Landslide Mitigation Using Granular Shear Keys:
Observations from a Review of 38 Case Studies

by

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ABSTRACT

Canadian transportation industries have been using granular shear keys to remediate shallow slope instabilities for decades. Engineers have used limit equilibrium (LE) analyses as the standard method for designing these granular shear keys (Wyllie & Mah, 2004). These LE analyses do not account for the strain required to mobilize the shear resistance in the granular backfill. Finite element methods can model deformation but require calibration and may not always be practical for many applications. Recommendations featuring plots and/or nomograms based on a review of the design and resulting performance of an extensive collection of case histories could supplement current design methods and enhance design effectiveness and reliability.

This thesis presents a compilation of 38 case studies for trenched granular shear keys or rockfill column shear keys. The purpose of this compilation was to identify empirical relationships between shear key design and deformation, to enhance the economy and performance of future granular shear keys.

Existing granular shear key design guidelines were reviewed and compiled. Case studies featuring granular shear keys were then collected. Details from these case studies were summarized and data was compiled into a database. The data was analyzed and compared with recommendations from the existing guidelines. Additional analyses using deformation data yielded predictions for the magnitude and timespan of post-construction deformation.

ACKNOWLEDGEMENTS

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1 INTRODUCTION

Major transportation corridors in the form of highways and railroads typically run thousands of kilometers, resulting in high exposure to landslides. In Canada, the road network comprises over 1.4 million kilometers of roads and boasts one of the highest kilometers per capita in the world (Statistics Canada, 2006). Meanwhile, Canada's railway network is responsible for the annual delivery of over 70% of all intercity surface goods moved in Canada and half of Canada's exports (Railway Association of Canada, 2011). Altogether, the transportation industry in Canada contributed 6.3% to the GDP in 2000 (Statistics Canada, 2006), demonstrating the importance of the road and railway networks to the Canadian economy. These networks can be damaged or closed in response to the thousands of landslides that occur every year in Canada. Estimates for the annual costs associated with these landslides are between \$200 and \$400 million (Natural Resources Canada, 2017). For this reason, the active management and remediation of landslides is of great importance in maintaining Canada's transportation network.

Landslide mitigation techniques are employed when the need for repairs becomes too frequent due to high rates of slope deformation. These techniques can be used to slow down movement or stop the landslide altogether (Cornforth, 2005). One such technique is the granular shear key.

A granular shear key is a deep trench that is excavated below the slip surface of a landslide before being backfilled with granular material. This granular material possesses greater internal shear strength than the native soils it is replacing (Cornforth, 2005). The shear key is intended to increase shearing resistance along the slip surface by either having it pass through the stronger backfill material or forcing it to follow a deeper, longer path through more competent materials (Abramson, Lee, Sharma, & Boyce, 2002).

A wide range of variables must be considered in the design of granular shear keys. These include deciding how deep to excavate below the slip surface, the height and width of the shear key, and the material with which to backfill the shear key. Some site conditions that must be considered are the position of the groundwater table, the landslide mechanism, and the root cause(s) of instability.

Optimizing the design of the shear key dimensions and volume are essential to minimizing costs. Overdesign can result in unnecessarily high project costs and increased risks associated with larger, open excavations. Underdesign can result in inadequate performance, which can translate to long-term maintenance costs and even additional remedial work being required.

The design of granular shear keys is most often undertaken using limit equilibrium (LE) analyses (Wyllie & Mah, 2004). A shear key is inserted into a base model that has been calibrated to current site conditions, and its dimensions are modified until a stability target is achieved (Denning, 1994). There are some limitations with this methodology. The mobilization of shear resistance in response to deformation is not captured by LE methods. The design process also typically involves several degrees of conservatism. These include conservative estimates for the friction angles for the sheared materials or the granular backfill, or ignoring the benefits of improved drainage from introducing a shear key to a slope. Between the limitations in the design methods and these conservatisms, there may be considerable potential for enhancing the design of granular shear keys.

The performance and cost effectiveness of granular shear keys can be enhanced by adopting a design methodology that considers the amount of deformation that may occur. The relative performance of existing granular shear keys can be evaluated using slope deformation data from before and after construction. This data can be used to identify relationships which could be used to supplement the current design practice by providing a means by which to quickly estimate deformation without the use of more advanced techniques and potentially with limited inputs.

1.1 Purpose and Objectives

The purpose of this research was to develop design recommendations for the mitigation of landslides using granular shear keys, based on a review of multiple case studies. Design recommendations and plots developed through this research are intended to supplement the current design state-of-practice. Plots can serve as a tool for quickly estimating the required dimensions of a granular shear key and predicting the magnitude and timeframe of slope movements after construction. Design plots could also be used to obtain a starting point for more detailed designs and serve as a form of redundancy for comparison with preliminary designs. Used in these capacities, the cost effectiveness and performance of granular shear keys could be enhanced, benefiting the safety and serviceability of transportation routes.

The following objectives were established to satisfy the purpose of this research:

i) Review the existing literature on granular shear key design theory.

ii) Compile a large database of granular shear key case studies from which to draw design specifications and performance indicators for subsequent analyses.

iii) Compare granular shear key design theory from literature to case study designs.

iv) Identify relations between the design and performance of granular shear keys using predictions derived from soil mechanics theory.

v) Propose design recommendations that are supported by design plots and case study examples, which enhance existing design practice.

1.2 Report Scope and Outline

The literature on granular shear keys was reviewed for gaps, to identify a focus for this work. Several of the publications that were consulted contained high level design steps for granular shear keys. Some publications also contained construction details and logistics. Others yet focused on specific aspects of granular shear keys. For example, Abdul Razaq (2007) produced suggestions for the construction of rockfill column shear keys. Goughnour et al. (1991) present typical design parameter ranges and methods for calculating them. Yarechewski and Tallin (2003) compared the performance of a trenched granular shear key to a rockfill column shear key. A single extensive collection of granular shear key case studies did not appear to have been studied before though. The scope of this research was to conduct a review of the designs, site conditions, and associated performances of documented case histories. The case studies that were considered all feature a trenched granular shear key or a rockfill column shear key. Case studies involving bonding agents, such as lime or cement, were generally excluded.

The following is an outline for the contents of this thesis. In Chapter 2, an introduction to granular shear keys is given. It touches on their applications, other techniques commonly associated with their use, and the most common settings in which they can be found. This introduction is followed by a discussion of the mechanics of granular shear keys. The subjects that are discussed include the mobilization of shear resistance, shear key modes of failure, and how the shear strength is modelled. Last, the concept of using rockfill columns as a shear key is discussed.

Chapter 3 explores the existing granular shear key design guidelines and concepts suggested by literature. This chapter is structured to roughly follow the same steps taken in current practice, from modelling to post-construction monitoring.

Chapter 4 focuses on the case studies that were collected. The construction of a case study database is briefly discussed. The statistics from the database are presented for comparison with the typical ranges summarized in Chapter 3. The methodology that was adopted for evaluating performance and other aspects of design is also presented. The chapter then goes through the analyses performed on the database, with results compared against predictions from Chapters 2 and 3.

In Chapter 5, the recommendations derived from Chapter 4 are listed in the context of the principal phases of a remediation project. Additional considerations are also presented, touching on some of the complexities encountered in the case studies. This is followed by suggestions for future research.

Lastly, Appendix A contains summaries for each of the case studies that were collected. Each summary is structured to include a brief background on the site, details of the remedial work that was performed, information on the performance of that remedial work, and some lessons that may be learned from studying that case study. A summary of statistics and specifications for each site is also provided, with details such as landslide and granular shear key dimensions. The statistics are then followed by supplementary information in the form of borehole logs, site plans and cross sections, where they were available.

2 LITERATURE REVIEW

The concept of using granular shear keys to stabilize landslides has been practiced for decades. This chapter explains what granular shear keys are and how they function using soil mechanics theory. First, the fundamental concept behind their use is introduced along with explanations for the various dimensions which are used to describe them. The most common uses and settings for granular shear keys are also given. Next, the stabilization techniques most commonly used together with granular shear keys are discussed. The discussion then moves to the most common landslide settings where granular shear keys tend to be adopted. The site conditions are described in terms of the landslide characteristics, mechanisms and behaviours.

Next, the behaviour of granular shear keys and how it is modelled are examined. The mechanism behind their performance is reviewed, with the principal modes of failure presented first. The fundamentals of modelling shear strength are discussed next. This discussion touches on how it is assessed and modelled in current practice, and whether this practice is adequate. The discussion concludes with a brief introduction to an alternative constitutive model that may be a better choice for modelling the shear strength of granular materials.

The final section of this chapter delves into the concept of using rockfill columns as an alternative to the traditional trenched granular shear key. The mechanism, modes of failure, and landslide conditions for rockfill columns are compared and contrasted with those of trenched shear keys.

2.1 An Introduction to Granular Shear Keys for Landslide Stabilization

Shear keys, sometimes referred to as shear trenches (Ho, 2004), are a slope stabilization technique wherein a trench is excavated below the slip surface, then backfilled with a material possessing greater shear strength than the native material (Abramson, Lee, Sharma, & Boyce, 2002; Cornforth, 2005). Shear keys are constructed perpendicular to the principal direction of movement. A schematic example of a shear key is shown in Figure 2.1 along with a summary of the dimensions relevant to their design, in Table 2.1.

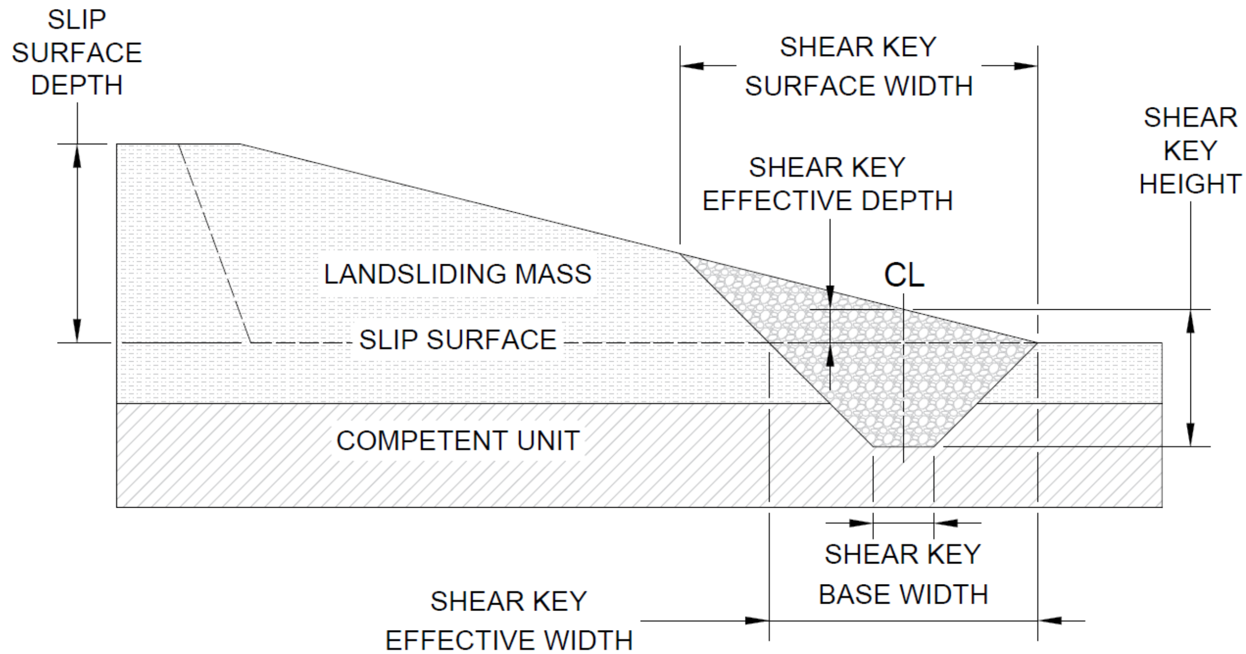


Figure 2.1: Schematic of an idealized shear key and landslide geometry indicating several key design dimensions.

Shear keys are considered a replacement technique. A variety of replacement techniques have been used for slope stabilization from as early as the 1830s, where the use of counterforts backfilled with stone was documented for the repair of the Bourgogne Canal in France (Collin, 1846). These stone-filled counterforts were excavated below the slip surface of the sliding mass and were also understood to act as French drains. These two ideas, of excavating then backfilling across the slip surface, and promoting drainage, are still the fundamental principals behind the design of modern shear keys.

Granular shear keys are commonly used at outfall locations, in bridge abutments and overpasses, and to promote the stability of dikes (Kenyon, 2016). They are often adopted in lieu of toe berms when space is limited, provided there is a strong material into which to excavate. They can also be added as a component of other earthworks, to lengthen the slip surface (Proudfoot D. , 2016). They are well-suited to sites where water levels are prone to changing. This is because the performance of granular backfill is materially unaffected by changes in river levels or rainfall because of its free-draining nature (Sills & Fleming, 1992). The construction of these types of structures is also relatively quick and possesses a small footprint. Thus, these methods are favourable for emergency repairs where the right-of-way is limited (Sills & Fleming, 1992).

Table 2.1: List of the dimensions used to describe the landslide and granular shear key geometries for this research, including the symbols that were used and definitions for each dimension.

Dimension	Symbol	Definition
Shear key surface width	b_s	The horizontal width of the shear key in cross section measured at the ground surface.
Shear key base width	b_b	The horizontal width of the shear key in cross section measured at the base of the excavation.
Shear key effective width	b_e	The horizontal width of the shear key in cross section measured at the elevation of the slip surface where it intersects the upslope side of the shear key.
Shear key height	H	The vertical distance between the surface and the base of the excavation, measured from the midpoint of the base of the excavation in cross section (CL).
Slip surface depth	D	The vertical distance between the ground surface and the base of sliding. Unless otherwise specified, this dimension is assumed to be measured from the slide scarp.
Shear key effective depth	d_e	The vertical distance between the ground surface and the slip surface, measured from the midpoint of the base of the excavation in cross section (CL).
Key-in depth	d_k	In the presence of a competent unit, the vertical distance from the top of said unit to the base of the excavation. Where no such unit exists, the vertical distance from the slip surface to the base of the excavation. Measured from the midpoint of the base of the excavation in cross section (CL).

2.1.1 Stabilization techniques commonly associated with granular shear keys

Granular shear keys can be combined with other measures to increase the overall improvement to stability. Cornforth (2005) remarks that in current practice, shear keys are typically used in conjunction with a buttress constructed of the same material. These structures are sometimes referred to as weighting berms, or counterbalance fills (Denning, 1994). In addition to buttressing the sliding mass, this structure has the effect of adding weight to the shear key to increase shear resistance (Cornforth, 2005). The granular backfill in shear keys is non-cohesive and its strength is derived from friction, which is proportional to the weight that is applied to it (Baker & Yoder, 1958). In addition to enhancing stability, buttress fills can be constructed from the same granular material being used in the shear key, or from the compacted material coming from the trench or nearby (Caltrans, 2014; Denning, 1994). This latter option can help to eliminate hauling costs. Caltrans (2014) reports these berms are often constructed flatter than the surrounding slope but should nevertheless be checked for stability. Another consideration is that settlement may increase in response to the additional loading from the construction of a berm (Caltrans, 2014). Barksdale and Bachus (1983) report that rockfill columns can also benefit from the addition of a weighting berm, and that there is an additional benefit from the development of stress concentration in the columns.

Granular shear keys can be constructed with or without the use of geotextiles (Ho, 2004). Cornforth (2005) recommends using geotextile along all rockfill/soil interfaces. This recommendation comes from the fact that geotextile tends to be relatively inexpensive, especially for the potential benefits it may provide. These benefits can include limiting the infiltration of fines, and increasing shear strength. However, the benefits or impacts of using geotextiles in granular shear keys were not directly investigated in this thesis.

Other slope stabilization techniques that are sometimes associated with shear keys include rockfill columns (Fiebelkorn, 2015), and rockfill trenches or ribs (Sills & Fleming, 1992). Rockfill columns, also known as stone columns, were included in this research for their appreciable similarities to trenched granular shear keys when installed across the width of a slide specifically for the purpose of resisting lateral loads. Rockfill ribs, constructed as panels aligned parallel to the direction of sliding, were not included since very few cases involving this technique were obtained.

2.1.2 Landslide conditions commonly addressed using granular shear keys

The applicability of granular shear keys can also be discussed in the context of the types of landslides most commonly addressed using this technique. An understanding of the mechanics and behaviour of those landslides, including the triggers that initiated movement, is central to successfully mitigating sliding.

Landslide characteristics

Several publications suggest the ideal application of granular shear keys is for the mitigation of shallow, planar landslides (Denning, 1994; Cornforth, 2005). The suitability of granular shear keys for addressing different landslide characteristics was reviewed to support and build on this recommendation.

The basic idea for granular shear keys is to replace weak, native soil with stronger granular backfill to increase the shear resistance within a landslide. This reliance on the granular material being stronger than the native soil imposes a limitation on the applicability of granular shear keys.

Intuitively, rocky or granular slopes would be similar in composition to the backfill, thereby limiting the benefit that could be had from a granular shear key. Thus, granular shear keys would be constrained to slopes composed of fine-grained, cohesive materials.

The velocities of landslides are grouped into seven categories by Cruden and Varnes (1996) (see Table 2.2). Landslides in fine-grained soils are usually classified as *slow*, meaning there is less than 13 m of deformation per month (432 mm/day). This is particularly typical of stiff clays and clay shales, where sliding is governed by the residual strength of the slip surface. Per Picarelli and Russo (2004), slow active slides tend to be either planar (Figure 2.2) or compound (Figure 2.3).

Table 2.2: Landslide velocity classes proposed by Cruden and Varnes (1996). Upper and lower velocity thresholds are presented for each class.

		Velocity	
Velocity class	Description	mm/sec	Typical units
7	Extremely rapid		
		5×10^{3}	5 m/sec
6	Very rapid		
		50	3 m/min
5	Rapid		
		0.5	1.8 m/hour
4	Moderate		
		5×10^{-3}	13 m/month
3	Slow		
		5×10^{-5}	1.6 m/year
2	Very slow		
		5×10^{-7}	16 mm/year
1	Extremely slow		

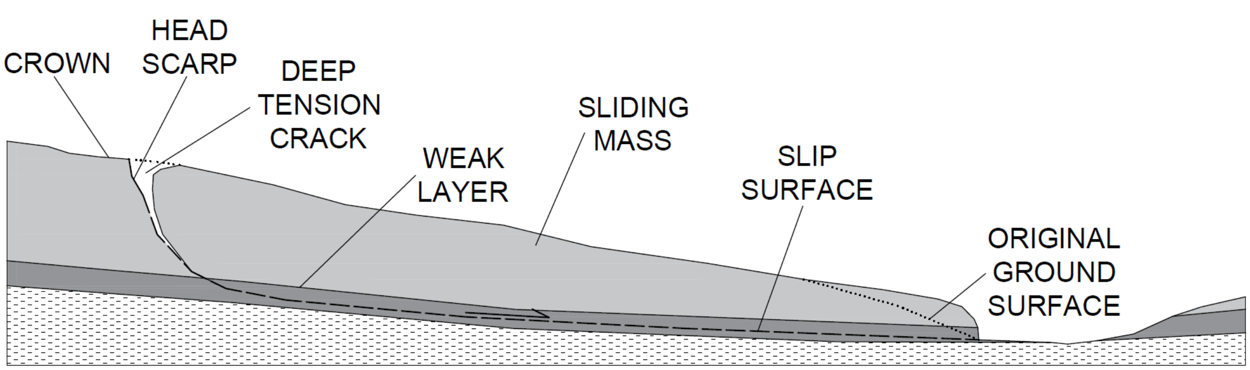


Figure 2.2: Schematic of a planar slide per the definition given by Hungr et al. (2014).

