CCR) facilities. Eighty-five percent (85%) of respondents noted that most to virtually all of their TSF projects involve "conventional" tailings dams that are raised on a regular schedule over a long period of time. In this context, facilities that are designed, constructed and operated to a single configuration that is modified only through a separate and subsequent design and construction process, similar to a water storage dam, were considered to be "unconventional." This latter process is more common with CCR facilities.

About 85% of respondents declared they are "aware" or "very much aware" of the challenges in providing EoR services for TSFs. Thirty-seven percent (37%) indicated their firms either already have formal internal policies defining standards for tailings EoR responsibilities or are in the process of developing such policies. Eighty-eight percent (88%) also indicated they believe there is a need for an industry document that clarifies the role of EoR for tailings dams and the responsibilities of owners and engineers in maintaining the safety and security of these facilities.

Respondents indicated their opinion on what constitutes the elements of TSF projects considering a list of studies, programs or engineering work products that reflect the standard of practice for a TSF project. These elements essentially comprise the TSF services discussed below.

2.3 *TSF Services Typically Performed by the Respondents*

Survey respondents were asked which work products (from the list in Figure 1) are incorporated into the projects they undertake, and how frequently these work products are produced. Many of these elements are engineering service products, but they may not necessarily be originated by the EoR and sometimes are prepared or conducted by the owner or other consultants. The most common services, as indicated in Figure 1, were conceptual and feasibility-level designs; geotechnical site characterization and borrow material investigations; development of issued for construction (IFC) plans and specifications; dam safety reviews (including inspections and/or instrumentation review); surface water management plans; site-wide water balance analyses; development of operation, surveillance, and maintenance (OMS) manuals; and development of facility closure plans and cost estimates. The least common elements of service were risk assessments, failure modes and effects analyses (FMEA), preparation of environmental monitoring and response plans, regulatory compliance programs, and input or support to NI 43-101 reports.

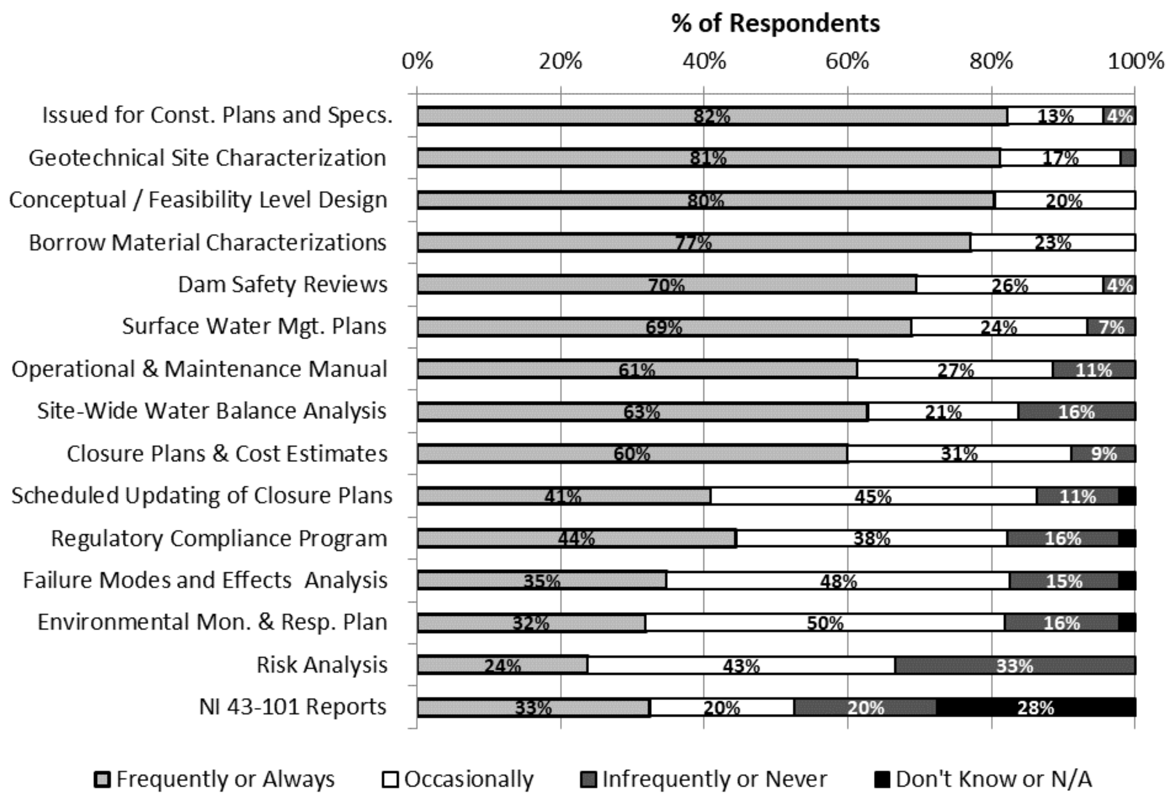

Figure 1. TSF elements incorporated into projects requiring EoR services.

Survey respondents were also asked to describe the typical work products provided during the design phase versus the work products and services performed during construction and/or operations phases. No clear trends were evident from the survey, but it appears that for most projects, some degree of continuity of services from the designer is usually carried forward into construction and operations. Thus, it appears from the survey that changes in the EoR from the design phase into the operations phase may be the exception rather than the rule.

2.4 *Owners' Responsibilities*

Respondents were also asked to provide their opinions on the responsibilities of TSF owners from a provided list, the results of which are illustrated in Figure 2. Respondents believed, in general, that the owner is responsible for providing qualified personnel, managing safety and health programs, procuring services based on qualifications, managing risk, and regulatory compliance and auditing programs. The majority of respondents viewed that owners had the responsibility to provide an independent technical review board (ITRB), particularly for high risk facilities. Respondent comments identified two additional owner responsibilities, including procedures to address EoR concerns and processes for implementation of corrective actions.

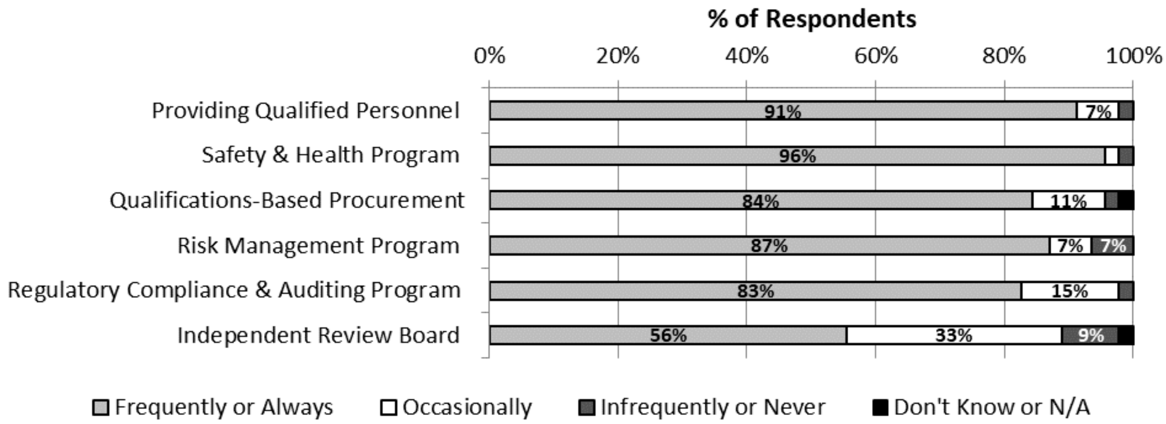

Figure 2. Survey respondents' position on responsibilities of the Owner on TSF projects.

2.5 *Oversight and Review in TSF Design, Construction, and Operation*

One of the recommendations brought forward in the aftermath of the Mount Polley failure is the use of independent expert review panels or other third-party reviews throughout the design, construction and operational phases of a TSF (IEEIRP, 2015). Review panels are not a new concept in the tailings dam practice (e.g., Ridlen et. al., 1997; McKenna, 1998; Martin et al., 2002; Morgenstern, 2010). The survey respondents were asked to comment on the relative frequency that external reviews or oversight (e.g., regulatory agency, third-party review, etc.) occur on TSF projects. Figure 3 illustrates the relative frequency of the types of formal oversight of TSF projects experienced by the survey participants.

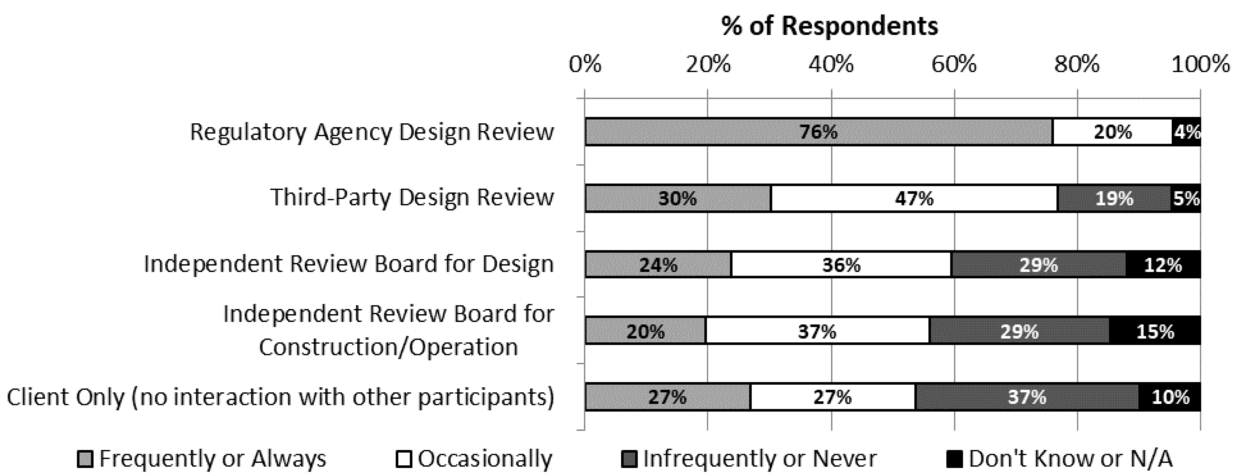

Figure 3. TSF project participants and relative frequency of involvement in TSF projects.

Only about 24% of respondents said that independent design review boards were frequently used on projects for which they are involved, and only 20% said that review boards were used regularly during construction and operation. Nearly 30% indicated that review boards were infrequently or never used on their projects, while an additional 12 to 15% did not know. Of those who responded on whether or not regulatory agencies are involved for design review, 76% indicated that regulatory agencies are frequently or always involved. In limited cases, the client is the only other project participant that the designer interacts with on TSF projects.

3 CONCERNS RAISED BY SURVEY RESPONDENTS

Survey respondents were asked to indicate if they viewed any of the issues from the list presented in Figure 4 as concerns when providing EoR services. The results are presented in Figure 4.

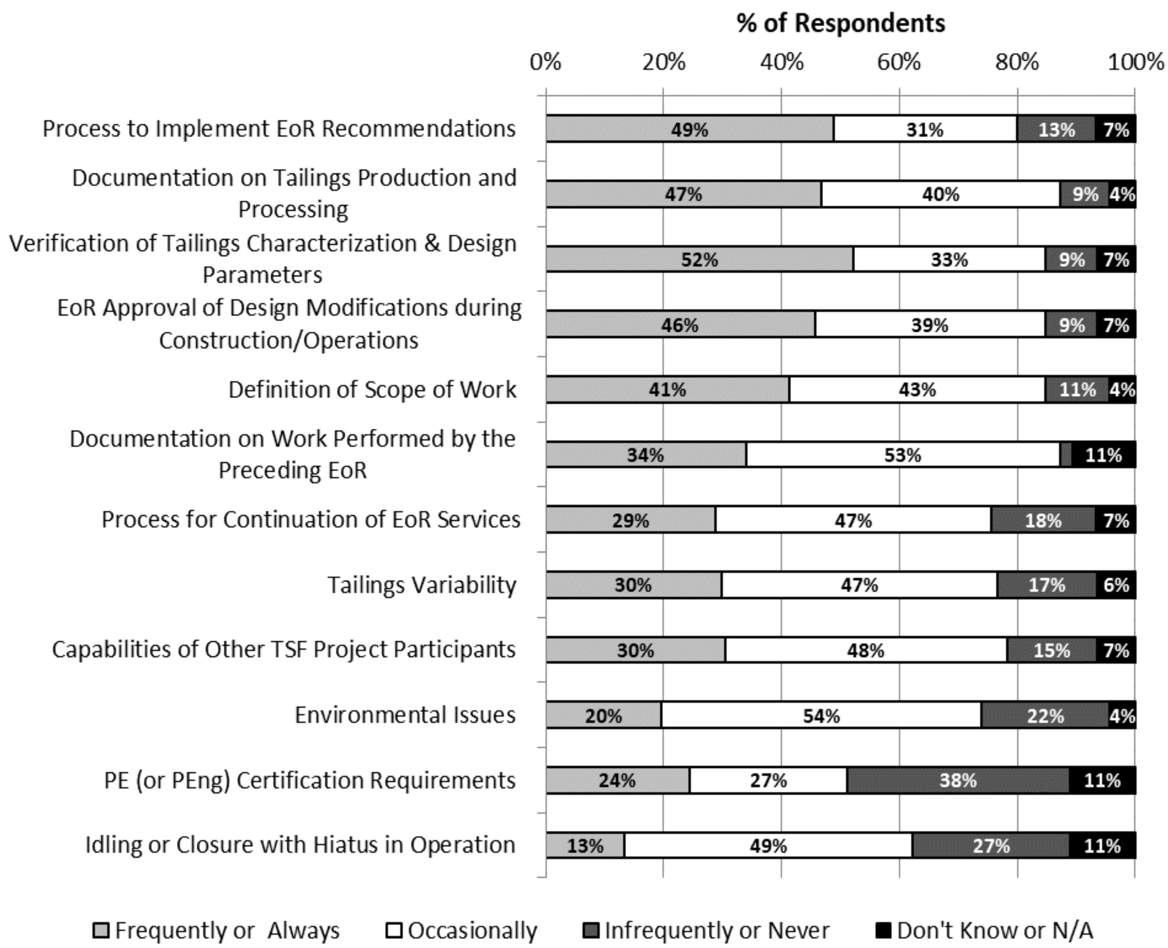

Figure 4. Survey respondents' concerns with providing EoR services.

Interestingly, the issue generating the most frequent concern was verification of tailings characteristics and design parameters, a primarily technical issue. Tailings production and characteristics are affected by variations in the ore body, as well as mining and processing activities. In most cases, the performance of the facility is highly dependent on the realized tailings characteristics over the life of the facility, and how well they correspond to the design assumptions. Since most tailings dam designs invoke the observational method, it is important to observe the site conditions that develop over time and potentially adjust the design accordingly. GBA (formerly ASFE, Undated) developed a document titled, *"The Observational Method in Construction – A Message to Owners"* that further elaborates the observational method for providing geoprofessional services.

The concern expressed by the survey respondents may reflect difficulty in gaining access to the information necessary to confirm the design assumptions, as well as concerns that the operators may not fully appreciate the need for ongoing observation and reconciliation. It can be inferred from these responses that more emphasis on routine confirmation and documentation of tailings production, material characteristics and comparison of actual parameters to the design parameters by owners is required.

Additional concerns encountered frequently or always by the respondents relate to the clarification of responsibilities and designation of proper authority that should accompany those responsibilities. Specifically, the following items ranked high on the list of concerns:

- Process for implementing EoR recommendations
- EoR approval of design modifications during construction/operations
- Definition of the scope of work (of the EoR)

The responsibilities and expectations of the design EoR are generally relatively straightforward and often are defined in distinct contractual scopes of work that have significant precedent and examples to guide each party. Responsibilities during construction/operations, however, can vary significantly. Ideally, the design EoR has a role in quality assurance/quality control (QA/QC) processes, which may include a resident engineer during initial construction works. However, with dams that are raised sequentially over long periods of time, the construction and operational phases of these facilities overlap, and many owners experience market pressures to reduce operational costs, often by reducing (or eliminating) the scope of the EoR and his or her team to support operations. In many cases, mine operations self-perform the construction instead of a contractor, and the owner may have such technical resources on staff as construction managers, geotechnical engineers, civil designers, and other technical specialties. As a result, there is no "standard" or "typical" role and scope of work of the EoR during construction and operations, and the roles and responsibilities often become blurred with those of the mine owner's employees.

The observational method inherently involves adjusting the design in response to actual conditions; thus, changes to the design frequently occur over the life of the facility. Under these conditions, the understanding and continuity of the intent and underlying assumptions of the original design may become lost over time, especially if the EoR does not have regular, ongoing involvement. The risks associated with losing continuity of the EoR throughout the TSF life cycle are what bring the concerns expressed by the survey and workshop participants to the forefront, and are becoming better understood by the industry. One of the recommendations for improvement adopted by the International Council on Mining and Metals (ICMM, 2016) is for widespread implementation of *"a formal change management process that is designed to ensure that when material changes are contemplated and subsequently made to the life of facility plan or to the Engineer of Record they are fully considered, formally adopted and embedded into the operations, maintenance and surveillance manuals, into budgets and into training and that the implications of the change are communicated"* (Golder, 2016). Workshop participants were in agreement with this recommendation.

As a facility makes a transition into a relatively routine operation, usually with recurrent cycles of construction, it is customary for the EoR to make regular site visits (typically at least once per year) to observe progress and consider whether the actual conditions are consistent with the design intent. Preparation of an annual dam safety inspection (DSI) is mandated in some jurisdictions, and should be considered a Best Management Practice (BMP). If an ITRB, third-party reviewer, or auditor is in-place, site visits are performed on a regular frequency with similar intent as the EoR's inspection. A clear process is needed for addressing observations of concern raised by the EoR, ITRB or other parties, with follow-up to ensure that recommendations are followed, or a sound technical reason given for why they are not implemented (if they pose a safety risk). Although not always clearly understood within many organizations, the EoR and independent reviewers are valued by a company's stakeholders. Most leading mining companies have established, or are now establishing, internal stewardship, governance or oversight departments or committees dedicated to ensuring that risks related to TSFs are managed according to the company's strategy and objectives.

Survey respondents emphasized the importance of a clear definition of the responsibilities and authorities of the EoR (and the EoR team), the owner, any third-party reviewers, and regulators. In addition to the importance of communication throughout the engagement, the respondents cited the value of identifying risks and concerns upfront, implementing design reviews and QA/QC programs.

The process for transitioning from one EoR to another was identified as a concern by several of the participants. This concern applies whether a planned hand-over of responsibilities from the design phase to the construction and operational phases exists, or when a change in the EoR due to contractual or other business reasons is contemplated. Several potential additional actions are considered or applied when respondents have taken over the EoR services from another party, including dam safety reviews (including design review based on comprehensive historical documentation) as part of the transition and independent assessment of potential risks.

Finally, almost two-thirds of the respondent expressed concern about the risk of angering mining clients in the process of clarifying (and attempting to enforce) the role of an EoR in the tailings dam lifecycle. The consensus of these participants was that most major mining companies understand the need for — and value of — an EoR with clearly-defined responsibilities and authorities. More inconsistency exists in the perspectives of junior mining companies. Approximately 75% of the respondents perceived that the clients they worked for were supportive or very supportive of the EoR concept, and only about 8.5% remarked that some clients are antagonistic or hostile to the notion. However, the broad consensus of respondents was that any risk of losing business by angering clients was worth taking because of the potential consequences to the public safety and environment, as well as the integrity of the profession.

4 WORKSHOP RESULTS

During the January 2017 workshop, attendees were divided into two breakout sessions. Participants of the first breakout session were tasked with collectively addressing the rationale for the position of TSF EoR, while participants of the second breakout session focused on identifying the specific roles and responsibilities of TSF project participants. This latter exercise resulted in the creation of a RACI (*R*esponsible, *A*ccountable, *C*onsulted, *I*nformed) matrix for the various elements of TSF projects.

4.1 *Value Provided by a Competent EoR*

The workshop participants agreed that designation of an EoR is anticipated to provide the following benefits:

- Continuing involvement of the responsible engineer having in-depth knowledge of the TSF, capable of implementing the observational approach to ensure the design philosophy and intent is met over the life of the project
- Delivering leadership across disciplines, providing a resource for the owner in making sound technical and business decisions
- Demonstrating owner's safety and sustainability commitments to project stakeholders
- Fulfilling regulatory requirements and ensuring QA/QC programs are implemented, project documentation is completed, and inspections are conducted and submitted
- Confirming that the owner's staff understands the proper methods of operating the facility and are prepared to respond should adverse conditions develop

The workshop participants expressed a firm belief that owners have the responsibility to arrange for independent technical review and to appoint an ITRB for high consequence facilities. The presence of independent technical review, and particularly an ITRB, can provide assurance that the EoR's observations and recommendations are addressed or other corrective actions implemented, and also provides assurance to the owner that the EoR's recommendations are consistent with the current state of practice.

4.2 *On the Definition of Engineer of Record*

Following the Mount Polley failure, the Canadian Dam Association (CDA), with input from the Mining Association of Canada (MAC) and several other organizations, has spent considerable effort in developing a consensus definition of the EoR for a tailings dam for inclusion into the CDA dam safety guidelines (e.g., Small & McLeod, 2015). The workshop reviewed the definition in progress at the time, and generally found agreement with the concept and framework of the definition, but identified several comments and some concerns.

The draft CDA definition frames the EoR for a tailings dam as an individual. From a regulatory standpoint, this makes sense because an agency typically wants a single, competent engineer—duly registered in the relevant jurisdiction—to "sign off" on the design of a facility, and in some cases "certify" or "affirm" that the facility was constructed in accordance with the design. As a note, GBA (formerly ASFE, Undated) has brought problems associated with "certifications" to the attention of engineers, owners and regulators, and offers several alternatives for addressing the issue (e.g., replace *"I certify"* with *"I state in my professional opinion"*). Likewise, in many jurisdictions, dam safety inspections are required on a regular basis (often annually), and these are also to be signed and sealed by a registered professional engineer; however, from a contractual and professional liability perspective, most owners prefer (perhaps even demand) that the licensed professional is backed up by a firm with substantial resources that can provide the financial assurances that the design conforms to the standard of care normally exercised on these typically high-risk facilities. It is likely that regulators also view the full resources of an established firm as preferred. Typically, owners will contract with an engineering firm who then designates a duly-registered individual as the EoR. As insurance companies have become wary of the risks associated with tailings dam design and operations, it may become increasingly difficult for individuals to obtain even minimal levels of insurance (professional liability or commercial general liability) to cover services related to tailings dams, making it increasingly difficult for an individual to function as an EoR independent of a firm. In the GBA breakout session, the group reached consensus (although not unanimously) that the EoR is a designated individual employed by a firm with necessary financial resources to manage liability, and that flexibility should exist for the EoR to be an employee of the owner.

The EoR team concept (Morrison & Hatton, 2016) was discussed at length in one of the breakout sessions. Participants confirmed that safe and responsible management of a TSF requires a team; however, this concept was clarified that the EoR is a member of the team and must rely on various experts to supplement his or her own technical expertise and experience.

Minimum requirements for EoRs were discussed at length. In addition to professional registration in the jurisdiction of the project, workshop participants indicated that an EoR for a TSF should possess a minimum of 10 years of relevant experience, but that more experience would be needed as complexity, scale, and downstream consequence increases.

4.3 *Roles & Responsibilities*

The RACI charts developed during the workshop breakout session provide perspective on the obligations of the various TSF project participants. The TSF project was broken into typical phases: design; initial development and construction for startup; operation and ongoing construction; and closure. Within each phase, specific TSF project elements were identified by workshop participants and RACI designations assigned to the project parties (EoR, owner's management and various designated positions, independent technical reviewer/board, and regulator). Table 1 presents general TSF elements incorporated in the various phases for the RACI charts.

Subsequent to the GBA workshop, the Dam Integrity Advisory Committee (DIAC) of the Alberta Chamber of Resources, members of which participated at the workshop, developed RASCI (*R*esponsible, *A*ccountable, *S*upport, *C*onsult, *I*nform) tables for the following two scenarios: (i) large organizations with multiple dams and sophisticated internal resources; and (ii) small organizations with few dams and limited internal resources (DIAC, 2017; Boswell & Martens, 2017). These RASCI tables divide accountabilities and responsibilities among the participants (e.g., Accountable Executive, Operations Manager, Dam Safety Responsible Engineer [DSRE], Engineer of Record [EoR] and Design Engineer [DE]) pertaining to organizational requirements; investigation and design; construction; operations and maintenance; surveillance and reporting; emergency preparedness and response; decommissioning and closure; and risk, documentation and review.

Table 1. TSF elements and project phases for RACI charts.

Design	Initial construction for start-up	Operation and ongoing construction	Closure
– Designation of design team – Plans and specifications – Cost estimation – Permitting support – Site investigation, analysis & reports – Operation, maintenance & surveillance (OMS) manual – Emergency action plan (EAP) – Action threshold levels – Regulatory reporting – Environmental impact analysis and protection – Risk and FMEA – Closure plan – Financial assurance	– QA & QC – Instrumentation – Construction management – Construction reports and as-builts	– Operations tailings management – Inspection and monitoring – Instrumentation and key performance indicators (KPIs) – Dam safety program – Closure plan updating – Environmental management – Change management – Succession planning – Permitting support – Periodic site investigations – Risk and FMEA – Training programs – Design changes – Major construction changes	– Final closure plan – Construction management – QA & QC – Dam safety program – Long-term care and monitoring

The FMEA process (or similar processes such as Failure Modes Effects and Criticality Analysis [FMECA] or Potential Failure Modes Analysis [PFMA]) was identified during the workshop as a potential step in accepting an EoR engagement on an existing project. The authors note that while engineering risk assessments and FMEAs have been widely-used in water dam practice for several years, the mining industry has only recently begun to increase the use of these methods.

5 NEXT STEPS

Through collective identification by TSF professionals of the elements of TSF projects, definition of the EoR, and TSF participant responsibilities, the standard of practice can be communicated and the potential risks and liability for the parties recognized. An EoR and an EoR’s firm can then begin to identify the internal and external programs to help manage the responsibilities and risks. Internal programs may include the firm’s QA program and special risk assessment, design review, and construction/operation review steps, along with contract positions and insurances. External programs, such as the adoption of industry practice guidelines, are most effective when broad support by practitioners, owners, and even regulators is obtained. The need for direct engagement with industry groups (such as MAC, ICMM, and others) was emphasized at the workshop and by the survey respondents.

Using information from the workshop supplemented by survey results and the work of others (e.g., CDA, DIAC), GBA is in the process of developing a document titled “*National Practice Guideline for the Tailings Storage Facility (TSF) Engineer of Record (EoR)*” that is planned for release by the end of 2017. The purpose of the guideline is to assist in identifying the roles, responsibilities and accountabilities of the EoR for TSFs, as well as other project participants including the Designer of Record (DoR) where this differs from the TSF EoR, the owner or operator, the regulator, and the third-party reviewer or ITRB.

The guideline will address the responsibilities and range and nature of services that should be included in the scope of service for the DoR, who is the engineer who signs and seals instruments of professional service, and the additional responsibilities and scope of service(s) required for the DoR to accept designation as the TSF EoR. The guideline is intended to clarify the roles and responsibilities of the DoR and the TSF EoR for projects undertaken in the United States. It is typically anticipated that the DoR will become the TSF EoR, providing continued support during TSF operations. The guideline presents concepts that, when consistently applied, have proven extremely beneficial in terms of reducing risk of failure of TSFs.

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Risk Assessment and Management Using Layers of Protection Analysis and Bowties

P. Bedell
Golder Associates Ltd., Vancouver, BC, Canada

J. Reinson
Goldcorp Inc., Vancouver, BC, Canada

W. Danaher
Risk Management Intercontential Pty Ltd, Brisbane, Australia

M. Malgesini
Golder Associates Inc., Lakewood, CO, United States

ABSTRACT: This paper describes the use of a "layers of protection analysis" (LOPA) for a risk assessment for the design and operation of tailings dams. Presented using a bowtie diagram, the LOPA is considered to improve upon the more typical risk assessment approach of using unmitigated and mitigated risk heat maps. High consequence / low likelihood risks are difficult to effectively communicate and manage by simple risk assessment tools like a heat map; tailings dam failures fall into this category. The focus, therefore, needs to be on the controls—the barriers—and not the risk ranking. The identification of independent controls that are effective in preventing the consequence are useful in design, review, and operations. The auditability of the barriers and their reliability are significant strengths to this approach and aid in decisions regarding the adequacy (type and number) of barriers. The bowtie analysis and presentation style allow for ease of clear and complete communication and may be readily linked to management systems and documentation whilst of critical importance in design, review, operations, and audits. An example case study is presented and discussed.

1 INTRODUCTION

1.1 *Risk assessment*

The assessment and management of the risks associated with a tailings storage facility (TSF) is recognized as international good practice. The objective of a risk assessment is to understand and determine the risk levels of the tailings storage facility TSF. Through identification and mitigation of risks, successful long=term operation and performance of the TSF may be achieved.

This paper discusses the layers of protection analysis (LOPA) approach to a risk assessment and management for a TSF. The key risk—the incident of interest—is the release of tailings; the prime objective being to avoid this from occurring.

The typical approach of risk assessment, using consequence and likelihood to determine the level of risk, is discussed along with its limitations. The LOPA approach is described along with its advantages for properly understanding the risks, identifying controls, and communicating the components. A case study of its use is presented in Section 3.

The LOPA approach focuses on the controls to avoid the release of tailings. It also identifies the controls to reduce the consequences post-release; as discussed below, the focus of a properly designed, managed, and operated TSF is on avoiding the release of tailings as reducing the consequences is difficult to infeasible. As the consequences of a tailings release from a TSF are difficult to infeasible to reduce, the key is to focus on how to avoid the tailings release in the first instance. As discussed below, the LOPA approach promotes the identification of the threats that would result in a tailings release and what controls are to be in-place to provide barriers (or independent protection layers) to the tailings release. It also promotes the controls to be put in-place to reduce the consequences of a tailings release. It is recognized and emphasized that avoidance of the tailings release is the primary focus of the design, management, and operation of a TSF. Post-tailings-release controls are critical components of TSF operation but are no replacement for those to prevent the tailings release in the first instance.

1.2 *Typical risk assessment approach*

The typical approach to such a risk assessment is to reduce the risks of a tailings storage facility (TSF) through a consequence-and-likelihood assessment; risk being the product of these two parameters. Risk assessments are carried out pre- and post-mitigation such that the risk is decreased to an appropriate level. Mitigation measures are captured as part of the risk mitigation and used during the design, operation, and management of the TSF to keep the risk level from increasing. The results of the risk assessment are commonly used to develop the risk ranking for the mine site.

The risk of an incident is defined as the product of the consequence—what happens if the incident occurs—and the likelihood of the consequences. Incidents of interest are those that are unwanted; for the purposes of this paper, the incident is the release of tailings from a TSF. The results from this type of risk assessment are presented in a heat map as shown in Figure 1.

Consequences are typically defined in an increasing level of impact from low to significant to high to very high to extreme. The definitions of these levels are typically sourced from the Canadian Dam Association's *Technical Bulletin: Application of Dam Safety Guidelines to Mining Dams* (CDA 2014). The consequence level is presented in the vertical axis of the heat map in Figure 1.

Likelihoods are typically defined on a frequency-basis or a qualitative-basis. Typical likelihood levels are rarely, unlikely, possible, likely, and almost certain. The likelihood level is presented in the horizontal axis of the heat map in Figure 1.

The risk assessment level is determined by the levels shown in the heat map in Figure 1—low to medium to high to very high—based on the consequence and likelihood. The risk level is then, typically, input into a mine site risk ranking register.

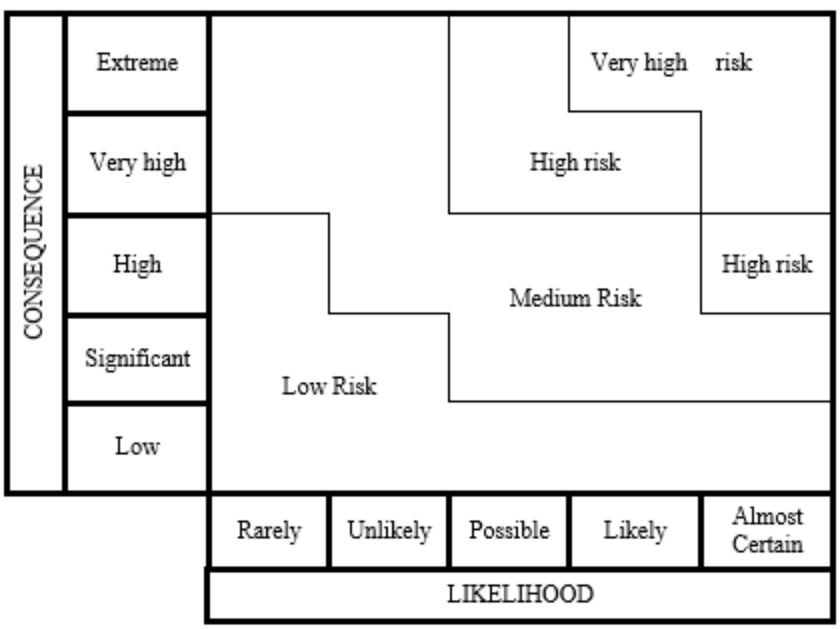

Figure 1. Risk assessment level heat map.

1.3 *Limitations to the typical risk assessment approach*

Through the development of a typical risk assessment (Section 1.2), most TSFs are classified as medium risk facilities as a result of their high consequence / low likelihood status. The key limitation to the typical risk assessment approach and the associated heat map is that the key controls that must be in-place to properly understand, manage, and control the likelihood of an incident from occurring are not communicated clearly. This limitation often results in a fundamental misunderstanding of TSFs; namely, that the consequences of a tailings release cannot be readily mitigated. The key realization is that the likelihood of a release of tailings from the TSF must be assessed continuously and properly managed at all times—this is not self-evident from the typical risk assessment approach and its associated heat map. A typical high consequence / low likelihood risk ranking does not advance understanding.

1.4 *A suggested approach to risk assessment and management—layers of protection analysis*

The objective to successful TSF operations is to avoid the release of tailings, in the first instance, and to reduce the consequences post-release. The key to achieving this objective is through the use of controls to stop the release of tailings and those to reduce the consequences should a release occur. The method suggested to assess this is the LOPA approach and the associated bow tie diagram. The suggested approach is described below and a case study of its use is presented in Section 3.

2 LAYERS OF PROTECTION ANALYSIS APPROACH

2.1 *Description of approach*

The use of a layers of protection (LOPA) approach to risk management is considered to be a more appropriate tool for the proper design, management, and operation of a high consequence / low likelihood facility such as a TSF. The focus of a LOPA is on the controls to avoid the release of tailings and those to reduce the consequences of a release rather than simply a risk ranking. As noted above and discussed below, the primary focus of a properly designed, managed, and operated TSF is on avoiding the release of tailings as reducing the consequences is difficult to infeasible.

The LOPA approach was developed in the petrochemical industry and has been successfully used for many years. The approach focusses on identifying threat-incident-consequence pathways. A threat is defined as a condition that may initiate the incident. The incident is defined as the unwanted event; in this case, the release of tailings from the TSF. The consequence is the possible outcome from the incident having occurred.

2.2 *Bowtie diagram*

The LOPA analysis is presented visually in what is commonly referred to as a "bowtie diagram"; an example of a simple bowtie diagram is shown in Figure 2. The centre of the diagram shows the incident. The threats are presented on the left-hand side and the consequences on the right-hand side. As the threats and consequences fan out from the incident when presented, the resulting bowtie shape provides the diagram with its name. The bowtie diagram presents the threat-incident-consequence pathways developed during the risk assessment.

A key attribute of the LOPA is the bowtie diagram itself—the visual presentation of the threat-incident-consequence pathways, the controls, and the measures required to ensure that the controls remain effective—seen in Figure 2. The effective communication from the use of the bowtie diagram is, as discussed below, invaluable for designers, owners, operators, and reviewers. It provides a clear illustration of the key items to be managed to properly manage the risk of a TSF to avoid the release of tailings—from design to management to operation to auditability perspectives. The effective communication of the LOPA approach through the development of the bowtie is discussed in the case study below.

2.3 *Role and description of controls*

Each threat-incident-consequence pathway is considered to be feasible; no preferential pathway is identified or considered as all pathways result in an unwanted event occurring and adverse consequences resulting. Therefore, controls are required to be put in-place to act as barriers for both the threats (pre-incident) and the consequences (post-incident). Key requirement for the controls in a LOPA analysis is that controls are functional, available, reliable, survivable, independent, and auditable. With a "strength in depth" approach, each control for a given threat or consequence must act as a barrier should any of its sister controls fail (often, a bow tie analysis does not include this level of rigour). The controls for each threat and consequence are shown in Figure 2. Refer to Figure 3 for symbology definitions.

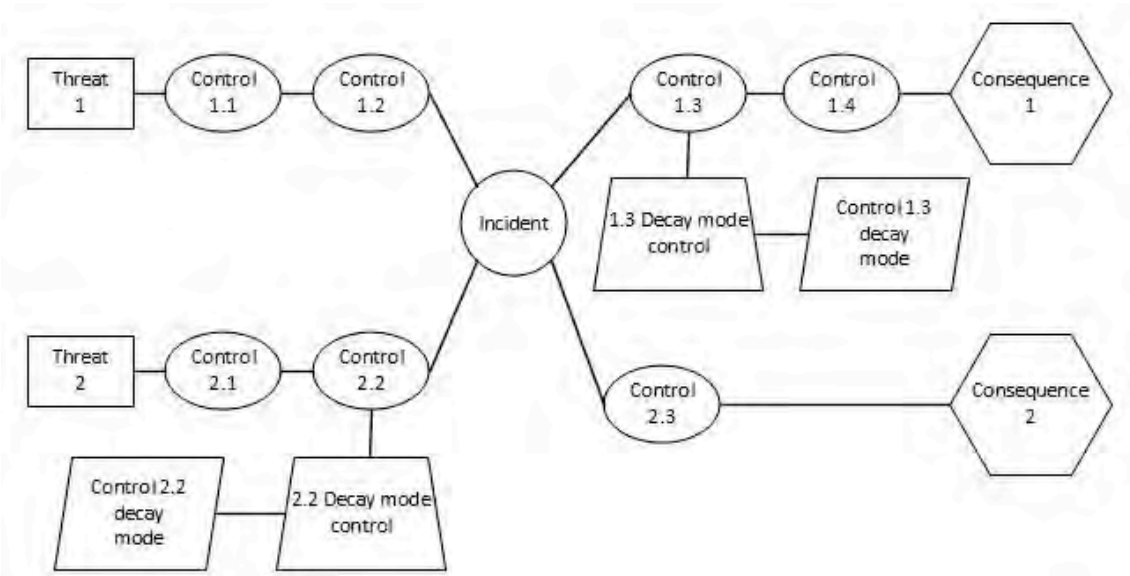

Figure 2. Example bowtie diagram.

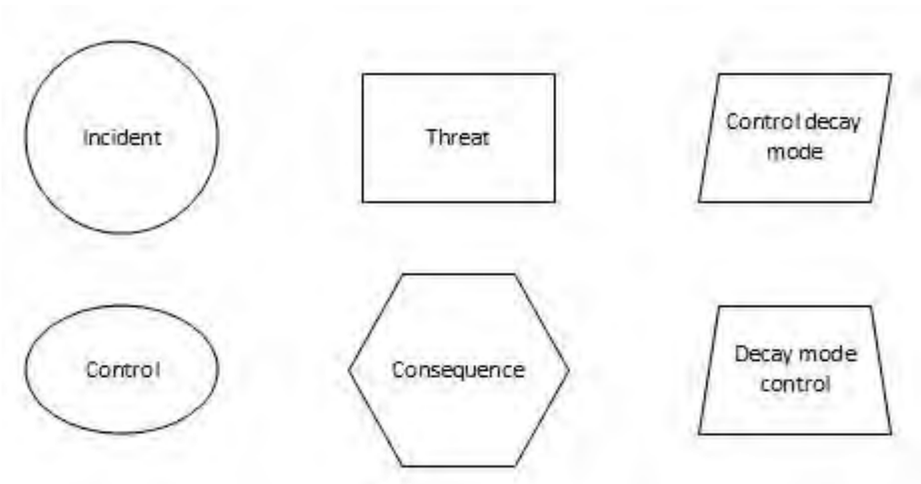

Figure 3. Bowtie diagram symbology.

2.4 *Reliability of controls*

With the absolute requirement for independent controls to act as barriers, consideration to the degradation of controls must be given. The question "what could compromise the effectiveness of this control?" results in the identification of control degrade modes. Decay mode controls are then identified. Control decay modes and decay mode controls are also shown in Figure 2. Refer to Figure 3 for symbology definitions. With decay mode controls in-place, the reliability of the controls is maintained. Recognition of decay modes provides a linkage to management systems.

In summary, threats are to be stopped before an incident occurs and consequences are to be mitigated after an incident occurs. The use of independent controls is used to achieve both. The reliability of the controls is key; therefore, identifying the decay modes for each control, and the corresponding decay mode controls, are keys to having reliable controls in-place.

2.5 *Types of controls*

There are, broadly, three types of controls: passive engineering, active engineering, and procedural. Passive engineering requires no human interaction for a control to be initiated while an active engineering requires human interaction for a control to be initiated; a procedural control requires complete reliance on human performance to be in-place. The control type hierarchy, from highest to lowest, is passive engineering followed by active engineering and then procedural.

As an example, consider an incident of a building fire to have occurred. To avoid the consequence of loss of life: a passive engineering control is construction using non-combustible materials; an active engineering control is the fire extinguisher (someone must use the extinguisher); and a procedural control is the fire evacuation plan (relies on people knowing and executing the plan).

Good practice involves having a suitable number of controls in-place for each threat and consequence with emphasis on having passive and active engineering controls as priorities over procedural controls. While not always possible to have passive engineering controls in-place and, at times, relying on only procedural controls, the type of controls is an important consideration when developing the LOPA approach. The case study presented in Section 3 provides additional comments regarding control types and their use.

2.6 *Misuse of LOPA approach*

It is recognized that the misuse of the LOPA approach is possible. The identification of overly-generalized threats and non-truly-independent controls are two common errors. The identification of specific threats and independent controls are critical for the proper use and application of the LOPA approach. It is recognized that too much detail may be captured in the LOPA (e.g., extremely detailed listing of threats), but it is the experience of the authors that this rarely adversely affects the outcome of the LOPA approach; more detail is better than too generalized considerations. It is essential that barriers used in LOPA are functional, available, reliable, survivable, independent, and auditable.

3 USE OF LAYERS OF PROTECTION ANALYSIS APPROACH—A CASE STUDY

3.1 *Project description*

The use of the LOPA approach is described for the design of a TSF. The TSF has been in operation since 2008 and is raised in stages in order to provide tailings storage in support of mining operations. Key considerations for the design included a downstream community, on-going operations, and construction of the current stage, and future development stages. A risk assessment was a key component of the design and included the use of the LOPA approach.

3.2 *Project bowtie*

The LOPA approach was developed early on in the design process and the associated bowtie was updated and revised throughout the design stages. The bowtie, simplified for presentation purposes, is shown in Figure 4. For clarity, controls, control decay modes, and decay mode controls are not shown in Figure 4; select examples of these will be discussed below. Relating to the incident of the release of tailings from the TSF, nine threats and four consequences were identified to develop the threat-incident-consequence pathways. It is noted that the actual bowtie for the design had a greater number of threats; in consideration of the presentation herein, the threats shown in Figure 4 have been generalized.

A variety of stakeholders participated in the design: dam design team, owner team, operators, community relationship team, environmental team, mine management, and the independent tailings review board. The bowtie was an invaluable tool for risk communication amongst the stakeholders to ensure that the appropriate threats and consequences were identified and that their controls were developed.

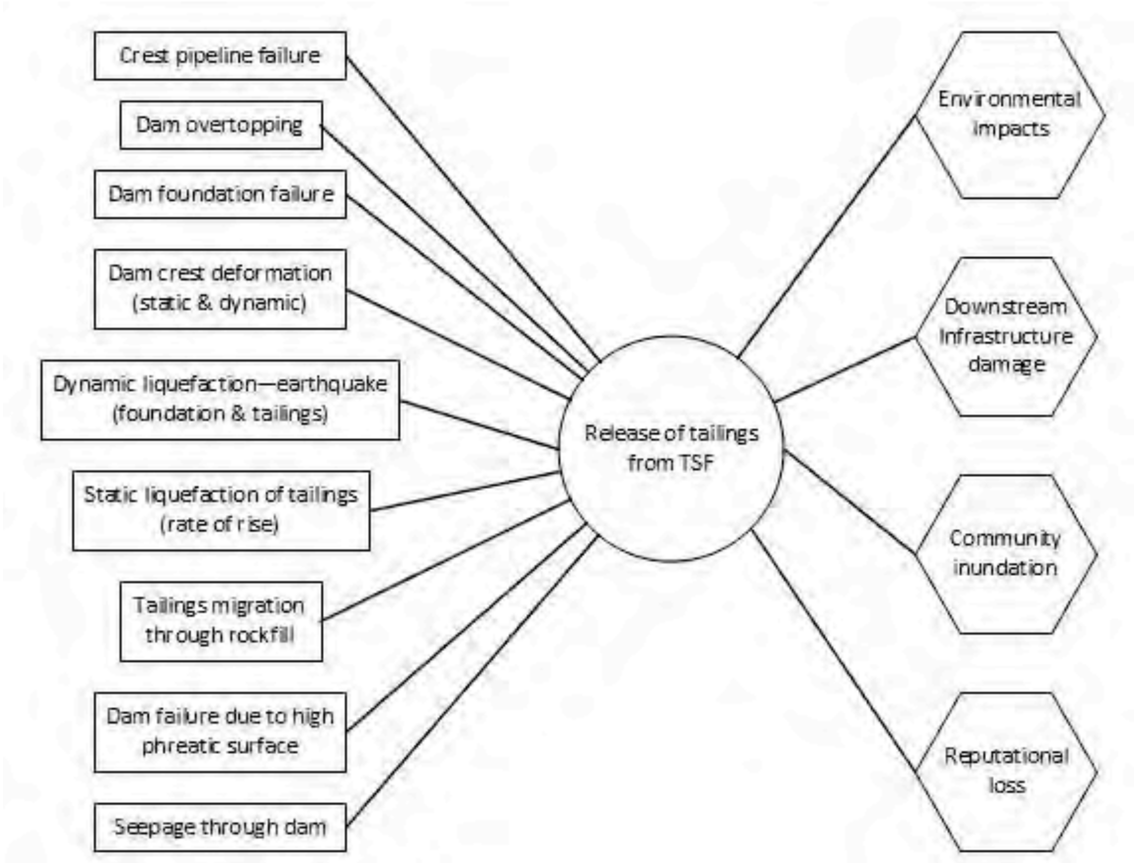

Figure 4. Simplified project bowtie diagram.

3.3 *Bowtie excerpt No. 1: threat-control-incident pathway*

An excerpt from the bowtie showing the threat of dam crest deformation, under static or dynamic (i.e., seismic) conditions, that could result in the release of tailings from the TSF is shown in Figure 5. Refer to Figure 3 for the bowtie diagram symbology. There is but a single control for this risk—a dam design based on the site-specific seismic and deformation analyses. The bowtie clearly illustrates that this control is the sole barrier to the incident from occurring, thus underlining the importance of the control. An inadequate design is the control decay mode and is avoided through the use of the decay mode control: the review of the design by the independent tailings review board. The threat-control-incident branch shown in Figure 5 illustrates how the bowtie is able to present individuals and groups involved in the LOPA; in this case, the dam design team (responsible for the design) and the independent tailings review board (responsible for the design review).

3.4 *Bowtie excerpt No. 2: threat-control-incident pathway*

A second threat-control-incident excerpt from the bowtie is shown in Figure 6.. The threat of static liquefaction of the tailings (due to the rate of rise of the facility) is the threat and two controls were developed to act as barriers to the release of tailings from the TSF. One of the controls is "instrumentation monitoring leading to intervention"—the process by which monitoring results trigger action by operations. The control decay mode to this control is "inadequate response to monitoring"—namely, monitoring for the sake of monitoring. In order to ensure that the control is effective, the decay mode control put in-place is the trigger-action-response-plan (TARP) that defines what actions are to be taken based on the results from the monitoring. As shown in Figure 6, the specific section of the operation, maintenance, and surveillance (OMS) manual is cited in the bowtie diagram. The inclusion of this information is a very useful tool for operators and for auditors of the LOPA; namely, management and the independent tailings review board. This illustrates the use of the bowtie diagram as a design tool, an operation tool, and an audit tool that is clearly and readily communicated.

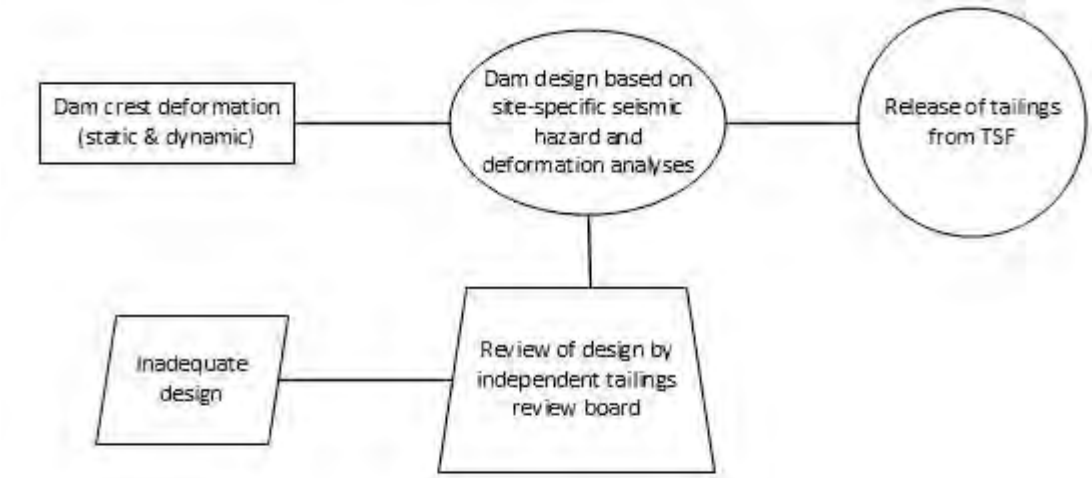

Figure 5. Excerpt No. 1 from project bowtie diagram: threat-control-incident pathway.

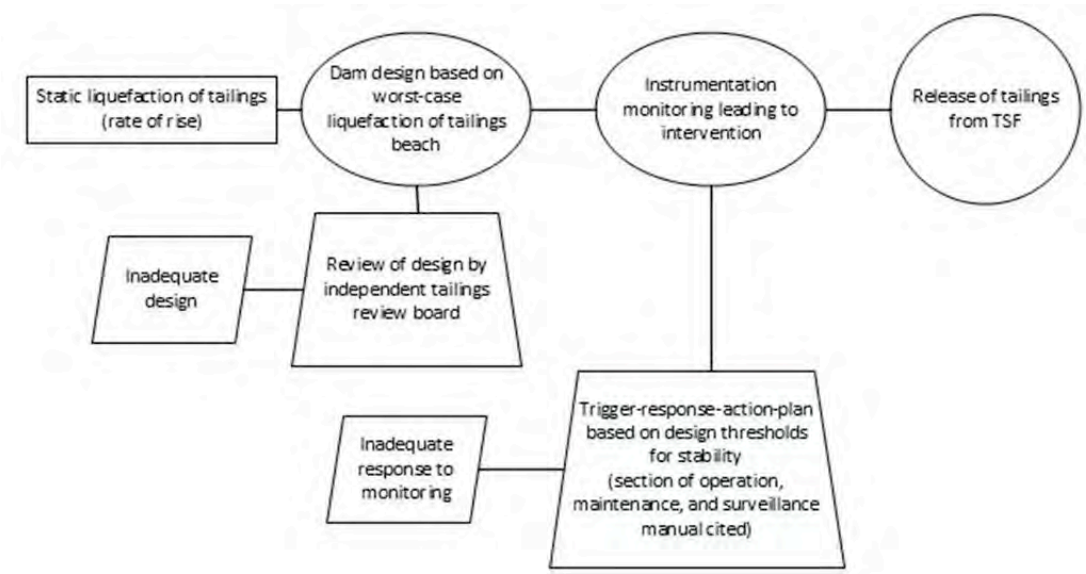

Figure 6. Excerpt No. 2 from project bowtie diagram: threat-control-incident pathway.

3.5 *Bowtie excerpt No. 3: incident-control-consequence pathway*

An excerpt from the bowtie showing an incident-control-consequence pathway is shown in Figure 7. This pathway contains the critical consequence of the project: the inundation of the community downstream of the TSF. Located some 500 m from the TSF, the community would be inundated in the event of a tailings release as determined the dam break assessment carried out during the feasibility design. The control to mitigate the consequence is the emergency plans; namely, the emergency response plan and emergency preparedness plans. While these plans will not avoid the inundation of the community in the event of tailings being released from the TSF, they will serve to avoid loss of life through the notification and evacuation of the community. As shown in Figure 7, the control is ensured through the communication and drilling of these plans.

It is emphasized that the controls put in-place on the post-incident side of the bowtie do not replace or de-emphasize those required to prevent the release of tailings from the TSF in first place. Dam designers, owners, and operations must undertake all appropriate and feasible approaches and actions to ensure that tailings are not released from a TSF; the use of international good practices, including the engagement of an independent tailings review board, through all phases of the mine life is strongly encouraged by the authors.

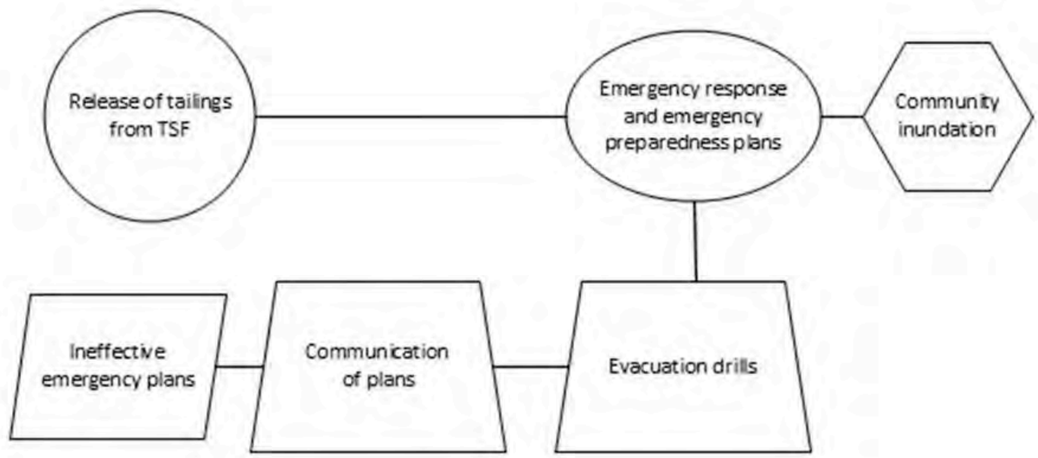

Figure 7. Excerpt No. 3 from project bowtie diagram: incident-control-consequence pathway.

4 SUMMARY

The assessment and management of the risks associated with the management of tailings is recognized as international good practice. The objective of a risk assessment to understand and determine the risk levels of the TSF. Through this, the proper operation and performance of the TSF over the long-term may be achieved.

The limitation of the traditional risk assessment approach and its heat map output is that the criticality of controls required to ensure that high consequence incidences, such as the release of tailings from a TSF, is not always recognized and jeopardizes the ability to keep likelihood as low as possible at all time. Application of a simple risk ranking may detract from the proper understanding of the control of the risks from being fully developed and communicated to all stakeholders. For this reason, the use of the LOPA approach is strongly encouraged during the risk assessment, design, and operation of a TSF.

The use of the LOPA approach as a risk assessment and management for a TSF enables the identification of threat-incident-consequence pathways and the controls required to avoid the release of tailings and those to mitigate the consequences. The bowtie diagram developed during the LOPA is an excellent communication tool for the various stakeholders involved with the TSF. The bowtie is developed to include groups/individuals who are involved in the controls and also specific references to such documentation as OMS manuals.

The LOPA approach focuses on the controls to avoid the release of tailings. It also identifies the controls to reduce the consequences post-release; the focus of a properly designed, managed, and operated TSF is on avoiding the release of tailings as reducing the consequences is difficult to infeasible. As the consequences of a tailings release from a TSF are difficult to infeasible to reduce, the key is to focus on how to avoid the tailings release in the first instance. Post-tailings-release controls are critical components of TSF operation but are no replacement for those to prevent the tailings release in the first instance. The proper development of a bowtie diagram requires the identification of specific threats, controls, and consequences; over-generalized items are inappropriate for the proper risk assessment of a TSF. Methodologies for ranking the quality of barriers are available, but not able to be discussed herein.

REFERENCE

Canadian Dam Association (CDA) 2014. *Technical Bulletin: Application of Dam Safety Guidelines to Mining Dams.*

Oil Sands Closure
and Reclamation

Closure landform design for an oil sands external tailings facility

Monica Ansah-Sam
Canadian Natural Resources Ltd, Calgary, AB, Canada

Gord McKenna[1] & Erin Olauson[2]
BGC Engineering Inc., [1]Vancouver, BC, [2]Calgary, AB, Canada

ABSTRACT: Canadian Natural Albian Sands' Muskeg River Mine External Tailings Facility (ETF) is located approximately 70 km north of Fort McMurray, Alberta. It is at its maximum dyke elevation with hydraulic sand infilling operations ongoing. The South Expansion Area, which abuts the ETF, will reach its final height in the next few years.

The closure landform designs for these facilities have progressed from the conceptual design stage through to the most recent planning-level design. Detailed designs will begin after coarse sand tailings infilling activities are completed. To support the evolving designs, ongoing monitoring and annual site investigation programs will provide confirmatory details for the landform design.

One of the key design goals is to be able to delicense the structures by converting them to solid earthen structures, with the intent that the structures will not be regulated as dams. This conversion will be accomplished through stabilization or removal of potentially mobile material, designing to avoid excessive ponding of water, establishing robust drainage outlets for the plateaus, and designing for geotechnical and erosional stability of slopes.

After coarse sand tailings infilling is complete, mine waste will be placed on the resulting surface to direct surface water and provide topographic diversity. The resulting surface will receive a reclamation cover, and be revegetated to create an interconnected system of boreal forest uplands, riparian areas, and wetlands for wildlife habitat and traditional use. This paper provides details of the integrated design and site investigation processes along with recent insights and opportunities for similar landforms.

1 INTRODUCTION

Canadian Natural Albian Sands (Canadian Natural) is in the process of developing closure landform designs for its External Tailings Facility (ETF) and the South Expansion Area (SEA) at its Muskeg River Mine (MRM). For large landforms such as the ETF and SEA constructed over a number of years, there are typically several levels of closure/reclamation design. The site-wide closure landscape plan (Ansah-Sam et al. 2016) provides the initial closure designs for these structures and sets the overall drainage layout. Further work on a feasibility-level design, dyke stability assessments, and annual site investigation programs provided the main inputs for the next stage of design, labeled the planning-level design and discussed in the following sections. A planning-level design provides a surface for mine planners to estimate volumes of mine waste fill (mostly lean oil sand) and cut into coarse sand tailings, as well as riprap and filter material volumes for channel armouring. It advances the feasibility design, allows reasonably detailed tailings planning and scheduling, and identifies specific items that require additional assessment during the detailed design phase.

There is limited precedent for closure landform designs of large oil sands tailings facilities (McKenna & Cullen 2008; Russell et al. 2010), and none as yet has entered the new dam delicensing process (OSTDC 2014). The design presented in this paper is based on guidelines and publications that use oil sands-specific data, and the experience and judgment of the authors with closure landform design, construction, and monitoring at other operations. This case history also highlights how closure landform design involves balancing a few key design criteria with the long-term performance of the landform.

2 BACKGROUND

Muskeg River Mine is an operating oil sands open pit mine, extraction plant, and processing facility located approximately 70 km north of Fort McMurray, Alberta, Canada. The ETF and SEA are adjacent to the Athabasca River / Highway 63 and the Muskeg River (Figure 1) and cover approximately 13 km^2. The crest elevation of the ETF is 340 masl, with a maximum dyke height of approximately 60 m. The final SEA crest elevation will be 306 masl, with a maximum dyke height of approximately 30 m. Canadian Natural's tailings operation is focused on closure landform construction, with dam delicensing and reclamation certification as the ultimate end goals for these structures.

During hydraulic construction of the dyke and its beaches, clay-rich fines segregate and densify in the central area of the ponds to form a fluid fine tailings deposit (COSIA 2012). These fluid tailings must be removed or otherwise treated to allow delicensing and reclamation of the facility. The ETF pond infilling plan calls for low density tailings (less than 55% solids by weight) to be displaced by hydraulically placed coarse sand tailings (CST) and dredged to a containment cell in a mined-out area or placed in thin lifts for drying. Tailings with solids contents of 55 to 65% will remain in the northwest corner of the ETF for future treatment. It is anticipated that tailings with densities greater than 65% will be captured below the CST during infilling. Some of the tailings in the northeast portion of the ETF will be capped in-place with CST.

The SEA is a CST-filled structure; CST is beached from the west, south, and east dykes. The fluid fine tailings are removed during construction and pumped to the ETF. Some entrapped pockets of fluid fine tailings are expected to remain in the SEA.

Figure 1. Location of Muskeg River Mine ETF and SEA.

For both the ETF and SEA, the plan is to track-pack the top 4 m of CST using dozers to create a compacted layer of trafficable sand. This compacted CST is intended to form the base unit upon which mine waste will be hauled and spread to create the mesotopography of the closure landforms (e.g. McKenna et al. 2011).

3 PLANNING-LEVEL CLOSURE LANDFORM DESIGN

The planning-level closure landform design for the ETF and SEA builds on the feasibility-level design, with more detail from the latest infilling plans, site investigation data, and dyke stability analyses to continue moving the design toward the goals of closing, delicensing, and obtaining reclamation certification for these structures. The specific objectives of the design are to:

- Estimate the material quantities for mine waste fill, riprap armouring, and filter materials, to allow Canadian Natural to develop a cost estimate and schedule for construction.
- Identify risks and opportunities for upcoming construction, and mitigation options for these risks where needed.
- Establish a work plan for detailed design and subsequent construction. This work plan includes details regarding site drilling, sampling, and instrumenting of the existing tailings deposits and newly infilled areas, and the required analyses to support the detailed design and construction.

A design surface (the end of construction topography) was developed with an overall surface water drainage plan for the ETF and SEA, which includes channel designs for the estimated flows plus other elements to support the wildlife habitat end land use. The design surface includes various sizes of hummocks constructed using mine waste to provide topographic variation. The hummocks were designed using limit-equilibrium slope stability analyses and estimates of tailings strengths. The design includes considerations for settlement based on one-dimensional settlement analyses.

There are several key project risks that were previously identified and were the focus of much of the design effort. The risks included the regulatory and technical guidelines for delicensing the ETF and SEA dykes (given the regulatory process has yet to be formally applied to any structure and the delicensing guideline has not been adopted by the regulator), the ability to construct topography using mine waste on soft tailings, managing settlement of low density tailings, and constructing long-term drainage systems on erodible tailings substrates. Other design considerations include the need for integrated annual site investigations to confirm the as-built conditions as new areas are infilled, a freeboard assessment to transition the ETF and SEA from closed ring dykes to landforms with outlets (as the pond area shrinks with time as infilling progresses), and development of methods and potential contingencies to manage settlement of soft tailings areas remaining after infilling.

4 DESIGN APPROACH

The following section describes the overall design approach used to develop the planning-level closure landform design for the ETF and SEA.

4.1 *Developing the design basis*

A design basis was prepared for the planning-level landform design, following the framework described by Ansah-Sam et al. (2016) and incorporating many of the site-wide closure plan design objectives and criteria. The design basis defined specific objectives and design criteria to support the overall ETF and SEA goals of delicensing and eventually obtaining reclamation certification. The specific objectives included:

- Design to provide safety for personnel and equipment, and meet dam safety criteria for all stages of tailings infilling, stabilization, and reclamation.

- Create a safe and geotechnically stable landform (including the plateau, drainage outlet, and slopes) that requires only minimal long-term maintenance.
- Control surface water through design of topography, water courses, and wetlands such that water is safely conveyed off-landform to the proposed lease-wide surface water drainage system in a way that minimizes erosion and sedimentation.
- Integrate the landform with the lease-wide closure landscape including natural areas and adjacent mining landforms.
- Allow efficient tailings placement and capping to form a surface that can be reclaimed using typical mine reclamation equipment.
- Allow progressive reclamation as areas of the landform become available.

4.2 *Predicting the tailings zones at the end of infilling*

The ETF and SEA are undergoing major changes as CST is being used to displace the fluid fine tailings toward the fluid return systems for removal to in-pit cells or for other treatment technologies. The infilling plan was used to create a map showing the anticipated locations of various tailings types at the end of infilling. The site investigation data from existing tailings zones were used to characterize the existing tailings and help develop the design tailings conditions for the newly infilled areas. This map will be continually updated, as infilling progresses and site investigations in the newly infilled areas document the actual stratigraphy and conditions.

4.3 *Determining the main design constraints*

A full list of design constraints for the ETF and SEA structures were developed, with the main constraints outlined as follows:

- Maximizing waste placement within a specific topographic design envelope: Canadian Natural and its dyke designers developed an envelope representing the maximum height and required offsets from the dyke crests for mine waste placement on the ETF and SEA plateaus. During the design process, this constraint was revisited with dyke designers to allow for a more robust drainage plan.
- Minimizing disturbance to already reclaimed areas: Some of the ETF and SEA slopes have already been covered with reclamation material and revegetated; re-disturbance of these areas is minimized in the design.
- Locating the ETF outlet to connect with the outlet channel and to minimize relocation of existing infrastructure: The ETF outlet will be connected to a channel indicated in the site-wide closure plan that directs surface water northeast toward the end pit lake (EPL), which will be constructed prior to the end of mine life. There is a mine waste dump to the west and existing infrastructure to the east that constrains the outlet and channel location.
- Maximizing the volume of surface water draining to the EPL: The design directs as much surface water flow as possible toward the EPL; any flow towards the SEA sump will be managed separately.

4.4 *Developing the drainage plan*

The overall design concept for the ETF and SEA is to direct surface water away from the dyke crest as quickly as possible. The main constraint affecting this concept is the waste placement envelope required for dyke stability. This envelope limits waste placement within 50 to 250 m of the dyke crest, and limits the maximum height of waste to 2 m in the north and 10 m in the south of the ETF, and 3 m for the SEA. This constraint effectively removed the option of doming the ETF and SEA such that surface flow would be directed off the plateaus and down the dyke slopes. It also removed the possibility of building up along the dyke crests (raising the dykes) to drain the entire ETF and SEA plateaus in toward a central channel and direct all surface water through the outlets.

