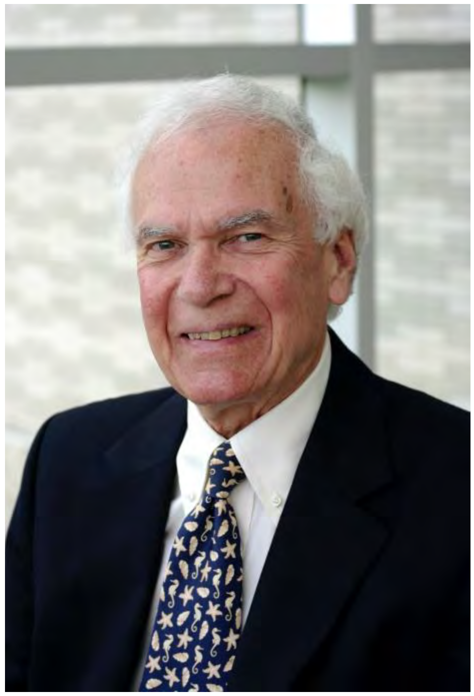
The Inaugural Lumb Lecture



The Inaugural Lumb Lecture was given by Professor N.R. Morgenstern at the Hong Kong Exhibition and Convention Centre on 10 May 2000. The following introduction was given by Professor C.F. Lee.

When the decision was made in December 1998 to establish the Lumb Lecture, the immediate task was to find the Inaugural Lumb Lecturer. It turned out to be an easy task for the Lumb Lecture Committee. Professor Norbert Morgenstern was everybody's first and obvious choice, and there was hardly any discussion required.

It was an easy decision because of two reasons. Firstly, Professor Morgenstern is a geotechnical engineer and researcher of global distinction. Secondly, he has been a guiding light for landslide hazard mitigation in Hong Kong for many years.

A native of Canada, Professor Morgenstern graduated from the University of Toronto in 1956 with a BASc in Civil Engineering. He then worked for the geotechnical consulting firm of Geocon Limited, before moving on to the Imperial College for post-graduate studies in Soil Mechanics. He was awarded the DIC and PhD in 1964. He taught at the Imperial College until 1968, when he returned to Canada and took up a Professorship in Civil Engineering at the University of Alberta. He became a University Professor in 1983, and a University Professor (Emeritus) in 1999. He also served as the Department Chair between 1994 and 1997.

Professor Morgenstern has a most remarkable and exemplary career in geotechnical engineering practice and research. He has published close to 300 papers on a wide spectrum of geotechnical subjects, many of which have become classics in our field. Even more remarkable is his long list of consulting engagements. A partial listing includes 80 dams and tailings projects; over 50 landslides and slope stability projects; 42 highway, bridge and foundation projects; 14 pipeline projects; 16 tunnelling and underground excavation projects; 10 offshore geotechnical engineering projects and numerous other geotechnical studies. In short, his insights and advice are sought on a global scale, covering a wide spectrum of major geotechnical projects. He is simply a most outstanding researcher as well as a practitioner, something that is extremely difficult to achieve simultaneously.

In recognition of his outstanding contributions to the geotechnical profession, Professor Morgenstern has been showered with no less than 42 major honours and awards, including the prestigious BGS prizes, the Canadian Geotechnical Society prizes and ASCE prizes. He has been a Rankine Lecturer, Terzaghi Lecturer and Cassagrande Lecturer. He has served on numerous prestigious commissions and committees, including the Presidency of the International Society of Soil Mechanics and Foundation Engineering.

Honours and distinctions aside, one more thing should be said about Nordie. He is as friendly and as helpful to any young student as to peers and senior officials alike. He commands tremendous respect from everyone, not just because of his contributions to the geotechnical profession, but also because of his very affable personality and his strong sense of fair play and equity among fellow human beings. He is therefore very popular and much admired both in and out of the profession. Despite his very busy schedule, he is always poised and at ease with himself and others alike. Whatever jobs that he works on, he always sets a high standard and depicts a high degree of professionalism. We are extremely proud to have someone of the calibre and standing of Professor Morgenstern as our Inaugural Lumb Lecturer.

C.F. Lee
Department of Civil Engineering
The University of Hong Kong

The Inaugural Lumb Lecture
May 10, 2000, Hong Kong

Performance in Geotechnical Practice

Norbert R Morgenstern
University Professor (Emeritus) of Civil Engineering, University of Alberta

The end product of successful Geotechnical Engineering is to ensure performance. Performance requires consideration of safety, serviceability and affordability. Serviceability criteria apply to such considerations as limited deformations, limited leakage and consistency with environmental constraints. The difficulties in assuring performance are often underestimated. It is a complex process that underpins all of the value-added contributions of Geotechnical Engineering. When it goes wrong, the penalties are severe.

The value of prediction in performance assurance has been over-estimated. Case histories will illustrate the limits of prediction. Additional case histories will be summarized to provide examples of both unanticipated unsuccessful performance and successful performance.

Recurrent sources of uncertainty will be summarized. The lecture will emphasize that in the face of the intrinsic uncertainties associated with geotechnical practice, the consistent application of Consequential Risk Analysis is essential to provide assurance of performance. Consequential Risk Analysis involves a number of techniques, including the Observational Method, and they will be briefly described.

Keywords: Risk Management, Uncertainty, Failures

Peter Lumb - An Appreciation

The memorial Lumb Volume, Selected Topics in Geotechnical Engineering, includes a list of Professor Lumb's publications. It is not large, but it does contain a disproportionate number of classics. His early paper with Gibson on numerical analysis of consolidation was well ahead of its time. His papers on decomposed granites and slope failures in Hong Kong remain essential references for anyone working on these subjects. Study of residual soils leads naturally to recognition of the implications of soil variability and Peter Lumb was a pioneer in advocating statistical and probabilistic approaches to geotechnical engineering. The value of pursuing these directions is only now being recognized in practice.

Professor Lumb's career was founded on excellence - the excellence of his research and engineering practice, the building of excellence at the University of Hong Kong and excellence in education and training as evident by the achievements of many of his students.

Introduction

In geotechnical engineering, as in other aspects of engineering, the overriding obligation is that the constructed (manufactured) entity or process fulfills its intended function. That it should do so safely, economically and in an environmentally acceptable manner are also generally desirable but, when taken together, are not always essential. Examples include the dam which must store water in a safe, economical and environmentally approved manner; the foundation which must support the load in a safe and economic manner, and the landfill which must function in a contained manner, while still being economical and in compliance with environmental regulations.

The geotechnical engineer has a long tradition of success in meeting these requirements under conditions that differ from many other types of technological endeavours. However, in some areas of application inadequate performance remains distressingly frequent. Morgenstern (1998) listed eleven serious incidents over the period 1995-1998 associated with mine tailings and waste overburden management, each involving geotechnical input from consultants, well-known by either national or international standards. He concluded that "......a well-intentioned corporation employing apparently well-qualified consultants is not adequate insurance against serious incidents". Failures associated with landslides and earthquake-induced ground movements remain alarmingly high in many parts of the world indicating that, notwithstanding the successes of the past, there is no place for complacency in the future. One might even sense more concern over future practice as a result of the increased ease of computational effort and the reduced exposure to geological reality that characterizes many of our educational programmes.

The natural materials that the geotechnical engineer must deal with are complex and do not afford the luxury of replication. Geotechnical undertakings, either in-situ or associated with unit construction processes themselves, are performed under circumstances very different from the controlled environment of a manufacturing plant. The construction and testing of a prototype, prior to production, is a procedure rarely available to the geotechnical engineer. As a result, uncertainty is a perpetual component of geotechnical design and construction.

The literature on uncertainty and attempts to model it is extensive (e.g. National Research Council, 1994). For practical purposes Morgenstern (1995) adopted the following three sources of uncertainty:

i) Parameter uncertainty
ii) Model uncertainty
iii) Human uncertainty

Parameter uncertainty is readily understood and has received considerable attention in the geotechnical literature. It is concerned with input variables such as the spatial variations of parameters like strength or compressibility and the lack of data for key parameters. Many examples exist in the literature in which the statistical distribution, say, of strength is specified and the traditional Factor of Safety is replaced by a probability of failure. The model, an equation for Factor of Safety based on limit equilibrium assumptions, is taken as certain. However, if, for the real problem, the model itself is a major source of uncertainty, the seemingly sophisticated calculation is meaningless because the major source of uncertainty has not been addressed.

This is not to suggest that probabilistic analyses emphasizing parameter uncertainty are not useful. There are many instances where the opposite is true and their capacity to consider parameter uncertainty has become extremely powerful. Christian et al (1992) provided an example of the probabilistic design of dikes on soft clay illustrating how the components of uncertainty including data scatter, spatial variation, and systematic uncertainty of each soil parameter involved in the design can be considered.

Model uncertainty arises from gaps in the scientific theory that is required to make predictions on the basis of causal inference. Model uncertainty abounds in geotechnical practice. Vick (1994) has listed components of model uncertainty that affect the reliability of assessing failure of a particular dam. He emphasized that they encompass not just approximations in various methods of numerical analysis, but also uncertainties associated with conceptualization and interpretation of all of the various processes that could lead to the failure of a particular dam. Examples included seismic liquefaction triggering, post-earthquake behaviour, undrained versus effective strength characterization, and the progressive development of internal erosion.

While the objective of science is to provide explanations, that of engineering is to provide performance. Performance of engineering systems cannot be provided independent of human involvement and the functioning of social organizations. Human error can obviously overwhelm an otherwise effectively operating system and risk analysis that ignores or understates human involvement in geotechnical practice borders on naivety. Even corruption is not unknown.

The value-added component of geotechnical engineering is closely linked to performance assurance. When performance is inadequate the penalties are severe for all involved. The fundamental premise of this lecture is that the complexity of performance assurance has been underestimated. This requires broad recognition. The correct application of comprehensive risk management tools provide the only way forward and deserves appropriate rewards. Risk management can only be successful if critical sources of uncertainty are understood. Case histories illustrate examples of unanticipated unsuccessful performance here, while other case histories illustrate successful performance assurance even though geotechnical behaviour was uncertain.

The presentation concludes with an overview of risk management tools appropriate for enhancing performance assurance.

Prediction and Performance

Assuring performance through prediction is attractive. It presupposes sufficient knowledge and precisely enough models to allow quantitative forecasts of behaviour. Relying on prediction does not preclude intervention by application of the observational method to re-direct the performance assurance process.

Lambe (1973) has provided the strongest defense of prediction in geotechnical engineering. His classification of predictions; Class A - Before event, Class B - During event and Class C - After event, has entered the lexicon of geotechnical engineering. Lambe was an advocate of Type A predictions. He regarded them as more useful than Type B predictions even though he was fully cognizant of the limitations of the data available to the engineer and of the reality that mechanisms involved are rarely fully and correctly identified. Lambe was of the view that it would be desirable if Class A predictions permitted all the judgement decisions to be made at one stage and be clearly identified and discussed.

No one can doubt that quantitative forecasts of events to come are important in geotechnical engineering. For many assignments, such as those involving the assessment of settlement, they are the essence of the geotechnical undertaking. However, it is debatable whether an emphasis on Class A prediction in geotechnical practice is appropriate when compared with reliance on Class B prediction and other components of geotechnical risk management. At least it is of interest to evaluate the accuracy with which Class A predictions can be made before emphasizing their value.