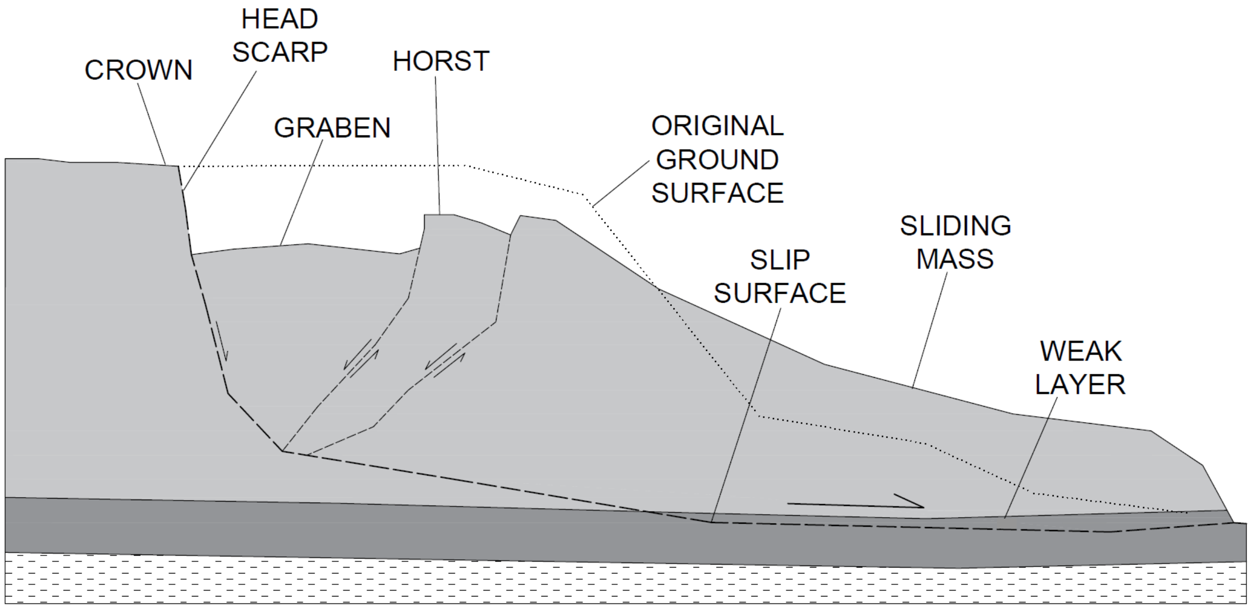


Figure 2.3: Schematic of a compound slide per the definition given by Hungr et al. (2014).

The size of a granular shear key affects the costs and safety of the remedial work. As the depth of the excavation increases, the volume becomes increasingly large. The excavated material must be hauled and backfill must be acquired to fill the excavation. There are costs associated with hauling, and with obtaining the raw material used as backfill. Thus, there is an economic limitation on the depth of sliding that can be practically addressed with a granular shear key. A larger excavation will also have a greater destabilizing effect on the slope. The risk associated with the excavation thereby increases. For this reason, shear keys are best suited for the remediation of shallow slides. It should, however, be noted that a study found the unit repair cost decreased as landslide volume increased, for volumes up to approx. 15 300 m^3 (Deschamps & Lange, 1999).

In considering the soil types, landslide velocity and type, and the depth of sliding most suitable for granular shear keys, it can be concluded that the ideal application of this landslide mitigation technique is for slow-moving, shallow translational slides in fine-grained soils.

Landslide mechanics and behaviour

Identifying the root cause of the instability is also critical to the successful mitigation of slope instabilities. Landslides can be triggered by many different mechanisms, such as the addition of a surcharge, or changes to the groundwater system. Abdul Razaq (2007) produced a flowchart outlining many of the potential landslide triggers, after the Highway Research Board's version from 1978. Abdul Razaq's flowchart broadly categorizes landslide triggers by whether they result in shear strength decreasing, or shear stress increasing. Cruden and VanDine (2013) listed four broad categories of landslide causes: ground conditions, geomorphological processes, physical processes, and artificial processes. Rogers (1992) highlighted three fundamental factors to consider in addressing slope instabilities: i) the relative position of the groundwater table, ii) the fluctuation of groundwater levels and flow volumes, and iii) the identification, confirmation and characterization of ancient and active landslide slip surfaces, including their geometric extent. Lastly, Cornforth (2005) remarked rainfall intensity is strongly linked to the occurrence of landslides, as opposed to the cumulative amount of rainfall.

In the previous section, it was concluded that granular shear keys are best suited for the mitigation of slow-moving, shallow translational slides in fine-grained cohesive soils. For these slides, the driving force tends to be constant, with movement triggered by changes in the resistance, or by viscous deformations. The resistance can change in two ways; primarily, seasonal pore pressure fluctuations, and additionally, by erosion and soil weakening due to weathering or additional deformations (progressive failure – see Figure 2.4). The slipping mode for these landslides is described as viscous-plastic, with variable slip rates especially if the slide is shallow (Picarelli & Russo, 2004). It was found by Campbell et al. (1995) that under the same conditions, slides of different sizes traveled with the same average velocity but the shear rates and frictional resistance varied. The shear rates and shear resistance for thicker landslides were smaller than those for thinner, smaller slides (Campbell, Cleary, & Hopkins, 1995). Campbell et al. (1995) also notes from simulations that the bulk of a landslide does not travel as a solid block as is assumed by convention, but experiences shear throughout. A study of the mechanisms of shear for clay soils found that clay soils with a clay fraction less than 20-25% exhibited turbulent (i.e. non-laminar) shear, and those with a clay fraction exceeding 50% exhibited sliding shear. Those soils that did not exclusively exhibit either mechanism were classified as transitional (Cornforth & Fujitani, 1991).

Other studies have shown that a non-linear relationship exists between the rate of displacement and increases in pore pressure. It has also been found that at the same pore pressure, the velocity of an identical slide differs depending on whether that sliding resulted from either a rise or a decrease in the groundwater table. Picarelli and Russo (2004) stated that the reactivation and halting of a slide can occur at different pore pressures, indicating the threshold for movement is not defined by a single static value of pore pressure. They found movement was initiated at a mobilized shear strength equal to approximately 95% of the soil's residual shear strength. Furthermore, each reactivation of the same slide required greater pore pressure than the previous reactivation for a case study on the Worbarrow Bay mudslide in Dorset (Picarelli & Russo, 2004).

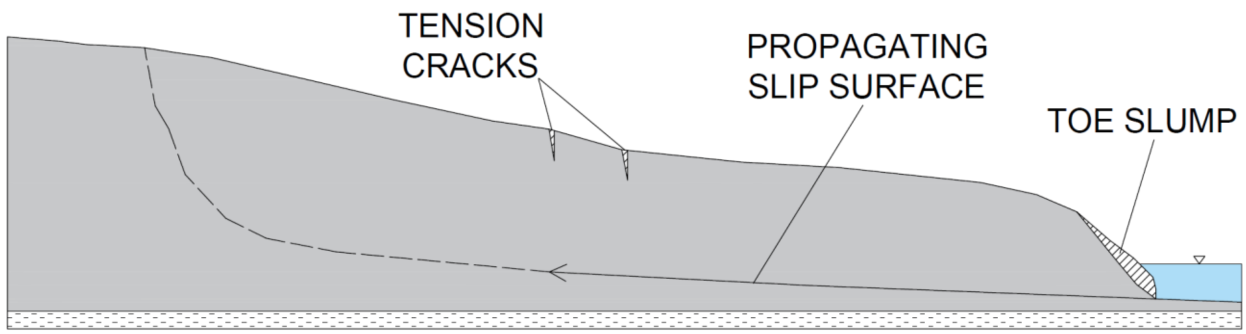


Figure 2.4: Schematic showing the propagation of a slip surface and the appearance of tension cracks as a hypothetical slope undergoes progressive failure, and toe erosion results in a toe slump.

The landslide characteristics and behaviours presented are diverse and there has clearly been extensive research on these subjects. The diversity of these conditions presents a challenge to adopting a generalized approach to design. These conditions no doubt also have an influence on the suitability of granular shear keys and long-term performance. A more complete understanding of their influence on the slide system can be had from focusing on the shear key itself.

2.2 Understanding and Modelling Granular Shear Key Behaviour

A granular shear key is expected to promote stability by increasing the shear strength along the slip surface. How that shear strength is described has been the subject of considerable study, resulting in the development of many different constitutive models. A design engineer must select an appropriate model for the shear strength of the soil and backfill, to execute the general design procedures that are later presented, in Chapter 3. This includes selecting appropriate input values for the model (or hand calculations). To gauge what constitutes an appropriate model, the behaviour of these materials must first be understood.

2.2.1 Granular shear key modes of failure

It was stated previously that granular shear keys promote stability by increasing the shear strength along potential slip surfaces. This is achieved by either forcing the potential slip surface deeper, thereby making it longer, or through the stronger material of the shear key itself (Abramson et al., 2002; Caltrans, 2014). The modes of failure exhibited by shear keys can be understood by studying rockfill buttresses. Rockfill buttress are highly similar except for the fact that the downslope face is not confined by native soil. There are three principal modes of shearing failure observed in rockfill buttresses: shearing failure across the buttress, through the native soil below the buttress, or along the interface between the buttress and the native soil (Baker & Yoder, 1958). These modes of shearing failure are illustrated schematically in the context of a granular shear key in Figure 2.5. In the presence of a competent unit into which the granular shear key can be excavated, Modes 2 and 3 would not typically be exhibited.

In the first failure mode, the location of the slip surface remains unchanged. This mode of failure requires that the shear force exerted by the sliding mass exceed the shear resistance of the granular backfill. The second and third modes of failure involve shearing through the native soil, either along the interface with the granular backfill, or at depth. The new slip surface will have a greater length and thus must overcome greater shear resistance than along the original path. The shear strength of the native soil may also be greater with depth, which makes shear keys a particularly effective solution if stronger material is close to the existing slip surface (Abramson et al., 2002).

In evaluating the stability of the granular shear key, there is a choice to carry out the analysis under drained or undrained conditions. The long-term stability of granular shear keys is generally checked with drained conditions, using effective stress analyses. The excavation of the trench during construction can be considered short-term and may be subject to an undrained or total stress analysis. However, the actual site conditions should always be considered before choosing an appropriate method of analysis (Canadian Geotechnical Society, 2006).

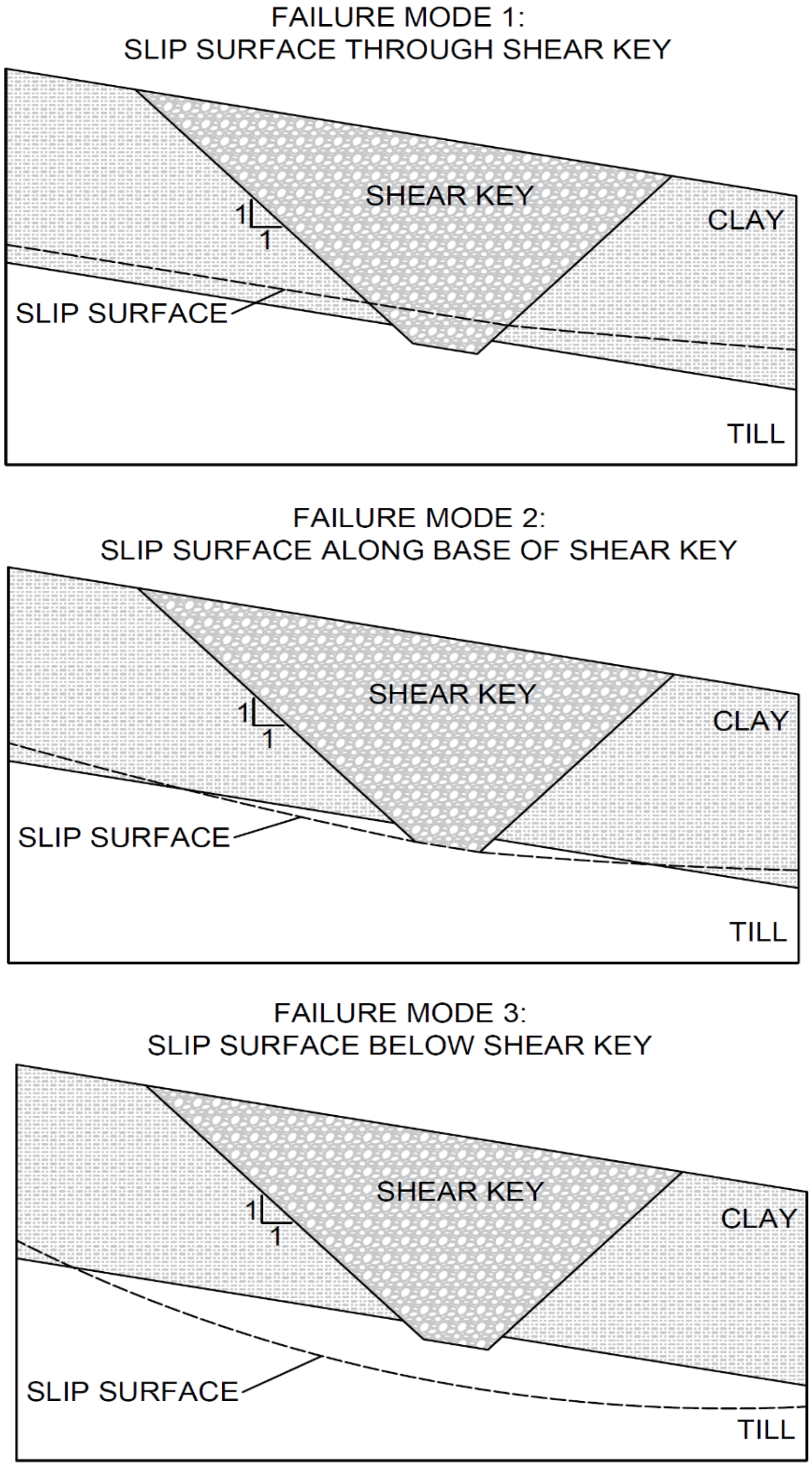


Figure 2.5: Schematic illustrations of the three modes of shearing failure which can occur in response to the introduction of a granular shear key.

2.2.2 Appropriateness of assuming a direct shear mechanism

In each of the three modes of failure presented in the preceding section, is it assumed the materials undergo direct shear. No instances were encountered throughout the undertaking of this thesis where a granular shear key had been excavated after construction to examine the shape or position of the slip surface. In a select few cases, slope inclinometers were installed between rockfill columns and, in one case, a slope inclinometer was installed within a trenched shear key. In these cases, displacement profiles which had exhibited relatively discrete zones of movement before construction became smoother once a shear key was introduced. It is difficult to reach a conclusion as to whether the slip surfaces were passing through the full width of the shear keys with the data that was available. However, the shape of the slip surface can be estimated and the appropriateness of assuming a direct shear mechanism can be evaluated by applying Rankine earth pressure theory.

By taking the granular shear key to be in a passive state, the shape of the slip surface can be estimated as following a curved path similar to what is portrayed in Figure 2.6. The slip surface exhibits a certain degree of curvature which is controlled by the friction angle of the material through which it passes. The horizontal distance over which this curvature would be exhibited before causing the slip surface to intersect with the ground surface, $2x$, can be estimated using the friction angle of the granular backfill, ϕ, and the depth, h, of the midpoint of the curved slip surface, as labelled in Figure 2.6. Statistics for the dimensions of the granular shear keys featured in the case studies that were collected will be presented in Chapter 4. However, a typical depth would be 5 m and the corresponding width would be 7 m. Assuming a value of 45º for ϕ and h equal to 5 m, the horizontal distance over which this curvature would be exhibited would be equal to 24.2 m. This greatly exceeds the granular shear key width of 7 m. This calculation shows that the slip surface would not be able to deflect very much while passing through the shear key, instead taking a path that could be approximated by a flat line. Thus, it would be appropriate to assume the granular material undergoes direct shear.

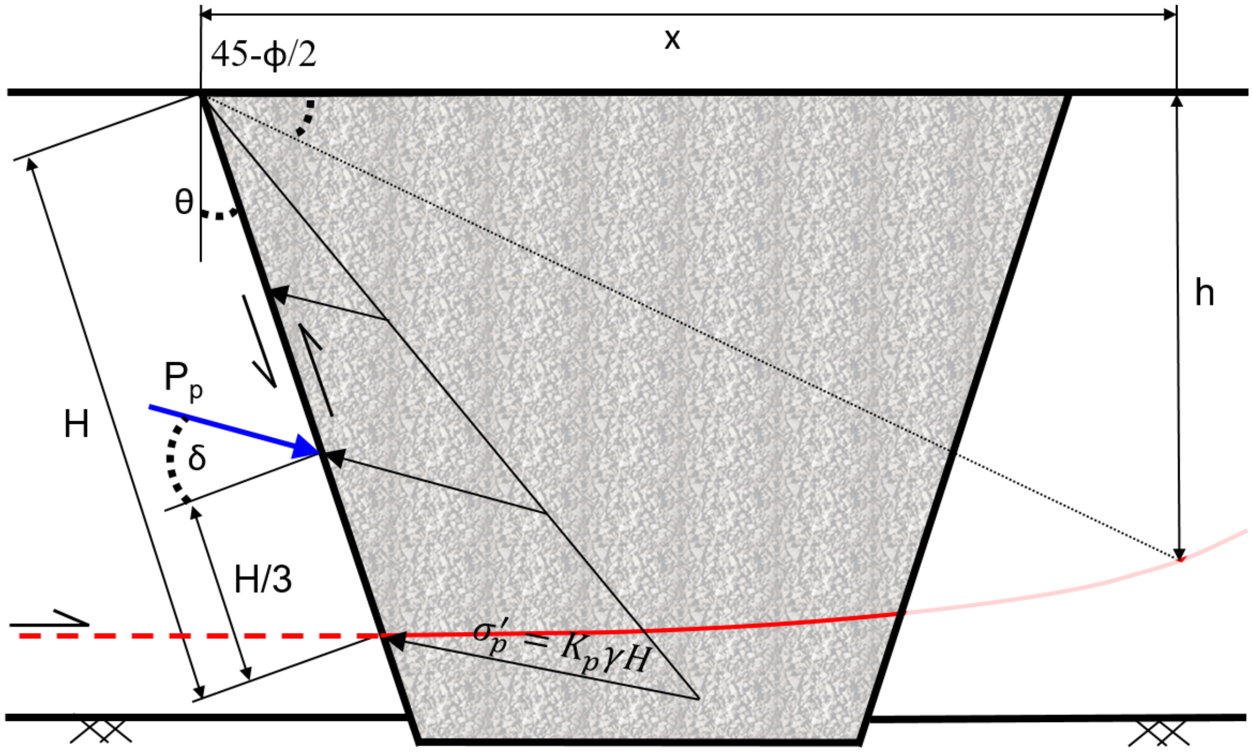


Figure 2.6: Schematic of a granular shear key in cross section with a slip surface (in red) portrayed in accordance with Rankine earth pressure theory. The shear key is in a passive state, with the shape of the slip surface through the granular backfill estimated using its friction angle.

2.2.3 The shear strength of granular backfill

Granular shear keys as a passive method

Granular shear keys are a passive stabilization method, wherein the shear resistance is mobilized in response to deformation caused by an external force. The source of the external force is the landsliding soil mass, which moves in the direction of the shear key. The strain required to mobilize passive earth pressure is large relative to active earth pressure. This can be seen in Figure 2.7. The granular shear key will continue deforming until the magnitude of passive shear resistance is equal to the driving shear stress.