The ETF and SEA designs were initially divided into the Perimeter Zone (described below) design and the plateau design. Within the Perimeter Zone, surface water is designed to shed outward. Beyond the Perimeter Zone, further in toward the centre of the plateau, surface water drains inward through collector channels to a central channel that connects to the outlet. After review of the long-term risks associated with shedding water outward and down the dyke slopes, Canadian Natural engineers and consultants revisited the mine waste envelope constraint with dyke designers. The mine waste envelope was adjusted, which resulted in a revised drainage plan. The new drainage plan has mine waste being used to build up the base topography to drain the entirety of the Perimeter Zones and plateaus inward through collector channels, to a central channel that connects to the outlet. Hummocks are used to create additional topographic variation and water table control on the ETF. Figure 2 is an illustration showing the closure landform design developed for the ETF and SEA.

4.4.1 *Perimeter Zone and Geotechnical Buffer Zone*

The ETF and SEA designs use a Perimeter Zone and Geotechnical Buffer Zone to delineate key areas adjacent to the dyke crests (see CEMA 2014), as part of reducing dam safety risks and increasing the potential for delicensing. The Perimeter Zone is defined as the area immediately upstream of the dyke crest and is designed to eliminate the potential for ponding water. The Geotechnical Buffer Zone is defined as the area upstream of the Perimeter Zone, where water may be allowed to pond temporarily during large precipitation events, but not contain permanently ponded water. Figure 2 provides a sketch of the Perimeter and Geotechnical Buffer Zone designs described below. Preventing water from ponding in these zones greatly reduces the risk of dyke slope instability, piping, and erosion.

The Perimeter Zone for the ETF and SEA closure landform design is defined as the area within 200 m of the downstream dyke crest, where the topography is sloped inward at minimum 1%. The Geotechnical Buffer Zone is defined as the 400 m inside the Perimeter Zone (toward the centre of the facilities). This area was defined based on limiting the hydraulic gradient to less than 5% from the edge of the Geotechnical Buffer Zone to the top of the starter dyke, to limit the potential for piping. Channels within the Geotechnical Buffer Zone are designed as vegetated waterways with a typical minimum slope of 0.5% toward the main plateau channel. They are designed to limit the potential for backflooding from the main plateau channel.

4.4.2 *Channel design*

Channels to convey surface water runoff through the landscape were designed to pass the design event with acceptable erosion, without channel failure due to downcutting/loss of armour, lateral migration (excessive bank erosion), or avulsion. Channels on the ETF and SEA plateaus were designed as vegetated channels that follow the Golder (2004) Vegetated Waterways Design Guidelines, with the ETF central channel following guidance provided in the CEMA wetland manual (CEMA 2014). Channels within the ETF and SEA Perimeter Zones were designed as armoured channels able to pass the peak flow estimated for the probable maximum precipitation (PMP) event, as part of the delicensing strategy.

Figure 2: Illustration of the closure landform design for the ETF and SEA. (Illustration by Derrill Shuttleworth)

4.4.3 *Setting the outlet elevations*

The ETF and SEA require outlets to safely direct water from the plateaus to a downstream channel and receiving area. As highlighted by the OSTDC (2014), the outlets require a robust design (hydraulically and geotechnically), as they are critical to both long-term performance and delicensing.

Maximizing the ETF and SEA tailings storage volume was a main factor in developing the outlet invert elevations. The second factor considered was maintaining a nominal gradient from the furthest channel on the plateaus to the outlet. The overall minimum gradient selected was 0.3%. While some settlement along the channel alignments is expected, this gradient was chosen to maintain overall positive drainage toward the outlet in the long-term. The third factor was to limit the extent of pond formation as a result of potential blockage at the outlet and with some settlement. A 3 m high blockage (a beaver dam being a plausible blockage) combined with 1 m of plateau settlement, creating a maximum 4 m deep pond at the outlet, was considered as the design scenario. A pond of this depth on the ETF would be about 1500 m long and 150 m wide; on the SEA, the pond would be about 800 m long and 150 m wide. While these are large ponds, the steeper slopes of the collector channels limit backflooding and keep the pond within the main channels, and honour the Perimeter Zone and Geotechnical Buffer Zone criteria.

Both the ETF and SEA outlets are designed to be excavated into compacted dyke fill, to limit the potential for settlement. They are designed to be armoured with durable riprap to provide erosion control and help to deter beavers from building within this portion of the main channels (Eaton et al. 2013). While designed to accommodate some beaver dams and settlement, annual inspection and maintenance of these critical outlets will likely be required in the near term.

4.5 *Planning for settlement*

The tailings remaining in the northwest portion of the ETF between an estimated 55% to 65% solids are planned to be treated to achieve a specific solids content and strength. Settlement of these tailings must be limited; excessive areas and volumes of ponded water would preclude delicensing of the structure. Controlling settlement means that the solids content of the material likely needs to be above 75% solids at the end of mine life or prior to placement of the designed landform. Analysis indicates that construction of the designed landform requires a minimum peak undrained shear strength of approximately 30 kPa in the deposit.

While Canadian Natural develops amendment plans that will meet the settlement and shear strength requirements in the design, options for contingencies are being developed. One contingency is the removal of these materials from the ETF and replacement with CST. The closure landform design adopted this contingency as its basis since this is not expected to be significantly different from the criteria outlined above after successful amendment of the tailings; the main plateau channel is specifically located in this material, to limit the potential for excessive settlement directly upstream of the outlet.

4.6 *Creating topographic variation with hummocks*

Once a drainage plan was developed, hummocks were added to the ETF plateau to add topographic variation (McKenna et al. 2011) and to help control the water table. The ETF hummocks were designed to fit within the waste placement envelope constraint. The SEA waste envelope limited the SEA plateau design to a maximum 3 m of waste; the topography was gently sloped toward the central channel and no hummocks were designed.

The design approach for developing the topography on the ETF plateau was based on the following:

- Review of current tailings deposit strengths: site investigation and sampling results from the many years of annual tailings investigations carried out at the ETF and SEA were reviewed to identify the strengths of the currently deposited materials and what might be expected for the areas that have yet to be deposited.
- Developing reasonable geometry for hummocks: Using the maximum waste placement envelope, and considering the different tailings strengths of the various tailings types, hummock heights and geometries were developed along with offsets to channels.

- Limit-equilibrium slope stability analysis of the hummock geometry: Typical cross sections for the different tailings types were developed to evaluate the geometry, and heights, slopes, and offsets to channels were modified to meet factor of safety criteria.

This approach was used to guide the closure landform design; however, it is expected that the approach will need to be regularly reviewed and updated, as annual site investigations are carried out documenting the as-built conditions of newly infilled areas. These investigations will provide the data to support or refine the design assumptions. Additional analyses (slope stability and deformation analyses) are also planned to support future levels of design. Instrumented test fills, loaded to failure to allow for back-analysis and calibration of the models, are also being considered.

4.7 *Settlement predictions*

The ETF is planned to contain some volume of soft tailings at the end of infilling and/or amendment activities that will undergo consolidation. At the SEA, while the majority of the soft and low-density tailings are planned to be removed, there will likely be trapped layers (AMEC 2013) that will undergo consolidation. Post-reclamation settlement of the materials within the ETF and SEA is a critical landform design parameter.

The key item for planning purposes is that once the final topography has been constructed and graded, monitoring of the ongoing consolidation and landform performance is required for many years. This monitoring will help to confirm the estimates of total consolidation settlement and the consolidation rate. Observations of the performance of the landscape will also help to understand the level of maintenance required, and the potential amount of additional fill that may be required to counteract the effects of consolidation, as well as the length of time required to manage the chemistry of the release water.

Due to limited MRM-specific tailings laboratory consolidation and hydraulic conductivity testing data, and because some of the tailings areas have yet to be deposited, estimates of settlement were extrapolated from current in-situ data and experience with similar tailings. They are intended to provide base case estimates, and were used to develop strategies to manage and potentially mitigate the post-reclamation settlements.

Using a one-dimensional spreadsheet approach, the total consolidation settlement of some of the tailings types that will remain in the ETF was estimated. The existing tailings solids contents obtained from annual sampling programs and published compressibility curves for various oil sands tailings were used in the analysis, as well as the weight of tailings layers above and the weight of hummocks constructed of mine waste material. While this approach provides an estimate of settlement, it is highly dependent on the compressibility curves used, and the amount and type of bitumen and other hydrocarbons present in the tailings (some contain high levels of asphaltenes). It also does not consider the time for total consolidation, which may occur over very long time periods.

Some of the settlement values predicted from this preliminary analysis were medium (less than 5 m) to large (greater than 5 m); laboratory large-strain consolidation testing is being undertaken by Canadian Natural to obtain compressibility and hydraulic conductivity curves specific to its tailings to refine these results and also evaluate the time for consolidation. Once settlement predictions are refined, they will be used to develop a management strategy focused on landform design and final reclamation.

Landform design for large settlement values is challenging, as many landforms will not perform as intended with more than 2 to 4 m of settlement (McKenna et al. 2016). There are options for managing post-reclamation settlement that may need to be considered, such as the use of wick drains to accelerate dewatering (e.g. Wells & Caldwell 2009), staged placement of mine waste over several years, strength and density amendment of certain areas (see CTMC 2012), and removal to an in-pit cell.

5 A LOOK BACK AT THE DESIGN PROCESS

The design process described above was carried out as a collaborative process between Canadian Natural and its consultants, over a period of several years, with the process expected to continue over many more years as annual site investigations are conducted and infilling progresses. With limited precedents for reclamation of large tailings facilities, much of the design and construction will require observation and adjustments to methods as the work progresses (Russell et al. 2010). Specific learnings from the current ETF and SEA closure landform design are listed below to provide guidance for future similar work that Canadian Natural and other oil sands operators will undertake for other ex-pit and in-pit tailings deposits.

5.1 *Set clear goals and objectives for the closure landform design*

The end goals for the ETF and SEA landforms are delicensing and reclamation certification; these two over-arching goals were declared at the outset of the design stage by Canadian Natural. Stating these goals up front in the design process provided the context for all design discussions and decisions, and helped to steer the design path toward these targets. Declaring the supporting objectives (listed above) guided the hundreds of design decisions. These goals and objectives, along with the criteria used to evaluate the design, formed the design basis, which established the overall framework for the design.

5.2 *Challenge design constraints*

The main waste placement topographic envelope design constraint for dyke stability provided a clear starting point for designing the surface drainage plan and topography for the ETF and SEA landforms. The envelope also allowed for mine planners to estimate a maximum volume of fill that could potentially be used, before the closure landform design process began.

During the initial design process, this constraint limited the ability to create a drainage plan for the ETF and SEA that would drain the entirety of both plateaus toward main channels, feeding the outlets. By having some drainage outward (the outward slope for the Perimeter Zone), armoured channels were initially required along the dyke crests and along the ETF slopes.

As mentioned earlier, given a desire to reduce costs and reduce the risk of downstream dyke slope erosion, the topographic envelope was revisited for key sections of the dyke crest. Raising the dykes in certain areas was shown to be possible, allowing for drainage from the dyke crests inwards to the main channels, creating a more robust overall drainage plan.

5.3 *Determine early in the design process when the outlets are required*

The ETF and SEA are currently designed and constructed with 3 m of operational freeboard. Most of the catchment area is occupied by the pond area (open water), with 100 to 150 m wide beaches around the perimeter. Runoff from the beach reports to the pond. During a precipitation event, the water level in the pond increases and the freeboard is temporarily reduced; pumps are used to return to 3 m of freeboard. The low-angle perimeter beaches provide protection against wave action and are an integral part of the freeboard design requirements. During operation, the pond occupies almost the entire watershed, so the rise in water level during extreme events is small.

Determining when to build the outlets is critically important, as these tailings storage facilities transition from management of storm events with simple pond storage behind ring dykes, to landforms with much reduced pond areas that pass major storm events through outlets. During infilling, the capacity of the dykes to contain the design storm event is reduced. Freeboard assessments are required to understand when the dyke containment is no longer sufficient to contain the design storm. Before this point in the infilling process is reached, the outlets need to be constructed. The outlets can only be constructed however when the environment is ready to receive the flows from the landform plateaus; temporary routing of the outlet channel may need to be considered.

5.4 *Designing with uncertainty*

The closure landform designer needs to be comfortable with the uncertainty that comes from the challenge of designing a landform on substrates that do not yet exist. The annual site investigation data collected by Canadian Natural during its tailings investigations programs provided significant data that can be used to estimate the final infilled tailings properties. These programs will continue as infilling progresses, to ensure that as-built information on the tailings stratigraphy, composition, strength, and consolidation behaviour is documented. This information will be used in the next level of design, to confirm and update the design parameters. Additional information from proposed instrumented test fills will be used to understand the potential tailings behaviour and expected deformations during construction.

The regulatory process to delicense a dam has not yet been formally applied to any structure, which means that the owner needs to work with the regulators through the process. Decommissioning requires potential dam failure modes be addressed to the satisfaction of the operator and regulator to reduce liability. Dams such as the ETF and SEA are operated according to Canadian Dam Association (2013) dam safety guidelines, which require regular inspection and maintenance. Thus, these licensed structures may require ongoing care and maintenance for a period of time.

There is also uncertainty in the long-term maintenance requirements of the surface water drainage system. There is little precedent for creating large channels that are stable for large flood events such as the probable maximum flood and that have adequate reliability in the absence of ongoing monitoring and maintenance. Many of these channels will be subject to incursion of vegetation, beaver activity, settling substrates, sediment loading from adjacent landforms, icing, and weathering of rock armour. The need for some level of ongoing maintenance is likely, to reduce the uncertainty in future performance. The objective is for the operator to demonstrate to the regulator that the residual risk for the structure has been reduced to a level compatible with that of the natural analogues (OSTDC 2014). The natural areas in the oil sands region, as in all other areas of the planet, undergo a geomorphic process of landscape evolution. As part of this natural process, slope instabilities, surface erosion, wave and river erosion, groundwater level fluctuation, fires, etc., occur in the region (OSTDC 2014). This would be expected to occur in a closed tailings facility, consistent with the scale and effects in the natural analogue.

6 LOOKING AHEAD

The SEA infilling is scheduled to be completed in the next few years, allowing for detailed landform design on this part of the landform to be completed. Learnings and experience from the detailed design, construction, and reclamation of the SEA will be applied to the ETF (and other tailings facilities).

As noted throughout this paper, for large tailings landforms, the design, site investigation, construction, and reclamation occur in parallel over decades, with learnings from each phase of the project being applied in real time, allowing designs to be adjusted and optimized. The case history presented above represents a snapshot in time (the end of construction: the point at which a planning-level landform design is developed). The design process will continue to evolve as each of the over 30 oil sands tailings facilities in the region is closed and reclaimed.

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A Practical Framework to Address Performance Monitoring Scaling Issues for Watershed Design to Support Oil Sands Reclamation Certification

Jason Song, Andrew Baisley & Mike O'Kane
O'Kane Consultants Inc. Calgary, AB, Canada

ABSTRACT: The oil sands industry spends millions of dollars every year to initiate, develop, and evaluate technologies, from tailings production to reclamation; all with the purpose of optimizing oil sands tailings management. Automated monitoring and manual measuring are usually utilized in the process of technology evaluation at the prototype and pilot scale. A common process to evaluate a technology is to gradually increase test scales. Compared to the watershed scale, at which tailings materials are usually reclaimed, other smaller scales, for example at the prototype and pilot scale, in comparison are "point scale. Design and monitoring becomes more challenging when evaluation is at the watershed scale, because of temporal and spatial variations of tailings properties and natural variables. One critical question that arises when "scaling–up": how do we use results obtained from the point scale monitoring to inform design and performance monitoring at the watershed scale to support oil sands reclamation certification?
In response to the issue, O'Kane Consultants have developed a practical framework, which is based on the differences communicated through the conceptual models describing performance at the pilot and watershed scale, as well as with key performance indicators (KPIs), and monitoring parameters usually applied at the point scale and watershed scale. Connections between KPIs and performance monitoring parameters were built up at the point scale and watershed scale through tailings characteristics and water balance components.
The framework indicates the importance of having a watershed scale view in mind when designing performance monitoring at the point scale and knowing outputs from tests at the point scale when designing a watershed scale oil sands reclamation.

1 INTRODUCTION

1.1 Background

Oil Sands operations are ultimately seeking reclamation certificates for mined portions of their leases, and may be a required to provide proof of reclamation prior to seeking lease extensions. The Alberta Energy Regulator (AER) is the issuing body of reclamation certificates for energy resource activities in accordance with Environment and Sustainable Resource Development (ESRD). Equivalent land capability in the Conservation and Reclamation Regulation is defined as "the ability of the land to support various land uses after conservation and reclamation is similar to the ability that existed prior to an activity being conducted on the land, but the individual land uses will not necessarily be identical" (Alberta Queen's Printer, 1993).

Additionally, operators are required to adhere to Directive 085 set by the AER (2016) which sets out requirements for managing fluid tailings volumes for oil sands mining projects. Under Directive 085, the Tailings Management Framework for the Mineable Athabasca Oil Sands

(TMF) provides policy direction to the AER to manage fluid tailings volumes during and after mine operation to manage and decrease liability and environmental risk resulting from the accumulation of fluid tailings on the landscape. All fluid tailings associated with a project need to be ready to reclaim (RTR) ten years after the end of mine life of that project (AER, 2016).

RTR is intended to track treated fluid tailings performance during the operational stage of the deposit, ensuring deposits can be reclaimed as predicted in the life-of-mine closure plan, in the time specified. To allow active treated tailings deposits to be removed from the fluid tailing inventory, assessments to determine whether tailings are on a predicted trajectory needs to be conducted. Each treated tailings deposit will have approved indicators that are measured to determine if the performance criteria have been achieved. The concept of RTR tailings supports the AER criteria of equivalent land capability of reclaiming oil sand mining projects to a self-sustaining boreal forest ecosystem that is:

- Integrated with the surrounding area (watershed approach); and
- Consistent with the values and objectives identified in local, sub-regional, and regional plans.

To evaluate whether active treated tailings deposits are on a trajectory to meet the high-level objective, two sub-objectives address different aspects of performance:

(1) The deposit's physical properties are on a trajectory to support future stages of activity.

(2) To minimize the effect the deposit has on the surrounding environment and ensure that it will not compromise the ability to reclaim to a locally common, diverse, and self-sustaining ecosystem.

Generally, these objectives can be reduced to 1) the deposits characteristics (physical, chemical and biological) and 2) the desired water balance of the deposits landform to support future reclamation efforts. Fluid tailings management plans must therefore include information to support the assessment of proposed performance criteria, which establishes when a deposit meets RTR status. AER requires the proposed performance criteria are supported by the proposed indicators for each deposit.

1.2 Statement of Problem

The practice of a technology development in oil sands tailings management indicates that performance indicators are changing with deposit scales. Therefore, there is a clear need to scale learnings from point scale measurements of various lab, bench and field pilot scale experiments to inform design and performance monitoring of oil sands reclamation at watershed scale. Scaling difficulties arise largely due to:

(1) Heterogeneity of materials;

(2) Boundary Conditions (hydrogeological and upper boundary); and

(3) Topography / geometry.

Without a clear framework that identifies how point scale data can be scaled to assess predicted performance of watershed scale reclamation, it is challenging to design final reclamation landforms. It is also not possible to ensure tailings deposits are on the trajectory of the tailings management plan and evaluate ultimate performance of the reclaimed landform without adequate performance measures.

This paper describes a practical framework connecting point scale measurements and watershed scale measurements. The connections are based on key performance indictors at these scales. The framework can serve as rationale for justification of performance indicators for a tailings deposit within the life of a project, as well as design tools for performance monitoring at various scales for developing a tailings management technology.

2 FRAMEWORK / METHODOLOGY

2.1 Conceptual Models

Developing conceptual models will be the first step in developing any performance monitoring design. Only once clear and concise reclamation objectives have been established will resulting design criteria be developed; which they will be evaluated against.

Conceptually, a tailings deposit at a point scale, such as a test cell or a test plot, features reduced surface footprint, flat or single slope, well configured layer(s), homogeneous material in a layer, and controlled boundary conditions (Figure 1). Conversely, a reclaimed watershed can feature hectare area footprints, multiple landforms or varied topography, various thicknesses of layer(s), heterogeneous materials layering, and various boundary conditions.

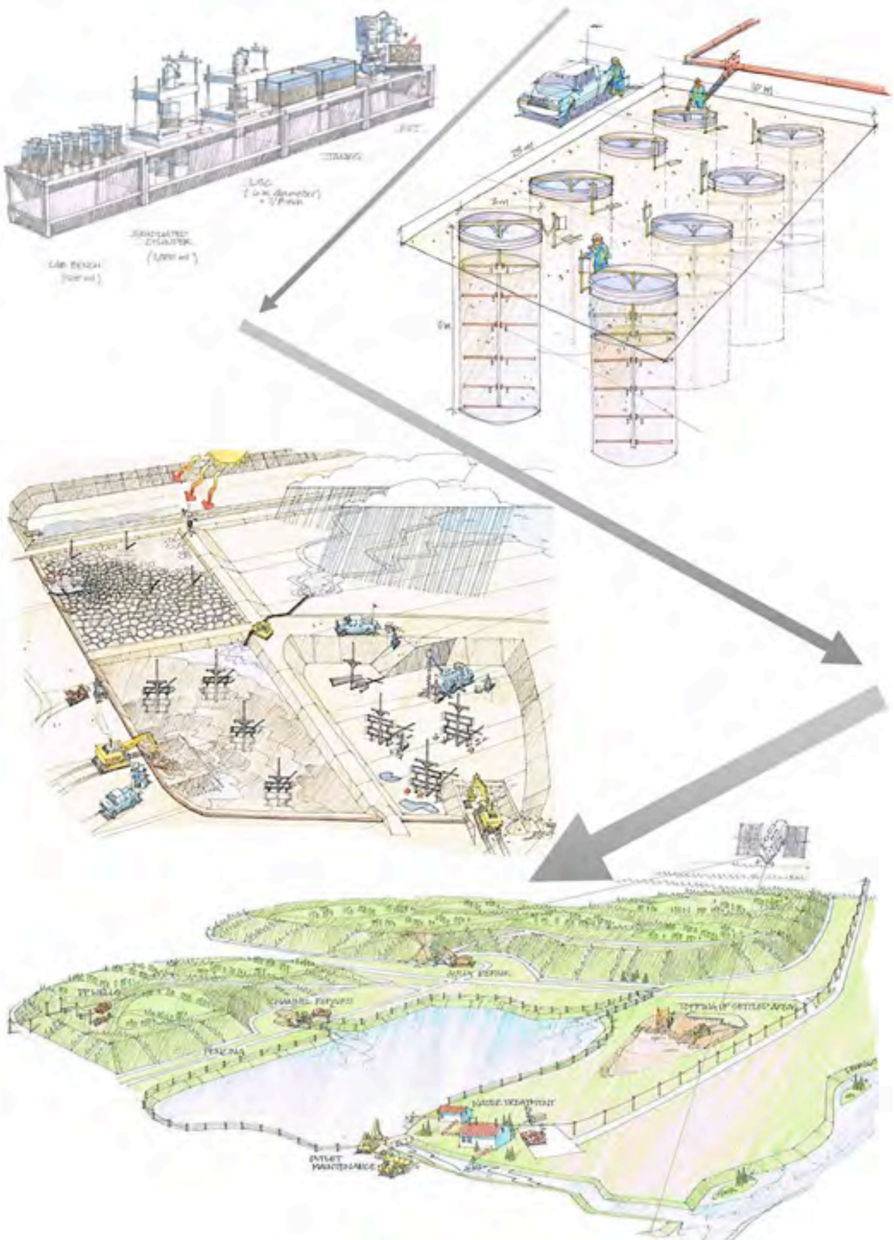

Figure 1. Integrating point scale deposit learnings into watershed scale deposits.

2.2 *Key Performance Indicators (KPIs) and monitoring parameters*

The *TMF* provides the flexibility for operators to develop performance criteria that are suitable to the type of tailings, technology, and deposit. The selection of performance criteria and how they are measured depends on:

- characteristics of the deposit (technology used, properties of the treated tailings),
- stage of the deposit in its development,
- deposit's targeted landforms and site-type, and
- position of the deposit in the landscape (e.g., below- or above-grade, proximity to sensitive areas or water bodies, etc.).

The AER expects that tailings deposits with higher uncertainty or more complexity, including with the surrounding environment, may have more indicators, measures, and performance criteria associated with them (AER, 2016). These treated tailings deposits will require more stringent performance criteria and rigorous monitoring.

At different tailings deposit scales, performance monitoring parameters and KPIs change to suit purposes of tailings deposition. Table 1 presents major KPIs for tailings deposits at the point scale and watershed scale.

Table 1. Key performance indicators for tailings deposits at point and watershed scales.

Item	**Tailings deposit in a test cell (point scale)**	**Tailings deposit in a reclaimed test plot (point scale)**	**Tailings deposit in a Reclaimed watershed (watershed scale)**
Performance Criteria	Strength > χ	Strength > χ Vegetation > χ	Geotechnical Stability > χ Vegetation > χ Water Quality / Quantity > χ
Specific Monitoring Parameter	Solids Content or Water Content Settlement Shear Strength Water Balance Soil Characteristics (physical)	Water Content or Solids Content Settlement Shear Strength Water Balance Pore-water Chemistry Soil Characteristics (physical and chemical) vegetation	Water Content Water Table Settlement Shear Strength Water Balance Pore-water Chemistry Soil Characteristics (physical, chemical, and biological) Vegetation
General Monitoring Parameter	Meteorological Parameters	Meteorological Parameters	Meteorological Parameters

χ indicates a threshold criteria value.

2.2.1 *Performance Monitoring of Tailings Deposits in a Test Cell*

The most important parameters to evaluate the performance of a tailings deposit in a test cell include solids content of the deposit, settlement, and shear strength. These parameters change over time, as well as over the profile of the deposit. Evaluating water balance is relatively easier in the test cell due to higher boundary condition control (e.g. an impermeable cell base). In most cases, solids content and settlement can be used to verify some of water content components. Physical properties of the deposit are the most important parameters for soil characterization, including sedimentation, consolidation, and drying (i.e. soil water characteristics curve).

2.2.2 *Performance Monitoring of Tailings Deposits in a Reclaimed Test Plot*

Comparing to the tailings deposit in a test cell, the tailings deposit in a reclaimed test plot has increased complexity of surrounding environment. Living vegetation at the deposit surface or overlying detritus material surface requires monitoring of water content and pore-water chemistry in the deposit in order to evaluate vegetation performance of the reclaimed test plot. Solids content, settlement, and shear strength are still important parameters that should be monitored in the deposit. Quantification of the water balance becomes more challenging due to the larger role evapotranspiration and/or runoff play on the water balance, and their larger uncertainty.

2.2.3 Performance Monitoring of Tailings Deposits in a Reclaimed Watershed

The Guide to the Landscape Design Checklist in the Athabasca Oil Sands Region (CEMA, 2008) requires a landscape designer to develop a monitoring and mitigation program for throughout construction, post construction and into reclamation; at which point the landform is considered stable and suitable for reclamation certification. The major monitoring items would therefore fall into the categories of:

- biological stability (i.e. sustaining vegetation);
- geotechnical stability (i.e. slope stability, differential deformation, erosion); and
- geochemical stability (e.g. water quality and quantity).

Accordingly, the KPIs should include water content, water table (e.g. phreatic surface), settlement, pore-water chemistry, water balance, and tailings deposit characterization. At this scale, water balance evaluation becomes complex because of increasing variations of water balance components. Physical, chemical and biological properties of the deposit are important to tailings deposit performance in a watershed scale.

2.3 Connections between KPIs at point scale and at watershed scale

To effectively apply knowledge learnt at a point scale to inform watershed design, it is necessary to build up connections between KPIs at these different scales. Derivation of water balance and soil characteristics shows potential to transfer knowledge from the point scale to the watershed scale as presented in the following sections.

2.3.1 Water balance (equation)

Tracking water balance in the tailings deposit from the deposition to final placement is a crucial aspect to evaluate tailings performance in the tailings management plan. A general water balance in the deposit can be expressed in Equation (1), regardless of the scale.

$$\Delta S = P + GW_{in} - AET - GW_{out} - RO \tag{1}$$

Where

ΔS = water storage change in the deposit;
P = precipitation;
GW_{in} = groundwater flow into the deposit;
AET = actual evapotranspiration from the deposit surface;
GW_{out} = groundwater flow out of the deposit; and
RO = surface water run off the deposit.

At the point scale, variations or differences of each water balance component are not substantial over the deposit, so it may be fair to say that all water balance components are at the same scale level. As a result, determination of the monitoring position is not difficult. However at the watershed scale, variations or differences of each water balance component may be significant over the deposit.

The water balance equation may be simplified at the point scale because of a controlled bottom boundary condition, such as a lined base (impermeable base) underlying the deposit of the test cell or test plot. In addition, each water balance component can be monitored so that the water balance equation can be calibrated. Comparing to evaluating water balance at the point scale, evaluating water balance at the watershed scale is much more complex. In most cases, the water balance equation is utilized to calculate one of the components. Table 2 compares water balance components at these two scales.

Table 2. Comparisons of water balance components at the point and watershed scales.

Component	Point Scale	Watershed Scale	Note
ΔS	• Measured-water content sensors • Measured-deposit settlement • Measured-deposit profile • Higher confidence	• Calculated-water balance equation • Measured-selected profiles • Lower confidence	Spatial variations
P	• Measured- gauges • High confidence	• Measured -gauges • Measured- gauge network • Measured- snow transects • Relatively high confidence	Spatial variations
GW_{in}	• Calculate- Consider zero if the test cell or plot is lined at bottom • Calculated by water balance equation • Calculated- hydraulic gradient and hydraulic conductivity • GW_{in} and GW_{out} non synchronous	• Measured- wells • Calculated- hydraulic gradient and hydraulic conductivity • GW_{in} and GW_{out} synchronous	Hydrogeological information
AET	• Measured-instrumentation • Calculated- meteorological parameters. • Calculated- energy balance	• Measured-eddy covariance / aerial estimations. • Calculated- meteorological parameters.	Spatial variations
GW_{out}	• Calculate- Consider zero if the test cell or plot is lined at bottom • Calculated by water balance equation • Calculated- hydraulic gradient and hydraulic conductivity • GW_{in} and GW_{out} non synchronous	• Measured - wells • Calculated- hydraulic gradient and hydraulic conductivity • GW_{in} and GW_{out} synchronous	Hydrogeological information
RO	• Measured- pumped volume • Measured- flow gauge, flout, tipping buckets	• Measured-stream gauging at outlet	

At the watershed scale, not all water balance components are measured at the same scale level. For example, runoff can be measured with respect to the entire watershed at its downstream outlet (i.e. at the watershed scale), while water content (used to calculate ΔS in the deposit) may only be monitored at the selected locations within the watershed (i.e. at the point scale). The accuracy of water balance evaluation for the watershed will be dependent on how measurements at these selected locations can represent the values of the entire watershed. Therefore, how to decide monitoring points (where and how many) would be a challenge for performance measurements at the watershed scale even if KPIs are known.

It is important to identify how variables are affected by scales of the deposit when using learnings obtained from the point. The variables may include landform types and changes of boundary conditions that caused by each landform within the watershed. This identification provides a screening tool to determine monitoring points required in the designed watershed.For example, when GW_{out} is marginal during point scale tests due to low hydraulic conductivity of the tailings and not by the underlying foundation material, monitoring points (or emphasis) should be focused on the tailings compared to the underlying foundation. In general, it is basic principles or fundamental observations obtained from the point scale monitoring that are transferable to the watershed scale monitoring and then these fundamental observations guide performance monitoring design and implementation at the watershed scale.

Specialized instrumentation can sometimes solve scaling issue on water balance components through the use of integrative measures. For example, AET can be measured over a large area at the watershed as can runoff at a catchment outlet. However, for most cases performance monitoring at the watershed scale has to be conducted through monitoring at a number of points.

2.3.2 Soil characteristics

The deposit's characteristics directly affect its performance at either the point scale or the watershed scale. At the point scale, soil characterization usually focuses on the deposit's physical properties, including particle size distribution, geotechnical index such as Atterberg limits, specific gravity, soil water characteristic curve, saturated and unsaturated hydraulic conductivity, consolidation. These tests are generally appropriate with performance measurements of the deposit at the point scale, in which solids content, settlement, and shear strength are the KPIs.

The performance monitoring results at the point scale inform the deposit's physical performance at the watershed scale when they are linked to the soil characterization results. Linking the point scale information to the watershed scale can be done many ways, but as an example:

1) Use performance monitoring results obtained from point scale to calibrate soil characterization results. Variations of boundary conditions applied to obtain these results should be identified to interpret the deposit performance at the point scale.
2) Compare variations or differences of boundary conditions of the deposit at the watershed scale and at the point scale, then apply learnings from 1) to inform watershed scale deposit performance.
3) Based on the informed performance in Step 2), performance monitoring at the watershed scale can be planned.

In addition to variations of boundary conditions, deposits have a greater likelihood of being heterogeneous than homogeneous at the watershed scale. Therefore, material characterization should seek to quantify the full range of properties caused by commercial placement. The performance results obtained from the point scale need to be adjusted to incorporate this variability when making inferences at the watershed scale. It may be more useful to group materials as coarser- and finer-textured groups at the point scale, characterizing each group separately. An approach such as this will aid numerical models in addressing material uncertainty and predicted performance expectations of landform units.

The deposit at the watershed scale will also have chemical and biological performance criteria associated with specific KPIs of pore-water chemistry or water quality. Chemical and biological characteristics of the deposit provide static results of water chemistry / water quality. Evaluation of water content and water balance versus time shows dynamic water volume changes, hence providing water chemistry of the deposit along time. Therefore, it is important to test chemical and biological properties of the deposit at the point scale even if pore-water chemistry is not a focus of the performance measurement at the point scale. Again, chemical and biological characterization of the deposit should be tested at coarser- and finer-textured groups.

3 PERFORMANCE MONITORING FOR INTEGRATED LANDSCAPES

After understanding how performance monitoring between scales can be linked, must now examine how performance monitoring will be conducted on final reclaimed landscapes. Tailings may be deposited in different landforms within a watershed. Key landform components of the oil sands region post closure include end pit lake (EPL), dedicated disposal area (DDA), tailings storage facility (TSF), wetlands, and uplands. Each landform features generic performance monitoring considerations (Table 3).

Table 3. Generic performance monitoring considerations for landforms.

Landform	**Performance monitoring considerations**	**Link to point scale monitoring**	**Difference**
TSF	• Material properties • Water balance • Tailings volume (settlement /SC) • Strength	• Material properties • Water balance • Tailings settlement / SC • Strength	Temporal and spatial variations of materials, boundary conditions changes,
DDA	• Solids content • Residual settlement • Strength • Pore-water chemistry	• Solids content • Material properties (consolidation) • Strength • Pore-water chemistry	Spatial variations
UPLAND	• Water content • Water balance • Residual settlement • Stability • Pore-water chemistry	• Water content • Water balance • Material properties (consolidation) • Strength • Pore-water chemistry	Spatial variations and boundary condition changes
WETLAND	• Water level • Water balance • Settlement • Water quality • Pore-water chemistry • Biodiversity	• Water level • Water balance • Settlement • Water quality • Pore-water chemistry	Boundary condition changes
EPL	• Water level • Water balance • Water quality • Sediment quality • Aquatic ecosystem	• Water level • Water balance • Water quality • Sediment quality	Boundary condition changes

*SC= solids content

Figure 2 shows instrumentation monitoring considerations for an upland with tailings underlying a cover system. Although most instrumentation focuses on the cover layers, instrumentation monitoring provides measurements to evaluate interactions between the cover system and underlying tailings.

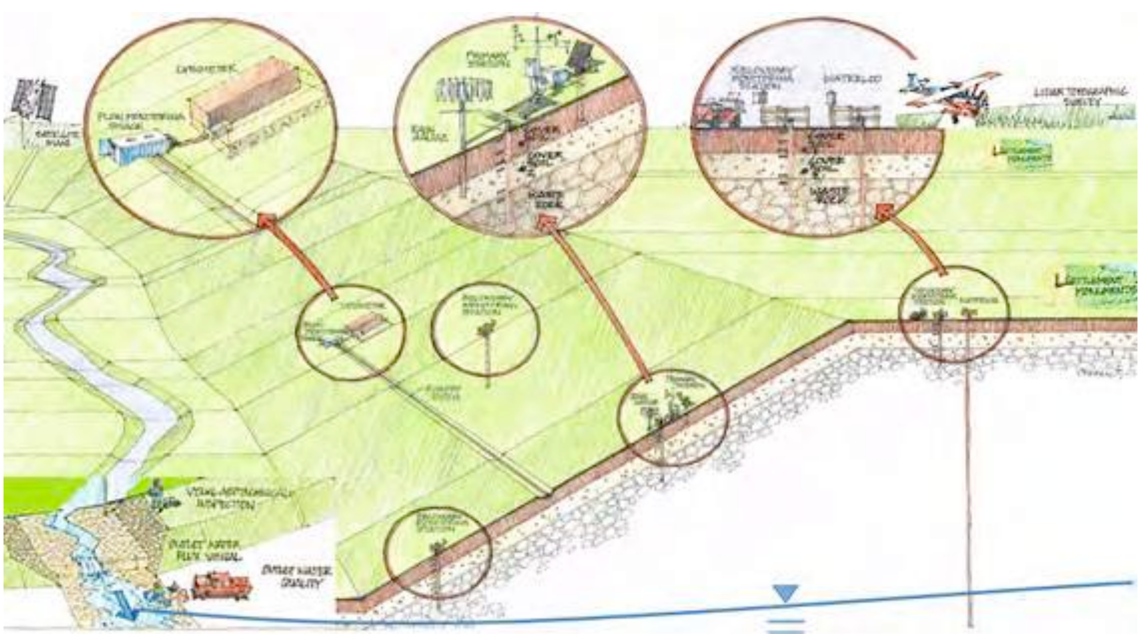

Figure 2. Instrumentation design for monitoring performance of an upland watershed.

The deployed instrumentation network in Figure 2 is explained as in Table 4 follows:

Table 4. Instrumentation consideration for performance monitoring of an upland.

Parameter	**Instrumentation**	**Considerations**
Meteorological	• Anemometer • Net radiometer • Air temperature sensor • Humidity sensor • Vapour pressure sensor	• Measurements at near surface and 2m height often required for flux measurements
Water Content	• Time domain / frequency domain reflectometry sensor • Neutron probe access tube • Diviner 2000 • Geophysics	• Slope variations • Profile variations
Thermal Regime	• Thermal conductivity sensor • Thermistors	• Slope variations • Profile variations
Precipitation	• Weighing gauge • Tipping bucket • Floating/ syphon gauge • Snow depth ruler • Snow pillow • Manual sample collection • Snowpack analyzer	• Spatial variability considerations • Regular maintenance and calibration required
Actual Evapotranspiration	• Evaporation pan • Bowen ratio energy balance apparatus • Eddy covariance system	• Fetch considerations • Night time measurement difficulties
Runoff	• Pump and metering • Weir	• Glaciation • Sedimentation.
Interflow	• Collection, pump and metering • Weir	• Sedimentation • Glaciation
Net Percolation	• Lysimeter • Piezometer (hydraulic gradient) • Guelph permeameter (hydraulic conductivity) • Infiltration ring (hydraulic conductivity)	• Slope variations
Storage Change	• Water content sensors	• Spatial variability
Phreatic Surface	• Saturated wedge pipe • Standpipe piezometer	• Spatial variability
Settlement	• Ruler/monument • Settlement plate • Settlement cell • LiDar • Satellite	• Spatial variability
Pore-water Chemistry	• Water sampling • Soil sampling • Electrical conductivity sensor • Geophysics	• Spatial variability

As stated before, there are differences of spatial and boundary conditions between monitoring an upland and at the point scale. Instrumentation is usually installed at top, mid and toe of the slope to capture spatial variations, while setting up monitoring of each boundary is to capture boundary differences for the upland.

4 CONCLUSIONS

Performance monitoring of reclamation landscapes has been mandated by many regulatory bodies for the oilsands lease holders to abide to. Currently, many sites are conducting point scale monitoring on tailings storage facilities to determine when they are RTR and to better understand long term performance of post closure landscapes. Monitoring has also been identified as a key component of adaptive management. However, with all of these requirements and the extensive amount of point scale quantification be untaken, a framework to provide linkages to the watershed scale has not been developed. As this paper discusses, reclamation objectives need to be clearly outlined so that design and performance criteria at the watershed scale can be developed. The development of landscape conceptual performance will help identify which performance measures will be needed during closure. In a sense working backwards, by understanding what monitoring will be required to adequately assess performance of the various reclamation landforms will help guide support studies of point scale measurements. On the other hand, designing and monitoring at point scale needs to provide transferable knowledge as much as possible to support performance monitoring of reclaimed landscapes. This framework can be used to understand which monitoring components are directly transferable between the point scale and watershed scale. Similarly, it will identify the measures which are integrative (i.e. runoff) and those which are not directly scalable. This paper outlines the key facets which lead some measures to not scale correctly (heterogeneity, spatial variability, and boundary condition differences) and how this can be addressed. Early conceptualization of the key factors influencing the reclamation KPIs will allow more appropriate monitoring of reclaimed landscapes in addition to better focusing point scale monitoring.

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Challenges in Cover Design, Reclamation and Revegetation of Terrestrial Tailings in the Athabasca Oil Sands Region

Lisa Kempenaar
Golder Associates Ltd., Saskatoon, SK, Canada

Bjorn Weeks & Audrey Wagenaar
Golder Associates Ltd., Vancouver, BC, Canada

ABSTRACT: In the Athabasca oil sands region, reclamation success is largely determined by the presence of functional, locally common boreal forest ecosystems with the ability to support similar end land uses to those that existed prior to disturbance. Tailings reclamation poses a particular challenge in achieving these objectives for several reasons. These include chemistry; physical stability of the cover system, which may be affected by plant rooting, animal burrowing or human activity; and long-term use of the reclaimed landscape. Regulations in the region dictate capping depths and materials, growth medium placement, and revegetation guidelines. Although tailings technologies continue to evolve, existing reclaimed landforms using proven tailings technologies exist in the oil sands region and provide a good source of information regarding terrestrial tailings reclamation. This paper explores the performance of existing reclaimed tailings landforms as well as discussing other important considerations for the long-term sustainability of reclaimed tailings landforms.

1 INTRODUCTION

This paper provides descriptions of the performance of existing reclaimed tailings landforms and the key considerations for reclamation success and sustainability.

Reclamation design can be prepared either by determining the end land use and designing the landform to meet that goal, or by designing the reclaimed ecosystem to match the final landform and attributing land uses to the resultant landscape. In the case of oil sands terrestrial tailings, the landform is usually pre-determined based on material properties, safety, and trafficability; therefore, ecosystems must be targeted to match the final landform. To successfully reclaim terrestrial tailings, several factors must be considered, including: potential end land uses, target ecosystem, topography, soil profile, rooting depth of dominant species, wildlife and human access, palatability of vegetation species, and cover integrity.

2 EXISTING RECLAIMED TAILINGS LANDFORMS

For the landforms described in this paper, oil sands were extracted and then processed using a warm to hot water extraction processes. The resulting tailings consisted of water, substrate particles, and residual bitumen mixed into a slurry. Tailings were pumped to tailings ponds where the coarsest tails settled first, closest to the deposition point. As velocities reduced, finer and finer particles settled out, until those that remained were suspended in water. Over time these suspended solids settled out to make Mature Fine Tailings (MFT). The MFT still contained a large fraction of water and so required further management to be reclaimed as a terrestrial landform (BGC Engineering Inc. 2010).

Data presented in this paper was collected at existing reclaimed tailings landforms. These landforms were constructed using older oil sands terrestrial technologies, with one of the landform construction processes described below.

2.1 Landform Construction

Environmental Protection and Enhancement Act (EPEA) approvals for oil sands operations require that treated tailings materials in terrestrial situations be capped with a minimum of 1.0 m of tailings sand or overburden prior to placement of 50 cm of reclamation material (AENV 2010). Terrestrial tailings landforms are generally designed using a geomorphic approach to mimic natural environments by creating gently undulating slopes in the range of 0.5 to 2%. Vegetated channels and/or protected channels are constructed to direct runoff toward closure wetlands or lakes.

This paper includes data collected from existing reclaimed tailings landforms. The data comes from landforms that consist of a combination of the following materials:

- **Consolidated tailings (CT)**: CT is a combination of MFT, coarse sand, and gypsum. CT may be deposited in layers several meters thick. This process creates a non-segregating deposit that is planned to be capped and reclaimed as a terrestrial landform (BGC Engineering Inc. 2010). The long-term behavior of CT is somewhat uncertain and may include settling and deformation (i.e., due to melting of ice lenses or frozen layers, cracking, or rehydration of tailings)
- **Regular tailings**: also called conventional or segregating tailings, are the material that forms when extraction tailings are deposited into in-pit beaching areas to form solid sand dump structures. The sand beaches trap a portion of the fine material extracted from the ore during bitumen separation (approximately 50% of fines).
- **Structural fill**: clean overburden or tailings sand that is considered suitable for construction of waste dumps, dams, roads, or reclamation features. The material is associated with relatively low long term environmental impacts, compacts easily and has minimal settling expected.

The landforms that were monitored were constructed with several meters of either CT or regular tails and all were capped with at least 1 m of structural fill and at least 50 cm of reclamation material.

More recent processes, not included in the landforms evaluated in this paper incorporate other treated tailings processes, such as those that employ polymers to assist in dewatering tailings. The resulting treated tailings are usually deposited into an exhausted mine pit or in a Dedicated Disposal Area (DDA) where it can be capped with structural fill, or where it may be capped with water.

3 ROOTING DEPTHS

One of the challenges in terrestrial tailings reclamation is choosing vegetation types that help protect the surface of the reclaimed structure and help create functional ecosystems. This vegetation should not compromise the tailings cover. The quality of the vegetation as well as waters found on the area should not compromise the health of wildlife that may use either the water or the vegetation on the reclaimed surface.

It is often considered important to choose plant species whose root systems do not penetrate into the tailings themselves as root systems may provide a conduit for movement of water from the surface into the tailings deposit and movement of process-affected water from the deposit to the surface. Table 1 shows the typical rooting habits of some of the common boreal species that could be planted over tailings deposits. Figure 1 shows the typical rooting habits of common boreal species.

Trees with root systems that are shallow and not widely spread run the risk of wind throw during high wind events. This may cause the tree to tip over, pulling up a large chunk of the root system and the associated underlying cover soil. A wind throw may lead to thinning of the tailings cover or exposure of the underlying tailings. Figure 2 demonstrates how this may occur.

Table 1. Typical Rooting Habits of Common Terrestrial Boreal Species

Common Name	Scientific Name	Vegetation Layer	Rooting Habit*	Reference
jack pine	*Pinus contorta*	tree	most roots in top 46 cm, tap roots up to 270 cm	BCMOF 2016
black spruce	*Picea mariana*	tree	most roots in the top 20 cm, to a maximum depth of 60 cm	BCMOF 2016
white spruce	*Picea glauca*	tree	generally shallow rooted to a maximum of 120 cm with tap roots up to 300 cm	BCMOF 2016
trembling aspen	*Populus tremuloides*	tree	roots spread extensively laterally and will put up suckers every few meters, sinker roots may descend to 300 cm	BCMOF 2016
paper birch	*Betula papyrifera*	tree	most roots in the top 60 cm, tap roots do not develop	BCMOF 2016
potentilla	*Potentilla fruticosa*	shrub	roots to a depth of 300 cm	Canadell et al. 1996
pasture sagewort	*Artemisia frigida*	forb	roots to a maximum depth of 170 cm	Canadell et al. 1996
common horsetail	*Equisetum arvense*	forb	roots to a maximum depth of 300 cm	Canadell et al. 1996
marsh horsetail	*Equisetum palustre*	forb	roots to a maximum depth of 250 cm	Canadell et al. 1996
Canada goldenrod	*Solidago canadensis*	forb	roots to a maximum depth of 340 cm	Canadell et al. 1996
red fescue	*Festuca rubra*	grass	most roots in the top 10 cm, descending to about 40 cm	Brown et al. 2010
June grass	*Koeleria cristata*	grass	shallow rooted to a depth of about 60 cm	CRI 1995
Kentucky blue grass	*Poa pratensis*	grass	deeply rooted to about 450 cm	CRI 1995

*assumes no growth restrictions and typical nutrients

If an average cover over a tailings deposit of 150 cm is assumed (based on EPEA approval conditions), trees such as jack pine, white spruce, and trembling aspen would penetrate into the tailings. Shrubs such as potentilla and grasses such as Kentucky blue grass and bluejoint would experience the same issue. Trees such as black spruce and paper birch would not be expected to penetrate into the tailings, but would be subject to wind throw if mature individuals are exposed to high winds.

Options for controlling rooting depth may include constructing a deeper structural fill cap or a root barrier layer. Root barriers prevent root penetration either by creating a physical barrier that the roots cannot move through, or by creating conditions that are not conducive to root growth. Root growth can be limited by providing insufficient fine particles, moisture holding capacity, or nutrients. Barriers can be created using rock fill, high-density polyethylene (HDPE) barrier sheets, or geogrids. Because of the scale of tailings deposits in the oil sands, root barriers may not be practical due to cost or availability of materials.

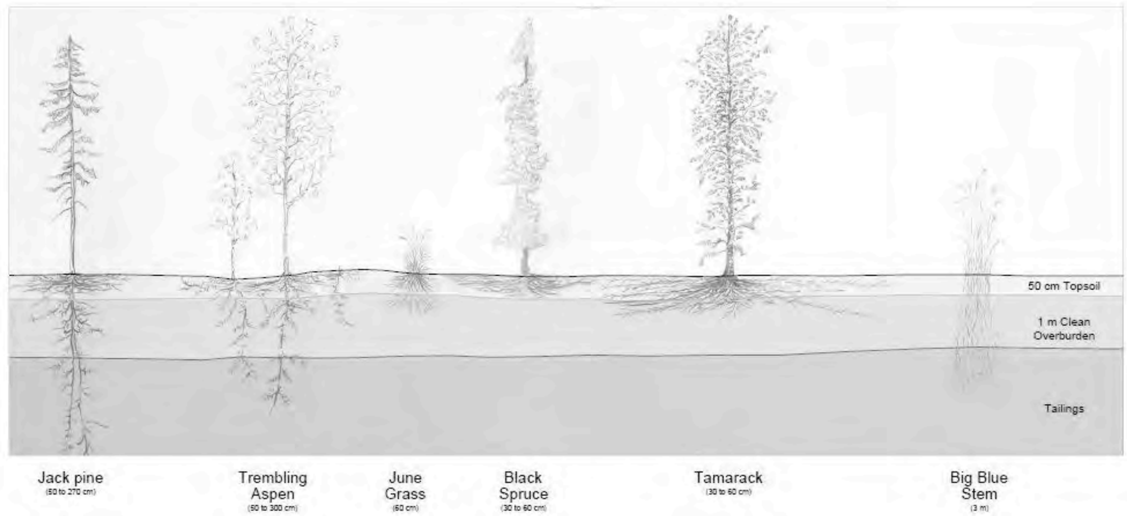

Figure 1. Rooting Habits of Typical Boreal Species.

Figure 2. Example of the Effects of Wind Throw on Tailings Cover.

If a deeper cap or a root barrier are not feasible, good vegetation choices for the landforms discussed in this paper may be shallow rooted shrubs, forbs, and grasses like red fescue and June grass. These species have the combined advantages of being unlikely to fully penetrate the cover, and not being subject to wind throw. If such species were used, the target ecosystem would be a shrubland / meadow complex.

4 VEGETATION UPTAKE OF SUBSTANCES

One of the key concerns in revegetating tailings landforms is the risk that tailings constituents could be incorporated into vegetation, with theoretical risks of impacts on the vegetation growth, or of bioaccumulation. Either as public perception issue or a real risk, this has been a concern in the reclamation of tailings at many types of mining sites, whether in the case of attempts to directly revegetate on tailings (as part of a phytostabilization project or for cost effective reclamation), or in the case of reclamation covers where roots may penetrate into the underlying tailings (see for example Bruce et al 2003; Neuman and Ford 2006).

As part of an environmental risk assessment, correlated samples of surface soil (top 50 cm) and vegetation (leaves, berries, roots) were collected in the summer of 2016 on reclaimed oil sands tailings landforms to help indicate if there was any uptake of tailings constituents into vegetation.

Maximum chemical concentrations in surface soil were compared to surface soil and vegetation screening values to select constituents that required further investigation. Screening values were established using various local and national regulations (AEP 2016; CCME 1999, 2008, 2010; US EPA 2016).

Vegetation (willow leaves, balsam poplar leaves, trembling aspen leaves, dandelion leaves, rose hips berries, kinnikinnick berries, ratroot roots and cattail roots) were collected as part of the environmental risk assessment. Concentrations in vegetation were compared against mineral tolerances for domestic animals (NRC 2005).

The constituents assessed were:

- Metals: iron, manganese and vanadium.
- Organics: benzo(a)anthracene, benzo(b+j+k) fluoranthene/benzo(b&j) fluoranthene, C1 benzo(a)anthracenes and C1 chrysenes, C1 benzofluoranthenes and C1 benzo(a)pyrenes, C2 benzo(a)anthracenes and C2 chrysenes, C2 benzofluoranthenes and C2 benzo(a)pyrenes, and benzo(a)pyrene total potency equivalents (alkylated PAHs).
- Carcinogenic PAHs (including alkylated PAHs).

Concentration ratios were calculated for correlated soil and vegetation samples for each reclaimed tailings landform. Results of these analysis are shown in Table 2. Concentration ratios of 5 or higher were chosen to highlight those constituents that may require further investigation. Those concentration ratios that are below 5 are not likely to be of concern."

Table 2. Concentration of Constituents of Interest in Vegetation Planted on Reclaimed Terrestrial Tailings Landforms in the Oil Sands region

Vegetation Type	Landform-substrate	Species Analyzed	Average CR (kg soil/ kg tissue wet)	Average CR 5 to 10	Average CR >10
Grass	Structural Fill – Consolidated Tailings	wheatgrass, tufted hair grass, wild rye, smooth brome, bluejoint	0.0079 to 5.00	-	-
	Structural Fill – Regular Tailings	fescue, smooth brome, tufted hair grass	0.0089 to 16.8	-	K
Leaf	Structural Fill – Consolidated Tailings	willow, balsam poplar	0.0067 to 5.00	-	Zn
	Structural Fill – Regular Tailings	trembling aspen, balsam poplar, willow, dandelion,	0.01 to 15.9	Cd, Ca, Mg, Zn	K
Berry	Structural Fill – Consolidated Tailings	rose hips (only one sample collected at consolidated tailings)	0.00078 to 5.00	-	-
	Structural Fill – Regular Tailings	rose hips	0.0042 to 22.2	-	K

CR = Concentration Ratio, K = Potassium, Ca = Calcium, Zn = Zinc, Mg = Magnesium, Cd = Cadmium

Occurrences of organics and carcinogenic PAHs were either not detectable or were below a concentration ratio of 5. Metals such as cadmium and zinc were found in concentration ratios greater than 5 in leaves found on regular tailings. Zinc was found at a concentration ratio greater than 10 in leaves on consolidated tailings.

Preliminary screening against MTL (maximum tolerable levels) for domestic animals (NRC 2005) suggest that all total concentrations as mg/kg for constituents that have defined levels (including zinc) showed concentrations in the vegetation samples well below MTLs for animal feed. MTL is "the dietary level that, when fed for a defined period of time, will not impair animal health and performance (NRC 2005)"

Table 3. Concentration of Constituents of Interest in Vegetation Compared Against Available Maximum Tolerable Levels (NRC 2005)

Constituent	Concentration Range in Sampled Vegetation (mg/kg)	MTL (mg/kg)
Cadmium	<0.0050 to 0.18	1
Calcium	1,040 to 5,610	10,000
Magnesium	271 to 2,300	5,000
Molybdenum	0.10 to 2.01	7
Potassium	2,230 to 6,690	30,000
Zinc	4.96 to 86.4	500

Many detectable constituents were nutrients (e.g., K, Ca, P, Mg, Zn) that are necessary for plant growth and would be expected to be actively taken up by healthy plants regardless of the substrate. Measured concentrations of many of these constituents (K, P, Ca) in the closure landform substrates are similar to concentrations measured from other background locations in the oil sands region (e.g., Athabasca Oil Corporation [2013]; Total E&P Joslyn Ltd. [2010]).

Several of the species analyzed have typical rooting depths greater than the mandatory 1.5 m cap, suggesting that root systems have likely penetrated into the underlying tailings deposits. Further investigation is required to determine if rooting depth is influencing uptake of constituents, the level of uptake and the potential risks for humans and wildlife associated with those uptakes.

In Figure 3, constituents that were present in each landform type were compared based on their uptake into grass samples. These trends show that the concentration ratio between soil and grass was highest on regular tailings. The data shows little correlation between soil concentration and uptake in vegetation for most constituents that were assessed. There appears to be some correlation in potassium and zinc, but a larger sample size would be required to confirm that correlation. All measured concentrations shown for grass samples are well below the MTLs for domestic animal feed (NRC 2005).

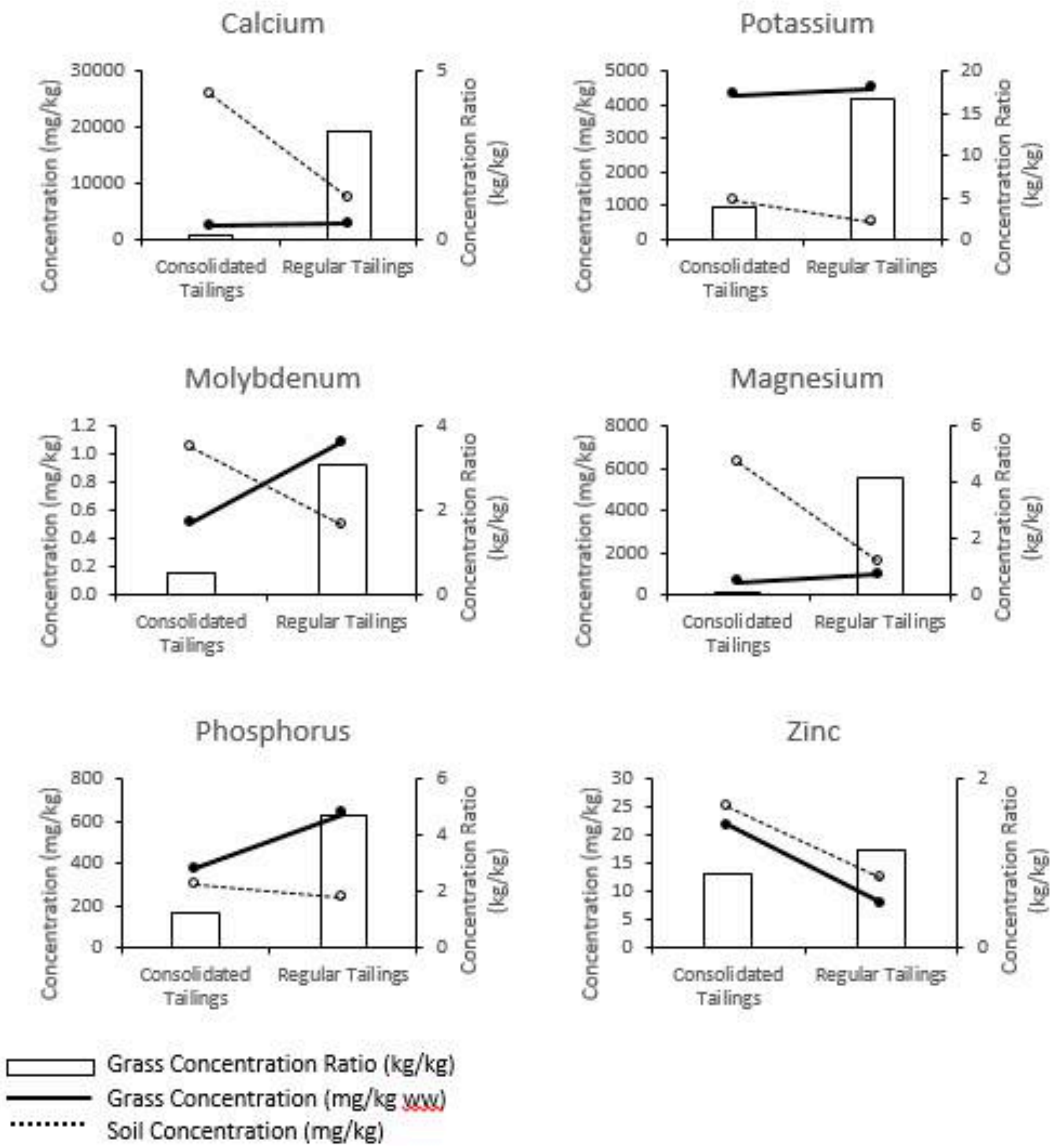

Figure 3. Comparison of the Uptake of Select Constituents in Grass

5 END LAND USE

Part of the design considerations for a reclaimed tailings landform are end land use targets. End land use targets for an oil sands mines typically include wildlife habitat, traditional uses (e.g., hunting, trapping, medicinal plant gathering, food plant gathering), recreation, and forestry, where applicable. Tailings landforms will likely support a variety of end land uses over time as the ecosystem evolves. Consideration must be given to which ecosystems can be expected to thrive on the reclaimed tailings landform, and which land uses can be practiced without compromising the integrity of the reclaimed tailings landform.

The design should also incorporate mitigations to protect the landform from the impacts of end land uses. These may include access prevention, limiting of palatable species, increased cover depth in target areas, and use of rocks, woody debris and other materials to protect sensitive areas.

5.1 Access and Wildlife Usage

Consideration must be given to whether humans will be likely to, and should be allowed to, access the reclaimed landscape. Low-impact recreational activities such as hiking, bird watching, camping, hunting, trapping, or berry picking are not likely to cause significant impacts. If these are the anticipated or planned end land uses, the design will probably not require special modifications, nor will access limitations to prevent these uses be required.

High-impact recreational activities such as use of all-terrain vehicles does have the potential to cause damage to vegetation and cover. Where this kind of use is anticipated and the cover performance could be vulnerable to its impacts, vehicle access can be prevented through landform design, fencing and signage, or through the placement of natural barriers (e.g., boulders, rock piles, woody debris piles).

One must also consider attractiveness, or the palatability, of planted species to wildlife. Will the new vegetation attract wildlife to the tailings deposit? Could those wildlife potentially damage to the tailings cover and risk exposing the underlying tailings deposit? Burrowing small mammals and digging bears can damage the tailings cover.

Preventing wildlife access is neither practical, nor in line with the development of functional ecosystems, a goal of the reclamation process. However, a balanced approach can be taken to mitigate potential impacts and shape the use of the landscape by wildlife, creating a more resilient system in closure. Key considerations for this may include:

- avoiding highly palatable species such as legumes and large patches of berries
- consider increasing capping depths in sensitive areas or in areas that may be particularly attractive to wildlife
- place rock piles, woody debris piles, or standing woody debris in sensitive areas

6 IMPLICATIONS FOR RECLAMATION PLANNING

EPEA Approvals for oil sands operations require an end land use target of self-sustaining, locally common, boreal forest ecosystems that are integrated with the surrounding area and support a variety of end land uses (AENV 2010). In the case of terrestrial tailings landforms, to accommodate the special considerations discussed in the paper, reclamation planning may require some design criteria that are not typical for other reclaimed landforms in the oil sands region.

This may include capping tailings with a deep cover of fill (i.e., $\geq$300 cm) so that a typical forested ecosystem can be targeted. Availability of clean structural fill material in proximity to the tailings deposit will be a key factor in determining the feasibility of deeper capping. Consideration must also be given to the effect the additional weight of the cap may have on the consolidation of the underlying tailings deposit. Excessive or uneven consolidation of the tailings may lead to cracking, erosion, or unplanned water movement on the reclaimed landform.

If deeper capping is not feasible, a modified vegetation community may be an appropriate mitigation. A shrubland / meadow ecosystem that focuses on shallowly rooted species may provide appropriate erosion protection and ecosystem development without the risk associated with either root penetration of the tailings or windthrow potential. A more open shrubland / meadow ecosystem may provide grazing and browsing habitat for wildlife as well as hunting areas for birds and small carnivores.

A combination of deeper capping and shallowly rooted species may provide the most practical solution by reducing the amount of material required while also providing a complex of variable vegetation communities and ecosystems. Landform and ecosystem design may include a variable landform with a more irregular undulating surface than is typically found on tailings deposits. Mounds of deeper fill that allow for deep rooted tree species to establish can be built up, transitioning into lower elevations with shallower caps that can support shallow rooted tree species that will be protected from the wind by the adjacent mounds. The lower elevation areas would act as channels to help direct precipitation and runoff toward wetlands, watercourse and waterbodies.

7 CONCLUSIONS

As regulatory focus continues to move toward reclaiming tailings in terrestrial landforms, and tailings technologies continue to evolve, the importance of understanding how to reclaim terrestrial tailings landforms will only increase. Considerations should be given to long term sustainability of landforms, covers, vegetation, and ecosystems. Tailings landforms should be designed to help meet the regulatory requirements of establishing functioning, locally common boreal forest ecosystems. Landform design and reclamation techniques that are not typical to the oil sands may need to be considered to effectively reclaim functional landscapes on tailings deposits.

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Operational water quality-based strategies for successful oil sands mine closure

Skya Fawcett & J.P. Bechtold
Golder, Calgary, AB, Canada

Jerry Vandenberg
Golder, Kelowna, BC, Canada

ABSTRACT: Water associated with drainage from oil sands closure landforms can include salinity, organics, and metals, which can pose challenges to operational water management as well as closure drainage. Accurately predicting the concentrations of these parameters in closure drainages depends on whether the data gathered during mine operations are representative of conditions at closure. Understanding and anticipating deviations from predicted closure water quality early in mine life and during operations is key to avoiding costly long-term mitigation at closure. To that end, this paper outlines strategies that can be implemented during operations to detect deviations in model assumptions, with the goal of reducing liability at closure. Specifically, past experience in water quality modelling of closure landforms, including pit lakes and covered tailings, has shown that predictions are sensitive to: 1. Assumed degradation rates; 2. Process water chemistry; 3. Flow paths through cover materials and underlying mine waste and its resulting effect on exiting water quality; and 4. Potential evolution of water chemistry originating from mine waste over time. For each of these inputs and assumptions, strategies will be described that can be used to confirm the validity of the model during operations, increasing the likelihood that water quality from closure landforms can be managed in a way that will meet closure targets.