Over the past twenty-five years or so a number of prediction competitions have been convened in geotechnical engineering. A synthesis of the results from these events provides some basis for assessing the accuracy of prediction in geotechnical engineering. However, it should be noted that even these competitions exaggerate the reliability of quantitative prediction in geotechnical practice. They usually present more comprehensive data than is normal, sites tend to be homogeneous, there is a consensus that model uncertainty is not overwhelming and human uncertainty is essentially eliminated.

In order to assess the quality of the predictions the descriptors presented in Table 1 are proposed.

Accuracy of Prediction (% actual)	**Quality Class**
95 - 105% (within ± 5%)	Excellent
85 - 95% or 105 - 115% (within ± 15%)	Good
75 - 85% or 115 - 125% (within ± 25%)	Fair
50 - 75% or 125 - 150% (within ± 50%)	Poor
<50° or >150%	Bad

Table 1 - Prediction Quality Classes

The first competition reviewed here is the MIT Prediction Symposium (MIT, 1974) involving the loading to failure of a large embankment on Boston Blue Clay. On the basis of comprehensive data provided before the embankment was loaded, ten teams or individuals were asked to predict the failure height of the embankment, as well as a number of pore pressures and displacements. Only failure height will be discussed in the following.

The ten predictions from the MIT competition discounting ranges of prediction, are summarized in Figure 1 according to Quality Class. The range was 43 - 144% of the correct answer. Poor to Bad predictions embraced 70% of the attempts.

A more comprehensive prediction competition for an embankment on soft clay was undertaken in Malaysia (MHA, 1989) and it attracted 31 participants. A substantial amount of shear strength data was collected as well as index testing to characterize the site variability. Details are provided in MAH (1989) and the results of the predictions are compiled by Poulos, Lee and Small (1990) and Brand (1995). Only predictions of the collapse height of the embankment are considered here.

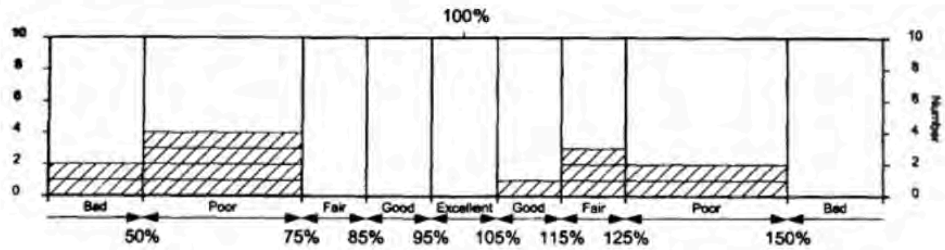


Figure 1. Prediction Quality Classification, MIT Embankment Prediction Competition (10 predictors)

Figure 2 summarizes the predictions according to Quality Class. The range was 52-170% of the correct answer. Poor to Bad predictions accounted for 55% of the attempts.

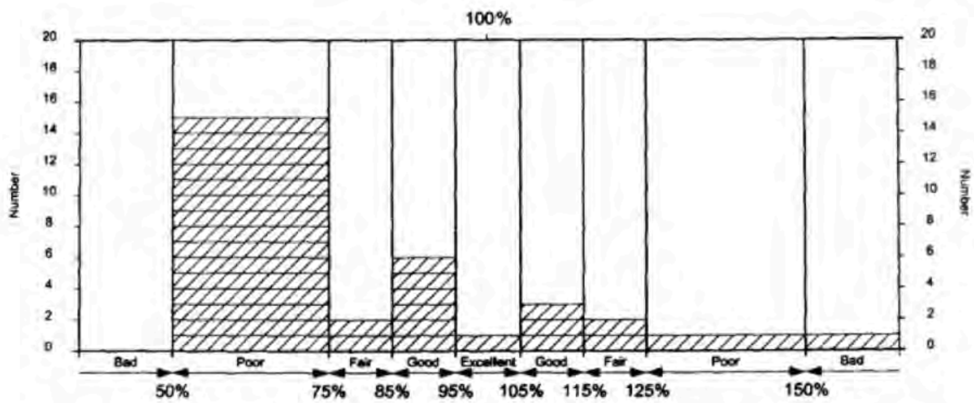


Figure 2. Prediction Quality Classification, Muar Embankment Prediction Competition (31 predictors)

Turning to foundation behaviour, Briaud and Gibbens (1995) summarized the test results and accuracy of predictions for five large spread footings on sand. Soil characterization and data were obtained by almost every conceivable test, presumably to a high level of care and accuracy. Thirty-one predictors participated.

Five different footings were involved, requiring some consideration of scale effects. Load-settlement and behaviour for each footing was determined and predictors were asked to forecast footing capacities at 25 mm settlement and 150 mm settlement (taken as failure), as well as settlements under the design load. In a typical design, one uses a factor of safety of 3 on the ultimate load and uses as the design load in this competition the minimum of the load leading to 25 mm settlement and one third of the ultimate load. Only the 2.5 m footing is analyzed here.

Figure 3 shows the distribution of predicted maximum bearing pressure according to Quality Class. The range is 15% to 230% and 63% are classified as Poor to Bad predictions.

The distribution for forecast settlement under the design load is given in Figure 4. Here the range is 0 to 347% and 90% were Poor to Bad predictions.

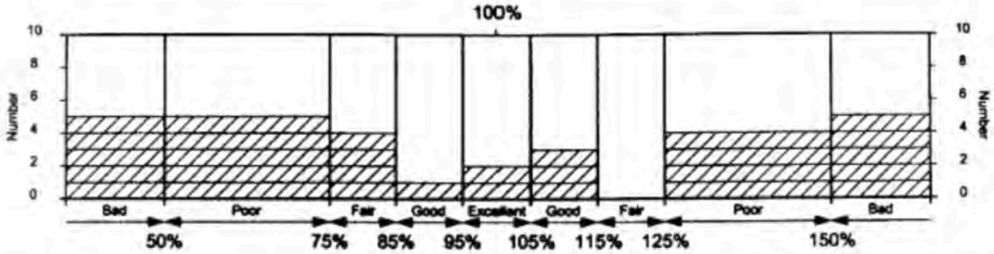


Figure 3. Prediction Quality Classification, Spread Footings on Sand, Maximum Bearing Capacity (30 predictors)

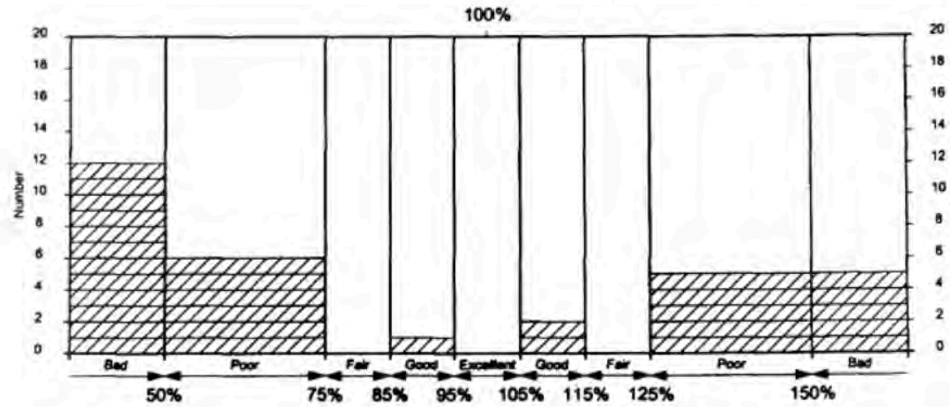


Figure 4. Prediction Quality Classification, Spread Footings on Sand, Settlement Under Design Load (31 predictors)

Finally, reference can be made to a competition to predict the performance of a single driven steel pile and an identical steel pile with a jet grouted bulb at its base driven into sand at a test site near Dunkirk in Northern France. This competition was part of a symposium to mark the 50th anniversary of the postgraduate course in Soil Mechanics at Imperial College. The results are summarized in Anon (1999). There were 16 predictions. Only the results of load capacity of the single driven steel pile are considered here.

Figure 5 shows the distribution according to Quality Class of the prediction of shaft resistance. The range is 21% to 159% and 87% are classified Poor to Bad. Figure 6 shows the distribution of the predicted base resistance. The range is 19% to 302% and 63% are classified Poor to Bad.

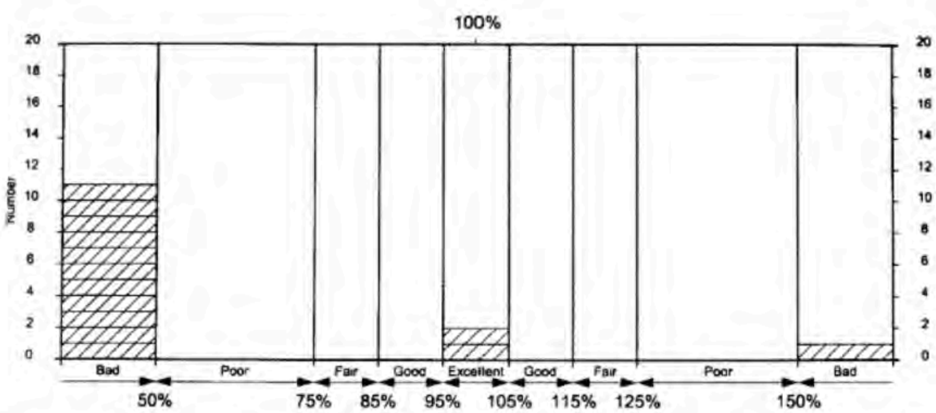


Figure 5. Prediction Quality Classification, Driven Pile, Maximum Shaft Resistance (16 Predictors)

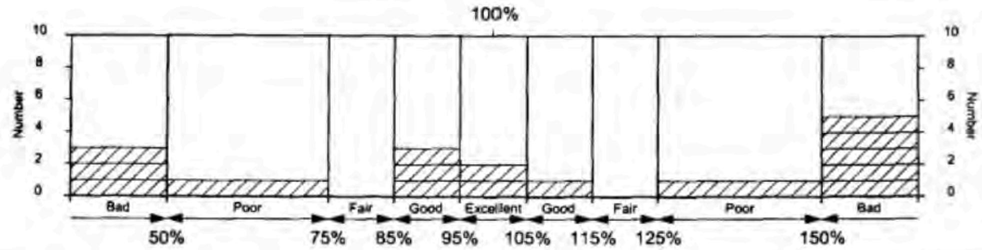


Figure 6. Prediction Quality Classification, Driven Pile, Maximum Base Resistance (16 predictors)

A fifth prediction competition concerned with the capacity of driven steel piles and cast-in-place drilled piers also leads to the conclusion that prediction capability is poor (Finno, 1989). Kay (1993, 1995) has previously drawn attention to the large variation in prediction revealed by prediction competitions and has analyzed the components of uncertainty involved.

It is evident that even under the near ideal conditions of a prediction competition, the accuracy of geotechnical prediction is poor. This is not to criticize the value of such competitions. Indeed, to be reminded of the limitations of quantitative predictions may be one of their most important contributions.

In the face of the intrinsic uncertainty associated with geotechnical engineering, it is wise to remember Southwood's caution (1985).

"The things that we would like to know may be unknowable"

It is rare for the geotechnical engineer to rely on quantitative prediction to meet his objectives. In practice reliance on quantitative prediction is only worth contemplating under the most idealized circumstances. The practice of the geotechnical engineer is more modest. Risk must be managed to overcome the limitations of site characterization, knowledge of material properties, other unknowns and the vagaries of construction practice. In order to manage this risk effectively, it is essential that the geotechnical engineer maintain an on-going awareness of factors that contribute to unsuccessful performance and introduce this awareness into comprehensive risk management tools.