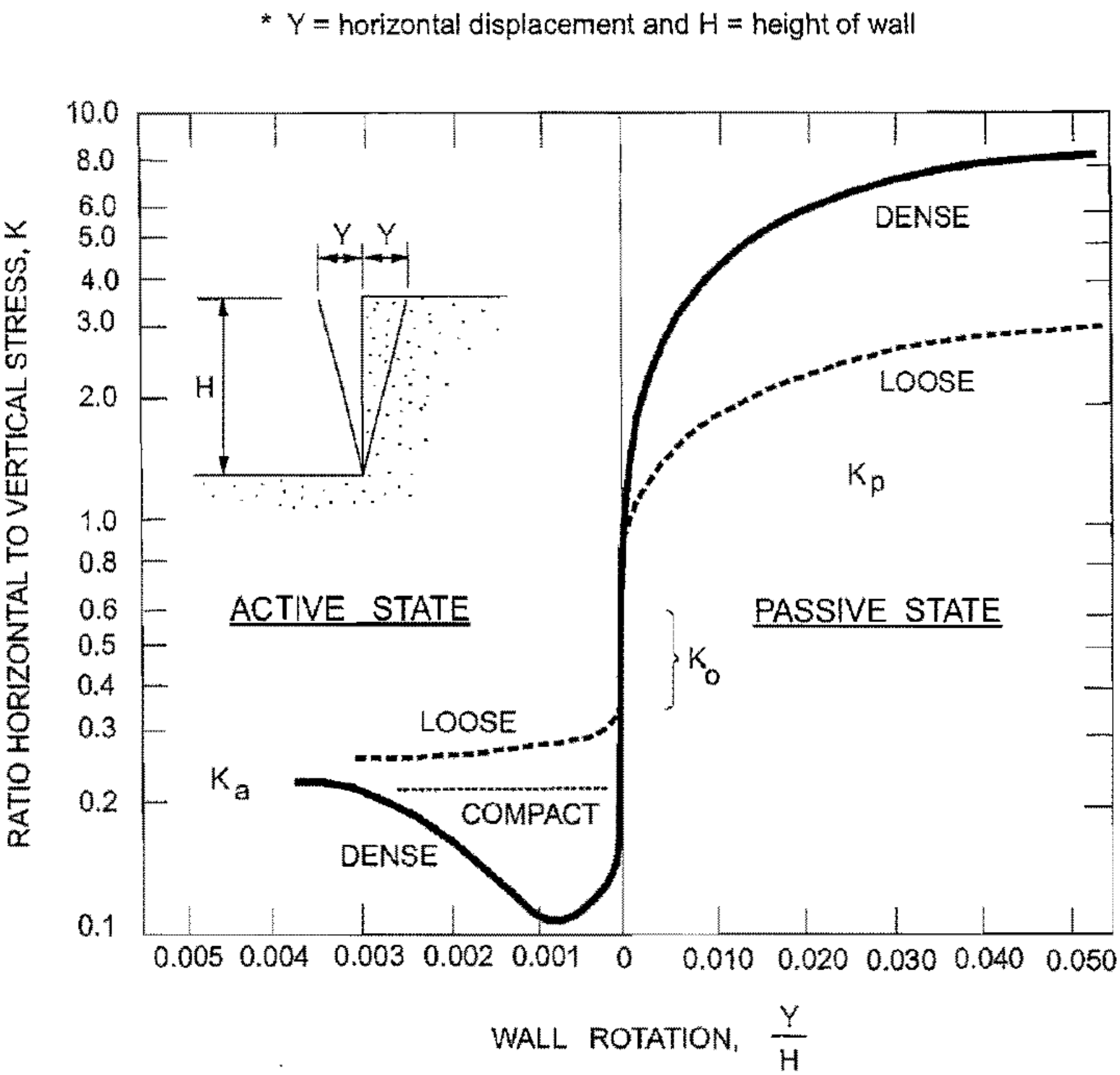


Figure 2.7: Schematic illustration of the mobilization of active and passive earth pressures in cohesionless materials in response to deformation. Figure reproduced from the Canadian Foundation Engineering Manual 4th ed. (2006), with permission from the Canadian Geotechnical Society.

The finite element method can be used to illustrate the deformations that will be experienced by the granular shear key in response to landsliding. Figure 2.8 shows shear strain contours for an idealized slope wherein a granular shear key has been placed near the toe. The deformed boundaries are shown, revealing how the granular shear key might deform in response to loading.

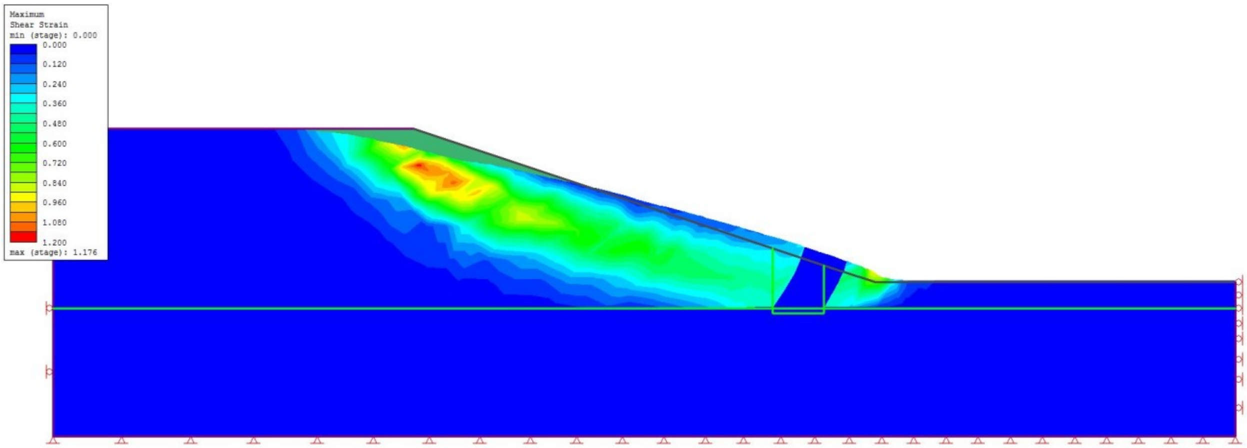


Figure 2.8: Shear strain contours plotted over an idealized slope with a granular shear key near the toe. Deformed boundaries have been enabled, showing how the granular shear key geometry may change in response to loading.

Mohr-Coulomb as the current standard of practice

The shear strength of granular soils is a function of interparticle friction, the rolling action of the particles resulting from the eccentricity of intragranular force, and the mean path of the particles (Marsal, Mechanical Properties of Rockfill, 1973). In modelling, an attempt is made to encompass these mechanisms using constitutive models. For granular shear keys, the Mohr-Coulomb (M-C) constitutive model for soil strength is most commonly used. Under this constitutive model, the shear resistance, τ, is represented as a linear function,

$$\tau = \sigma'_{\mathrm{n}} \tan \varphi' + c' \tag{2.1}$$

where φ' is the effective internal angle of friction, σ'_{n} is the normal effective stress at the depth where the shear failure occurs, and c' is the effective cohesion. Clean granular backfill is usually assumed to be cohesionless, as the particle size is sufficiently large that the role of colloidal forces is insignificant (Marsal, Mechanical Properties of Rockfill, 1973).

For this approach, the peak mobilized friction angle of the backfill is estimated from direct shear testing under the anticipated normal load conditions. The friction angle is then typically factored for modelling, introducing a degree of conservatism to the design process.

Despite its frequent adoption, some limitations have been identified with the M-C model. Marsal (1973) states that the Mohr criterion assumes failure develops along the plane of maximum obliquity but that this assumption is only valid for triaxial tests. Soils were observed to fail along a 45° plane during plane strain testing, with failure being controlled by the maximum shear stress and the mean normal stress. The behaviour of granular shear keys is likely better represented by plane strain conditions. Another issue with the use of the M-C model is that studies on the mechanical properties of rockfill have shown its shear strength is nonlinear (Marsal, Mechanical Properties of Rockfill, 1973). The friction angle that can be used to describe the shear strength of a granular material is therefore not a constant. The friction angle can be shown to vary significantly in response to several factors which are discussed in the following section.

2.2.4 Effect of grain size, density, and confining stress on friction angles

The friction angle of a granular material is affected by the grain size distribution (GSD), the density of the material, and the confining stress that is applied. Duncan et al. (2014) provide a formula developed from data for rockfills, gravels and sands, which relates the effective friction angle, φ', to each of these factors:

$$\varphi' = A + B(D_{\mathrm{R}}) - [C + D(D_{\mathrm{R}})] \log_{10}\left(\frac{\sigma'_{\mathrm{N}}}{P_{\mathrm{a}}}\right) \tag{2.2}$$

where A is a parameter which is used to reflect the type of material, B controls the amount by which φ' increases as the relative density, D_{R}, increases from 0 to 100%, C controls how the decrease in φ' from an increase in confinement, σ'_{N}, is greater for larger grain sizes, and D reflects the same minor effect that confinement has on all materials. P_{a} is the atmospheric pressure, which is used to normalize σ'_{N}.

The values for these parameters given by Duncan et al. are summarized in Table 2.3.

Table 2.3: Summary of the parameter values determined by Duncan et al. to relate the effective friction angle to the gradation, relative density, and confining stress for sands, gravels and rockfills.

	Parameter values (Degrees)				Std. dev. (Degrees)
	A	B	C	D	
Gravel and cobbles (C_{u} > 4)	44	10	7	2	3.1
Sand (C_{u} > 6)	39	10	3	2	3.2
Sand (C_{u} < 6)	34	10	3	2	3.2

Effect of grain size on friction angles

Nicks and Adams (2013) found from large-scale direct shear tests on open-graded aggregates that the friction angle decreased as the aggregate size decreased. This is consistent with Duncan et al. (2014), who found that friction angles for gravels and cobbles were larger than for sands. Bishop

(1966) showed that dilation was inhibited at lower confining stresses for a rockfill than for a sand though. He concluded that well-graded rockfills placed in a dense state would be the best option for settings in which the fill would be saturated and subjected to high stresses. This is consistent with Marsal (1967), who found from large-scale triaxial tests on rockfill materials of various origins that well-graded materials with low void ratios possessed the greatest shear strengths. Duncan et al. supported this by stating that well-graded soils had greater friction angles than uniformly graded soils. This was attributed to the fact that smaller particles would fill the voids and thereby increase the density of the packing. Marsal also observed appreciable differences in the shear strength measured for different rockfills with similar gradations. He postulated these differences were due to the intrinsic characteristics of the particles, namely the crushing strength of the particle contacts, P_{a}, and the particle breakage, B. Simoni and Houlsby (2006) investigated the potential impact of grain size distribution on shear strength for sand-gravel mixtures. They found that volume fractions of gravel as low as 0.1 caused an increase in the peak friction angle. The gravel caused greater dilatancy of the mixture at failure and resulted in a higher constant volume friction angle as well (Simoni & Houlsby, 2006).

Effect of density on friction angles

Several works have shown that an increase in the relative density, D_{R}, will increase the friction angle. For example, Marsal (1967) found that lower void ratios corresponded to higher shear strengths for rockfills. Simoni and Houlsby (2006) demonstrated higher peak friction angles for sand and sand-gravel mixtures could be achieved by increasing D_{R}. Duncan et al. (2014) stated that for an increase in D_{R} from 0 to 100%, the friction angle increased by approximately 10°. Abdul Razaq (2007) found that, for the same normal stress, a dense rockfill specimen (D_{R} > 90%) could exert almost twice the shear resistance of a loose specimen (D_{R} < 15%). The peak strength of these specimens was demonstrated to depend on both the initial void ratio and the effective normal stress that was applied. Having identified these factors in his work on sands, Bolton (1986) introduced the relative dilatancy index, I_{R}, which he defined in terms of the relative density and effective stress level. Bolton found there was not a one-to-one relationship between the friction angle and the density though. He attributed this to the fact that the friction angle has a component related to dilation, and another component related to the mineralogy of the grains. The dilation component of the friction angle causes the mobilization of shear resistance to be a function of the volume change from shearing. This volume change in turn depends on the initial density (Abdul Razaq, 2007). Bishop (1966) demonstrated this by plotting the rate of volume change against the confining stress for a dense and a loose sand. The dense sand exhibited much greater rates of volume change than the loose sand, especially at low confining stresses. Lastly, the Canadian Foundation Engineering Manual 4th Ed. (2006) states that the strain required for the full mobilization of passive resistance in sand depends on its density. Thus, the density has been shown to have a great influence on the friction angle.

The stiffness of the granular backfill can vary significantly depending on how uniformly it is compacted. Abdul Razaq (2007) found the shear stiffness of dense rockfill was about four times greater than the shear stiffness of loose samples. As the shear key deforms, a redistribution of pressure can occur along its upslope boundary since stiffer materials tend to take on greater stress in accordance with the equal strain assumption (Barksdale & Bachus, 1983). This process results in the development of soil arching (Canadian Geotechnical Society, 2006). The granular shear key is itself composed of soil, and so soil arching can develop along the downslope boundary as the backfill moves towards the native soil. It is possible this soil arching could promote the performance of granular shear keys beyond that which might be expected if left unaccounted.

Effect of confining stress on friction angles

Charles and Watts (1980) performed large-scale drained triaxial compression tests on heavily compacted samples of different types of rockfill over a range of confining pressures. The drained conditions are very likely to be representative of field conditions, since rockfill tends to be free-draining (Duncan et al., 2014). Charles and Watts found the Mohr failure envelopes showed pronounced curvature, particularly at low confining pressures. This is consistent with Nicks and Adams (2014), who state the friction angle and dilation angle decrease in a logarithmic function as the applied normal stress increases. Abdul Razaq (2007) also observed an increase in the dilatancy in rockfill samples as the normal applied stress decreased. Nicks and Adams found the secant friction angle for each applied normal stress was greater than the friction angle that was estimated from the line of best fit for the Mohr envelope. Charles and Watts thus concluded the current practice is often over-conservative because the Mohr strength parameters underestimate the shear strength.

The strain boundary conditions, or how the confining stress is applied to the granular material, will also influence strength (Duncan et al., 2014). Charles and Watts (1980) state that rockfill structures will in general approximate to plane strain conditions rather than triaxial conditions. The use of triaxial data is expected to be acceptable, as Marsal (1973) observed similar behaviour at failure between tests performed on rockfill subjected to both conditions. The friction angle under triaxial conditions is typically smaller than that exhibited under plane strain conditions. This was demonstrated by Bishop (1966), who went on to show that the difference is more pronounced for denser materials. Friction angles from triaxial tests were smaller than those obtained from plane strain tests by 1 to 6° in the dataset analyzed by Duncan et al. (2014). Differences of 3 to 6° were observed for the densest materials, which supports the finding by Bishop. Thus, there is an acceptable degree of conservatism in using strength parameters determined under triaxial conditions.

For drained triaxial tests on rockfill, the effective angle of internal friction can be described in terms of the principal stress ratio at failure, $\left(\frac{\sigma_1'}{\sigma_3'}\right)_{\mathrm{f}}$, as

$$\varphi' = \sin^{-1}\frac{\left(\frac{\sigma_1'}{\sigma_3'}\right)_{\mathrm{f}} - 1}{\left(\frac{\sigma_1'}{\sigma_3'}\right)_{\mathrm{f}} + 1} \tag{2.3}$$

The maximum size of grains that can be tested in the laboratory is often constrained by the equipment that is available. It is typical to overcome this limitation by adopting a grain size distribution (GSD) curve that plots parallel to the field GSD on a semi-logarithmic graph. This technique introduces other limitations though. The maximum grain size has been shown to have an influence on the angle of internal friction. However, this influence is relatively minor compared to confining pressure and initial porosity.

Charles and Watts (1980) tested four different rockfill materials. At low confining pressures (27 kN/m^2), tests on a sandstone showed marked dilation, whereas there was very little dilation at greater confining pressures (282-695 kN/m^2). This dilation results in a curved strength envelope. However, the axial strain at failure, $\varepsilon_{\mathrm{a-f}}$, increased from 2% at 27 kN/m^2 to 6% at 695 kN/m^2.

The angle of internal friction used for the design of large rockfill embankments (more than 100 m) per Charles and Watts (1980) often ranges from 37° to 45°. Nicks and Adams (2014) state that a default friction angle of 34° is typical of transportation agencies, regardless of the material. These values are very conservative for applications where the confining pressure would be relatively small, since the use of a constant value ignores the curvature of the strength envelope. The implication of this was demonstrated by Charles and Watts who showed that the factor of safety for a translational slide in an infinite slope increased dramatically as the depth of the slip surface decreased. To model this curvature, the shear strength, τ, can be described using a power curve:

$$\tau = a(\sigma_N')^b \tag{2.4}$$

where σ_N' is the normal effective stress, and a and b are power curve parameters that can be determined by regression.

The curvature in the strength envelope of granular materials was stated to be a result of the high rates of dilation at failure observed at low confining pressures. The reduction in the angle of internal friction and the corresponding reduction in the rate of dilatancy at high confining pressures was linked to particle breakage by Marsal (1973). As dilation is eliminated, the behaviour of the granular material at failure approaches the critical state, which can be represented by a constant angle of internal friction. Power curves can therefore only be used to describe shear strength over a limited range of normal stress. For the rockfills tested by Charles and Watts (1980), this range is 40-400 kPa. By performing a regression on test data obtained for a greater range of normal stresses, the b parameter will approach unity; this would represent an envelope that is a straight line. It therefore would be reasonable to suggest that the power curve parameters to be used for design should be obtained from tests performed over the stress range that is anticipated for the slip surface being remediated. The use of power curve parameters determined for a very large range of confining pressures would result in there being an insignificant difference between the M-C and the power curve strength envelopes. This was likely the case for De Mello (1977), who performed tests on rockfill under confining stresses up to 5000 kPa.

Charles and Watts (1980) argue that a higher factor of safety but more accurate models would be superior to current practice. Nicks and Adams (2014) also supported the use of more realistic strength models in design. For the normal stress range the slip surface is anticipated to fall within, the best approximation to the actual curved strength envelope using the Mohr criterion will involve $c' \neq 0$. However, c' is typically set to zero for design. This introduces an unspecified factor of safety to the failure envelope (Charles & Watts, 1980). By using more accurate models for shear strength in the form of power curves, the introduction of this unspecified factor can be avoided.

The use of power curves to describe shear strength can considerably improve the economy of designs especially at shallow depths where the confining pressure is low. Morgenstern and Dusseault (1978) stated that failing to recognize the curvature of failure envelopes and consequently underestimating the shear strength can result in designs that are uneconomical. This could cause potentially favourable techniques such as granular shear keys to be rejected as a viable remedial option. However, the shear strength at shallow depths would also be highly susceptible to the effect of pore water pressure. The introduction of positive pore water pressure would result in a significant reduction in strength; conversely, negative pore water pressure (i.e. suction) would have the opposite effect (Charles & Watts, 1980).

2.2.5 Role of dilation in granular materials and the factors that control dilatancy

What can be concluded from the previous section is that the increase in the angle of internal friction at low confining pressures is attributed to high rates of dilation. As the confining pressure increases, this dilation is reduced and the angle of internal friction decrease. The peak strength envelope will therefore be curved until a confining stress is reached at which dilation is fully inhibited. The friction angle at this confining stress and beyond will be equal to the critical state value (Abdul Razaq, 2007). Thus, dilation plays a significant role in the strength of granular materials subjected to lower confining pressures. Consequently, there has been a considerable amount of work (Rowe, 1962; Marsal, 1967; Charles & Watts, 1980; Bolton, 1986) performed to characterize the relation between strength and dilatancy for granular materials.

Changes in the stress field result in the displacement, rotation, and breaking of particles (Marsal, Mechanical Properties of Rockfill, 1973). Consequently, dilation that develops in response to these processes has been found to depend on particle shape, particle strength, and the degree of compaction (Charles & Watts, 1980). Studying the shear strength of Athabasca Oil Sands, Dusseault and Morgenstern (1978) attributed the very high rates of dilation before failure to a dense interpenetrative structure and grain surface rugosity.

Dilation was increasingly suppressed in response to increases in the normal stress, which resulted in the shearing of grains (particle breakage). The apparent cohesion in granular materials at high normal stresses is a result of this shearing (Dusseault & Morgenstern, 1978). Marsal (1967) found that an increase in particle breakage resulted in a decrease in the shear strength of materials. A plot of the principal stress ratio at failure, $\left(\frac{\sigma_1'}{\sigma_3'}\right)_{\mathrm{f}}$, against the particle breakage, B, showed a lower bound being established for values of B exceeding 20%. For values of B below this though, the data spread rapidly increased and no correlation could be identified (Marsal, 1967).

Particle breakage was found to be progressive and dependent on the gradation, the crushing strength of the particles, and the stress levels experienced by the particles (Marsal, Mechanical Properties of Rockfill, 1973). It has been observed that the GSD in turn shifts toward smaller grades as a result of particle breakage during shear, with higher confining stress resulting in greater shifts (Marsal, 1967; Bishop, 1966). The relation between particle breakage and the crushing strength of the particles can be explained by the work of Dusseault and Morgenstern. They found the amount by which the rate of dilation decreased in response to an increase in the normal stress seemed related to the grain mineralogy and competence. Intuitively, these factors will influence the crushing strength of the corresponding particles. Further to this point, the shear strength of Ottawa sand specimens was considerably less affected by increasing normal stress than the two other sands tested by Dusseault and Morgenstern. This was attributed to the tangential grain contacts and the high strength of the grains (Dusseault & Morgenstern, 1978). These tangential grain contacts would result in lower concentrations of stresses than highly angular grain contacts, supporting the previously stated relation between dilation and particle shape. Dusseault and Morgenstern also found increased grain angularity and surface rugosity resulted in greater residual strength.