1 INTRODUCTION

Oil sands operators may face water quality challenges at closure, such as elevated concentrations of organics (and their resulting toxicity), salinity, and metals in residual process water, tailings pore waters and, potentially, runoff from mine-affected areas. Water quality at closure is forecast by interpretation of geochemical test work and use of modelling tools. Conceptual and predictive water quality models in particular are a key component of operational and closure planning. The more accurate the assumptions used regarding model inputs and material reactivity, the more likely the water quality predictions will represent the conditions realized at closure. Accurate forecasting is important, because it allows operators to identify potential risks to closure success early in mine life, which allows the greatest range of mitigations to be incorporated into operations, and can result in a reduction in the environmental risk and liability of a project, as well as the costs associated with long-term management. In general, more effective and less costly mitigation options are available earlier in operations compared to closure.

Based on past experience in water quality modelling of oil sands closure landforms, inputs with the greatest influence on water quality predictions consist of: organic degradation rates, process water chemistry, chemistry of water flowing through cover materials and the underlying mine waste material, and weathering of waste materials from mine facilities. The purpose of this paper is to illustrate the influence of these inputs and assumptions on closure outcomes and provide recommendations on how to identify deviations in forecasted water chemistry early in operations.

2 EVALUATING WATER QUALITY MODEL INPUTS AND ASSUMPTIONS

For the purpose of illustrating the influence of the four aforementioned inputs, a water quality model was developed for a hypothetical pit lake that has characteristics that are typical of an oil sands pit lake. The model was developed using the Golder Pit Lake Model, described in detail in Teck (2015). The conceptual pit lake model is illustrated in Figure 1, and characteristics are provided in Table 1.

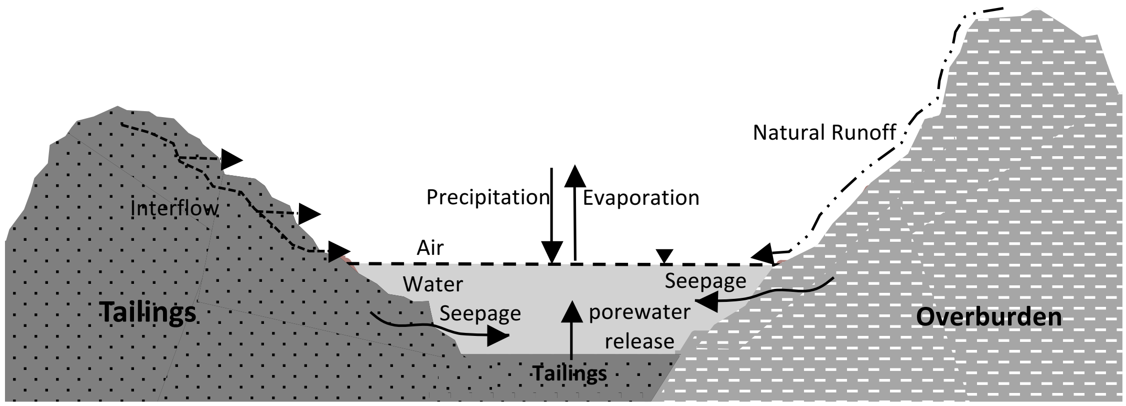

Figure 1. Pit Lake Conceptual Model

Table 1. Pit Lake Model Dimensions and Flows

Characteristic	Units	Base Case
Pit Lake Volume	Mm^3	300
Initial Tailings Volume	Mm^3	200
Surface Area	Km^2	10
Residual Process Water Inventory Placed into the Pit Lake	Mm^3	20
Runoff from Undisturbed / Natural Areas	Mm^3/year	5.7
Seepage from Surrounding Tailings Deposits	Mm^3/year	0.8
Surface Flow / Interflow from Surrounding, Reclaimed Tailings Deposits	Mm^3/year	1.8
Seepage from Surrounding Overburden Dumps	Mm^3/year	1.9
Evaporation	Mm^3/year	1.2
Pumping Rate from Athabasca River[1]	Mm^3/year	5
Filling Period	Years	4
Average Residence Time	Years	18
Stratification	NA	None

Notes

NA = Not Applicable

[1]Athabasca River water will be pumped to the pit lake only during filling

Variation to the values assigned to the inputs shown in Table 1 will affect closure water quality; the values shown were selected to be broadly representative of an oil sands pit lake for the purposes of providing a base upon which to illustrate the sensitivity of forecasting models to chemistry inputs. Model inputs for the sensitivity scenarios are shown in Table 2. The influence of the four aforementioned assumptions on pit lake water quality predictions are described in the following sub-sections.

Results are presented for total dissolved solids (TDS), refractory naphthenic acids (RNAs), and cadmium. This paper focuses on these three parameters, because they:

- Have been identified as constituents of potential concern in drainage from closure landforms in the oil sands region;
- Demonstrate different levels of sensitivity to model assumptions and inputs; and
- Represent a range of constituent types, namely:
  - organics that undergo decay and contribute to toxicity (e.g. RNA); and
  - inorganics that do not decay (e.g. TDS and cadmium).

Labile naphthenic acids (LNA) predictions are not included because they have been shown to degrade quickly and therefore are not as sensitive to model input concentrations as RNAs over the long term.

Table 2: Model inputs used for the sensitivity scenarios

			RNA Degradation Rate	RNA (mg/L)	Total Dissolved Solids (mg/L)	Cadmium[1] (mg/L)
Base Case	Process Water Chemistry (PWC)[1]		0.03	40	2200	0.0007
	Interflow from Tailings Deposits		0.03	40	2200	0.0007
	Surface Flow from Reclaimed Tailings Deposits, and Natural Undisturbed Areas		0.03	0.6	250	0.00007
	Overburden Seepage		0.03	0.6	250	0.00007
	Athabasca River Water		0.03	0.4	209	0.00003
Sensitivity Scenarios	Degradation Rate		0.003	40	-	-
			0.015	40	-	-
			0.045	40	-	-
			0.3	40	-	-
	Process Water Chemistry	Average	0.03	34	1400	0.00081
		Salty	0.03	69	3200	0.002
	Interflow from Tailings Deposits	50% PAW (Base Case PWC)	0.3	20	1225	0.0005
		100% PAW (Base Case PWC)	0.3	40	2200	0.0009
	Weathering of Mine Waste Materials	Tailings[1]	0.3	-	6000	0.07
		Overburden	0.3	-	3450	-

Notes

[1]The process water chemistry value provided for cadmium is for seepage and interflow from the tailings landforms; cadmium in process water inventory and flux from consolidating tailings is 0.024 mg/L in all scenarios

RNA= Refractory Naphthenic Acid, PAW = Process Affected Water

"-" = not included in the sensitivity model run

2.1 *Degradation Rates*

Most organic compounds degrade naturally over time. The rate of degradation is compound specific and dependent on variables such as temperature and oxygen levels. Degradation rates influence not only the concentration of organics in closure landform drainage, but also the level of toxicity in water. The sensitivity of closure water quality to changes in degradation rates is illustrated here using RNA predictions. The assumptions regarding partitioning and decay of naphthenic acids are consistent those recommended in the CEMA End Pit Lake Guidance Document (CEMA 2012). Specifically, the document outlines the following assumptions:

- For all process-affected inflow sources, such as direct inputs of process-affected water or seepage inflows from tailings areas, 25% of total naphthenic acids are assumed to be labile, with a half-life of 0.22 years (degradation constant (k) = 3.2 $year^{-1}$).
- For all process-affected inflow sources, 75% of total naphthenic acids are assumed to be refractory, with a half-life of 23 years (k = 0.03 $year^{-1}$).
- For all natural sources, 100% of total naphthenic acids are assumed to be refractory, with a half-life of 23 years (k = 0.03 $year^{-1}$).

The degradation rates (k) were varied from +/- 50% to +/- an order of magnitude compared to the Base Case (Table 2).

As shown in Figure 2, the effect of changing degradation rates is moderated to a large extent during the filling period by the dilution provided by the waters used to fill the lake, which consist largely of Athabasca River water. Once full, the addition of water from the Athabasca River ceases, and the influence of varying degradation rates becomes more apparent as RNA concentrations continue to decline with time, leveling off in some cases as the system reaches equilibrium. These results illustrate the influence of the approach used to initially fill the pit lake and how more rapid degradation will result in lower RNA concentrations, which could, in turn, reduce water quality issues at closure. Hence, pit lake planning should include considerations of how to encourage aerobic decay of naphthenic acids.

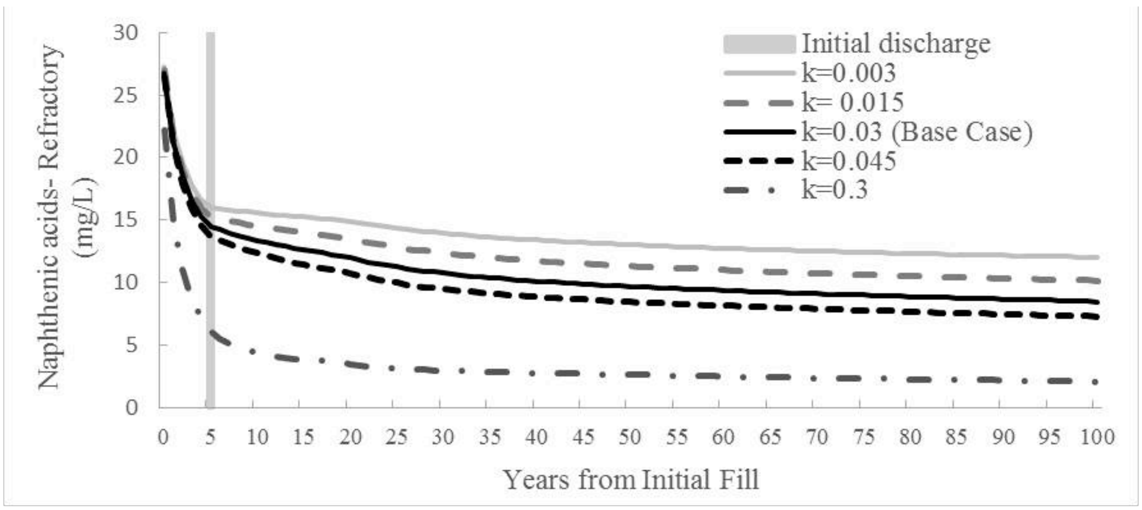

Figure 2: Degradation rates sensitivity results, k = degradation rate

2.2 *Process Water Chemistry*

Forecasting closure conditions requires assumptions to be made concerning the quality of: the process water remaining on site at closure (process water inventory), seepage from tailings landforms, and release water from tailings as they consolidate. The chemistry of process water is dependent on factors including ore quality, processing technology, use of processing additives, management of saline groundwater, use of closed-circuiting, rate of recycle, age of operation and climate. With the current use of closed-circuiting and water recycling, process water chemistry changes over time; it also changes over time in response to changes in the characteristics of the underlying ore body being mined. Consequently, what is measured today at one site may not be reflective of what conditions will be like at another site in the future.

The chemistry of water seeping from tailings landforms or being released through consolidation will reflect site process water chemistry at the time the tailings were placed. For this reason, it is important to consider when the tailings will be placed when deriving model inputs for tailings landforms that drain to a given area or when defining the quality of the water being released from consolidation.

To illustrate the sensitivity of closure forecasting relative to process water chemistry, three scenarios were run. The RNA, TDS and cadmium concentrations considered across the three scenarios (see Table 2) are reflective of the range of conditions that exist in the Athabasca Oil Sands Region. As previously noted, there a number of process and management considerations that will result in a given process water chemistry. The scenarios used in this evaluation were derived to be reflective of the following:

- Scenario 1 (Average PWC): process waters are maintained in a closed-circuit, recycle rate is moderate (i.e., operation is of moderate age and maintains a relatively large freshwater make-up), Basal depressurization water is discharged into the process water loop, Basal water is of moderate salinity, and release of major ions (salts), metals, and naphthenic acids from the ore being mined is moderate.
- Scenario 2 (Base Case): similar to Scenario 1, but water undergoes extensive recycling over a long mine life (e.g. >30 years).
- Scenario 3 (Salty PWC): Similar to Scenario 2, but Basal depressurization water reporting to the closed-circuit has elevated concentrations of TDS and recycled water has elevated cadmium.

As shown in Figure 3, TDS, RNA, and cadmium pit lake water quality predictions are sensitive to process water chemistry inputs, and the model input affects whether or not predicted pit lake water quality will be lower than chronic effect benchmarks defined for TDS and RNA, as outlined in Teck (2015) (i.e., TDS = 1000 mg/L, RNA = 7.1 mg/L [interim]) or water quality guidelines for cadmium (i.e., 0.00037 mg/L at a hardness greater than 280 mg/L, CCME 1999) in the future.

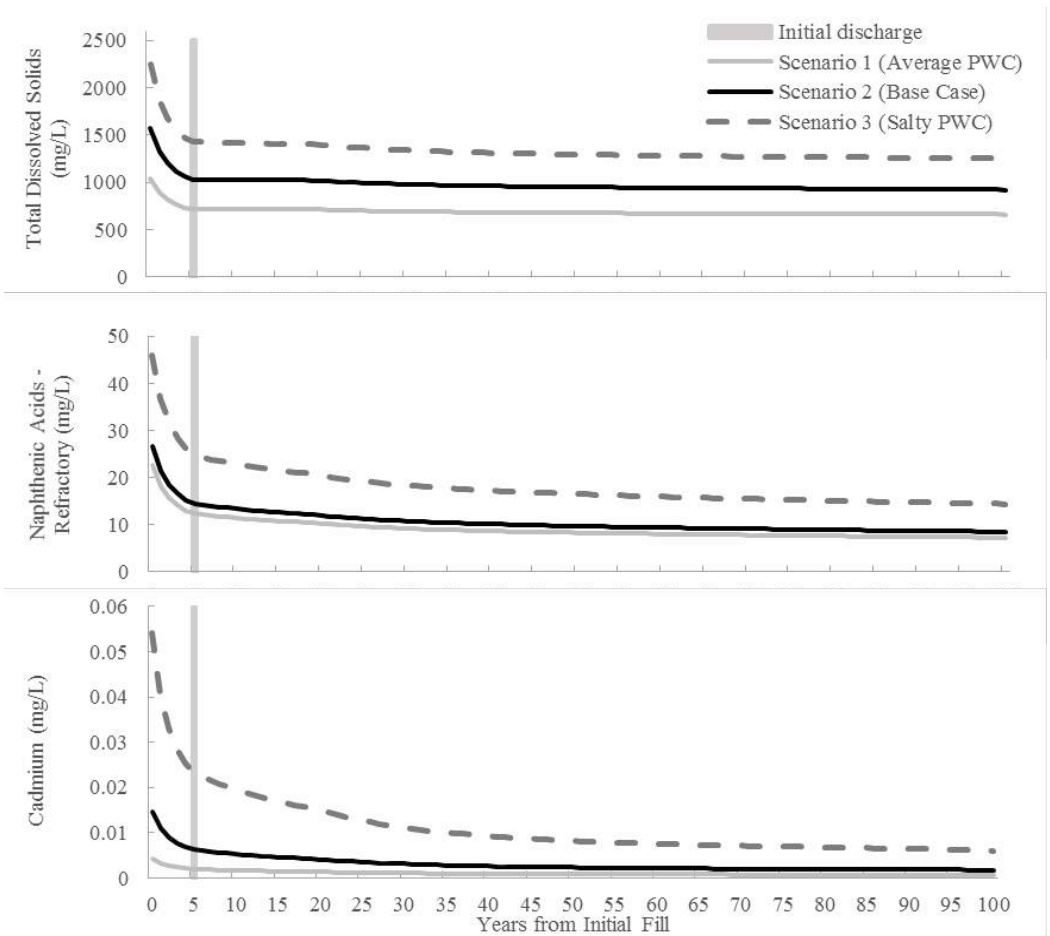

Figure 3: Process water chemistry sensitivity results. PWC = Process Water Chemistry

2.3 *Interflow*

Interflow is a physical runoff component; it refers to water that infiltrates into the subsurface and flows laterally downslope to enter the closest receiving surface waterbody without passage through a principal groundwater body. The water stays relatively close to the surface, the exact depth being dependent on the subsurface composition and other conditions such as slope. In a hydrological modelling context, interflow is the runoff component that arrives in the stream slower than the surface runoff, but faster than groundwater discharge.

With respect to closure forecasting, an assumption must be made as to whether interflow will be confined to the cleaner reclamation layer, or, if not, how far into the underlying waste material it will flow and what effect this will have on the quality of the interflow when it discharges to a pit lake or other receiving waterbody. To illustrate the influence of interflow chemistry on closure water quality in a pit lake, three scenarios were run where the interflow chemistry is (Table 2): 1. Similar to natural background, 2. Similar to Base Case PWC, and 3. Half of the flow is similar to natural background and half is similar to Base Case PWC.

As shown in Figure 4, pit lake water quality model predictions are sensitive to interflow chemistry. TDS and RNA predictions are all notably lower in the 50% and 100% natural background interflow scenarios compared to the Base Case. Accordingly, pit lake planning should be considered a basin-wide endeavor, as subtle changes to the reclaimed landscape, such as thickness of reclamation material, could affect the water quality reporting to the pit lake. Cadmium shows less sensitivity in the first 100 years after filling because the dominant source of cadmium is porewater released from tailings through consolidation.

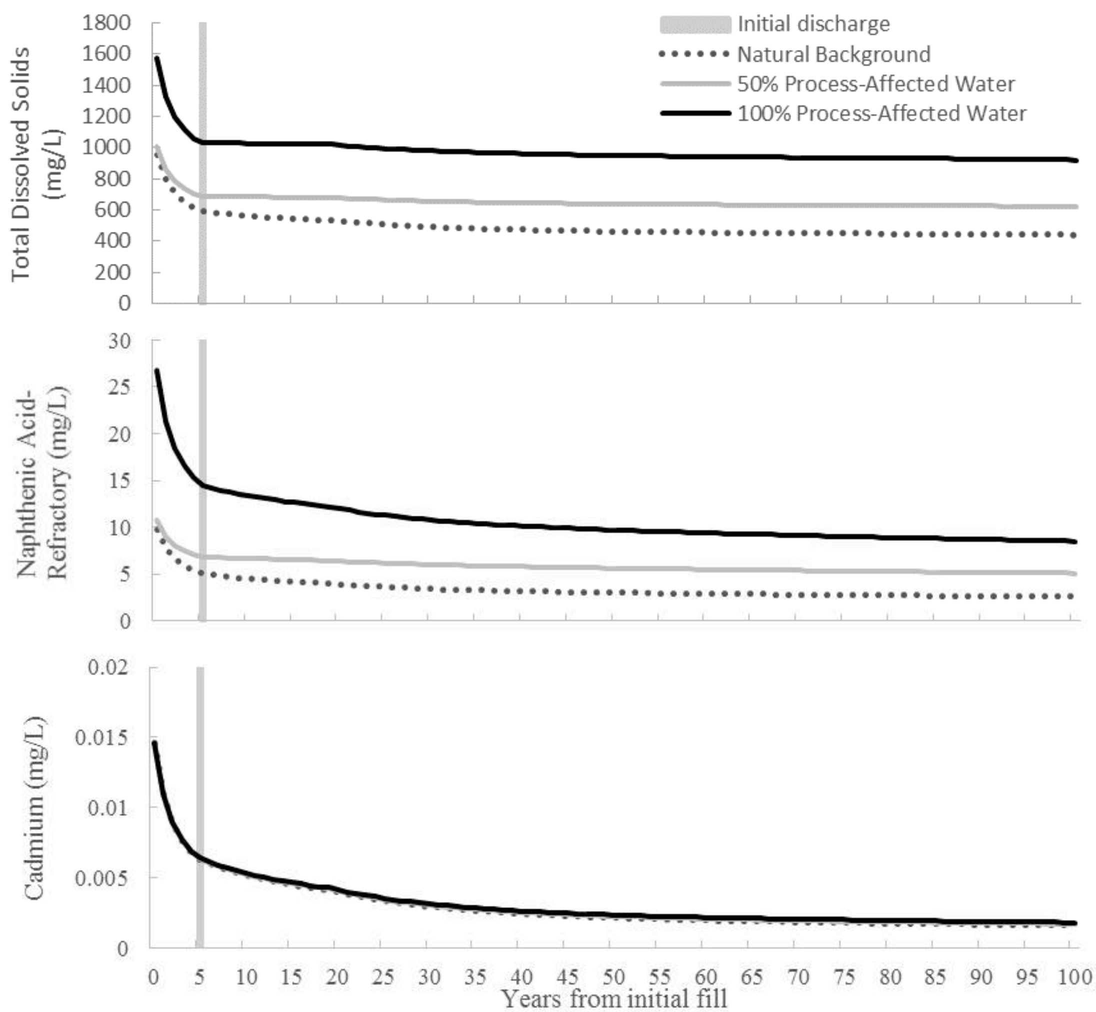

Figure 4: Interflow water chemistry sensitivity results

2.4 *Weathering*

Site-specific knowledge of material characteristics improves the robustness of future water quality predictions. The process of removing soils during project development enhances natural rates of weathering of minerals, notably in materials with elevated sulphide content that are exposed to air. This type of weathering of mine waste materials over time can contribute to saline, acidic, and/or metalliferous drainage. For example, Kuznetsov et al. (2015) and Kuznetsova et al. (2016) documented acidic drainage in accelerated weathering lab tests using Froth Treatment Tailings. These researchers found that drainage evolved over the course of the test from a neutral pH of ~7 to 2 and that, once the pH declined, the rates of metal release increased. Overburden piles that are composed of the pyrite-rich Clearwater shale formation may also weather to produce saline and/or acidic drainage (Chapman 2008, Appels et al. 2017).

Two scenario were considered to illustrate how weathering of the above-noted materials could influence water quality at closure:

1. Clearwater overburden seepage reporting to the pit lake is saline water typical of Clearwater shale overburden landforms.
2. 30% of the inflow from the above-grade tailings landform (7% of inflow to the pit lake) is saline water originating from high-sulphide tailings that have undergone subaerial weathering.

The water chemistry inputs for weathering sensitivity scenarios are provided in Table 2, and the results of the sensitivity analysis are shown in Figure 5.

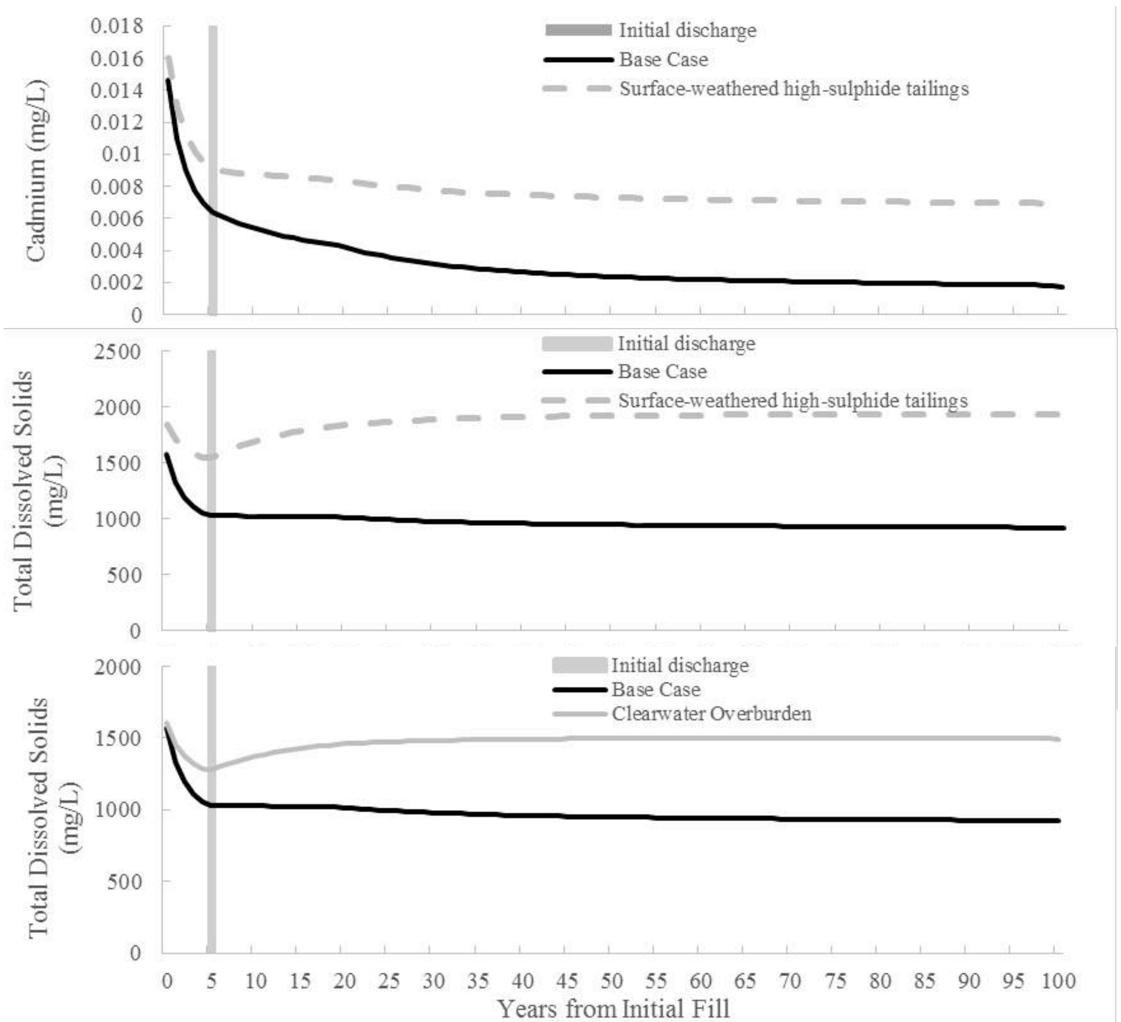

Figure 5: Surface weathering of high-sulphide tailings and overburden sensitivity results. Note in the tailings sensitivity scenario high-sulphide surface-weathered tailings water chemistry values were applied to 30% of the drainage from above-grade tailings.

This analysis focuses on TDS, and cadmium in the case of tailings weathering scenario, because monitored and geochemical data was available for these constituents from materials undergoing subaerial weathering. The results show that weathering of Clearwater shale and tailings can influence pit water quality results. TDS concentrations are notably higher than the chronic effects benchmark (1000 mg/L) in both sensitivity scenarios, which suggests that weathering of suphides is potentially an important process that should be given due consideration in tailings and waste management plans. Similarly, cadmium in the tailings weathering sensitivity scenario is notably higher compared to Base Case, as is typical in water bodies receiving acid mine drainage waters.

3 RECOMMENDATIONS

As discussed in the previous section, closure water quality forecasting is sensitive to a number of chemistry inputs and assumptions, and variations in the values used can often affect the projected viability of a given pit lake. Therefore, it is important to detect deviations in input assumptions or from forecasted water quality early to protect the environment and minimize economic risks. The following recommendations were developed to help in the identification of deviations from forecasted predictions, and confirm the accuracy of model assumptions, during operations when mitigation options are more numerous and less costly.

3.1 *Degradation rates*

Despite considerable research into the composition and analysis of naphthenic acids, there are few published studies that provide degradation rates. Given the range of outcomes that are possible under different degradation rates, site-specific testing of naphthenic acid and associated toxicity degradation rates would reduce uncertainty in predicting closure water quality and therefore closure liability.

Aerobic degradation of naphthenic acids in process water and closure landforms is likely the most cost-effective way to reduce both concentrations and toxicity associated with naphthenic acids, which emphasizes the need for aerobic pit lakes. There are other effective means to reduce naphthenic acids levels. They include, as detailed in Kannel and Gan (2012) and Whitby (2010), ozonation, photolysis, phytoremediation, and adsorption onto petroleum coke. However, these options are more costly than passive treatment within an aerobic pit lake.

3.2 *Process water chemistry*

We recommend regular, consistent collection of water samples from the process plant, tailings ponds, and process water storage ponds that make-up the process water loop during operation. The frequency of monitoring will be site-specific and dependent on process water chemistry variability and operational considerations (e.g., changes to processing, ore quality, and water management). Recommended analytes are those included in baseline monitoring programs outlined in recent oil sands Environmental Impact Assessments (e.g. Teck 2015). Where site-wide monitoring programs are already established, it is recommended these data be compared to water quality model inputs, and interpreted in the context of closure success.

If a deviation in process water chemistry is detected, the source of the deviation should be identified to understand how the magnitude of deviation is likely to change over time during operations. Closure forecasts should be revisited and updated accordingly to reflect the observed deviation in process water chemistry. If necessary, closure planning activities should be altered to reflect the change in process water chemistry to maintain a viable closure landscape.

3.3 *Interflow chemistry*

To determine whether interflow chemistry will follow expectations, the following should be measured and monitored during periods of progressive reclamation:

- Depth of reclamation material
- Surface slopes of reclamation layer and underlying tailings material
- Water table elevation / zone of saturation within reclamation layer and within underlying tailings material

- Water chemistry within the reclamation layer and in the waste materials underlying the reclamation layer
- Water flow rates and water quality at the point(s) of discharge from reclaimed landforms

Where appropriate, measurement points should be sufficiently distributed over the surface of a reclaimed landform to account for heterogeneity in the application of reclamation materials.

Should this type of monitoring suggest that interflow water quality may be different than assumed, closure forecasts should be revisited and updated accordingly. Column testing to measure the rate of flushing of process water from the pores of the waste in the interflow zone should be considered, where appropriate.

3.4 *Weathering of mine waste materials*

There are many options related to waste deposition planning, treatment, and management that can be used to prevent geochemical risks associated with long-term drainage quality. To identify the risks early, and hence have capacity to implement the strategies if needed, we recommend that a subset of mined and processed materials undergo geochemical testing. Geochemical testing is conducted to identify the potential for waste materials to contribute to saline, acidic, and/or metalliferous drainage. Several guidance documents provide recommendations for geochemical testing, though some procedures need to be modified due to the influence of bitumen on test results. Documents commonly used in regulatory jurisdictions in Canada and abroad include:

- Prediction Manual for Drainage Chemistry from Sulphidic Geologic Materials, MEND Report 1.20.1, December 2009; and
- The Global Acid Rock Drainage Guide, International Network for Acid Prevention (INAP), 2009.

The number of samples to undergo geochemical testing should be selected based on the variability of the materials that will be evaluated and the amount of material that will be produced.

Geochemical characterization testing is typically phased, with initial screening level static tests aimed at identifying samples with the greatest potential for generating acidic, saline, and/or metalliferous drainage, and subsequent kinetic tests aimed at confirming this potential and better understanding long-term environmental behavior. Static tests include acid base accounting, net acid generation testing, mineralogical and metal analysis, and short-term leach testing. Kinetic tests can be performed in the laboratory or in the field. Laboratory tests (ASTM, 2013), are designed to enhance mineral reaction rates relative to field conditions. Field scale tests are performed to confirm that the results of laboratory testing are representative of chemical weathering rates achieved under site conditions. Kinetic tests include humidity cell tests, field barrel tests, and column tests.

In addition to geochemical testing, we recommend that runoff from, and seepage through, closure landforms be monitored during operations to confirm assumptions regarding the long-term weathering behavior of mine waste used in water quality forecasting. This testing can be combined with that outlined above with respect to interflow chemistry.

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Oil Sands Modelling

Step-Loading and Seepage Induced Approach to Determine Consolidation Properties of Mature Fine Tailings

G. Gjerapic
Golder Associates Inc., Denver, CO, USA

H.J. Kim & D. Znidarcic
University of Colorado, Boulder, CO, USA

ABSTRACT: Mature Fine Tailings (MFT) containing significant portion of clay minerals are characterized by slow rates of consolidation and are expected to undergo large deformations during the closure and post-closure periods. The determination of MFT consolidation properties, the void ratio – effective stress (compressibility) and the void ratio – hydraulic conductivity (permeability) relationships, typically involves non-standardized large strain compression testing. As the vertical load is applied in step increments, the testing often requires several weeks or even months to complete depending on the sample properties, laboratory set-up configuration and the adopted loading and permeability procedures.

This paper presents a comparison between the step-loading method and the seepage induced consolidation technique. Based on the closed form constitutive relationships in the literature, the seepage induced consolidation technique allows for the determination of MFT parameters using the inverse problem solution approach. This methodology reduces the number of loading steps and, consequently, the amount of time required to define consolidation relationships over the stress range of interest.

For a given set of MFT parameters, the step-loading and seepage induced consolidation methods were evaluated by considering sample heights of 50 and 100 mm. Analyzed scenarios indicate that the seepage forces at the unit gradient condition induce displacements ranging from approximately 43 to 48 percent of the displacements caused by the maximum step-load of 100 kPa. The application of seepage stresses at the bottom of the sample of up to 5 kPa, has the potential to reduce the testing time by more than 50 percent.

1 INTRODUCTION

1.1 *Background*

Selection of effective closure strategies for the tailings ponds containing oil sand tailings remain a significant challenge due to the high clay content of Mature Fine Tailings (MFT). Settlement predictions for the existing impoundments indicate that the consolidation of MFT tailings may require periods of more than 100 years to approach steady-state conditions. With the advent of new technologies to improve initial MFT densities and the need for accurate dewatering and settlement predictions, there is continuous effort to improve methodologies for determination of MFT consolidation parameters. Laboratory techniques that are commonly used for determination of MFT consolidation properties include settling columns (hindered sedimentation and compressibility standpipe tests) and large strain consolidometer (LSC) testing (see e.g. Scott et al. 2008). As the range of consolidation parameters determined from settling column tests is generally restricted to low effective stresses (typically below 1 kPa), the LSC test results are often more relevant to engineering designs concerned with geotechnical properties of tailings at closure.

Laboratory determination of MFT properties using LSC cells typically involves the step-loading approach where the vertical load on the sample is doubled in each sequential loading increment. The starting load applied on the surface of the sample is typically on the order of 0.1 kPa. At the end of each loading increment, the hydraulic conductivity is determined by applying an upward gradient of less than 0.5 using the constant head technique. The sample height for the LSC test typically varies from 0.1 to 0.5 meters (see e.g., Jeeravipoolvarn et al. 2015). Depending on the number of loading increments and sample characteristics (geometry and material properties), the LSC test may require several weeks to several months to complete.

The Seepage Induced Consolidation Test (SICT) was developed at the University of Colorado at Boulder (Abu-Hejleh and Znidarcic 1994, Abu-Hejleh et al. 1996) with the goal to provide reliable determination of consolidation parameters for soft slurries. The test utilizes seepage forces to trigger the consolidation process at low effective stresses followed by the step-loading to determine a void ratio-effective stress points at higher effective stresses and direct hydraulic conductivity measurement at higher effective stresses. The test was successfully used to determine consolidation characteristics for MFT (see e.g. Znidarcic et al. 2011 and Estepho et al. 2013).

1.2 *Material Parameters*

The MFT consolidation properties are commonly described in terms of the void ratio – effective stress (compressibility) and the void ratio – hydraulic conductivity (permeability) relationships. Soft slurries are capable of undergoing large changes in void ratio over the anticipated range of effective stresses. One of the first compressibility relationships used in the industry to describe non-linear behavior of soft tailings deposit was the power-law function introduced by Somogy (1979)

$$e = A\sigma'^{B}. \tag{1}$$

To account for the double curvature of the compressibility function (i.e. an "S"-shaped curve) and to allow for the finite initial void ratio of the tailings deposit, the modified power-law function (Liu and Znidarcic 1991)

$$e = A(\sigma' + Z)^{B}, \tag{2}$$

and Weibull function (see e.g., Jeeravipoolvarn et al. 2009) may be employed:

$$e = A - B\exp(-E\sigma'^{F}). \tag{3}$$

In the above equations, e stands for the void ratio and σ' denotes the effective stress. Parameters A, B, Z, E, F are determined by fitting the selected form of the compressibility curve to laboratory measurements. The permeability relationship is typically expressed as a power-law function (Somogy 1979)

$$k = C\,e^{D}, \tag{4}$$

Where k stands for the hydraulic conductivity of the tailings deposit and C and D are material parameters determined from laboratory measurements.

1.3 *Motivation*

The LSC test on MFT samples presents challenges when attempting to account for non-linearity of the void ratio profile at low effective stresses with the testing duration often exceeding several months to complete. Typically, the LSC test is conducted on a single sample with the loading applied by weights or using pressure cylinders by doubling the applied vertical stress on the sample in successive increments. This paper compares potential advantages in applying the inverse problem solution approach and seepage induced consolidation technique to improve interpretation of the test results and to reduce the test duration of the conventional LSC test on MFT samples. A comparison between the conventional step-loading approach and the seepage induced consolidation technique was conducted by simulating laboratory testing for constitutive relationships defined by Equations (2) and (4) with the model parameters selected based on the available literature data on fine oil sand tailings. Consolidation parameters for the fine tailings sample reported by Jeeravipoolvarn et al. (2009) and parameters for MFT samples by Estepho et al. (2013) are compared in Table 1 to provide an indication of variability of consolidation properties for different fine oil sand tailings samples. Numerical models used in this study utilize the consolidation parameters for MFT-1 sample.

Table 1. Parameters for fine oil sand tailings based on data from Jeeravipoolvarn et al. (2009) and Estepho et al. (2013)

	A	B	Z	C	D
	$(1/\text{kPa})^B$	()	(kPa)	(m/day)	(-)
Fine Tailings	4.027	-0.3603	0.400	6.91×10^{-6}	3.69
MFT-1	3.00	-0.177	0.035	1.21×10^{-6}	3.55
MFT-2	3.52	-0.236	0.181	9.50×10^{-7}	3.79
MFT-5	3.27	-0.195	0.074	2.16×10^{-6}	3.39
MFT-6	3.20	-0.169	0.007	1.38×10^{-6}	3.71

Compressibility curves for the fine oil sand tailings based on data from Jeeravipoolvarn et al. (2009) and Estepho et al. (2013) are presented in Figure 1.

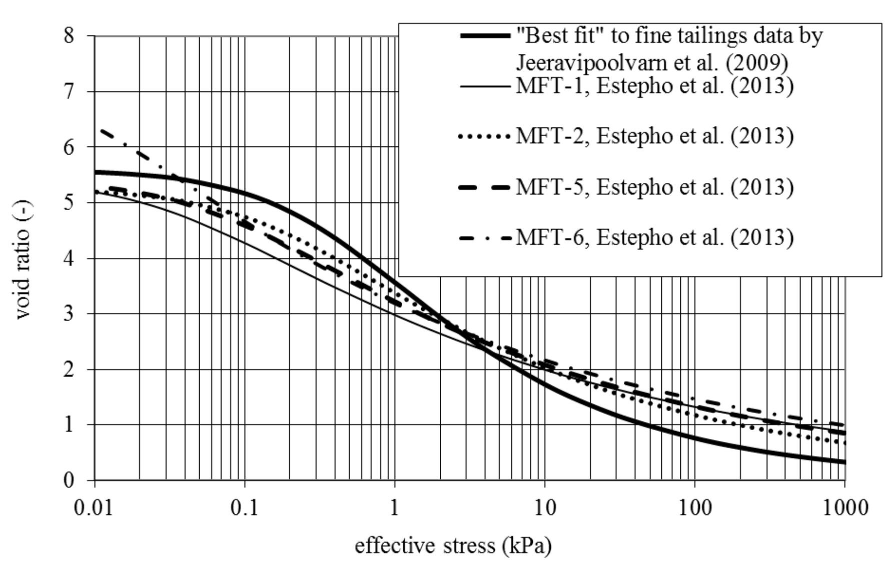

Figure 1. Compressibility Curves for Fine Oil Sand Tailings

Permeability curves for the fine oil sand tailings based on data from Jeeravipoolvarn et al. (2009) and Estepho et al. (2013) are presented in Figure 2.

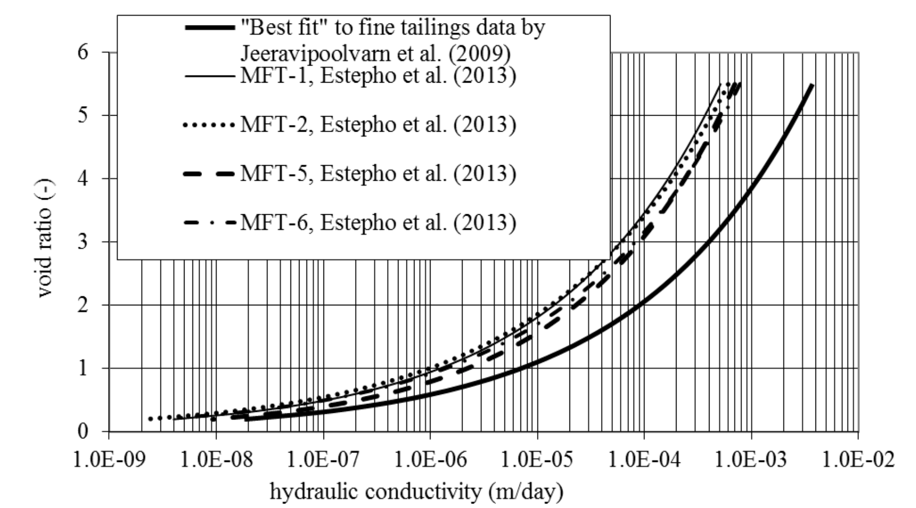

Figure 2. Permeability Curves for Fine Oil Sand Tailings

2 APPROACH

At low effective stresses, the void ratio distribution within the sample is non-linear and may show significant variability due to the influence of self-weight consolidation stresses. The variability of the void ratio profile is more pronounced for the tests where the sample is subjected to seepage forces (see Figure 3). By employing constitutive models capable of capturing the void ratio variability within the sample of soft tailings, test results can be used to determine permeability and compressibility parameters for a given set of boundary conditions imposed during the loading process. In terms of effective stresses at the bottom of the sample, one can write:

1) For the step-load test.

$$\sigma'_{BOT} = \sigma'_{TOP} + \int_0^{Hs} (G_s - 1)\gamma_w \, d\lambda = \sigma'_{TOP} + (G_s - 1)\gamma_w H_s \quad (5)$$

2) For the seepage induced consolidation test.

$$\begin{aligned} \sigma'_{BOT} &= \sigma'_{TOP} + \int_0^{Hs} (G_s - 1)\gamma_w \, d\lambda + \int_0^{H_s} \frac{v}{k(e)} \gamma_w (1+e)\, d\lambda \\ &= \sigma'_{TOP} + (G_s - 1)\gamma_w H_s + \Delta u \end{aligned} \quad (6)$$

Where

σ'_{TOP} = effective stress at the top of the sample

σ'_{BOT} = effective stress at the bottom of the sample

γ_w = unit weight of water, 9.81 kN/m^3

Hs = total height of solids. Note that the average void ratio, e_{avg}, is calculated from the current sample height, H, and the total height of solids as e_{avg} = H/Hs-1.

dλ = height of solids increment

v = downward flow rate through the sample induced by the flow pump

Δu = pressure difference across the sample (total head difference across the sample multiplied by γ_w). This value if often measured directly by using the differential pressure transducer.

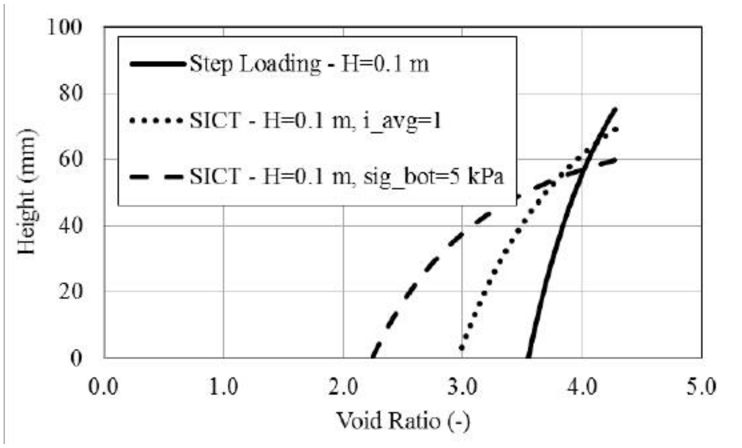

Figure 3. Void Ratio Distribution for Seepage Induced Test at Surface Load of 0.1 kPa.

It is of importance to note that Equations (5) and (6) are valid only for the steady-state conditions, i.e. for the condition at which both the height of the sample and the velocity across the sample remain constant with time. Equation (6) can be generalized to express the effective stress at any point within the sample as a function of the height of solids, λ, as

$$\sigma'(\lambda) = \sigma'(0) + (G_s - 1)\gamma_w \lambda + \int_0^{\lambda} \gamma_w \frac{v}{k[e(\sigma')]}[1 + e(\sigma')]d\lambda \tag{7}$$

The height of the sample can now be determined by integrating the void ratio profile as a function of the effective stress

$$H = \int_0^{Hs} (1+e)\, d\lambda = \int_0^{Hs} \{1 + e[\sigma'(\lambda)]\} d\lambda \tag{8}$$

When combined with the constitutive equations (2) and (4), equations (7) and (8) allow for the determination of the final height of the sample at the end of the consolidation process (i.e. after reaching the steady-state conditions). Equation (8) allows for validation of the selected consolidation parameters by monitoring the height of the sample at different stages of the test. By employing the inverse problem solution approach, one can now determine constitutive parameters A, B, C, D and Z for an arbitrary set of seepage and loading conditions. A minimum of three tests are required to determine compressibility parameters A, B and Z. Similarly, a minimum of two permeability tests are required to determine parameters C and D. Additional loading and permeability tests are typically conducted to provide data redundancy and confirm validity of the adopted constitutive models.

Seepage induced consolidation test can be used to apply arbitrarily low gradients through the sample allowing for the controlled decrease of void ratios at low effective stresses. An increase in the seepage flow velocity imposes higher gradients through the sample, resulting in the gradually decreasing sample heights and increased average densities. Consequently, the SICT may be conducted to allow for gradual increase in average effective stresses prior to increasing the surface load on the sample by using weights or adding the load by pressure cylinders.

To allow for data redundancy and validate adopted constitutive model parameters, it is recommended to determine the void ratio at "zero" effective stress (i.e. the effective stress smaller than 0.1 kPa) by using results from the settling column tests, see e.g. Scott et al. (2008) and Stianson at el. (2016). Additional consolidation data are proposed to include seepage consolidation tests with the surface load of approximately 0.1 kPa and the bottom effective stress ranging from 0.5 to 5 kPa. While a single SICT is sufficient to determine consolidation parameters defined by Equations (2) and (4), at least two SICTs are recommended when testing MFT materials for the purpose of data validation and to provide gradual transition towards the step loading measurements. The consolidation testing is concluded by conducting step loading tests with seepage permeability measurement until reaching the maximum effective stress of interest for the design. It is recommended for the successive loading steps not to exceed the order of magnitude when applying the next stress increments. E.g. for the maximum stress at the bottom of the sample of 5 kPa induced by SICT, the step loading phase should not apply the stress exceeding

50 kPa. In addition, a gradual loading of softer tailings samples may be needed after completion of individual loading phases in order to prevent "squeezing" of material between the top piston and the sides of the holding chamber, i.e. the surface of the sample may need to gain sufficient strength prior to applying the target load. However, when applying intermediate loads for the sole purpose of increasing the sample strength, these additional loads are typically applied relatively rapidly, e.g. well before reaching 90 percent consolidation.

3 STEP-LOAD TEST EXAMPLE

Material parameters for MFT-1 sample in Table 1 were used to demonstrate the potential to reduce testing times for MFT samples utilizing smaller initial heights and by introducing the seepage induced consolidation technique at effective stresses between 0.1 and 10 kPa. Laboratory samples with the initial heights of 0.1 and 0.05 meters were selected to simulate LSC and SICT procedures.

3.1 *Results for 0.10 m column*

Results for the MFT sample with the initial height of 100 mm are summarized in Table 2.

Table 2. Results for MFT Sample – Initial Height = 100 mm.

Surface Load (kPa)	t95 (day)	t100[1] (day)	Test Duration for t_95 (day)	Test Duration SICT t_95[2] (day)
0.1	90	210	90	
0.75	36	44	126	
1.5	33	42	159	
3.0	30	42	188	233 (150)[31]
6.0	24	42	212	
12	11	33	223	
25	4	22	227	
50	3	14	230	
100	3	4	233	

1) t_100 times were determined for sample heights within 0.5% of the final value
2) SICT results illustrate potential reduction in testing time when introducing average unit gradient conditions across the sample. The value in parentheses denotes estimated testing time for which the imposed seepage gradient resulted in the pressure difference of approximately 5 kPa across the sample at the end of SICT for constant flow rate conditions. Value in brackets denote estimated testing time for the constant pressure difference of 5 kPa across the sample.

Values t95 and t100 in Table 2 denote times for the MFT sample to reach 95 and 100 percent consolidation at each loading increment. The time t100 was determined from the calculated settlement values using sample heights within 0.5 percent of steady-state conditions. At the end of testing, a cumulative duration of 233 days was determined by adding t95 values for individual loading tests, i.e. the estimated test duration based on t95 values should be viewed as a best-case estimate. The actual test duration in the laboratory is expected to be significantly longer than the duration estimate from t95 values because the laboratory test termination criteria (for each loading increment) are commonly defined in terms of the steady-state conditions and due to required times to conduct permeability measurements after reaching the steady-state. Estimated duration for the simulated step-loading test on MFT-1 sample up to the vertical stress of 3.0 kPa is illustrated in Figure 4. The SICT results, determined for a comparable stress range, are included in Figure 4 for comparison.

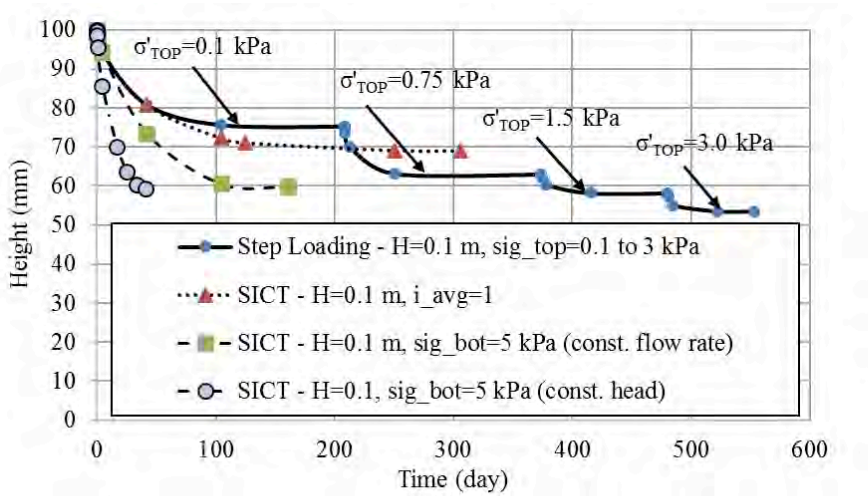

Figure 4. Time Settlement Curve for Sample Height of 100 mm.

3.2 *Results for 0.05 m column*

In order to reduce the time of testing, shorter soil columns may be utilized (see e.g., Estepho et al. 2013). Results for the MFT sample with the initial height of 50 mm are summarized in Table 3.

Table 3. Results for the MFT Sample – Initial Height = 50 mm

Surface Load (kPa)	t95 (day)	t100[1] (day)	Test Duration for t_95 (day)	Test Duration SICT t_95[2] (day)
0.1	34	42	34	
0.75	26	42	60	
1.5	10	34	70	
3	4	22	74	43 (35)[11]
6	3	9	77	
12	3	4	80	
25	2	4	83	
50	1	4	84	
100	1	3	85	

1) t_100 times were determined for sample heights within 0.5% of the final value.
2) SICT results illustrate potential reduction in testing time when introducing average unit gradient conditions across the sample. Value in parentheses denotes estimated testing time for which the imposed seepage gradient resulted in the pressure difference of approximately 5 kPa across the sample for constant flow rate conditions. Value in brackets denote estimated testing time for the constant pressure difference of 5 kPa across the sample.

Results in Table 3 illustrate the potential for significant reduction in the required testing time when allowing for the increase of effective stresses due to seepage gradients, i.e. by introducing the seepage induced consolidation test (SICT) approach. It is important to emphasize that the majority of the testing time is expended to consolidate the sample at lower effective stresses.
For LSC test, the duration of 74 days was required to complete the consolidation process up to 3 kPa while only additional 11 days were required for the remaining loading increments up to 100 kPa. For SICT conducted at unit gradient conditions, approximately 43 percent of the total settlement (induced by the load of 100 kPa) is realized at the surface effective stress of 0.1 kPa and induced seepage forces during the first 43 days of testing. Results in Table 3 indicate that the majority of consolidation water is expelled at lower effective stresses leading to significant time savings when comparing SICT to LSC approach. SICT durations reported in Tables 2 and 3 (with the exception of values in square brackets) are conservative as they are calculated for con-

stant pump rates. Further reduction in the required testing times are achieved if the ultimate bottom stress is maintained at constant value by using variable pump speeds or by imposing the constant head difference across the sample as illustrated in Figure 4.

4 CONCLUSIONS

This paper demonstrates the benefits of using the inverse problem solution approach to determine compressibility and permeability parameters of oil sand tailings. The use of closed-form relationships required to match laboratory data, using the range of effective stresses of interest for the design, requires numerical fitting of constitutive model parameters regardless of the laboratory technique used to determine individual effective stress – void ratio – permeability measurements.

The use of constitutive models validated in the oil sand tailings design practice during the last several decades provides the mean to reduce the number of individual laboratory tests and simplify testing methodology that is currently used to determine compressibility and permeability relationships of oil sand deposits.

The complexity and extent of laboratory testing should match the design requirements, allow for accurate representation of variability of material properties, and provide sufficient redundancy to validate adopted constitutive model against laboratory measurements. The use of different testing techniques, e.g., seepage induced consolidation and step-loading techniques may be combined to improve reliability of test results and reduce testing times.

The seepage induced consolidation test is suitable for testing soft tailings with low permeability values, such as MFT, as it accounts for the non-linear void ratio distribution within the sample due to the self-weight of tailings material and the induced seepage forces. Consequently, the test improves the reliability of calculated consolidation parameters at low effective stresses with the added benefit of reduced testing duration as it allows for faster dissipation of excess pore pressures due controlled seepage gradients induced by the flow pump.

Total duration of the conventional LSC test on MFT samples can be significantly reduced by using smaller initial heights, by applying seepage induced consolidation technique at lower effective stresses, and by reducing the number of step-loading tests at higher vertical loads. Minimum requirements to complete the consolidation test on MFT sample within the time-frame of 6 to 8 weeks are as follows: 1) perform settling column test to determine the void ratio at "zero" effective stress (i.e. to determine the "soil-forming" void at or below the effective stress of 0.1 kPa, 2) conduct SICT using the initial sample height of approximately 50 mm with the target bottom effective stress between 1 to 5 kPa, 3) gradually, over the period of several hours, load the sample until reaching the maximum design stress, and 4) perform direct hydraulic conductivity measurement (after reaching the degree of consolidation of at least 95 percent at the maximum design stress).

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Delft3D Modelling of Sand Placement on an Oil Sands Treated Tailings Deposit

Monica Ansah-Sam
Canadian Natural Resources Limited, Calgary, AB, Canada

Ben Sheets & Jim Langseth
Barr Engineering and Environmental Science Canada Ltd., Calgary, AB, Canada

Luca Sittoni & Jill Hanssen

ABSTRACT: Incorporating sand layers in a soft high-fines tailings deposit may be an effective means of improving tailings drainage, consolidation, and strength. Various methods have been used for sand placement, ranging from relatively aggressive mechanical approaches to more gentle barge-based sand raining. In between these two endmembers, hydraulic placement of sand, using coarse sand tailings flows, is of interest for its potential as a relatively low-cost approach. Hydraulic sand deposition onto a centrifuged deposit was performed at the Canadian Natural Albian Jackpine Mine External Tailings Facility (Dedicated Disposal Area 1). A delta was formed near the sand outfall, which resulted in displacement of the majority of previously deposited centrifuge deposit. Cone Penetration Testing and deposit samples showed some mixing of sand into the deposit. This paper discusses a Delft3D slurry model used to model the deposition, and explores the controls on hydraulic placement of granular material on fine tailings.

1 INTRODUCTION

The placement of centrifuged fluid fine tailings into the Jackpine Mine (JPM) Dedicated Disposal Area (DDA1) is a component of reducing the fluid fine tailings inventory at JPM as part of the Canadian Natural Albian Sands (Canadian Natural) program to meet the requirements of the Tailings Management Framework issued by the Government of Alberta in March 2015. The centrifuge product has average solids content of 45%, produced from fluid fine tailings (about 25% solids) dredged from a tailings pond at JPM. The centrifuged material is being deposited to create a deep deposit at the JPM DDA1. Additional details on the centrifugation process at JPM are provided in Graham, et al. (2016).

Characterization of tailings deposits provides necessary information to support reclamation and closure design (Ansah-Sam et al, 2016). Annual tailings investigations have been conducted since the start of operations in 2013 at JPM DDA1 to monitor the volume and material properties of the centrifuge deposit and also to assess the potential to co-deposit sand and thickened tailings with the centrifuged material. The annual tailings investigations also provide an understanding of the historic behavior and current conditions, as well as the ability to predict the long-term geotechnical behaviour of the deposit. Another step towards the closure and reclamation goal for the JPM DDA1 was to understand the feasibility of sand capping the centrifuge deposit.

In general, the main goals of placing a cap are typically to:

1. Enhance Consolidation (provide load, manage release water etc.).
2. Provide a buffer between the reclamation cover and the deposit.

3. Provide and maintain long and short term trafficability (allow access for placement of other loads, allow access for wick drains installation etc.).
4. Manage surface water.
5. Provide nutrients for plants and animals.
6. Minimize acid mine or rock drainage, by creating water capped deposits or end pit lakes.

The first and third points are the main geotechnical drivers for successful sand placement on treated tailings deposits.

Modelling studies were performed to evaluate mechanical and hydraulic sand placement methods, followed by areas identified for capping test trials. The JPM DDA1 was identified as a good candidate for hydraulic sand placement due to the presence of the necessary infrastructure at this facility as a result of dyke or beach construction.

Delivery of coarse sand tailings (CST) to DDA1 was initiated with the potential for capping, mixing, or increasing strength and solids content of the centrifuge deposit. Coarse Sand Tailings was delivered to the northwest corner of DDA1 in August 2016 and the resultant deposit was sampled.

To help understand the conditions under which successful sand capping of centrifuge deposits might be expected, the Delft 3D model was used on an experimental basis. The main objective of the modelling was to determine whether the observed CST/centrifuge interaction could be reproduced by the model, and therefore develop a tool to estimate necessary conditions for capping. The modelling work presented in this paper is preliminary and exploratory, as the version of the Delft3D software used herein is currently under development in partnership with Canada's Oil Sands Innovation Alliance (COSIA).

1.1 *August 2016 Deposition*

Coarse sand tailings was placed using an end-of-pipe energy dissipation device into the northwest corner of DDA1 at 7700 m^3/hr for approximately seven hours, split over the course of August 24 and 25, 2016. Figure 1 shows aerial photography of the DDA1 northwest corner before and after the trial.

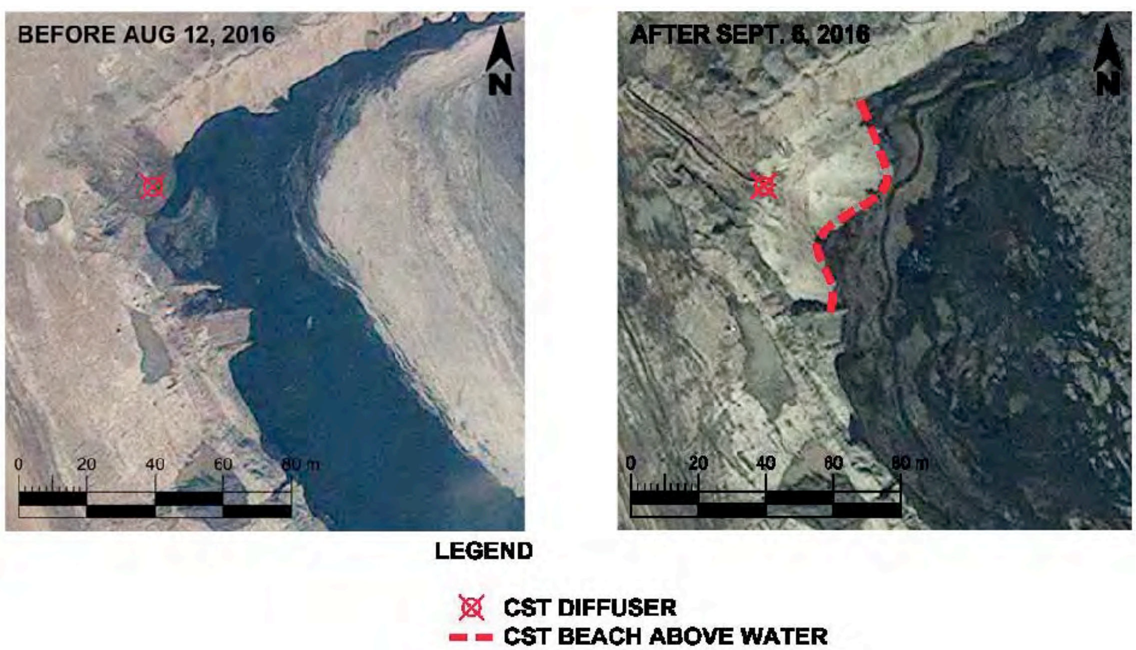

Figure 1. 2016 Trial aerial photography

The centrifuge deposit was sampled during the 2015 annual investigation program at DDA1 prior to the 2016 trial. The mixed CST/centrifuge deposit was sampled after the trial in late 2016, with some sampling locations corresponding to the 2015 locations, as well as some additional locations. Comparison of deposit characteristics can be made before and after the trial with the assumption that the 2015 sample results were representative of the centrifuge deposit.

The following illustration (Figure 2) provides a simplified cross section (D-D') through the northwest corner of DDA1, approximately along the primary direction of CST flow. Figure 2 shows the sampling/Cone Penetration Testing (CPT) data, elevation of the 2015 hard bottom (HB) surface and the 2015 centrifuge deposit surface. The laboratory sample data and a simplified interpretation of these data as used in the model are also shown on the figure, which summarizes the primary deposit types interpreted from laboratory data. Cone Penetration Testing and core locations farther east from those on Figure 2 show no influence of CST sand. A more detailed interpretation of the laboratory and CPT data has subsequently been prepared, the details of which will be subject of another paper.

The post-trial deposit investigation indicates that a significant CST sand beach deposit formed in the northwestern-most corner of DDA1 adjacent to the CST discharge. Deposit sampling showed that this CST deposit was characterized by solids contents of 80% and higher, and fines contents below 5%. This deposit is indicated by dark grey shading on Figure 2.

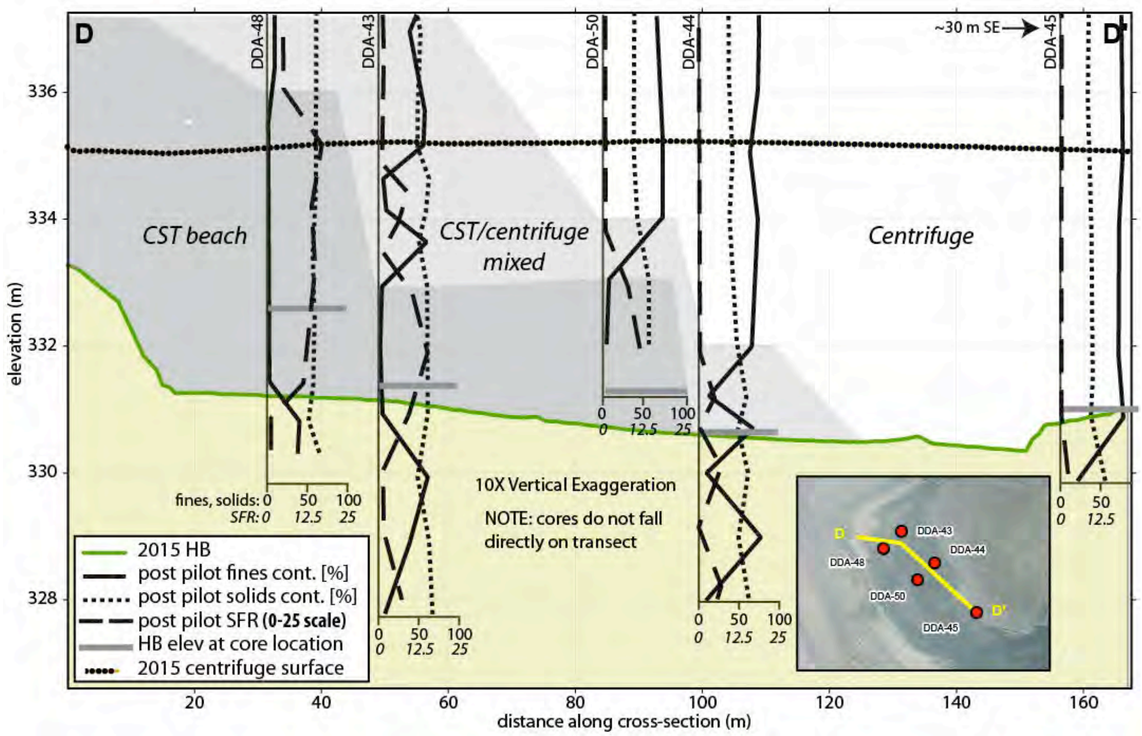

Figure 2. Post-trial fines content, solids content, and sand-to-fines ratio (SFR). Simplified deposit interpretation discussed in text shown by grey shading and labels.

In portions of the pond unaffected by the CST placement, the centrifuge deposit remains largely unchanged, though the elevation of the centrifuge has risen around two meters between 2015 and 2016 sampling events, primarily due to centrifuge placement prior to the 2016 trial. The pure centrifuge deposit in DDA1 shows very high fines contents (90%+) and those deposits are not shaded in Figure 2.

In between the CST deposit and pure centrifuge deposit, a mixed deposit of variable thickness, sand content, and strength was found. This deposit was thickest near the CST discharge location, and 'thinned' with distance into the pond. Solids contents, fines contents, and sand-to-fines ratios (SFR) varied in this mixed deposit, and generally showed consistent trends away from the CST discharge point. These trends indicate that mixing of CST sand into the centrifuge deposit was greatest near the discharge point (higher mixed zone sand content, solids content, and SFR). This CST/centrifuge mixed zone is indicated by intermediate grey shading on Figure 2.

2 DELFT3D MODELLING

No standard "off-the-shelf" modelling approach exists to simulate the CST/centrifuge mixing processes occurring in DDA1. While there are a number of commercially available modelling software packages that deal with sediment transport and mixing in Newtonian fluids, particularly water, essentially no standard tool exists for non-Newtonian dynamics of this sort. Thus, at Canadian Natural we employed a modelling approach that is currently under development by Deltares in partnership with COSIA, which is described in the following subsection.