Examples of Unsuccessful Performance

Fissured and Jointed Soils

Failure of a Cofferdam in Till (Author's files)

During the winter of 1971 two concrete bridge piers were to be constructed in a river in Western Canada. Pier No. 1 was closer to the shoreline of the river and Pier No. 2, which was started first was towards the middle of the river. A plan and section of a typical pier is shown in Figure 7.

Construction began by thickening the ice and then cutting a slot approximately 11m x 24m to facilitate driving to design depth of interlocking sheet piles. As pump out proceeded the supporting frames and struts were installed. The ground conditions were a few metres of sand and gravel alluvium containing boulders, followed by about 27m of dense till composed of various till sheets and underlain by an extensive regional sandstone acquifer displaying an artesian head of about 20m at the base of the till.

Pile driving for Pier No. 2 was difficult because no attempt was made to excavate the boulders in the sand and gravel layer. Final driving was hard with a set of about 10 blows/cm. Difficulties due to blow-ins and some entry of artesian water were encountered during drawdown leading to various attempts to seal the piling by re-driving and grouting. Complete dewatering was never achieved due to river inflows before the failure of Pier No. 1.

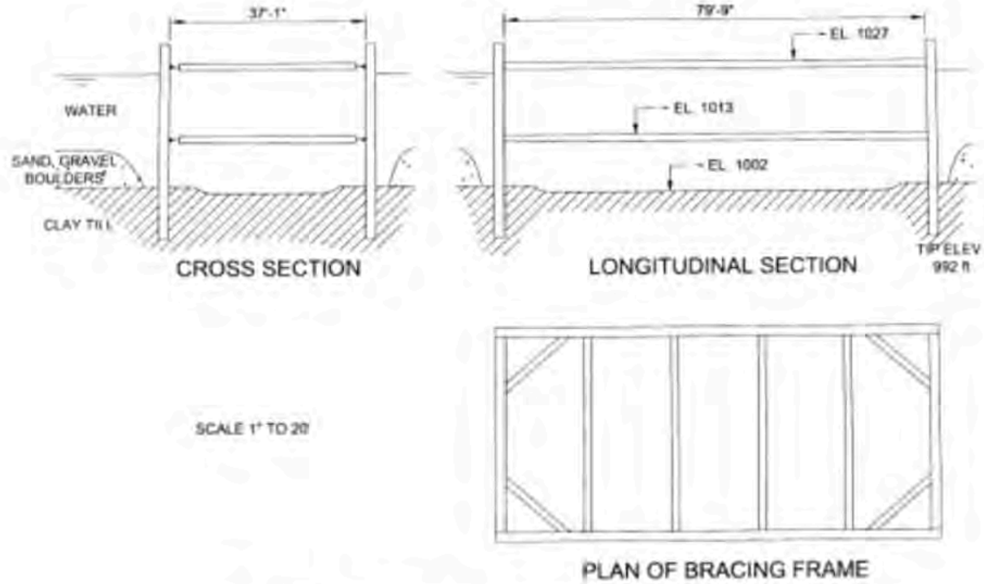


Figure 7. Failure of a Cofferdam, Plan and Sections

For the construction of Pier No. 1, boulders were excavated before pile driving was started. Thick shoes were welded to the tips of the piling during driving and driving to final elevation was easier than for Pier No. 2 cofferdam. The middle and top frames were hung in place and pumping was started. Only joint sanding was required to effect a seal and the cofferdam was effectively dewatered and excavation for the lower frame bracing was underway when the cofferdam collapsed.

The Pier No. 1 cofferdam failed violently. The collapse occurred with a loud noise and ice was broken and thrown some 10 m from the cofferdam. Eye-witnesses state that they saw the cofferdam propelled into the air and even saw the bottom of some of the piles rise clear of the ice. Collapse of the cofferdam took less than 1 minute. Fortunately, no one was in the excavation at the time.

Until the third and bottom set of struts was in place, the cofferdam relied on the passive pressure of the soil to maintain stability. This was recognized by the designers who calculated the depth of embedment of the piles based on the mobilization of this passive resistance. The passive pressure was calculated using the undrained shear strength of till samples, simple earth pressure theory and the application of a reasonable, conservative factor of safety. Subsequent investigations showed this to be in error.

Failure occurred by inadequate passive resistance, rotation at the base inward to the excavation and vertical acceleration of the cofferdam by upward water forces acting on one side of the cofferdam. The error in geotechnical design was in ignoring the influence of fissures and joints on the behaviour of the till.

The till was fissured and jointed, although stiff to hard when intact. Fissuring would be aggravated as a result of pile installation. On unloading fissures and joints would tend to open so that the soil mass could not behave in an undrained manner. The influence of the artesian pressure would further reduce the available passive resistance. Instead of the till behaving in an undrained manner, mobilizing the strength of intact till, it behaved in a drained manner, mobilizing a frictional resistance substantially reduced by uplift pressures. Passive pressures, calculated in this way were generally consistent with the mechanics of the failure.

Failure of an Embankment on Soft Fissured Clay (Crooks et al., 1986)

A highway relocation was required as part of a power project. The highway was to be carried over a mine haul road by means of a single span overpass structure with associated approach fills.

Following stripping of the surficial organics and topsoil, construction of the north fill began. Fill consisted mainly of weathered shale placed and compacted in 150 mm lifts, generally achieving in excess of the specified 95% of Standard Proctor Density. The design fill height close to the overpass structure was about 12 m above the surrounding ground surface. Just as it was close to completion, with 10-12 m of fill in place, cracking developed on both sides of the embankment, with subsequent failure as shown in Figure 8. There was little evidence of deformation on the fill surface prior to failure.

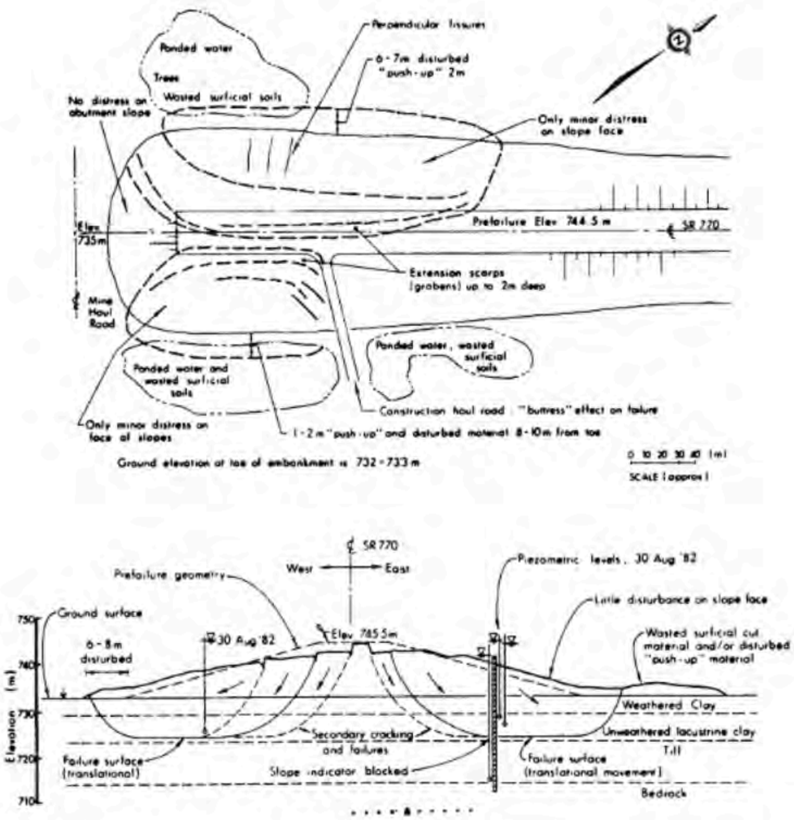


Figure 8. Failure of Embankment on Soft Fissured Clay (after Crooks et al., 1986)

The ground conditions, near Edmonton, Alberta, involve a highly plastic post-glacial lacustrine clay over glacial till. The lacustrine clay deposit can be subdivided into two units with the upper unit consisting of weathered soils, while the lower unit is unweathered. However, the lacustrine clay is fissured throughout its depth, with numerous slickensided surfaces. Original design was much influenced by a conservative interpretation of field vane tests. However, this proved to be excessively optimistic.

While laboratory UU tests of this material display considerable scatter, they reflect more clearly the influence of structure than the kinematically restrained vane test. Additional research into the clay showed that Ko varied within it as a result of having been deposited on an ablation till with subsequent melt-out. Moreover the stress-strain curve was strain weakening from a peak to a plateau mobilizing a frictional resistance of about 20º, before declining to a residual of 9º. Hence this clay was not amenable to simple normalization procedures for characterization and some element of progressive failure was involved in the embankment collapse. This clay has substantial variation of properties over a short distance, of the order of less than 100m. The post-depositional melting of the ice in the underlying till had two effects on the clay, namely:

i) the formation of slickensides and fissures as a result of internal stressing due to deformation at the bottom of the clay layer,
ii) the spatial variation in the lateral stresses in the deposit which appear to be related to modest changes in topography.

Details are given in Chan and Morgenstern (1987).

The limited resistance offered by soft fissured clays had been identified before this failure (Rivard and Lu, 1978) but had evidently not been given enough weighting in the original design.

Commentary

Fissures and joints often exercise an overwhelming influence on the geotechnical behaviour of a soil mass. They are commonly associated with stiff to hard clays and soft rocks. Indeed, they are so common in these deposits that the burden on site investigation should be to prove their absence if they are to be ignored. Moreover soft fissured clays also exist. Fissuring can also be aggravated by construction processes.
Fissuring and jointing affect all geotechnical properties. However, their influence is greatest in problems that involve unloading and the opening of discontinuities. Slickensides on fissures embrittle a clay mass, promoting progressive failure. Empirical methods dominate design in this class of materials and model uncertainty is high.

Clay Seams

Bentonites in Clay Shale (Chan and Morgenstern, 1987)

The bedrock beneath much of Interior Plains of North America is composed of compacted, non-cemented sediments deposited in epeiric seas influenced by widespread transgressions and regressions. Vertical variation from mainly marine shale at the base of the Upper Cretaceous to continental sandstone at the top is common. Volcanic activity, particularly in the Upper Cretaceous, resulted in deposits of bentonite, or bentonite enriched zones. Hence the predominantly flat-lying sedimentary rocks are comprised of sandstones, siltstones and mudstones (clay shales) with beds of coal and pure bentonite. The presence of bentonite, both as seams and as admixtures to various beds, has often been of controlling significance to a number of major engineering projects. This can be illustrated by the behaviour of the excavation for the Edmonton Convention Centre (Chan and Morgenstern, 1987).

The Edmonton Convention Centre is situated in a 20m deep excavation on the north bank of the North Saskatchewan River in the centre of the city of Edmonton. Site investigation had clarified the early landslide stratigraphy that was controlled by a deep-seated bentonite layer at Elv. 630m, about 50m below ground level, see Figure 9. The concept of draping the facility down the slope was precluded on geotechnical grounds, but approval was given to embed the facility in ground that had not participated in past landslides.