2.2.6 Using nonlinear failure envelopes in design

It has been established that granular materials possess nonlinear failure envelopes. To ignore the curvature of the strength envelope by using the critical friction angle in design can lead to over-conservative design. The curvature can be accounted for by using secant friction angles (Eq. 2.1) or by using power laws (Eq. 2.4) to represent the shear strength. Duncan et al. (2014) state that the secant effective friction angle, φ_{s}', at a specified confining stress, σ_3', can be determined from a curved strength envelope using

$$\varphi_{\mathrm{s}}' = \varphi_0 - \Delta\varphi \log_{10}\left(\frac{\sigma_3'}{P_{\mathrm{a}}}\right) \tag{2.5}$$

where φ_0 is the value of φ_{s}' for σ_3' equal to 1 atm, $\Delta\varphi$ is the reduction in φ_{s}' for an increase in σ_3' by a factor of 10, and P_{a} is the atmospheric pressure.

The decision to use nonlinear strength envelopes in design can be made by evaluating the impact it can have over the range of confining stresses that are typical for granular shear keys. Strength data for a sandstone rockfill and a basalt rockfill tested by Charles and Watts (1980) is shown in Figure 2.9. Using the density values reported by Charles and Watts (see Table 2.4), the confining stresses were converted to depths to put the data into the context of the depths recommended in the literature for granular shear keys (also shown on the plot). Two strength envelopes are plotted for each of the rockfills; one that is a linear fit using the friction angle for a dilation rate of zero (φ'_{ZDA}), and another that is a nonlinear fit using a power curve. The parameters used to plot these strength envelopes are summarized in Table 2.4.

It is clear from the plot that the Mohr-Coulomb model underestimates strength compared to the power law model. This holds true even at depths far exceeding what is typical for granular shear keys. What this implies is that the effects of dilation clearly apply for the effective stress ranges covered by granular shear keys, with particle crushing not beginning to impede these effects until much greater depths.

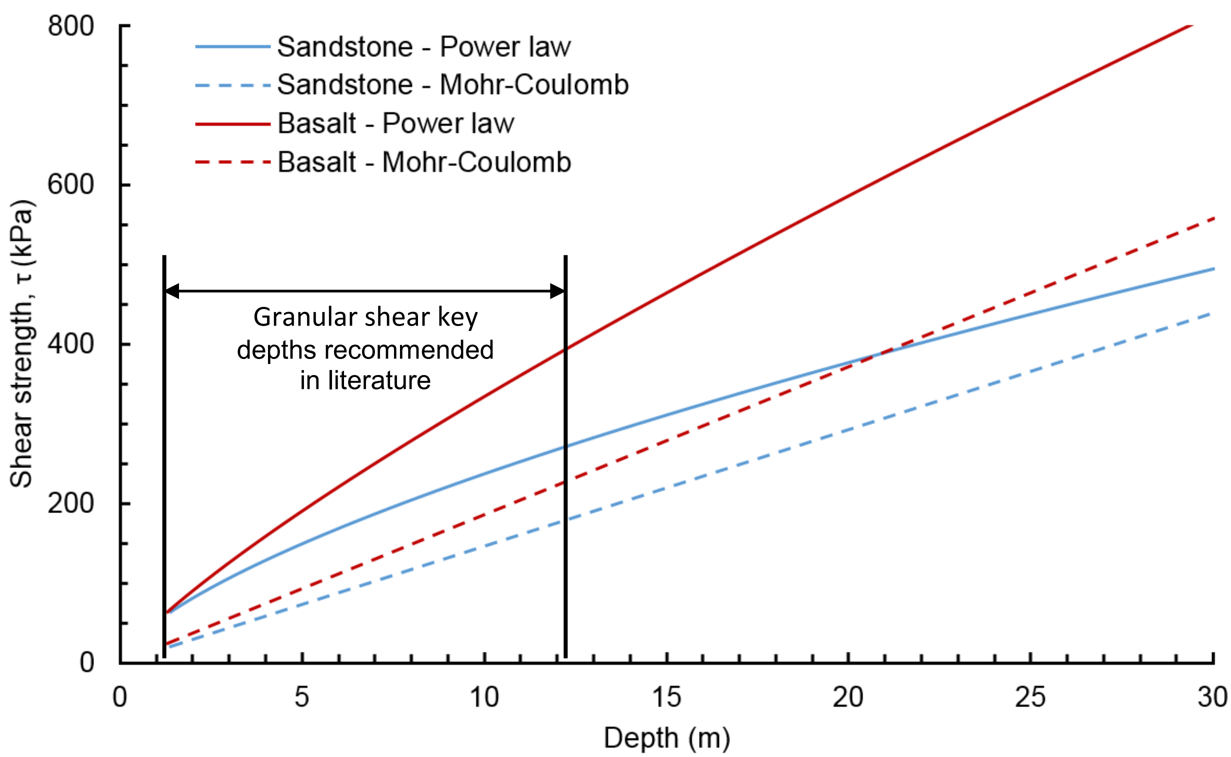


Figure 2.9: Linear and nonlinear strength envelopes plotted for a sandstone rockfill and a basalt rockfill, using test data from Charles and Watts (1980). The range of depths recommended in the literature for granular shear keys is also shown.

The difference between the two strength envelopes was plotted in Figure 2.10. By subtracting the linear estimate from the power curve, the component of the strength associated with the mineralogy of the grains could be eliminated and the effect of dilation can be isolated. The curves for the two materials appear to suggest that it is more conservative to use the Mohr-Coulomb model for the basalt rockfill than for the sandstone rockfill. This can be explained by examining the density and the grain sizes of the two rockfills. The density of the basalt rockfill was greater than that of the sandstone rockfill, so the effect of dilation would be expected to be greater for the basalt rockfill. The D_{10} and D_{60} sizes for the basalt rockfill were 1.3 mm and 16 mm, respectively. For the sandstone rockfill, they were less than 1 mm[1] and 7.3 mm, respectively. The larger grain sizes for the basalt rockfill would be expected to increase the rate of dilation at failure. Both of these processes would result in greater curvature in the strength envelope of the basalt rockfill compared to that of the sandstone rockfill, thereby resulting in a greater degree of conservativeness if the Mohr-Coulomb model is used to estimate its strength.

Table 2.4: Summary of the parameters used to model the linear and nonlinear strength envelopes against depth for the data reported by Charles and Watts (1980) from tests on a sandstone rockfill and a basalt rockfill.

			Power law parameters		
Material	Density (kN/m^3)	φ'_{ZDA} (°)	A	b	Reference
Sandstone	20	36.2	6.8	0.67	Charles and Watts 1980
Basalt	21	41.5	4.4	0.81	Charles and Watts 1980

Another observation that can be made in Figure 2.10 is that the curve corresponding to the sandstone rockfill appears to approach zero at a shallower depth than the basalt rockfill. This suggests that dilation in the sandstone rockfill is being suppressed at lower confining stresses than the basalt rockfill. The grading and grain angularity for the two rockfills was similar, so this can be explained by examining the crushing strength of the particles. Charles and Watts (1980) reported the parent rock strength indices for the two rockfills. The basalt parent rock had strength indices that were almost twice those of the sandstone rockfill, indicating the basalt particles would be stronger than the sandstone particles. Thus, dilation in the sandstone rockfill would be expected to be suppressed at shallower depths than the basalt rockfill.

[1] The GSD curve reported for the sandstone rockfill did not go below D_{30}, for which the grain size was 1 mm.

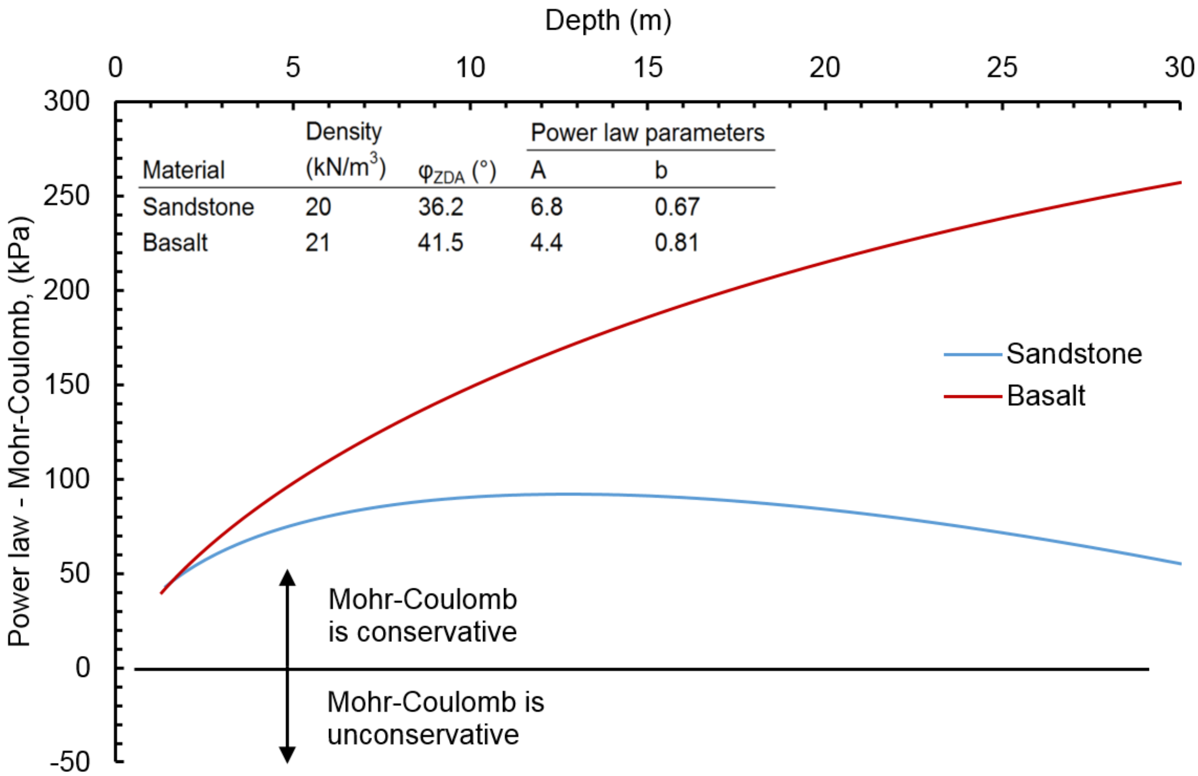


Figure 2.10: The difference between the strength estimated by the power law and the Mohr-Coulomb models plotted against depth.

Bolton (1986) warns that friction angles in excess of the critical friction angle are not permanent if strain continues to develop. Designing with realistic, nonlinear failure envelopes should therefore be approached with caution. A slope undergoing progressive failure could result in a granular shear key experiencing strains exceeding that which it can accommodate for the design strength to remain valid. The designer must be aware of the maximum strain that can be tolerated by a granular shear key before the peak shear strength is exceeded. The strain at which this occurs was shown to increase as the confining pressure increased (Charles & Watts, 1980). The implication of this finding in the context of granular shear keys is that shallower granular shear keys can be designed with greater strengths but will have a lower tolerance for movement than deeper shear keys. This can be seen in the data shown in Table 2.5, in which the axial strain at failure at various confining stresses is summarized from Charles and Watts (1980). The principal stress ratios and corresponding friction angles are also shown. The axial strain at failure is labelled for several points along the power law strength curves shown in Figure 2.11.

Table 2.5: The axial strain at failure, principal stress ratio and corresponding friction angles for triaxial compression tests on a sandstone rockfill and a basalt rockfill for various confining stresses, from tests performed by Charles and Watts (1980).

Material	Confinement, σ_3' (kPa)	Axial strain at failure, ε_f (%)	Principal stress ratio at failure, $(\sigma_1'/\sigma_3')_f$	Friction angle, φ_s' (Degrees)	Reference
	27	1.8	14.0	60.11	
	92	2.5	7.8	50.71	
	100	2.5	7.3	49.25	
	282	5.8	5.3	42.86	
Sandstone	695	5.9	4.3	38.48	
	27	1.2	19.2	64.30	Charles and Watts (1980)
Basalt	100	2.1	12.7	58.65	

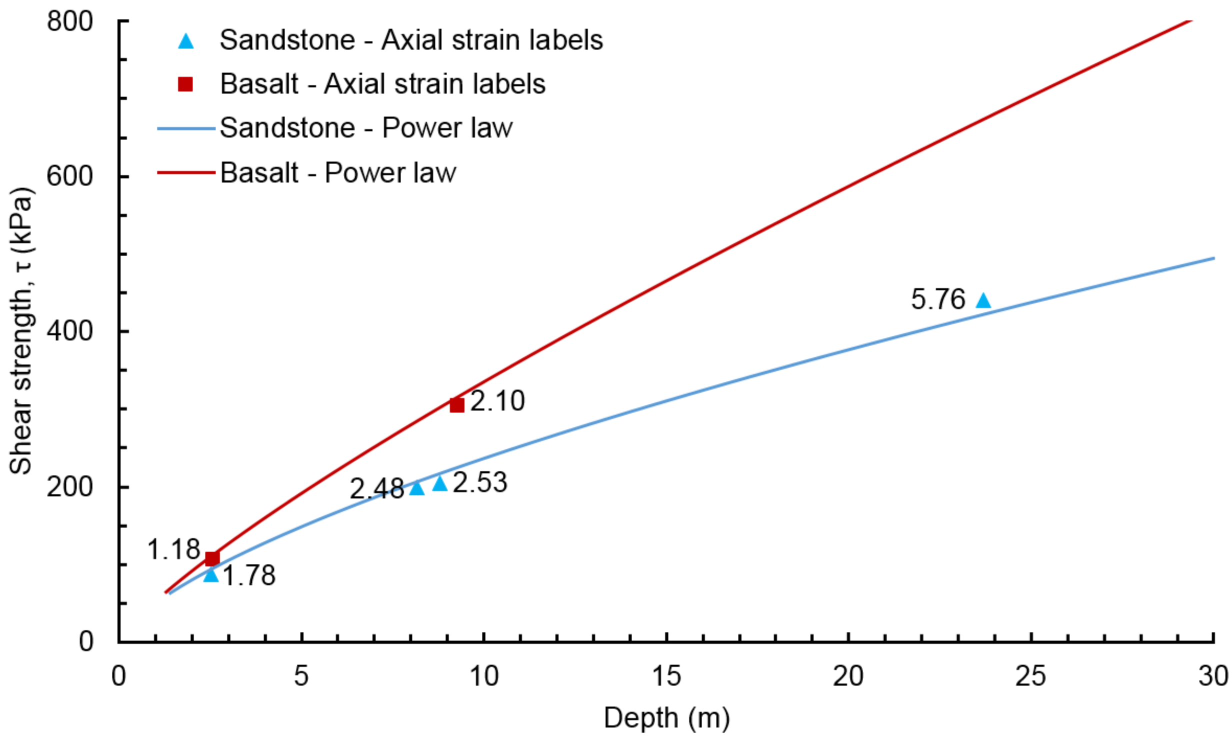


Figure 2.11: Power law failure envelopes for the sandstone rockfill and basalt rockfill tested by Charles and Watts (1980). The axial strains at failure (%) for several confining stresses are labelled.

2.2.7 Importance of using appropriate strength parameters

The use of the Power Law model to estimate shear strength has been shown to more accurately represent the real shear strength of granular materials at low confining stresses than the Mohr-Coulomb model. However, the strength parameters used to estimate shear strength are highly dependent on the range of confining stresses at which testing was performed. This is easily observed when evaluating the secant friction angle at different points along the strength envelope. The same applies to the strength parameters used as inputs for the Power Law model. It is therefore critical that these parameters be obtained from tests performed over a range of confining stresses which are as near as practical to the anticipated stress conditions along the slip surface.

To evaluate the benefit of adopting the Power Law model and highlight the importance of selecting appropriate strength parameters, 2D LE stability analyses were performed on a 3H:1V slope with an idealized slip surface, shown in Figure 2.12. The slope was modelled with a friction angle of 8°, cohesion of 2.75 kPa and a unit weight of 20 kN/m^3. Four materials were modeled with the parameters summarized in Table 2.6. Each material was modelled using parameters obtained for the Power Law model and three commonly-used friction angles for the Mohr-Coulomb model. The effective width of a modelled shear key was then modified until a target factor of safety of 1.3 was satisfied. The effective depth was kept constant at 4.8 m.

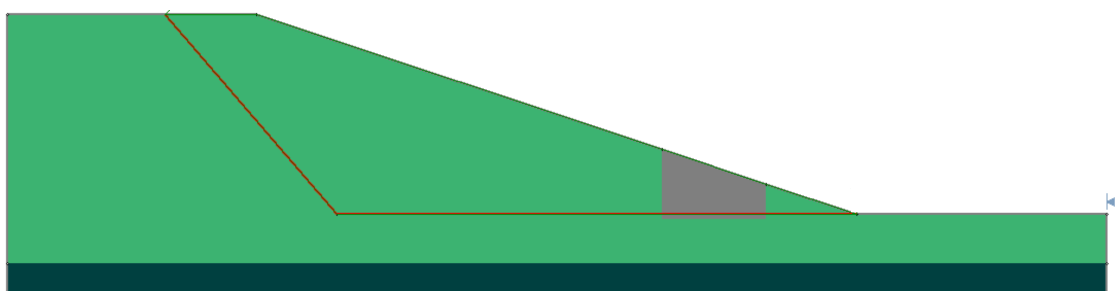


Figure 2.12: The idealized slope used to model the change in effective width required to satisfy a factor of safety of 1.3 for different strength parameters.

As can be seen in Table 2.6, to satisfy the target factor of safety, the effective width had to increase when switching from the Power Law model to the Mohr-Coulomb model for the Sandstone, Slate and Basalt. The stress ranges for which the strength parameters for these materials were obtained were reasonable given the depth of the slip surface in the model. On the other hand, the shear key effective width had to decrease when modelling the Crushed olivine basalt and switching from the Power Law model to the Mohr-Coulomb model for friction angles of 40° and 45°. This can be explained by observing the stress range for which the Power Law strength parameters were obtained. The stress range for the Crushed olivine basalt was considerably larger than those of the other materials. Over this large range, the strength envelope for the Crushed olivine basalt would appear more linear. This is because most of the curvature resulting from dilation would occur at the low end of this stress range. Consequently, the strength parameters would produce a more linear estimate for shear strength which would be a closer approximation of critical state conditions. Thus, adopting the Mohr-Coulomb model with a friction angle of 45° would yield a greater estimate for shear strength in this case.

The results in Table 2.6 show that the potential benefits of adopting the Power Law model, measured in terms of reducing the size of the shear key that is required, can be significant. The example provided using Crushed olivine basalt also clearly demonstrates that these benefits can be forfeited should modelling be performed using strength parameters obtained for conditions that poorly represent the anticipated field conditions.

Table 2.6: Summary of the materials and the strength parameters used to evaluate the change in the effective width required to satisfy a target factor of safety of 1.3.

Material	Unit weight	M-C	Power law		Stress range	Avg. D_e	b_e	F	Increase in b_e (%)
	kN/m^3	φ' (°)	A	b	kPa	(m)	(m)		
Sandstone A	20		6.8	0.67	27-695	4.8	6.9	1.310	-
	20	35				4.8	18.5	1.310	168%
	20	40				4.8	13.8	1.303	100%
	20	45				4.8	10.6	1.300	54%
Slate B1	21		5.3	0.75	100-500	4.8	5.5	1.308	-
	21	35				4.8	17.0	1.303	209%
	21	40				4.8	13.3	1.311	142%
	21	45				4.8	10.3	1.310	87%
Basalt C	21		4.4	0.81	27-695	4.8	4.8	1.312	-
	21	35				4.8	17.0	1.303	254%
	21	40				4.8	13.3	1.311	177%
	21	45				4.8	10.3	1.310	115%
Crushed olivine basalt	19.7		1.563	0.842	211-4501	4.8	17.0	1.301	-
	19.7	35				4.8	18.5	1.303	9%
	19.7	40				4.8	14.0	1.302	-18%
	19.7	45				4.8	10.8	1.301	-36%

2.3 Rockfill Columns as an Alternative to Trenched Granular Shear Keys

Rockfill columns used as shear keys, as introduced in § 2.1.1, were included in the scope of this work. Per Barksdale and Bachus (1983), rockfill columns have been used since the 1950's but are typically intended for resisting compressive loads. Used as shear keys, rockfill columns are installed to resist the lateral loads associated with slope movement. The following section aims to support the inclusion of rockfill column case studies in this work. This is achieved by first outlining the general similarities and differences between trenched shear keys and rockfill column shear keys. The rest of the section shadows the topics of § 2.2, showing how the mobilization of shear resistance, modes of failure, and landslide site conditions for rockfill column shear keys compare to those of trenched shear keys.