2.1 *Modelling approach*

The hydrodynamic component of the standard Delft3D modelling software, Delft3D-Flow, is a widely used open source, physics-based, numerical model developed and maintained by Deltares. The standard hydrodynamic and sediment transport modules simulate morphological changes resulting from transport of cohesive and non-cohesive sediments at relatively low concentrations in Newtonian flows comparable to those commonly observed in natural environments, such as rivers and coastlines (Deltares, 2014; Lesser, et al., 2004). In such flows the sediment dynamics are typically computed as a function of flow velocity, turbulence, sediment particle diameter, and specific gravity.

The Delft3D-slurry formulation used in this study is a research version of Delft3D developed specifically for non-Newtonian slurry flows comprising sand and mud, and exhibiting a finite (though potentially variable) yield stress. In fluids of sufficiently high yield stress and viscosity, sand may be suspended in the carrier fluid; such slurries are commonly referred to as non-segregating. A complication in simulating such flows arises from variability in yield stress with flow velocity. As flow velocity increases, so does shear rate, which decreases slurry yield stress and viscosity. Sand can settle more readily in lower-viscosity slurries, thereby affecting their rheology, and leading to non-linear flow and mixing behaviour. As such, modelling of such slurries is relatively new, and Delft3D-slurry is a state-of-the-art modelling approach still under development through a series of COSIA-funded projects.

The mathematical theory describing the relationship between rheology, flow, and sediment dynamics for non-Newtonian non-segregating slurries in the oil sands was described by Sisson, et al. (2012), and was significantly updated, including the Delft3D-slurry implementation, by Hanssen (2016), and Van Es (2017).

2.1.1 *Assumptions*

Given the "*beta*" (still in testing/development) nature of the Delft3D-slurry implementation of non-Newtonian flow, a number of assumptions and simplifications of the DDA1 environment were necessary to conduct the simulations presented in this paper:

1. No beach was formed. Currently the model allows sand to settle to the bed and leave the model domain, but the morphology of the bed does not change. This has significant implications for how the model results match the DDA1 observations, which will be discussed in Section 3.
2. CST/centrifuge mixing was modeled as a 2D phenomenon (vertical and with distance from CST discharge). Lateral flow (perpendicular to flow direction) was not simulated. Thus, a reduced CST discharge was used to simulate only a portion of the flow that would likely have impacted the D-D' cross section.
3. Centrifuge deposit properties were assumed to be spatially uniform, and adequately represented by the 2015 DDA1 characterization.
4. Water release during flow of CST, or resulting from CST interaction with centrifuge deposit, was not included in the modelling.
5. Post-depositional processes, such as consolidation and dewatering, were not included in the modelling.
6. The seven-hour trial duration was modeled continuously, rather than split over two days.
7. CST discharge and character (e.g. solids content) were assumed to be constant with time.

The effect of these assumptions is significant, but not critical for the general behaviour of the model. The local details, such as precise SFR, sand bed accumulation, post-depositional consolidation, or metre-scale mixing patterns in non-homogeneous centrifuge deposit were not expected to be accurately reproduced in this approach.

2.1.2 *Model boundary conditions*

A number of model simulations were performed to investigate a range of conditions and verify model performance. For the purpose of this paper, we present seven of those simulations that best capture the range of potential field behaviour, and simulate reasonable conditions under which capping might be possible.

In each of these simulations, certain boundary conditions are kept constant. These include the physical geometry (matched to the 2015 hard bottom and centrifuge deposit mudline), the CST discharge (7700 m^3/hr), and trial duration (7 hours). The CST discharge per unit width used in the modelling was 0.1 m^3/second/m. Table 1 lists the variable tailings property boundary conditions of the simulations (important variability *italicized* for emphasis).

Table 1. Preliminary Delft3D-slurry model boundary conditions

Simulation	Description	CST bulk density [kg/m^3]	CST solids content [%]	Centrifuge bulk density [kg/m^3]	Centrifuge solids content [%]	Centrifuge Yield Stress (τ_y) [kPa]
QN1	Base case	1664	63	1400	46	1
QN1b	▼CST solids	*1520*	*54*	1400	46	1
QN1c	▼CST particle density	*1400**	*62*	1400	46	1
QN2	▲centrifuge τ_y	1664	63	1400	46	*2*
QN2b	▲centrifuge τ_y; ▼CST solids	*1520*	*54*	1400	46	2
QN3	▲centrifuge density	1664	63	*1664*	*65*	20
QN6	▲▲centrifuge density	1664	63	*1700*	*68*	20

*Particle density = 1800 kg/m^3

2.2 *Modelling results*

As noted above, the metre-by-metre predictions of specific quantities such as SFR or mixing zone thickness are likely to diverge from DDA1 observations due to the computational assumptions required for the modelling approach. The overall trends in mixing patterns, however, are physically meaningful, and are representative of behaviours likely to occur with CST placement into soft tailings.

2.2.1 *Baseline simulations (1 kPa and 2 kPa centrifuge deposit)*

The initial simulations in the modelling program described here are intended to reproduce the properties of both CST and centrifuge deposit to reflect JPM DDA1 conditions within the modelling framework. Thus, CST and centrifuge densities were matched to observations, and other simulation conditions were scaled to represent those present in the 2016 DDA1 CST placement trial. The cross-sectional results of these simulations are shown on Figure 3. Comparison of these simulations to DDA1 observations will be presented in Section 3.

Figure 3 shows CST sand concentration and total deposit SFR for both QN1 and QN2 simulations, which were targeted at reproducing a reasonable bracket of centrifuge deposit strengths in DDA1. Color mapping on Figure 3 corresponds, as noted, to either the concentration of CST-derived sand relative to total deposit mass, or SFR computed from deposit sand: fines ratio at a given point.

Note that pre-existing centrifuge yield stress exerts a significant control on how far into the facility sand can mix, as well as the sand content of the resultant mixed deposit. Centrifuge deposit with an initial yield stress of 1 kPa leads to a nearly 300 m-long mixing zone, and doubling the yield stress to 2 kPa reduces the mixing zone length to less than 250 m.

Given a constant CST discharge and density, differing mixing zone geometries result in differing sand contents and SFR, with the stronger centrifuge deposit producing modelled SFR values as high as ~1.3:1, and weaker centrifuge leading to SFR no higher than 1:1. In other words, the same volume of sand delivery over seven hours is not as widely mixed across the simulated DDA1 with stronger centrifuge deposit, and thus leads to higher local sand concentrations. The mixed zone associated with stronger centrifuge is somewhat thicker than the mixed zone in weaker centrifuge deposits.

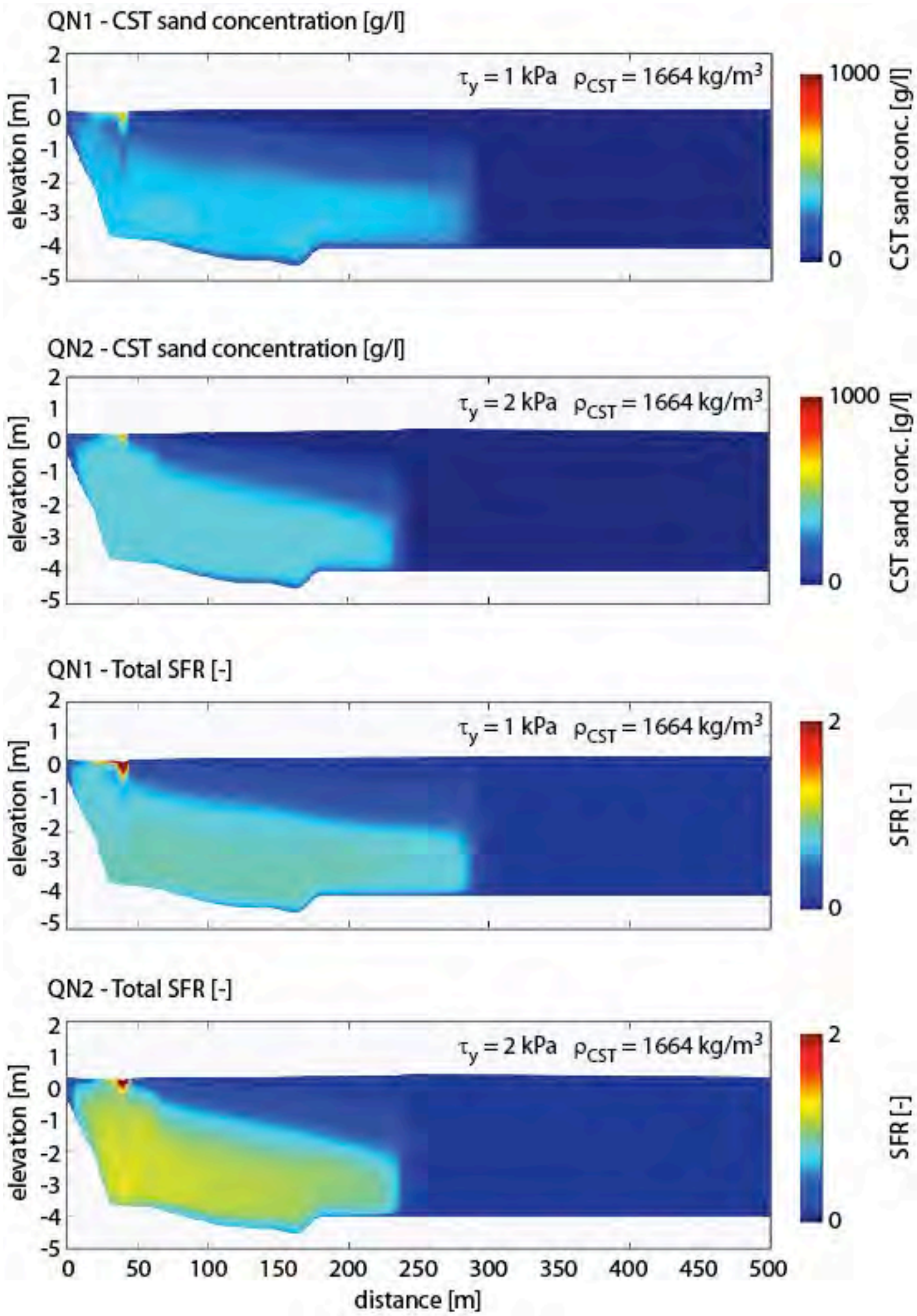

Figure 3. Baseline simulation model scenarios (QN1 and QN2)

2.2.2 *Comparison simulations (lower density CST)*

Figure 4 shows CST sand concentration for two simulations (QN1b and QN2b) with comparable centrifuge deposit density to that in QN1 and QN2, but reduced CST bulk density. These simulations were performed to investigate the possibility that the CST density (and concentration) entering the centrifuge deposit at the beach above water shoreline in DDA1 may not be the same as that measured in pipeline characterization. Due to deposition of CST sand during beach construction it is likely that flow concentration at the effective DDA1 shoreline is lower than that delivered at the end of pipe.

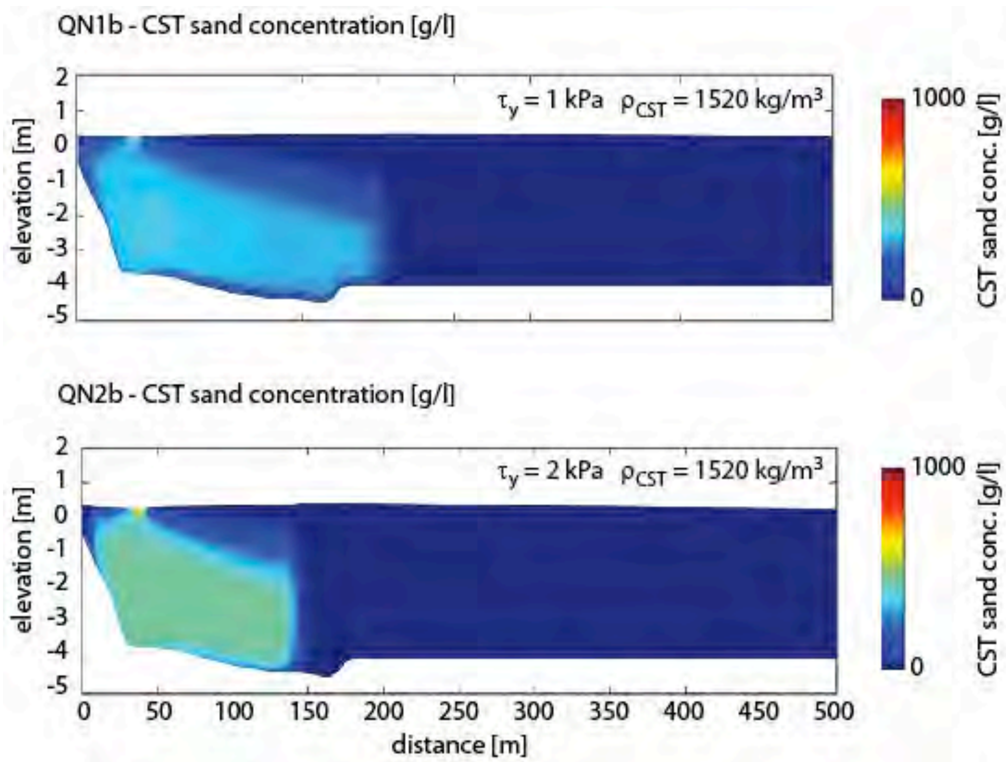

Figure 4. Reduced CST bulk density scenarios (QN1b and QN2b)

The lower CST solids concentration produces an effect consistent with the observations of the baseline simulations. Lower CST density, and therefore less total sand mass delivered to the simulated DDA1 over 7 hours, leads to shorter mixing distances. Interestingly, the reduced CST density also leads to generally higher CST sand concentrations (note different color scales on Figures 3 and 4). This behaviour is presumably due to a less-dense CST flow moving and mixing into centrifuge deposit less energetically than the higher density CST flows of the baseline simulations.

2.2.3 *Capping simulations (higher relative density centrifuge)*

The final set of modelling results we present here involve hypothetical situations (not directly based on existing conditions in DDA1) that might lead to sand capping behaviour over centrifuge deposit. Simulations were run to see if CST might cap centrifuge deposit if bulk densities were comparable. Thus, simulations QN3, QN6, and QN1c investigated various approaches to achieving comparable or lower CST bulk density than centrifuge deposit bulk density. The results of these simulations are presented as CST sand concentration in Figure 5.

Simulations QN3 and QN6 both involve increased centrifuge density, a situation that could be achieved via dewatering, solids content increase, and associated strengthening of the centrifuge deposit after placement. When CST and centrifuge bulk density are similar (QN3), CST sand is concentrated in the upper part of the deposit, but mixes through nearly the entire centrifuge deposit column. Only when centrifuge density is higher than CST density (QN 6) does modelling suggest cap-like behaviour is possible.

Interestingly, a reduction in CST bulk density accomplished via reduction in CST particle density (QN1c), leads to similar behaviour as exhibited in QN6. In this case, the CST solids content remains similar, but the particle density is analogous to coke, or other reduced density material, and predominantly capping behaviour (versus mixing behaviour) is predicted.

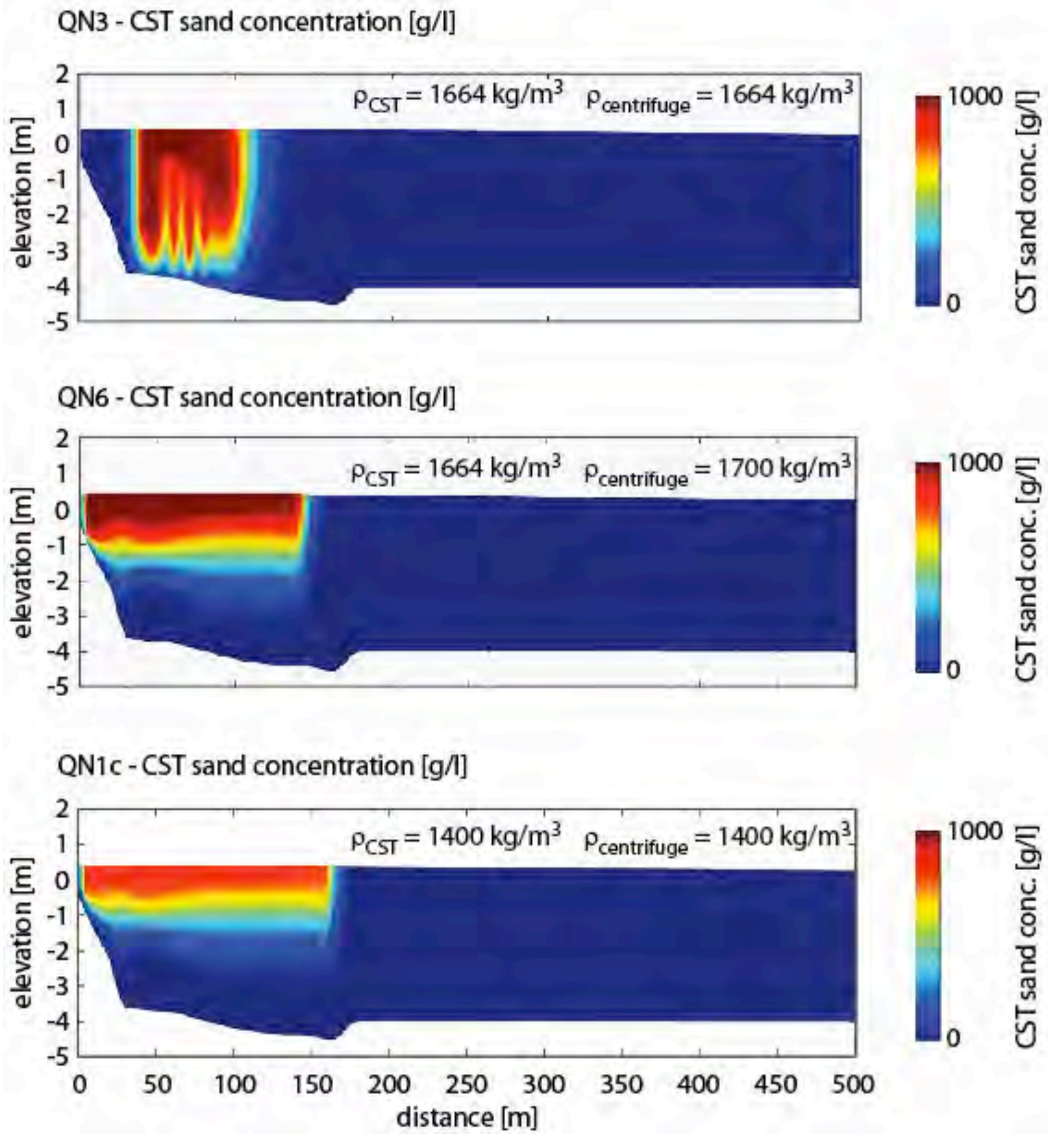

Figure 5. Capping scenarios (QN3, QN6, QN1c)

3 DISCUSSION

The results of the modelling described in this paper highlight several important factors that likely govern the potential success of CST sand mixing into the centrifuge deposit in JPM DDA1, but are likely universal to mixing of CST tailings flows with tailings similar to centrifuge deposit. This section discusses these findings, and identifies data gaps that could be addressed to improve the ability to model, predict, and optimize sand placement into centrifuge or other similar fine tailings deposits.

3.1 *General modelling implications*

The overall mixing/capping patterns predicted by the model are governed by the relative bulk densities of the centrifuge deposit and CST. In general, when the CST slurry bulk density exceeds that of the centrifuge deposit, the sand slurry tends to plunge into and underneath the centrifuge cake. When the centrifuge deposit has a bulk density higher than CST, sand tends to mix higher in the centrifuge deposit, in some cases effectively forming a sand cap.

In the situation investigated here, sand capping does not occur in the traditional sense of a sand layer deposited over undisturbed fine tailings (as might be accomplished from a spreading barge). Rather, centrifuge product and CST sand mix in an increasingly restricted thin zone near the surface of the centrifuge deposit as the centrifuge deposit bulk density increases. In contrast, increasingly dense CST slurries lead to progressively thinner mixed zones near the base of the centrifuge deposit. It is recognized that this form of modelling is limited to materials that behave like slurries or fluids. For instance, this model would be inappropriate as the centrifuge deposit solids content increases to the point that the deposit behaves more like a soil and less like a slurry.

Another key finding from the preliminary modelling is that the strength of the centrifuge deposit (primarily a function of solids content) affects how far (longitudinally into the basin) CST sand can mix. The weaker the centrifuge deposit, the easier it is for CST sand to propagate into, and mix with, the treated tailings deposit.

3.2 *Comparisons with field data*

As noted earlier, direct comparison of model results with DDA1 field observations is complicated by certain assumptions made in setting up the simplified modelling approach. The absence of beach formation in the modelling approach precludes simulation of the high solids content CST beach shaded in dark grey on Figure 2.

The overall geometry predicted in the baseline simulations (QN1 and QN2 in Figure 3) compares favorably with the mixing geometry observed in DDA1. Mixing of CST sand into a centrifuge deposit is strongest and thickest near the CST discharge, and tapers in thickness and sand content with distance into the pond. The SFR in the mixed region of the simulations is broadly comparable to, but lower than, that in the mixed regions of the DDA1 deposit (light grey shading on Figure 2).

Though mixing geometry and magnitude are comparable, the baseline simulations overpredict the distance over which mixing propagates into DDA1. The most likely reason for this discrepancy is that three-dimensional spreading of sand cannot be reproduced by the two-dimensional model. However, the simulations are consistent with the overall behaviour that occurred during the DDA1 field trial. The CST slurry plunged into the centrifuge deposit, forming a mixing zone that decreases in thickness and sand content away from the CST discharge point in both the model and field trial. Based on these observations, this modelling approach may be a beneficial way of predicting interaction between CST and centrifuge or similar tailings deposits, and therefore a methodology to evaluate commercial viability of sand mixing technologies. Due to challenges and complexities associated with modelling this behaviour, the model should to be used as a guide for planning, design, exploring likely outcomes, and for comparison of options rather than as an exact predictor of expected field conditions.

3.3 *Data gaps*

The preliminary modelling presented in this paper is only a first step towards a modelling approach that could become a robust tool for predicting hypothetical or planned situations. A number of gaps remain in our understanding of CST/centrifuge mixing that could be addressed numerically and/or via laboratory experimentation to improve the applicability of the model. Important data that may be beneficial to creating a more robust model are:

1. Inclusion of depositional processes, particularly the formation of CST beach deposits that dominate not only the geometry of mixing in DDA1, but significantly affect the mass-balance of sand throughout the mixing zone.
2. Inclusion of post-depositional processes, particularly the early stages of dewatering during particle settling, that affect the long-term character of the CST/centrifuge mix.
3. Observations of CST/centrifuge mixing and capping when CST and centrifuge bulk densities are comparable, or when centrifuge bulk density exceeds that of CST. The model simulations of these cases are physically reasonable, but there are no laboratory or field data to corroborate the predictions.

4 SUMMARY AND RECOMMENDATIONS

This preliminary study identified a modelling approach suited to simulation of mixing between a CST slurry and centrifuge deposit. The Delft3D-slurry modelling results were generally consistent with mixing observations made after the 2016 trial in JPM DDA1, suggesting that this may be a valid approach, though it would benefit from inclusion of additional capabilities. This could become a valuable tool for predicting and designing tailings mitigation and capping strategies.

4.1 *Recommendations for successful sand mixing*

Mixing of CST sand into centrifuge deposit is primarily controlled by the relative bulk densities of the CST slurry and centrifuge deposit. When the CST slurry bulk density exceeds that of the centrifuge deposit, CST is likely to plunge, forming a sand beach and bottom-hugging mixed zone that tapers with distance from the discharge point. In order for CST sand mixing to be longitudinally more extensive, a weaker centrifuge deposit should be targeted.

4.2 *Recommendations for successful capping*

Capping of centrifuge deposit with CST sand was shown to be viable when centrifuge deposit bulk density sufficiently exceeds CST slurry density. The sand cap is thinner and denser as density contrast increases. This can occur when:

1. Centrifuge deposit consolidation proceeds to the point that solids content (and bulk density) are sufficiently high i.e. greater than the 65% solids content);
2. CST bulk density is reduced by using lighter-than-silica particles; or
3. Theoretically, CST bulk density is reduced by decreasing the silica sand CST solids content (a more dilute sand slurry). This case was not simulated for this project.

4.3 *Recommendations for technology development*

The preliminary modelling presented in this report does not include several processes that play an important role in CST/centrifuge mixing patterns in a setting like DDA1. Foremost among these are bed formation (deposition) and post-depositional dewatering. Delft3D already includes numerical formulations for these processes, though they have only been used, to date, for conventional Newtonian flows. Ongoing COSIA funding is helping to develop these, and other, components of Delft3D-slurry.

A modest laboratory experimental program, such as small-scale mixing tests over a range of CST and centrifuge bulk densities should be evaluated. Accordingly, the central conclusion of the preliminary modelling work is that relative bulk density exerts first-order control on mixing dynamics. This conclusion has only been validated against the August 2016 field data, so additional tests across a broader relative range of bulk densities than is possible in field trials would allow verification and improved understanding of this behaviour.

Additional work is needed to understand the constraints on the thickness of CST that can be placed onto the centrifuge material, as well as other practical cap deployment considerations.

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Homogenization approach to simplify consolidation analysis of a layered deposit

S. Masala & S. Proskin
Thurber Engineering, Calgary, AB, Canada

ABSTRACT: A realistic analysis of consolidation of a tailings deposit may include numerous changes in filling rates (FR) and materials properties, which can easily exceed the input data limitations of the software used. The input variability should be narrowed down to make the analysis tractable without sacrificing the numerical accuracy while preserving the essential physics. The homogenization approach provides an approximate solution for a typical problem of a one-dimensional large strain consolidation analysis of a regularly layered deposit of repeating layers of two different tailings materials with arbitrary thicknesses. First, an "equivalent" continuous deposition schedule with a uniform FR, which preserves the total deposited dry solids mass of the layered deposit, is evaluated. Second, the "equivalent" consolidation properties for the homogenized deposit are developed that preserve the layered deposit consolidation behaviour "on average". The homogenization procedure was found sensitive to the contrast in material properties of the layers.

1 INTRODUCTION

1.1 *Why homogenization in consolidation analysis of tailings deposits*

Deposition of mine wastes into their storage areas is a process that can take decades, with filling schedules with numerous changes in filing rates and with pauses and interruptions, planned and unintended. Occasionally, multiple tailings ponds are simultaneously active and multiple pipelines are introduced into the storage area or removed from it. Such ponds may be filled with variable tailings types or materials of different properties, resulting from changes in the extraction / mineral processing or tailings treatment activities. Even with a relatively regular filling schedule, a large pond size and its long lifetime, as is the case with most oil sands tailings storage areas, will cause these changes to multiply excessively.

Performing a direct simulation of an entire deposit may become very complicated and impractical even when it is relatively straightforward. A complicated input extends the time for data preparation and makes difficult finding errors in it. The input data can easily exceed the limitations of the software used. It is possible and likely that numerical process itself may encounter problems with excessive discretization requirements related to accuracy, or due to computation process instability. Such an analysis may simply become computationally expensive because of high memory storage requirements or excessive computation time, especially with non-linear problems.

In all such cases, the problem theoretical definition and implementation, primarily the input variability, should be narrowed down to make the analysis tractable without sacrificing the numerical accuracy while preserving the essential physics.

1.2 *Homogenization as a physical problem*

The layered tailings deposits can possibly be best described as the composite materials, where the word "composite" means that two or more materials are combined to form the third one (Jones, 1999). When a composite manufacturing process is designed, the purpose is to improve the properties of the constituents or create some properties that neither of the component materials possesses. When composites results from natural processes, the purpose of their investigation is to develop understanding of their behaviour and approximate and simplify prediction of their performance, typically considering it as a single material at the macroscale.

One component is usually dominant in the composite and it is called matrix; the other components are usually called inclusions. A common classification of composites includes: (a) fibrous composites, with fibers in a matrix; (b) laminates, which consist of layers of various materials; and (c) particulate composites, with particles distributed in a matrix, with an irregular or a regular pattern (structure). Of course, combinations of the above three types are possible.

1.3 *Homogenization as a mathematical problem*

The homogenization methods are based on abstraction of the constituent materials properties (which are, as a rule, inhomogeneous and non-isotropic) and the actual structure of the composite, typically replacing it in the analysis with an "equivalent" homogenous and isotropic body, so that macroscopic behaviour of heterogeneous materials can be effectively and efficiently predicted. The material properties of the composite at a macroscale are determined from the material properties of the constituents and the internal structure of the composite at the microscale.

Homogenization methods can be classified as analytical, semi-analytical and computational.

Rigorous analytical methods exist for general analyses of composites, or mixtures, based on their description by the partial differential equations (PDE) of relevant physical processes, with corresponding initial and boundary conditions. For example, Bakhvalov et al. (1988) provide a treatise on the one-dimensional diffusion equation (a parabolic PDE), mathematically equivalent to the small strain consolidation theory by Terzaghi. Solutions are provided for mathematically very complicated cases with spatially variable coefficients and parametrically-dependent coefficients. Nevertheless, the large strain consolidation theory by Gibson et al. (1967, 1981), applied to a layered deposit in a tailings pond, is even more complex mathematically, with the coefficients – the compressibility and hydraulic conductivity functions – discontinuous and variable both in space and time, and with moving domain boundaries and variable boundary conditions. There is no analytical solution for such a problem.

The methods presented in the paper had therefore to be based on experience and useful analogies, and solutions obtained using numerical methods with available software. They are developed for two specific cases, without "verification" and a rigorous analysis of numerical stability and error margin, and identification of a strict area of applicability. The methods have been applied as "engineering tools" to two evaluation examples and showed to work well with certain ranges of input parameters.

2 HOMOGENIZATION FOR LARGE STRAIN CONSOLIDATION ANALYSIS OF A LAYERED DEPOSIT

2.1 *Definition of two illustrative homogenization examples*

The two demonstration examples for the homogenization approach to large strain consolidation analyses are hypothetical and based on authors' actual project work for the oil sands operators in the Fort McMurray area in Alberta and, although generalized, are still realistic and illustrative.

2.2 *Example 1: Discontinuous filling schedule*

The first example is intended to illustrate the effect of homogenization of a filling schedule on the consolidation behaviour by comparing the following two cases:

- An "actual" discontinuous deposition schedule "2+2 weeks", with 2 weeks of deposition followed by 2 weeks for quiescent consolidation; and

– A modified schedule with continuous deposition with a uniform FR that is 50% of the FR in the “actual” deposition schedule, to make the total masses of deposited dry solids equal in both cases.

The motivation for this example was found in difficulties to strictly simulate the design filling schedules with a large number of changes in filling rates; for example, with deposition events lasting a few days or weeks, followed by several days or weeks of quiescent consolidation, over a pond life time of a decade or two. The number of events (changes) in the filling schedule input section can accumulate to several hundreds, which is far beyond the input limitations for the applicable software.

The deposit was assumed homogeneous (the same material in both analyzed cases) to isolate the effect of deposition schedule. The pond shape was adopted as a cylindrical or prismatic, with the constant pond area - to avoid possible pond geometry influence.

The material used in the analysis was a typical oil sands fine tailings with SFR_{44}=0÷0.2 from the Thurber Engineering Ltd. (Thurber) tailings data base, see section 2.5 and Figures 1 and 2.

2.3 *Example 2: Layered deposit and equivalent homogeneous deposit*

The second example build on Example 1 by introducing deposition of another material with different properties during periods of quiescent consolidation in Example 1. It consists of the following two cases:

– A stratified deposit with the “2+2 weeks” deposition schedule (2 weeks of deposition of material 1, then 2 weeks of deposition of material 2), and
– A homogenized deposit with a modified - continuous - deposition schedule with a FR preserving the same total amount of dry solids as in the 2+2 weeks deposition schedule with individual layers of materials 1 and 2.

The procedures for determination of the equivalent consolidation properties for a substitute homogeneous deposit are described in Appendices A and B. The hydraulic conductivity of a homogenized material was calculated using a well-known layered model for seepage flow across the layers (or the model of heat conduction through a layered panel as a theoretically equivalent problem). The equivalent compressibility function was determined summing the settlements of the two layers in a sandwich (calculated using the individual compressibility functions of the layers) and making the sum equal to the settlement of a homogenized material of the same thickness as the two-layer sandwich.

The equivalent compressibility and hydraulic conductivity functions of the homogenized material are also functions of stress and void ratio. The equivalent properties of a homogenized stratified deposit depend not only on the individual material properties of the layers but also on the deposit configuration, i.e. the layer thickness ratio.

The materials used in this analysis correspond to two types of fine oil sands tailings from Thurber’s data base: one with SFR_{44}=0.7÷0.9 and the other, used in Example 1, with SFR_{44}=0÷0.2. The pond shape was again adopted a cylinder or a prism, with a constant area.

2.4 *Theoretical model and software*

The theoretical model was based on the 1D large strain consolidation (LSC) theory developed by Gibson et al. (1967, 1981). The software used for numerical analyses was FSConsol by GWP Software Inc., version 3.48, a common tool for LSC simulations in the oil sands industry. The “Pond Analysis” option was used to simulate a mine tailings pond scenario in which a slurry is gradually deposited into a settling pond, with user-specified material properties, pond geometry, filling rates, and boundary conditions.

It should be noticed that all 1D consolidation analyses suffer from the “reduced dimensionality” problem and ignore the physics of tailings flow, over beach above water or subaqueously. These can significantly alter consolidation behaviour of deposited tailings. Depending on its state, flow over dry sandy beach can produce various outcomes: tailings stream can be densified due to particle settling in shear flow and water absorption by sand or, on the contrary, the flow can result in segregation, if the slurry is shear-thinning, with the fines washed-out of tailings stream with released water. Additional segregation may occur when the slurry enters the ponded water. These processes are not included in any 1D model.

2.5 *Material properties*

The material properties required for a large strain consolidation analysis are the compressibility, which defines void ratio as a function of vertical effective stress, and the hydraulic conductivity, which is expressed as a function of void ratio. The two functions are inter-related which allows their simultaneous determination from test data or, as in this case of a composite material, from the material properties of the constituents.

The tailings data used to derive the consolidation properties are the public domain part of the oil sands tailings laboratory consolidation testing data base developed at Thurber. Using the actual test results as input for simulations gives practical significance to the outcome of presented analysis.

As usual in the LSC analyses, the compressibility and hydraulic conductivity are mathematically expressed as non-linear functions of void ratio, in this case the power laws (equations 1).

$$e = A\sigma'^{B} \quad k = Ce^{D} \tag{1}$$

The material properties adopted for this analysis are average values for several tailings types from the database, classified by their fines contents on the basis of 44 microns FC_{44}, or a more common in the oil sands industry sand-to-fines ratio SFR_{44}. The consolidation functions for defined SFR ranges are plotted in Figures 1 and 2; their parameters A to D are listed in Table 1.

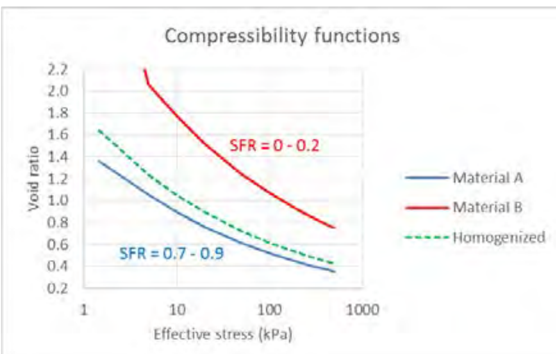

Figure 1: Compressibility, Thurber database.

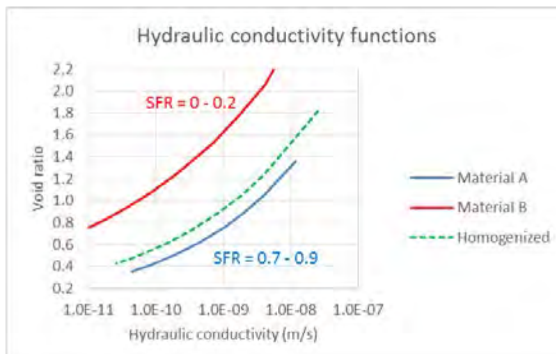

Figure 2: Hydraulic conductivity, Thurber database.

Table 1. Material properties for analysis.

SFR	Compressibility A (kPa^{-1})	Compressibility B	Hydraulic conductivity C (m/s)	Hydraulic conductivity D	Specific gravity Gs	Initial SC (%)
0÷0.2	2.939	-0.220	$5.58*10^{-11}$	5.973	2.50	35
0.7÷0.9	1.547	-0.238	$3.33*10^{-9}$	4.144	2.50	55

2.6 *Boundary conditions*

The boundary conditions were kept as an impermeable pond bottom and a permeable deposit top with the phreatic surface coincident with the tailing top elevation; therefore, the deposit was fully saturated throughout analysis. The top boundary condition assumes efficient surface drainage of consolidation released water; however, a surface water layer above tailings does not affect deposit consolidation and only changes the total pore pressure distribution over depth.

3 EXAMPLE 1 RESULTS - DISCONTINUOUS VERSUS CONTINUOUS DEPOSITION SCHEDULE

The deposition time had to be limited to 3 years because of the FSConsol input limitation for the number of FRs to maximum 80. The total number of FR changes for the 2+2 weeks schedule was 2 x 12 x 3 = 72, just below the maximum allowed number by the software.

The results of this comparative analysis are presented in Figures 3 to 10. Figures 3 to 8 present the time plots of tailings elevation (deposit height), the average SC and the average degree of excess pore pressure dissipation (DPPD). Figure 4, 6 and 8 are zoomed on the deposition period, the first 3 years. Figures 9 and 10 show the excess pore pressure profiles during deposition at 1 and 2 years.

The figures show the "visual equivalence" of the discontinuous (original) and continuous (modified) pond filling schedules in the post-deposition period, with the differences during deposition best visible in the zoomed deposition periods, particularly DPPD in Figure 8. The oscillatory nature of the average DPPD for the 2+2 weeks schedule is consonant with the periods of deposition and quiescent consolidation. The oscillations quickly attenuate with the increase of the deposit thickness and the drainage path length, which effect grows quadratically with deposit elevation (or time). The saw-tooth variation in the layered deposit elevation during deposition (Figure 4) will become larger with increases in FR and hydraulic conductivity and decrease in the duration of a consolidation event.

The pore pressure profiles in Figure 9 are shown for the end of a quiescent consolidation period (360 days) and the end of the subsequent deposition event (375 days). In this example, the two-week consolidation period is enough to dissipate incremental excess pore pressures in both filling schedules – the profiles at 360 days are coincident for the two deposition schedules. The increase in excess pore pressures after a deposition event (375 days) is approximately twice larger for the 2+2 weeks schedule because the 2+2 weeks FR is two times higher. Figure 10 presents the same profiles but at 2 years after the start of deposition. The differences among the profiles are of the same magnitude as in Figure 9, but they are twice smaller relative to the current pore pressures. These differences further diminish with time and their effects is negligible for practical purposes.

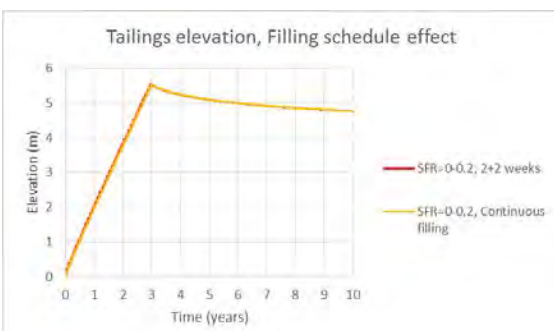

Figure 3. Tailing elevation over time.

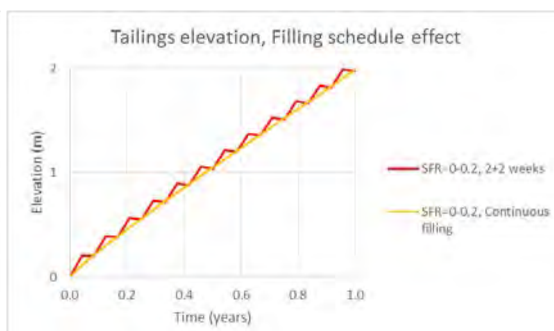

Figure 4. Tailings elevation, first year of deposition.

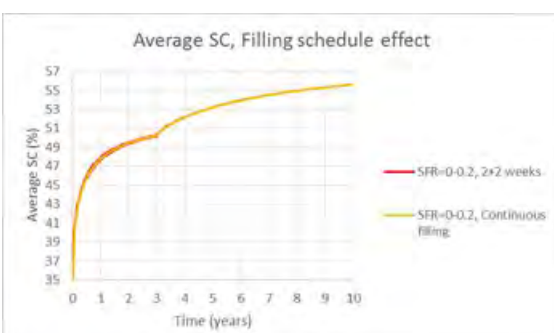

Figure 5. Average SC over time.

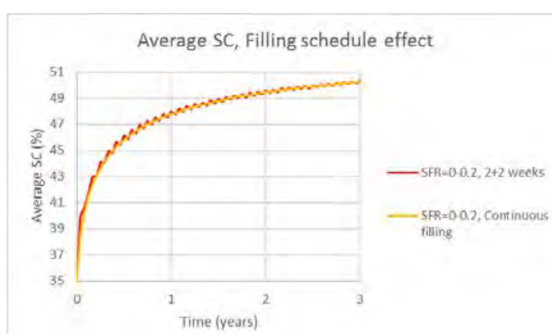

Figure 6. Average SC during deposition.

Figure 7. Average DPPD over time.

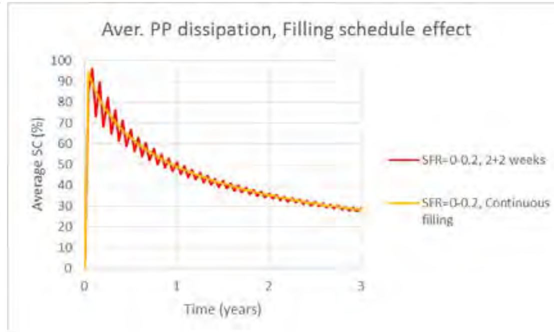

Figure 8. Average DPPD during deposition.

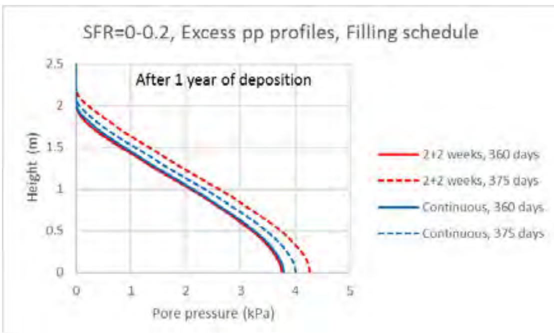

Figure 9. Excess pore pressure profiles at ~1 year.

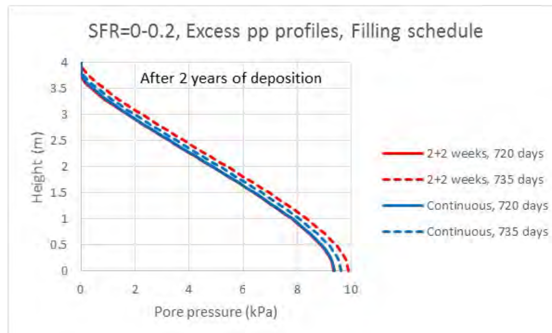

Figure 10. Excess pore pressure profiles at ~2 years.

4 EXAMPLE 2 RESULTS - LAYERED DEPOSIT VERSUS HOMOGENIZED DEPOSIT

The deposition time had to be limited to 1 year because of the FSConsol input limitation of the maximum number of material type changes to 30. The total number of material type changes for the stratified deposit with the “2+2 weeks” schedule was 2 x 12 = 24. The simulation time was 5 years.

The results of this comparative analysis are presented in Figures 11 to 18. Figures 11 to 16 present the time plots of tailings elevation (deposit height), average SC and average DPPD. Figure 12, 14 and 16 are zoomed on the deposition period. Figures 17 and 18 show the SC and hydraulic conductivity profiles at the end of deposition (1 year) and 5 years.

Similar to Example 1, the layered and the homogenized deposit exhibit visually identical behavior in the post-deposition period, with the differences during deposition best visible in the average SC and DPPD time plots in Figures 13-14 and 15-16, respectively.

The smoothening effect of increasing deposit height on the average properties of a deposit, characteristic for the layered deposits and discontinuous filling schedules, results from the physical nature of the underlying diffusion process. The differences in consolidation behaviour between the layered and homogenized deposit decrease with the number of layers, i.e. with the decrease of the thickness of RVE (the sandwich of materials 1 and 2) relative to the deposit height.

The above smoothening effect does not apply to internal distributions of material properties, as illustrated by the profiles of solids content and hydraulic conductivity in Figures 17 and 18. These profiles keep the differences in the solids content and hydraulic conductivity between layers 1 and 2 from year 1 to year 5, and will maintain them throughout the consolidation process, even at the fully dissipated state, merely because the two materials follow two different compressibility and hydraulic conductivity curves. An important consequence is that different layers will always have different strengths, dependent on their individual material properties and void ratios, i.e. indirectly excess pore pressures or stress states; the homogenization does not apply to deposit stability.

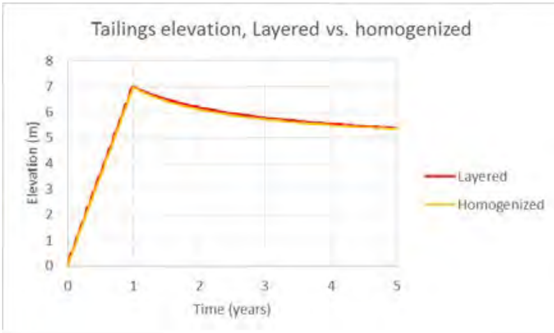

Figure 11. Tailing elevation over time.

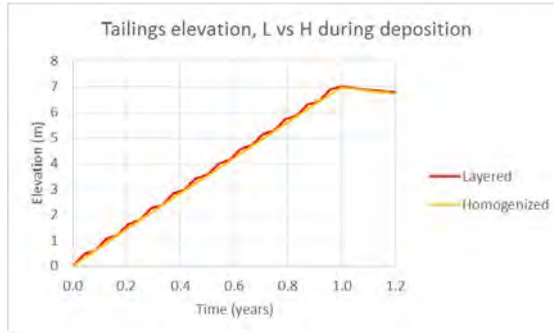

Figure 12. Tailings elevation, first year of deposition.

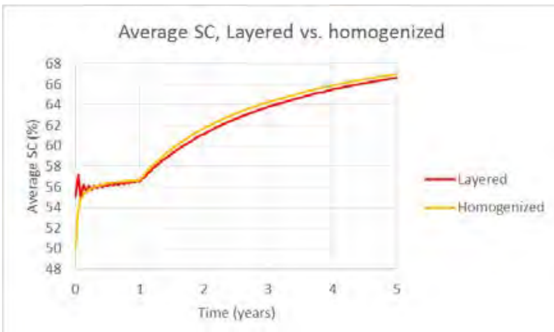

Figure 13. Average SC over time.

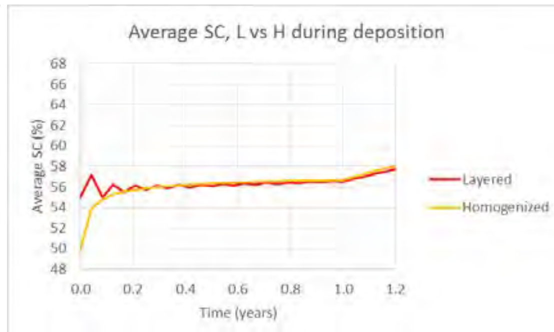

Figure 14. Average SC during deposition.

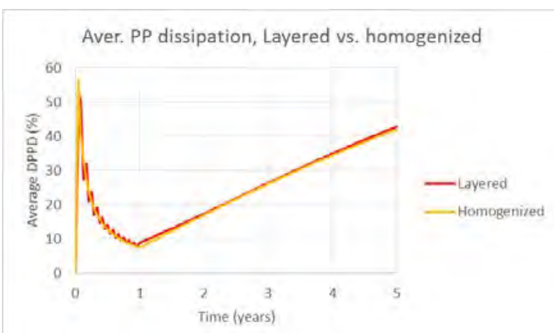

Figure 15. Average DPPD over time.

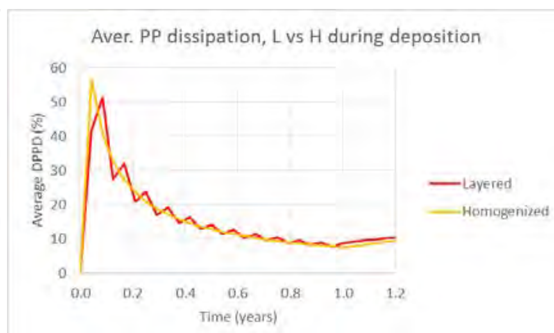

Figure 16. Average DPPD during deposition.

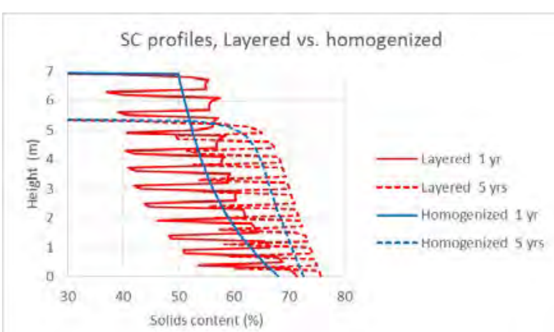

Figure 17. SC profiles at EOF and 5 years.

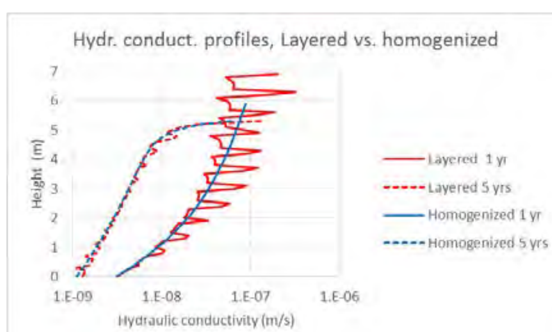

Figure 18. Hydr. cond. profiles at EOF and 5 years.

5 DISCUSSION WITH CONCLUDING REMARKS

Homogenization methods for 1D large strain consolidation analyses were presented that allow replacing discontinuous filling schedule with a continuous one, and substituting a layered deposit with a homogeneous one with equivalent consolidation material properties. The methods are illustrated with two examples typical for oil sands tailings ponds and materials.

While the overall layered deposit behavior in consolidation is well captured by homogenization, the specifics of different material properties must be kept in mind when assessing actual distribution of these properties in individual layers. The operational strength in a particular layer depends on its own strength function and stress state of that layer and cannot be homogenized.

The homogenization procedures for material properties were derived for the steady-state conditions. Throughout consolidation, the materials are in a transitional state and the derivations are only approximately valid. For example, the effective stress in the middle of a layer was used in derivation, but the stress state is variable across the layer and, moreover, the stress state evolves in different ways in different layers, as they consolidate with different rates following their own materials properties. Therefore, the results of this exercise should not be generalized without checking each specific case of different layer configurations and material properties, similar to what was presented here.

In the examples presented in the paper the consolidation behaviour of a layered and the "equivalent" homogenized deposit were remarkably well matched. This will not be the case if the contrast in the layer properties is further increased. The problem can be analyzed at a more general level by systematically varying the normalized material properties of individual layers in the RVE; this is the subject of ongoing analyses.

A particularly useful feature of homogenization is that it offers a quick and straightforward insight into the relative effects of influential parameters and helps concentrate the analysis on the most important of them.

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APPENDIX A: DERIVATION OF COMPRESSIBILITY OF HOMOGENIZED MATERIAL

The homogenization of a layered deposit was based on the assumptions of a regular deposit structure – the individual layer thicknesses remain uniform throughout deposition – and identification of a representative unit of the structure which is "small enough" compared to the size of the structure (deposit).

A representative structural unit or the representative volume element (RVE) for a two-layer deposit was adopted as a sandwich consisting of contiguous layers of materials A (matrix) and B (inclusions), as in Figure 19.

The first step consists of derivation of equivalent compressibility of the homogenized material under self-weight of the RVE - the A+B sandwich - itself. The initial thicknesses of the layers H_0^A and H_0^B are determined from: the initial solids content SC_0, the specific gravity of solids G_s and the filling rate FR expressed in dry solids mass, a explained below. The initial layer thicknesses are nominal values - before settlement due to self-weight consolidation.

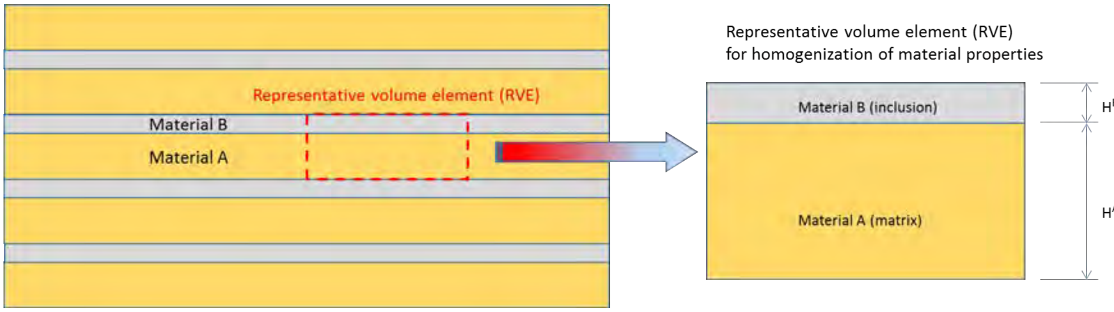


Figure 19. Representative volume element (RVE) for a two-layer deposit.

For each layer, the initial void ratio e_0, the total and dry unit weights ρ_0 and ρ_{0dry} and the masses of solids M_s^A and M_s^B are first calculated. The superscripts A and B are omitted in the formulae below.

$$e_0 = \frac{1\text{-}SC_0}{SC_0} G_s \tag{3}$$

$$\rho_0 = \frac{G_s + e_0}{1+e_0} \rho_w \tag{4}$$

$$\rho_{0dry} = SC_0 \cdot \rho_0 \tag{5}$$

$$M_s = \rho_{0dry} \cdot H_0 \tag{6}$$

The total and effective stresses due to material self-weight are then calculated for the middle of layers A and B and for the middle of RVE. The superscripts A and B denote layers, while the subscript 0 denotes the initial state. Layer B is assumed above layer A (Figure 14).

$$\sigma_0^B = \rho_0^B \cdot \frac{1}{2} H_0^B \tag{7}$$

$$\sigma'^B_0 = \sigma_0^B - \rho_w \cdot \frac{1}{2} H_0^B \tag{8}$$

$$\sigma_0^A = \rho_0^B \cdot H_0^B + \rho_0^A \cdot \frac{1}{2} H_0^A \tag{9}$$

$$\sigma'^A_0 = \sigma_0^B - \rho_w \cdot \left(H_0^B + \frac{1}{2} H_0^A\right) \tag{10}$$

$$\sigma_0^{A+B} = \rho_0^B \cdot H_0^B + \rho_0^A \cdot \frac{H_0^A - H_0^B}{2} \tag{11}$$

$$\sigma'^{A+B}_0 = \sigma_0^B - \rho_w \cdot \frac{H_0^A - H_0^B}{2} \tag{12}$$

The mid-layer stresses and their respective compressibility functions are used to calculate the sum of the settlements of layers A and B under self-weight: s_0^A and s_0^B. For example, for layer A:

$$e_1^A = A^* \left(\sigma'^A_0\right)^{B^*} \tag{13}$$

$$s_0^A = \Delta H_0^A = \frac{H_0^A}{1+e_0^A} \cdot \Delta e_0^A = \frac{H_0^A}{1+e_0^A} \left(e_0^A - e_1^A\right) \tag{14}$$

where A^* and B^* are the material parameters of the compressibility function for layer A.

The void ratio for the homogenized sandwich of layers A+B is computed using the sum of settlements of layers A and B and the average specific gravity for the RVE G_s^{A+B}.

$$G_S^{A+B} = \frac{1}{\frac{M_S^A}{M_S^{A+B}} \cdot \frac{1}{G_S^A} + \frac{M_S^B}{M_S^{A+B}} \cdot \frac{1}{G_S^B}} \tag{15}$$

$$SC_0^{A+B} = \frac{M_S^{A+B}}{H_0^{A+B}} \tag{16}$$

$$e_0^{A+B} = \frac{1-SC_0^{A+B}}{SC_0^{A+B}} G_S^{A+B} \tag{17}$$

The pair σ'^{A+B}_0 - e_0^{A+B} is the first point of the compressibility function for the homogenized material. The next step is to increment the effective stress and calculate a matching void ratio repeating the previous two steps: calculate settlement of layers A and B and their new thicknesses, then sum the individual layer settlements, make the sum equal to the settlement of the homogenized material, and finally calculate the corresponding void ratio for the homogenized material using formulae 13 and 14. The process continues with subsequent stress increments up to the desired maximum stress value.

The properties of a homogenized material depend on the properties of the component materials, the thickness ratio of individual layers (the RVE geometry) and the deposit size. The homogenization approximation works better when the contrast in the material properties of the layers is small and the layer thicknesses similar. The approximation error also declines with decrease of the RVE size (thickness) relative to the size (thickness) of the deposit.

The presented scheme does not adequately describe the physics of individual deposition of layers A and B at the deposit surface, when each material is deposited separately and the average stress and void ratio for the sandwich do not make sense. Therefore, the homogenized deposit response will not match well the actual – layered – deposit response near the deposit surface and, in general, when the stresses are small.

The homogenized deposit response will converge to the original layered deposit behaviour with increasing deposit thickness since the material properties of various tailings, both compressibility and hydraulic conductivity, converge with increasing stress.

APPENDIX B: DERIVATION OF HYDRAULIC CONDUCTIVITY OF HOMOGENIZED MATERIAL

Hydraulic conductivity of the composite A+B can be calculated using a textbook formula for the seepage flow across the layers in a layered material. Validity of Darcy's flow is assumed. Formula 18 was derived by applying a unit hydraulic gradient across the contiguous layers A and B and a homogenized material of the same thickness, then making equal the flow rates in the two cases.

$$\frac{H^A+H^B}{k^{A+B}} = \frac{H^A}{k^A} + \frac{H^B}{k^B} \tag{18}$$

This formula is applied to every step of the calculation of the compressibility function for homogenized material as the hydraulic conductivities of layers A and B are functions of void ratio, i.e. indirectly of effective stress. The void ratio values e^{A+B} are calculated during determination of the compressibility function of the homogenized material. The thicknesses of layers A and B are also functions of stress and are determined during calculation of the compressibility function for the homogenized material.

It should be noticed that the equivalent hydraulic conductivity k^{A+B} for the composite A+B depends on the configuration of the deposit, i.e. depends on the thickness ratio of layers A and B. The validity of equation 18 depends on:

- the ratio of the RVE thickness and the deposit thickness (the smaller, the better) and
- the ratio of the hydraulic conductivities of layers A and B (again the smaller, the better, since the effective stresses in the two contiguous layers will be close to each other).

Modeling of naphthenic acids transport and fate from oil sands tailing ponds

L. Ma, C. Huang & Z. Liu
National Research Council Canada, Vancouver, BC, Canada

ABSTRACT: The toxicity of the fluid wastes from oil sands tailings ponds has been traced to the complex mixture of organic acids believed to be naphthenic acids (NAs), which have both acute and chronic toxicity to aquatic organisms. In this study, a Multiphysics modeling methodology was developed to simulate physical and chemical interactions of NAs in oil sands tailings ponds and to predict how oil sands process-affected water (OSPW) seepages through dykes and foundation layers. Specifically, NAs' diffusion, dispersion, convection and bio-degradation are investigated respectively and coupled with the study of unsaturated OSPW flow in the subsurface layer. The proposed methodology was implemented in commercial software COMSOL for the prediction of NAs' transport and fate in both spatial and temporal scales. Furthermore, the tailing pond of the Muskeg River Mine in the Athabasca oil sands deposit was selected in the following case study. The comparison of seepage and NAs monitoring data to the predicted results from calculation showed good agreement. This study is supported by the project under the Program for Energy Research and Development.

1 INTRODUCTION

The province of Alberta, Canada has a long history for the exploration of oil sands. It is reported that some experimental and pioneer extraction work started as early as 1940s and commercial productions began in the beginning of 1970s. The Athabasca oil sands deposit located in the north of Fort McMurray has been found to be the largest oil sands reservoir in Alberta with an estimated 1.37 trillion barrels deposit (Woynillowicz 2005). The production of crude oil requires significant volumes of water, which brings a lot of water contamination concerns, for both surface water and ground water, during hydrocarbon recovery, transport, or as a legacy from reclaimed areas.

It is widely reported that the waste water from the oil sands tailings ponds is toxic. The toxicity has been traced to those chemicals added in the oil sands extraction processing, and also complex mixture of organic acids exists in the bitumen (believed to be naphthenic acids), which has both acute and chronic toxicity to aquatic organisms (Mackinnon 1986). In this study, naphthenic acids (NAs) are considered to be the main pollutant in oil sands process-affected water (OSPW) through literature survey (Grewera 2010).

As NAs are found to be chronically toxic to aquatic organisms, oil sands industry are not permitted by regulator to actively discharge the OSPW to natural environment. The tailings pond waters have typical NAs concentrations in the range of 40 to 120 mg/L (Schramm 2000). Apart from the increasing concentrations of the NAs in the tailings ponds, the volume of fine tailings is also accumulating at a rate of about 0.1 to 0.2 m^3 per ton of oil sands processed. It is estimated that if processes continue at the current rate, over 1 billion m^3 of tailings pond water will require reclamation by 2025 (Quagraine 2005), and the reclamation process requires possibly over hundreds of years. Thus the OSPW is accumulating in oil sands tailings ponds, which persistently attracts public concerns and may cause potential environment issues in future. The development and use of models for the prediction of NAs' transport and fate are critical to oil sands industry.

2 METHODOLOGY

The basic principles and methods used in this study are shown in Figure 1. Details about the mechanisms used in this study and the corresponding control mathematical equations will be given and explained in this section.

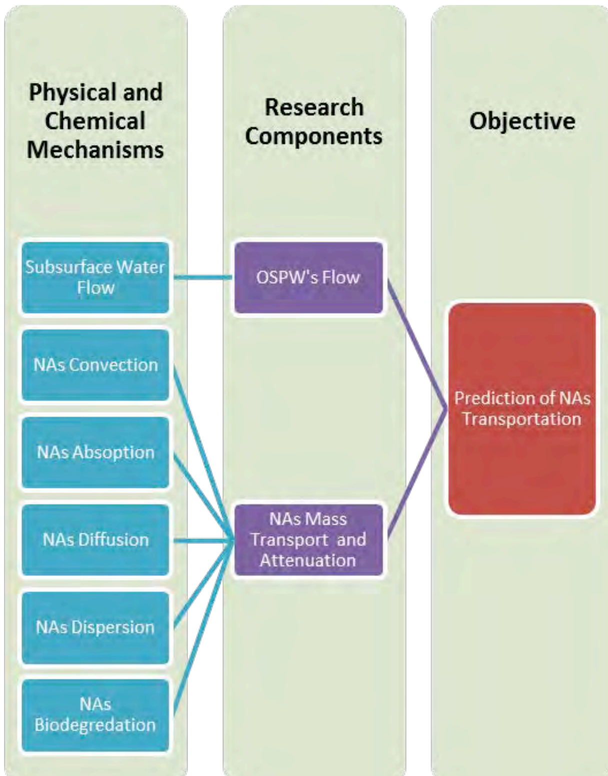


Figure 1. Principles and methods

Regarding the OSPW's seepage in the foundation of the tailing pond, the classic Darcy's law is adopted to calculate the water flow velocity under saturated conditions:

$$\bar{\mathbf{u}} = -K(\nabla p - \rho\bar{\mathbf{g}}) = -K\nabla H \tag{1}$$

where K and H denote hydraulic conductivity and hydraulic head, respectively. The following Richard's equation is usually to investigate the scenarios with unsaturated subsurface flow.

$$\frac{\partial}{\partial t}(\rho\theta) - \bar{\nabla}\cdot\left(K\left(\bar{\nabla}p - \rho\bar{\mathbf{g}}\right)\right) = 0 \tag{2}$$

It is observed that Richards' equation appears notoriously nonlinear, as the water content θ and hydraulic conductivity K vary for unsaturated conditions (negative pressure) and reach a constant value at saturated conditions (pressure of zero or above). The van Genuchten retention model reveals how the water content θ and hydraulic conductivity K relate to the fluid pressure p:

$$\theta[p] = \theta_r + \frac{\theta_s - \theta_r}{\left(1 + (\alpha p)^n\right)^m} \tag{3}$$

$$K[p]=K_s\left(\frac{\theta(p)-\theta_r}{\theta_s-\theta_r}\right)^{0.5}\left(1-\left(1-\left(\frac{\theta(p)-\theta_r}{\theta_s-\theta_r}\right)^{1/m}\right)^m\right)^2 \tag{4}$$

Then, Darcy's law was applied to calculate the subsurface water velocity for unsaturated conditions,

$$\bar{\mathbf{u}}=-K[p](\nabla p-\rho\bar{\mathbf{g}})=-K[p]\nabla H \tag{5}$$

Furthermore, Richard's equation/Darcy law and mass transport equation (diffusion, dispersion, absorption, convention and degradation) are coupled in the studied domains to investigate the release of NAs with OSPW (William 1998):

$$(\varepsilon+\rho_b k_d)\frac{\partial c}{\partial t}+\bar{\nabla}\cdot\left(-(D_d+D_e)\bar{\nabla}c+\bar{\mathbf{u}}c\right)=\mathrm{R} \tag{6}$$

where ε is the porosity, ρ_{b} is the bulk density, k_{d} is the absorption coefficient, D_{d} and D_{e} are the diffusion coefficient and dispersion coefficient respectively, $\bar{\mathbf{u}}$ is the flow velocity obtained from Richard's equation/Darcy law. In addition, R is the source term such as potential reaction rate due to NAs degradation. It is common to use the half-life of first order reaction to evaluate the bio-degradation rate of NAs (Toor 2013, Hana 2009),

$$\mathrm{R}=c\times\frac{\ln(2)}{t_{1/2}} \tag{7}$$

$t_{1/2}$ is the estimated half-life of NAs' degradation.

3 CASE STUDY FOR THE MUSKEG RIVER MINE

3.1 *Case introduction*

The tailing infrastructure of the Muskeg River Mine that was started in Dec 2002 by Albian Sands Energy Inc. was adopted in this case study. All of the monitoring data used for comparison and analysis come from open sources.