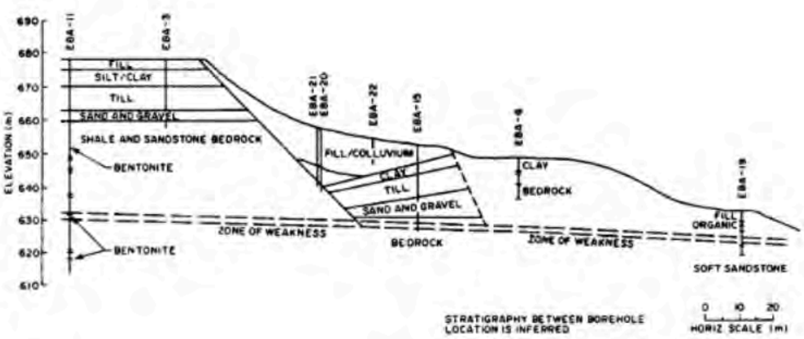


Figure 9. Edmonton Convention Centre, Geological Section (Chan and Morgenstern, 1987)

The resulting support system for the open-sided excavation is shown in Figure 10. The excavation was supported by tangent pile walls with six levels of permanently pre-stressed anchors embedded into the adjacent soil and bedrock. Design was based on providing an adequate factor of safety against overall failure for the mechanisms illustrated and minimizing movements at the top of the tied back wall in order to avoid damage to adjacent brittle buildings. The potential for progressive failure was recognized but analyses suggested that it would not be a problem at depth. This proved not to be the case. Analytical capabilities at the time (~ 1980) were limited to non-linear elastic formulations. The ability to handle strain-softening in an elasto-plastic analysis came later (Chan and Morgenstern, 1987).

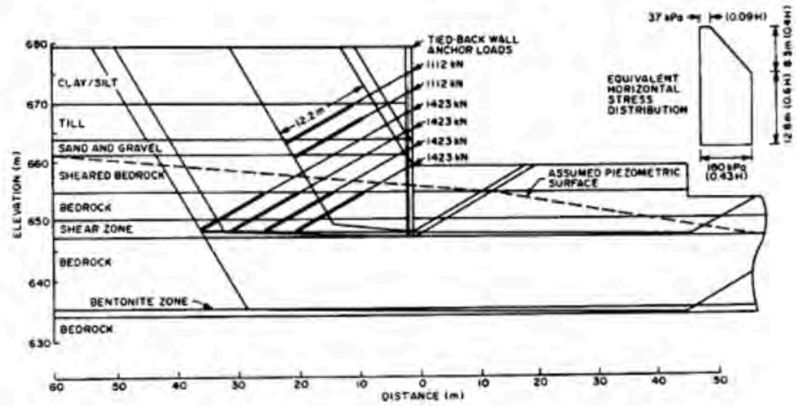


Figure 10. Edmonton Convention Centre, Support System (Chan and Morgenstern, 1987)

The performance of the excavation was carefully monitored to assure that ground movements remained within tolerable limits. Observations from the slope indicator measurements during the excavation revealed that substantial movement, in excess of what had been anticipated, was occurring along the bentonite layer at Elv. 630; see Figure 11. Even though the excavation was only 20m deep, shearing of the bentonite was occurring at a depth of 50m. This bentonite layer had been sampled during the site investigation stage and carefully inspected. It was not sheared in situ at the location of the slope indicator measurements prior to investigation. The large localized movements that occurred were a result of progressive failure due to the excavation process. This caused a reduction in the overall factor of safety against deep-seated failure. However the original design had a high enough initial factor of safety to accommodate this reduction without jeopardizing performance.

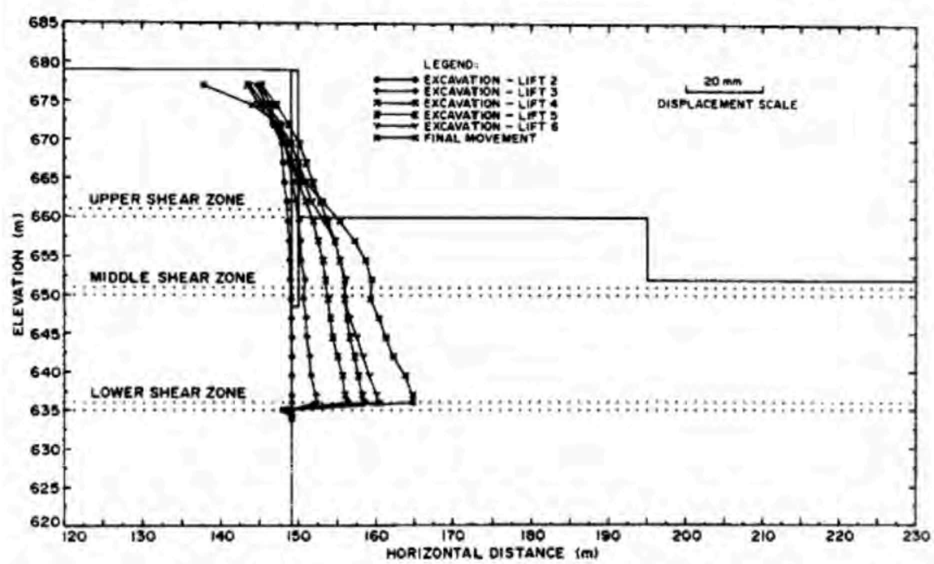


Figure 11. Edmonton Convention Centre, Shear Band Development (Chan and Morgenstern, 1987)

Kaolin in Saprolite (Parry, 1999)

Recent studies by the Geotechnical Engineering Office, Hong Kong (GEO) concluded that the distribution of clay-rich zones in Hong Kong saprolites may be widespread. A need for additional insight into origin, distribution and characterization was evident. This was prompted by the landslide failures at Fei Tsui Road and Shum Wan Road where shear surfaces developed preferentially along zones of abundant kaolin veins. Irfan (1998) has discussed the influence of structure on landslides in Hong Kong. Parry (1999) has recently completed a valuable review of the evolution of ideas with respect to the origin of kaolin in Hong Kong.

Early studies of weathering (pre-1985) concentrated on mineralogical investigations with conflicting reports on the preponderance of kaolin over halloysite. The role of weathering was clear, but the influence of hydro-thermal alteration was not. Lumb and Lee (1975) concluded that the environment, rather than the parent material, was the principal factor controlling the specific clay mineralogy of Hong Kong soils.

The influence of kaolin-enriched zones on slope stability became better recognized with the work of Irfan (1986) who emphasized a hydro-thermal alteration mechanism for the origin of kaolin seams. Subsequent studies of the kaolin shear zones at Fei Tsui Road and Shum Wan Road concluded that hydro-thermal alterations resulted in kaolin veins associated with certain mineralization. This was followed by weathering which may have contributed to the clay genesis and movements.

However Kirk et al (1997) made the following observations based on their study of the kaolin seams at Fei Tsui Road and Shum Wan Road. It was noted that the kaolin occurred within or close to the soil/rock interface and its distribution appeared mainly related to weathering, with hydro-thermal alterations only locally important. The kaolin accumulations at both sites were commonly associated with multiple minor movements reflected in shearing along clay stress relief joints and opening of sub-vertical joints. It was suggested that this movement then allowed kaolin in-filling to occur and that as the kaolin was deposited, it increasingly acted as an aquiclude promoting further kaolin deposition.

Hence an understanding of kaolin rich zones has involved a series of hypotheses, the first being in-situ weathering; then the influence of hydro-thermal events was emphasized and now the view that hydro-thermal effects are only locally important and that the main process of formation and redistribution of the kaolin is by weathering.

The presence of kaolin seams in saprolite exercises an important restraint on slope stability in Hong Kong. While the presence of such seams can be detected by conventional site investigation techniques, the interpretation of their significance on any specific site is hampered by a lack of an adequate genetic process model.

Commentary

Many geotechnical environments involve clay seams. The two examples here represent diverse origins, from deposition of volcanic ash in Upper Cretaceous marine sediments to secondary accumulation at rock head in saprolites. Clay seams affect all geotechnical properties and when they are reduced to residual strength, they dominate stability. Unsuccessful performance on a number of projects can be attributed to inadequate understanding of the presence of clay seams, inadequate site investigation and logging techniques and inadequate geomechanical characterization.

Loose Cohesionless Deposits

Sau Mau Ping (Knill, et al, 1976)

On 25 August 1976, soon after 10:00am, the fill slope immediately behind Block 9 of the Sau Mau Ping Estate in Hong Kong failed. The resulting mud avalanche buried the ground floor of the block killing eighteen people. This disaster had wider implications in that 71 persons were killed in a slope failure, also in fill and also at Sau Mau Ping in 1972. The Final Report of the Commission of Inquiry concluded, with regard to the 1972 failure, that no fault was found "with the manner in which the design and construction of the embankment was carried out". Since the 1976 failure made this conclusion suspect, the Government of Hong

Kong appointed an Independent Review Panel on Fill Slopes to advise on the cause and implications of the 1976 Sau Mau Ping failure. In retrospect, this proved to be a major turning point in the evolution of geotechnical engineering in Hong Kong.

The Panel was tasked to report on the following:

a) the cause of the recent failure.
b) assessment of risk of further failures in the recently failed slopes and in other fill slopes which may affect public housing areas.
c) feasibility of temporary and permanent remedial works for the recently - failed slopes.
d) assessment of risk of failures in fill slopes elsewhere taking account past design and construction practice in Hong Kong.
e) recommendation on design of future fill slopes.

The Panel, with the assistance of others (Binnie and Partners, 1976) recognized that the 1976 slope failure was the result of infiltration during intense rainfall, in end-tipped, loose fill, followed by loss of strength and consequent conversion of the upper few metres of the fill into a destructive mud avalanche. Loose fill was found in many other slopes in Hong Kong, and a program of re-compaction of the surface layer of fills was recommended. Improved specifications were to be adopted for the design and construction of future fill slopes. Finally the Panel recommended "that a control organization be established within the Government to provide continuity throughout the whole process of investigation, design, construction, monitoring and maintenance of slopes in Hong Kong".

The Government of Hong Kong accepted these recommendations. Compaction of old fill slopes has been an on-going remediation programme. A recent review by the Geotechnical Engineering Office has confirmed its effectiveness while observing that other methods of mitigation may be attractive under special circumstances. It is of interest to note that the recommendations of the Panel were not intended to eliminate slope instability. Given the steepness of fill slopes in Hong Kong, some instability is inevitable. However by densifying the fill, the transformation by liquefaction into a mud avalanche was eliminated, thereby substantially reducing the risk to public safety.

The Government also established the Geotechnical Control Office (GCO) which later became the Geotechnical Engineering Office (GEO). This organization has generally worked under the remit suggested by the Review Panel, but at a scale much greater than originally envisaged. Under the leadership of a series of distinguished Directors, it has grown to be an internationally recognized centre of excellence.

Stava Tailings Dam (Morgenstern, 1996)

On July 19, 1985 two upstream construction tailings dams near Stava, Italy failed catastrophically, resulting in the destruction of two villages and causing extensive property damage. A total of 190,000 m^3 of liquefied tailings flowed down the Stava valley resulting in the loss of 269 lives. The flow reached the village of Tesero, a distance of 4 km from the mine, in only minutes. The failures occurred without any warning and are not attributed to earthquake shaking or storm water flooding. Morgenstern (1996) provides additional references and a brief history.

The dams composed of cohesionless mine tailings, were generally constructed by the upstream method, utilizing cyclones. Pond water was kept well back from the shell of the dam by means of a decant system. The mine was shut down in 1978 and the dams were abandoned.

Under new ownership, mining operations recommenced in 1982. At this time, the technique of embankment construction was switched from hydraulic to mechanical placement. Soils for embankment construction were obtained from near the upstream edge of the supernatant pond, above the tailings dam. This material, consisting of sands and gravels, was placed in a somewhat segregated manner. There is no evidence of routine inspection by the new owner.