2.3.1 General comparison of rockfill columns and trenched shear keys

Rockfill columns are used to promote slope stability in many of the same ways as trenched granular shear keys. First, rockfill columns are a replacement technique wherein the native soil is replaced by granular material possessing greater shear strength (Abdul Razaq, 2007). The intent is to intercept an existing or potential slip surface in order to increase the shear resistance. Second, rockfill columns can promote drainage from the slope since the backfill is free-draining (Abdul Razaq, 2007). Drainage would, of course, be contingent on there being a conduit through which excess water could be discharged. The resulting relief of pore pressure further increases the shear resistance. These two characteristics are fundamental to both rockfill columns and trenched shear keys.

Some differences between rockfill columns and trenched shear keys are apparent too. One difference is that rockfill columns will lower the overall compressibility of a slope or embankment subjected to vertical loading (above the columns). This increase in stiffness will increase shear strength (Barksdale & Bachus, 1983). This difference could potentially result in improved performance and economy for rockfill column projects, but only where vertical loading is a factor. This theory is explored using case study data, in Chapter 4. Another difference is that rockfill column shear keys comprise individual columns, while trenched shear keys comprise a single, continuous trench. This difference can be accounted for through the relatively simple use of area replacement ratios, which is discussed at length, in Chapter 3.

2.3.2 Mobilization of shear resistance in rockfill column

As with trenched granular shear keys, rockfill columns are a passive technique. This means rockfill columns must also undergo deformation for shear resistance to mobilize (Abdul Razaq, 2007). Considerable work has been completed by researchers (Kim, 2007; Abdul Razaq, 2007; Thiessen, 2010) at the University of Manitoba in Winnipeg, Manitoba on the shear strain required to mobilize shearing resistance in rockfill columns. This research has included direct shear testing of scaled rockfill column models under various conditions and configurations (Abdul Razaq, 2007), and even a full-scale field test where rockfill columns were loaded to failure (Thiessen, 2010).

The degree of improvement resulting from the installation of rockfill columns is often evaluated in terms of the shear strength of the composite soil. The composite soil is defined as a combination of the rockfill column and the native soil surrounding it. Studies suggest the sum of the mobilized shear strength of each of these individual materials is greater than the shear strength of the composite soil. This has been supported by both theoretical and experimental data. This finding was especially relevant for cases where the shear strength of the native soil was below 30 to 40 kPa and/or where the rockfill columns were particularly strong whether from having a high friction angle or applied normal stress (Barksdale & Bachus, 1983).

2.3.3 Rockfill column modes of failure

Rockfill columns possess the same basic modes of failure as trenched granular shear keys, but some additional modes of failure have also been documented. The construction of a test embankment to intentionally fail a slope that had been stabilized with rockfill columns showed that the columns experienced bending deformations in addition to shear (Thiessen, 2010). The researcher demonstrated that granular material could support this bending though if the material could be kept in a state of compression.

Another study showed granular piles subjected to compressive loads could fail either by bulging, general shear failure, or by sliding (Sharma, Kumar, & Nagendra, 2004). Barksdale and Bachus (1983) found axial loading at the top of a single column created a bulge to a depth of 2 to 3 m below surface. The bulge was noted to increase the lateral stress and thus the confinement on the surrounding native soil. The grouping of rockfill columns was found to reduce this susceptibility to bulging. This is supported by Sharma et al. (2004), who found the dimensions of the granular piles and the configuration of the system of piles determined which mode of failure was exhibited. While the case studies collected for this work were for rockfill columns constructed to resist lateral loads, designers should nevertheless be aware of these compressional modes of failure too.

2.3.4 Landslide conditions commonly addressed using rockfill columns

The conditions where rockfill columns are used to mitigate landslides are mostly identical to those where trenched shear keys are used. While all rockfill column projects considered for this research involve landsliding, rockfill columns are particularly ideal when a combination of compressive and shear loads is expected (Proudfoot D. , 2016). As with trenched shear keys, rockfill columns are a suitable option when the right-of-way is limited, such as along a riverbank (see Figure 2.13).

Another similarity is in the type of soils at these sites. Abdul Razaq (2007) reviewed several studies on the use of rockfill columns in embankment and foundation applications, finding that rockfill columns were widely considered to be most effective when constructed in soils with shear strengths ranging from 10 to 50 kPa. Soils with low shear strengths introduce the risk of providing insufficient lateral support, whereas those with higher shear strengths may not benefit from replacement with rockfill.

This type of remediation technique benefits from some advantages over trenched granular shear keys. One advantage is the avoidance of the large destabilizing effect of a continuous trench by applying a hole-by-hole excavation sequence. Other advantages are their suitability for deep-seated slides due to the manner of excavation, and their ability to act as vertical drains to relieve excess pore water pressure (Thiessen, Alfaro, & Blatz, 2007). However, due to the somewhat specialized equipment involved, the cost of rockfill columns can be greater than trenched granular shear keys at shallower depths. A review of rockfill columns in Europe revealed typical column lengths ranged from 4 to 10 m but columns up to 21 m had also been reported in both the USA and Europe (Barksdale & Bachus, 1983). Yarechewski and Tallin (2003) remark that the depth of excavation for rockfill columns is not limited. It was recommended that the construction of columns of these lengths be approached with caution though due to problems concerning the stability of the hole, contamination of the backfill, and challenges ensuring proper densification (Barksdale & Bachus, 1983).

Figure 2.13: A large-diameter auger being used in the installation of a rockfill column along a riverbank underneath a bridge. Photo by Hugh Gillen.

One last difference between rockfill columns and trenched granular shear keys is that high pore pressures may be more frequently encountered when constructing rockfill columns. This is because rockfill columns are typically used to intercept deeper slip surfaces. Furthermore, the existence of upward gradients in the Red River Basin led practitioners in the region to observe poorer performance than expected. This reduction in effectiveness was a direct result of upward gradients reducing the normal effective stress at depth, particularly at sites where the rockfill columns were not installed to sufficient depths. This was more common for early projects in thick clays (up to 40 m), when rockfill column installation depths were limited to approximately 20 m (Tallin, 2017). In some of these cases, high ground water pressures at depth led to the development of slip surfaces in the clay below the rockfill columns. Improved drilling techniques capable of advancing the rockfill columns into the underlying till have largely solved this problem in recent years (Tallin, 2017).

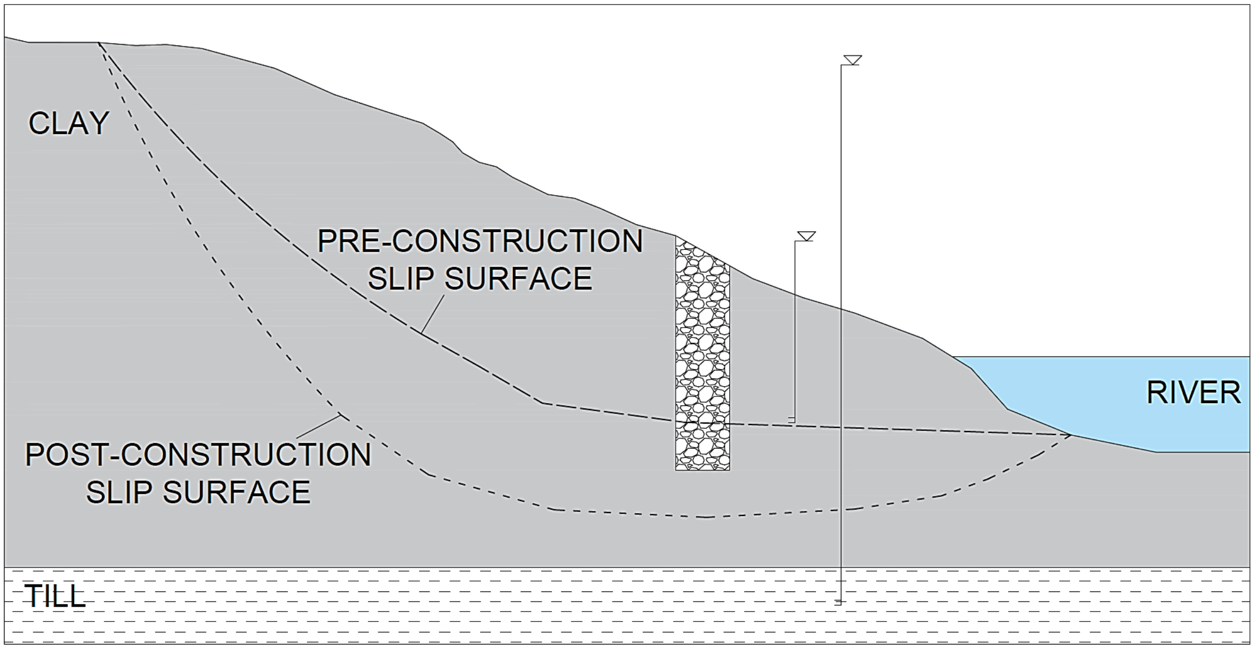


Figure 2.14: Schematic of a hypothetical slope in which an upward gradient exists between a till and the overlying clays.

2.4 Factors of Safety in Granular Shear Key Design

The degree of improvement made to a slope as a result of remedial works needs to be quantified during design. For granular shear keys, this is typically done by comparing the shear strength before and after the introduction of the shear key. This is captured by the factor of safety, F, which is defined as

$$F = \frac{Shear\ strength\ of\ the\ soil}{Shear\ stress\ required\ for\ equilibrium} \tag{2.3}$$

Slope stability analyses tend to focus on the shear strength because it is often the parameter for which there is the most uncertainty. The factor of safety applies to the individual components used to represent the shear strength (Duncan et al., 2014). Thus, if the M-C criterion is used to represent shear strength, the factor of safety would apply to cohesion, c, and the angle of friction, φ. This definition assumes the factor of safety is the same along the entire length of a slip surface in limit equilibrium analyses. The factor of safety can be shown to vary along the slip surface, using finite element analyses but the average value calculated by limit equilibrium methods has been accepted as a valid measure of stability (Duncan et al., 2014). Progressive failure could occur if the factor of safety at a point along the slip surface is below unity. This can be avoided if the overall factor of safety is sufficiently greater than unity and is calculated using shear strength parameters that can be mobilized at any point along the slip surface (Duncan et al., 2014).

Special considerations may be necessary when calculating the factor of safety in the passive zone of the slide (i.e. at the toe). Duncan et al. (2014) states there is a zone of passive earth pressures near the toe of a slide. This can lead to the development of very large compressive or tensile stresses. Duncan et al. explains these problems are due to the resultant force on the base of the last slice aligning with the interslice force in a direction far from that corresponding to minimum passive earth pressures. In doing so, the forces can combine constructively and create a much larger positive or negative force. The factor of safety calculated at this point may therefore be unreasonable in limit equilibrium analyses when the software is permitted to search for the critical slip surface. This can be resolved by applying passive earth pressure theory to obtain a reasonable starting angle for the search, and by constraining the search parameters such that very steep slip surface exit angles are not permitted (Duncan et al., 2014).

The target factor of safety for granular shear key projects tends to be between 1.3 and 1.5. These targets are intended to ensure the design can provide sufficient shear strength to resist reasonably foreseeable changes to the site conditions, and to provide a buffer against uncertainty (Duncan et al., 2014). For example, there is a typical coefficient of variation of 10% for friction angles measured in the laboratory (Nicks & Adams, 2013). The allowable uncertainty, Δ, expressed as a percentage by which the shear strength could be lower than anticipated, increases with the factor of safety that is selected:

$$\Delta = \frac{F - 1}{F} \text{ x } 100\% \tag{2.3}$$

The factor of safety that is targeted in design should also reflect the potential consequences of failure. As the consequences of failure become greater, the factor of safety must also increase (Duncan et al., 2014).

2.4.1 Selecting an appropriate factor of safety

Duncan et al. (2014) state that experience is the basis for recommending factors of safety. For example, they cite a factor of safety of 1.5 for long-term steady seepage, adopted by the U.S. Army Corps of Engineers for embankment and excavation slopes such as those relating to dams and levees. They note it is important to consider the context for which these recommendations have been established. The U.S. Army Corps of Engineers consistently apply the same methods of exploration, testing, and analysis to projects that do not widely vary in terms of the uncertainty that is involved (Duncan et al., 2014).

The factors of safety targeted for granular shear key projects in Canada are also based on experience. Practitioners have typically adopted a factor of safety of 1.3, or 1.5 if infrastructure is located on the slope. Catastrophic failures are almost nonexistent, putting into question whether these target values are appropriate or over-conservative.

The use of an inappropriate factor of safety is one of the seven types of recurring error in landslide studies and design observed by Cornforth (2007). Cornforth states that a factor of safety of 1.5 has become the norm for *slope stability studies*, and that this is appropriate given that these either involve relatively small earthworks, or are considered high risk projects. Cornforth reasons a factor of safety of 1.5 may be inappropriate for *landslide studies*. Landslides can range significantly in volume, in geological site conditions, and in the level of technical work that is performed on them. This variability can make it difficult in some cases to achieve a target of 1.5, deterring any action from being taken. On the other hand, for some cases a factor of safety of 1.5 would be very conservative. For these reasons, Cornforth argues that landslides should not be approached with a set limit but rather with a factor of safety deemed appropriate on a case-by-case basis. Furthermore, a major advantage in the study of landslides is that the factor of safety is known to be at unity at the onset of instability. This effectively eliminates the need to target factors of safety as high as those used in slope stability studies since even a marginal improvement can result in stability.

Cornforth (2007) suggests five factors that influence the factor of safety that is adopted: the velocity of the landslide, the level of study which has been performed, the size of the landslide, the potential consequences of continuing movements, and the experience of the geotechnical practitioner. In general, he suggests that the factor of safety be raised as the landslide velocity, level of uncertainty, and consequence of failure increase. As discussed previously, granular shear keys are typically adopted to address slow (<13 m/month) to extremely slow (<16 mm/year) landslides. The geometry of the slope, the location of the slip surface and its residual properties, and the properties of the granular backfill are all typically known. The consequences of the failure of a granular shear key are relatively low because of the slow rate of slope deformations. The high degree of tolerance for deformation exhibited by roads and railways also reduces the consequence of poor shear key performance. Thus, there is reasonable support for the adoption of a lower factor of safety for granular shear key design. On the other hand, the landslides that are addressed using granular shear keys do not tend to be large since the volume of the shear key can become prohibitively large and pose a risk to stability during construction. Cornforth recommends adopting a higher factor of safety for smaller landslides, so this would support the continued use of higher factors of safety. Finally, granular shear keys are a well-used technique across Canada and consequently, the geotechnical practitioners are experienced with the design of this type of structure. This level of experience would permit the use of lower factors of safety.

Cornforth (2007) suggests factors of safety between 1.15 and 1.50 are appropriate for use in landslide applications. On the other hand, Charles and Watts (1980) recommended using factors of safety between 1.5 and 2.0 for rockfills modelled using triaxial strength parameters and curved strength envelopes. This higher range of factors of safety is to accommodate for the considerable drop in shear strength post-failure for low confining stresses. The stark contrast between these two recommended ranges likely reflects the conservative nature of using the Mohr-Coulomb criterion as opposed to the more realistic power curve model. The factor of safety that is selected should therefore take into consideration the model used to describe shear strength.

From the points that have been presented, it can be concluded that the typical granular shear key target factor of safety of 1.30 in Canada is reasonable under the current practice of using the Mohr-Coulomb criterion to describe shear strength. Factors of safety as low as 1.15 should be considered where appropriate; this might include landslides where water levels do not change significantly, large landslides with minimal consequence of failure, and where there is extensive local experience. Should nonlinear models such as the power curve model be used, the factor of safety should be increased. Factors of safety of 1.50 should be more economically attainable by accounting for the more realistic and higher strengths at the typical range of confining stresses that apply to granular shear keys.

2.5 Summary of Literature Review

Granular shear keys were introduced as a method for landslide stabilization whereby an excavation is created to intercept an existing or potential slip surface and is backfilled with a granular material possessing a higher shear strength than the in-situ soils. Two types of granular shear keys were introduced: trenched shear keys, and rockfill column shear keys.

Granular shear keys promote stability by increasing shear strength and promoting drainage, both of which increase shear resistance. This technique is often adopted where the right-of-way is limited and in emergency situations. It is also common to find granular shear keys being combined with other structures, such as weighting berms. The key terminology used to describe granular shear keys is summarized in Table 2.1. This terminology is useful for stating the size of a shear key, and putting the landslide conditions in the context of the shear key.

The typical landslide remediated with granular shear keys seems to be comprised of fine-grained soils, and tends to be slow-moving and shallow. These would typically be classified as planar or compound slides. These landslide mechanisms are depicted in Figure 2.2 and Figure 2.3, respectively. Movements in these landslides are often controlled by changes in the shear resistance or by viscous deformations. Changes in the shear resistance are generally related to seasonal pore pressure fluctuations and erosion or soil weakening.

Granular shear keys are a passive type of structure, where the shear strength of the backfill is mobilized in response to deformations. The magnitude of the shear strength and the strain required for it to mobilize are controlled by a number of factors. The factors that practitioners tend to have the most control over are the relative density and the normal applied stress. Three modes of failure affect granular shear keys: shearing through, below, or along the base of the shear key. For each of these modes, stability is enhanced by either increasing the shear strength, the length of the slip surface, or both. The shear strength is typically modelled using the M-C constitutive model due to its relative simplicity and the ease with which it can be used in LE stability analyses. The principal limitation associated with the use of this model is that the shear strength of the granular backfill materials is nonlinear. The nonlinearity can in general be attributed to the dilatant behaviour of granular materials at low confining stresses. The power law model is proposed as a potential alternative to the M-C model. Adopting this more realistic approach to modelling shear strength can lead to more economic designs being realized. However, these higher shear strengths are only valid for limited ranges of strain, after which the materials will tend toward residual strengths.

Rockfill columns can be adopted as an alternative to trenched shear keys. Many similarities exist between the two, such as how they are both replacement techniques, their suitability for mitigating weak clay slides, and their ability to perform in the presence of variable water conditions. Some differences exist, raising a need to perform comparative analyses to quantify these differences.

The factor of safety is used as a means to quantify the degree of improvement made to the stability of a landslide after a granular shear key has been constructed. The factor of safety is defined as the ratio of the shear strength of the soil to the shear strength required for equilibrium. Granular shear keys enhance stability by increasing the average shear strength of the soil. Selection of an appropriate factor of safety requires an evaluation of the site conditions to which a granular shear key will be exposed, and an assessment of the level of risk that can be tolerated. A number of factors were presented along with the corresponding influence they have on the factor of safety. These factors are the velocity of the landslide, the level of study which has been performed, the size of the landslide, the potential consequences of continuing movements, and the experience of the geotechnical practitioner. The typical factor of safety that is targeted for granular shear key projects in Canada is 1.3. This increases to 1.5 when infrastructure is present. An argument can be made for the adoption of factors of safety as low as 1.15 given the facts that a marginal degree of improvement can satisfy stability and that the properties of the materials involved are known with relative certainty. Factors of safety should be increased if estimating strength using nonlinear strength envelopes, to accommodate for the considerable drop in shear strength (from peak to residual) for low confining stresses. By adopting this more realistic (and less conservative) estimation of shear strength, higher factors of safety should become more attainable though.

To aid practitioners, a compilation of granular materials and their properties is presented in Table 2.7. The corresponding grain size distribution curves are presented in Figure 2.15. Due to the number of materials that are plotted, the extents of the grain size distribution curves were outlined for gravels and for sands. The D_{10}, D_{30} and D_{60} sizes for these materials are summarized in Table 2.8. These resources are intended to serve as a quick reference for obtaining realistic estimates for the strength and types of gradations associated with typical granular shear key backfill materials.