The Muskeg River Mine is located between the Athabasca River and the Muskeg River. An open pit mine is situated in the northern part of the mine site and the tailing pond is located in the south of the mine site near the confluence of the two rivers. In this study, a selected small area at the southern edge of the tailing pond is investigated. The oil sands tailing pond at the Muskeg River Mine is enclosed by a ring tailing dyke, which was constructed by the tailing sand produced from bitumen extraction. In the beginning of the dyke building, an "overburden starter dyke" was constructed on the original ground with relatively impermeable materials called lean oil sands. With the water level in the tailing pond rises, the dyke was raised and expanded over the starter dyke by using relatively coarse tailing sand. In the Muskeg River Mine, it is reported that the Pleistocene sand (Pf-sand) and associated larger grain size granular are widely presented over the source of oil sands - McMurray formation. In addition, many peat layers usually can be found over fluvial sand and gravel deposit in the Athabasca mine area. However, most of the peat layers in the Muskeg River Mine were removed before mining operation and only a small portion is left on site.

To provide deeper understandings in how NAs release in the tailing pond of the Muskeg River Mine, the simulation of the seepage of OSPW with NAs' transportation has been performed. Specifically, mechanisms of NAs' diffusion, dispersion, convection and bio-degradation are investigated respectively while the OSPW flow in the subsurface layer is also studied. In the end, the calculated OSPW's seepage profile is integrated with the simulated NAs' mass transport and attenuation to predict the how NAs discharge into the surrounding environment. The modeling tool developed based on the commercial software COMSOL is used in this case study to construct these fully coupled tailing pond models.

In general, the geometry configuration of the tailing pond of the Muskeg River Mine is shown in Figure 2. The tailing pond locates at the top right corner of the figure. The domain in yellow denotes the body of the dyke which is made by tailing sands. ASE's data from exploratory boreholes revealed that some of the boreholes struck thick (<3m) Pf-sand. For simplification, a Pf-sand layer with 3m thickness (2m in wet area) is assumed on the top of the McMurray Formation foundation in the model. In addition, the starter dyke and the peat soil layer with 3m thickness has also been included in this study.

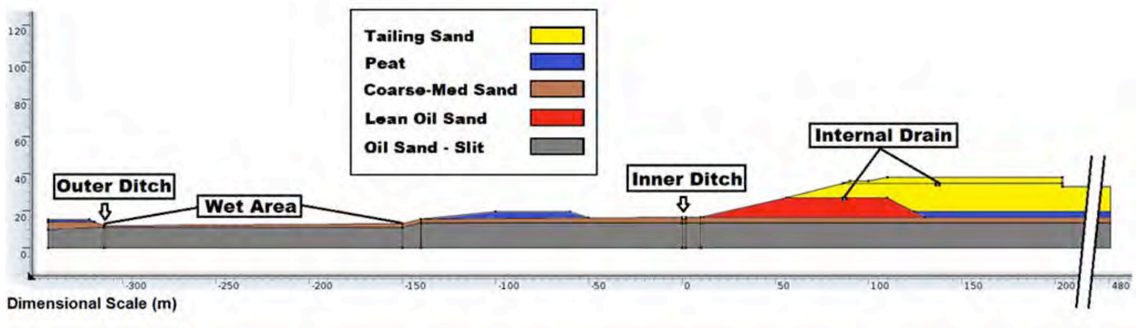


Figure 2. Geometry configuration of the tailing pond of the Muskeg River Mine.

3.2 *Hydrogeology modeling*

All physical properties of geologic materials and dyke materials were estimated by a limited number of laboratory experiments and the typical value reported by ASE. Monitoring data related to hydrogeology used in this study mainly come from the previous work did by Yasuda (2006). In his study, the hydraulic modeling was simply calculated by separated 1D and 2D domains using measured hydraulic heads for all boundary conditions. While in our study, we integrated the studied section as a whole in 2D manner and apply the measured hydraulic heads only for the boundary conditions of two ending edges. The calculated hydraulic head profile within the domain is used to compare with those measured data for model validation.

The hydraulic conductivity for each studied layer is estimated by monitoring data given in Table 1. The anisotropy of hydraulic conductivity is considered in the model. Due to lack of water retention properties, nonlinearity of permeability is neglected and the Darcy's law is simply applied to calculate the groundwater flow. In addition, the bottom surfaces of the McMurray Formation layer are defined as no flux for both of OSPW flow and NAs transport. Based on the hydraulic head profiles obtained from the paper of Yasuda (2006), it is observed the precipitation seems to have very limited impact on the groundwater, thus no recharge from precipitation is considered in the modeling. According to the measurement shown in Table 2, the hydraulic head boundary conditions on the bottom of tailing pond and the model left edge are set to 299.77m and 277.3m, respectively.

The approximate average flow rate measured at the outtake drain pipe was estimated to be 0.6L/s, which is mainly contributed by seepage water collected by internal drains. It is reported that the drain pipe covers ~150m wide section of the dyke, so the discharge rate per meter length on the internal drains is estimated to be 4e-3 L/s/m. The approximate average flow rate in the outer ditch is estimated to be 50 L/s. According to previous study, much of this flow come from the dyke drain collected from inner ditch and it is considered that only a small portion (<5%) of the total flow rate in the outer ditch comes from the discharge of the wet area. As a ~250m wide section of the wet area is observed to contribute to the seepage into the outer ditch, the discharge rate per meter length on the surface of wet area is estimated to be <0.01 L/s/m. In addition, the discussion of the groundwater flow in our model is assumed to be in steady state for simplification without considering any seasonal fluctuation.

Table 1. Hydraulic conductivity for each layer

No.	Domain	Materials	K_h(m/s)	Anisotropy(K_v/K_h)
1	Tailing Dyke	Tailing Sand	2e-5	0.1
2	Peat Layer/Peat Soil Heap	Peat	9e-6	0.5
3	Pf-sand	Coarse-Med Sand	2e-4	1
4	Overburden Starter Dyke	Lean Oil Sands	1e-8	0.5
5	McMurray Formation	Oil Sands - Silt	2e-8	0.1

A parametric study on water discharge rates is performed to calibrate the proposed subsurface water flow model. The model is calibrated by perturbing the water discharge rate on the bottom of inner ditch as currently no techniques is able to measure this rate. According to the simulation of Yasuda (2006), the discharge rate on the bottom of the inner ditch is estimated about 4e-3 L/s/m. In our study, the perturbation of this discharge rate remains within a reasonable interval, which ranges in a single order of magnitude from the initial values (from 0.1x to 10x). In addition, the water discharge rate into the wetting area is also very difficult to measure or estimate onsite. It is mentioned above that less than 5% of the total water flow rate in the outer ditch is estimated to be the discharge rate contributed by wet area in previous study. Through our preliminary calculation, it is found that 5% of the total water flow rate is a little high, which is not able to generate a hydraulic head profile to match the onsite measured one. As a result, we also perturbed the water discharge rate into wet area from 1% to 5% of total flow rate into the outer ditch in our parametric study.

Based on parametric study, the scenario with 4e-2 L/s/m and 4% of total flow shows the most similar pattern with what we found from the monitoring data. The contour of calculated hydraulic head is plotted in Figure 3. The comparison between the measured hydraulic heads and simulated one at several monitoring locations is given in Table 2, which shows good match.

Table 2. Comparison of measured and calculated water head

Locations	Measured Head (m)	Calculated Head (m)
Tailing Pond (B.C.)	299.77	299.77
Inner Ditch North (Cross Section)	High 280.23, Low 279.98	280.63
Inner Ditch South (Cross Section)	High 280.08, Low 279.83	280.42
Wet Area North (Cross Section)	High 277.61, Low 277.58	277.50
Outer Ditch North (Cross Section)	High 276.95, Low 276.87	276.95
Outer Ditch South (B.C)	High 277.44, Low 277.19	277.3

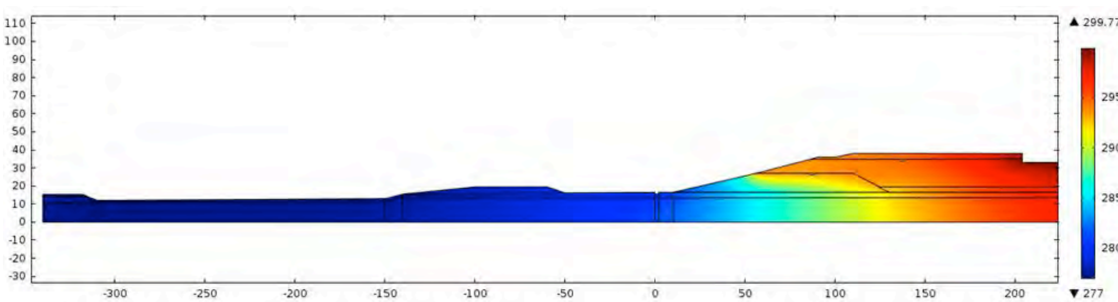


Figure 3. Contour of calculated hydraulic head

3.3 *NAs' transport modeling*

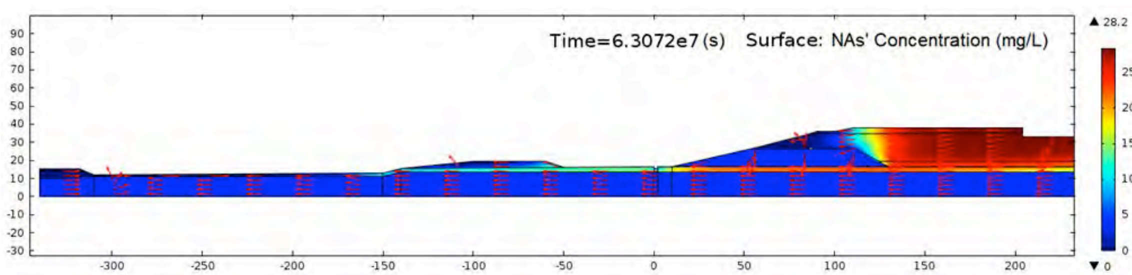
To further investigate the NAs transportation with OSPW flow and their fate, the profile of water flow velocity calculated from hydrogeology modeling will be imported to the convection term presented in Equation 6. The inflow NAs concentration at the bottom of tailing pond is set to 29mg/L according to the monitoring data shown in Table 3 and the model left edge is set to open boundary condition for NAs transportation. It is assumed that the NAs have a uniform diffusivity 5×10^{-6} cm^2/s in the water phase. In addition, the NAs are considered to be biodegraded in the pond and the half-life is set to be 3.9×10^{4} days (Wang 2013). The initial NAs concentration in the McMurray Formation layer and starter dyke is defined as 4.4mg/L, which is considered to represent the maximum contribution from the lean oil sands as background according to onsite observations. Regarding the absorption term, it is found that the NAs absorption coefficient k_d may significantly correlated to the amount of bitumen doped on the sands. k_d may range from 0-10 ml/g depending on the residual bitumen content (Wang 2013). In this study, two separated parametric studies (from 0.1 to 10 ml/g) are performed on the absorption coefficients for the pf sand layer (low bitumen content) and also for the body of the dyke (high bitumen content).

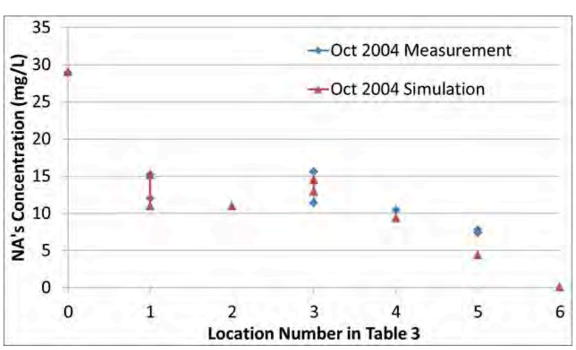
Table 3. Measurement of NAs concentration

No.	Locations	Oct, 2004 NAs (mg/L)	Jun, 2005 NAs (mg/L)	May, 2006 NAs (mg/L)
0	Tailing Pond	28.9	29.3	15.5
1	Inner Ditch North			
	Deep	15.2	14.7	-
	Mid	12.5	14.3	-
	Shallow	12.0	21.2	-
2	Inner Ditch Water	-	11.6	-
3	Inner Ditch South			
	Deep	15.6	14.2	-
	Mid	15.5	13.8	-
	Shallow	11.4	13.4	-
4	Wet Area North	10.4	11.3	-
5	Wet Area			
	Measurement 1	7.4	-	9.7
	Measurement 2	7.8	-	10.3
6	Outer Ditch South			
	Deep	-	<1.0	-
	Mid	-	<1.0	1.2
	Shallow	-	1.7	-

Due to the high heterogeneity of the dyke materials, the effect of dispersion is also considered in our models. The dispersion coefficient usually can be expressed as dispersity times flow rate. The study of Wang (2013) shows that the NAs dispersity for a similar sand material is 90cm, which is based on a 10cm lab column testing. It should be mentioned that the dispersity in the field is different from the values measured in lab. It is commonly observed that the dispersity is proportional to the scale of the geometry. As there is no measurement for the dispersity of those layers in the Muskeg river mine, here another parametric study on the NAs dispersity perturbed based on Wang's measurement (from 0.1x to 10x) is performed. Consequently, two NAs absorption coefficients for different layers and also the NAs dispersity are swept simultaneously to calibrate NAs' transport modeling in our study.

Through comparing the simulation results from parametric sweeping, it is determined that the absorption coefficient 0.1mg/L for the pf sand layer, 0.2mg/L for the body of the dyke, and also 0.1x for dispersity which means dispersity 9cm based on 10cm column testing shows the best matching with the NAs' monitoring data in the field.

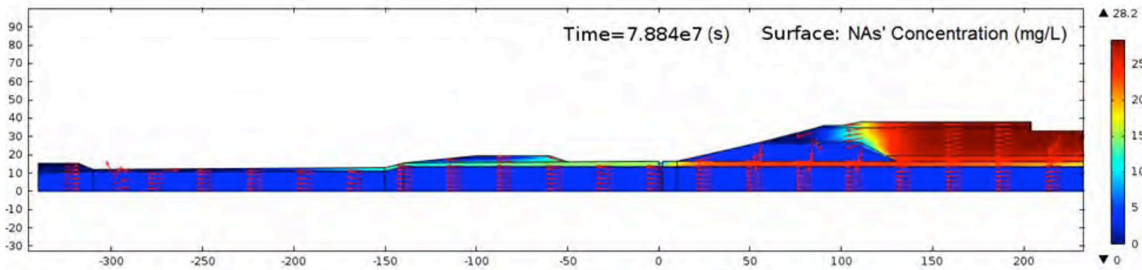


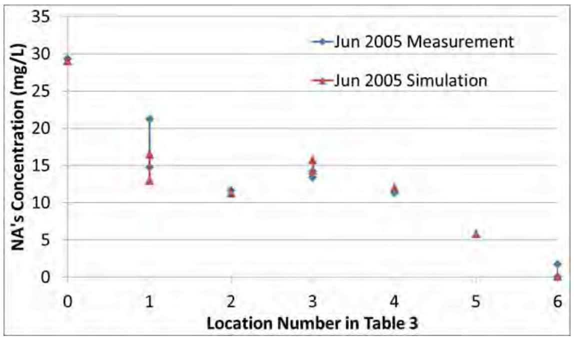


(a) Top: Simulated concentration and transport direction

(b) Left: Comparison of measurement and simulation

Figure 4. NAs concentration in Oct 2004

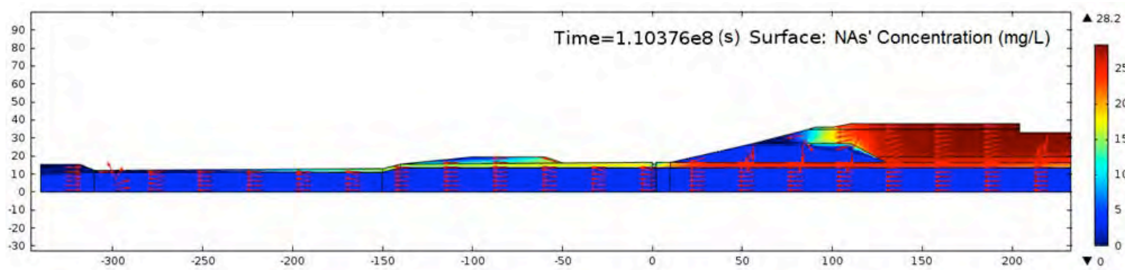


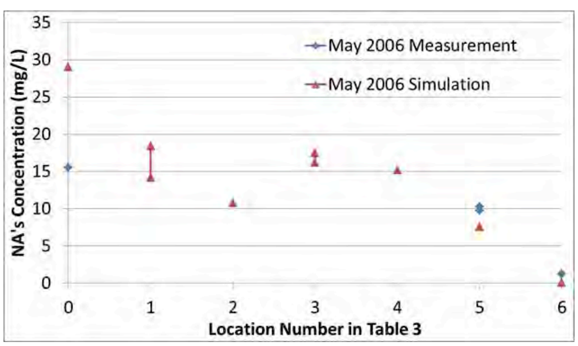


(a) Top: Simulated concentration and transport direction

(b) Left: Comparison of measurement and simulation

Figure 5. NAs concentration in June 2005





(a) Top: Simulated concentration and transport direction

(b) Left: Comparison of measurement and simulation

Figure 6. NAs concentration in May 2006

The simulated NAs contour and NAs transport direction with time evolutions from the best matching scenario are plotted in Figure 4-6. In addition, comparisons between the simulation results with the monitoring data are given. The reason for the NAs concentration drop in the tailing pond in May 2016 is unclear. Generally, a good agreement is shown between the modeling and the measurements. For the monitoring location far away from the dyke (wet area and outer ditch south), the simulation concentration is below the measured value. The reasonable explanation is that the operation stage with only the starter dyke is not simulated in our study, which may make the NAs seepage further and faster in the field than the scenarios in simulation.

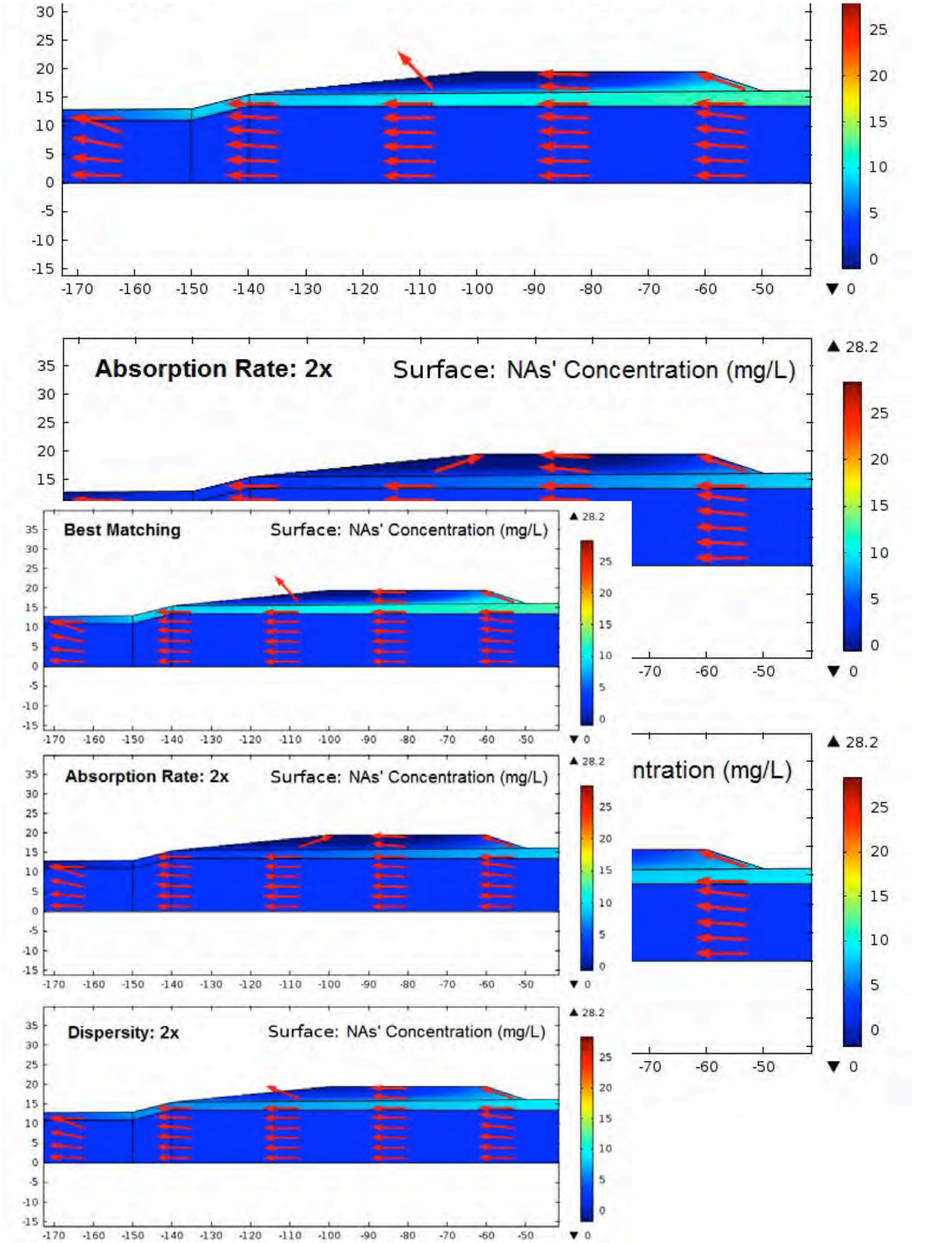


Figure 7. Impact of absorption rate and dispersity

In conclusion, the simulation results in this study reveal that the NAs' transport under the tailing pond of the Muskeg River Mine is mainly along with the OSPW seepage flow in the pf-sand layer. However, the simulations illustrate that NAs should not be able to escape this tailing collection system due to the negative direction of subsurface flow around the outer ditch region, which results from its hydraulic management design.

4 SUMMARY

In this paper, a Multiphysics modeling methodology was developed to predict OSPW seepages in field and also to simulate physical and chemical interactions of NAs with the flow of OSPW.

A case study on the tailing pond at the Muskeg River Mine is conducted to validate and refine the developed models. The simulation results show good agreement with the onsite monitoring data in terms of hydraulic heads and NAs concentration profiles. Moreover, the observation from our modeling results also reveals the potential pathway for NAs' seepage and confirms the validity of its hydraulic design to prevent NAs release to environment. In the future, the developed NAs reactive transport model coupled with hydraulic simulation can be applied to investigate other contaminants fates, if the corresponding interaction properties are available.

ACKNOWLEDGMENT

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Development of a numerical model for the prediction of tailings deposition and fines capture efficiency

B. Lavoie, Z. Bouazza, L. Piciacchia & J.A. Vides
BBA Inc., Mine Waste Management and Environment, Montréal, QC, Canada

ABSTRACT: Hydraulic deposition in tailings storage facilities is often used in Canada. These tailings are generally composed of fine and coarse particle that deposit on the beach above water (BAW). Some of these fine particles, however, are not captured and may end in the beach below water (BBW), transforming into mature fines tailings which causes a number of problems for pond reclamation and security. Among these problems are the very long time of fines consolidation that can be more than 30 years and the presence of chemicals in the remaining water that becomes toxic to fish or birds. As an illustration of the magnitude of the problem, the total surface of oil sands tailings ponds in Alberta was estimated to be 176 square kilometers in 2015 (Steward 2015). The present paper propose a numerical model for the prediction of fines capture based on the coupling of a 2D numerical model and a capture model proposed by Sisson et al. (2012). The intent is to illustrate the possible benefits of optimizing beach lengths to promote fines capture. This is emphasized by the results of the model.

1 INTRODUCTION

A number of current tailings storage facilities in Canada use conventional sub-aerial hydraulic deposition where tailings streams are sent to embankment facilities. The resulting impoundment is composed by perimeters dykes, deposition beaches, water retention dykes and water pond. The tailings contain fine and coarse particles that deposit during the flow along the beach above water (BAW). An unneglectable quantity of fines is however transported to the beach below water (BBW) and eventually transforms into mature fines tailings (MFT). Reducing the amount of fines deposited in the water ponds could reduce environmental and geotechnical risks during operations, improve soil strength in ponds and facilitate pond reclamation. The aim of this work is to develop a numerical model to simulate tailings flow as well as fines capture. The model is built by coupling a two-dimensional model for non-newtonian flows, FLO-2D, with a fines capture prediction method elaborated by Sisson et al. (2012). This results in a model that is two-dimensional for the flow component and one-dimensional for the deposition/capture component. A brief literature survey is presented, followed by the methodology, an application of the method and a conclusion.

2 LITERATURE REVIEW

2.1 *Beach Slope Prediction Models*

Substantial work has been done on the prediction of beach slopes and interesting methods are available. These methods give no information about the fines deposition and capture but may be useful for geometric analysis.

Fitton developed a method that is based on equilibrium between erosion and deposition on the beach (Simms et al. 2011). This equilibrium leads to the maximum slope which is characterized by a critical velocity. Once the critical velocity is known, the Colebrook-White equation can be used to calculate the friction coefficient and the final slope is given by the Darcy-Weissbach equation.

The stream Power Method was elaborated by McPhail and relates the beach slope to the energy dissipation; i.e., the beach slope is considered equal to the power curve. An equation of power along the beach is therefore given and differentiated in order to provide the power curve for the whole beach length. Beach elevations may then be approximated by solving the slope equation (Simms et al. 2011).

The lubrication theory is a simplification of the Navier-Stokes equations with the following assumptions: the flow depth and length ratio is small and the viscous and gravitational forces are more important than the inertial force. The simplified momentum equation is then combined to the hydrostatic pressure to give the permanent profile of the fluid (Bingham fluid with viscosity lower than yield stress) (Simms et al. 2011).

2.2 *Pore Capture Model*

The pore capture model, elaborated by C. Marsh states that the segregated sand that deposits on the bed has a certain ratio of voids that are completely filled with the carrier fluid which is a mixture of fines, water and oil. The oil component is relatively small and may be neglected. If the volumetric ratios between voids and solids as well as voids and sand are known, one can compute the ratio between sand and fines in the beach deposit (SFR_D).This in turn may be used to calculate the fine capture which is defined with respect to the slurry SFR (SFRs) (AMEC Environment and Infrastructure 2013).

2.3 *Barr-Deltares Model*

Sisson et al. (2012) have provided an analytical model that allows the prediction of flow parameters as well as evolution of sediment concentration and fines capture. This model can be separated in five main components that respectively investigate flow, sand settling, concentration, morphology and deposition.

The flow, which is assumed to be laminar and at equilibrium, follows a Ostwald-de-Waele law or power law. The sand settling is calculated using a stokes type equation which is modified to consider the hindered settling due to high sediment concentration. Mass balance of settling solids and the sand continuity equations are used to compute the sediment sand concentration and sedimentation rate. This method states that deposited sand will entrap the carrier fluid which contains fines. The volume of captured fines can then be derived from the known porosity and volume of settled sand.

This method is of great interest because it addresses both flow characteristics and fines deposition. However, it neglects the 2D or 3D effects as well as the evolution of the beach geometry.

2.4 *Numerical Models*

Computational fluid dynamics has also been applied to this kind of problems. Yang (2009) used ANSYS CFX to study the oil sands slurry flows and the deposition of fine and coarse particles. Spelay (2007) elaborated a numerical model to solve Navier-Stokes equations for non-newtonian fluids in one dimension with a finite volume method. Babaoglu (2011) used the open source smoothed particle hydrodynamics (SPH) code SPHYSICS to model the flow of high density tailings but did not address the evolution of concentration or fines deposition.

A few open channel hydraulic software may be used to model non-newtonian flows; FLO-2D (2009) and RiverFlow-2D (Hydronia 2017) have this capability but do not allow for simultaneous computation of sediment deposition. Hansen (2016), Van Es (2017) and Deltares have been working on simulation of fines capture with Delft-3D software but this option of Delft-3D is not made available yet.

2.5 *Proposed Refinement*

The above mentioned models are all interesting but none of them is completely satisfying due to the high complexity of the physics of tailings deposition. The Barr-Deltares model is very interesting but neglects the two-dimensional effects while FLO-2D consider two-dimensional flows but lacks the study of concentration and fines deposition. This paper proposes a refinement to the Barr-Deltares model in order to take into account the two-dimensional flow by coupling its concentration and morphological components to a two-dimensional non-newtonian flow model, FLO-2D.

3 METHODOLOGY

The principle behind this methodology is to take advantage of the sand settling, concentration, morphology and deposition capabilities of the Barr-Deltares model while replacing its flow component with a two-dimensional model, FLO-2D. The coupling of these two models has the following advantages:

- Two-dimensional flow;
- Prediction of sediment concentration at multiple sections along the beach;
- Prediction of Fines Capture;
- Takes in account the topography and evolution of bed elevation; and
- A general gain of precision.

The model applies to laminar flows of non-segregating tailings (NST). This section presents the methodology in three steps; flow prediction, fines capture prediction and coupling process.

3.1 *Flow Prediction – FLO-2D*

FlO-2D is a two-dimensional hydrologic and hydraulic model capable of simulating water as well as hyperconcentrated sediment flows. The model uses an explicit finite differences scheme to solve the shallow water equations. The continuity and momentum equations are solve independently in eight directions on a cartesian grid:

$$\frac{\partial h}{\partial t} + \frac{\partial hV}{\partial x} = 0 \tag{1}$$

$$S_f = S_0 - \frac{\partial h}{\partial x} - \frac{V}{g} \cdot \frac{\partial V}{\partial x} - \frac{1}{g} \cdot \frac{\partial V}{\partial x} \tag{2}$$

Where, S_f is the friction slope, S_0 the bed slope, h the flow depth, V the velocity, g the gravitational acceleration, x the position and t the time.

To account for Non-Newtonian Fluids, FLO-2D uses a special friction slope that considers a yield slope (S_y), a viscous slope (S_v) and a turbulent dispersive slope (S_{td}):

$$S_f = S_y + S_v + S_{td} = \frac{\tau_y}{\gamma_m \cdot h} + \frac{K\mu V}{8 \cdot \gamma_m \cdot h^2} + \frac{n_{td}^2 \cdot V^2}{h^{4/3}} \tag{3}$$

Where, τ_y is the yield stress, γ_m the specific weight of the sediment mixture, K a resistance parameter, μ the viscosity and n_{td} an equivalent Manning for the turbulent and shear stress components.

3.2 *Fines Capture Prediction – Adapted from Barr-Deltares Model*

This method was presented by Sisson et al. (2012), it has been modified in the sense that the flow component is not used in the current procedure and the equation of mass balance settling solids (6) neglects the vertical variation of concentration, which was not the case in the initial method.

The settling velocity of sand (w_0) can be determined with a stokes type equation:

$$w_0 = \frac{1}{18}\frac{(\rho_s - \rho_f)g \cdot d^2}{\mu} \tag{4}$$

This is then modified for hindered settling (w_s) according to the Richardson and Zaki theory (1954):

$$w_s = w_0 \cdot (1 - k \cdot C)^{\beta} \tag{5}$$

Where, ρ_s is the solids density, ρ_f the carrier fluid density, d the solids diameter, k an empirical constant, β an empirical exponent and C the volumetric concentration of sand in the flow.

The evolution of concentration is given by the mass balance of settling solids (Yang 2011):

$$\frac{dC}{dt} = -\frac{Pw_s C}{h} \tag{6}$$

Where P is a parameter that represents the probability for deposition, for the purpose of the present work, it is assumed that deposition is independent of the shear stress (P=1). Therefore, equation 6 can be integrated to give:

$$C = C_0 - \frac{w_s C_0 \Delta t}{h} \tag{7}$$

Where, C_0 is the initial concentration entering the beach and Δt the travel time from the deposition point to the studied section. The sedimentation rate is derived from the sand continuity equation of the suspension (Sisson et al. 2012):

$$v_{sed} = \frac{\partial C}{\partial x}\frac{q}{C - C_{bed}} \tag{8}$$

Where, v_{sed} is the sedimentation rate, C_{bed} the bed solids concentration and q the flow per unit width. Once the sedimentation rate is known one can use the porosity of settled sand to calculate the volume of carrier fluid that is entrapped and, using the ratio between mass of fines and mass of carrier fluid (fines and water), the volume of captured fines. The settled sand porosity has to be measured with laboratory experiments prior to calculations.

3.3 *Coupling Procedure*

The coupling procedure consists in doing multiple time iterations, in all of which the FLO-2D model is executed, its results are read and sent to the adapted Barr-Deltares Model whose results are finally processed to be reintegrated into FLO-2D. These operations are all controlled by a MATLAB script that is in charge to call the models, process their results and ensure a consistent formatting of input files. The different steps of the coupling process are described hereafter in the order as they are encountered within one iteration (see figure 1).

Prior to iterations, a FLO-2D model has to be created; this includes the digital terrain model (DTM), the incoming discharge and its concentration in sediments, the roughness coefficient of the beach and an outlet boundary condition. Also, a few cross sections have to be selected along the beach, these sections are made of a series of FLO-2D cells where the depth and velocity are averaged and are used for sand settling and deposition calculations (Fig. 1). This model will remain exactly the same during all the procedure with the exception of the DTM elevations that will be modified at the end of each iteration following the calculation of sediment deposition.

Steps for one iteration:

1. The FLO-2D calculations are launched which provides the velocity and flow depth at all nodes of the grid;
2. The averaged flow depth and velocity are calculated at previously defined cross sections;
3. The settling velocity (4) and hindered settling velocity (5) are calculated for each cross section.
4. The concentration is calculated for each cross section (7).
5. The volume deposition (8) and fines capture are calculated for each cross section

6. The bed elevations are modified in the DTM in function of the sedimentation rate of each cross section (8).

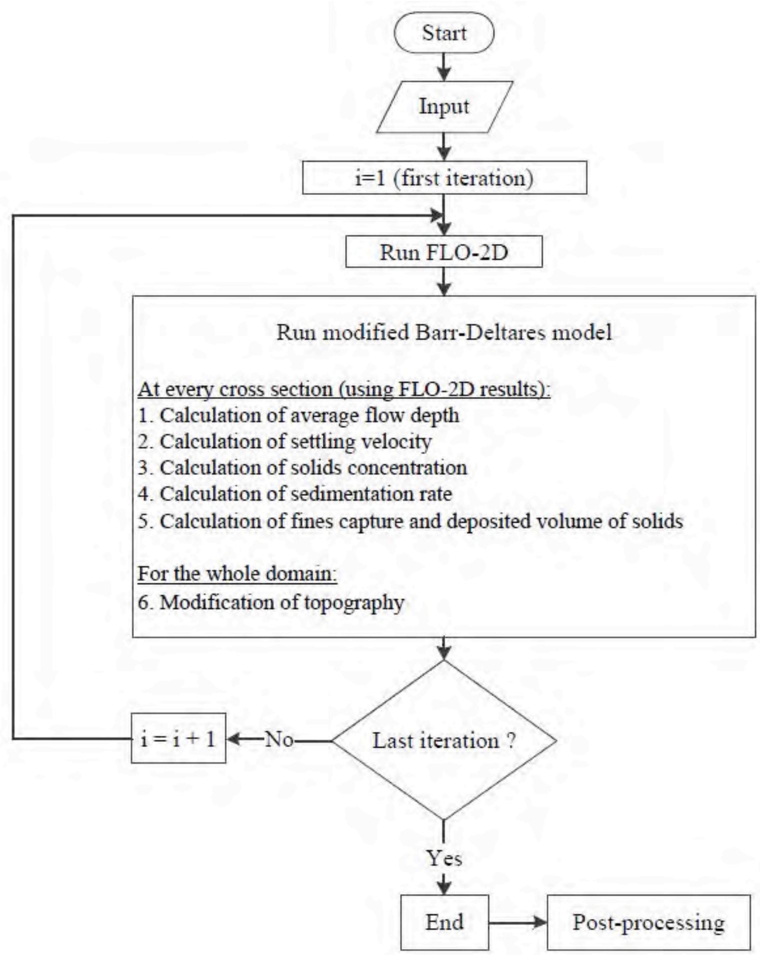

Figure 1. Simplified flow chart of the coupling process of FLO-2D and Barr-Deltares deposition model

4 APPLICATION AND RESULTS

The model is run on a simple theoretical case to show the capabilities of the method and study its behaviour; this does not include any experimental validation.

The idealized case is composed of a beach with a constant slope and deposition at its top. The beach has a width of 588 meters and a length of 996 meters and the flow, deposition and concentration are studied all along the progression of the wave front over the beach. Sections where deposition is studied are positioned every 68 meters for a total of 16 sections, from the very top of the beach to its downstream end. In FLO-2D, the upstream boundary condition is a constant total discharge (water and solids) of 0.5 m^3/s and a free outflow condition which approximates normal flow is applied downstream.

All parameters are kept constant during the simulation. As seen in the *Methodology* section, the model uses many parameters which determination would require experimental tests in a real case. Such data was not used for the present article; therefore the parameters were chosen to be representative of a normal deposition situation. These are presented in table 1.

An iteration of the whole coupled model has duration of 360 seconds (6 minutes) which should not be mistaken has the FLO-2D time step which range from 1 to 5 seconds, i.e., every iteration of the coupled model includes several iterations of FLO-2D. This is to ensure some balance between computation time and precision.

Table 1. Parameters used in the test-case simulation

Parameter	Symbol	Value	Unit
Total discharge	Q	0.5	m^3/s
Cell area (FLO-2D mesh)	A_c	4	m^2
Empirical constant for HS*	k	1	[-]
Empirical exponent for HS*	β	4.65	[-]
Slurry density	ρ_s	1717	kg/m^3
Carrier fluid (water + fines) density	ρ_f	1165	kg/m^3
Mean sand diameter	d	2.20e-04	m
Vol. solids concentration	C_0	0.43	[-]
Empirical constant for viscosity (Eq. x)	α	0.00283	[-]
Empirical exponent	$\beta_{viscosity}$	23	[-]
Vol. solids concentration in the bed	C_{bed}	0.558	[-]
Bed porosity	Θ	0.442	[-]
Sand over fine mass ratio	SFR	5.9	[-]
Fine over fine plus water mass ratio	FOFW	0.23	[-]

*HS: Hindered Settling

The importance of beach length is emphasized in figure 2 where the wave front is presented at different times of the simulation with corresponding concentration and fines capture rate.

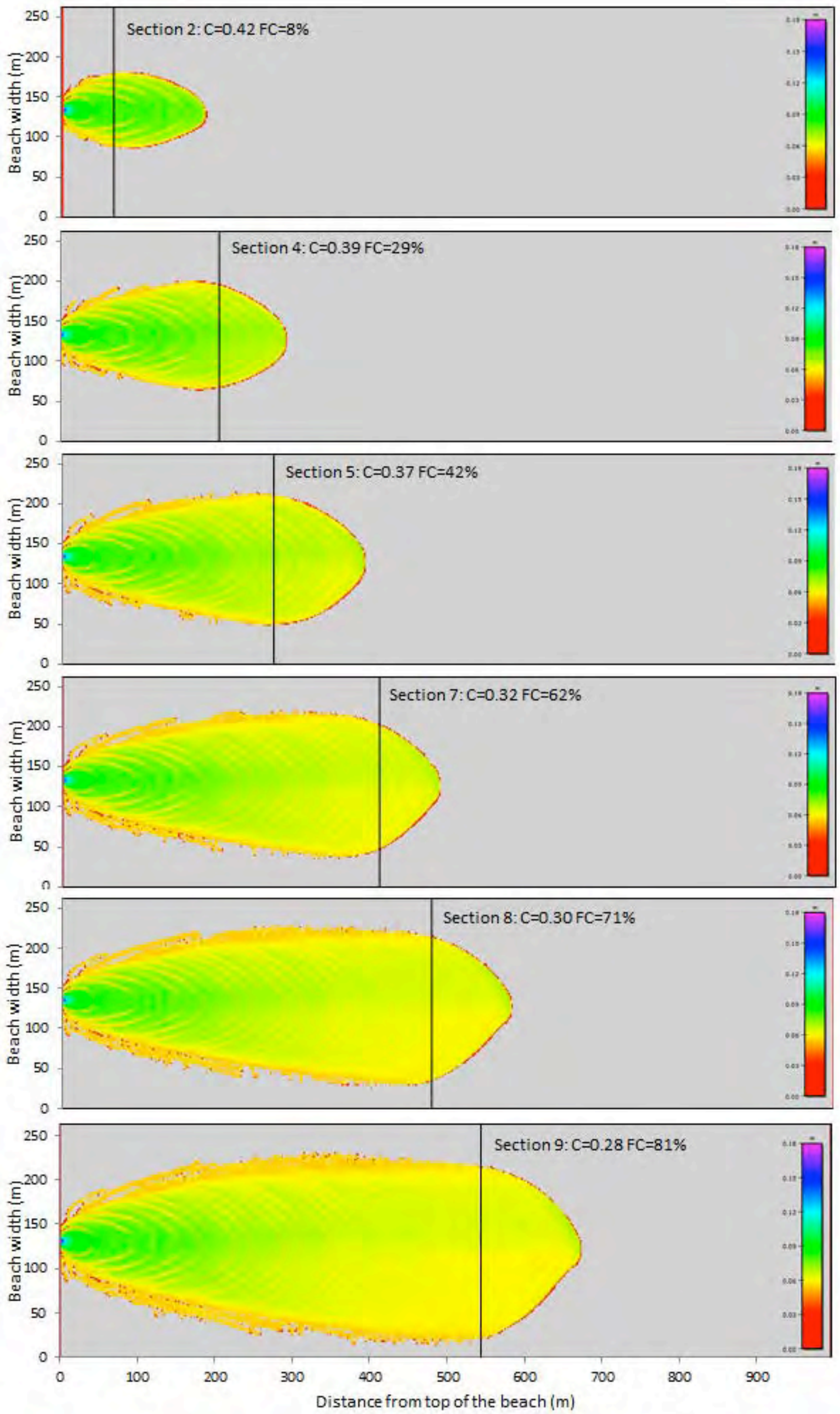

Figure 2. Progression of wave front, concentration (C) and fines capture (FC) over the beach at different time. From top to bottom: t=30 min, 60 min, 150 min, 180 min, 210 and 240 min.

Figure 3 shows the evolution of volumetric solids concentration (fine and coarse particles) and fines capture with respect to the distance from top of the beach after 3 hours of simulation. The fines capture is calculated as the mass proportion of initial fines in the slurry that are deposited on the beach.

The fines capture seems to be increasing faster than the concentration reduction. Also, the beach length has a clear impact on the fines capture especially between 100 m and 300 m where the gradient of the fines capture curve is higher.

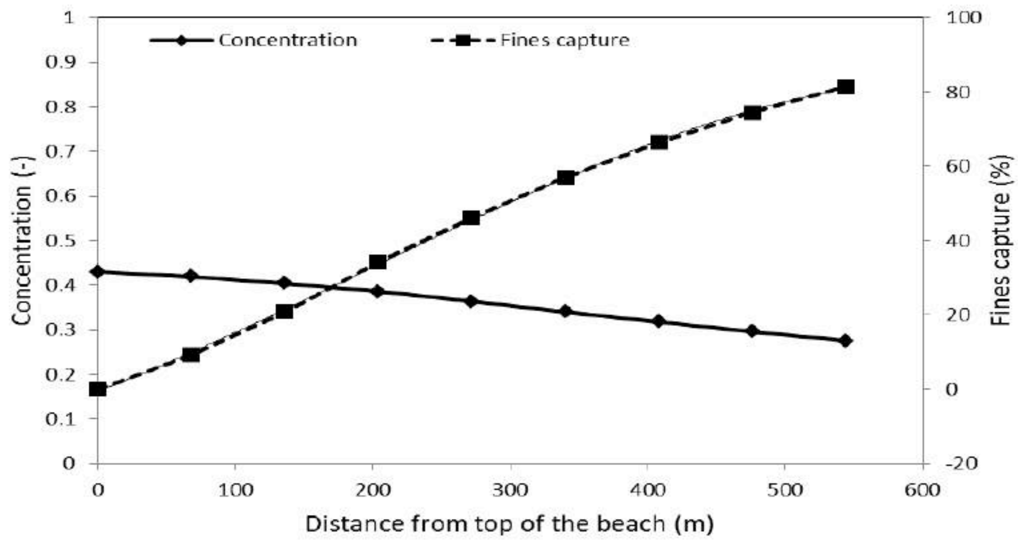

Figure 3. Solids volumetric concentration (fines and coarse particles) and fines capture in function of upstream distance after 3hours of simulation

A polynomial regression curve is fitted on the fines capture data and provides the following equation:

$$FC = -3 \cdot 10^{-7} \cdot x^3 + 0.0002 \cdot x^2 + 0.1433 \cdot x - 0.5378 \qquad \text{With } R^2 = 0.9996$$

The correlation coefficient is very good, therefore such a regression could be used to interpolate fines capture for beach locations where calculations were not made during the simulation and eventually extrapolate for longer beaches. However, this has to be verified upon experimental data.

5 CONCLUSION

A numerical model for fines capture on deposition beaches was presented in this paper. The model is created by the coupling of a 2D numerical model for non-newtonian flow, FLO-2D, and an analytical model for fines capture and solids concentration evolution proposed by Sisson et al. (2012). The results from a theoretical case are presented and suggest that the beach length has a great importance on fines capture along the beach. These results were not validated by experimental data; this has to be the next step in the model validation. However, the model capabilities seem to be promising and it is an encouraging avenue for the development of new prediction techniques.

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Oil Sands Tailings Dewatering

Statistical Data Mining Approach to Improve Understanding of Flocculant Dewatering of Fluid Fine Tailings

Zubin Kuvadia, Michael K. Poindexter, Paul A. Gillis, Lizbeth Rostro, Wu Chen & Jason Tubbs
The Dow Chemical Company, Freeport, TX, USA

ABSTRACT: Mining operations of oil sands frequently result in the creation of fluid fine tailings (FFTs) which reach approximately 30-35 wt % solids content. The remediation of the tailings is challenging due to the geological variations between mining operations and the inherent colloidal stability of the clay particles. Chemical flocculants are frequently used to aid the various dewatering processes of the FFTs. A novel clay dewatering flocculant (XUR) has been identified using high-throughput screening methods and demonstrated several unique performance attributes. Statistical data mining approaches and tools were utilized in this study to identify key property relationships and improve the understanding of flocculant effects on dewatering. Laboratory experiments were conducted that provide dewatering data over the course of several weeks. The rate as well as the extent of consolidation were considered as key performance attributes. Various input parameters were explored which included 1) tailings properties such as clay content and solids content, 2) the operational parameter mixing speed and 3) the additive parameter dosage. The findings from an in-depth data analysis study spanning several years of experimental work will be presented.

1 INTRODUCTION

Excellent recovery of bitumen from oil sands ore is achieved via the Clark hot water process (Clark 1932, Clark 1944, Masliyah 2004). During the commercial scale-up of this process, the concomitant generation of tailings streams was found to be resistant to consolidation of the clay-containing waste stream which has led to the accumulation of slow-dewatering high-clay content slurries (Camp 1977). The inherent stability of clays towards dewatering is well known and mainly due to their small particle size and electrical double layer (van Olphen 1963).

To speed the dewatering process, a number of mechanical approaches have been examined with several of them becoming commercial (Sobkowicz 2009, BGC 2010, Oil Sands Tailings Roadmap 2012, Read 2014). In most of these processes, the addition of a chemical amendment is part of the treatment approach. With the recently released Directive 85, it would seem that the use of chemical amendments will continue to be part of any treatment program envisioned (Alberta Energy Regulator 2016).

The compositional complexity and variability of tailings streams further complicates finding solutions for the treatment of tailing streams (Botha 2015). Due to this complexity, the evaluation of amendments is generally empirical and often further simplified by using model clays e.g. kaolinite, illite, etc. (Mohler 2014, Govedarica 2016, Botha 2015). The approach taken in this paper was to data mine a large number of experiments where numerous fluid fine tailing (FFT) streams were amended and dewatered on the same scale. The primary goal of this work was to determine what broad encompassing performance trends might exist across the numerous experimental runs where several input parameters were varied including the nature of the tailings.

Prior to this work, amendment screening and product development progressed through several size scales. At the start and as needed thereafter, high-throughput robotic liquid handlers produced 5-6 mL sample sizes of treated FFT. These samples were used to determine the solids settling performance as well as floc size via image analyses (Mohler 2012). The best performing candidates from the high-throughput experiments were then followed by studies using in-line static mixer flow loops that produced larger samples ranging from 100 mL up to 2 L (Gillis 2013). The flow loop apparatus allowed a better understanding of key mixing parameters such as blending, floc formation kinetics and shear. To better mimic field conditions, a continuous flow mixing rig was then constructed where the amendment was added to a flowing stream of FFT followed by transport through a dynamic mixer and then through various lengths and diameters of hose with the ultimate production of 5 gallon samples. These treated samples were studied for their solids dewatering performance and initial yield stress values (Poindexter 2016).

All of the experiments reported in this paper used the continuous flow mixing rig, and the sample sizes here are identical to those reported in a recent amendment dosage optimization study (Poindexter 2016). The utility of the continuous flow mixing rig was also demonstrated in the filling of laboratory geocolumns (Stianson 2016a, Poindexter 2015) in addition to casings for a field pilot project (Stianson 2016b). Throughout all three test scales, amendment dose response profiles and various mixing conditions were explored as part of the test program dewatering. Over time, numerous FFT samples having different clay and solids content were tested using the continuous flow mixing rig. Data from experiments on this scale was analyzed using various statistical techniques to determine key independent properties affecting the rate and extent of clay dewatering performance when experiments were conducted for extended periods of time and across multiple sources of FFT.

2 EXPERIMENTAL

All FFT samples were received in 275 gallon totes and agitated for a minimum of 2 days prior to distributing into 55 gallon drums. During agitation with an ITM 7000 Tote Mixer from Dynamix at 175 rpm, the totes were periodically manually scraped to stir-up any settled solids at the bottom and the corners of the totes. Once no appreciable amount of solids were at the bottom and corners, samples of FFT were collected for analysis. The samples were collected once every 4 hours, and the solids content and MBI (methylene blue index) were analyzed. Once these properties were consistent, the tote was distributed into drums by pouring through a screen with 1/2" openings to remove any large clumps of solids. It is important to note that the FFT was not diluted and was used as received. All experimental runs were conducted with FFT distributed into 55 gallon drums. The content of a drum was homogenized with a drill powered paint mixer prior to testing.

MBI values are reported in units of meq MB/100 grams of clay (Omotoso 2008). Solids weight percent values were determined using a Mettler Toledo HB43-S halogen moisture analyzer.

The XUR flocculant was hydrated using process water to make a 0.4 wt% aqueous solution and added to FFT at varying dosage levels. The dosage was based on the FFT solids content. The FFT was mixed with XUR flocculant solution using a proprietary dynamic mixer with speeds ranging from 0-1500 rpm. Upon exiting the dynamic mixer, the treated FFT flowed through a hose of 1" inner diameter at 6-10 gpm and was poured into 5 gallon graduated pails. The pails were covered to prevent water evaporation and left undisturbed for at least 1000 hours. The mudline was measured to determine the solids content. It was critical to monitor the samples for an extensive time frame to realize the long-term dewatering performance of the XUR flocculant.

Yield stress values were determined using a Brookfield DVT-3 Rheometer with the V-73 vane spindle rotated at 0.2 rpm. Measurements were taken within a few minutes of collecting the treated FFT. If any water had been released at the time of the measurement, a pipette was used to carefully remove the water. In this way, it was possible to submerge the spindle consistently in all samples (i.e. placement of the spindle notch at the mud-air interface).

All statistical analyses were conducted using JMP Software version 12.

3 EVALUATING DEWATERING PERFORMANCE FROM SETTLING CURVES

An end goal in FFT pond remediation efforts is to return the area that was mined back to its original state – a trafficable area which can sustain animal and plant life. Two critical issues are the shear strength of the soil (directly related to the solids content) and the time required to achieve this strength to meet mine closure timelines. One method of examining these two variables together is the use of settling curves. These plots track the solids content (below the mud line) over time of a treated FFT sample that has been placed in a vessel. Although the phrase "settling curve" is used, this metric includes both the settling and consolidation of the solids.

Figure 1 shows examples of settling curves. This figure shows the dewatering of 8 different FFT samples treated with a particular flocculant (Dow Chemical's XUR) at a similar dosage. The colors of the curves denote the MBI level of that particular FFT. The combination of marker shape and color allows the identification of each of the FFT samples (A through J). In this particular set of settling curves, a logarithmic scale was used on the x-axis (time). This provides a means of seeing a clearer differentiation of the dewatering performance for the various FFT samples.

Settling curves have been previously published to demonstrate the effect of operational and design variables on FFT treatment. In 2015, this technique was used to illustrate the differences in performance between two flocculant treatments in transparent ~3m high x 0.6m diameter geocolumns (Poindexter 2015). In the following year, additional settling curves were presented to show the effect of flocculant chemistry on dewatering performance (Poindexter 2016).

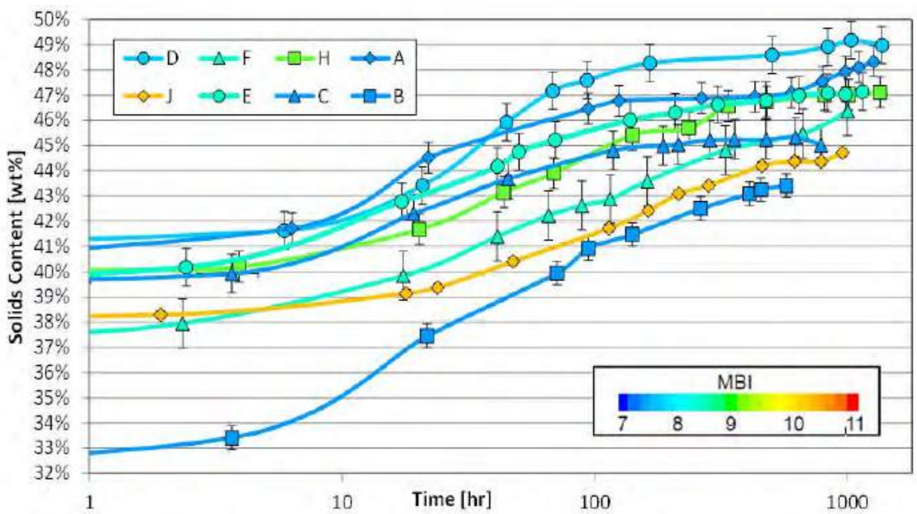


Figure 1: Logarithmic settling curves for 8 FFT samples.

Qualitative information can be gleaned from Figure 1. Key performance characteristics that can be deduced from these curves include the final solids content, the change in solids content and the slope (rate) of dewatering. It appears that this flocculant effectively dewaters a wide variety of FFT samples. The starting point of the solids content (left most value) indicates the approximate initial solids content of the FFT sample. Furthermore, it appears that there might be a dependence on the extent of dewatering (final solids content) and MBI. The curves are colored by MBI, and it appears that a lower MBI FFT might dewater to a greater extent than FFT with higher MBI values. This observation can be further probed by altering the performance plot to show the change of solid content on the y-axis. This plot (for the same data set) is given by Figure 2. Figure 2 shows that the relationship between dewatering performance and FFT characterization is rather complex. Excluding sample C, an overall trend can be seen showing that lower MBI values produce larger changes in solids content. The effect of initial solids content on the solid weight percent change can be seen in the blue square markers (sample B) which corresponds the lowest initial solids content (shown in Figure 1). However, these observations are purely qualitative. The next step is to use methods to produce statistically-based relationships between operational parameters and dewatering performance.

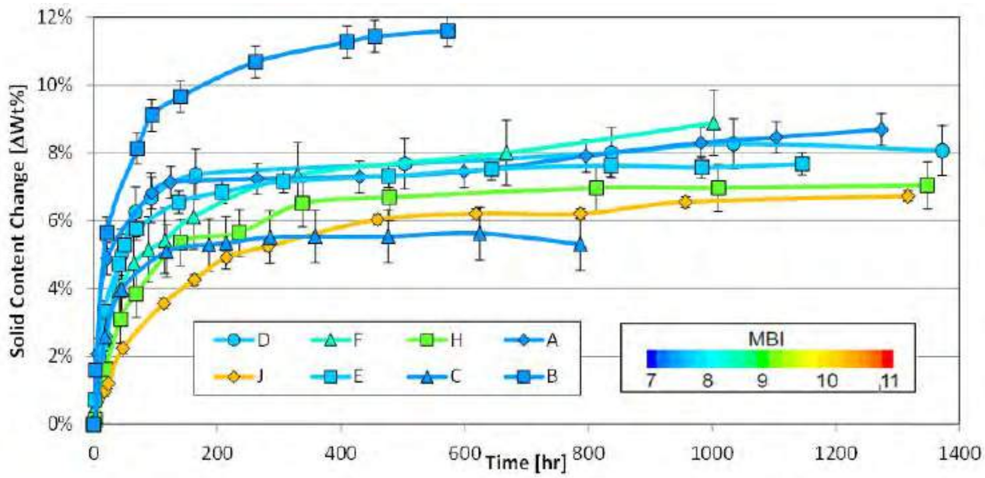


Figure 2: Solids content change as a function of time for 8 FFT samples.

4 STATISTICAL DATA ANALYSIS – CURVE FIT APPROACH

Statistical methods were further utilized to extract useful information from settling curves for quantitative comparisons of settling data across different tailing samples and at different operational parameters . The approach adopted is a curve-fit method, i.e., use a single mathematical function to fit the settling data and capture the trend. A 3-parameter exponential curve was found to give good fits for the settling data investigated. The form of the equation used is as follows:

$$Solids\ wt\ \%\ (t) = Asymptote + Scale * \exp(Growth\ Rate * t) \qquad (1)$$

where *Asymptote, Scale and Growth Rate* are the 3 equation parameters. Three parameter exponential models continually increase in value, but the rate of growth slows so that the model reaches an asymptote. The *Asymptote* term is the long-term (final) solids weight percent (wt%) achieved after settling for several weeks (> 1000 hours). The *Growth Rate* term is indicative of slope/steepness of the curve and is a negative value. A higher magnitude (more negative) means a faster growth rate. The *Scale* term correlates with the total gain in solids content (Final Solids wt% reached – Initial Solids wt% at the start of the settling after treatment). It should be noted that the additive solution will reduce the solids content of the starting tailings suspension depending on the dosage used. Figure 3 demonstrates the suitability of the 3-parameter exponential model by fitting this model to a settling curve.

R-squared values (R^2), often called the coefficient of determination, are a statistical measure of the relationship between data and a regressed line (Alfassi 2005, Harnett 1972). R-squared values estimate the proportion of variation in the response that can be attributed to the model rather than to random error. The model is a good fit if the R-squared value is close to 1. With the 3-parameter exponential growth model, most of the data in our sample space showed a very good fit with R-square > 0.95. Hence, this simple 3-parameter exponential model was determined to be appropriate for our purpose. There is precedence of using double exponential functions and more complicated exponential fits in the literature (Garmsiri 2012), but these were not explored in detail in this study for the sake of simplicity and the ease of interpretation of the parameters provided by the 3-parameter exponential function.

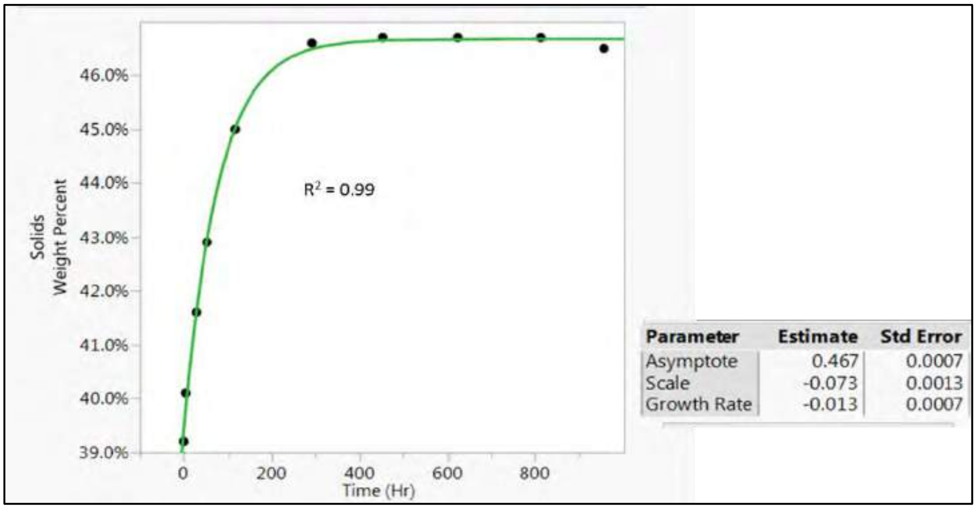


Figure 3: A 3-parameter exponential model fitted against a settling curve (solids content as a function of time, showing an excellent fit R^2 =0.99).

5 RESULTS AND DISCUSSION

Data obtained from 195 settling experiments was selected for this initial study with dynamic data spanning several weeks for each experiment. Based on the curve fits, the output parameters to compare across different settling curves are *Asymptote, Scale, Growth Rate* as well as Measured Yield Stress. The first step in the modeling process is to carry out preliminary analysis including plotting the data to explore possible relationships. Using JMP Software to make histograms and simple plots is an appropriate way to do this. Figure 4 shows histogram plots of key input parameters for the data set. The data set consists of 17 different FFTs with varying solids content and MBI values. These experiments also explored a range of additive dosages and dynamic mixer speeds. It is often useful to start data exploration and analysis by clicking on the histogram bars and visually mapping the different inputs and outputs in order to design and select the form of the model for quantitative analysis. For example, from Figure 4, it can be observed that for the FFT Tote N (which is highlighted), several different dosage levels and dynamic mixer speeds were explored. Figure 5 highlights the corresponding output variables.

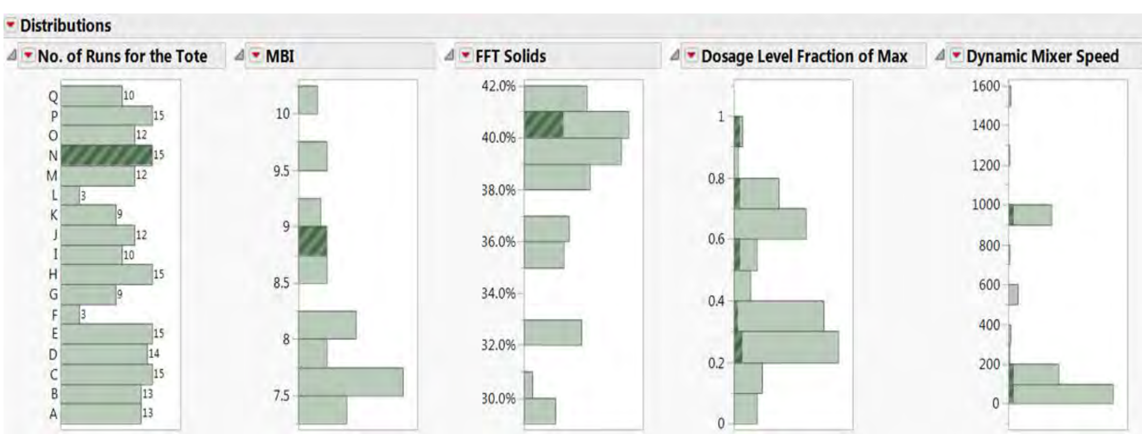


Figure 4: Histogram plots of 17 tailings totes, labeled A through Q, along with critical input parameters such as MBI, initial solids content, additive dosage level and dynamic mixer speed. The first distribution plot shows the number of runs for each tote.

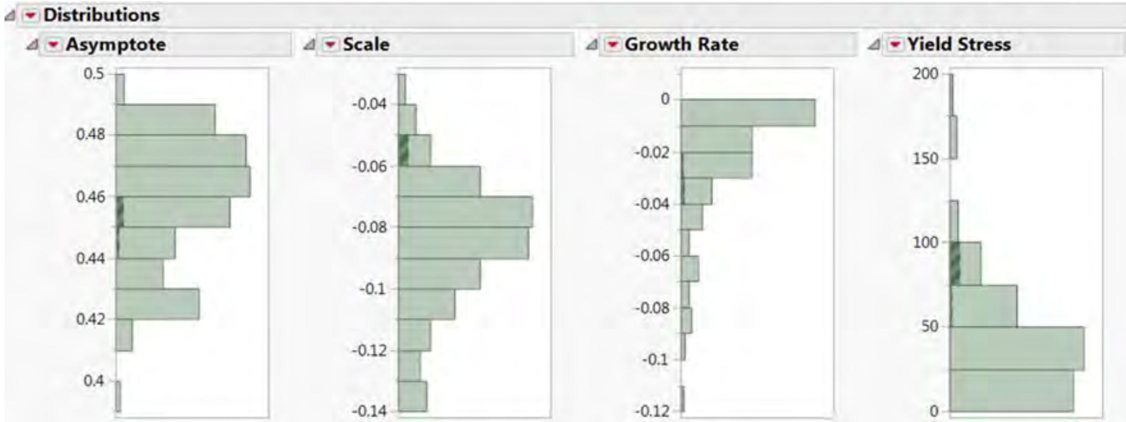
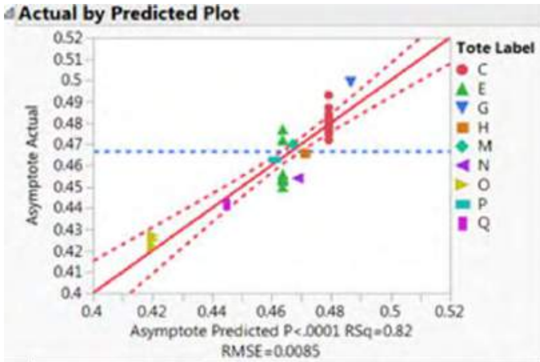


Figure 5: Histogram plots showing the distribution of key output variables (*Asymptote, Growth Rate, Scale* and Yield Stress) for the totes and input parameters corresponding to those shown in Figure 4.

5.1 *Influence of Tailings Properties*

To conduct the various analyses, it was often necessary to select certain input data ranges to remove the influence of parameters which needed to be held constant. A subset containing 32 experiments spanning different FFT totes, all performed at a dosage level equal to 0.3 times the maximum dosage and at a 0 rpm dynamic mixer speed, was selected for this analysis. A first order multiple predictor regression model was explored to understand the impact of the tailings properties on the long-term dewatering performance (indicated by the *Asymptote* term). Multiple predictor regression is generally preferable compared to univariate analysis, or one variable at a time. Multiple regression can separate the effects of different factors and pick up significant predictors that are masked when univariate analysis is used. The coefficients were estimated using the least squares method.