Tailings were deposited through a single fixed cyclone which was located on the left side (looking downstream) of the basin. This manner of deposition resulted in a significantly flatter beach deposit.

Eventually, the cyclone was completely abandoned in favour of a single, fixed spigot point discharge. Tailings deposited in this manner formed a sloping delta, displacing the pond water, and allowing the pond to encroach on the main embankment (Figure 12).

Figure 12. Stava Tailings Dams Before Failure (note impounded water distributions)

D'Appolonia and Morgenstern (1988) conducted a forensic investigation that lead to views that differed from previous studies. They concluded that failure of the upper dam was caused by a high phreatic surface within the steep sand shell, creating the initial failure which strained sufficiently to induce liquefaction of the sand materials. With the change in operational conditions in 1982, the pond was allowed to encroach on the main dam, changing seepage patterns. Since the dam was constructed with sand, but with extensive gravel layers, the water had a direct conduit through which to enter the sand shell. Failure was initiated under drained conditions. Since the sand shell was in a loose state, the straining which took place generated positive pore pressure, sufficient to induce flow liquefaction under static loading conditions. The sand shell began to flow. This then triggered undrained failure of the slimes as they suddenly lost their confinement.

Granular Waste Dumps (Dawson, Morgenstern and Stokes, 1998)

Waste dumps constructed under seemingly dry conditions are among the simplest of geotechnical structures and, provided they are located on strong foundations, might be thought to present few problems. Unfortunately this has not proven to be the case in the coal industry of British Columbia. Here mining is carried out in steep mountainous terrain and haulage costs are very sensitive to the proximity of the free dump waste piles to the pit areas. Since the late 1960's, when higher production mining was initiated, there have been incidents of long runout waste dump flowslides, mostly in the British Columbia mountain coal mines. The number increased during the 1980's as coal production increased.

These waste dumps are typically end-dumped fills on foundation slopes steeper than 15°. The foundation materials overlying bedrock typically consist of granular colluvium derived from weathering and slope wash and/or alpine dense glacial tills.

There have been about 50 high runout facilities during the last 25 years with a mean runout of 980m and a maximum exceeding 3,000m. Most were founded on dense till or alluvium. Failure at dumps, including fatalities, have occurred at abandoned dumps, indicating that the processes triggering mobility are not uniquely tied to active dumping.

Recent detailed field, laboratory and theoretical studies have shown that the most plausible failure mechanism is collapse inducing liquefaction flowslides. The materials composing the dumps are sandy gravels in grain size and they form layers parallel to the dump face as a result of end-dumping. This was also a factor at Sau Mau Ping. They exhibit collapse behaviour at void ratios greater than 0.3, which readily develops in the upper 50m of the dump. Water is needed to generate high pore pressure. Limited perched water from captured snow, infiltration or blocked springs are adequate to facilitate liquefaction. High mobility results from the strain-weakening behaviour of the loose fill.

Commentary

The emphasis in recent years on earthquake-induced liquefaction has resulted in loss of appreciation of the factors contributing to statically-induced liquefaction, a phenomenon of potentially very high risk. A large number of materials are disposed to liquefaction and flow. They range from quick-clays, recent marine silts, loose sandy gravels, poorly compacted decomposed granite fill and other loose fills, to loose sands, both natural and mine tailings. It has been known for a long time that liquefaction can be triggered by both undrained and drained processes. However, a basic understanding of this initiation is more recent (Sasitharan et al., 1993).

The observational method is limited in its capacity to eliminate flow liquefaction. Warning of the onset is often minimal and the phenomenon is brittle. As emphasized by Martin and McRoberts (1999), reliance on traditional effective stress approaches to design can be dangerous.

A better understanding of both the fundamentals of loose soil behaviour and their response to different field loading paths is necessary to assure improved performance in geotechnical practice.

Construction Induced Defects

Paddle River Dam (Author's files)

The Paddle River Dam is approximately 140km northwest of Edmonton, Alberta. The valley in which the dam is located is about 600m wide and 34m deep. The embankment and hydraulic structures are primarily intended to provide flood control downstream of the site.

The dam is earthfill with a wide central plastic clay core, shells composed of locally excavated granular alluvium, an upstream impervious blanket, berms on both upstream and downstream slopes and down stream relief wells. There is a low level conduit made of concrete, located approximately one-third of the embankment length from the left abutment. The conduit barrel has a 2.6m wide by 2.6m high horseshoe shaped section and is 330m long. The flow through the low level conduit is regulated by an automatic gate control designed to maintain the reservoir elevation for provision of flood attenuation on the Paddle River. The service spillway is a concrete structure located on the right abutment.

In the valley, alluvial silts, sands and gravels mantle a grey clay till which in turn overlies an interbedded succession of silts, silty clays, sands and clayey sands (referred to as sub-till sediments), resting directly on bedrock.

The sub-till sediments are generally clayey at higher elevations, grading to sandy deposits at depth. They appear to be an alluvial, or possibly lacustrine, succession which was deposited as the glaciers advanced. The clayey member contains zones of high plasticity with discontinuous slickensided surfaces. These zones are remarkably brittle dropping from a maximum frictional resistance of 19.4° ($c' = 0$) to close to the residual value of 7.2° ($c' = 0$) after only a few millimeters of travel in the shear box. Clearly concern over progressive failure in the foundation was a dominant issue in the design.

Detailed studies of the grey till revealed windows upstream of the dam centre line, which had arisen as a result of post-glacial erosion. Because of the concern that the regional artesian pressures might communicate to the dam, a clay blanket was introduced as an upstream extension of the core.

The final cross-section of the dam is shown in Figure 13. It reflects extensive berming in the downstream direction required to support the structure as a result of progressive failure and high pore pressures in the foundation during construction. While somewhat larger than anticipated in the original design, the potential for progressive failure in the foundation was anticipated at the outset.

Due to improved foundation conditions in the upstream direction, only a small berm was required in the original design. As shown in Figure 13, extensive flattening in the upstream direction was subsequently required and the need for this was not anticipated at the outset. The instability requiring this extra support arose not from movements in the natural soil in the foundation, but as a result of construction induced defects.

In December, 1981 samples were taken from the clay till core and upstream blanket compacted fill to determine the quality of bonding in the fill. The results of the investigation indicated generally good bonding with only occasional planes of discontinuous poor bonding. The compacted material was regarded as acceptable.

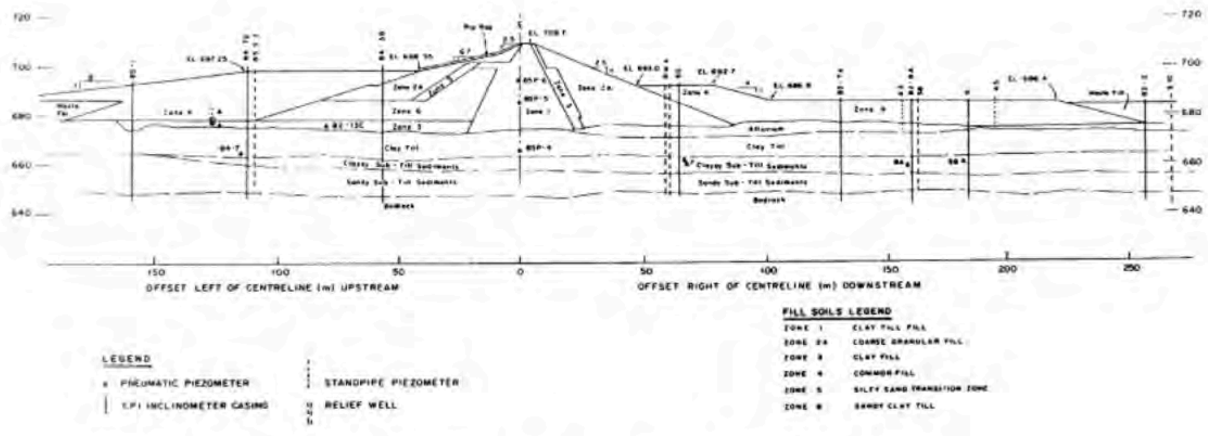


Figure 13. Paddle River Dam, Cross-Section

On September 15, 1983 the low level conduit was surveyed to determine the position and elevation of approximately 60 monitoring pins installed in the conduit invert. At that time the fill elevation was approximately 698m (10m above the berms) and there were no signs of significant settlement or horizontal extension of any of the conduit joints. On October 4, 1983, with the fill halted at 702.5m, the inside of the conduit barrel was inspected and it was found that significant extension had occurred in four of the conduit joints upstream of the gate shaft. The sum of all the joint spacing horizontal extensions was progressing at a rate of approximately 10 mm/day from October 5 through 10, and decreasing to 5 mm/day on October 15. These upstream horizontal displacement rates were confirmed by slope indicator measurements which also revealed that the displacement was taking place within the upstream clay blanket. The clay blanket shear strength had been reduced to a residual of about 15° ($c' = 0$) and a substantial upstream berm was required to achieve ultimate height of the dam and support rapid drawdown conditions.

Subsequent inspection of the compacted clay revealed that compaction had induced discontinuous slickenslides in the clay mass creating an embrittlement that is not evident in laboratory specimens compacted in the usual manner. This facilitated progressive failure in the compacted soil. It is ironical that a project so concerned with progressive failure in the natural foundation soils, experienced large movements in compacted soils. Special attention must be paid to construction induced fabric when using fills of high plasticity.

Water Treatment Plan (Author's files)

A high lift pumphouse was being constructed as part of a large water treatment plant set in a river valley. The general site conditions involved about 10 m of soft clay, silty sand and gravel overlying bedrock composed of interbedded clay shales and uncemented clayey sandstones. The pumphouse was founded on sandstone. A thin mud slab was placed to protect the bedrock surface. Some heave was noted at the time. Subsequently a structural slab, 0.76 m deep, was poured. The structural floor slab cracked within a day of completion and settled, rapidly at first and then stabilized within one to two weeks. Settlements in the range of 2 - 2.5cm occurred. This settlement response is symptomatic of void closure. A number of explanations were put forward ranging from thaw of the foundation which had been allowed to freeze to bedding closure as a result of inefficient drainage which had resulted in bed springing. In the view of the Author, it was unlikely that the foundation sandstone would have displayed such compressible behaviour had it not been allowed to freeze. The effect of freezing is two-fold. It makes the foundation impervious and promotes hydraulic jacking over a large area. It also generates frost heave with extra settlement upon thaw. However, the possibility that the mudslab alone, in the absence of freezing, could have inhibited drainage and influenced uplift effects cannot be discounted as a contributing factor. Regardless of detail, the unsuccessful performance was caused by construction-related defects.

Commentary

Assurance of geotechnical performance does not end with site investigation and design. While the observational method can be successful in detecting conditions that depart from the design basis, it alone is not sufficient to control the construction process to ensure that performance is as intended. This is dealt with by construction specifications and quality assurance programmes. It is important that the potential problems that may arise in geotechnically sensitive construction be analyzed with care to ensure that specifications and quality assurance are properly focussed.

Human Uncertainty

Kwun Lung Lau Landslide (Morgenstern, 1994; Wong and Ho, 1997)

At about 8:53pm, on July 23, 1994 a landslide occurred below Block D at Kwun Lung Lau, Kennedy Town, Hong Kong. The landslide resulted in five fatalities and more injuries.