Table 2.7: Compilation of granular materials and material properties. φ_0 is the value of φ'_s for σ'_3 equal to 1 atm, and $\Delta\varphi$ is the reduction in φ'_s for an increase in σ'_3 by a factor of 10.

Material	Density (kN/m^3)	Stress range, σ'3 (kPa)		Friction angles, φ'		Power curve parameters		Source
		Low	High	φ_0	$\Delta\varphi$	A	b	
Sandstone (A)	20	27	695	50.3	nd	6.8	0.67	Charles & Watts (1980)
Slate (B1)	21	100	500	55.8	nd	5.3	0.75	Charles & Watts (1980)
Slate (B2)	18	100	500	43.4	nd	3.0	0.77	Charles & Watts (1980)
Basalt (C)	21	27	695	59.2	nd	4.4	0.81	Charles & Watts (1980)
El Infiernillo crushed basalt	21.0	490.5	2452.5	55.7	13.9	1.5	0.83	Marsal (1967)
El Infiernillo blasted granitic gneiss	19.4	490.5	2452.5	44.9	9.8	0.7	0.95	Marsal (1967)
	15.9	490.5	2452.5	42.2	14.8	0.6	0.97	Marsal (1967)
El Infiernillo diorite	nd	39.24	2452.5	46.2	8.8	1.1	0.87	De Mello (1977)
Idem silic. Conglomerate	nd	39.24	2452.5	47.8	8.3	1.27	0.846	De Mello (1977)
Pizandaran sand and gravel	nd	39.24	2452.5	48	5.4	1.27	0.876	De Mello (1977)
VDOT 21 (limestone and granite)	21.8	43.1	205.9	53	12	1.491	0.804	Duncan et al. (2014)
Pinzadapan gravel	20.8	38.3	2441.9	51	9	1.341	0.858	Duncan et al. (2014)
Netzahn Dam rockfill	18.7	181.9	2441.9	50	10	1.307	0.841	Duncan et al. (2014)
Infiernillo Dam rockfill	16.6	38.3	1627.9	46	9	1.119	0.858	Duncan et al. (2014)
Mica Dam rockfill	19.4	488.4	2441.9	44	9	1.062	0.849	Duncan et al. (2014)
VDOT 21B (limestone and granite)	19.8	30.2	2106.7	44	10	1.049	0.843	Duncan et al. (2014)
Rawallen Dam sandy gravel	21.2	172.4	1053.4	58	10	1.755	0.840	Duncan et al. (2014)
Oroville Dam rockfill	23.3	861.8	4309.2	53	8	1.426	0.875	Duncan et al. (2014)
Basalt rockfill	21.0	488.4	2441.9	52	10	1.405	0.842	Duncan et al. (2014)
Oroville Dam gravel	23.9	220.2	2729.2	50	7	1.268	0.889	Duncan et al. (2014)
Round Bute Dam basalt rock	15.6	191.5	1340.6	51	14	1.411	0.774	Duncan et al. (2014)
VDOT 57 (limestone and phyllite)	17.4	28.7	210.7	48	11	1.224	0.825	Duncan et al. (2014)
Mica Dam sandy gravel	nd	689.5	3112.2	41	3	0.892	0.952	Duncan et al. (2014)
Crushed Olivine basalt	19.7	210.7	4500.7	55	10	1.563	0.842	Duncan et al. (2014)
Pyramid Dam shell	17.5	210.7	4500.7	53	9	1.440	0.859	Duncan et al. (2014)
Crushed Venatio sandstone	18.5	210.7	2729.2	43	4	0.964	0.937	Duncan et al. (2014)

Table 2.7 (continued)

Material	Density (kN/m^3)	Stress range, σ'3 (kPa)		Friction angles, φ'		Power curve parameters		Source
		Low	High	φ_0	$\Delta\varphi$	A	b	
Pumicieous sand (w=18%)	13.2	191.5	1340.6	48	10	1.216	0.841	Duncan et al. (2014)
Monterey No. 0 sand	16.5	28.7	1149.1	45	3	1.025	0.954	Duncan et al. (2014)
	14.5	28.7	114.9	35	0	0.700	1.000	Duncan et al. (2014)
Glacial outwash sand	17.6	95.8	3926.2	44	4	0.998	0.938	Duncan et al. (2014)
Port Allen lock sand	16.5	86.2	373.5	44	3	0.989	0.954	Duncan et al. (2014)
	15.7	86.2	373.5	40	1	0.846	0.985	Duncan et al. (2014)
Silica sand	16.9	95.8	488.4	37	0	0.754	1.000	Duncan et al. (2014)
	15.7			30	0	0.577	1.000	Duncan et al. (2014)
Pumicieous sand (w=25%)	12.1	191.5	1340.6	36	12	0.832	0.764	Duncan et al. (2014)
Sacramento River sand	16.3	95.8	3926.2	45	7	1.063	0.889	Duncan et al. (2014)
	15.4	95.8	3926.2	41	5	0.907	0.919	Duncan et al. (2014)
	14.8	95.8	3926.2	37	3	0.772	0.951	Duncan et al. (2014)
	14.1	95.8	3926.2	35	2	0.711	0.967	Duncan et al. (2014)

nd: Data not recorded.

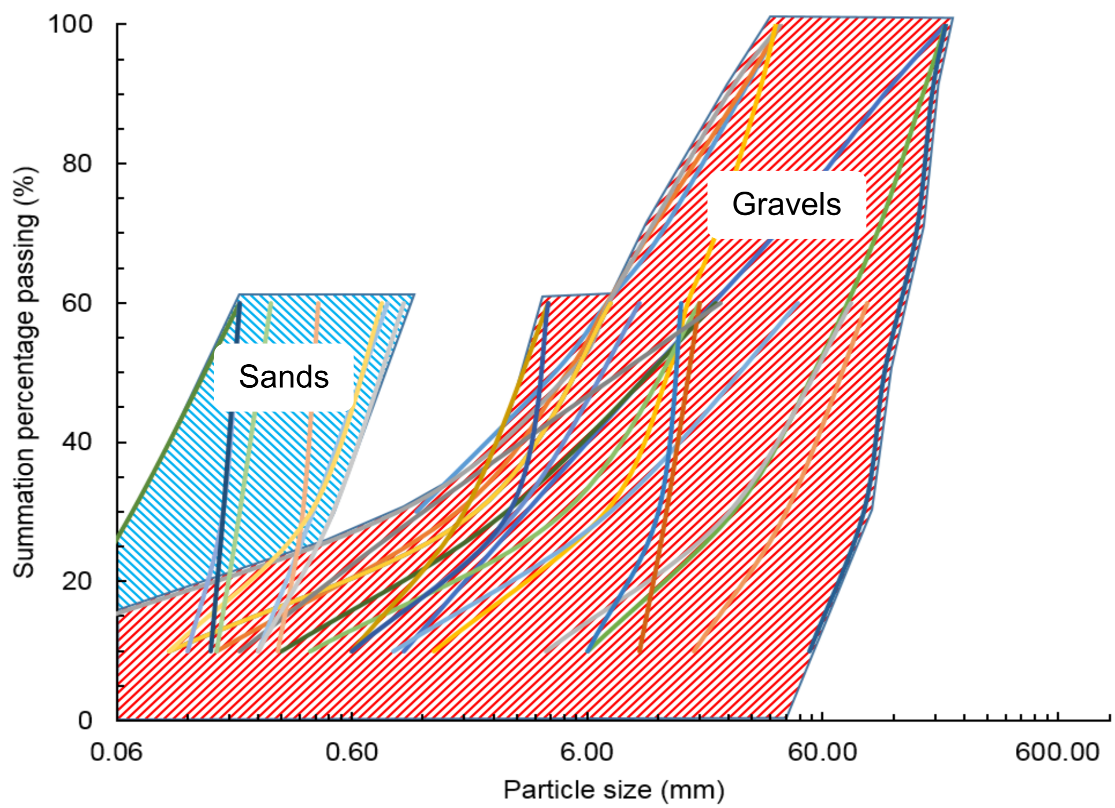


Figure 2.15: The grain size distributions of the granular materials listed in Table 2.6, with the extents shown for gravels and sands.

Table 2.8: Summary of the grain characteristics for the granular materials that were compiled.

			Particle size (mm)			
Material	USCS	Grain shape	D_{10}	D_{30}	D_{60}	Reference
Sandstone (A)	GW	Angular	nd	1.13	7.3	Charles & Watts (1980)
Slate (B1)	GW	Angular	0.16	1.68	7.3	Charles & Watts (1980)
Slate (B2)	GW	Angular	0.02	0.94	7.4	Charles & Watts (1980)
Basalt (C)	GW	Angular	1.33	7.2	16	Charles & Watts (1980)
El Infiernillo crushed basalt	GW	Angular	1	3.3	nd	Marsal (1967)
El Infiernillo blasted granitic gneiss	GW	Angular	6	25	nd	Marsal (1967)
	GP	Angular	53	90	nd	Marsal (1967)
El Infiernillo diorite	GW	Angular	20	nd	nd	De Mello (1977)
Idem silic. Conglomerate	GW	Angular	5	nd	nd	De Mello (1977)
Pizandaran sand and gravel	GW	nd	0.2	nd	nd	De Mello (1977)
VDOT 21 (limestone and granite)	GW	Angular	0.1	2	7.5	Duncan et al. (2014)
Pinzadapan gravel	GW	Subrounded	0.3	2.7	21	Duncan et al. (2014)
Netzahn Dam rockfill	GW	Subangular	0.9	7.5	47	Duncan et al. (2014)
Infiernillo Dam rockfill	GW	Angular	17	42	93	Duncan et al. (2014)
Mica Dam rockfill	GW	Subangular	4	24	79	Duncan et al. (2014)
VDOT 21B (limestone and granite)	GW	Angular	0.1	2	7.5	Duncan et al. (2014)
Rawallen Dam sandy gravel	GP	Rounded	0.6	3	10	Duncan et al. (2014)
Oroville Dam rockfill	GP	Rounded	0.4	4.8	18	Duncan et al. (2014)
Basalt rockfill	GP	Angular	1	3.6	19	Duncan et al. (2014)
Oroville Dam gravel	GP	Rounded	0.4	4.6	13.2	Duncan et al. (2014)
Round Bute Dam basalt rock	GP	Angular	6	12	15	Duncan et al. (2014)
VDOT 57 (limestone and phyllite)	GP	Subangular	10	13	18	Duncan et al. (2014)
Mica Dam sandy gravel	GP	Subangular	0.2	1.2	22	Duncan et al. (2014)
Crushed Olivine basalt	SW	Angular	0.6	1.8	4.1	Duncan et al. (2014)
Pyramid Dam shell	SW	Angular	0.6	2.8	4.1	Duncan et al. (2014)
Crushed Venatio sandstone	SW	Angular	0.03	0.07	0.2	Duncan et al. (2014)
Pumicieous sand (w=18%)	SP	Angular	0.24	0.41	0.85	Duncan et al. (2014)
Monterey No. 0 sand	SP	Rounded	0.29	0.37	0.43	Duncan et al. (2014)
	SP	Rounded	0.29	0.37	0.43	Duncan et al. (2014)
Glacial outwash sand	SP	Subrounded	0.1	0.4	0.8	Duncan et al. (2014)
Port Allen lock sand	SP	Rounded	0.12	0.17	0.2	Duncan et al. (2014)
	SP	Rounded	0.12	0.17	0.2	Duncan et al. (2014)
Silica sand	SP	Rounded	0.16	0.2	0.27	Duncan et al. (2014)
	SP	Rounded	0.16	0.2	0.27	Duncan et al. (2014)
Pumicieous sand (w=25%)	SP	Angular	0.24	0.5	1	Duncan et al. (2014)
Sacramento River sand	SP	Rounded	0.15	0.17	0.2	Duncan et al. (2014)
	SP	Rounded	0.15	0.17	0.2	Duncan et al. (2014)
	SP	Rounded	0.15	0.17	0.2	Duncan et al. (2014)
	SP	Rounded	0.15	0.17	0.2	Duncan et al. (2014)

nd: Data not recorded.

3 GRANULAR SHEAR KEY DESIGN – A REVIEW

Many guidelines exist for advisement on landslide best practices, both nationally and internationally. Perhaps most relevant for Canadian practice is the *Canadian Technical Guidelines and Best Practices related to Landslides: a national initiative for loss reduction*, a series of publications by the Geological Survey of Canada and available online through Natural Resources Canada. It should be noted though that specific guidelines outlining the design process that should be adopted for granular shear keys are infrequent. This section identifies those design guidelines containing information relevant to the design of trench-based or rockfill column shear keys consulted for this chapter. Supplemental information from guidelines for similar techniques or from individual case studies was also included where applicable. The guidelines cited in this section are summarized in the order they were published, in Table 3.1.

The discussion of these guidelines is structured in the same manner as granular shear key design would be approached. First addressed is the establishment of a base model as a starting point for the rest of the design process. This is followed by a discussion on the selection of an appropriate backfill material. The actual design of granular shear keys and the accompanying stability analyses are presented next, followed by considerations to be made during construction to successfully implement the design. Finally, expectations for the timeframe over which slope movements will persist are presented and the reasons for the given timeframes are discussed.

3.1 Establishing a Base Model

The first step in designing a granular shear key is to define the geometry of the slope that is moving. A site investigation is critical to determining the extent of the problem and characterizing the landslide. This step is advocated by almost all the guidelines, and its importance cannot be overstated. There are case studies where remediation efforts were unsuccessful for years, after which a more thorough site investigation revealed information which would have averted the waste of resources expended up to that point. Monitoring programs can also greatly contribute to the success of granular shear keys. They can provide information about the type of movement exhibited by the slope, help delineate the zone(s) of movement, and serve as a baseline against which post-remediation movements can be evaluated. This information feeds into the site model and reduces uncertainty. While monitoring programs are highly advisable, it is also understood that granular shear keys are often adopted to satisfy emergency or unplanned landslide repairs where the time required to monitor a slide may not be available. Following the establishment of a site model, the geometry is typically transcribed onto two-dimensional limit equilibrium software for stability analyses.

Table 3.1: A list of the guidelines consulted for the design of granular shear keys or similar structures, in chronological order.

Author(s)	Published by	Title	Year
R. Baker & E. Yoder	Highway Research Board Committee on Landslide Investigations	"Chapter 9: Stability Analyses and Design of Control Methods", in *Landslides and Engineering Practice*	1958
R. Barksdale & R. Bachus	Federal Highway Administration	Design and Construction of Stone Columns (Vol. I & II)	1983
R. Goughnour, J. Sung & J. Ramsey	American Society for Testing and Materials	"Slide Correction by Stone Columns", in *Deep Foundation Improvements: Design, Construction, and Testing*	1991
C. Denning	United States Department of Agriculture	"6G. Shear Trenches", in *Slope Stability Reference Guide for National Forests in the United States Vol. III*	1994
D. Cornforth	John Wiley & Sons	Landslides in Practice: Investigations, Analysis, and Remedial/Preventative Options in Soils	2005
Caltrans	California Department of Transportation	Caltrans Geotechnical Manual	2014

Landslides for which granular shear keys are being constructed typically have already experienced some amount of movement. For landslides in clay soils, it is well understood that slide movement can reduce the shear strength of the sliding materials to a residual state (Denning, 1994). In this state, the cohesion is assumed to have been eliminated and a residual friction angle is used to represent the shear strength of the soil.

The residual friction angle of the native soil is either estimated from repeated direct shear tests on soil samples taken from the zone of sliding, or from back analyses of the failure (Denning, 1994). Cornforth and Fujitani (1991) remark that weathered siltstones studied in Oregon required more than three stress reversals in a direct shear test before the laboratory strength properties approached the actual residual properties of a slide. Back analysis is commonly employed once reasonable confidence in the rest of the model has been established. In this approach, the factor of safety of the existing slope is taken to be between 0.95 – 1.00 (Denning, 1994). Using strength properties estimated via back analysis of the slide tends to provide better results than those attained from laboratory tests (Cornforth, 2005). Denning (1994) notes the availability of charts where simple index test results can be used to estimate the residual shear strength too.

3.2 Considerations When Selecting a Backfill Material

The backfill material is an essential component of granular shear keys. The strength of this material will influence the size of the shear key required to achieve the stability targets. At the same time, the proximity of this borrowed material will affect the hauling costs. The suitability of granular shear keys is thus highly dependent on the availability of nearby, high strength backfills. The following section describes the ideal backfill provided it can be sourced economically.

3.2.1 Recommended backfill materials and gradations

Caltrans (2014) states quarry spalls or similar materials are commonly used. These high-shear-strength materials are relatively easy to place below the groundwater table and are relatively easy to compact. The Oregon Department of Transportation recommends the use of backfill consisting of clean, hard, angular, and durable gravel or rock. Barksdale and Bachus (1983) state that crushed stone is preferred but, based on availability, natural gravel is often used too. Barksdale and Bachus state that a coarse (12-75 mm), open-graded stone is typically used, whereas Denning (1994) recommends using a backfill that is well-graded from coarse to fine but also free-draining. This latter recommendation is consistent with the conclusion by Marsal (1967) that well-graded rockfills possessed the greatest shear strength. The requirement that the backfill be free-draining may relate to the prevention of pore water pressure build-up in response to shear. Dusseault and Morgenstern (1978) found the rate of displacement had almost no effect on the shear strength of sand; this was likely due to the free-draining nature of the material. Potentially helpful in satisfying this criterion, the specific permeability of a compacted granular medium was found by Kenney et al. (1984) to be almost exclusively dependent on the D_5 grain size parameter. The shape of the gradation curve was not found to have a significant effect (Kenney et al., 1984). Alternatively, Hazen (1892) proposed an empirical relationship between the permeability of uniform, loose sands and the D_{10} grain size parameter.

Barksdale and Bachus (1983) recommend using Alternative No. 1 or No. 2 gradations. These gradations are summarized in Table 3.2, along with Alternative No. 3 and No. 4. The grain size extents corresponding to each of these gradations are shown plotted in Figure 3.1.

Table 3.2: Summary of the gradations for Alternative No. 1, No. 2, No. 3, and No. 4 given by Barksdale and Bachus (1983).

Sieve size	Percentage passing								
Inches	0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0
mm	12.7	19.1	25.4	38.1	50.8	63.5	76.2	88.9	101.6
Alternative No. 1	0 - 5	0 - 10	nd	nd	40 - 90	nd	90 - 100	na	na
Alternative No. 2	nd	nd	2	nd	100	na	na	na	na
Alternative No. 3	0 - 5	0 - 10	nd	0 to 60	nd	25 - 100	nd	90 - 100	100
Alternative No. 4	0 - 5	10 - 55	20 - 100	nd	65 - 100	100	na	na	na

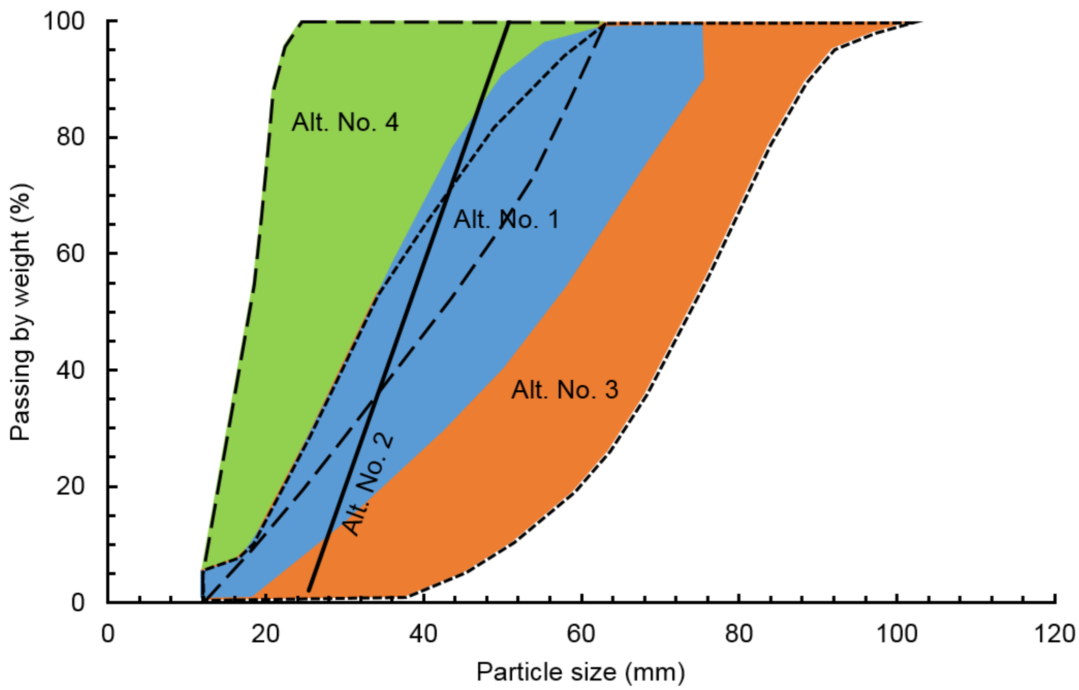


Figure 3.1: The extents of the grain sizes corresponding to the Alternative No. 1, No. 2, No. 3, and No. 4 gradations given by Barksdale and Bachus (1983).