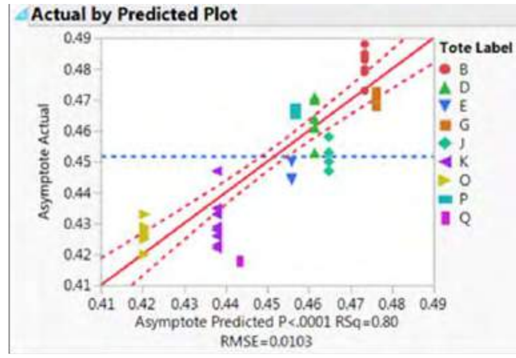
A simple first order model with FFT solids content and MBI values as predictors was chosen to fit the data. The plot of Actual Response versus Predicted Model (Figure 6 a) indicates that the model fits the experimental data well, and the data points fall very close to the straight line shown on the graph. The solid (red) line represents the model, while the other two dotted lines represent “error bounds” about that model at 95% confidence. The horizontal (blue) line represents the overall mean of the data. The Summary of Fit table indicates that this model is significant because of a high R^2 value (0.82). The Parameter Estimates table provides a list of the model estimates along with their standard errors and develops a t-test for testing the significance of each parameter. The small p-values (less than or equal to 0.05) indicate statistical significance. Hence, from the model, it can be inferred that the higher the initial solids content of the tailings, the greater the long-term extent of dewatering (*Asymptote*). Additionally, high clay content (MBI) impedes the extent of dewatering as shown by the negative sign of the Parameter Estimate.



Summary of Fit	
R- Square	0.82
R- Square Adj	0.81
Mean of Response	0.47
Observations	32

Parameter Estimates				
Term	Estimate	Std Error	t Ratio	Prob > |t|
Intercept	0.449	0.0024	184.7	< 0.0001*
FFT Solids	0.027	0.0033	8.24	< 0.0001*
MBI	-0.016	0.0026	-6.02	< 0.0001*

a)



Summary of Fit	
R- Square	0.8
R- Square Adj	0.79
Mean of Response	0.45
Observations	46

Parameter Estimates				
Term	Estimate	Std Error	t Ratio	Prob > |t|
Intercept	0.446	0.0021	213.1	< 0.0001*
FFT Solids	0.023	0.0025	9.38	< 0.0001*
MBI	-0.012	0.0027	-4.42	< 0.0001*

b)

Figure 6: a) Model Fit Analysis for a subset containing 32 settling experiments for multiple FFT totes, at a constant dosage level (0.3 times max value) and 0 rpm dynamic mixer speed. b) Model Fit Analysis for a subset containing 46 settling experiments for multiple FFT totes, at a constant dosage level (0.7 times max value) and varying dynamic mixer speed (0 to 1500 rpm range).

Another subset containing 46 experiments spanning different FFT totes, all performed at a dosage level equal to 0.7 times the max and at varying dynamic mixer speeds, was selected. The same first order model was chosen to compare this dataset with the previous dataset selected. The plot of Actual Response vs Predicted Model (Figure 6 b) shows that the model fits this dataset reasonably well also. The R^2 value of 0.80 for this dataset is only slightly lower than lower than the R^2 of the previous dataset (0.82). This suggests that the same conclusions about the influence of tailings properties hold at the two dosage levels considered. Additionally, the variability introduced in the current dataset due to different dynamic mixer speeds causes a slight spread in the data, but does not diminish the model fit significantly.

This influence of FFT solids content and MBI values on the long-term dewatering performance can also be visualized as a regression partition tree (Figure 7) in JMP (Breiman 1998). Figure 7 includes data from 135 settling experiments at several dosage levels and mixing speeds for multiple FFT totes. The individual data points are colored by their MBI values (blue = low MBI, red = high MBI) and different marker shapes represent different FFT totes. Based on recursive partitioning algorithm, the dataset is initially split into 2 branches at FFT Solids 39%. Each branch is further split based on the MBI values. For each partition branch, lower MBI values correspond to greater *Asymptote* values. This helps to visually reinforce the observation of the first order model fit discussed above that higher solids content results in greater dewatering performance, and a higher clay content results in lower dewatering performance.

5.2 *Influence of Additive Dosage*

A subset of 5 FFT totes was chosen to explore the influence of the additive dosage on the dewatering performance. It can be observed from Figure 8 a) and b) that for the dosage levels explored in this dataset, the dosage level has a pronounced influence on the *Growth Rate* parameter, but not a significant influence on the long-term dewatering performance (*Asymptote*). A higher dosage level of the additive results in a more negative growth rate.This will mean steeper settling curves and relatively greater settling performance in the first couple of days. An important learning from this analysis is that it is easy to misinterpret the short-term settling data. The data suggests a strong influence of dosage on the initial solids settling rates, but this effect gradually diminished over time. It becomes essential to collect long-term settling data over the course of several weeks to fully understand the impact of experimental parameters and source material properties.

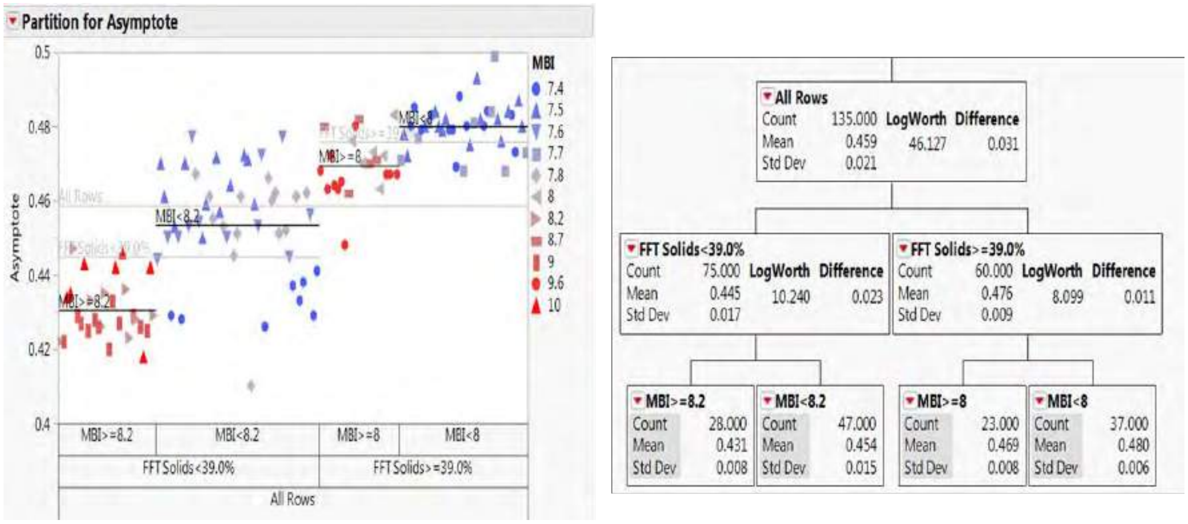
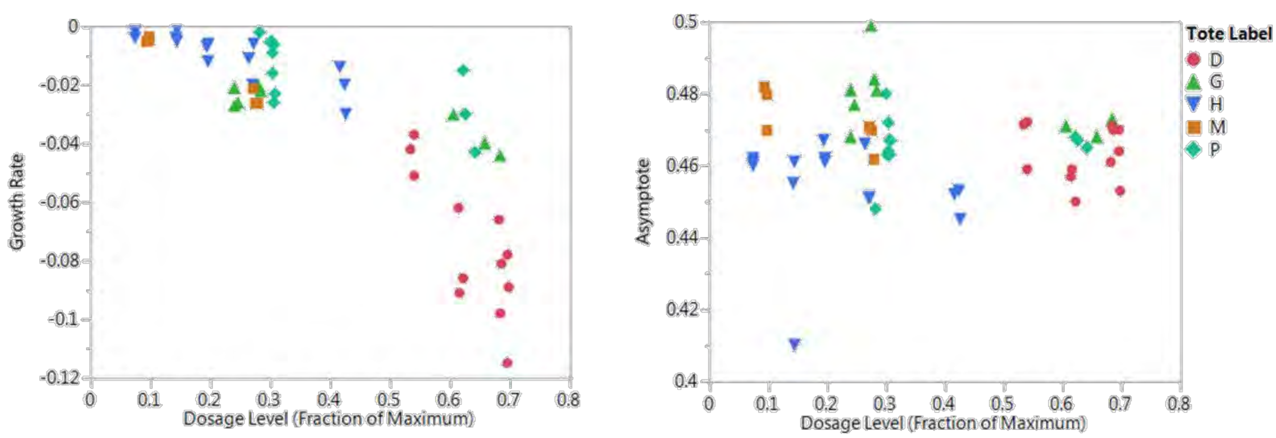


Figure 7: Recursive partitioning pictograph for 135 settling experiments at different dosage levels and mixer speeds showing the effect of initial solids content (partition branch at 39% solids) and MBI values.



a) b)

Figure 8: a) Growth Rate versus Dosage Level and b) Asymptote versus Dosage Level.

5.3 *Yield Stress and Pipeline Transport*

Yield stress can serve as a useful output variable for efficient pipeline transport. A plot of the yield stress by FFT solids weight percent is provided in Figure 9. Plotting the yield stress measurements (natural log transformation) against FFT solids weight percent showed a high statistically significant correlation ($R^2 = 0.79$). This result is consistent with previous results (Poindexter 2016) as well as related work where higher percent solid streams give higher yield stress values (McCaslin 2016).

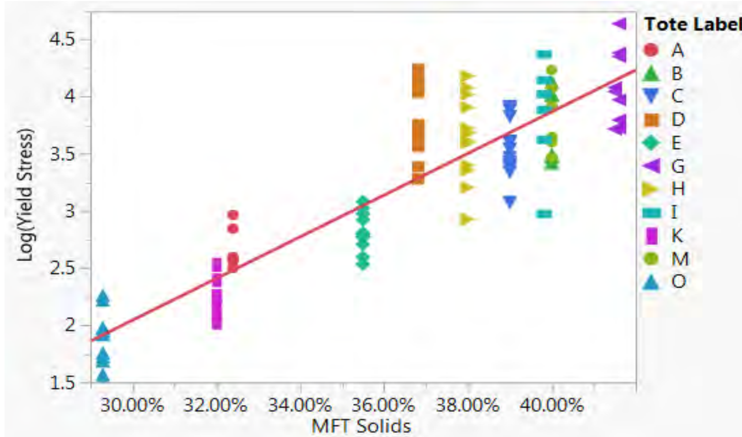


Figure 9: Plot of XUR-treated FFT yield stress versus solids weight percent for different FFT totes.

6 CONCLUSIONS

Statistical data mining approaches and tools were utilized in this study to identify key property relationships and improve the understanding of flocculant effects on dewatering. Laboratory experiments that provide long-term settling data over the course of several weeks were conducted. The dewatering rate as well as the extent of dewatering were considered key performance attributes. The tailings properties such as initial solids content and the clay content (MBI values) impacted the dewatering performance significantly. Tailings with high initial solids content show a greater dewatering performance, while tailings with high clay content were resistant to separation of water. In the data sets considered, additive dosage levels had a greater impact on the initial rate of dewatering compared to the extent, thus confirming the need for long-term settling data spanning several weeks. Tailings with low initial solids content gave lower yield stress values which may benefit pipeline transportation.

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Use of calcium hydroxide as a coagulant to improve oil sands tailings treatment

M. Tate
Graymont Inc., Genoa, OH, USA

J. Leikam, J. Fox
Graymont Inc., Salt Lake City, UT, USA

N. Romaniuk
Nikolas Romaniuk Consulting, Edmonton, AB, Canada

ABSTRACT: Fluid Fine Tailings (FFT) from oil sands mining operations are difficult to dewater and reclaim. Though much work has been done with the use of flocculants to improve dewatering characteristics, concerns about reclaiming treated tailings remain. The use of calcium hydroxide coagulation to address these concerns by modifying clay particles is explored in this study.

Calcium hydroxide increases *pH* as well as provides divalent calcium cations for coagulation of FFT. These calcium cations preferentially react with bicarbonates until a pH of 11 is reached and have low solubility in process water. As *pH* increases above 11, calcium becomes soluble and combines with clay through cation exchange. This increases particle size and enhances settling. As *pH* increases above 12, pozzolanic chemical reactions between calcium and clay improve strength and dewatering characteristics of the FFT. Optimization of calcium hydroxide dose to improve geotechnical characteristics along with water chemistry is explored.

1 OBJECTIVE

The objective of this work is to understand the effects of calcium hydroxide coagulation on the reclamation properties of Fine Fluid Tailings produced by oil sands mining operations.

2 BACKGROUND

2.1 *The reclamation of oil sands mining tailings has been challenging*

The Clark Hot Water Extraction (CHWE) process was developed by Dr. K.A. Clark and his co-workers in the 1930's to extract bitumen from an ore-water slurry in Alberta oil sands mining operations (Clark 1939, Clark and Pasternack 1932). Though some modifications have been made, this process serves as the basis for processing ore from current oil sands operations. The CHWE process initially utilized sodium hydroxide to adjust the *pH* of the ore water slurry to 8 – 8.5 which activates asphaltic acids to produce a water-soluble surfactant species that liberates bitumen from the oil sand with good recovery efficiencies (Moschopedis et al., 1980, Speight and Moschopedis, 1977/78, 1980). Sodium compounds added for *pH* adjustment as well as sodium compounds in the ore are quite soluble and have accumulated over time in process water which is recycled from tailings treatment to the extraction process. The accumulation of sodium along with other inorganic and organic ions and molecules results in a complex water chemistry which complicates the reclamation of oil sands tailings.

A tailings slurry containing process water, sand, clay, and small amounts of residual bitumen is produced as a byproduct of the extraction process (Kasperski 1992). This slurry is typically discharged into a diked area to settle and consolidate solids and provide water for the extraction process. Sand particles (>44 µm) from the tailings can be removed by cyclones or settle quickly in ponds and can be used as beach sand or for dike construction. Reclamation of the fines less than -44 µm portion of the tailings is more problematic. A portion of these fines will be captured with sand to form beaches in a contained area. The remaining fine solids, water and residual bitumen, called thin fine tailings, drain as an approximately 10-15% solids slurry to form ponds within contained areas. In the pond, the thin fine tailings settle to 20% solids within a few days and to 30-35% solids within a few years to form a fluid-like colloidal material called fluid fine tailings or FFT. FFT (also called mature fine tailings (MFT)) becomes a structurally stable slurry which is predicted to remain in a fluid state for decades because of its slow consolidation rate. Slow consolidation, limited solids strength and poor water quality of FFT has limited reclamation options and resulted in large tailings ponds at every oil sands mining operation.

During the last four decades, significant effort has been devoted for the development of novel technologies to reduce FFT production and improve the reclamation characteristics of oil sand tailings. A number of technologies have been explored (BCG 2010) including whole tailings treatment, non-segregating tailings (NST) production, composite tailings (CT) production, tailings reduction operations (TRO), atmospheric drying, thickened tailings, blending with overburden and centrifuge technology with limited commercial success. Canadian Oil Sands Innovation Alliance (COSIA) continues to explore novel technologies to reduce FFT inventory (COSIA 2012).

2.2 *Coagulation versus Flocculation*

The use of coagulants and/or flocculants is a key component of most oil sands tailings treatment technologies. The ability of treated tailings to settle to high solids content is important to achieve proper geotechnical properties as well as minimize the volume required for containment and future reclamation. However, the fine FFT particles have proven difficult to consolidate. Kasperski (1992) indicates that electrostatic, steric, Van der Waals and hydration forces influence the ability of fine particles to consolidate. Coagulants and flocculants can influence these forces by modifying the surface of the clay particles and/or binding them together to form large particles that settle and dewater more easily. However, coagulants and flocculants function using very different mechanisms (Drew 1994).

Coagulants primarily act by destabilizing the electrostatic repulsion of suspended solids through cation exchange or chemical reaction. This neutralization of charge combined with bridging mediated by multivalent cations helps link individual particles together, which increases the average particle size. High shear mixing can be used to ensure that inorganic coagulants are well dispersed in FFT and able to diffuse to the clay-water interfaces. Previous FFT studies have focused on investigating lime, gypsum, alum and carbon dioxide as coagulants (Mathews et al 2002).

Flocculation is commonly used to increase the settling rate of fine suspensions by using very large organic polymers to capture suspended solids and form large agglomerates called flocs. While the clay particles are chemically unchanged, being trapped in flocs increases their size and density which allows for rapid settling. The effectiveness of the polymer is dependent on a number of factors including its type (anionic, cationic or nonionic), molecular weight, degree of ionization, and mixing procedure. Undermixing of polymers with FFT results in poor fines capture; however, overmixing or high shear conditions during slurry conveying will result in the breakdown of flocs. Some studies have investigated double polymer treatments using cationic then anionic polymers to enhance flocculation by bridging and charge neutralization (Lu et al 2016).

2.3 *Soil Treatment – An example of the impact of lime coagulation of reactive clays*

The use of calcium hydroxide (also referred to as lime) to modify the surface of fine clays to improve strength, increase particle size and reduce moisture content is well established in soil treatment applications. Fine- grain clay soils (min. 25% through a 74 mm sieve) with a plasticity index above 10 are considered to be good candidates for soil treatment with lime (National Lime Association 2004). Treatment with lime can either modify or stabilize the soil.

2.3.1 *Soil Modification*

Calcium ions from the hydrated lime migrate to the surface of the clay particles and displace univalent cations through cation exchange. This exchange facilitates the release of water from the clays, increases particle size and reduces the plasticity index of the soil. Benefits from this process can be seen within a few hours to overnight.

2.3.2 *Soil Stabilization*

As calcium hydroxide increases *pH* above 10.5 alumina and silica from the clay become soluble and react with calcium cations to form cementitious products. The calcium silicate and calcium aluminate hydrates formed result in a more permanent modification compared to cation exchange. When lime is added to a reactive soil, it has been proven by extensive laboratory and field testing that plasticity index decreases, water release occurs, resilient modulus increases and compressive strength exceeding 1,400 kPA can be obtained (Little, D.L. 1999). A testing protocol has been developed to determine the proper dose of lime for stabilization to occur (National Lime Association 2006).

2.4 *Exploring potential roles of calcium hydroxide as a coagulant in treatment*

There have been many other studies related to the use of lime as a coagulant on its own or in combination with gypsum or polymers to treat whole tailings, non-segregating tailings and fluid fine tails (Baillie & Malmberg 1969, Caughill et al 1993, Chalaturnyk et al 2002, Donahue et al 2008, Erno and Hepler 1981, Hamza et al 1996, Lane 1983, Lorentz et al 2014, Liu et al 1980, Mathews et al 2002, Ozum 2013, and Sworska et al 2000). This work differs from other studies by focusing on FFT and increasing treatment process *pH* in a manner very like methods used for soil treatment. The impact of cation exchange and pozzolanic reactions on the FFT properties will be observed. The ability of calcium hydroxide to modify the size, plasticity index, strength and dewatering ability of the tailings with increasing *pH* will be studied. A key difference between soil and oil sands tailings treatment is the presence of process water in tailings that contains high levels of bicarbonates and organics that complicate oil sand treatments. Calcium hydroxide has other benefits to potentially improve water chemistry that will be discussed.

3 FLUID FINE TAILINGS CHARACTERISTICS

Four different FFT samples were used for this work. The characteristics of each sample are presented in Table 1. All calcium hydroxide doses are based on the wet mass of FFT.

Table 1. Fluid fine tailings sample properties

FFT	Mineral Solids Content %	Bitumen Content %	Methylene Blue Index	Clay %	Na^+ (ppm)	Carbonate Alkalinity (ppm)	*pH*
1	32.5	2.9	9.2	66	362	579	7.9
2	33.4	1.3	11.2	80	877	937	8.3
3	33.8	1.5	9.9	71	827	860	7.6
4	39.6	1.5	8.6	62	894	782	7.7

4 RESULTS

4.1 *Settling Rate*

The addition of calcium hydroxide to several FFT samples without dilution has shown little, if any, improvement in settling. Samples of FFT-1 were diluted with an equal amount of process water to study the impact of calcium hydroxide dosage at a diluted FFT concentration. As seen in Figure 1, very little change in settling rate was seen until a dose of 1250 ppm on a calcium oxide basis was added. Water chemistry results obtained from this test indicated that the *pH* exceeded 11. Prior to reaching the 11 *pH* level, soluble calcium levels dropped slightly from the control but above 11 *pH* an increase in soluble calcium was noted. Soluble sodium and potassium cations also increased when the *pH* went over 11 which suggested that the calcium cations were modifying the surface of the clays by cation exchange. As calcium hydroxide dose increased beyond 1250 ppm settling was observed to decrease despite additional soluble calcium. Some residual organics were seen at the surface of some of the settling rate tests at doses below 11 *pH*. Residual organics were not as visible as the *pH* increased above 11. Even the best settling sample did not reach the dilution point though settling did appear to be improved from the control sample. The use of calcium, whether from lime or gypsum, resulted in clear water above the settled solids. This test was also repeated using distilled water to dilute the sample with similar results.

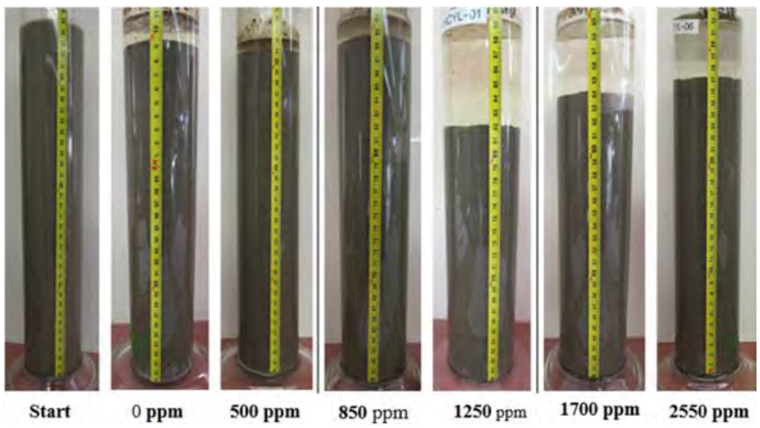

Figure 1. FFT 1 settling test – 50% dilution at 3 weeks

4.2 *Particle Size Distribution*

The particle size distribution of FFT was measured by a Cilas 1190 laser diffraction particle size analyzer. Residual bitumen was removed from the sample by washing with heptane. As seen in Figure 2, the addition of a calcium hydroxide slurry resulted in an increase in the median particle size of both FFT-1 and FFT-2. The median size increased from 2 microns to nearly 30 microns for FFT-1 and from 6 microns to 14 microns for FFT-2 at the highest dosage. It is important to note that as *pH* exceeded 12.0 for the 7000-ppm dose, the median particle size increased by more than 100% for FFT-1.

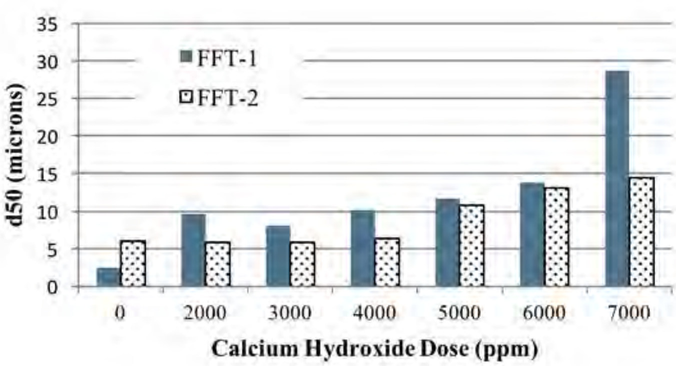

Figure 2. Impact of calcium hydroxide dosage on FFT-1 and FFT-2 median particle size (d50)

4.3 *Atterberg Limits*

Atterberg limit tests were run on FFT-1 and FFT-2 to determine the impact of calcium oxide addition on the plasticity limit of the treated tailings (Burden, 2016). Calcium oxide was added directly to the FFT samples in a manner consistent with typical soil treatment tests. The dosage range was selected to cover the wide range of dosages seen in soil applications when high levels of sodium are present in the soil. Test results, seen in Figure 3, show that the plasticity index declined sharply with the addition of calcium oxide to FFT up to a dose of approximately 7000 ppm. Above a dose of 7000 ppm further reduction in plasticity index was seen but at a slower rate. The reduction of plasticity index for FFT-2 was greater than that of FFT-1.

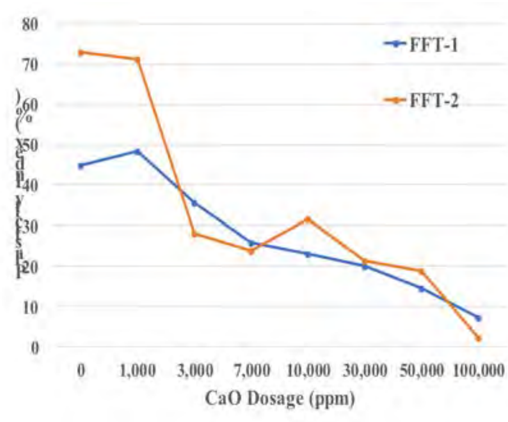

Figure 3. Effect of CaO dose on Plasticity Index (after 28 days treatment)

4.4 *Yield Stress*

Yield stress measurements were taken on a control sample and samples of FFT-1 and FFT-2 treated with lime and gypsum at three different time intervals as seen in Figure 4. Test results show that little change was seen in the yield stress of the control sample over time as expected. Test data shows that the addition of calcium hydroxide resulted in significant yield strength gain over time. Calcium hydroxide yield strengths were generally higher than those obtained with gypsum. A substantial gain in yield strength was seen after 15 days with the addition of 10,000 ppm of calcium hydroxide.

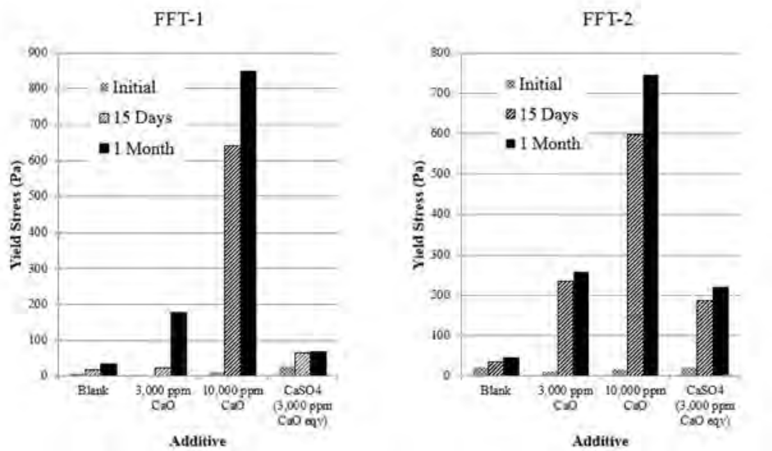

Figure 4. Increase in yield stress for CaO and gypsum treated FFT in one month

4.5 *Pressure Filtration*

A 150 gram FFT sample was subjected to 100 psi air pressure. A Whatman 50, 2.7 micron filter was used to separate water from the sample. Each sample was pressurized for up to 120 minutes. Test results, seen in Figure 5, show that optimal results were obtained at the 4,000 ppm dosage level. As seen in Figure 6, the rate of water loss was influenced by the dosage level. At high dosage, the treated FFT released water much more quickly. This quick water release created cracks in the treated FFT which released the pressure and ended the test prematurely, which means the maximum solids contents shown in Figure 5 may be underestimated for the 5,000 ppm and 10,000 ppm dosages.

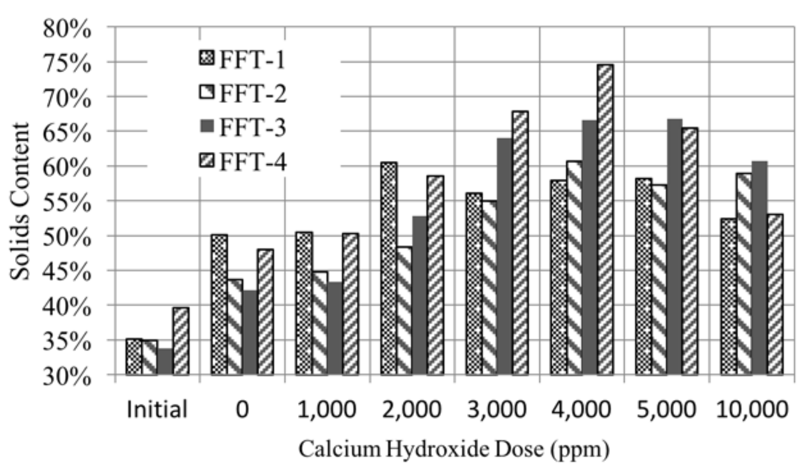

Figure 5. Percent solids of FFT after pressure filtration testing

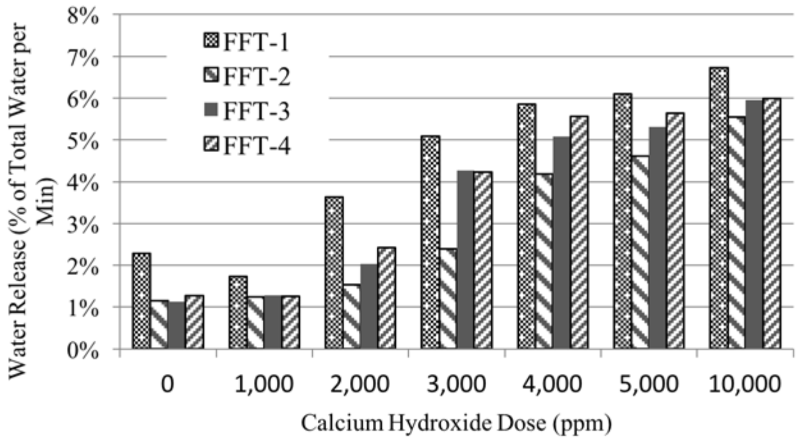

Figure 6. Rate of water release after 5 minutes of pressure filtration

4.6 *Water Chemistry*

Filtrate collected from the pressure filtration tests was analyzed for *pH* and basic water chemistry. As seen in Figures 7 & 8 the concentration of calcium and sodium both decreased as calcium hydroxide was added to each FFT up to a *pH* of approximately 11.5. This is similar to water chemistry results from the settling tests. As seen in Figure 9, the *pH* level increased rapidly with dosage until the maximum *pH* level of approximately 12.5 was achieved with a 4,000 ppm dosage of calcium hydroxide.

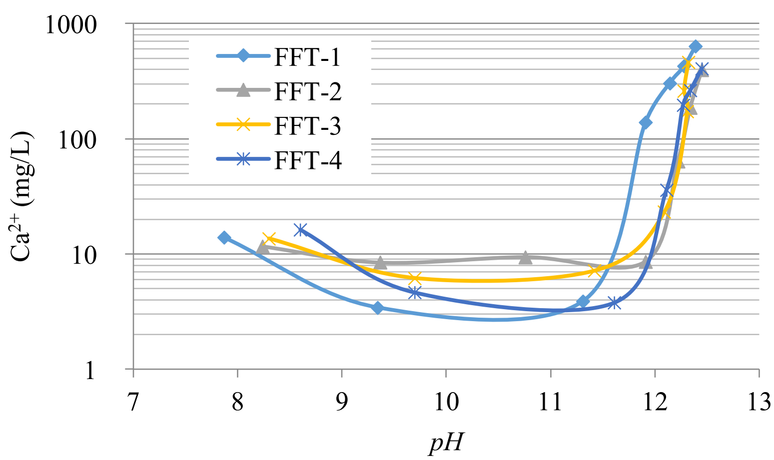

Figure 7. Impact of *pH* on FFT pressure filtrate calcium concentration

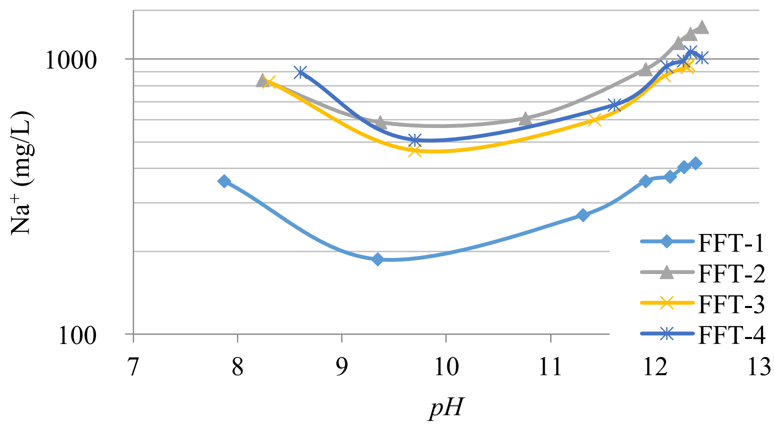

Figure 8. Impact of *pH* on FFT pressure filtrate sodium concentration

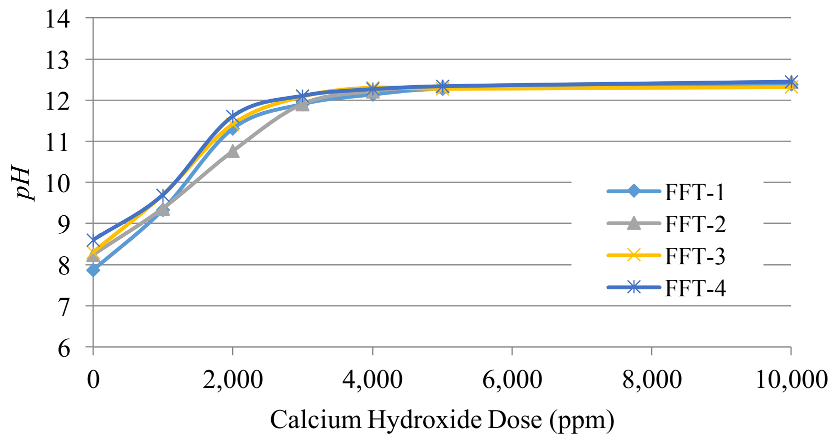

Figure 9. Impact of calcium hydroxide dosage on pressure filtrate *pH*

4.7 *Lime slurry concentration*

The impact of lime slurry concentration on the performance of pressure filtration tests was studied with each of the FFT samples. As seen in Figure 10, at the same 4,000 ppm dose of lime, the lime slurry concentration can have a substantial impact on the outcome of this test. In general, it appeared that lime slurry concentrations must be 10% or lower to optimize the outcome of filtration tests. For FFT-2, the optimal percentage of solids in the lime slurry was 5% or lower.

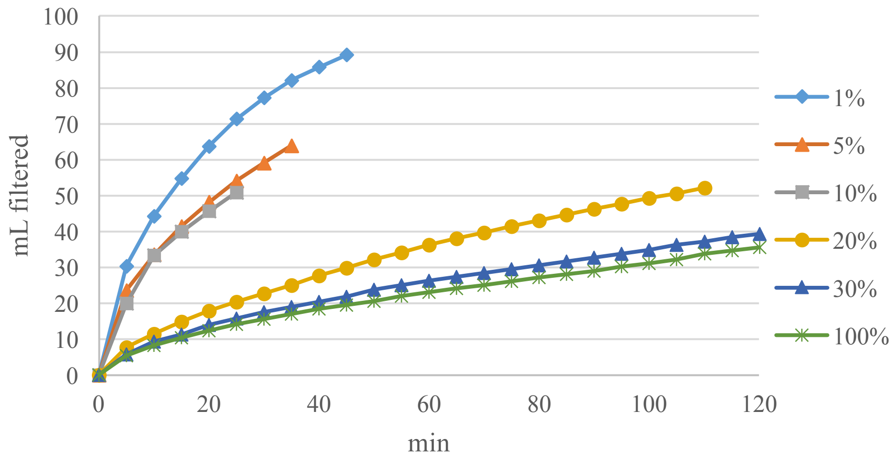

Figure 10. Impact of lime slurry dilution on dewatering in pressure filtration for FFT-3 treated with 4,000 ppm calcium hydroxides.

5 DISCUSSION

Rogers and Glendinning (1996) indicates that in soil modification, calcium hydroxide improves the geotechnical characteristics of soils in two ways: cation exchange and pozzolanic reactions. Cation exchange reduces the thickness of absorbed water known as the electrical double layer. This allows coagulation, increases the internal angle of friction, changes the texture and reduces the plasticity index of fine clay particles in the soil. Pozzolanic reactions occur as calcium is available to react with dissolved silica and alumina from the clay at a high *pH* level. This reaction forms calcium silicate hydrate and calcium aluminate hydrate gels which can bond particles together as a weak cement.

Based on our work, it appears that calcium hydroxide can react with FFT clays in a similar way but the application is considerably more complex due to water chemistry and organic contaminants. The presence of sodium both in the form of sodium bicarbonate and sodium hydroxide limits the ability of soluble calcium to be available for both cation exchange and pozzolanic reactions. Naphthenic acids and other organics further complicate the reaction of calcium hydroxide with the clay. Finally, the solubility of calcium from calcium hydroxide is significantly influenced by the introduction of lime as dilute slurry rather than dry powder.

When cation exchange occurs univalent cations, such as sodium and potassium, will be replaced by divalent calcium cations on the surface of clay. Higher levels of sodium and potassium should be seen in the process water after cation exchange takes place. Water chemistry from settling rate and pressure filtration tests shows that calcium cations have low solubility (they actually decreased in concentration) until bicarbonates are largely depleted at a *pH* over 11. It appears that calcium hydroxide preferentially reacts with bicarbonates to form insoluble calcium carbonates until this *pH* is reached and is not available for cation exchange. Sodium and other exchangeable cations also decreased in concentration which is further evidence that cation exchange is not occurring. Once the *pH* reached 11, soluble calcium cations were seen in the water chemistry as well as increased sodium and potassium levels which suggests that cation exchange was occurring. Other signs of cation exchange at this *pH* include improved settling rate, increased particle size, increased yield strength, improved dewatering and a lower plasticity index. Despite signs of cation exchange and settling as calcium becomes available, settling rates decline rather improve as the dose of calcium hydroxide is increased above a *pH* of 11.

Though some water release was evident as calcium became soluble, pressure filtration tests showed that the ability of calcium hydroxide treated FFT to release water is not optimized until *pH* increases to above 12. Despite poor settling conditions at this *pH* level, it was clear that calcium hydroxide had impacted the ability of the solids in each FFT to dewater. The presence of high levels of clay in FFT appeared to make dewatering difficult. The percent solids of treated samples look promising for the use of calcium hydroxide without a flocculant at *pH* level above 12. Currently with the use of coagulants and/or flocculants the typical percent solids concentration is 55% or less for centrifuging technology. Calcium hydroxide addition in the 3,000 to 4,000 ppm range showed promise to exceed the 55% solids level but at higher doses the treated FFT solids concentration decreased. Test results suggest that increasing calcium hydroxide dose provided an increase in the rate of water loss from the FFT after five minutes. At the highest lime doses, the release of water stopped early into the experiments due to the formation of cracks. In many cases the testing ended because of crack formation well prior to the 120 minute target. For example, at 4000 ppm, the typical length of the test was 40 to 45 minutes before cracks formed ending the test. At 10,000 ppm, test typically lasted 25 to 30 minutes before cracks formed. Lorentz et al (2014) indicated that the use of lime or a lime/gypsum blend resulted in cracking which was useful in rim ditching drying. While cracking is useful for drying operations, it can be detrimental in optimizing pressure filtration. Other types of mechanical dewatering processes such as other pressure and vacuum filtration process and centrifuging will be studied to further optimizing these results.

Tate et al. (2106) indicated that sodium hydroxide could be produced by the reaction of sodium bicarbonate with calcium hydroxide. Sodium hydroxide could help to increase *pH* but would make calcium hydroxide less soluble due to the common ion effect. The importance of soluble calcium in this system is evident based on the better performance of dilute lime slurries. Limited solubility could impact both cation exchange as well as pozzolanic reactions. Carbon dioxide absorbed from the air would eventually convert sodium hydroxide to sodium bicarbonate.

The role of bitumen and other organics is also important. Romaniuk et al. (2015) has observed that residual bitumen could play a significant role in tailings treatment. Residual bitumen could coat the fine clay particles and be difficult to remove. They observed bitumen release during settling tests at a *pH* level below 11 when FFT was treated with calcium hydroxide. The release of residual bitumen could expose new areas for cation adsorption on the clays. This could be the reason why sodium and potassium concentrations decreased in this *pH* range when calcium hydroxide is added. This could explain the drop in soluble sodium and potassium seen below the *pH* of 11. Above a *pH* of 11, the level of bitumen release was either reduced or not evident. As *pH* increases Havre (2002) indicates that the formation of calcium naphthenates is possible. Calcium naphthenates are typically insoluble in both oil and water. The extent to which insoluble calcium naphthenates form and their impact on the settling of tailings is unclear. Additional work is underway to understand this better.

In dewatering tests, clearly the percentage of solids of the lime slurry made a difference on both the rate of water loss and the final percent solids obtained in this testing. The reason for this difference is not clear but likely relates to the difficult environment for solid calcium hydroxide to solubilize in water. Slurries with low percent solids contained a higher ratio of soluble to insoluble calcium as well as providing a dilution effect. There is an optimal percent solids level that changes based on the FFT characteristics. The effect on test results was significant.

The geotechnical characteristics of FFT appear to improve as calcium hydroxide dosage increases. Yield stress measurements are low initially but improve over time. This is consistent with observations in soil stabilization where soil strength gradually improves. Atterberg limit testing show a reduction in the plasticity index as calcium hydroxide dosage increases. The upper portion of the range was set to duplicate the dosage for a successful project involving a highly saline soil at the site of the Mexico City airport construction project. This soil had a sodium concentration of approximately 40,000 ppm. Good reduction was seen in the plasticity index for both samples but calcium hydroxide had bigger impact on the sample with a higher MBI.

As with soil stabilization, this work shows *pH* targets with oil sand tailings that help predict performance. Other work has been done which demonstrates the importance of *pH* on the interaction of calcium cations with sodium rich kaolinite clay. Atesok, et al (1988) studied flocculation of Na-kaolinite of by calcium and polyacrylamide. As shown in Figure 11, the adsorption of calcium on the surface of Na-kaolinite was highly dependent on *pH* and time. High *pH* facilitates calcium adsorption based on our work as well as theirs. We have also observed that water chemistry changes over time, such as reduction of high *pH* as bicarbonates are reabsorbed back into the process water which results in a significant reduction of the soluble calcium cations during aging. The impact of aging on the water chemistry will be explored in future publications.

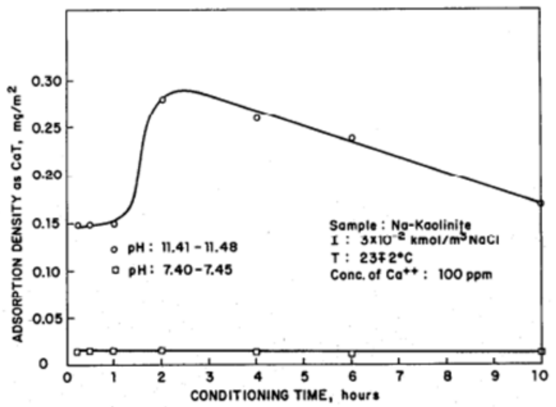

Figure 11. Effect of conditioning time on adsorption of calcium to Na-Kaolinite (Atesok, et al 1988)

Process water discharged with high *pH* levels will have high levels of calcium cations in solution that could be of concern in the extraction process. Several possible methods have been identified to address this. Settling ponds will naturally absorb carbon dioxide from the air to lower their *pH* and remove soluble calcium. Water chemistry tests show that samples with over 100 ppm soluble calcium and be reduced to less than 10 ppm in one month. This process could be accelerated by the injection of carbon dioxide into the filtrate prior to the pond system to lower *pH* and soluble calcium. A third option is to use some of this water to react with bicarbonates from process water to regenerate sodium hydroxide prior to extraction, which would reduce alkali needs. Finally, the calcium laden water could be used to dilute the FFT prior to centrifuge or filtration processes. The soluble calcium would be lower the calcium hydroxide requirement initially but the impact of recycling on the build-up of sodium hydroxide in the water and its effect on dewatering require further investigation.

6 CONCLUSIONS

This work demonstrated that the use of calcium hydroxide can be beneficial for the coagulation of fluid fine tailings. The effectiveness of calcium hydroxide addition is influenced by the sodium concentration of the process water as well as residual bitumen in the system. Though clay modification happens at high *pH*, it is not always evident because of poor FFT settling characteristics. A force, such as that in pressure filtration, is needed to release the water contained in FFT treated with calcium hydroxide. The following conclusions can be made from this study:

- The preparation of slurries containing calcium hydroxide is a critical factor in the rate of reaction and filtration efficiency of calcium hydroxide treated FFT. In FFT treatment, addition of calcium hydroxide as dilute slurry performed significantly better compared to a dry powder. The ideal lime slurry concentration is dependent on FFT properties but in general the calcium hydroxide slurry concentration should not exceed 10% by weight.
- The rate and amount of FFT dewatering is dependent on the calcium hydroxide concentration. It appears that to optimize dewatering the *pH* must be raised above 12 and pozzolanic reactions occur. Dependent on the process requirements, overdosing calcium hydroxide may reduce the percent solids generated in some dewatering methods due to premature cracking.
- Though promising results were obtained employing pressure filtration to dewater lime treated FFT, further work is needed to optimize the dewatering potential of calcium hydroxide to treat FFT in centrifuge, filtration or related processes.

- High levels of bicarbonate in process water prevents soluble calcium cations until a *pH* of 11 is reached which is the pH at which dissolved bicarbonate has been converted to a calcium carbonate precipitate.
- Cation exchange with FFT clay is not apparent in the water chemistry or settling tests until soluble calcium is seen at a *pH* above 11 in process water with the addition of calcium hydroxide.
- Once cation exchange occurs, there is an immediate benefit for settling but as more lime is added, less settling occurs.
- Similar to soil stabilization applications, the yield stress and Atterberg limit data suggest that strength development is dependent on the calcium hydroxide dosage and that geotechnical properties improve as the treated FFT ages.
- The role of organics in this system is important for both cation adsorption and settling. Calcium naphthenate formation could impact settling of FFT when calcium hydroxide is added at high *pH*.
- Waters released from calcium hydrate treated FFT can have high levels of *pH* and soluble calcium. Both characteristics are reduced over time as natural carbonation occurs.

Several areas were identified for further study:

- Further optimization of pressure filtration, vacuum filtration and centrifuge processing of the treated solids to high solids content treated FFT with minimum solids released in filtrate.
- Understanding calcium naphthenate formation and its effect on the settling rate.
- Further investigation of the impact of aging on the characteristics of water quality and FFT properties.
- Methods to reuse or treat release waters from lime treated FFT that are high *pH* and soluble calcium.

This work suggests that the use of calcium hydroxide as a coagulant can produce a material with good geotechnical properties for land reclamation. Like soil treatment applications, an additional benefit is that these materials appear to improve time. The basic science shows promise but further work is needed to optimize these results.

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Effect of floc size on geotechnical properties of oil sands fluid fine tailings: Experimental study

Jordan Elias
Natural Resources Canada, CanmetENERGY, Devon, AB, Canada

Nicholas Beier
University of Alberta, Edmonton, AB, Canada

ABSTRACT: Large volumes of fluid fine tailings (FFT) pose difficult engineering challenges because they do not readily dewater and due to their fluid state, require containment. A common approach to reduce FFT volume by water removal is to treat the clay suspension with a coagulant and flocculant to form highly porous aggregates called flocs. This work looked at the effect of increased floc size on geotechnical properties of FFT. Floc dimensions were determined using images captured during deposition of flocculated FFT. Geotechnical properties measured included compressibility, vane shear strength, and hydraulic conductivity.

1 INTRODUCTION

Oil sands fluid fine tailings (FFT) are an aqueous suspension consisting primarily of dispersed fine mineral particles along with residual bitumen and soluble inorganic salts and organics (Kasperski 1992). Large volumes of FFT pose difficult engineering challenges because they do not readily dewater and due to their fluid state, require containment. One closure goal for FFT is to convert the fluid tailings into a soil-like material having a low water content and adequate geotechnical strength to sustain dry landscape reclamation efforts. A common approach to treating FFT to achieve an adequate reclamation material is through coagulation and flocculation. In this paper coagulation refers to changing the suspension chemistry by the addition of an inorganic salt to promote the formation of larger aggregated particles. Flocculation refers to the agglomeration of coagulated particles using a polymer flocculant to form flocs.

The focus of this research was to investigate effects of FFT treatments on geotechnical properties and specifically, what effect did floc size and high coagulant dosage have on geotechnical properties of treated FFT. Previous studies have looked at the effect of floc size and shear on tailings properties and found that reduction in floc size by shear can affect permeability and shear strength. Some work has also been done on the effect on compressibility of high-dosage coagulant treatments. Treatment of FFT with coagulant and flocculant opens the door to an almost infinite number of treatment recipes due not only to the number and differences in possible components but also to the large number of suitable dosage combinations, so one goal of this study was to determine the optimum cation dosage for coagulation prior to flocculation. This study investigated the potential advantage to increasing floc size through treatment with a coagulant and flocculant. The paper describes the development of a floc size measurement system, a method to create measurable flocculated samples, and subsequent geotechnical characterization of the treated samples.

1.1 *Background*

Both floc size and high coagulant dosage have the potential to affect geotechnical properties of treated FFT. Work regarding the effect of shear and floc size on tailings properties has been carried out by Jeeravipoolvarn (2010), Jeeravipoolvarn et al. (2014), Derakhshandeh et al. (2016) and Mizani (2016). The effect of high coagulant dosage on consolidation properties of consolidated tailings was investigated by Boratynec (2003). Da Silva (2014) used high coagulant dosages and polymer flocculation to enhance tailings dewatering and consolidation in geotextile bags.

Jeeravipoolvarn (2010) compared compressibility of in-line thickened tailings (ILTT) where cyclone overflow tailings were treated with anionic and cationic polymer flocculants. The effect of shear on ILTT was studied by emulating pipeline transport shear then characterizing the resulting tailings product. While it was found that shearing ILTT substantially reduced floc size, it was concluded that shearing to reduce floc size had little to no effect on compressibility at void ratios less than 5 but can have marginal effects at a void ratio greater than 7.5. Jeeravipoolvarn et al. (2014) found that with ILTT, shear causing floc breakage reduced both permeability and vane shear strength. However, Derakhshandeh et al. (2016) found that shearing flocculated MFT had marginal effects on initial dewatering but overall increased the rate of dewatering. Derakhshandeh's conclusions supported the reduction in yield stress caused by shearing.

Mizani (2016) carried out a rheological study on polymer-amended FFT that included the effect of shear history on apparent yield stress. It was concluded that disturbances to floc structure induced by oscillatory rheometry were reversible and benefits of polymer addition could be retained. Boratynec (2003) studied the effect of increased gypsum dosages on the consolidation of composite tailings (CT). Results from consolidation testing showed little difference in compressibility while SEM micrographs showed more open structure at higher dosages. Da Silva (2014) investigated the effect of dewatering tailings in a geotextile bag as a means to improve dewatering and consolidation of treated FFT. The effect of high dosages of Ca^{2+} on zeta potential were studied and it was found that elevated dosages of Ca^{2+} reduced the zeta potential, and increased fines capture and total suspended solids.

2 MATERIALS AND METHODS

2.1 *Materials*

Two oil sands FFT samples were used for initial coagulant screening. They were analyzed to determine composition (bitumen, solids, and water), mineralogy, clay content by methylene blue index, particle size distribution, and water chemistry. Mineralogy as determined by XRD on Dean Stark solids is shown in Table 1. Dean Stark analysis for each FFT gave similar bitumen, water, and mineral fractions. However, mineralogy, particle size distribution, and methylene blue clay content revealed that the two FFT samples were quite different. Characterization data for the samples is also included in Table 1. Water chemistry for each FFT sample is shown in Table 2 with the most notable difference being the higher ionic strength of FFT-2 compared to FFT-1. FFT-1 was chosen for use in the bulk of the study since it was more similar in fines content and mineralogy to the cyclone overflow studied by Jeeravipoolvarn (2010).

Gypsum ($CaSO_4 \cdot 2H_2O$) and K-alum ($KAl(SO_4)_2 \cdot 12H_2O$) were used for coagulation. The flocculant used was a solution of a high molecular weight anionic polyacrylamide at a concentration of 0.4-wt%.

Table 1. Tailings characterization. Clay wt % as determined by methylene blue index.

	Composition (wt%)		
	FFT 1 – as received	FFT 2 – as received	Cyclone O/F (Jeeravipoolvarn 2010)
Kaolin in mineral solids	32.4	16.4	45.5
Illite in mineral solids	22.2	16.7	4.5
Quartz in mineral solids	36.0	60.1	41.0
< 45μm in mineral solids	96.1	74.3	94
Clay in mineral solids	74.1	57.9	-
Bitumen in FFT	4.3	3.6	3.3
Water in FFT	54.3	52.2	-
Mineral solids in FFT	39.9	43.3	-

Table 2. Water chemistry for fluid fine tailings samples

	Conductivity (mS/cm)	Aqueous Ion Concentration (mg/L) Ca^{2+}	Na^{+}	K^{+}	Mg^{2+}	$Al^{3+/2+}$	Cl^{-}	SO_4^{2-}	HCO_3^{-}	CO_3^{2-}	pH
FFT-1	1.75	23	378	15	12	0	216	2	750	0	8.08
FFT-2	2.93	9	551	7	4	0	222	7	1015	27	8.31

2.2 *Methods*

Floc size measurements were made using image capture followed by image analysis. This method was chosen for ease and ability to capture macroscopic flocs. To capture suitable images the tailings treatment recipe had to be designed such that there was a large contrast between flocs and water. Since a clear supernatant was important it was better to use higher coagulant dosages as a means to minimize turbidity. Coagulant dosage was optimized based on coagulant screening tests aimed at minimizing the capillary suction time (CST). From the CST dosage study each coagulant showed an obvious dosage whereby any further increase in dosage had minimal effect on CST. Flocculant dosages were chosen based on the minimum dosage where flocs were visible and supernatant was clear.

Coagulation testing was done by diluting FFT samples to 20-wt% solids with tap water (0.30 g/L total dissolved solids and conductivity of 0.37 mS/cm). Diluted FFT was treated in 300-g batches with either of the two coagulants, gypsum or K-alum, at varying dosages. Coagulant powder was added to the slurry while continuously mixing. Coagulated slurry was typically mixed for a minimum of 4 h after which CST testing was carried out (5 replicates). The remaining sample was used for water chemistry analysis to determine major inorganic ions, conductivity, and pH. Flocculation of coagulated FFT required further dilution to 10-wt% solids to better distinguish the flocs.

Creation of flocs in FFT needed to be repeatable to ensure consistent floc size distribution (FSD) for the same tailings treatments so the experimental setup was designed to run with a minimum of handling to mitigate human error. Flocculation was done in-line with both slurry and polymer pumped through a static mixer then into a vessel with a dynamic mixer. The dynamic mixer was a benchtop overhead mixer with a Visco Jet®-120 mm impellor and was included to provide extra mixing for improved flocculation and also to control floc size. From the dynamic mixing vessel the flocculated slurry was discharged from two ports, one to a sample container and one to a glass tank (30 cm tall, 50 cm wide, 5 cm deep) for floc-size measurements.

As flocs settled through the water in the tank they were photographed at approximately 30-s intervals using a Canon™ EOS 60D DSLR camera. The images were then analyzed using Axiovision 4.8™ image analysis software to determine the number of flocs and average diameter. Floc size data were converted from a frequency distribution to a mass fraction and plotted as cumulative percent versus average floc size diameter to obtain the FSD. See Figure 1 for the flocculation setup and Figure 2 for a typical photograph of flocs used in image analysis for floc size determination.

Two samples treated with the same dosage of K-alum and flocculant but mixed at different speeds were selected for consolidation testing (see Table 3 for conditions). The intent was to compare properties of samples with the same chemical amendments but with different FSD. Consolidation testing was carried out in a large strain consolidometer (LSC) according to the Geotechnical Centre at the University of Alberta procedure described by Jeeravipoolvarn (2005). Vane shear measurements were made during consolidation testing according to ASTM D4648-16 using laboratory vane apparatus VJT5300 made by VJ Tech. Shear strength was measured after each load step by first removing the load and decanting tailings water.

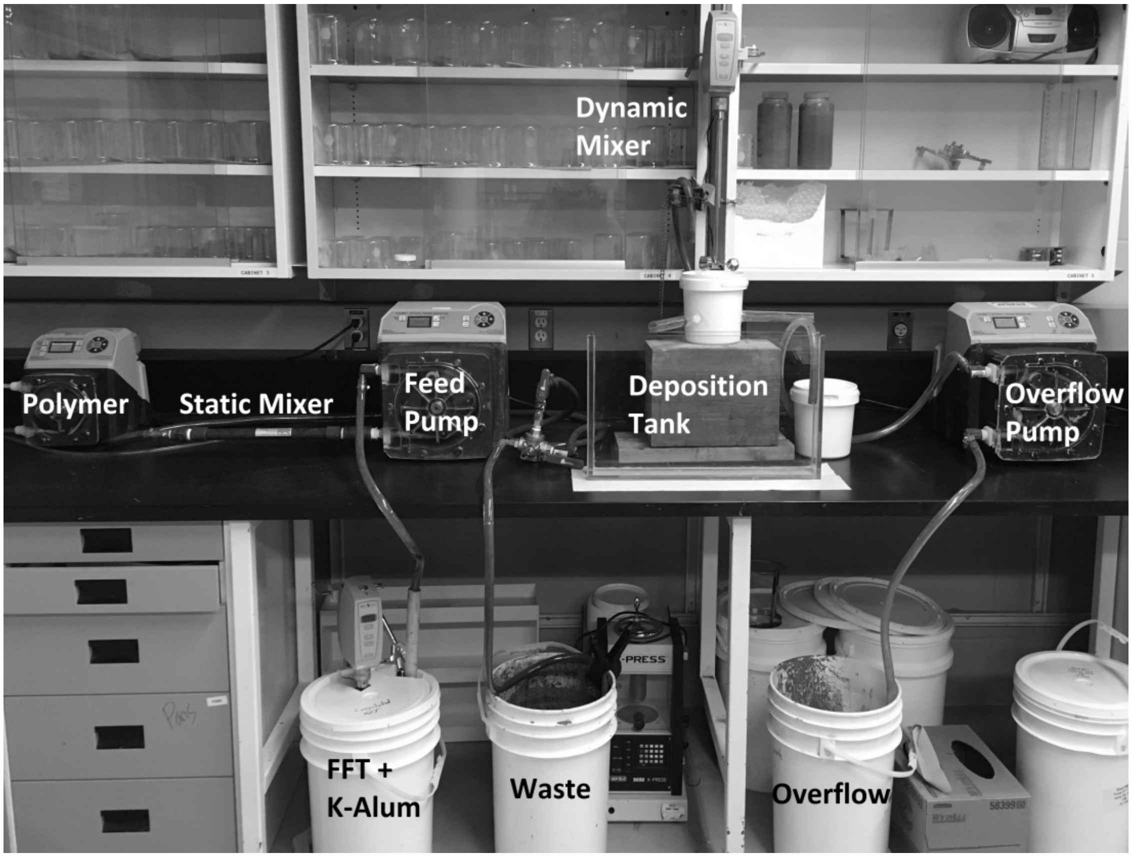

Figure 1. Flocculation setup with in-line polymer injection, static mixer, dynamic mixer, and deposition tank for floc size measurements

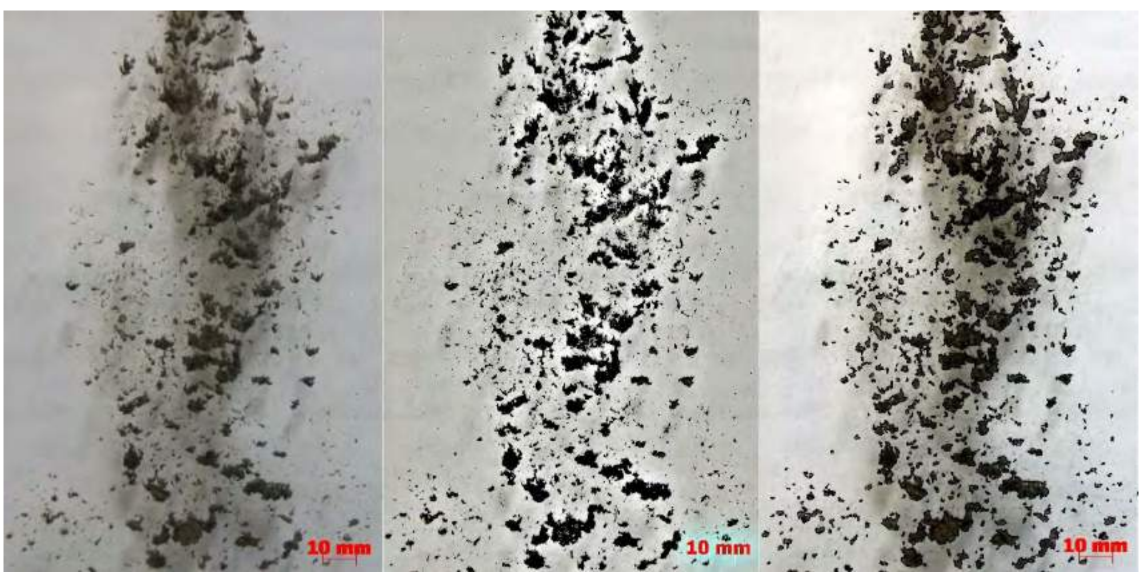

Figure 2. Photos of flocs and throughout image analysis process

Table 3. Sample and testing summary for flocculated FFT

FFT	Coagulant cation	Cation dosage (g/tonne solids)	Polymer dosage (g/tonne solids)	Solids content (wt%)	Dynamic mixing speed (RPM)	LSC testing
FFT-1	Al^{3+}	350	1245	9.19	350	X
FFT-1	Al^{3+}	350	1245	10.82	900	X

*K-alum dosage is 6154 g/tonne solids

3 RESULTS AND DISCUSSION

Coagulation screening results for FFT are shown in Figure 3. Coagulant dosages are plotted in terms of the multivalent metal cation rather than the entire salt to simplify dosage comparisons. These dosage plots were used to select suitable coagulant dosages for conditioning FFT prior to flocculation with polymer. Dosages were selected near the inflection point of the curve since an increase in dosage beyond this point did not yield much benefit in terms of CST reduction. Dosages obtained from the inflections points in Figure 3 were 450 g/tonne for Ca^{2+} and 350 g/tonne for Al^{3+}. These dosages were deemed as the optimum coagulant dosage for the purpose of this work although only K-alum was used for flocculation studies.

Flocculated FFT samples (coagulated with K-alum) with different FSDs were made by mixing with the dynamic mixer at either 350 RPM or 900 RPM during flocculation. A mixing speed greater than 900 RPM yielded flocs too small to be measured by the image capture and analysis technique, while mixing below 350 RPM did not fully mix the polymer into the tailings. Insufficient mixing was judged by observation of free polymer on the surface of the tailings slurry.

Figure 4 shows the FSD of the two flocculated samples while Table 4 shows the summary statistics. Although there were differences in the number of smaller flocs there was not much difference in the number of larger flocs. Even though the overall FSD of the two samples were similar each sample was still subjected to geotechnical testing. Both FSD and geotechnical test results are compared to those reported by Jeeravipoolvarn (2010) with FSD being compared in Figure 4.