This landslide provoked considerable public concern in Hong Kong and resulted in technical detailed inquiries by the Geotechnical Engineering Office (GEO), Hong Kong Government (1994) and Morgenstern (1994). The case history is cited here because Hong Kong probably has the most advanced landslide reduction program of any major city in the world.

Through the efforts of GEO, Hong Kong has developed a comprehensive catalogue of slopes, a risk-based ranking system and a phased approach to upgrade slopes, both in the public and private sectors. In addition, a landslide warning system has been developed based on correlations between landslide occurrence and rainfall. This warning system utilizes an extensive automated rain gauge system, operating in real time, and it has been demonstrated that few incidents precede that warning.

Together with these measures, the GEO has been instrumental in improving local geotechnical practice, highlighting responsibility for routine maintenance of slopes as well as instigating a number of other initiatives directed toward landslide prevention and risk reduction. There is little doubt that the work of the GEO has been very effective in reducing overall landslide risk in Hong Kong. Therefore it was particularly unsettling to discover that the Kwun Lung Lau landslide involved a slope and retaining wall that had been catalogued, a configuration that had been subjected to a preliminary study and assessed as adequate, a site that had been inspected periodically by qualified consultants, even shortly before the unfortunate occurrence, and had occurred when the landslide warning was in effect. These discoveries raised questions about the effectiveness of the whole risk management system and therefore extensive technical and public policy inquiries ensued.

A cross-section through the slide is shown in Figure 14. The slope was supported by a thin masonry wall that was in place by 1901. The soils consisted of loose fill and partially weathered volcanics and the slope was covered with chunam. The slope had been inspected at the time of heavy rainfall, about six hours before the slide, and no defects were detected. Rain was light at the time of the collapse although it had been extremely heavy some hours before. Failure was sudden, taking place over a very short period of time. The masonry wall burst out at about mid-height, followed by the instant collapse of the wall and slope. Had the failure mode been more ductile, it is likely that the disaster could have been avoided.

Detailed studies revealed that the landslide occurred as a result of sub-surface infiltration from defective, buried drainage systems. Flow from leaking storm water drains and a failed, foul-water sewer saturated the soil mass. Initially the wall, which was thin, provided support, but it finally buckled and collapsed in a brittle manner. The volume and mobility of the slide mass was actually increased by the presence of the wall. Had it been thicker, it likely would have deformed in a more ductile manner which could have provided some warning of the impending danger. Further inquiry revealed that the drawings relied upon by GEO in their preliminary assessment of stability did not portray the wall correctly. The wall was shown to have a base width of 4m on drawings approved in 1965 instead of the actual width of about 750mm. Had the preliminary analysis been based on the actual width of the wall, the wall would likely have been found to be unsafe and the possibility of future instability may have been foreseen.

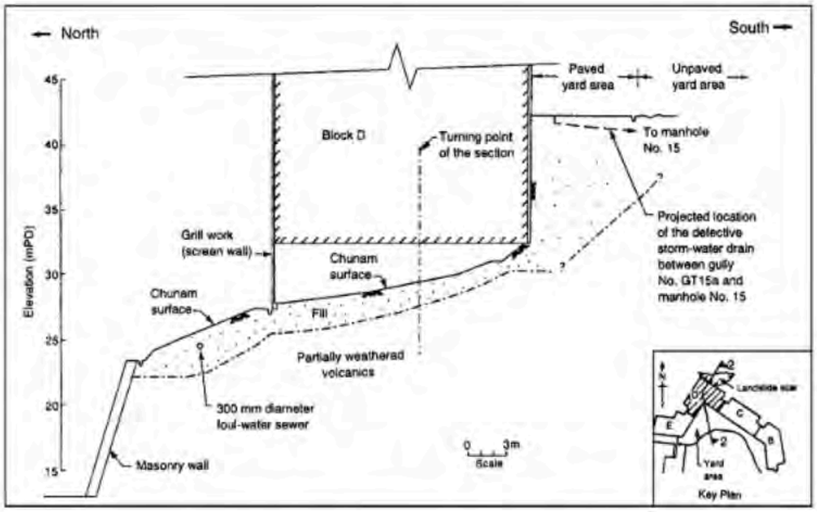


Figure 14. Kwun Lung Lau Landslide (after Wong and Ho, 1997)

While considerable attention was being paid to potential surface infiltration into slopes in Hong Kong, only limited attention was being given to sources of subsurface infiltration, particularly if a potential source was some distance away from the specific location of the slope under assessment. This has since been rectified.

The example of Kwun Lung Lau entailed parameter uncertainty, since geological and material characterization entered into the evaluation of stability. It also contained elements of model uncertainty since the study utilized limit equilibrium analyses, finite element analyses, distinct element analyses and analyses of flow through both saturated and unsaturated soils. However, these sources of uncertainty were overwhelmed by human uncertainty. In this case the filing and approval of inaccurate documents and the limited appreciation of potential sources of subsurface infiltration were dominant factors.

Commentary

Human factors as a major cause of geotechnical failure have been discussed, among others, by Peck (1973) and Sowers (1991). Li and Lee (1991) provide an extended discussion noting that human error, inadequate supervision, lack of communication between project parties during construction, and ignorance of or failure to use prevailing knowledge have all been encountered. Human uncertainty exists in all branches of engineering and an extensive literature exists (Senders and Morary, 1991).

Performance in geotechnical practice cannot be assured without invoking appropriate risk management practice to minimize the detrimental effects of human uncertainty.

Examples of Successful Performance

The Alberta Oil Sands

The Alberta oil sands, located primarily in north-eastern Alberta, comprise a vast resource of hydrocarbons. Of the 270 billion m^3 (1700 billion bbls) contained in these Cretaceous deposits, less than 10% are within the surface - mineable region, in the vicinity of Fort McMurray, Alberta. Estimates suggest that about 60% of the surface-mineable deposits are recoverable with current technology.

Commercial mining and processing of oil sands was pioneered by Suncor Ltd. (originally Great Canadian Oil Sands). The original mining scheme required removal of overburden, followed by mining with bucket-wheel excavation operating on a three-bench mining configuration. Suncor began operations in the mid-1960's. The bucket-wheel excavations have now been abandoned in favour of a high capacity truck and shovel operation.

Slightly more than a decade later, in 1977, Syncrude Canada Ltd. came on stream. At Syncrude annual production is currently about 13 million m^3 (80 million bbls) of synthetic crude oil and, together with Suncor's production, represents about 20% of Canada's petroleum needs.

Geotechnical engineering has contributed to the emergence of this success industry in a number of ways (Morgenstern, Fair and McRoberts, 1988). The examples of dragline mining and the construction of a major tailings dam, both at Syncrude, are described below.

Highwall Stability at Syncrude

Although initial experience in the industry had been with bucket-wheel mining, Syncrude adopted a dragline scheme as the primary mining method. Concerns over pit floor stability, the prospects for selective mining and higher productivity were major considerations at the time in the evaluation of the appropriate mining method. Among the negative considerations were the geotechnical restraints associated with highwall stability. While the draglines are still productive, new mine developments favour the current generation of truck and shovel equipment which are replacing draglines in new mines.

At Syncrude, the oil sand is initially exposed by a truck and shovel operation which removes 15 to 25m of overburden. Four draglines then excavated the oil sand from a 50m high operating bench. The oil sand was placed in a windrow for subsequent transfer by bucket-wheel reclaimers. Following transfer, the ore is brought to the plant site for processing by means of conveyors. Figure 15 illustrates the mining scheme.

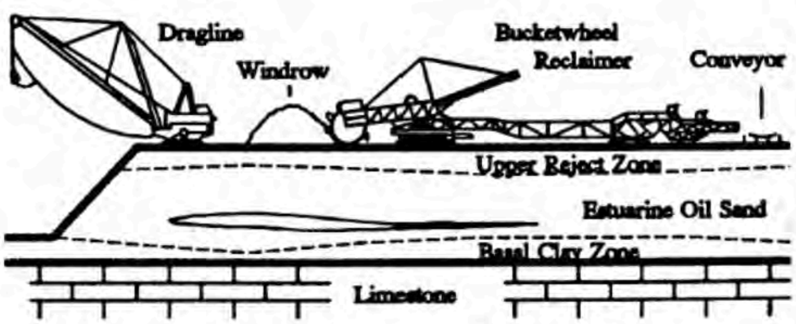


Figure 15. Syncrude Canada Ltd. Mining Scheme

As is evident from the figure, slope stability is of paramount consideration. Slope angles of 45°-50° are required to facilitate production, although the initial mine design was based on the ability to maintain somewhat steeper (60°) slopes. The geological conditions suggested that this could have been problematical.

The oil-bearing deposit of the McMurray Formation is composed of successive fluvial, estuarine and marine deposits on a previously eroded Devonian carbonate surface. The depositional setting of the McMurray Formation at this location can be broadly termed tidal. Drainage of the intertidal flats was accomplished by a network of interconnected meandering tidal channels which continuously reworked the tidal flat area. Deposition of fines can take place along the convex side of the channel resulting in the formation of point bars contain inclined layers. This depositional environment is shown in Figure 16. Estuarine clay layers can dip up to 20° normal to the channel and can control stability locally. Because of the extreme variability within the deposits, it is not possible to establish stratigraphy except by a very close spacing of boreholes. Sometimes even this is not effective if borings are made in an area of channel overlap.

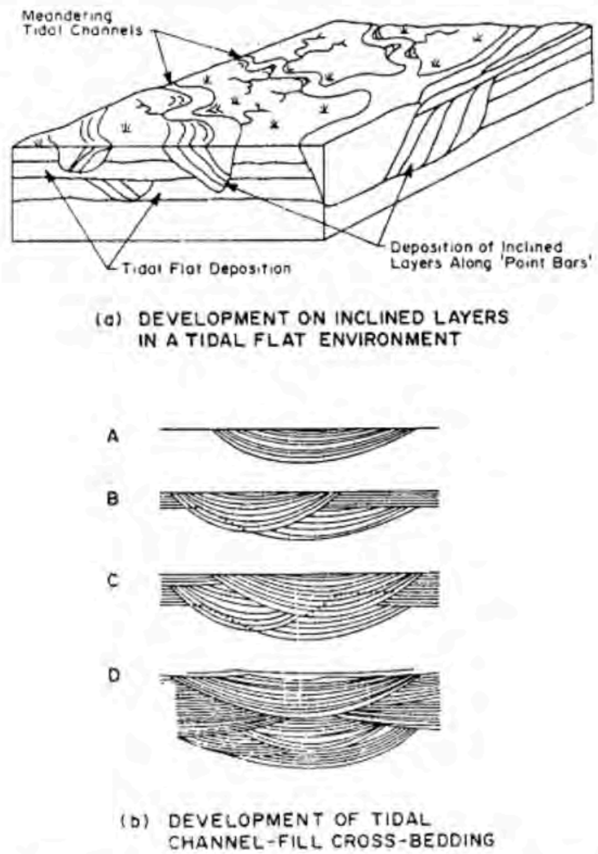


Figure 16. McMurray Formation Deposition Model

The lower estuarine oil sands typically contain in excess of 12% bitumen by weight, which is quite high grade. The sand grains are water-wet, being surrounded by a thin film of water, and the bitumen fills the bulk of the pore space. There is an appreciable quantity of gas in solution, primarily methane and carbon dioxide. It is thought that most of the gas is dissolved in the bitumen. When confining stresses are reduced, by sampling or by the mining process, gas comes out of solution and forms a bubble in the bitumen. This material is immobile at ground or room temperature due to its high viscosity and the oil sand swells. There are two important geotechnical consequences: 1) undisturbed sampling of oil-rich sands is virtually impossible, 2) gas exsolution will create high wall deformations in high grade oil sands.