Barksdale and Bachus (1983) recommend the gradation be sufficiently coarse to settle out rapidly. However, they warn that the gradation selected for a project may shift toward finer particle sizes when installing rockfill columns in very soft soils. They attributed this observation to the fact that vibratory methods can promote the intrusion of these finer materials into the void spaces of the backfill. To mitigate this intrusion, they suggest adopting finer gradations when constructing in softer soils. For soils with a shear strength less than 12 kPa, they recommend gradations on the finer limits of Alternative No. 2, No. 3, or No. 4. Sand can also be used in these cases. Bottom feed systems may be required when fine gradations are adopted (Barksdale & Bachus, 1983).

3.2.2 Typical friction angles corresponding to recommended backfills

As stated in §2.1.2 (Chapter 2), Charles and Watts (1980) cited typical friction angles of 37° to 45° for large rockfill embankment design, and Nicks and Adams cited a default friction angle of 34° for transportation agencies. The frictional range corresponding to the well-graded, angular quarry spalls and gravels for the compilation of granular material properties presented in Table 2.7 is 43.4° to 59.2° at 1 atm of confining pressure. The average density for these materials is 19.9 kN/m^3, which would equate to a depth of approximately 5.1 m. This depth is well within the typical range for granular shear keys. Applying a factor of safety of 1.3, the factored friction angles range from 33.4° to 45.5°. This is consistent with the range of friction angles used in design cited by Charles and Watts, and by Nicks and Adams.

3.2.3 Drainage considerations

The use of these recommended granular materials tends to result in improved drainage since they are more permeable than the native clay soils that are being replaced. The deep trench that forms the shear key acts as a groundwater interceptor trench (Denning, 1994), so the design of granular shear keys almost always incorporates subdrains (see Figure 3.2) located within the heel of the trench (Rogers, 1992). These subdrains conduct the water toward horizontal drains which discharge the water from the trench (shown in Figure 3.3). If groundwater is not permitted to exit from the shear key, a bathtub effect can develop which can weaken the surrounding soils by creating a deeper wetting area (Alberta Transportation, 2006). When it is anticipated that it will not be possible to fully drain the shear key, it becomes necessary to use the buoyant density of the rockfill in design calculations and models (Cornforth, 2005). The buoyant density will result in lower shear strength being achieved, and so, if disregarded, designs may be under-conservative. This consideration is particularly relevant for slope remediation works planned near sources of surface water or where the groundwater table is high.

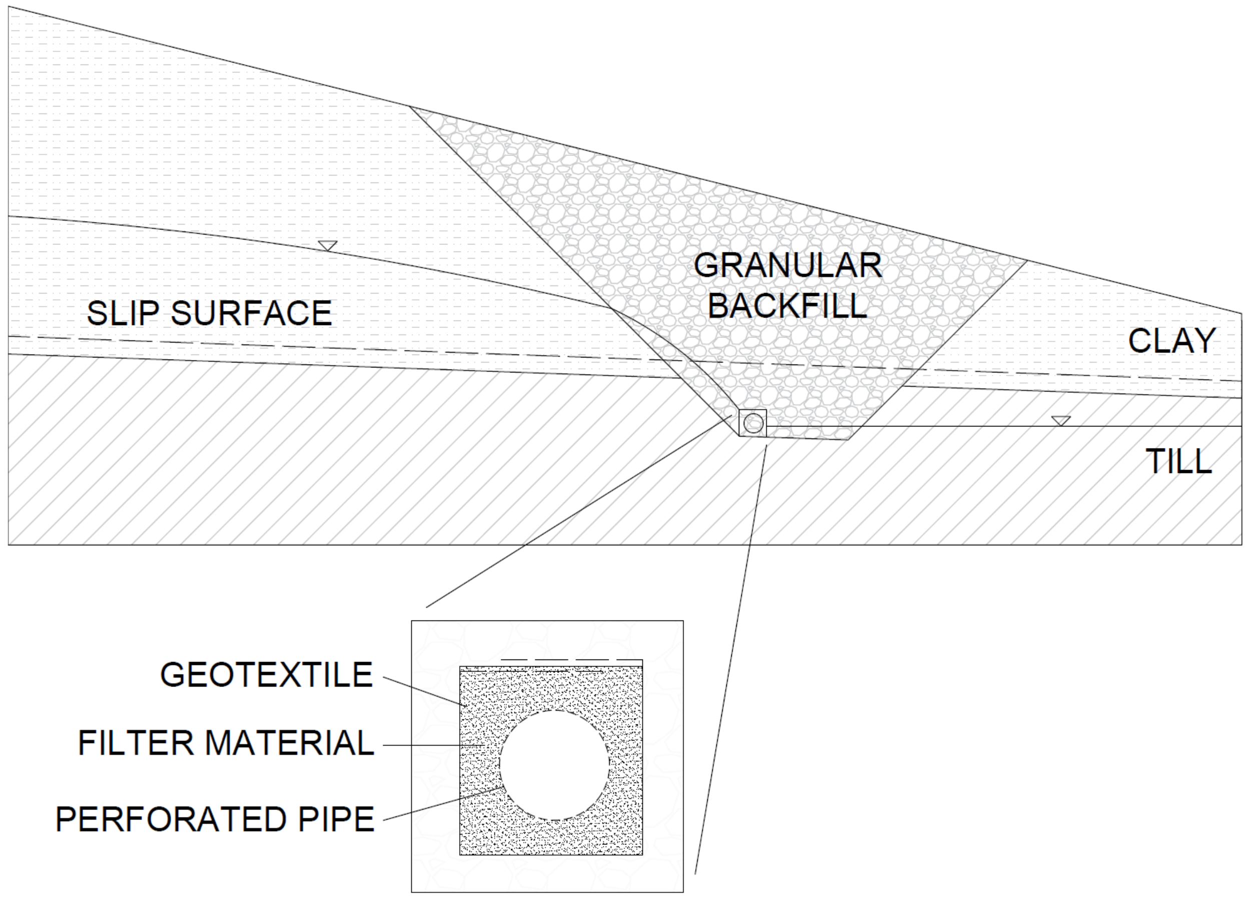


Figure 3.2: Schematic cross-section of a granular shear key with a subdrain located in the heel of the trench. The water table is shown getting drawn down to the elevation of the subdrain. The subdrain typically consists of a perforated pipe surrounded by filter material and a geotextile (shown enlarged). The subdrain outlet is not shown in this section.

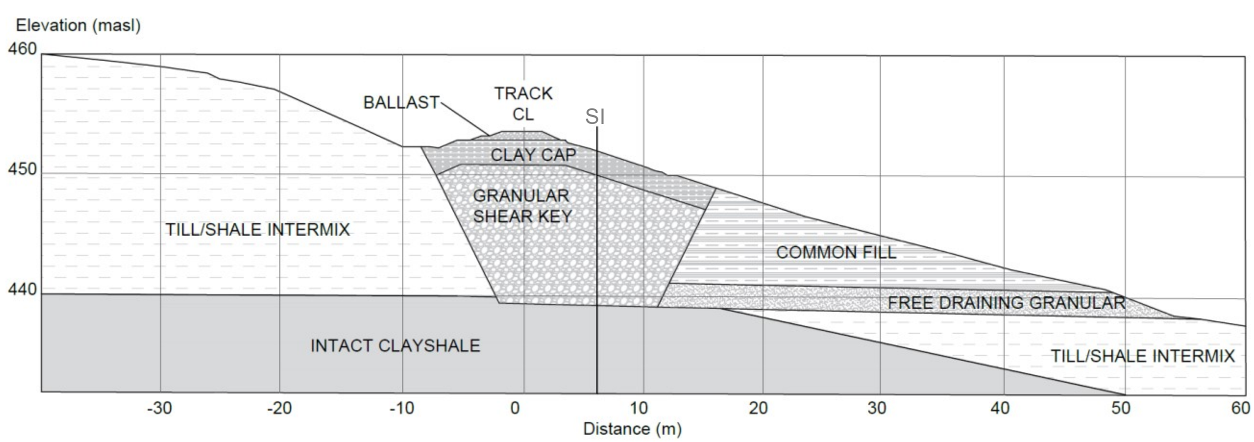


Figure 3.3: A horizontal drain shown in cross-section, indicated by the free-draining granular material. A slight gradient at the base of the shear key allows water to discharge via the horizontal drain.

3.2.4 Rockfill column-specific backfill considerations

For high quality, crushed rock to be used in rockfill columns, Barksdale and Bachus (1983) recommend the adoption of a maximum angle of internal friction between 42° and 45°. For gravels, they recommend an angle of 38° to 42°. The angle of internal friction that is used for design should also take into consideration the undrained shear strength of the native soil that is to be replaced. This is because the degree of improvement from rockfill columns relies on the strength of the composite soil (the rockfill and the soil between the columns). For example, with crushed rock, an angle of internal friction of 45° can be used for native soil undrained shear strengths exceeding 38 kPa, an angle of 42° might be adopted for undrained shear strengths between 10 to 24 kPa, and an even further reduced angle might be adopted for undrained shear strengths below 10 kPa (Barksdale & Bachus, 1983). This recommendation relates to the differential mobilization of shear resistance observed in composite soils, as introduced in §2.3.2 (Chapter 2).

3.3 Granular Shear Key Design and Stability Analyses

Once the LE model has been calibrated to the existing landslide conditions, a shear key is introduced and modified until the desired factor of safety is achieved (Denning, 1994). The typical factors of safety targeted for granular shear key design range from 1.3 to 1.5, as discussed at the end of Chapter 2. This approach may be inadequate in that the granular material must deform for shear resistance to mobilize, and the LE method cannot fully capture that process. Despite this potential shortcoming, LE stability analyses are the current standard of practice for the design of granular shear keys. This is not expected to change, so the following section will continue to outline the design process in the context of this type of analysis.

In some instances, 3D modelling may also be performed. This type of modelling can help the illustrate the effects the sides of a landslide and the sides of the shear key trench will have on stability. The sides are expected to provide additional support, so more economic designs can be produced by accounting for this. However, as the width of a slide becomes large relative to the slide thickness, the added benefit of considering the sides becomes small. This can considerably reduce the justification for 3D analyses.

3.3.1 Positioning the granular shear key within the slide

Varying the placement of a shear key within a slide can yield different results depending on the type of slide and the mechanism. The orientation of the principal stresses will rotate from the head to the toe of a slide and can be related to the inclination of the slip surface (see Figure 3.4).

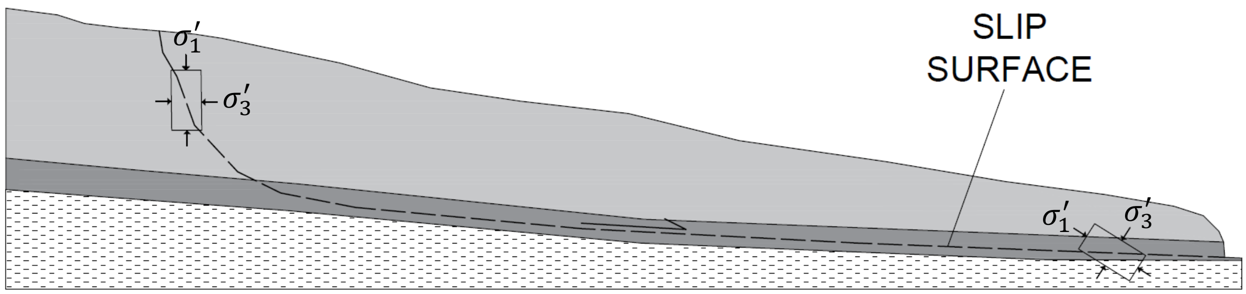


Figure 3.4: Schematic of a planar slide with the orientation of the principal stresses shown along the slip surface near the head and the toe of the slide.

Shear keys are typically placed near the toe of a slope. For a planar slide, the inclination of the slip surface in this region of the slope tends to be relatively shallow. For a rotational slide, the slip surface can be estimated as exiting at an inclination of approximately 30° from horizontal (Duncan et al., 2014). The major principal stress would be directed roughly toward the side of the shear key and the weight of the shear key would be aligned with the minor principal stress. This would be expected to increase the magnitude of the shear resistance that can be mobilized as this can be likened to increasing the confining stress in a triaxial compression test.

Some shear keys are placed further upslope rather than near the toe. The use of lightweight backfill can reduce the driving force acting on the portion of the slide downslope of the shear key. In these cases, the shear key is expected to also resist the portion of the slide that is upslope. By isolating the downslope section of a slide from the driving forces upslope, as depicted in Figure 3.5, movement in the downslope portion can be reduced (Cornforth, 2005). However, if downslope movement persists, the removal of passive confinement may impact the performance of the shear key. This may be a more pertinent consideration for the design of rigid structures, but should nevertheless be considered for shear keys built in the upper portion of a slide.

Caltrans (2014) describes shear keys as functioning in much the same way as toe berms except that shear keys are placed below fill embankment as opposed to adjacent to them. For rotational slides, it has been recommended that toe berms or buttress fills be placed such that the back of the fill intersects the slip surface where it is roughly tangent to the horizontal (Baker & Yoder, 1958). Granular shear keys are often incorporated with these structures and may benefit from similar placement.

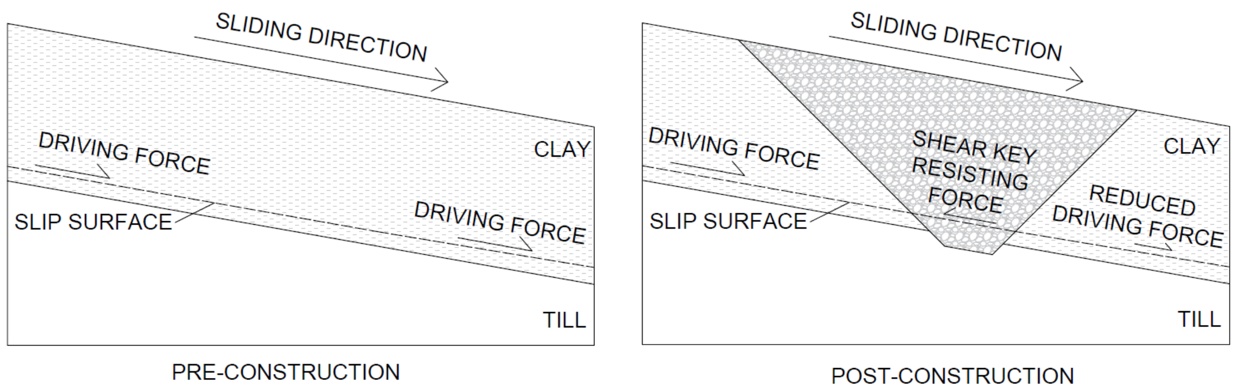


Figure 3.5: Schematic depiction of the effect of a granular shear key used to isolate the lower portion of a slide from the upslope portion. The shear key will resist the upslope driving force, and in doing so, reduce the driving force acting downslope.

After the site investigation, design engineers should also attempt to identify shallower parts of the landslide, where the excavation volume can be kept to a minimum (Denning, 1994). In line with this recommendation, Caltrans (2014) remarks they are frequently placed at the toe of embankments.

Other than offering a qualitative appreciation for how the shear key will affect the driving and resisting forces in a slide system, none of the consulted literature went into further detail or justification for the placement of shear keys. The tendency for their placement near the toes of slides appears to have a primarily economic justification. A quantifiable measure of how shear keys perform based on their location in a slide does not appear to exist. This may be due to the considerable number of variables that are affected by changing the location. For example, a shear key placed further upslope would intercept groundwater earlier. The slip surface will also likely be intercepted at a greater depth, increasing the normal effective stress acting on the slip surface. The weight of the shear key will contribute to the driving force though. Considering these factors, there potentially exists an optimum location for the shear key and it may not be at the toe.

3.3.2 Determining the size of the granular shear key

Once the model inputs have been selected and a preliminary target location for the remedial work has been established, the geometry of the granular shear key must be determined. The two dimensions that are typically altered are the width and the depth of the shear key. These dimensions are altered until the calculated factor of safety meets or exceeds the target factor of safety (Denning, 1994). While designers often cite the base depth and width in completion reports, it is the depth and width of the shear key at the elevation of the existing or potential slip surface that governs the magnitude of shear resistance that is provided. For this research, these were designated the *shear key effective depth* and the *shear key effective width*, respectively. These dimensions can be seen in the context of a shear key, in Figure 2.1, and are defined with other relevant design parameters in Table 2.1. Since increasing either dimension can positively affect stability, the aspect ratios of shear keys vary considerably depending on the conditions for which they were designed.

For initial calculations, it can be assumed the slip surface propagates as a flat plane across the shear key (Baker & Yoder, 1958). The base model that has already been established can be used to estimate the driving force and resistance force acting on the system. At this stage, it is assumed that the slide factor of safety (FOS) is at unity:

$$\mathrm{FOS} = 1.0 = \frac{\text{Resisting force}}{\text{Driving force}} \tag{3.1}$$

The mechanism is assumed to be a shear failure so the resisting force can be written as the shear force, V. To achieve the desired factor of safety, $\mathrm{FOS}_{\mathrm{desired}}$, the shear key must contribute extra shear force, V_{extra}. Thus, Equation 3.1 can be rewritten:

$$\mathrm{FOS}_{\mathrm{desired}} = \frac{V + V_{\mathrm{extra}}}{\text{Driving force}} \tag{3.2}$$

Rearranging Equation 3.2 to solve for the extra shear force that is required yields,

$$V_{\mathrm{extra}} = (\text{Driving Force} \cdot \mathrm{FOS}_{\mathrm{desired}}) - V \tag{3.3}$$

The shear force can be expressed in terms of the shear resistance, τ, and the area over which the resistance is acting, A, as follows:

$$V = \tau \cdot A \tag{3.4}$$

Thus, the extra shear resistance, τ_{extra}, and the area over which the shear key must span, A_{replaced}, can be used to express the extra shear resistance, using Equation 3.3 and Equation 3.4:

$$V_{\mathrm{extra}} = \tau_{\mathrm{extra}} \cdot A_{\mathrm{replaced}} \tag{3.5}$$

The extra shear resistance derives from the difference between the shear resistance of the backfill, τ_{backfill}, and the shear resistance of the native material, τ_{native},

$$\tau_{\mathrm{extra}} = \tau_{\mathrm{backfill}} - \tau_{\mathrm{native}} \tag{3.6}$$

Each can be modeled using the Mohr-Coulomb criterion, with both assumed to be cohesionless because of the granular nature of the backfill and the residual state of the native material.

$$\tau_{\mathrm{backfill}} = \sigma'_{\mathrm{n \cdot backfill}} \cdot \tan(\varphi'_{\mathrm{backfill}}) \tag{3.7a}$$

$$\tau_{\mathrm{native}} = \sigma'_{\mathrm{n \cdot native}} \cdot \tan(\varphi'_{\mathrm{native}}) \tag{3.7b}$$

At the depth of the slip surface, the normal effective stress, σ'_{n}, depends on the normal total stress, σ_{n}, and the pore water pressure, u_{w}. If the water table is at a height, h_{w}, above the slip surface, this can be expressed as,

$$\begin{aligned} \sigma'_{\mathrm{n}} &= \sigma_{\mathrm{n}} - u_{\mathrm{w}} \\ &= \gamma \cdot d_{\mathrm{e}} - \gamma_{\mathrm{w}} \cdot h_{\mathrm{w}} \end{aligned} \tag{3.8}$$

where γ is the unit weight of the soil, γ_{w} is the unit weight of water, and d_{e} is the effective depth of the slip surface.