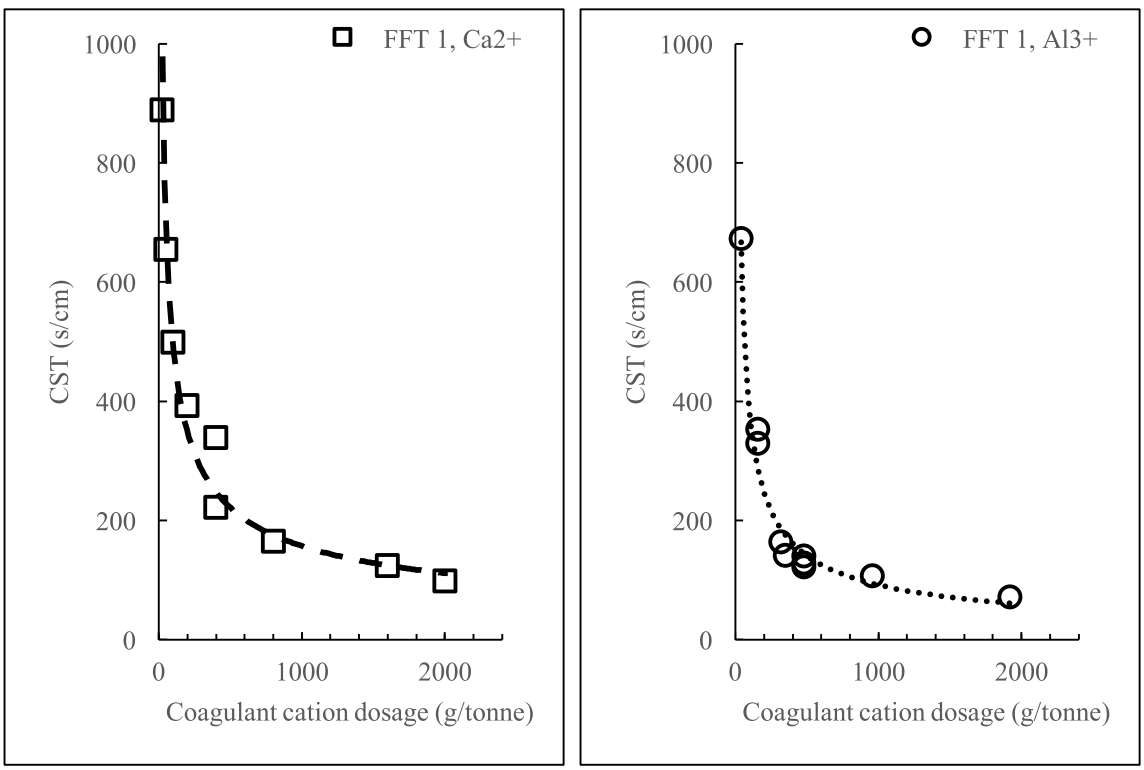

Figure 3. Coagulant dosage screening for FFT-1 using K-alum and gypsum as coagulants

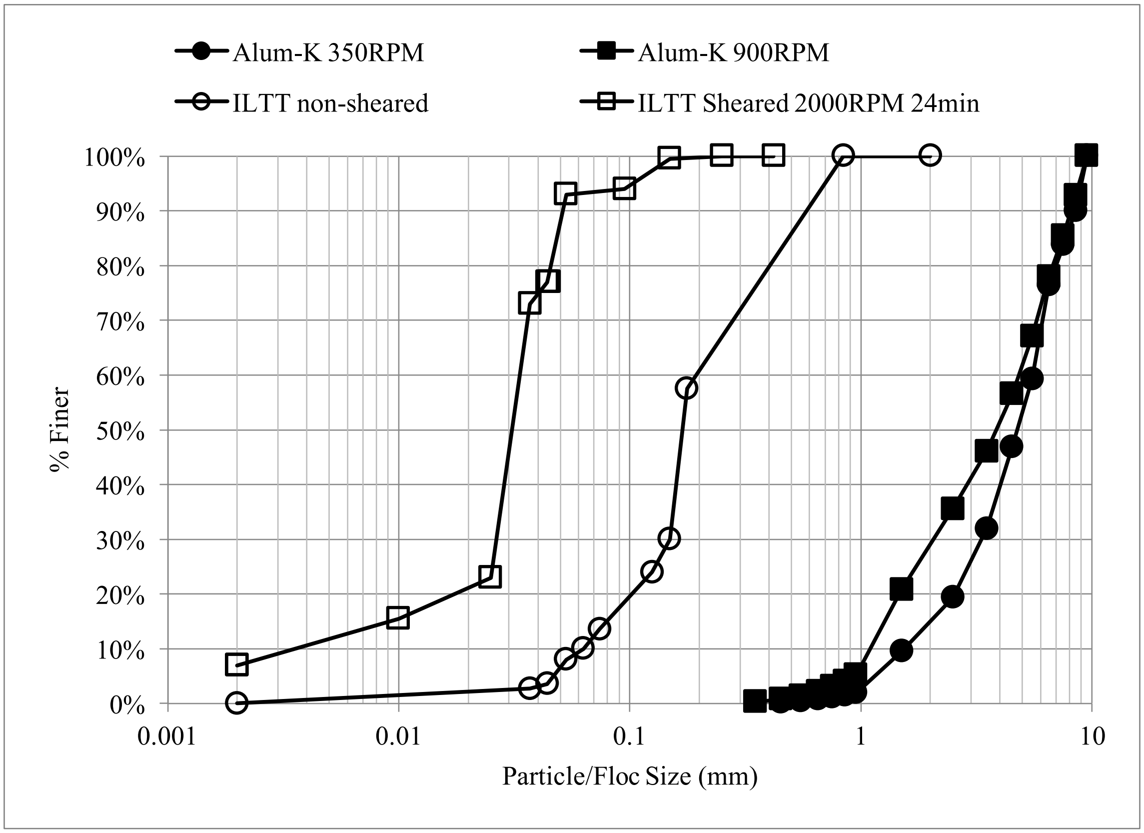

Figure 4. Floc size distribution for treated FFT from this study and in-line thickened tailings (ILTT) from Jeeravipoolvarn (2010). Flocs in this study measured from images while Jeeravipoolvarn used a combination of sieve and hydrometer

Table 4. Summary statistics for flocculated FFT-1 coagulated with K-alum with dynamic mixing set to either 350 RPM or 900 RPM

	Flocculation with dynamic mixer at 350 RPM	Flocculation with dynamic mixer at 900 RPM
D90 (mm)	8.5	8.1
D75 (mm)	6.4	6.2
D50 (mm)	4.8	3.9
D25 (mm)	3.0	1.8
D10 (mm)	1.5	1.1

Compressibility was assessed with LSC testing on the two flocculated FFT samples coagulated with K-alum. The relationship between void ratio and effective stress are shown in Figure 5 along with the increase in vane shear strength with reduction in void ratio. Marginal differences in compressibility were seen between FFT samples having different FSD at a void ratio greater than 4, but appeared to compress the same at a void ratio less than 4. Compressibility and vane shear strength as reported by Jeeravipoolvarn (2010) are also shown in Figure 5 for comparison. Flocculated FFT samples, having larger flocs than ILTT, appeared to be less compressible. Vane shear strength at lower effective stress was greater for the two samples with larger flocs while shear strengths at higher effective stresses were the same for all samples. Figure 6 shows hydraulic conductivity as a function of void ratio for each FFT sample and non-sheared ILTT. Considering the variability in hydraulic conductivity typically observed for treated oil sands tailings (Jeeravipoolvarn 2005) the differences observed in Figure 6 are not significant.

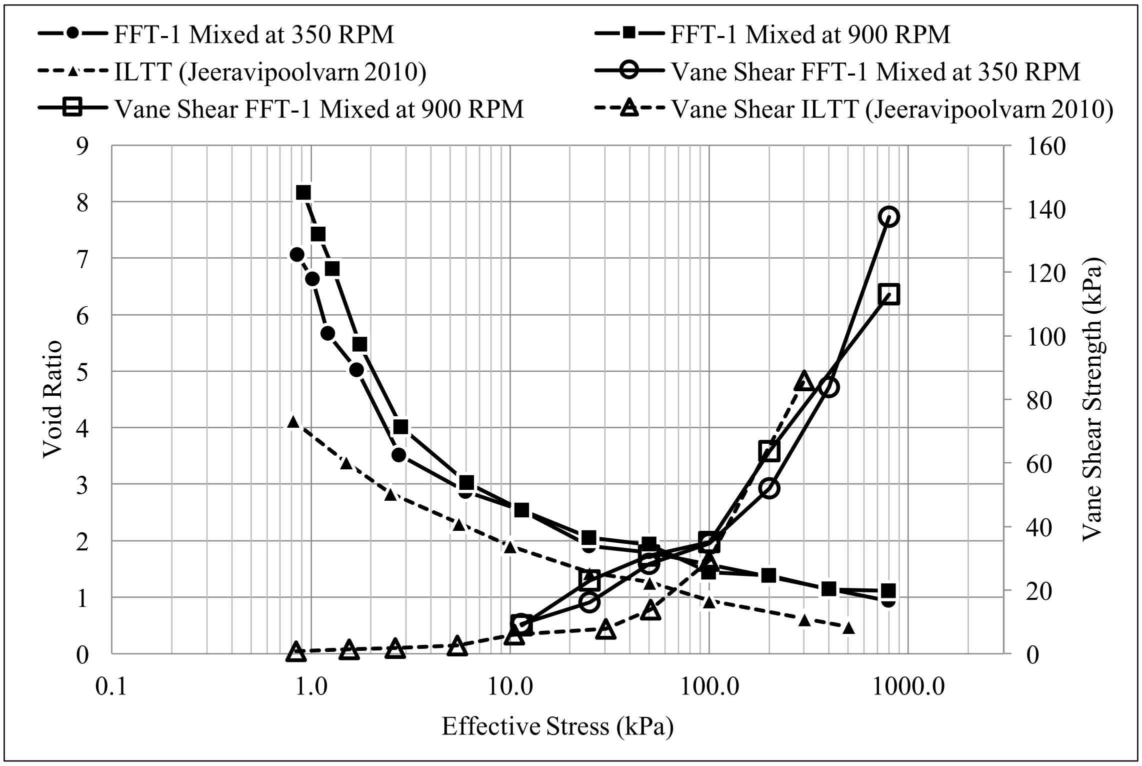

Figure 5. Compressibility (solid symbols) and vane shear strength (open symbols) for FFT-1, and ILTT as reported by Jeeravipoolvarn (2010)

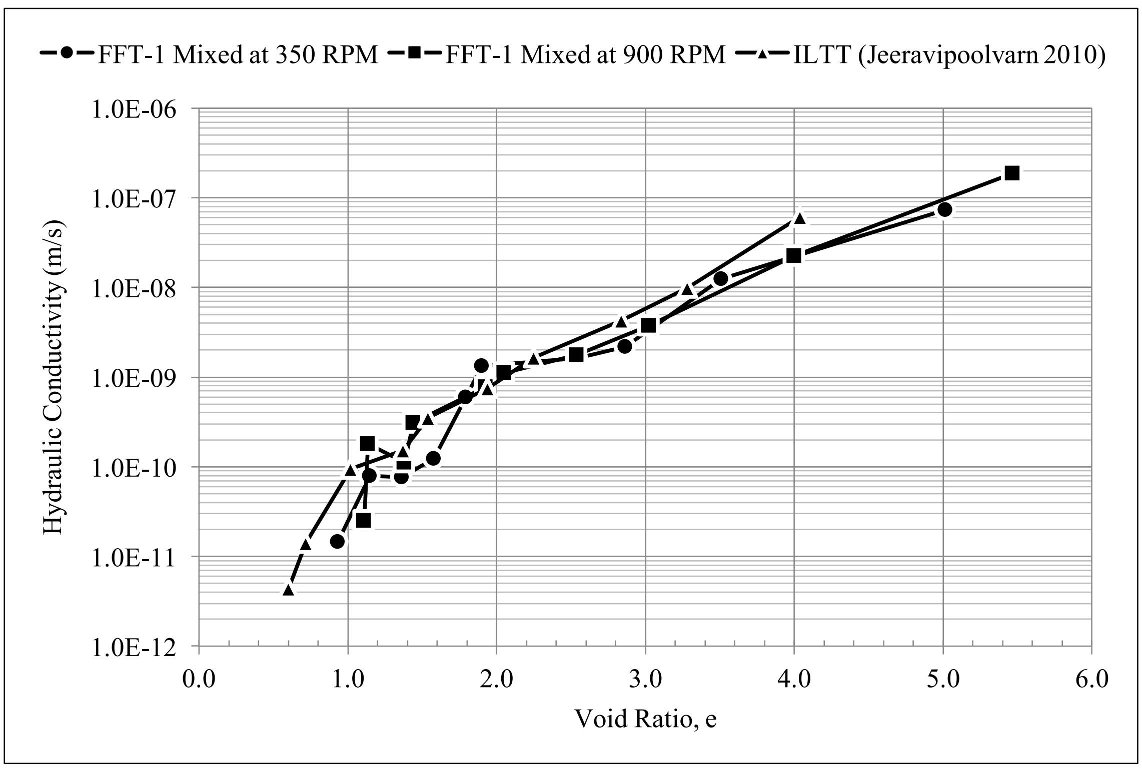

Figure 6. Hydraulic conductivity of FFT-1 mixed at 350 RPM or 900 RPM compared to non-sheared ILTT Jeeravipoolvarn (2010)

4 CONCLUSIONS

The protocol developed in this study was effective at determining floc sizes and in producing bulk samples for further testing. Limitations to the flocculation setup were that a low solids content and high coagulant dosage were required to allow for the necessary contrast for image analysis. This protocol is better for the creation and analysis of larger floc sizes; study of smaller flocs (less than about 0.3 mm) was problematic because of limitations of the imaging system.

Coagulant dosage screening using CST worked well as a method to select a suitable dosage to minimize turbidity and avoid unnecessary overdosing of coagulant. The high coagulant dosages used were suitable for this study but are not necessarily optimal for field application.

For the flocculated FFT in this study there were only minimal differences in compressibility for the different FSDs at lower solids contents; at higher solids contents of the two different FSDs there was no difference in compressibility, shear strength, or hydraulic conductivity. Differences in FSD are apparent when comparing flocculated FFT from this study to non-sheared ILTT. These differences in FSD may be responsible for lower compressibility of flocculated FFT samples compared to ILTT but since the initial tailings materials are different it is difficult to make this claim. Flocculated FFT, having larger flocs than ILTT, exhibits higher vane shear strength up to 50 kPa effective stress but converges after 100 kPa effective stress.

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Co-disposal in the Oil Sands: Blending of FFT and overburden materials

Ralph Burden & G Ward Wilson
University of Alberta, Edmonton, Alberta, Canada

ABSTRACT: Co-disposal of fine tailings and waste rock or overburden material is an emerging technology which has the potential to significantly improve the way mine waste is managed. In the oil sands this approach could offer an effective and economical way of dewatering and managing the large volumes of fine tailings that have thus far proven problematic. A conceptual model, based on the approach used by Wickland et al. (2006), has been developed. It is proposed that the mix ratio can be used as the principle parameter to control the properties of a blend, and that the conceptual model can be used to predict the properties of a blend of a given mix ratio. It is shown how simple compaction tests can be used to estimate the relationship between mix ratio and configuration at the time of placement. There is a need for further research to understand the moisture transfer kinetics and model the long-term behaviour of these deposits, and a need for more experimental verification of the conceptual model.

1 INTRODUCTION

Research at base and precious metal mines has shown that waste rock and fine tailings can be blended to produce a new material consisting of a waste rock skeleton with fine tailings occupying the void spaces (Wilson et al. 2008). This material has been shown to have favorable properties; it combines the high shear strength and low compressibility of the waste rock skeleton with the water retention properties and low permeability of the tailings. In practice this reduces the total volume of waste storage required, reduces the need for containment of fine tailings and can mitigate the problem of oxidation that can be associated with waste rock dumps.

This paper discusses the application of this “co-disposal” technique to oil sands mining; blending Fluid Fine Tailings (FFT) with Clearwater shale overburden. Dewatering of FFT remains a significant, largely unresolved challenge, and represents a barrier to reclamation and closure. Whilst most approaches currently in use or development aim to increase the solids content of the FFT by removing water, essentially here we are increasing the solids content by adding solids. This has great potential because of the high water demand of the overburden material. Upon mixing, water transferred from the FFT to the shale, promoting rapid dewatering and strength gain.

2 BACKGROUND

2.1 *Geological background*

The Clearwater formation is the predominant deposit overlying the McMuarry (oil sand) formation, in the main mining region on the west side of the Athabasaca River. It is well described and characterised in the literature (Isaac, Dusseault et al. 1982, O'Donnell and Jodrey 1984); only a brief summary is given here. It consists mainly of silts (around 50%), marine clay shales (around 44%) and some beach and shoreface sands (around 6%), deposited in a mixed marine and continental environment. The clay fraction is mainly composed of illite and smectite. The Clearwater formation on Syncrude's lease was found to consist of 7 sub-layers, each readily identifiable by a distinct geophysical response. These are informally identified by Syncrude as KCW, A, B, C, D, E and F respectively.

It is noteworthy that the in-situ moisture content of the Clearwater formation is generally at or below the plastic limit. This is significant because it implies that the shale has a high water demand; upon mixing it will draw the water out of the tailings, causing rapid dewatering and strength gain.

2.2 *Previous studies on FFT and Clearwater Shale mixtures*

Several studies were carried out in the 1980s to investigate the properties of clay shale overburden lumps, or "balls", blended with Mature Fine Tailings (MFT). A large scale field trial was conducted at Syncrude (Lord and Isaac 1989), in addition to several laboratory-based studies (Ash 1987, Dusseault and Ash 1987, Mimura 1990). The focus was on creating hydraulically placed overburden dumps rather than on tailings disposal, which at the time was not considered to be the priority area that it is today. However, the work is worthy of re-examination in this context. A comprehensive review of earlier work is given by Burden and Wilson (2016). The concept has recently been revisited by Syncrude (Mikula, Wang et al. 2016), using conveyors to blend and stack the materials, as opposed to hydraulic placement.

3 CONCEPTUAL MODEL

The objective of the conceptual model is to develop a theoretical basis for the prediction of the geotechnical properties of the blends; thus allowing design of blends with specific geotechnical properties.

3.1 *Blend structure and configuration*

In practice, blends of shale overburden and FFT are not perfectly mixed but consist of "lumps" of shale, with FFT and/or air occupying the void space between lumps. Wickland, Wilson et al. (2006) showed that blends of waste rock and fine tailings can exist in 3 distinct structural configurations. The same principle may be applied by likening the waste rock particles to shale "lumps". This is shown schematically in Figure 3-1.

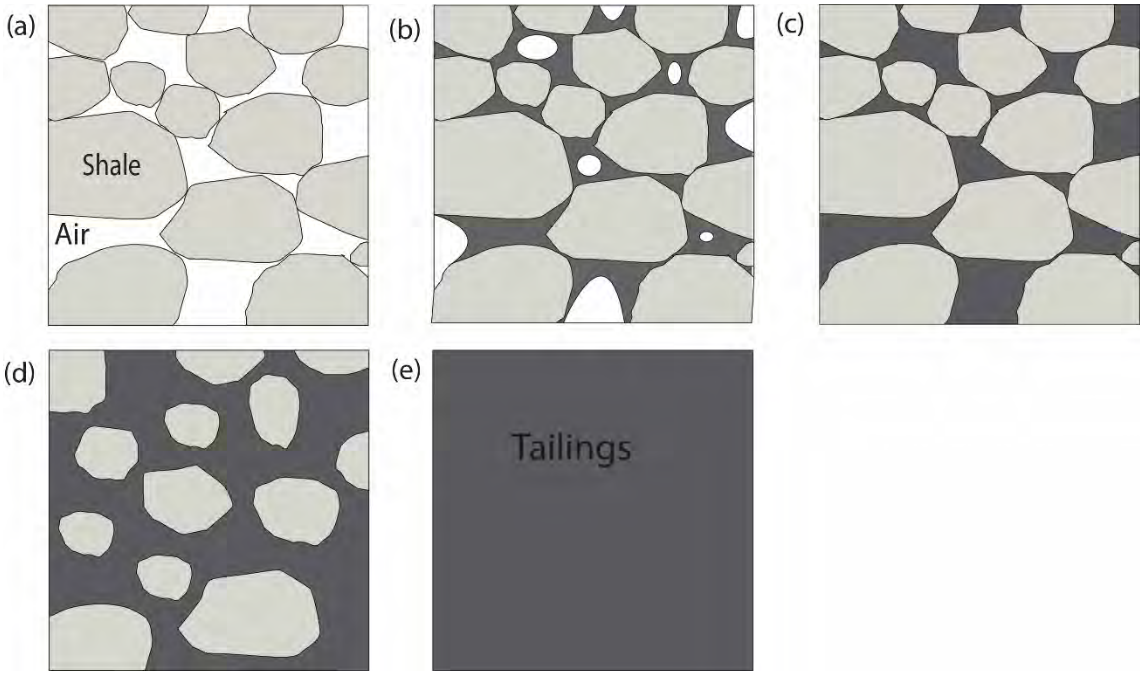

Figure 3-1. Particle structure configurations for Clearwater shale - FFT blends: (a) Shale lumps only; (b) Shale lump matrix partly filled with FFT; (c) "Just filled" condition; (d) "Floating" shale lumps in an FFT matrix; (e) FFT only

This “macro scale” particle configuration is critically important because, in the most general sense, it defines the properties and behavior of the blend, for example compressibility, permeability, pore-pressure response etc.

At the time of blending, the shale lumps are unsaturated and contain shale solids, porewater and air voids. The internal structure of a shale lump is shown schematically in Figure 3-2.

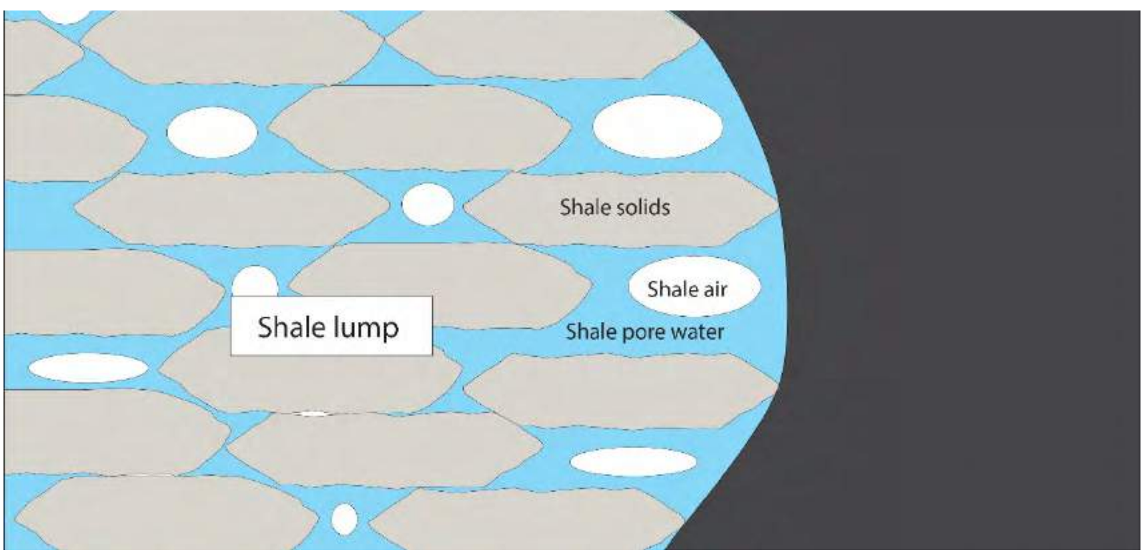

Figure 3-2. Schematic showing internal structure of shale “lump”

It is convenient and often reasonable to assume that the tailings are fully saturated. However, experience has shown that the tailings often contain entrained air bubbles; this is particularly true in the case of centrifuge cakes. This may be considered a separate air phase in the conceptual model. In summary, 3 air phases are proposed as follows: Air in the unsaturated shale pores, occluded air in the fluid tailings and “macro”-scale air voids.

3.2 *Phase relationships*

Figure 3-1 shows the phase diagram, and the notation used in this paper to describe the mass and volume of respective phases.

Volume V	Shale volume V_s	Shale solids V_{ss}
		Shale water V_{sw}
		Shale air voids V_{sa}
	Tailings volume V_t	Mineral solids V_{ts}
		Bitumen V_{bit}
		Tailings water V_{tw}
		Tailings air voids V_{ta}
	Macro air voids V_{ma}	

Mass M	Shale mass M_s	Shale solids M_{ss}
		Shale water M_{sw}
	Tailings mass M_t	Mineral solids M_{ts}
		Bitumen M_{bit}
		Tailings water M_{tw}

Figure 3-3. Phase relationships and notation for shale – fine tailings blends

The blend can be characterised using a global void ratio, defined as:

$$e = \frac{Total\ volume\ of\ voids}{Total\ volume\ of\ solids} \quad or \quad e = \frac{V_{sw} + V_{sa} + V_{tw} + V_{ta} + V_{ma}}{V_{ss} + V_{ts} + V_{sw} + V_{bit}} \qquad [1]$$

The "macro scale", or shale lump skeleton void ratio, is defined as:

$$e_m = \frac{V_t + V_{ma}}{V_s} \qquad [2]$$

In addition, we have the internal shale lump void ratio,

$$e_s = \frac{V_{sw} + V_{sa}}{V_{ss}} \qquad [3]$$

and the tailings void ratio:

$$e_t = \frac{V_{tw} + V_{ta}}{V_{ts} + V_{bit}} \qquad [4]$$

3.3 *Mix ratio*

In general terms, the principal variables that govern the properties of the blend are the minerology of the shale and the FFT, the particle size distributions of the shale and the FFT, the initial moisture contents of the shale and FFT, the bitumen content of the FFT and the mix ratio. The "macro" scale particle size distribution, i.e. the sizes of the shale lumps, must also be considered. In practical terms, the mix ratio is the main design parameter that can be used to control the properties of the blend.

A convenient mix ratio parameter for producing blends in practice is Bulk Mass Ratio (BMR), defined:

$$BMR = \frac{Bulk\ mass\ of\ shale}{Bulk\ mass\ of\ tailings} \quad or \quad BMR = \frac{M_{ss} + M_{sw}}{M_{ts} + M_{tw} + M_{bit}} \qquad [5]$$

Wickland, Wilson et al. (2006) proposed the mix ratio parameter R, defined as the ratio of waste rock to tailings by dry mass, as the primary design variable for waste rock and tailings blends. This is a useful parameter for defining mix ratio when investigating the relationship between the mix ratio and properties of the blend, since it is independent of variations in the properties of the original constituents of the mix, such as moisture content, bitumen content and void ratio. Note that for the purposes of this paper, R is defined as:

$$R = \frac{Mass\ of\ shale\ solids}{Mass\ of\ tailings\ mineral\ solids} \quad or \quad R = \frac{M_{ss}}{M_{ts}} \qquad [6]$$

Given that gravimetric moisture content (w) and bitumen content (b) of the constituents of the blend can easily be measured, it is straightforward to calculate the mass proportion of a blend at a given mix ratio. Figure 3-4 shows the mass proportion diagram for a typical FFT-Clearwater shale blend.

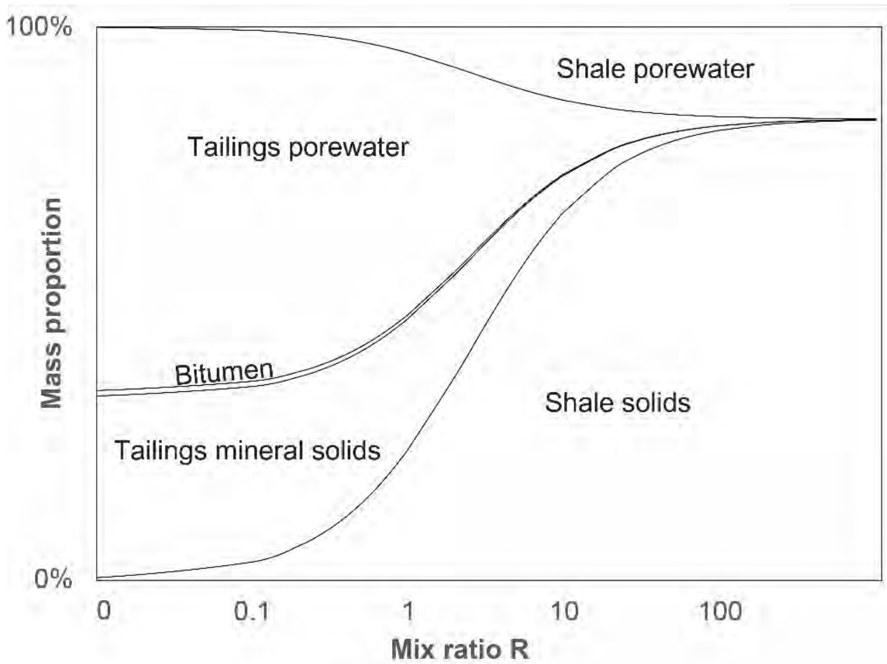

Figure 3-4. Mass proportion with respect to mix ratio for a typical FFT – Clearwater shale blend

Solids content (s or s_m) is widely used in the industry to characterise tailings. It is useful to express mix ratio in terms of the final solids content, defined:

$$s = \frac{M_{ss}+M_{ts}+M_{bit}}{M_{ss}+M_{ts}+M_{sw}+M_{tw}+M_{bit}} \qquad [7]$$

$$s_m = \frac{M_{ss}+M_{ts}}{M_{ss}+M_{ts}+M_{sw}+M_{tw}+M_{bit}} \qquad [8]$$

Given that gravimetric moisture content (w) and bitumen content (b) of the constituents of the blend can easily be measured, it is straightforward to calculate the relationships between the different mix ratio parameters, and these can be used interchangeably. Figure 3-5 shows these relationships for a typical FFT-Clearwater shale blend.

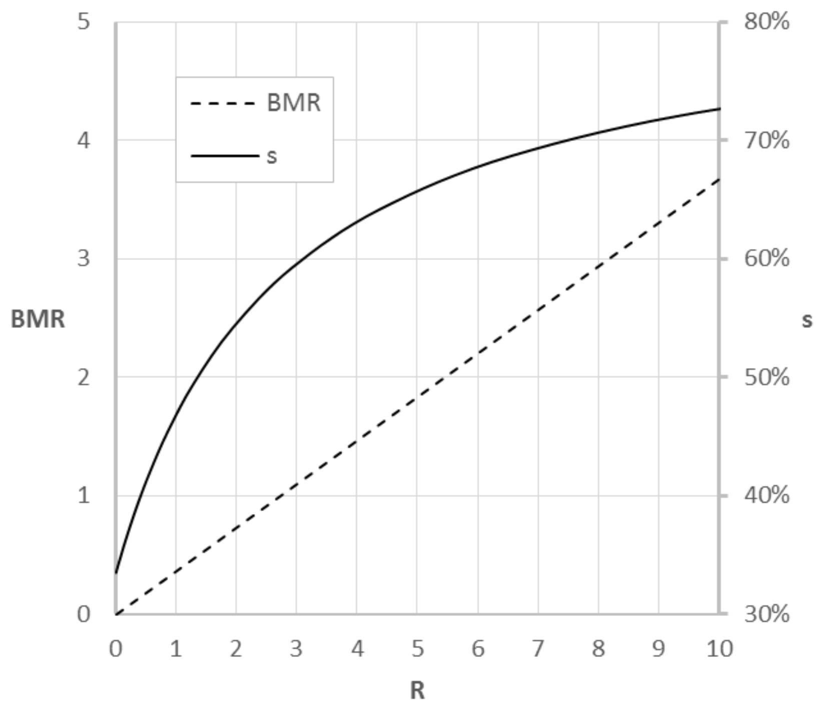

Figure 3-5. Bulk Mass Ratio (BMR) and solids content (s) with respect to dry mass ratio (R) for a typical FFT-Clearwater shale blend (w_s = 20%, w_t = 200%, b = 8%).

3.4 *Predicting configuration from mix ratio*

Configuration is important because it defines the geotechnical behavior of the blend. In practice, controlling mix ratio is the only way in which we can control the properties of the blend. Therefore, to design a blend with specific geotechnical properties, we need to predict configuration from mix ratio.

3.4.1 *The critical mix ratio*

The "just filled" condition is represented by a critical mix ratio, denoted R_{crit}. If tailings are added to a blend in the "just filled" condition ($R < R_{crit}$), then the shale lumps will move apart and the blend will be in the "floating" condition, shown in Figure 3-1 (b). If tailings are removed from a blend of a constant volume in the "just filled" condition ($R > R_{crit}$), then the blend will be in the unsaturated condition shown in Figure 3-1 (d). The critical mix ratio (R_{crit}) is a function of the void ratios of the constituents of the blend (e_s and e_t) and the macro-scale (shale lump structure) void ratio (e_m).

4 EXPERIMENTAL MIXING TRIAL

This section describes a simple experimental mixing trial. The objective was to verify the conceptual model described in the preceding section, and to predict the critical mix ratio.

4.1 *Predicting the critical mix ratio*

Experimental measurements of e_m, e_s and e_t are difficult. Prior to mixing, tailings void ratio (e_t) can easily be measured. The internal shale lump void ratio (e_s) may be assumed to be equal to the in-situ void ratio of the shale at initial conditions. Tailings may be assumed to be fully saturated. Given these assumptions, the full volumetric proportions may then be back-calculated from measurement of total volume.

4.2 *The Proctor test*

The Proctor test is a useful means of evaluating the properties of blends of different mixture ratios because it is quick, simple, well established in geotechnical practice and allows for easy volume measurement. Tests were conducted on a KCA Clearwater shale – Centrifuge cake blend, starting with pure KCA at its natural moisture content (w=22.6%). Blends used for subsequent tests were wetted by adding centrifuge cake (w=90.9%) and mixed by hand. In all cases the KCA was passed through a No. 4 sieve prior to mixing. At each stage, the blend was compacted using the Standard Proctor method and the bulk mass and gravimetric moisture content are measured. Given that the volume of the mold is known, the bulk density can easily be calculated. The test was repeated using the modified Proctor method.

4.3 *Results and discussion*

The standard Proctor compaction curve is shown in Figure 4-1. The dashed lines represent global degrees of saturation (S_r) of 100% and 80%.

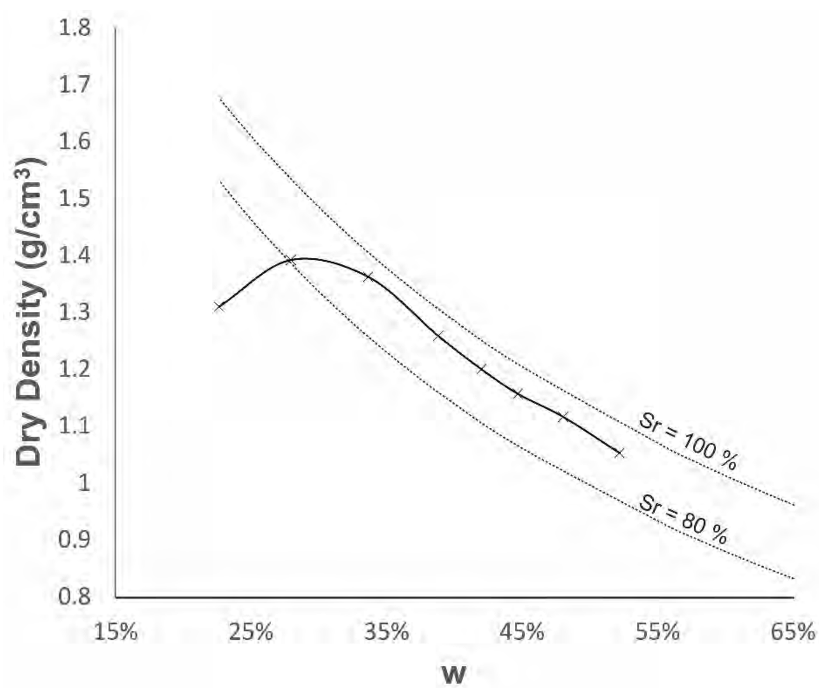

Figure 4-1. Standard Proctor compaction curve for Centrifuge cake – KCA Clearwater shale blend.

Given that the water content of the final blend, the water contents of the shale and tailings and the bitumen content of tailings are known, if perfect mixing is assumed the mass proportion of all of the phases can be evaluated and the mixture ratio can be back-calculated. Figure 4-2 shows the dry density with respect to mix ratio. Theoretical curves representing the fully saturated condition, and zero "macro" air voids, using an assumed e_s of 0.7, and assuming the tailings are fully saturated, are plotted.

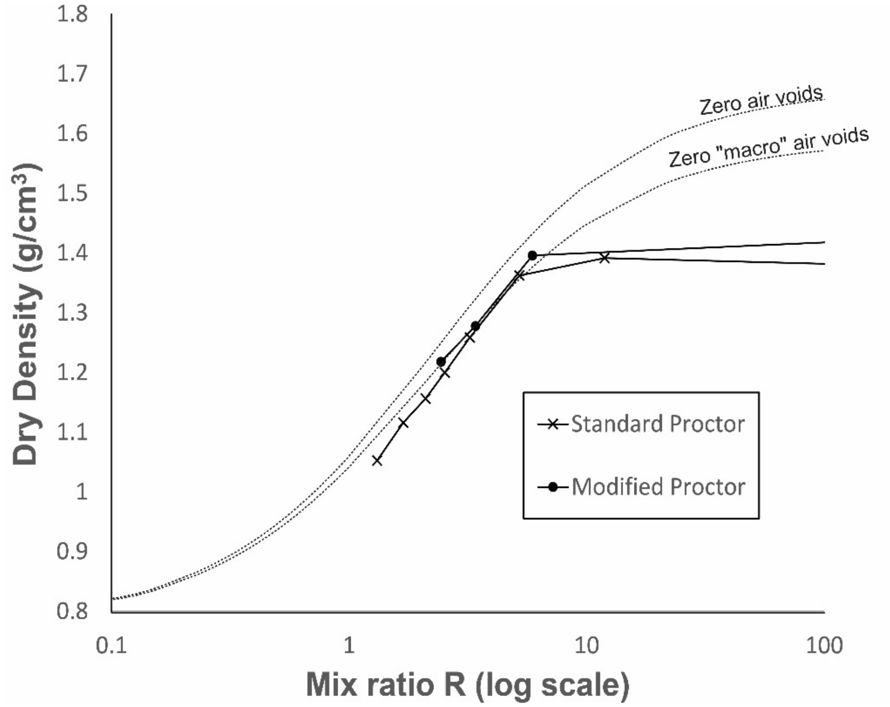

Figure 4-2. Dry density with respect to mix ratio for Centrifuge cake – KCA Clearwater shale blends compacted using the standard and modified Proctor method.

If the points plot on the zero macro air voids line, that implies that the blend is in either the "floating" or "just filled" configuration (Figure 3-1 (c) and (d)). If the points are below the line, that implies that they are in unsaturated configuration (Figure 3-1 (b)). Based on the conceptual model, we would expect that points would not plot above this line irrespective of the degree of compaction. In other words, 100% saturation will not be achieved, because the floating shale lumps remain unsaturated.

In general, the results show the expected behavior. At low mix ratios the blends are in the "floating" configuration; shale lumps floating in a fine tailings matrix. When the mix ratio is higher than a critical value (R_{crit}) the blends consist of shale lumps, fine tailings and macro scale air voids. The point where the blends become unsaturated with increasing mix ratio represents the "just filled" point, where the blend has optimum density. As might be expected, the critical mix ratio is slightly higher for the modified Proctor than the standard, suggesting that R_{crit} is a function not only of the material properties but also of degree of compaction or confining stress. The critical mix ratio for this material appears to be around R = 6, corresponding to a final solids content (s_m) of around 75 % and a Bulk Mass Ratio (BMR) of 3:1 shale : tailings. This suggests that blends optimised for tailings disposal will generally be in the "floating" configuration.

It should be noted that conceptual model introduced here is generally only valid at initial conditions. Upon blending, moisture is transferred from the tailings to the shale under a suction gradient, and the shale lumps will swell. This is a complex process with many variables, and is the subject of on-going study. Whilst it is still useful to develop a conceptual model to characterise the properties of the material at the time of placement, further research is needed to model long-term behavior.

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Oil Sands Tailings Consolidation

Geotechnical characterization of a frozen and thawed centrifuge product

Louis K. Kabwe, G. Ward Wilson, Nicholas A. Beier, Kelsey A. Cunningham & J. Don Scott
University of Alberta, Edmonton, AB, Canada

ABSTRACT: Centrifugation is a commercially deployed technology to increase the solids content of fluid fine tailings. In an initial study, a 1-meter thick centrifuge product was subjected to one cycle of freeze and thaw (F/T) treatment in a flume kept under laboratory controlled conditions. The flume was then left in the cold room kept at about 0^oC for more than two years. This study focuses on the geotechnical characterization of the two-year old frozen and thawed centrifuge product (about 60 cm thick). The geotechnical properties being studied include the solids content profiles, shear strength and suction profiles and large strain consolidation. Results of large strain consolidation tests indicate that the F/T treated centrifuge product has a considerably greater hydraulic conductivity than the untreated centrifuge product and therefore will consolidate much faster. Results also show that both the untreated and F/T treated products have approximately the same shear strengths but indicate that centrifugation coupled with F/T treatments alone cannot achieve the levels of solids content and shear strength necessary to establish a trafficable surface. Additional technologies such as surcharge and self-weight consolidation or drying/shrinkage are required to further increase solids content and shear strength.

1 INTRODUCTION

By 2012, there were already 22 tailings technologies in commercial use to dewater fine fluid tailings (FFT) and thereby increase the solids content. Many of these technologies are mature and others are in an advanced development stage (Sobkowicz 2012; Fair and Beier 2012). A promising technology is to add flocculants to the fluid fine tailings and use thickeners or centrifuges to increase the solids content (Kabwe et al. 2015). A further technology would be to then use freeze/thaw processes to further thicken the tailings. An additional promising technology is to thicken the tailings by atmospheric drying. The objective of the research reported in this paper was to characterize a two-year old thick centrifuge product that was subjected to one-cycle of freeze and thaw treatment then coupled with self-weight consolidation and evaporation under laboratory controlled conditions. Large strain consolidation tests with shear strength measurements were also performed on the F/T treated and on the untreated products to evaluate and compare the effects of F/T treatment on consolidation and shear strength of these centrifuged tailings products.

2 MATERIAL AND METHODS

2.1 Characterization of the Centrifuge Product

A sample of fine tailings was treated with a flocculant and gypsum ($CaSO_4$) prior to centrifugation and then shipped to the University of Alberta Geotechnical Centre in a 200-L drum. The methods of production of the flocculated centrifuged tailings product (FCTP) samples is beyond the scope of this paper and is not discussed. The solids content, void ratio, specific gravity and index properties of the FCTP sample (Table 1) were determined upon delivery to the University of Alberta Geotechnical Centre. The initial properties of the samples immediately after centrifuging were not provided.

Table 1. Initial centrifuge product characteristics

Characteristics	Initial	4 months	32 months
Solids content	55.5%	63%	76%
Void ratio	2.08	1.52	0.81
Specific gravity	2.58		
Fines content	87%		
Liquid limit	57%		
Plastic limit	26%		
Plasticity	32%		
Bitumen	0.37%		
Shear Strength	0.4 kPa	5 kPa	75 kPa

Table 1 shows the initial, after 4 months and after 34 months properties of the FCTP sample. The initial solids and fine contents of the FCTP sample were 55.5% and 87% respectively. The FCTP sample had a liquid limit and plastic limit of 57% and 26% respectively. The FCTP sample had a bitumen content (by total mass) of 0.37%.

The flume filling was completed about the end October, 2014 and the flume was allowed to self-weight consolidate and drain for about 2 weeks before freezing commenced. The total freezing time was 38 days and when freezing was complete, thawing commenced for 53 days during which time self-weight consolidation and atmospheric drying continued. The first characterization test then was performed 10 days after thawing was complete or about 4 months after flume filling. The second characterization test was performed about 28 months later or 32 months after flume filling.

2.2 *Frozen and Thawed Treated Centrifuge Product*

Figure 1 presents the product flumes before (A) and after (B) the freeze and thaw (F/T) treatment respectively. The length, width and height of the flume were 1.5 m, 0.5 m and 1.0 m respectively. The flume was installed at an angle of about 1.5 degrees to allow drainage during thawing. The sample was frozen one-dimensionally from the top using a freezing plate with a gradient temperature of -24 $^{o}C/m$ for about a month and then permitted to thaw by warning the room to about 22 ^{o}C. The freezing plate was removed after thawing and the product was exposed to room conditions. The method and description of the F/T treatment are beyond the scope of this paper and are not discussed. This paper discusses only the post freeze/thaw treatment of the product as shown in Figure 1(B). The initial thickness of the product before the F/T treatment was 90 cm and was reduced to about 65 cm after the F/T treatment. The flume was kept in a freezing room kept between 0 and 2 ^{o}C.

Figure 1. (A) Filling of the Flume with centrifuge product (B) the product after freeze/thaw treatment.

2.3 *Large Strain Consolidation*

A large strain consolidation testing apparatus (slurry consolidometer) (Figure 2) was used to determine the consolidation characteristics of the FCTP tailings samples and provide the compressibility and hydraulic conductivity relationships. As well, vane shear tests at each effective stress (> 1 kPa) were performed to provide undrained shear strength (void ratio-shear strength relationship).

The large strain consolidation apparatus used in this research confined the slurried material in a consolidation cell 10 cm in diameter x 15.5 cm in height (Scott et al. 2008). Drainage during consolidation was only upward with the bottom sealed. A plate load of about 1 kPa was applied as the first load. Subsequent loads were applied by dead loads up to about 10 kPa and then by an air pressure bellofram. The vertical stresses were doubled for each load step until the maximum vertical stress was reached (500 kPa in these tests). During consolidation, the change in height of the sample was monitored and plotted against time. When the height change stopped at each load step it was assumed that consolidation was complete for that load step. At this stage the excess pore pressure was also monitored at the base of the sample to ensure that the excess pore pressure had fully dissipated. The hydraulic conductivity was measured at the end of consolidation for each load step. An upward flow constant head test was performed with the head loss being kept small enough so that seepage forces would not exceed the applied stress. The undrained shear strength was also measured following the hydraulic conductivity test. The sample surface was exposed, and the shear strength was measured using a Rheometer for strengths up to 1 kPa. A vane shear apparatus was used for shear strengths greater than 1 kPa. A subsequent load was then applied after the shear strength measurement.

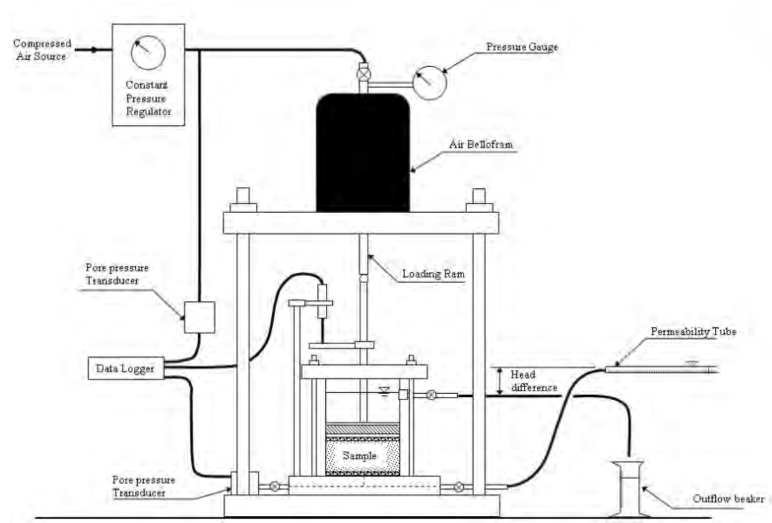

Figure 2. Schematic diagram of large strain consolidation set up at the Geotechnical Centre of the University of Alberta.

2.4 *Sampling and Measurement*

Moisture and solids contents were determined by oven-drying using the ASTM standard test procedure. Shear strength was measured using a motorized vane shear apparatus. Matric suction was measured using a T5 tensiometer connected to an Infield device and temperature was measured using a digital temperature probe. These properties were measured from the same sample specimen. The T5 tensiometer was first inserted into the product at a specified depth until the reading became stable. The tensiometer was then removed from the product and in-situ shear strength was measured at the same depth using the vane shear apparatus (Figure 4 (B)). The tested sample was then extracted from the product block using a Shelby tube for moisture and solids contents determination by oven-drying. Figure 3 shows the diagram of the test locations on the product. The test locations were spaced at about 10 cm intervals. The measurements were conducted in 2014 and in 2017, 4 months and 34 months after filling the flume

Note: UPS1(15) represents an upstream test done at location 1 in 2015.

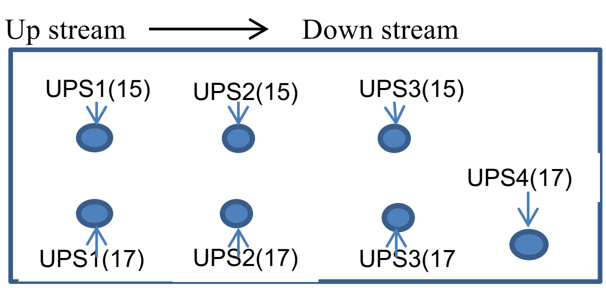

Figure 3. Diagram showing the test locations on the centrifuge product.

Figure 4. (A) Frozen/Thawed centrifuge product treated under laboratory controlled conditions for 30 months test period and (B) In-situ measurement of shear strength using a vane shear apparatus.

3 RESULTS AND DISCUSSION

3.1 *Large Strain Consolidation*

Figures 5 and 6 compare the consolidation results of untreated and F/T treated FCTPs. Figures 7 and 8 compare the shear strengths. Figure 5 indicates that at consolidation effective stresses up to 500 kPa the F/T treated FCTP was at a lower void ratio. This lower void ratio for the F/T treated FCTP is a function of the lower initial void ratio and does not reflect that it is more compressible. Figure 8 shows the shear strength as a function of void ratio and Figure 7 shows the shear strength as a function of effective stress. In Figure 8 the shear strength of the F/T treated FCTP is slightly less than the shear strength of the untreated FCTP at the same void ratio. It is postulated that the freeze/thaw process results in a soil structure which is weaker which then gives the F/T treated FCTP a lower shear strength which in turn would make it slightly more compressible. Figure 6 shows the hydraulic conductivity of the F/T treated FCTP is considerably greater, about one order of magnitude, than that of the untreated FCTP at void ratios greater than 1.0. This larger hydraulic conductivity is the most important benefit of the F/T treatment. An increase in hydraulic conductivity will result in the F/T treated FCTP consolidating much faster.

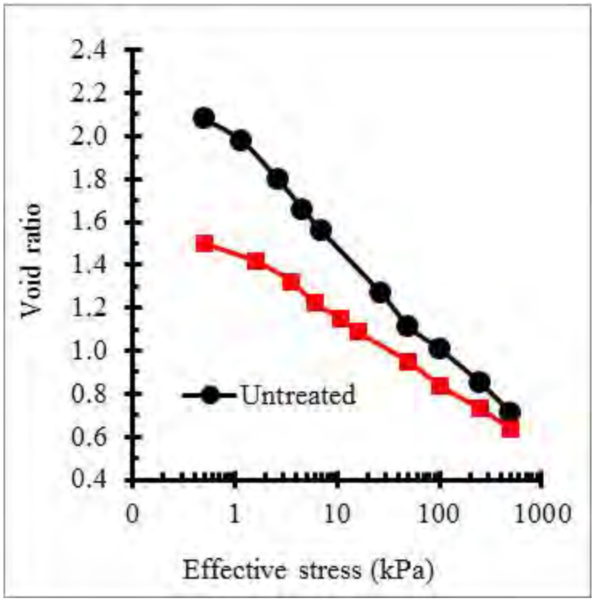

Figure 5. Compressibility curves.

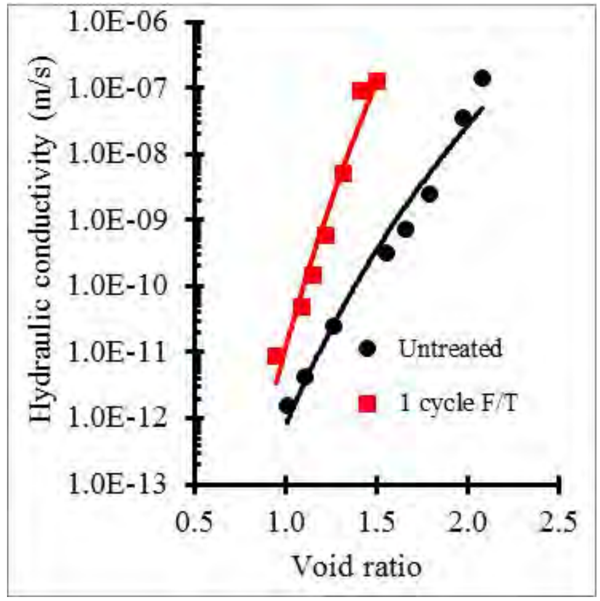

Figure 6. Hydraulic conductivity curves.

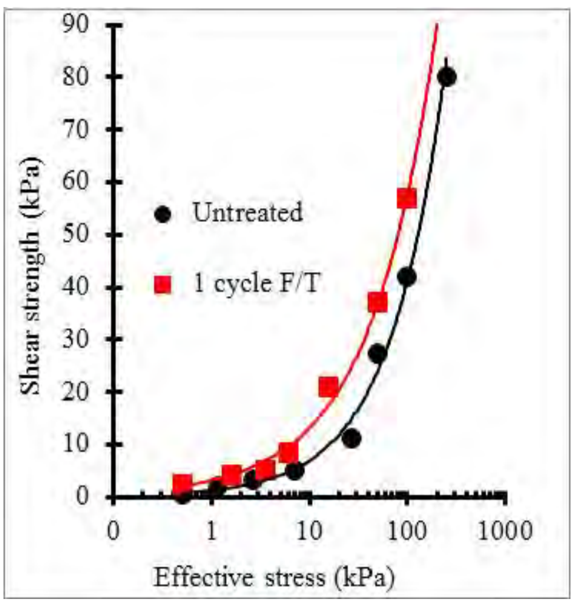

Figure 7. Shear strength vs effective stress curves.

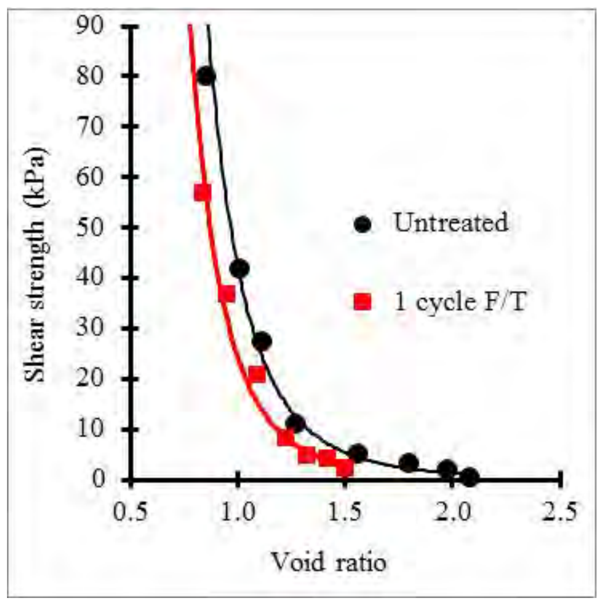

Figure 8. Shear strength vs void ratio curves.

3.2 *Post Freeze and Thaw Product Characterization*

Figures 9 through 11 present the post freeze/thaw/atmospheric drying characteristics of the treated FCTP. The post F/T product characterizations were conducted twice as follow: i) within 3 months after the F/T treatment and ii) 33 months after the F/T treatment. The flume product was kept in a cold room kept between 0 and 2 ^{o}C after F/T treatment. The temperature profile measured in the product was about 0 ^{o}C when the cooling system was turned on (Figure 12, Temp. 1). During the tests periods however, the cooling system was turned off and the tests were conducted at room temperature. The temperature profile measured during the tests periods was about 17 ^{o}C (Figure 12, Temp. 2). It should be noted that the product characterization test study focused only on the half part (upstream) of the treated FCTP (about 60 cm long from the flume crest) (Figure 1B). The other half part (downstream up to the toe of the flume) of the product was detached from the other half part and was very dried and desiccated. The initial thickness of the product before F/T treatment was about 90 cm with a solids content of about 55.5%. The initial shear strength of the product was about 0.4 kPa upon deposition in the flume.

Results of the first characterization test (February, 2015) after F/T treatment show that the thickness of the product was reduced to about 65 cm representing a strain of 28% (Figure 9). The total settlement from flume filling to the first characterization test was about 25 cm. Based on the large strain consolidation test about 6 cm of this was from self-weight consolidation with the other 19 cm from atmospheric drying.The solids content profiles measured at three locations on the product along the flume slope from the crest to the toe (UPS1, UPS2 and UPS3) (Figure 3) remain almost constant throughout the profiles except at the near surface due to evaporation. The solids contents profiles slightly increase from the crest to the toe of the flume slope (i.e., UPS1 < UPS2 < UP3). This is due to the fine migration process along the flume slope (1.5 ^{o}) during thawing. The average solids content at the 3 locations was about 63% and that represent an increase in solids content of about 8% (i.e., from 55% to about 63%). During the first product characterization the shear strength was only measured at the near surface (i.e., up to 15 cm deep) and the shear strength values range from 3 to 5 kPa (Figure 10). No suctions were measured during the first characterization. The flume was left in the cold room kept at about 0 ^{o}C for about 32 months after the first product characterization.

During the second characterization (July 2017) the thickness of the product was further reduced from 65 cm to about 55 cm (Figures 9 and 10). This settlement of 10 cm resulted from 3.5 cm self-weight consolidation and 6.5 cm from atmospheric drying. This additional settlement of 10 cm represents an additional strain of about 15% for a total strain of about 43% since flume filling. . Measurements of the solids contents profiles and shear strengths were carried out close to the previous test locations UPS1 (17), UPS2 (17) and UPS3 (17) (Figure 2). The solids contents profiles exhibit approximately the same trends as those of the first characterization (Figure 9). The solids contents profiles slightly increase from the crest to the toe of the flume slope. The average solids content at the 3 test locations is about 76%. There is an increase in solids content from 63% (first characterization) to 76% (second characterization). The associated shear strengths were only measured at the near surface (i.e., 20 cm deep) (Figure 10). The average shear strength at the 3 locations is about 75 kPa. An additional set of solids content, shear strength and suction profiles were measured near the toe of the product at location UPS4(17) (Figures 2, 10 and 11). This set of measurements yield averages solids contents and shear strengths of about 80% and 140 kPa respectively. The suction profile was measured in the product profile using a T5 tensiometer (Figure 12). All suction profile measurements indicated a value of about -85 kPa and that is the maximum range of the T5 instrument. Therefore, the product suction was greater than 85 kPa. In summary, the solids content, shear strength and suction measured during the second characterization tests indicate a trafficable condition of the product (Figures 4(A) and 13).

The effects of various treatments on solids content and shear strength of FFT are summarized in Figure 13. The plots indicate that centrifugation treatment (treatment 1) of FFT yields a product with 55% solids content (i.e., from 30% - 35% solids content of untreated FFT (0)). The freeze/thaw treatment (treatment 2) yields a product with solids content of 63%. Further medium-term (two years) atmospheric drying/shrinkage processes yield a product with solids content and shear strength of about 76% to 80% and 75 kPa to over 80 kPa respectively. In summary, Figure 12 indicates that centrifugation and freeze/thaw alone cannot achieve a product with a trafficable surface. Additional technologies are required to increase the solids content and shear strength of the product (i.e., atmospheric drying, long-term surcharge and self-weight consolidation).

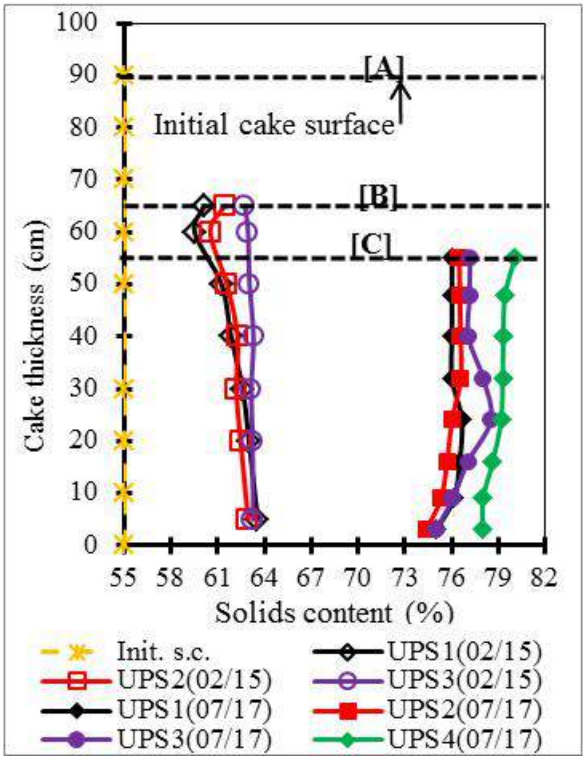

Figure 9. Solids content profiles at various times.

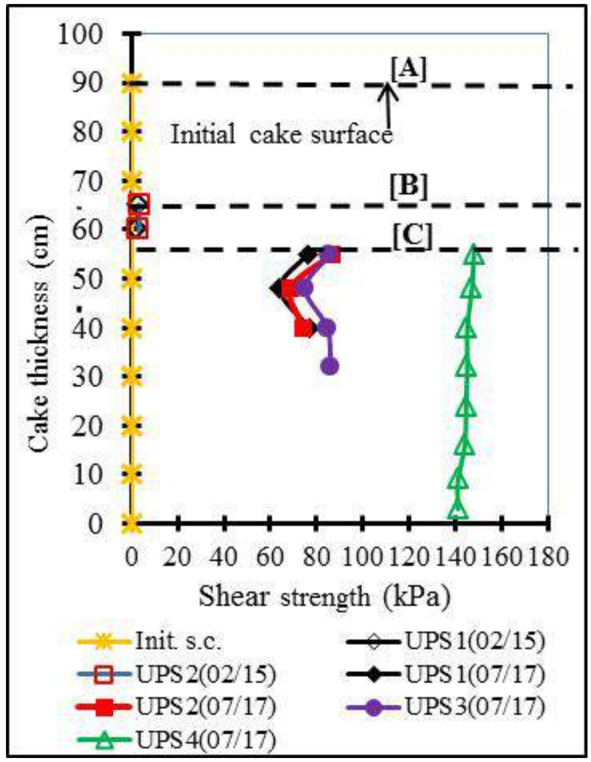

Figure 10. Shear strength profiles at various times.

[A] = initial product surface, [B] = product surface 3 months after F/T treatment and [C] = product surface 30 months after F/T treatment. Cake = centrifuge product.

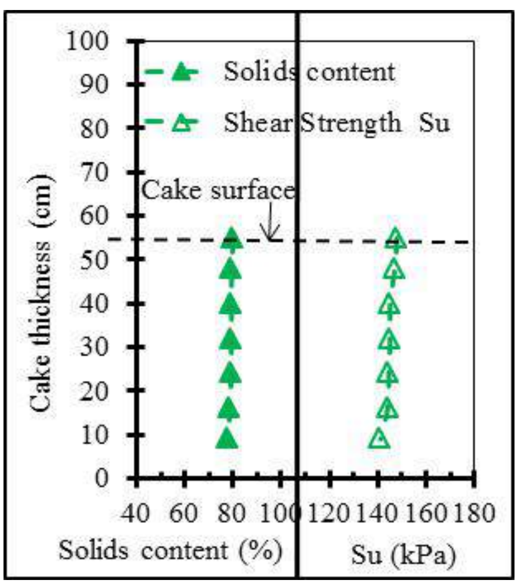

Figure 11. Solids content and shear strength profiles.

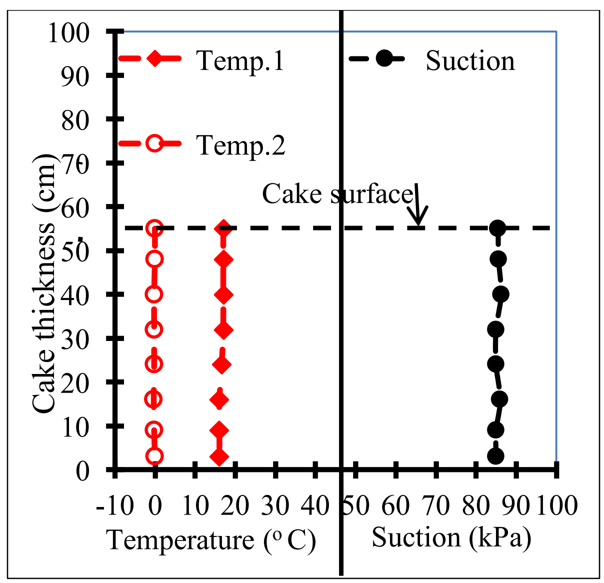

Figure 12. Temperature and suction profiles.

Measurements made in centrifuge product (cake), 30 months after freeze/thaw treatment. S_u = undrained shear strength.

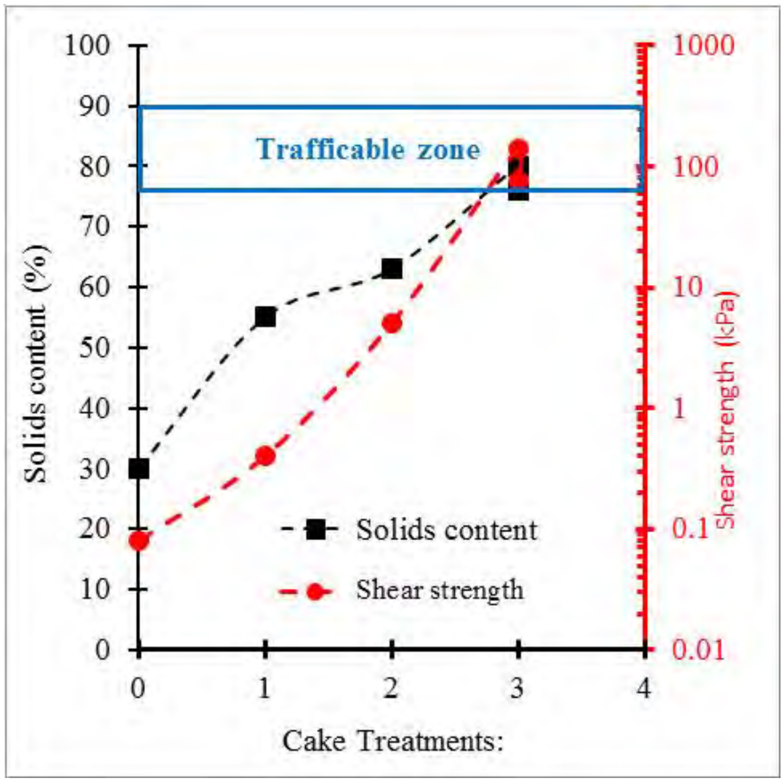

Figure 13. Effects of various treatments on solids content and shear strength of the treated FFT.

0=Untreated FFT, 1=Centrifugation, 2=Freeze/Thaw, 3=Medium-term atmospheric drying.

4 SUMMARY AND CONCLUSIONS

The consolidation and shear strength results indicate that the F/T treated FCTP was slightly more compressible than the untreated FCTP. An increase in compressibility would result in the F/T treated FCTP compressing more under a surcharge stress and during self consolidation. The hydraulic conductivity of the F/T treated FCTP was greater than that of the untreated FCTP by approximately an order of magnitude. An increase in hydraulic conductivity will result in the F/T treated FCTP consolidating much faster. Results also indicate that the F/T treated FCTP has a slightly higher shear strength than the untreated FCTP as a function of effective stress.

Results show that the 1-cycle F/T treatment increased the solids content of the FCTP by 8% (i.e., from 55% to 63 %). Similarly, the 1-cycle F/T treatment increased the shear strength of the FCTP from 0.5 kPa to about 5 kPa. These results indicated that centrifugation and F/T alone cannot produce the deposit that has the strength and stability necessary to establish a trafficable surface.

Further medium-term (i.e., 30-month period) atmospheric drying processes have increased the solids content and shear strength of the F/T treated FCTP from 63 % to about 76% and from 5 kPa to about 75 kPa respectively.

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Effect of rate of rise on the consolidation of fine tailings in deep ponds

S. Masala & S. Proskin
Thurber Engineering, Calgary, AB, Canada

ABSTRACT: The oil sands industry has focused recently on the separate tailings stream deposition and, particularly, on deep ponds with fines-dominated deposits. It is not uncommon to see in new designs a rate of rise (RoR) greater than 10 m/year, despite much lower RoRs in the past. The implications of these higher RoRs on tailings consolidation seem to have been overlooked: higher RoRs hinder dissipation of pore pressures during pond operation and increase the post-deposition settlements. These have adverse effects on tailings deposit performance and reclamation. The goal of this study was to investigate the existence of a threshold value of RoR that does not adversely affect the consolidation of a given fine tailings during deposition. It was found that increasing RoR has detrimental effects on all geotechnical parameters of a deposit, but that there is no a threshold value and the RoR influence on consolidation behaviour is smooth.

1 INTRODUCTION

Settling of mine tailings in storage areas (tailings ponds) is usually analyzed as a one-dimensional (1D) consolidation problem of a soil layer which increases in thickness with time (an accreting layer). As a rule, the consolidation is theoretically formulated in a rigorous manner, as a large strain problem (Gibson et al. 1967, 1981) with non-linear parameters – the material properties, compressibility and hydraulic conductivity, are defined as functions of void ratio. The pore water pressures are generated by the increasing weight of overlaying material, and the drainage path length varies with time (deposit thickness) in a non-linear fashion due to settlements. These unusual features make analytical methods of analysis inapplicable; the numerical methods are standard means. A disadvantage of numerical approach is shortage of general understanding how various parameters and initial and boundary conditions affect the solution.