The upper portion of the McMurray Formation is capped by a shallow marine sequence, which is divided into various shoreface facies. One contains continuous sub-horizontal clay layers up to 500mm thick. This is an additional defect controlling stability when these facies constitute the upper portion of the dragline mining bench. Conditions are also encountered where marine clay layers can overly inclined estuarine clay layers.

For the dragline mining scheme to be adopted, it was necessary to demonstrate that the overall strength of the oil sands was adequate and that the defects within it could be managed. The first issue was resolved by instrumented trial excavations, the study of natural slopes and investigations into the geotechnical behaviour of the oil sands themselves. Conventional triaxial tests on routine samples of oil sand indicated that the material would perform like a dense sand. Natural slopes behaved much better. As a result of studies performed on special samples, Dusseault and Morgenstern (1978) concluded that the oil sands had frictional shear strength of 60º or greater which arose from the interlocked fabric of the sand. They called this class of materials locked sands. When this fabric is destroyed, it behaves as a normal sand. These findings provided confidence that overall stability would be ensured and shifted the focus to management of defects within the clay. While there was agreement at the outset that dragline mining was feasible, attention was drawn to the risk associated with the scheme and the need to alleviate this risk by the ongoing application of observational methods.

The complex and variable geological conditions encountered across the Syncrude mine site govern the many different types of slope failure mechanisms. Morgenstern et al (1988) provide a summary. The most critical mechanisms are: i) blocks slides, ii) slope bulging due to gas exsolution and iii) translation failure in marine sediments. These critical failure mechanisms require ongoing monitoring and have their own unique impact on mining operations. Current practice with respect to monitoring has been summarized by List (1992).

Gravity driven slides in estuarine materials continue to be the greatest single threat to dragline safety. They occur where relatively shallow blocks can slide on clay beds that dip in excess of 10º towards the pit. Documented block slides have extended as far as 100m along the highwall crest with crest retrogression of up to a maximum of 75m. Characteristically, they occur rapidly with little surface expression or warning of impending failure.

The prevention of block slides relies on on-going programs of investigation, highwall mapping, past performance evaluation and monitoring. Although stratigraphy is not readily established by coring, a major contribution to hazard assessment has been made by the development of a small diameter downhole dipmeter tool. This micro-resistivity device determines the in-situ dip and dip direction of clay beds and facilitates the recognition of those portions of the highwall that will be at risk due to block slides. Dipmeter surveys are used routinely. When critical structures are identified, remedial action might be undertaken and intensive monitoring is invariably required.

Slope deformation in the lower third of the highwall has been evident in the mine since the beginning of mining and since 1984 in the central third of the highwall slope. This bulging characteristic, shown in Figure 17, is generally associated with higher grade ore and is thought to result from gas exsolution within the bitumen and subsequent expansion and breakdown of the oil sand structure which then flows under gravity. Lower slope deformation has resulted in outward flow in excess of 20m for as much as 1km along the highwall toe, resulting in as much as 20m of crest retrogression. It generally occurs on the new highwall slope within days of mining and stabilizes with time, prior to the next panel excavation. Deformation of the central third of the highwall has been more detrimental to mining, with crest retrogression and cracking up to 35m having been recorded.

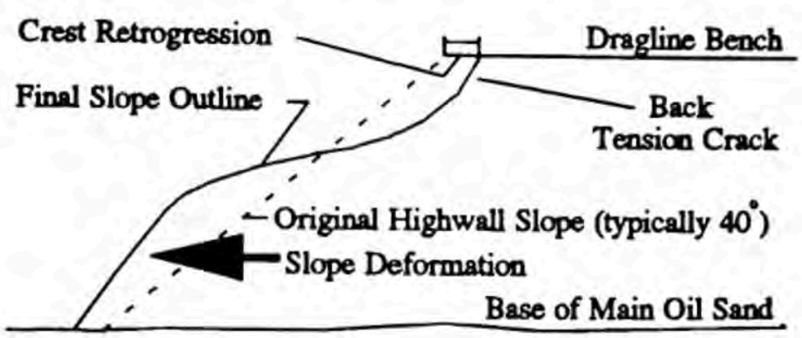


Figure 17. Slope Bulging Due to Gas Expansion

As noted by List (1992), in excess of 250 translational failures along marine clay layers have occurred in recent years, primarily affecting windrow stability. Studies indicate that these sediments are at their residual strength and it is thought that progressive failure as a result of windrow loading might be a significant factor. Windrow movements have not severely impacted production. However, there is concern that failure due to dragline shoe loading might affect dragline stability.

Production at Syncrude Canada Ltd. has increased continually since start-up. It is evident that geotechnical monitoring is an essential component contributing to increasing production while minimizing risk. Slope inclinometers are used extensively and instrumentation development has been on-going. A recorder-processor-printer (RPP) system has been adopted to aid in slope inclinometer data management and a remote slope inclinometer reading system has been successfully implemented. In order to monitor surface translational movement beyond the range of the slope inclinometer, Syncrude initiated the development of a remote robotic optical survey monitoring system. Other monitoring instruments such as pneumatic, electric, and vibrating wire piezometers, heave gauges, horizontal extensometers, sliding micrometers, tiltmeters, pressure cells and others have been used to evaluate mechanisms and performance.

Monitoring criteria have evolved empirically and are subject to on-going review. Slope inclinometers are installed in all potential block slide areas, both in the mining panel and the subsequent panel, to ascertain the total extent of a potential block slide. A reading frequency of once every 1 to 2 hours is initiated in the event lateral displacement is detected. When the velocity attains 0.45mm/hr. at a reading depth, continuous monitoring commences. To ensure no additional movement zone is left undetected, the entire slope inclinometer is read hourly. When the velocity at a reading depth exceeds 0.75mm/hr., the dragline is walked off the cut and the situation assessed. It should be noted that this criterion, although established to give advance warning of a potential block slide, does not mean that a block slide will occur. It is merely a point at which the risk to the dragline associated with on-going mining has become unacceptable. Response to the dragline walkoff is based on the detailed assessment of the situation.

Slope degradation due to highwall bulging is a more ductile process. Moreover, it occurs primarily in the newly excavated highwall as stress is released and hence the dragline is not so much at risk compared with the threat of block slides. With experience, a less restrictive set of criteria governing dragline walk-offs has been developed. They are based on a classification of cracks and monitoring their initiation and progression. Their implementation requires vigilance, frequent monitoring and a good understanding of all failure mechanisms. Criteria associated with translational failure mechanisms in the marine clay have been discussed by List and Czajewski (1991).

As a measure of risk assessment, the original geotechnical consulting board to the Syncrude project advised that management must contemplate the loss of a dragline every ten years. This has not occurred and mine production is now greater than anticipated at the outset. Syncrude has mined safely through in excess of 600km of highwall. That it has done so is due in no small measure to the geological and geotechnical understanding that has evolved and to the application of the observational method.

Syncrude Tailings Dam

The oil sand industry involves earth moving on a grand scale. At Syncrude alone, approximately 5 tonnes of material are moved every second. Accordingly, waste disposal is a major consideration. Large tailings dams are used. Challenging foundation conditions have had to be dealt with and innovations in hydraulic fill construction have contributed to safe cost-effective waste management.

The wet tailings produced by bitumen extraction require out-of-pit storage by means of a tailings pond until sufficient space has been created to turn to in-pit storage.

The tailings are predominantly a mixture of uniform fine-grained sand and water, but also contain well-dispersed silt and clay as well as some residual bitumen. On deposition in a tailings pond, the sand fraction settles out to form dykes and beaches and the oily silt and clay fractions suspended in the water forms a sludge, called fine tailings, that flows into the pond. All of these materials must be stored since only a portion of the water in the pond is clean enough for re-use in the plant. Practice indicates that in the early years of mine development, the storage volume required has been about three times the volume of oil sand mined and processed. As pond sedimentation processes advance and water recycling becomes possible, a bulking factor of 1.4 is appropriate.

The Syncrude tailings pond was designed to provide storage for 550 x 10^6 m^3 of sand, 370 x 10^6 m^3 of fine tails and 50 x 10^6 m^3 of free water. To accommodate these volumes approximately 18km of dyke, ranging from 32 to 90m in final height and enclosing a 22km^2 area has been constructed. In terms of volume of engineered fill, this is one of the largest earth structures in the world.

The tailings dam has been constructed on till overlying Cretaceous clay shales (Clearwater Formation) which are underlain by the dense oil sand formation. Glacial drag processes have sheared the clay shale in-situ to its residual strength which is very low. In addition, the low permeability of the fine-grained clay shales results in exceptionally low rates of dissipation of excess pore pressure consequent upon construction. Thus a combination of low shear strength and high pore pressure leads to low factors of safety.

Movements beneath a portion of the dyke developed along a pre-sheared, overconsolidated clay-shale layer lying practically horizontal at a depth below original ground of about 20m. A section is shown in Figure 18. Movements were predominantly in the K_{ca} horizon.

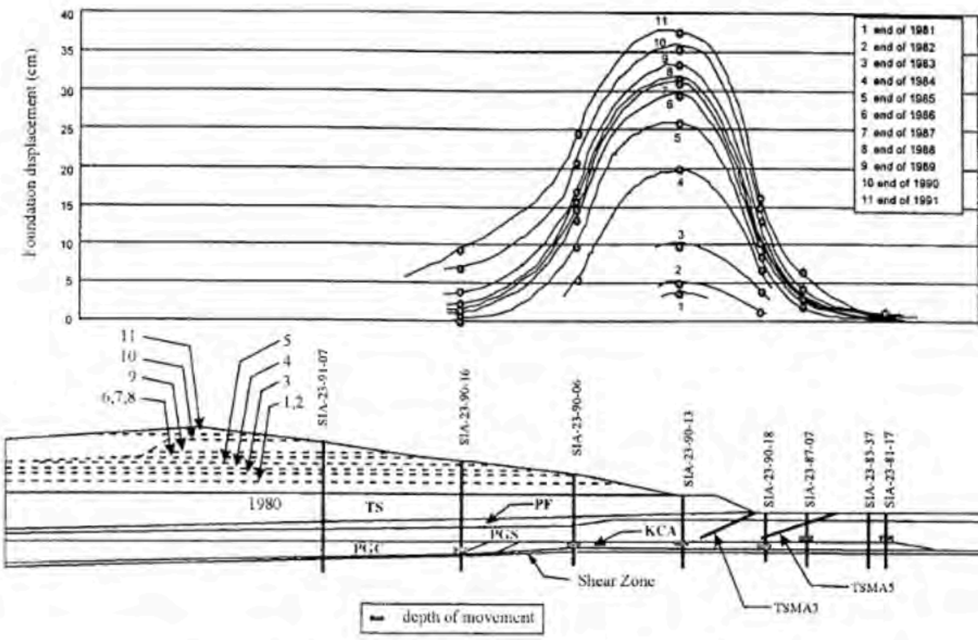


Figure 18. Syncrude Tailings Dam, Section and Movements

The largest foundation movement recorded up to 1986 exceeded 25 cm (Fair and Handford, 1986). Piezometric measurements showed substantial increases in pore pressure during construction and little indication of dissipation. Due to the slow rate of construction of the dyke and careful monitoring of extensive instrumentation, the re-designed dyke, which involved some flattening was completed to its ultimate height utilizing observational methods. At the worst location, about 43cm of slip have been accommodated in bringing the dyke to its final elevation. Nicol (1994) describes the evolution of the dyke and provides performance history.

Obviously the factor of safety against sliding in the foundation was low. Notwithstanding abundant testing for residual strength and numerous piezometers it became increasingly impractical to rely on limit equilibrium analyses to ensure stability. The dyke was essentially maintained in place by the break-out or passive resistance provided by the till (P_{gs}) and sand (P_f) overlying the weak pre-sheared clay shale (K_{ca}). Monitoring of strain by inclinometers and sliding micrometers were used increasingly to evaluate stability. Therefore a theoretical understanding of strain development in this critical area was necessary.

This was achieved by development of a finite element model of the foundation behaviour incorporating localized slip along a weak plane and using observed pore pressures as input. Details are provided by Alencar, Morgenstern and Chan (1994). Results comparing field measurements with calculated values to 1986 are shown in Figure 19. The excellent fit provided considerable insight into the stresses that might lead the structure to failure, as well as confidence that the model can be used to project allowable incremental strains in the critical restraining section. For the class of potential failure mode exhibited by the Syncrude tailings dyke, it is both possible and effective to construct a numerical model using a history - matching approach. This provides insight for the evaluation of both past and future performance.

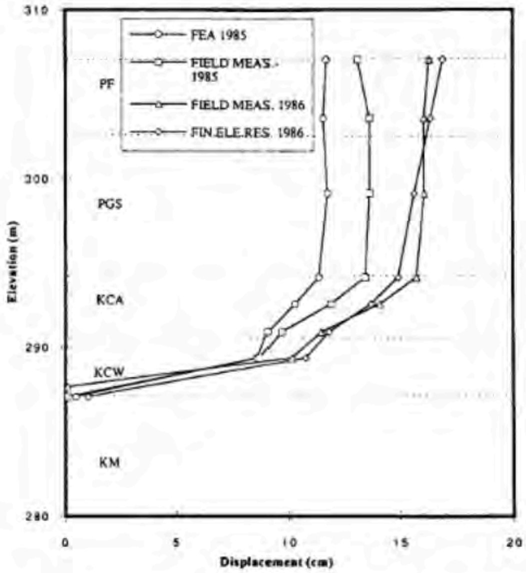


Figure 19. Correspondence Between Observed and Computed Slip (after Alencar et al., 1994)

Comme ntary

The successful application of geotechnical engineering to the oil sands industry has relied on a number of contributions including: 1) basic soil property studies, 2) advanced analytical studies, 3) geophysics, 4) instrumentation. But above all, there has been an intimate interaction between the analysis of the geological environment and geotechnical behaviour, with on-going application of the observational method.

The mining environment provides near ideal circumstances for the application of the observational method in that the resistance to making change as a result of contractual relations is lessened. However, risk-taking is part of the mining industry. Successful geotechnical practice allows one to compensate for this increased risk-taking by insisting that every effort be made to take advantage of the near ideal conditions for the application of the observational method.

Risk Management

Uncertainty is chronic in geotechnical practice and quantitative prediction of behaviour, even under ideal circumstances, is unreliable. One way forward might be increasing reliance on quantitative risk analysis (QRA) as a basis for design. However Melcher (1993) has drawn attention to some of the difficulties that affect QRA such as:

i) the limited ability to predict relevant hazard scenarios and in particular the prediction of extremely unlikely scenarios.
ii) treatment and use of data when it is extremely scarce, non-existent or incomplete.
iii) description of consequences making no allowance for their variability and uncertainty.
iv) limitations in incorporating human error.

All of these are familiar to the geotechnical engineer.

As noted by Morgenstern (1995), gathering data for QRA raises difficult issues in geotechnical engineering. Failure is normally a rare event. Many projects are unique and hence data based on replicate behaviour cannot be obtained. Design might be controlled by extreme events such as the Maximum Credible Earthquake, or extreme properties, such as the lowest residual strength which might control a foundation stability problem. Much readily obtainable data such as information from Standard Penetration Tests is only a crude cipher for real-world behaviour. Some of these limitations are overcome when the problem is rich in data (e.g. Hardingham et al., 1998) or by invoking subjective data, based on expert opinion.

Practical examples of QRA applied to dam safety, based on subjective probability assessment have been described by Vick (1994). He emphasized the ability of this type of analysis to identify failure modes, to make the judgmental processes more transparent and to facilitate risk communication. The significance of risk communications should not be under-emphasized, particularly on projects of a multi-disciplinary nature. QRA is not exclusive of other risk management tools and can be complementary to more qualitative procedures.

There has been a considerable effort expended in recent years to develop a reliability-based design approach that can accommodate different kinds of uncertainty, such as uncertainty in the geological model, parameter uncertainty, cost uncertainty and other important considerations. The work of Freeze and his students (Freeze et al., 1990; James and Freeze, 1993) is noteworthy in this regard. Reliability based design can even be compatible with the observational method provided that Bayesian statistical updating is used. However there still may be a conflict with acceptable practice if the decision criteria controlling the designs are overly simplistic. The common risk-cost-benefit approach to design is based on the calculation of the probability of failure, P_f, or the reliability $(1 - P_f)$. However, as emphasized by Hashimoto et al (1992), reliability is not always a complete measure of technical performance. Often robustness is more important. A design is robust if it has the flexibility to permit adaptation to a wide range of potential conditions at little cost. When there is large uncertainty in future loads (or capacities) robustness is very desirable. The principle of designing to provide defense in depth, commonly adopted in the design of earth dams is an example of robustness. There may be economic tradeoffs between maximizing reliability and robustness.

Risk analysis is essential to assure geotechnical performance. However, it need not be strongly quantitative as a process. Consequential Risk Analysis (CRA) is the appropriate process for risk reduction in geotechnical engineering.

CRA assumes that an event occurs and attempts to prevent a particular outcome from occurring or to mitigate the impact of the outcome. Some of the techniques used involve analytical processes such as sensitivity analysis and optimization, and contingency processes that mitigate consequences such as emergency response planning and warning systems. It should be emphasized that the common risk management process in geotechnical engineering applied to assure the performance of major structures relies on the observational method that includes a high degree of intervention and potential for enhancement. That is, it utilizes risk analysis but of the consequential kind.

However, while the observational method is powerful, it can be applied in an inappropriate manner and can be thwarted in practice. To assure reliable geotechnical performance, it should be bolstered by the systematic application of other tools of qualitative risk analyses as advocated by Morgenstern (1995). These include:

i) PPA (Potential Problem Analysis)
ii) HAZOP (Hazard and Operability Study)
iii) FMEA (Failure Modes and Effects Analysis)
iv) PHA (Preliminary Hazard Analysis)

Each has different protocols and Neowhouse (1993) provides a convenient overview.

Dushnisky and Vick (1996) have emphasized the value of FMEA to rank and prioritize risks from various sources and they provide examples of its application to mining projects with geoenvironmental consequences.

Judgment is essential to assure successful geotechnical performance. This has been emphasized by many commentators. The need to apply the observational method is also recognized, but its limitations are sometimes underestimated. This lecture advocates the systematic application of qualitative and consequential risk analysis to the design and control of geotechnical projects. Such application would provide structure to the judgment process, make it more transparent and facilitate risk management. This part of a project development requires the highest level of experience. It should be recognized as adding the highest value and rewarded accordingly.

Concluding Remarks

In 1980 Professor Lumb presented the Rupert H. Myers lecture at the University of New South Wales, Australia. This was a masterful review of thirty years of soil engineering in Hong Kong. He concluded with the observations:

......"Engineering judgment dominates design in soil engineering, but it is tremendously difficult to transfer a sense of judgment from the experienced to the inexperienced".

He went on to say:

......"In the soil engineering world it is all too easy to spend time on calculating what can be calculated rather than on what should be calculated, to giving an over precise answer to the wrong question.......".
At about the same time Peck (1980) concluded with regard to improved dam safety and geotechnical practice in general:

......"It depends on our ability to bring the best engineering judgment to bear on problems that are essentially non-quantitative, having solutions that are non-numerical. To develop this judgment and to bring it to bear require a reordering of our present views of what constitutes the highest form of engineering. Without detracting from the necessity for reasonable and meaningful engineering calculations and from the rewards to those who can carry them out, at least equal professional prestige and responsibility should be accorded men of judgment, even when that judgment is not expressed in numerical form.....".

This lecture has highlighted the limitation of quantitative prediction in geotechnical practice and, by a series of examples, illustrated the diversity of factors that can result in unsuccessful geotechnical performance. Examples are also given of the need for a broadly integrated approach to assure successful performance on major projects.

This lecture builds on the advice of Lumb and Peck to emphasize the role of judgment. However it goes further in advocating that the application of judgment should be structured within a framework of qualitative and consequential risk analysis, without precluding any other quantitative studies currently used in practice. Qualitative and consequential risk analysis should make use of tools such as Potential Problem Analysis, Failure Modes and Effective Analysis and the consistent application of the observational method. Their use should be introduced into graduate instructional programmes and they should be applied on all, but the simplest, geotechnical assignments.

The assurance of geotechnical performance would be enhanced if geotechnical engineering shifted from the promise of certainty to the analysis of uncertainty.

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A Vote of Thanks

Ir Y C Chan, Assistant Director (Geotechnical-Development), Civil Engineering Department

In this stimulating and informative lecture, Professor Morgenstern shared with us his wealth of case histories. The rich details and the clear conception of causation annotate his deep involvement in them. That he was invited to look into the many cases of under-performance and was able to come to the insightful findings tell us of his skill and the respect he enjoys for this.

The bulk of the lecture is on case histories, and they illustrate the various forms of uncertainty in geotechnical engineering well. However, Professor Morgenstern reminded us that we must not lose sight of the role of judgement.

Judgement is the subconscious balancing of limited knowledge leading to a choice of a course of action. Important to this process are insights and perceptions that are usually the result of one's learning from his past action. They are not proven, but could be surprisingly useful within the constraints of his past experience, particularly if he has been through lots of events, and with an active mind to benefit from them. They are by nature intimate to the person's past, and hence Professor Lumb's observation of the difficulties of transferring a sense of judgement from the experienced to the inexperienced.

A decision may be by judgement alone. In engineering, a combination of analysis and judgement is more likely. In general, the heavier a decision depends on judgement, the more difficult it is for another party to assess and appreciate the reliability of the decision, apart from references to the track record of the person who judged. This accounts at least in part for the reluctance of some stakeholders to accept judgement in the decision process. It also adds to the difficulties of transferring a sense of judgement.

There has been much effort to improve the transparency and consistency of judgement. Professor Morgenstern described PPA, HAZOP, FMEA and PHA. They are tools that regulate application of judgement. Quantitative Risk Assessment (QRA) is for calculating risk. Professor Morgenstern has shown us in many occasions that the value of this technique lies substantially in its providing a framework for examining engineering processes. It helps the articulation of processes into components of a wide range of suitability to scientific analysis, so that our effort on analysis could be kept in context, and judgement be made where necessary.

In today's lecture, Professor Morgenstern introduces to us the method of Consequential Risk Analysis, a framework he formulated for applying judgement. Implicit in the framework is the presumption that under-performance of a geotechnical design could occur, and focuses on contingency planning including detection and remediation of problems. It is particularly useful to cases where avoidance of under-performance is not affordable. It will be a powerful tool in the hands of the experienced, and a useful framework for the less experienced to build up his sense of judgement. For that, and for the wonderful case histories, we must thank Professor Morgenstern again for the excellent lecture.