An accreting layer consolidation formulated as a 1D small strain problem was analyzed by Gibson (1958) who reduced it to a linear integral equation and obtained a closed form solution for the case of deposition at a constant rate, with the only material property - the coefficient of consolidation – adopted constant. Gibson and Shefford (1968) analyzed the role of blanket drains in the consolidation of embankments and obtained an analytical solution for a substitute case: a sand-clay sandwich as a representative volume and an existing deep layer subsequently loaded by deposited soil, with the material parameters considered constant. Recently, a similar problem of the interbedded sand layer deposit concept was analyzed by Masala et al (2016) in two ways: using the Gibson and Shefford (1968) small strain approach, but formulated in 2D and with variable material parameters, and applying a simplified version of Gibson et al. (1981) large strain consolidation theory; both problems were solved numerically.

The authors are not aware of analytical solutions for a 1D large strain consolidation problem with accreting layers nor a systematic numerical analysis of the same problem conducted to investigate the influential parameters and to generalize their effects.

Simulation of the filling of a tailings pond is a regular design task in the oil sands tailings management planning. The issues of a filling rate versus pond storage capacity and related efficiency of pond area are regularly discussed, but apparently never treated in general terms. Such an attempt is described in this paper, motivated by the question of a functional relationship between the deposit geotechnical conditions at the end of filling (EOF) and the rate of filling, which varied widely in the tailings management plans.

The analyzed problem geometry, a cylindrical container with a constant pond area, was adopted instead of a realistic truncated cone in order to eliminate the decrease in the rate of rise (RoR) with filling time and deposit thickness. This occurs even with nearly uniform annual rates of tailings production and is always beneficial. The theoretical approach is based on the large strain theory. The material consolidation properties, compressibility and hydraulic conductivity, are adopted as non-linear functions of void ratio, as usual. The material properties are adopted as average values for several tailings types classified by fines content on the basis of 44 microns FC_{44} or a more common sand-to-fines ratio SFR_{44}. The tailings data used to derive the consolidation properties are the public domain part of the oil sands tailings laboratory consolidation testing data base developed at Thurber Engineering Ltd. (Thurber). Using the actual test results as input for simulations gives practical significance to the outcome of presented analysis.

2 SOURCE DATA FOR TAILINGS MATERIAL PROPERTIES

2.1 *Thurber database of laboratory consolidation test results for oil sands tailings*

The database includes results of laboratory consolidation tests conducted on various oil sands tailings. They are expressed as points specifying the compressibility $\sigma'(e)$ and hydraulic conductivity $k(e)$ functions, where σ' is the vertical effective stress, k is the hydraulic conductivity and e the void ratio. All data are limited to fully saturated conditions.

The data base currently contains about 160 cases. About half of it is in public domain and that subset was used in the presented analysis, see section 2.2.

The majority of tests are historical records of the academic research at the University of Alberta, Edmonton and the tests conducted in Thurber's tailings laboratory, for which the original data and supplementary information were available. The tests that were conducted by others and presented at conferences or published in journals in graphical form were scanned and digitized; they typically miss some supplementary information. The database is surely not exhaustive; more work will be needed to include recent information from the meetings and journals in the past several years that were mostly related to flocculation by polymers.

The supplementary information for each tested material includes physical and geotechnical index properties, details of treatment methods (chemical, dosage, mixing procedure), slurry rheology parameters, shear strength, etc. Some of this information is missing, especially for older tests from 1980s and 1990s, when stating only, for example, "MFT from this or that operator" was considered sufficient for identification. It will also be necessary to include full specifications of index testing methods and procedures, as well as more detailed descriptions of consolidation equipment and testing procedures.

The large majority of consolidation test data were obtained using the large strain consolidation (LSC) apparatus, which exists in various forms in many geotechnical laboratories involved with the oil sands operators, and typically implementing testing procedures with the load increment ratio LIR = 1, i.e. doubling the stress in subsequent load steps. Based on authors' experience, differences in consolidation properties on the same materials tested using the same equipment by different laboratories should always be expected; however, to the best of their knowledge, a systematic study has never been performed, so that uncertainty in the data base records remains unquantifiable. This is not considered a limitation for its use, at least for the purpose of this paper, since the variation of consolidation properties with SFR was logical (see section 2.2) and the scatter of results within the same class (type) of tailings appeared commensurate with the anticipated variation of tailings feed in commercial production.

2.2 *Tailings types for the analysis*

Classifying the tailings data base records can be done in many ways, following various sorting schemes. Our decision for the purpose of this analysis was conservative: the data were classified according to the percentage of fines in mineral solids, measured on the basis of 44 microns, FC_{44} or the more common derived factor, the sand-to-fines ratio SFR_{44}. The subscript "44" is omitted in the following text. The FC and SFR values depend on the determination method, most commonly a combination of sieve and hydrometer (SH) or the laser diffraction (LD), but the differences between the methods were found sufficiently small at the particle size level of 0.044 mm (Senft et al. 2011), so that a potential uncertainty of SFR in the database was considered inconsequential.

The data base was first classified into two main groups: (a) coarse tailings with SFR>2 and (b) fine tailings with SFR<1. The "transitional" range of SFR=1÷2 did not have any records.

The coarse tailings group was then subdivided into four subsets: SFR>5 (FC<17%), SFR=4 (FC=20%), SFR=3 (FC=25%) and SFR=2÷2.5 (FC=30÷35%). The vast majority of coarse tailings data is in the two categories for SFR=3÷4, as the result of focus on composite tailings (CT) at Syncrude and Suncor in 1990s and 2000s. The class of SFR>5 was not interesting, being close to beach sand which behaviour does not differ much from natural sandy soils. The class with SFR=2÷2.5 was investigated only by Suncor and showed behaviour close to the class SFR=3. The fine tailings fraction in the coarse tailings group was treated with various methods: coagulation, thickening, flocculation, etc. but no differentiation was necessary since the effect of SFR was dominant.

The fine tailings group was subdivided into two subsets: SFR=0.7÷0.9 (FC=52÷58%) and SFR=0÷0.2 (FC=84÷100%). Each group contained tailings treated with various methods: thickening, flocculation, centrifugation, as well as untreated fine tailings. Again, no differentiation into subclasses due to treatment was performed.

3 DEFINITION OF CONSOLIDATION PROBLEM

3.1 *Theoretical model and software*

The theoretical model was based on the 1D large strain consolidation (LSC) theory developed by Gibson et al. (1981). The software used for numerical analyses was FSConsol by GWP Software Inc., version 3.48, a common tool for LSC simulations in the oil sands industry. The "Pond Analysis" option was used to simulate a mine tailings pond scenario in which a slurry is gradually deposited into a settling pond, with user-specified material properties, pond geometry, filling rates, and boundary conditions. The "Tank" option was used to analyze the reference cases in which the pond was assumed instantaneously filled to the nominal depth.

3.2 *Material properties for analysis*

Figures 1 and 2 present the ranges of variation of compressibility and hydraulic conductivity data for the tailings classes from Section 2.2. For each class, the ranges are shown as patterned areas, and the averages plotted as dashed lines. The average values are expressed as power law functions (equation 1) with parameters A to D listed in Table 1.

$$e = A\sigma'^{B} \quad k = Ce^{D} \tag{1}$$

The data sets in Figures 1 and 2 are distributed as anticipated due to variation of FC or SFR: compressibility becomes smaller and hydraulic conductivity higher with increasing SFR. There is certain overlap of the neighbouring data sets that can be attributed to tailings treatment and inherent variation in physical properties of the feed material.

For compressibility, the largest variation is at the small stress end, reflecting influence of various treatment methods. The variations progressively diminish with increasing stress: above 10 kPa the void ratio variation is narrowed to a band about 0.5 wide, regardless of treatment. The data ranges for coarse tailings and fine tailings are distinct. It is interesting to note that the subsets for coarse tailings with SFR=3 and SFR=4 overlap for stresses larger than 10 kPa, while the ranges of SFR=0÷0.2 and SFR=0.7÷0.9 for two fine tailings groups remain distinct up to several hundred kPa's. This indicates that the settlements of coarse tailings in deep ponds will not vary much with SFR, while such differences can be expected with fine tailings.

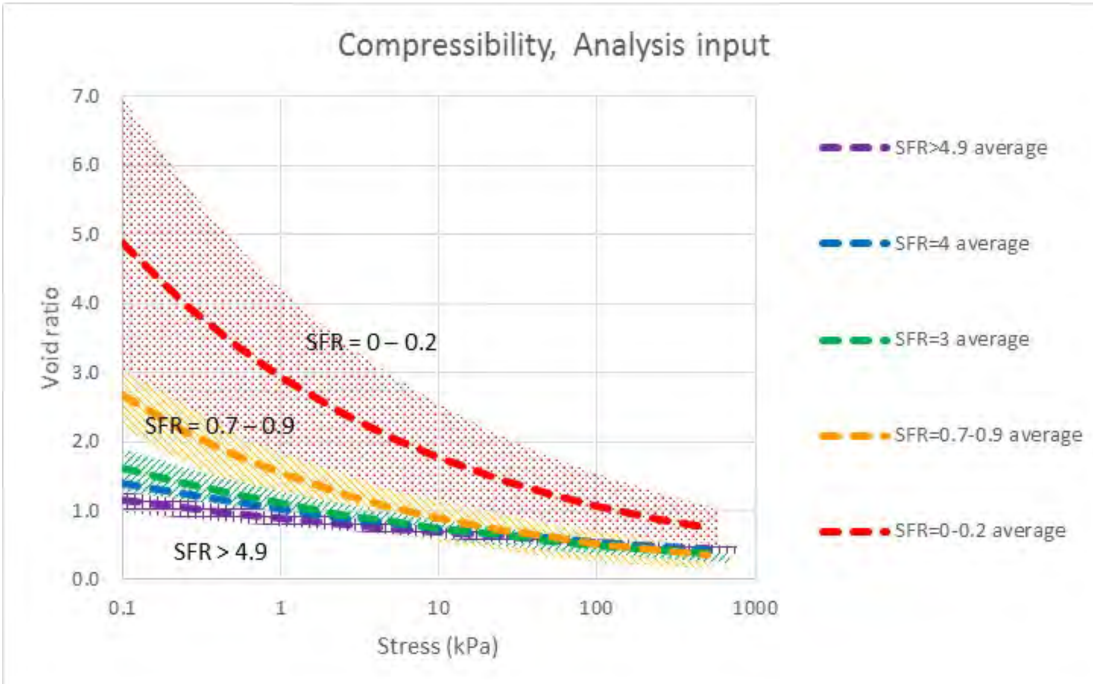

Figure 1. Compressibility ranges for various SFR and average function used as input for analysis

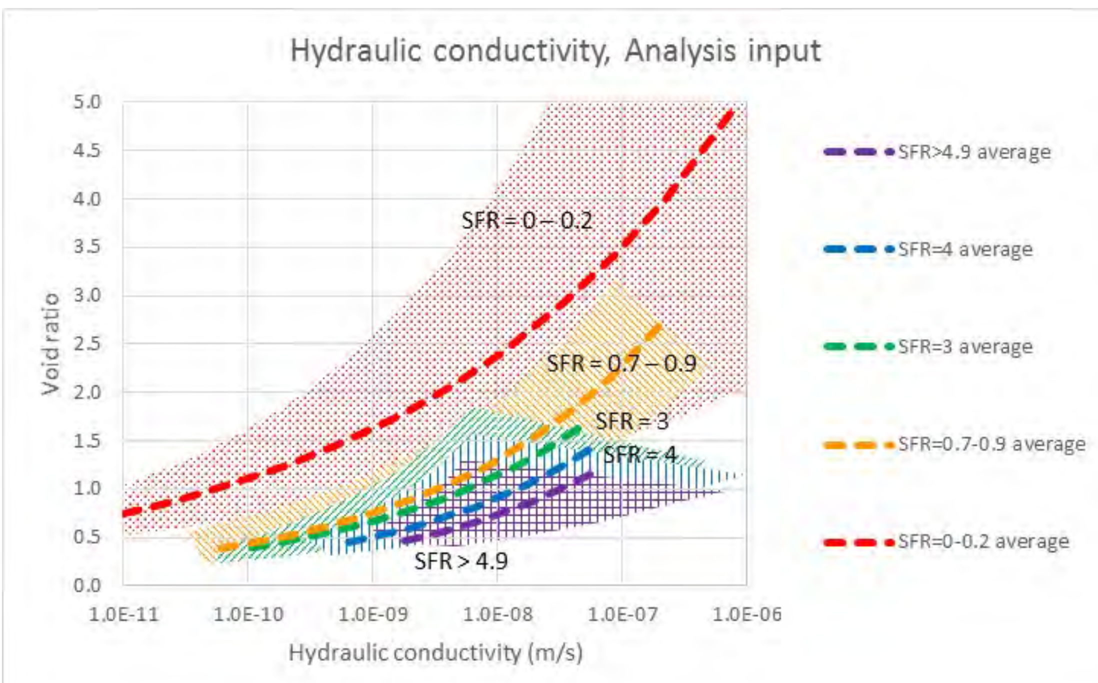

Figure 2. Hydraulic conductivity ranges for various SFR and average function used as input for analysis

The hydraulic conductivity data sets for the coarse tailings with SFR=3 and 4 become distinct after ~100 kPa, while the ranges of hydraulic conductivities of fine tailings with SFR=0÷0.2 and 0.7÷0.9 separate after larger stresses of 200-300 kPa. This indicates that the pore pressure dissipation rates will progressively differ with increasing deposit depths.

Table 1. Material properties for analysis.

SFR	Compressibility A (kPa^{-1})	B	Hydraulic conductivity C (m/s)	D	Specific gravity Gs	Initial SC (%)
0÷0.2	2.939	-0.220	$5.58*10^{-11}$	5.973	2.50	35
0.7÷0.9	1.547	-0.238	$3.33*10^{-9}$	4.144	2.50	55
3.0	1.099	-0.170	$5.64*10^{-9}$	4.253	2.60	65
4.0	1.018	-0.138	$1.43*10^{-8}$	3.885	2.60	65
≥4.9	0.888	-0.112	$3.21*10^{-8}$	3.786	2.60	65

The solids content (SC) values of discharged slurries were different for various tailings type groups, based on the data base information: the coarse tailings with higher values of SC = 65%, and the fine tailings with lower values of SC=55% for SFR=0.7÷0.9 and SC=35% for SFR=0÷0.2. The values of initial SCs are within the ranges of tested data so that sedimentation stage, which the software cannot incorporate, is excluded from analysis. Some of the data base cases include unrealistically high void ratios at the so-called "zero effective stress state" as low as 0.01 kPa, which is below the accuracy of measurement.

It is worth noting that the compressibility laws do not include any possible time effects (creep or secondary consolidation – deformation after "full dissipation" of excess pore pressures). This deformation would not significantly affect the results of analysis, being orders of magnitudes smaller than the primary consolidation settlement and, declining exponentially with time, would become less relevant with time after pond filling. Similarly, the hydraulic conductivity laws do not include the effect of hydraulic gradient, for which most of data base records did not even contain data.

3.3 *Pond geometry*

A single case of pond geometry was analyzed: the pond was assumed cylindrical, with constant area, to isolate the effect of RoR. A more realistic geometry would be a truncated cone, with sloping sides at 3H:1V or 4H:1V. The conical geometry is beneficial as it tends to significantly reduce the RoR with the deposit height increase, thereby enhancing consolidation.

The pond depth of 40 m was adopted, as in typical out-of-pit oil sands tailings pond.

3.4 *Filing rates*

The FSConsol filling rates (FR) in Table 2 are expressed in terms of dry solids mass added to the system ("discharged into the pond") in a unit of time. They were selected in such way to produce – for the same total dry mass deposited - the nominal rates of rise of tailings elevation of 1, 2, 4 and 8 metres per year (the "nominal rates" meaning that consolidation was not accounted for during filling, as if the tailings hydraulic conductivity was infinitely small). The rates were constant during deposition.

The "Tank" reference case – the instantaneously created deposit with the same discharge SC and the same total dry mass - was added to the four RoR analyses, to elucidate the pattern of behavior observed from the RoR calculations.

Table 2. Filling rates in kg/day of dry mass for planned RoRs.

SFR	RoR = 1 m/year	RoR = 2 m/year	RoR = 4 m/year	RoR = 4 m/year
0÷0.2	$1.214*10^6$	$2.428*10^6$	$4.855*10^6$	$9.684*10^6$
0.7÷0.9	$2.249*10^6$	$4.498*10^6$	$8.996*10^6$	$1.794*10^7$
≥3.0	$2.968*10^6$	$5.936*10^6$	$1.187*10^7$	$2.374*10^7$

3.5 *Boundary conditions*

The boundary conditions were kept as an impermeable pond bottom and a permeable deposit top with the phreatic surface coincident with the tailing top elevation; therefore, the deposit was fully saturated throughout analysis. The top boundary condition assumes efficient surface drainage of consolidation released water. A surface water layer above tailings does not affect deposit consolidation in this particular problem and only changes the total pore pressure distribution over depth.

It should be noticed that all 1D consolidation analyses suffer from the “reduced dimensionality” problem and ignore the physics of tailings flow, over beach above water or subaqueously. These can significantly alter consolidation behaviour of deposited tailings. Flow over dry sandy beach can result in stream densification due to particle settling in shear flow or, on the contrary, it can result in segregation, if the tailings slurry is thin and prone to it, with the fines washed-out of tailings stream with released water. Additional segregation may occur when the slurry enters the ponded water. These processes are not included in the selected or any other 1D model.

3.6 *Performance indicators*

The performance indicators were founded on experience and historical criteria: (a) variations of the average SC and the degree of excess pore pressure dissipation (PPD) for the deposit over time, (b) settlements over time, with attention to the post-deposition settlements, from EOF to 10-year-after-EOF and to the ultimate state, and (c) the effective vertical stress profiles at 10 years after EOF. The performance criteria are entirely geotechnical.

The 10-year-after-EOF moment is typically the time to begin reclamation which, for fine tailings, starts with sand capping. The critical issue is trafficability for mine equipment used for spreading of sand. Shear strength is theoretically zero at the surface of saturated tailings deposits and gradually increases with depth. The relevant shear strength is the undrained one, and it can be estimated directly from the vertical effective stress using the undrained strength ratio.

The settlements after EOF may be the critical factor for the final landscape design. The portion between EOF and 10 years after affects how much overburden should be used for landscaping, while the ultimate settlement after the end of reclamation specifies a required “safety factor” in the landscape design, to preserve the functionality of landforms “in perpetuity”.

The consolidation effects of sand capping and overburden placement were not included in this analysis.

4 RESULTS

Space limitations allow presenting only selected results although the same analyses were conducted for all tailings types from Table 1.

4.1 *Case SFR = 0 ÷ 0.2*

Figures 3 to 5 show time plots of tailings elevation (settlements), the average SC and the average PPD, respectively, for the finest tailings class with SFR=0÷0.2. Qualitatively identical results were obtained for other analyzed tailings types, with their data magnitudes commensurate with their respective SFRs.

Figure 3 shows separate tailings surface elevation versus time curves for each RoR during deposition. The higher the RoR, the faster surface elevation rise with time. It is a clear indicator of the amount of consolidation occurring during deposition: the thicker the pond at a fixed time during deposition, the lower the degree of consolidation during deposition. Once the filling stops, all deposits behave in the same way: their settlements follow the reference curve for the instantaneously filled pond. This enables a relatively simple graphical estimation of the settlements for various RoR using the planned time of deposition, i.e. the known EOF, and the results of the tank analysis.

Figure 4 presents the average SCs for the deposits. A clear pattern can again be observed: during filling, each RoR has its own SC variation curve and the SC increase is slower at higher RoR. After EOF, the separate RoR lines merge with the line for the tank reference case and evolve in the same way, so that all deposits, given enough time, ultimately reach the same state regardless of their RoR.

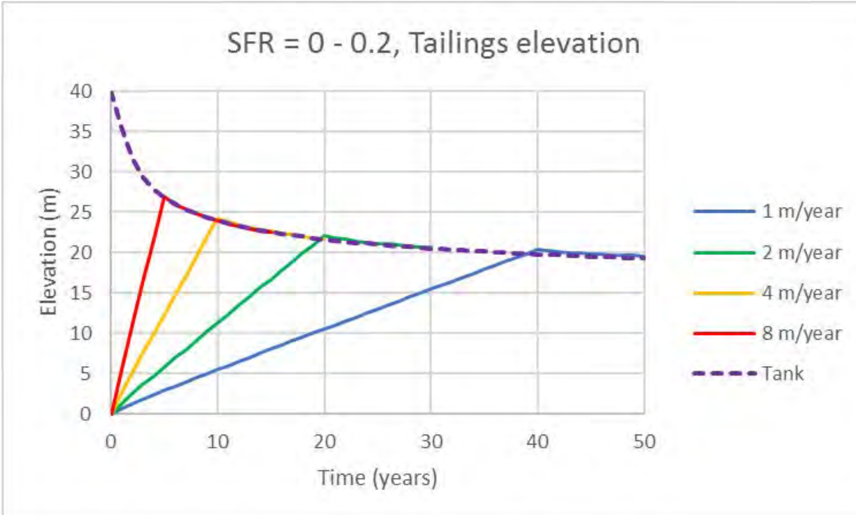

Figure 3. Deposit elevation over time, SFR=0÷0.2

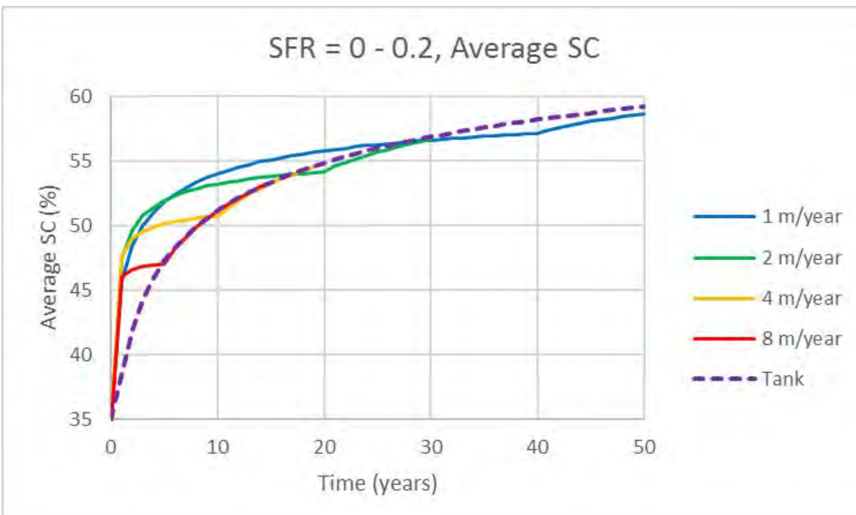

Figure 4. Deposit average SC over time, SFR=0÷0.2

Not surprisingly, the same behaviour can be seen with the average PPD lines in Figure 5: individual RoR lines merge with the tank line at their respective EOF times and further follow the reference tank case.

Figure 6 shows the effective stress profiles for various RoR 10 years after the end of deposition. Assuming the undrained strength as maximum 20%÷30% of the effective stress, it is unlikely that fine tailings can develop any significant strength within relatively short periods after EOF, regardless of RoR.

4.2 *Cases SFR=0.7÷0.9 and SFR=3.0*

Qualitatively identical results were obtained for other SFR cases. A summary of the results, with computed performance indicators, is presented in Table 3.

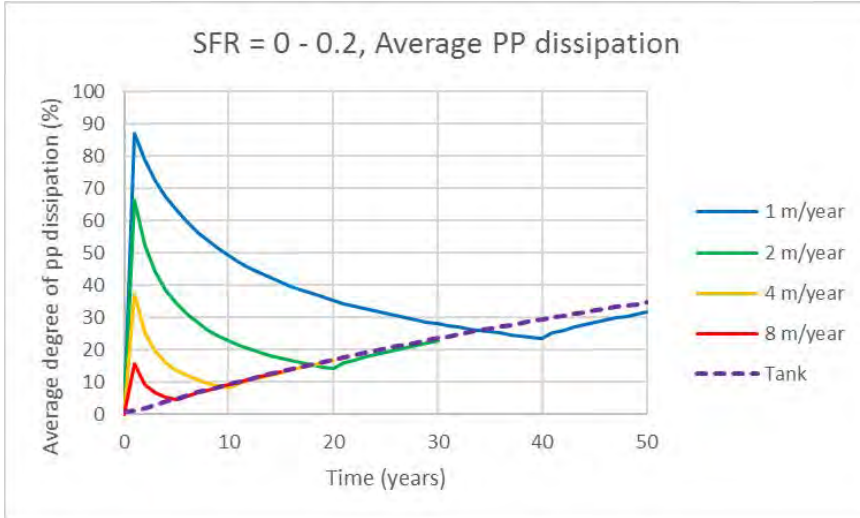

Figure 5. Deposit average PPD over time, SFR=0÷0.2

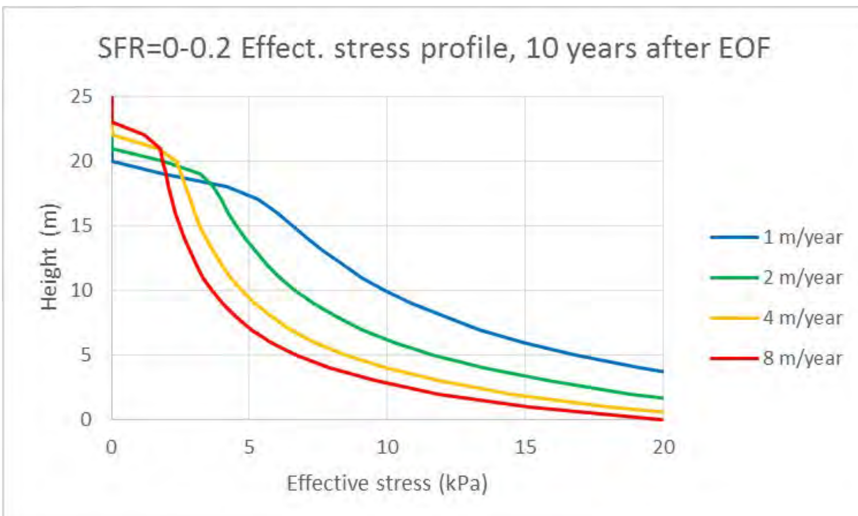

Figure 6. Effective stress profiles 10 years after the end of filling, SFR=0÷0.2

An illustrative example of some unintuitive features of the RoR effect is shown in Figure 7. The rate of generation of excess pore pressure at the pond bottom during deposition is essentially constant despite the change of RoR from 1 to 8 m/year. After touching the reference tank case line, a pore pressure dissipation for an individual RoR follows this – unique – tank line until the end of consolidation. So, the pore pressure at the bottom is apparently independent of the rate of rise and uniquely determined by the tailings surface elevation for a particular material.

Figure 8 presents the excess pore pressure at the bottom against the deposit surface elevation, for the same RoR=8 m/year and variable SFR. The pattern from Figure 7 is repeated, with each individual SFR line “scaled” according to its materials properties, in this case the hydraulic conductivity as the dominant parameter.

5 CONCLUSIONS AND RECOMMENDATIONS

The effect of RoR of tailings surface during deposition was investigated on a cylindrical pond using three different oil sands tailings types with SFR varying from 0 (100% fines) to 3 (25%

fines). Increasing RoRs have progressively adverse effects on all geotechnical performance indicators of a deposit. A higher RoR reduces consolidation during deposition, creating a less dense deposit with higher excess pore pressures and a lower shear strength at the end of filling, and will increase post-depositional settlement. The adverse effects of increased RoR will reduce the pond storage efficiency.

There is no a threshold value for the RoR effect; it is smooth, at least over the ranges of RoR and material parameters examined here. The same pattern of behavior was found for all tailings types, or SFRs (Table 3).

Table 3. Performance indicators.

SFR	RoR (m/year)	EOF Elevation (m)	EOF Aver. SC (%)	EOF Aver. PPD (%)	EOF Storage efficiency (kg dry mass/m^3)	Settlements: EOF→ 10 years (m)	Settlements: 10 years→ ultimate (m)
0÷0.2	1	20.38	57.1	24	869	0.81	2.91
	2	22.09	54.1	14	802	1.45	3.98
	4	24.29	50.7	8	730	2.59	5.04
	8	26.95	47.0	4	658	4.45	5.84
0.7÷0.9	1	24.47	74.3	19	1,342	1.01	2.75
	2	26.98	70.3	9	1,217	2.02	4.25
	4	30.68	65.2	4	1,070	4.07	5.90
	8	34.88	60.0	2	941	6.93	7.24
3.0	1	28.73	78.2	15	1,508	0.96	2.55
	2	31.41	74.6	6	1,379	1.99	4.20
	4	35.06	70.2	2	1,236	3.85	5.99
	8	37.51	67.5	1	1,155	4.78	7.51

A peculiar feature of the RoR effect combined with a time-independent compressibility function was found by "scaling" the analysis results against deposit elevation instead of time: all filling curves for various RoRs (and the same material) collapse onto a unique line.

Another consequence of the time-independent compressibility functions adopted in the analysis is that, after the end of deposition, and regardless of the RoR, the deposit follows a unique evolution path as for the instantaneously created deposit with the same total dry mass. That is because the ultimate state of a deposit is the same regardless of RoR. At the ultimate state, with all excess pore pressures dissipated, the effective stress profile will depend only on the amount of dry mass deposited. The ultimate thickness of the deposit is independent of the RoR and all RoR cases should asymptotically converge toward it.

The influence of time-dependent deformation (creep) was not considered because of lack of experimental data and analytical tools. Although creep deformation is not expected to be critical, creep will increase settlements and will certainly not be beneficial for pore pressure dissipation.

Thurber's oil sands tailings geotechnical database contains data from both published sources and confidential testing. We believe there is a wealth of unpublished data that remains locked away. Our suggestion to the various stakeholders is to, as much as possible, share their unpublished data through COSIA and make it more widely available. By contributing unpublished data on consolidation testing to the tailings data base, it would expand its coverage of tailings types and, particularly, recent treatment technologies. This collaborative action should improve access to a wider variety of data sets for comparing and assessing dewatering technologies.

The software used for consolidation analyses of oil sands tailings deposits is, as a rule, commercial and proprietary. In some situations, when numerical details become important, it is tedious or time-consuming to discern how the theory was implemented. Given the importance of this software in predicting tailings volumes, COSIA should conduct a round-robin comparison of various software used by the oil sands industry, with analysis cases carefully tailored for control of critical theoretical and numerical issues.

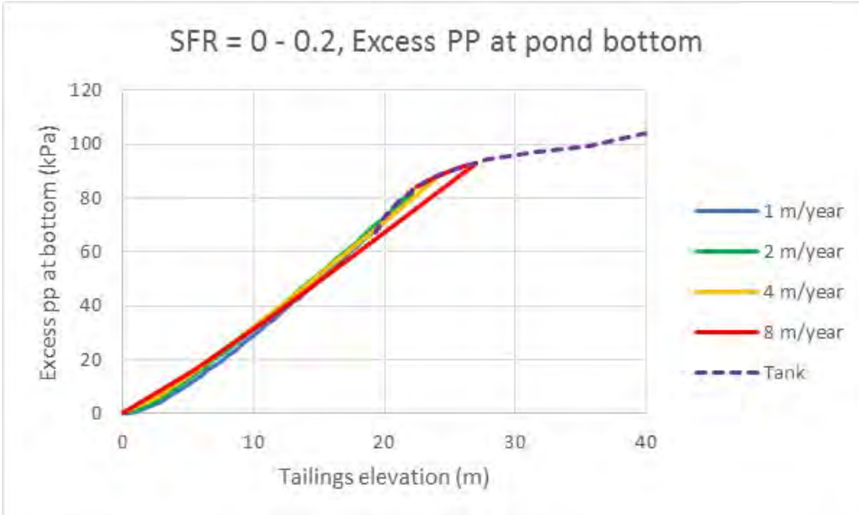

Figure 7. Excess pore pressure versus tailings surface elevation for variable RoR, SFR=0÷0.2

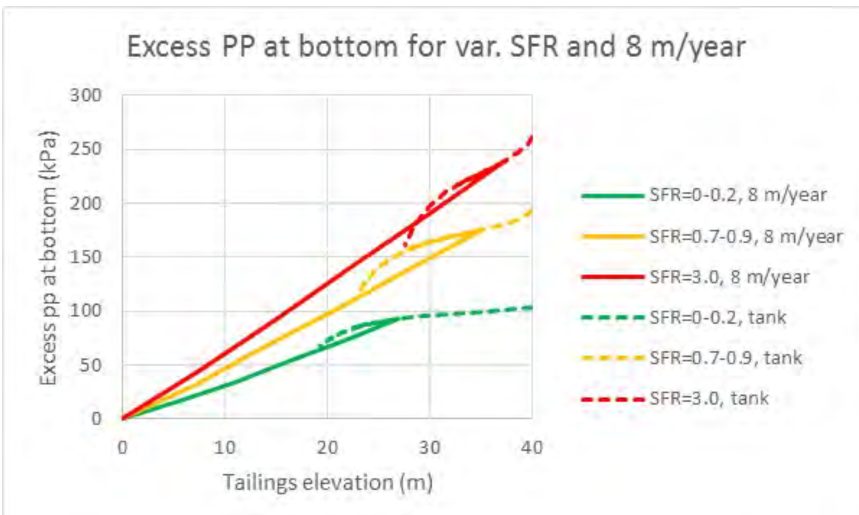

Figure 8. Excess pore pressure versus tailings surface elevation, RoR = 8 m/year, variable SFR

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Laboratory Plate Load Testing of Non-Segregating Tailings

M. Pinheiro & S. Proskin
Thurber Engineering Ltd., Calgary, AB, Canada

B. Li
Canadian Natural Resources Ltd., Calgary, AB, Canada

ABSTRACT: Non-segregating tailings (NST) are tailings produced by mixing mature fine tailings or thickened tailings with cyclone underflow coarse tails, and a coagulant to accelerate solids settling and densification. NST is prepared at a target sand-to-fines ratio of about 4 or more, thereby producing a predominantly frictional material. A research project was setup to measure the bearing capacity of NST, in order to assess its readiness for capping or landform construction activities. Five plate load tests were carried out on NST dewatered to different solids content. Index testing, and strength testing (using pocket penetrometer, handheld field vane and a Rimik® cone device) were also conducted to further characterize the tailings. The results revealed that the ultimate bearing capacity of NST increases with plate settlement, as expected for a frictional material whose shear strength depends on stress and confinement levels. For a small settlement (20 mm), the ultimate bearing capacity varied from 3–16 kPa. For larger settlements (60 mm), ultimate bearing capacities reached nearly 30 kPa for the NST at 80.7% solids content.

1 INTRODUCTION

In 2009, the Alberta Energy Regulator (AER), then known as Energy Resources Conservation Board (ERCB), issued "Directive 074: Tailings Performance Criteria & Requirements for Oil Sands Mining Schemes." Under this directive, oil sands operators were required to come up with strategies to meet specific targets, such as having a trafficable tailings deposit with a minimum shear strength of 10 kPa within five years after tailings has been deposited in a dedicated disposal area. Directive 074 proved itself to be difficult to comply with, and has since been replaced in 2016 by "Directive 085: Fluid Tailings Management for Oil Sands Mining Projects" (AER, 2009, 2016). Directive 085, which is currently being completed and enhanced, does not set out a particular strength target for trafficability. Instead, it sets out a tailings management framework in which performance criteria are first developed by the operator for each tailings deposit. The proposed performance criteria are then assessed by the regulator for approval. Each criterion must identify indicators (e.g. material properties, residual settlement, trajectory to trafficability) and measures (e.g. solids content, sand-to-fines ratio, cone penetration testing) that will be used to assess and track progress towards the various stages of reclamation – trafficability being one of them.

Non-segregating tailings (NST) are tailings that have been significantly dewatered by using thickeners and cyclones. They are produced by mixing either mature fine tailings or thickened tailings with sand derived from cyclone underflow coarse tails. Coagulants (e.g. CO_2, CaO) are added to accelerate solids settling and densification. NST is typically prepared at a target sand-to-fines (SFR) ratio of about 4 or more, thereby producing a predominantly frictional material.

Plate load testing and ultimate bearing capacity are here investigated as an alternative gauge to assess the shear strength of NST. The underlining concept would be to conduct plate load tests on NST samples prepared at different solids contents by weight (SBW), and estimate the ultimate bearing capacity of each sample. Solids content would then be used as a measure to assess the deposit's trafficability and readiness for capping or landform construction activities and monitor progress towards reclamation.

A research project was setup, and the scope of work entailed the following tasks:

- Develop procedures for the index testing, strength characterization, and plate load testing.
- Design and fabricate a plate load assembly for testing the NST under laboratory conditions.
- Conduct plate load tests.
- Conduct strength characterization tests on the NST samples.
    a. Cone penetration testing using a hand-held electronic cone penetrometer.
    b. Shear vane testing using a hand-held shear vane device.
    c. Pocket penetration testing using a hand-held pocket penetrometer.
- Collect samples for geotechnical index testing, including Atterberg limits, solids content, bitumen content, and particle size distribution.
- Analyze results from index, strength characterization and plate load testing programs.
- Determine the ultimate bearing capacity of the NST material by means of analytical methods.

2 METHODOLOGIES

The following sections briefly describe the methodologies employed to prepare the NST and to characterize it in a laboratory setting. NST production and testing, including plate load testing were all carried out at Canadian Natural's "Bitumen Production / Applied Process Innovation Centre," located at Horizon Oil Sands mine, just north of Fort McMurray, Alberta, Canada.

2.1 *NST Material Preparation*

The NST material was produced at a SBW of 64.4% and at a SFR of about 5. The tailings material was then discharged into four 1.0 m × 1.0 m × 0.7 m (length × width × height) aluminum pans: Pans #12, #11, #09 and #08 (Figure 1). The initial plan was to prepare batches of NST at solids contents ranging from around 65% to 85%, and discharge each batch into separate pans. However, this plan was found to be impractical – it would take more time and resources to prepare several batches at distinct solids content than to prepare a single homogeneous material. Moreover, NST produced at SBWs above 80% would not be representative of field conditions.

The tailings in each pan was then subject to different dewatering conditions so that by the end of two weeks, they were expected to have reached target solids contents of 65%, 70%, 75% and 85%. The dewatering strategies involved:

- Laying a 0.10 m thick layer of dry and loose beach sand at the bottom of the pan to provide bottom drainage, at least to the maximum storage capacity of this sand layer.
- Installing a French drain at the sand layer to allow the under-seepage water to drain out, and enhance the bottom drainage.
- Decanting the "free" water accumulated on the tailings surface.
- Mounting lights, and subjecting the surface of the tailings material to varying light schemes and intensity (i.e. number of light bulbs on), to enhance evaporation.
- Setting up fans to blow air moisture away from the tailings surface, and enhance evaporation.

Table 1 summarizes the various dewatering conditions applied to the NST in each pan.

Table 1. Pan setup and dewatering conditions.

	Pan #12	Pan #11	Pan #09	Pan #08
Average target SBW	65%	70%	75%	80%
Sand beach layer at the bottom of pan	Yes	Yes	Yes	Yes
French drain	No	No	Yes	Yes
Removal of decanted water	No	Yes	Yes	Yes
Lights / Light intensity	No	No	Yes / Medium	Yes / High
Fan	No	No	No	Yes

Figure 1. (a) Overview of pans and lab setup. (b) Overall plate load test setup showing pan, reaction frame, hydraulic ram, load cell, LVDT, data acquisition equipment and laptop.

All four pans were initially placed on top of Desna scales to monitor the change in weight (in kilograms) of the tailings material with time prior to plate load testing. Figure 1a shows the pans, light stands and Desna scales used in this research project. Figure 1b depicts the plate load testing setup, which will be described in the next subsection.

NST dewatering occurred over a two-week period. Weights were recorded at irregular intervals, usually during weekdays, as no one worked full time in the lab. The results of the dewatering program are presented in Figure 2, in terms of solids content. The NST material dewatered rapidly after deposition as indicated by the sharp jump in solids content from 64.4% to almost 75% in Pans #11, #09 and #08. The NST material in Pan #12 also underwent similar rapid dewatering; however, this is not readily observed in Figure 2 because the decant water was not pumped out from this pan. Figure 2 also shows target (from Table 1) and actual average SBW values achieved prior to plate load testing. The solids content of the tailings in Pan #08 was on target, but the others were not. Anyway, an NST at a solids content below 75% would have been too soft/weak to be tested, as discussed in Section 2.2.

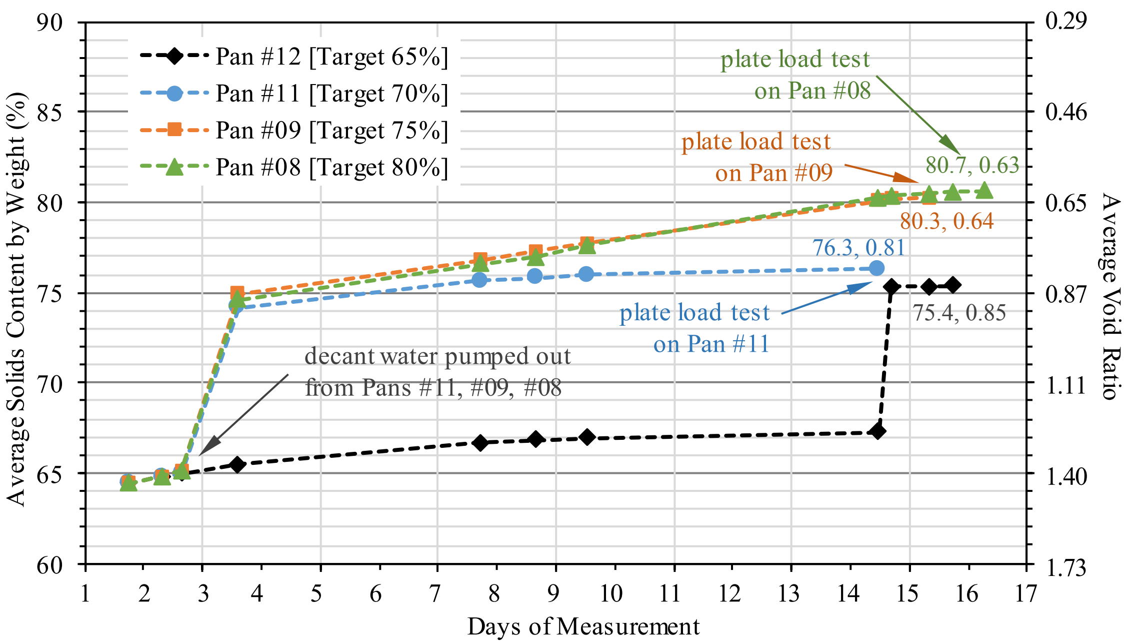

Figure 2. Average solids by weight of the NST in each pan over an approximately two-week period.

The NST in Pans #09 and #08 developed millimetre-to-centimetre thick cracks. Desiccation was not uniform because of uneven wind and light exposure (some of the lightbulbs fixed to the stand were burnt out, and air was blown from a fan located at one of the corners of the pan). Unsaturation in the driest areas may have reached down a couple centimetres. The topmost surface of the NST was notably glossy, likely due to bitumen migration during deposition and sedimentation. The topmost layer was also visibly richer in fine particles than the underlying one.

2.2 *Plate Load Testing*

The plate load testing equipment included the following:

- Custom-made load frame.
- Custom-made 203.2 mm (8-inch) diameter steel plate.
- Custom-made adapters and extension rods.
- 4-ton capacity hydraulic Porta-Power ram assembly.
- 1-ton capacity electronic load cell.
- Linear Variable Differential Transformer (LVDT).
- Data acquisition system and laptop.

The load frame was especially designed for this project to accommodate the dimensions of the pans, the expected maximum loads, as well as transportation requirements. The load frame, steel plate, adapters and extension rods were all manufactured in Calgary. The hydraulic ram, the disassembled frame and companion parts were shipped to Horizon mine, and reassembled on site, as shown in Figure 1b.

The plate load test consisted of pushing the steel plate onto the tailings material, and recording both plate displacement (in millimetre) and load (in kilogram). Any desiccated/crusted tailings material within the testing area was removed, and the exposed surface evened out with a spatula before laying down the steel plate.

The data acquisition system was set to automatically collect and store readings of displacement and load every second. These readings were plotted on a laptop in real-time for quick interpretation of the tailings behaviour. The first displacement increments were very small (0.01 – 0.05 mm) because the initial NST response was relatively stiff. As the NST deformed and softened, the imposed displacement increments were increased in steps up to a maximum value of about 1 – 3 mm. The recorded total displacement and load values were transferred to an Excel spreadsheet to calculate the relative total displacement of and applied pressure under the plate. These were then plotted to help assess the results and determine when to terminate the test.

Plate load tests were completed on Pans #11, #09 and #08. No test was carried out on Pan #12 due to time constraints and because its NST was still very soft. Moreover, the bearing capacity of the tailings material in Pan #12 might have been below the sensitivity of the load cell. The results of the plate load testing are discussed in Section 3.3.

2.3 *NST Strength Characterization*

Strength properties of the NST material were also indirectly assessed by other means based on pocket penetrometer test, field shear vane test and cone penetration test. The methodology of each test is briefly described below. Desiccated and cracked material was removed and, the exposed surface levelled before testing.

The pocket penetrometer test consisted of pushing the piston of a Humboldt pocket penetrometer device onto the NST, and recording the unconfined compressive strength, S_q (in tons/ft^2 or kg/cm^2) directly from a scale indicator. An adaptor foot was attached to the piston to increase the device's accuracy in the lower range of measurement.

The field shear vane test consisted of pushing a Humboldt vane into the NST material to the desired depth, slowly turning the handle clockwise at constant speed until failure, and recording the value, S_v, registered on the graduated scale of the shear vane device. A shaft rod was used to prevent friction between NST and extension rod. A large 50.8 mm × 101.6 mm four-bladed vane size was used to measure shear strengths in the range of 0 to 8.125 kPa.

The cone penetration test consisted of pushing a Rimik® CP140II cone vertically into the NST material at a constant speed until the tip of the cone reached the bottom of the pan. The cone was pushed at three locations. The Rimik assembly comes with two size cones (areas: 130

mm^2 and 323 mm^2). The larger size cone was used to improve the device's accuracy in the lower range of measurement. The Rimik device automatically tracks insertion depth, speed and load. It then averages the recorded data over a 25-mm range, and converts the load into a cone resistance index, q_c (in kPa).

The limitations of these tests and their applicability to NST-like material is well understood. Here, they were used as qualitative tools to improve our understanding of the NST material, to help interpret the plate load test results, and to compare the NSTs prepared at different solids contents – they provide a relative measure of resistance when comparing these materials.

2.4 *NST Index Characterization*

Two 4-litre pails (pails #1 and #2) were filled with the NST prepared on site at 64.4% SBW, and shipped to Calgary for index characterization, including Atterberg limit tests, solids content and bitumen content determination, and sieve-hydrometer analyses. These tests were conducted according to the following methodologies:

- Atterberg limit tests were completed according to ASTM D4318 standards.
- Solids contents were measured using the conventional oven technique, as per ASTM D2216 standards.
- Bitumen content was determined by means of Dean-Stark extraction method.
- Sieve-hydrometer analyses were carried out on non-bitumen extracted samples according to ASTM E11-09 and D422-63 standards, respectively. Mining fines content, defined as the mass of fines (< 44 μm) divided by the mass of mineral solids and bitumen, was obtained directly from the particle size distribution curves.

The solids contents of the NST in Pans #11, #09 and #08 were also measured on site using the quick oven technique, as per ASTM D4959 standards. NST samples were collected at various depths by pushing a 30.5 cm (1-foot) long, 2.5 cm (1-inch) diameter cylinder sampler into the tailings, then digging around the cylinder, and placing a cap at the top of the cylinder to minimize water loss when carrying the sample for quick oven testing.

3 RESULTS

3.1 *NST Index Characterization*

The results of the index testing completed on the fresh NST tailings (before any dewatering took place) are summarized below:

- Atterberg limits could not be measured because of the high sand content (about 80%). The NST is classified as a cohesionless non-plastic material.
- The solids contents of the NST in pails #1 and #2 were 66.3% and 65.6%, respectively. These values were determined using the conventional oven drying method, and are slightly higher than 64.4%, measured on site using the quick oven method.
- Bitumen content, determined by Dean-Stark, ranged from 0.12% to 0.15%.
- Particle size distribution curves are shown in Figure 3 for pails #1 and #2. Mining fines content is about 16%, and SFR is about 5.2.

The solids content of the NST material in Pans #11, #09 and #08 was again determined immediately prior to plate load testing, as described in the previous section. The measured solids contents agree with the average values estimated based on the weights of the pans. Averaged void ratio values were around 0.65 or higher. Interestingly, Robertson et al. (2011) pointed out, based on field measurements on Suncor's composite tailings in Pond 5, that effective stress starts to develop at a critical void ratio of approximately 0.65. The strength characterization results presented in the following section corroborate their findings.

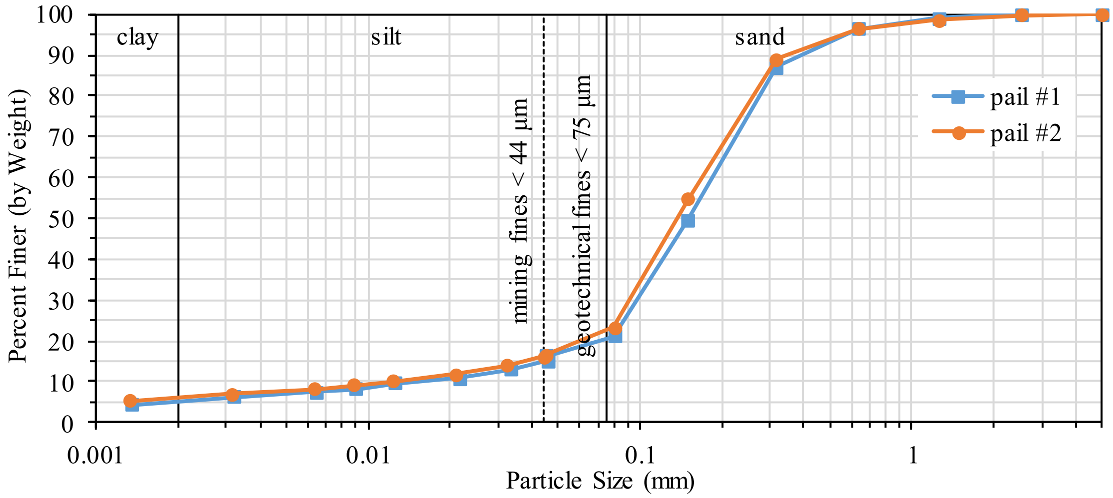

Figure 3. Particle size distribution curves of NST from pails #1 and #2

3.2 *NST Strength Characterization*

Both pocket pen and field shear vane tests generate readings that are directly correlated with shear strengths. The correlations depend on the testing device employed. For the Humboldt pocket pen, the shear strength, S_p (in kPa) was calculated using the following equation:

$$K_t = \left(1 - \frac{R^2 \tau}{c_a + \nu \tan \delta}\right)^4 k_1 \tag{1}$$

where S_q is the unconfined compressive strength; "95.76" is a factor that converts tons/ft^2 into kPa; and "16" is to account for the foot adaptor, which has an effective area sixteen times greater than the piston (Humboldt, 2010). For the Humboldt shear vane, the shear strength, S_f (in kPa), was calculated using the equation below:

$$K_t = \left(1 - \frac{R^2 \tau}{c_a + \nu \tan \delta}\right)^4 k_1 \tag{2}$$

where S_v is the value registered on the graduated scale of the vane device (Humboldt, 2011). Shear strengths obtained from field shear vanes are normally corrected to account for the plasticity and liquidity of the material being tested (Bjerrum, 1972). Such correction is not required here, as the NST is non-plastic.

The cone resistance index (q_c) has been correlated with several soil properties, including shear strength and friction angle (Kulhawy & Mayne, 1990). For the hand-held Rimik cone, the following equations were used for the shear strength, S_c (in kPa), and friction angle, ϕ (in degrees), respectively:

$$K_t = \left(1 - \frac{R^2 \tau}{c_a + \nu \tan \delta}\right)^4 k_1 \quad \text{and} \quad \tan \phi \; K_t = \left(1 - \frac{R^2 \tau}{c_a + \nu \tan \delta}\right)^4 k_1 \tag{3}$$

where σ_v is the total vertical stress; σ_v' is the effective vertical stress; N_{kt} is the cone bearing factor. N_{kt} is an empirical factor that ranges from 10 to 20 with an average of 15, the value assumed here (Eid & Stark, 1998).

Figure 4a shows the shear strengths calculated from the pocket pen and field shear vane measurements. We refrained from using the term "undrained" as we are not sure of the actual drainage conditions of the tests carried out for the project. For Pan #11, shear strengths increase gradually with depth, whereas for Pans #09 and #08, much higher shear strengths are encountered at the surface due to desiccation; however, at about 25 cm from tailings surface, shear strength values are almost as low as those measured in Pan #11.

Figures 4b,c display the shear strengths and friction angles estimated from the Rimik cone. For Pan #11, shear strengths are relatively constant. For Pans #09 and #08, much higher shear strengths are encountered at the surface of the NST due to desiccation – and near the contact between NST and beach sand layer due to under drainage. Inferred friction angles vary from about 18º–24º for the very loose NST in Pan #11 to about 27º–30º for the loose to medium dense NST in Pans #09 and #08. The desiccated NST, as expected, shows higher friction angles, above 40º in average. The results from the Rimik cone, pocket pen and vane shear tests are consistent with one another.

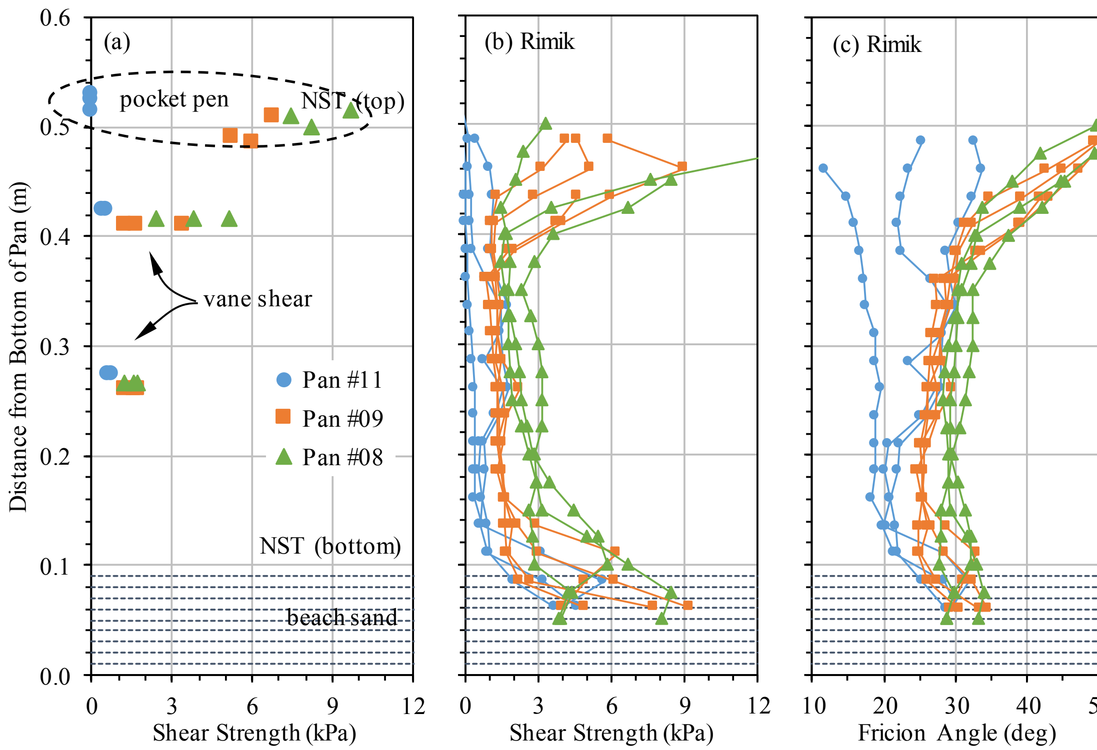

Figure 4. Profiles of (a) shear strength from pocket pen and vane shear tests, (b) shear strength from Rimik tests and (c) friction angle from Rimik tests for Pans #11, #09 and #08.

3.3 *Ultimate Bearing Capacity*

Five plate load tests were completed within two days: one in Pan #11, two in Pan #09 and two in Pan #08. The results of the plate load testing are presented in Figure 5, in terms of averaged pressure (force divided by plate area) versus displacement curves. These curves portray the typical plate load test response of loose to medium cohesionless soils (Lambe & Whitman, 1969). That is, for the loose to medium NST in Pans #09 and #08, the curves show an initial stiffer mechanical response followed by a softer behaviour after a sharp bend in the curve, corresponding to a local shear failure. The initial stiffer response is likely governed by the presence of desiccated material at the surface of the NST. For the very loose NST in Pan #11, the shear zones at the sides of the footing are not well defined and no surface heave was observed. This is termed “punching failure”, much like that observed in Pans #09 and #08 (Figure 6).

Typically, the “bearing capacity” (q_l) of the soil is defined as the pressure at which there is a pronounced change in slope, corresponding to the “knee” or bend in the pressure-displacement curve. The “ultimate bearing capacity” (q_{ult}) is the bearing pressure that causes a sudden catastrophic settlement of the foundation (Lambe & Whitman, 1969). This catastrophic settlement was not observed in our tests for two reasons: (a) the NST was loaded at a displacement-controlled mode, and (b) the shear strength of sand-dominated materials depends on stress and confinement levels; that is, when the load increases, so does the stress level and therefore the shear strength. As such, the ultimate bearing capacity of sands increases as more load is applied, and failure is not clearly defined. For that reason, some authors have defined the ultimate bearing capacity of coarse-grained soils as the bearing pressure for a settlement equivalent to 10% of the plate diameter (Briaud, 2013).

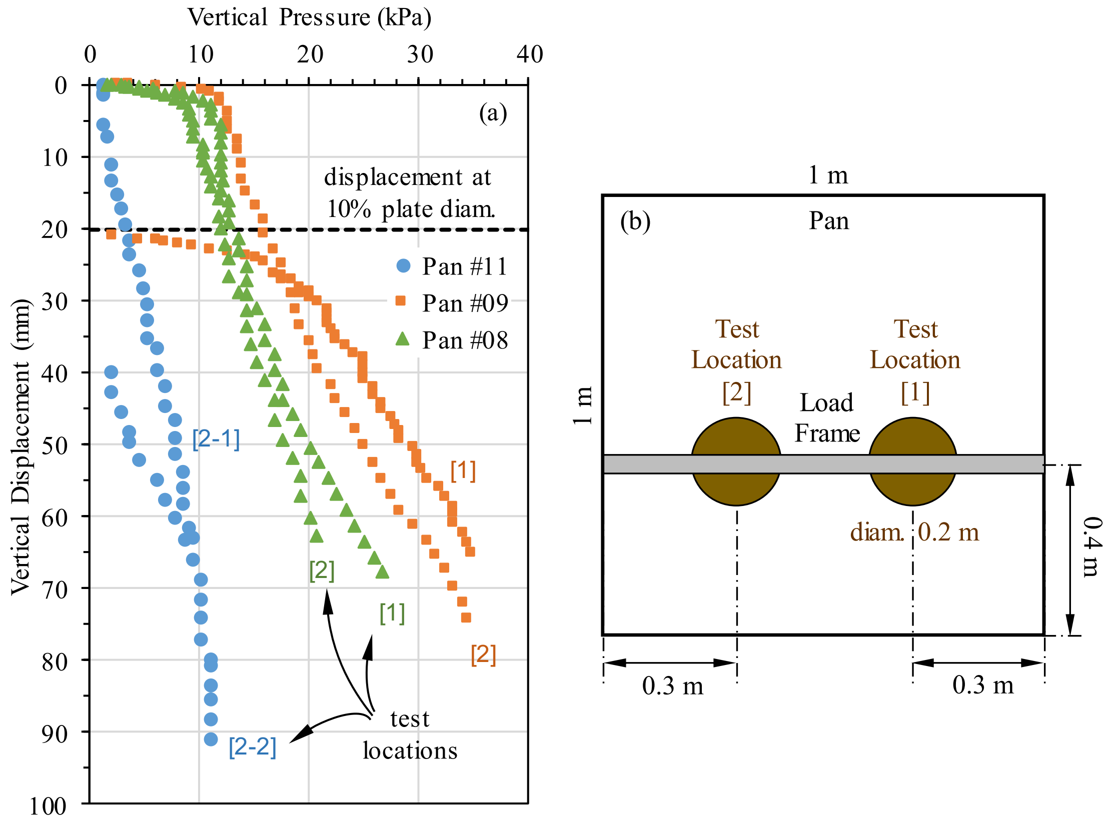

Figure 5. Plate load testing (a) results and (b) locations.

Figure 6. Punching failure observed on the NST in (a) Pan #11 and (b) Pan #08

In light of the above discussion, the bearing capacity of the NST material varied from about 1 kPa for the 76.3% SBW material to around 9–12 kPa for the 80.3–80.7% SBW material. If one resorts to Briaud's definition of ultimate bearing capacity, then q_{ult} varied from about 3 kPa for Pan #11 to around 12–16 kPa for Pans #09 and #08. Evidently, if larger settlements can be tolerated (e.g. in case of construction equipment), then higher ultimate bearing capacities can be expected. For example, for a plate displacement of 60 mm, an ultimate bearing capacity of 20-30 kPa is obtained.

Results of field plate load tests can be used to estimate the ultimate bearing capacity of actual footings (track or wheel footprint). For cohesionless soils, the ultimate bearing capacity of actual footings is proportional to the ultimate bearing capacity obtained from a plate load test by the ratio of footing size to plate size (Das, 2010).

4 SIMPLIFIED BEARING CAPACITY ANALYSIS

Several formulations are available to calculate the ultimate bearing capacity (q_{ult}) of soils. Terzaghi's, Meyerhof's, Hansen's and Vesic's methods are among the most widely used ones (Bowles, 1988). Meyerhof's seems to be the best method for frictional soils, and will be applied here. For concentrically loaded circular footings, Meyerhof's formulation (in SI units) reduces to the following equation:

$$K_f = \left(1 - \frac{R^2 \tau}{c_a + \nu \tan \delta}\right)^4 k_1 \quad (5)$$

where:
- c (in kPa) is the cohesion (or shear strength) of the soil.
- ϕ (in degrees) is the friction angle of the soil.
- γ' (in kN/m^3) is the effective unit weight of the soil.
- q (in kPa) is an external load (equal to $\gamma'D$ for loading plate embedded into the soil).
- B (in m) is the plate diameter; D (in m) is the embedment depth of the plate.
- K_p is the coefficient of passive resistance (= tan2(45º + ϕ/2)).
- N_c, Nq and N_γ are dimensionless bearing capacity factors; $N_c = (N_q - 1)\cot(\phi)$, $N_\gamma = (N_q - 1)\tan(1.4\phi)$, $N_q = e^{\pi \tan\phi\, Kp}$.
- s_c, s_q and s_γ are dimensionless shape factor; $s_c = 1 + 0.2Kp$; $s_q = s_\gamma = 1 + 0.1K_p$ for $\phi > 10º$.
- d_c, d_q and d_γ are dimensionless depth factor; $d_c = 1 + 0.2\sqrt{K_p}\,(B/D)$; $s_q = s_\gamma = 1 + 0.1\sqrt{K_p}\,(B/D)$ for $\phi > 10º$.

Table 2 compares measured and calculated ultimate bearing capacities under various modelling assumptions. The mechanical properties of the NST (i.e. shear strength and friction angle) were obtained from the Rimik cone data. The saturated unit weights were calculated directly from the solids contents in Figure 3. In all cases, Meyerhof's method overestimates the ultimate bearing capacities, with the best estimates being based on shear strength values. This suggests that the friction angles in Figure 4 are slightly overestimated.

In Meyerhof's formulation, the soil was assumed homogenous. This may not apply to Pans #09 and #08 because of the upper 5–10 cm NST material. Employing Meyerhof's method for a strong soil layer over a weak soil layer profile (Hanna & Meyerhof, 1980) yields ultimate bearing capacity estimates that are still comparable to the homogeneous case with the loading plate at surface.

Table 2. Measured versus calculated ultimate bearing capacities based on Meyerhof's method

		Pan #11	Pan #08	Pan #09
	SBW_{ave} (%)	76.3	80.3	80.7
	$\gamma_{s(ave)}$ (kN/m^3)	18.6	19.8	19.8
Measured q_{ult} for plate 20 mm and 60 mm below NST surface	$q_{ult\text{-}20mm}$ (kPa)	3	16	12–14
	$q_{ult\text{-}60mm}$ (kPa)	8	28–32	20–24
Plate at NST surface and 60 mm below surface (ϕ from Rimik data)	$q_{ult\text{-}surface}$ (kPa)	4	17	30
	$q_{ult\text{-}60mm}$ (kPa)	10	32	51
	ϕ_{ave} (deg)	22º	29º	32º
Plate at NST surface / NST as a cohesive material (S from Rimik data)	q_{ult} (kPa)	4	13	20
	S_{ave} (kPa)	0.7	2.1	3.2
Plate at NST surface / Two-layered frictional material (ϕ from Rimik data)	$q_{ult\text{-}surface}$ (kPa)	NST in Pan #11 is homogeneous	21	30
	$\phi_{ave\text{-}top10cm}$ (deg)		40º	41º
	$\phi_{ave\text{-}elsewhere}$ (deg)		27º	30º

5 CONCLUSIONS

The bearing capacity of the NST material varied from as low as 1 kPa to about 9–12 kPa. The ultimate bearing capacity was found to increase with plate settlement, as expected for a sand-dominated frictional material, whose shear strength depends on stress and confinement levels. Based on the plate load test results, it was also evident that the ultimate bearing capacity of NST increases with solids content. However, because we tested three pans effectively at two solids content (Pan #11 at about 76% SBW and Pans #09 and #08 at about 80-81% SBW), there is insufficient data to derive a reliable relationship between bearing capacity and solids content. This relationship is very likely to be non-linear, as is the case for undrained shear strength and void ratio (Sobkowicz et al., 2013; Moore et al., 2014).

While considering the above, it is also important to note that we have presented a preliminary evaluation based on laboratory measurements using about 0.50 m thick layers of laboratory prepared and deposited NST with a loading surface of 0.20 m diameter plate. Inferring field estimates of bearing capacity must consider the geotechnical issues associated with extrapolating laboratory data and empirical relationships to field scale designs. This NST was prepared under controlled lab conditions. The NST in the pans were exposed to dewatering conditions, and both may differ significantly from field deposits. Furthermore, field bearing capacity must consider the larger volume and depth of soil experiencing higher stresses imposed by larger scale loads.

The NST material was at a very loose state after deposition and initial dewatering. Liquefaction of loose, saturated sands may be caused by either static or dynamic, undrained loading. The assessment of static and seismic liquefaction potential is certainly advised as part of the design of NST deposits. A similar investigation is advisable if there were potential for deposits to be trafficked by equipment applying dynamic/cyclic loads.

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