The extra shear resistance can now be expressed in terms of Equations $3.7a$, $3.7b$, and 3.8:

$$\tau_{\mathrm{extra}} = (\gamma_{\mathrm{backfill}} \cdot d_{\mathrm{e}} - \gamma_{\mathrm{w}} \cdot h_{\mathrm{w}}) \cdot \tan(\varphi'_{\mathrm{backfill}}) - (\gamma_{\mathrm{native}} \cdot d_{\mathrm{e}} - \gamma_{\mathrm{w}} \cdot h_{\mathrm{w}}) \cdot \tan(\varphi'_{\mathrm{native}}) \tag{3.9}$$

where $\gamma_{\mathrm{backfill}}$ is the unit weight of the backfill and γ_{native} is the unit weight of the native soil being replaced.

From the design perspective, the part of this expression the engineer has most control over is the shear resistance provided by the backfill. Practically speaking, the backfill material is likely to be constrained to what is economically available at the site. The unit weight of this material can be increased somewhat through compaction. This then only leaves the engineer with the ability to further modify the normal effective stress by introducing a weighting berm or opting to push the location of the granular shear key further upslope. The angle of internal friction would be determined from direct shear tests under these loading conditions. Ideally, the engineer should aim to maximize the extra shear resistance in the most economic way possible.

Provided the most economic options for maximizing the extra shear resistance have been exhausted, the engineer can satisfy the required shear force by increasing the area over which this extra shear resistance will be applied. The area that will be replaced is dependent on the effective width of the granular shear key, the length of the shear key in plan view, the inclination of the slip surface relative to horizontal, and the angle of the trench walls from horizontal. Assuming a horizontal slip surface and vertical trench walls, and analyzing the stability of a granular shear key in cross section, this reduces to a sole dependency on the effective width, w_{e}. The extra shear force can thus be expressed as,

$$V_{\mathrm{extra}} = \tau_{\mathrm{extra}} \cdot w_{\mathrm{e}} \tag{3.10}$$

A quick trigonometric analysis can be performed to illustrate how the length of the slip surface is affected by changes to the trench wall angle and the inclination of the slip surface (see Figure 3.6). For this example, let the slip surface be inclined at 30° from the horizontal. It will cross a granular shear key with trench walls inclined at 45°. The length of this slip surface would be 50% longer than a horizontal slip surface measured from the upslope point of intersection with the trench. This calculation assumes the shear key has not deformed in response to shearing or from the motion of the landslide. It is also assumed the slip surface does not deviate upon passing into the shear key. As the angle of the trench walls nears vertical, and as the inclination of the slip surface approaches horizontal, the magnitude of this potential increase in the length of the slip surface decreases rapidly. This scenario therefore represents what could reasonably be assumed to be an upper limit for actual slip surface length. The effective width determined through Equation 3.10 can be considered the lower limit.

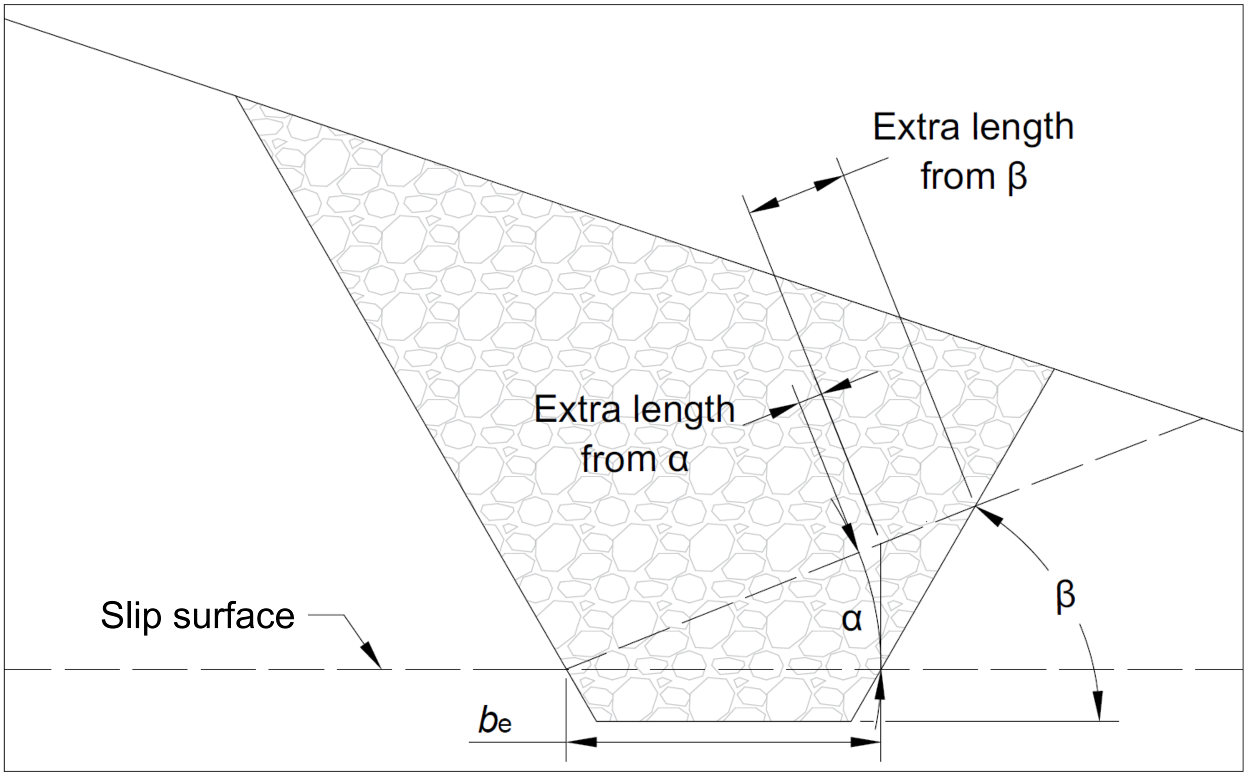


Figure 3.6: Schematic of the effect that the inclination of the slip surface, α, and the slope of the trench walls, β, will have on the length of the slip surface that passes through a granular shear key.

An inclined slip surface (i.e. $\alpha \neq 0$) will also affect the average depth of the slip surface; if $\alpha > 0$, the depth will decrease, and vice versa if $\alpha < 0$. This will impact the normal effective stress that can be used in the calculation for shear resistance. Thus, improved certainty regarding the path the slip surface will take would be beneficial to the design process.

The calculations discussed above can be performed easily and rapidly using 2D LEM software. However, it should be restated that this analysis requires that a representative cross section of a slide be selected. The unique shape of the landslide can result in significant variations in the shear resisting force from different cross sections. For this reason, it is recommended that the cross section producing the largest driving force be selected for analyses, so the lower limit for the granular shear key effective width can be established.

3.3.3 Extra considerations for the design of rockfill columns

While the design process presented thus far is largely the same for rockfill columns, several differences do exist. First, 2D models can be thought of as assuming the cross section that is being analyzed extends indefinitely into the plane. This is obviously not the case for a rockfill column represented in cross section, so there are methods to emulate the effect of having a row that alternatives between rockfill columns and native soil. These methods rely on what is called the area replacement ratio, A_{r}. The area replacement ratio can be expressed in terms of the plan area that the rockfill columns occupy, A_{rock}, and the total tributary area influenced by the columns, $A_{\mathrm{treatment}}$, as follows:

$$A_{\mathrm{r}} = \frac{A_{\mathrm{rock}}}{A_{\mathrm{treatment}}} \tag{3.11}$$

The tributary area is defined as the area of a regular hexagon about each column (Barksdale & Bachus, 1983). This is illustrated in Figure 3.7. Since trenched granular shear keys are continuous for the extent of the repair, the area replacement ratio would simply be 1.0.

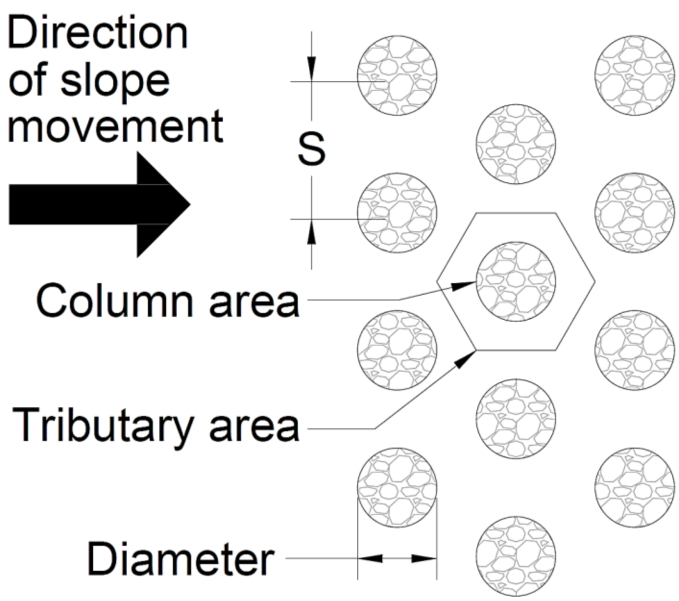


Figure 3.7: Schematic of three rows of rockfill columns in an equilateral triangular pattern. An example tributary area has been illustrated alongside the center-to-center column spacing, s, and the column diameter, d.

An alternative method of calculating the area replacement ratio is to use the diameter of the columns (d) and the center-to-center spacing (s) in an individual row, as shown:

$$A_{\mathrm{r}} = c\left(\frac{d}{s}\right)^2 \tag{3.12}$$

This approach also requires the input of a constant, c, which depends on the pattern or configuration of the columns. There are two commonly used patterns: equilateral triangular and square. By observation, the inter-row center-to-center column spacing i.e. in the direction of the slope, is equal to s when the square pattern is adopted, and $s \cdot \cos 30°$ when the equilateral triangular pattern is adopted (Goughnour et al., 1991). It follows, then, that for the equilateral triangular pattern, $c = \pi/4$, and for the square pattern, $c = \pi/(2\sqrt{3})$ (Barksdale & Bachus, 1983).

Abdul Razaq (2007) investigated the effect different area replacement ratios had on the mobilization and strength of rockfill columns using large-scale direct shear tests. It was found that increases to the area replacement ratio resulted in greater mobilized shear resistance but did not affect the shear stiffness of the system. With the maximum area replacement ratio of 1.0 effectively representing a trenched shear key, this suggests the shear resistance from rockfill columns will require the same amount of strain as trenched shear keys to become mobilized.

The area replacement ratio can be used to account for the native soil between each column in a row in several different ways. Two popular methods are to use composite soil-backfill material properties, or to represent the rockfill columns as strips with an equivalent width (Barksdale & Bachus, 1983). The procedures are summarized below but can be found in their entirety in Barksdale and Bachus (1983) or Goughnour et al. (1991).

The use of composite soil-backfill material properties is referred to as the average shear strength method. In this method, a weighted average for the material properties is calculated. The weighted averages of the shear strength parameters can be calculated with the area replacement ratio, as shown in Equations 3.13a, 3.13b and 3.13c for the cohesion, friction angle, and unit weight, respectively (Goughnour et al., 1991).

$$c_{\mathrm{average}} = c_{\mathrm{soil}}(1 - A_{\mathrm{r}}) \tag{3.13a}$$

$$\tan(\varphi_{\mathrm{average}}) = \frac{\tan(\varphi_{\mathrm{rock}})\, A_{\mathrm{r}} S_{\mathrm{r}} + \tan(\varphi_{\mathrm{soil}})\,(1 - A_{\mathrm{r}})}{1 + A_{\mathrm{r}}(S_{\mathrm{r}} - 1)} \tag{3.13b}$$

$$\gamma_{\mathrm{average}} = \gamma_{\mathrm{rock}} A_{\mathrm{r}} + \gamma_{\mathrm{soil}}(1 - A_{\mathrm{r}}) \tag{3.13c}$$

In Equation 3.13b and 3.13c, S_{r} is the stress ratio which accounts for the orientation of the slip surface, α, relative to horizontal. It is calculated as,

$$S_{\mathrm{r}} = 1 + (S_{\mathrm{rv}} - 1)\cos(\alpha) \tag{3.14}$$

where S_{rv} is the ratio of the vertical stress in the column, σ_{s}, to the vertical stress in the surrounding soil, σ_{soil}.

$$S_{\mathrm{rv}} = \frac{\sigma_{\mathrm{s}}}{\sigma_{\mathrm{soil}}} \tag{3.15}$$

The rockfill columns can then be drawn into the modelled cross section as strips of width d using the average properties (Figure 3.8). Goughnour et al. (1991) note that φ_{soil} can be set to 0 to model end-of-construction conditions, or c_{soil} can be set to 0 for long-term conditions.

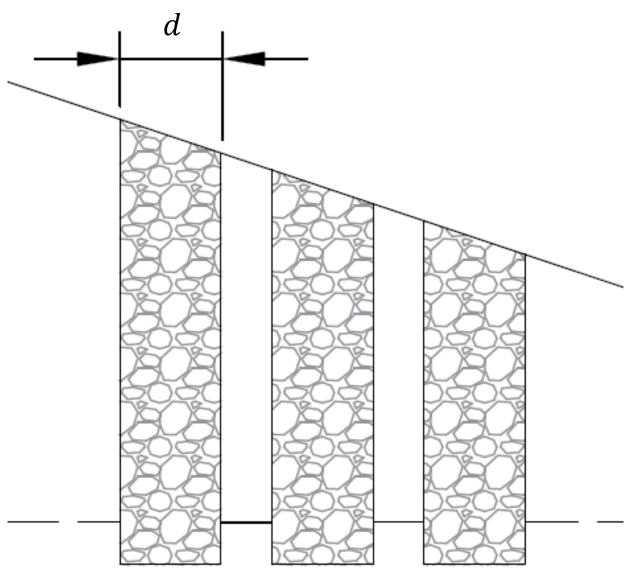


Figure 3.8: Schematic of three rows of rockfill columns in cross section, for modelling using the average shear strength method.

The representation of rockfill columns as strips of an equivalent width is known as the profile method. These strips are modelled as being continuous but are given a width, w, that yields the same volume of backfill as each row of rockfill columns. The width can be calculated as the plan area per rockfill column, A_{rock}, divided by the center-to-center spacing between them.

$$\begin{aligned} w &= \frac{A_{\mathrm{rock}}}{s} \\ &= \frac{\pi d^2}{4s} \end{aligned} \tag{3.16}$$

The width of the soil strips, w_{soil}, between each row can also be calculated using Equation 3.17.

$$w_{\mathrm{soil}} = \frac{A_{\mathrm{rock}}}{A_{\mathrm{r}} s} \tag{3.17}$$

A last consideration is the development of stress concentration in the rockfill columns. Stress concentration is a result of the stiffness contrast between the columns and the surrounding soil, which results in increased shear resistance in the columns, under the equal strain assumption (Barksdale & Bachus, 1983). This increase in shear resistance can be taken advantage of in the interest of improving the economy of rockfill column designs. The stress concentration is accounted for when calculating the effective vertical stress acting along the centerline of a column. The effective vertical stress along the centerline of a column, σ_{s}, is a combination of the weight of the column and the applied loading, which can be expressed as,

$$\sigma_{\mathrm{s}} = \gamma_{\mathrm{rock}} z + \sigma \mu_{\mathrm{s}} \tag{3.18}$$

where z is the depth below ground surface, σ is the added vertical stress from an embankment or weighting berm, and μ_{s} is the stress concentration factor (Goughnour et al., 1991).

The stress concentration factor for a column can be calculated using:

$$\mu_{\mathrm{s}} = \frac{S_{\mathrm{rv}}}{1 + (S_{\mathrm{rv}} - 1) A_{\mathrm{r}}} \tag{3.19}$$

For verification by hand, each of these methods can be used to quickly calculate the theoretical degree of improvement made to a slope by the rockfill columns. The difference between the shear strengths of the material representing the columns and the native material can be multiplied by the length of the potential slip surface crossing through the representations of the columns in cross section. If the average shear strength method is used, the shear strength of the native soil should be subtracted from that of the composite material, and if the profile method is used, the shear strength of the native soil should be subtracted from that of the backfill material.

A full-scale study of the degree of improvement using rockfill columns was performed by Thiessen et al. (2007). The conclusions are numerous and can be consulted in detail in Thiessen (2010).

3.3.4 Additional stability analyses

The preceding analyses focus on the factor of safety for slip surfaces passing completely through the granular shear key. Denning (1994) suggests analyses for active and passive wedges exiting from and upslope of the shear key also be performed (see Figure 3.9). Per soil mechanics theory,

- the angle from vertical at which a passive wedge will daylight is 45° plus half of the angle of internal friction of the material through which it passes, and

- the angle from vertical at which an active wedge will daylight is 45° minus half of the angle of internal friction of the material through which it passes.

Passive wedge scenarios should consider the potential for slip surfaces to daylight through or upslope of the shear key. Using the finite element method, a passive wedge daylighting upslope of a modelled shear key was identified by displaying the deformed boundaries and displacement vectors in a model (Figure 3.10). Active wedge scenarios should consider the possibility of slip surfaces daylighting through or downslope of the shear key.

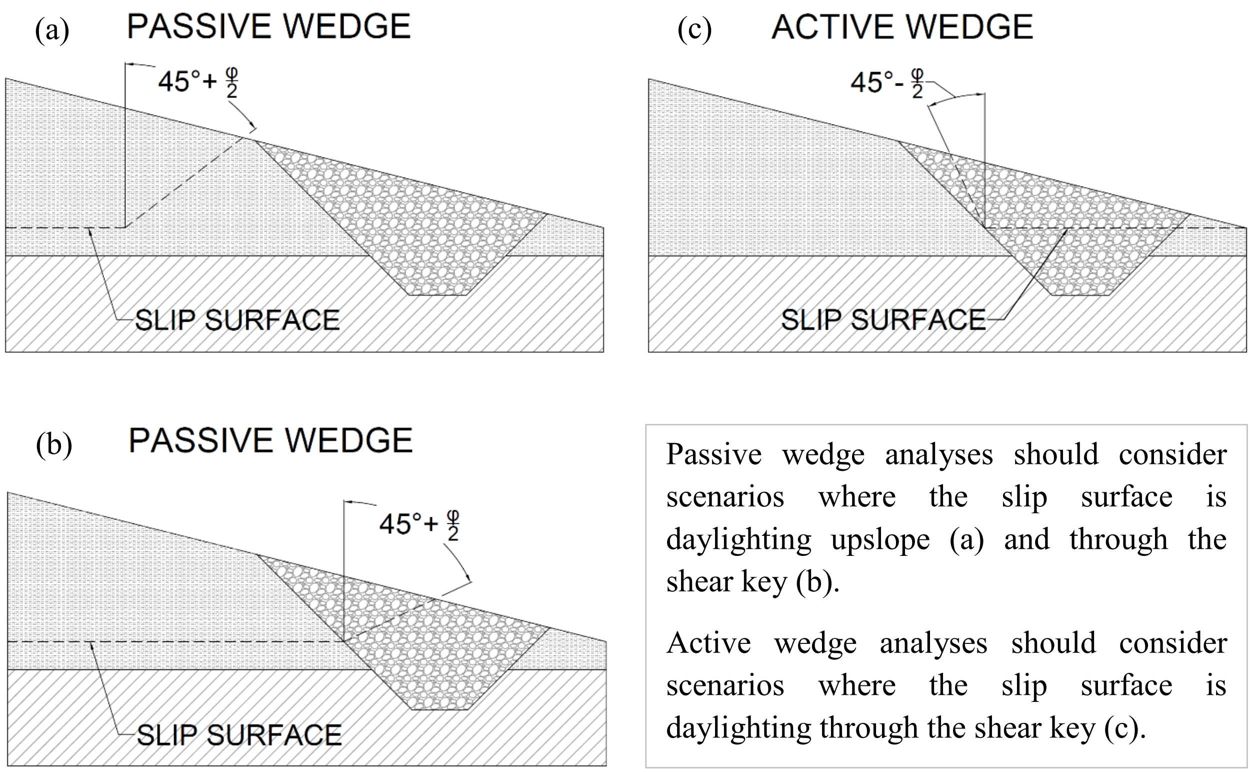


Figure 3.9: Schematics showing conceptual passive and active wedges in relation to a granular shear key. Adapted from Denning (1994) with permission.

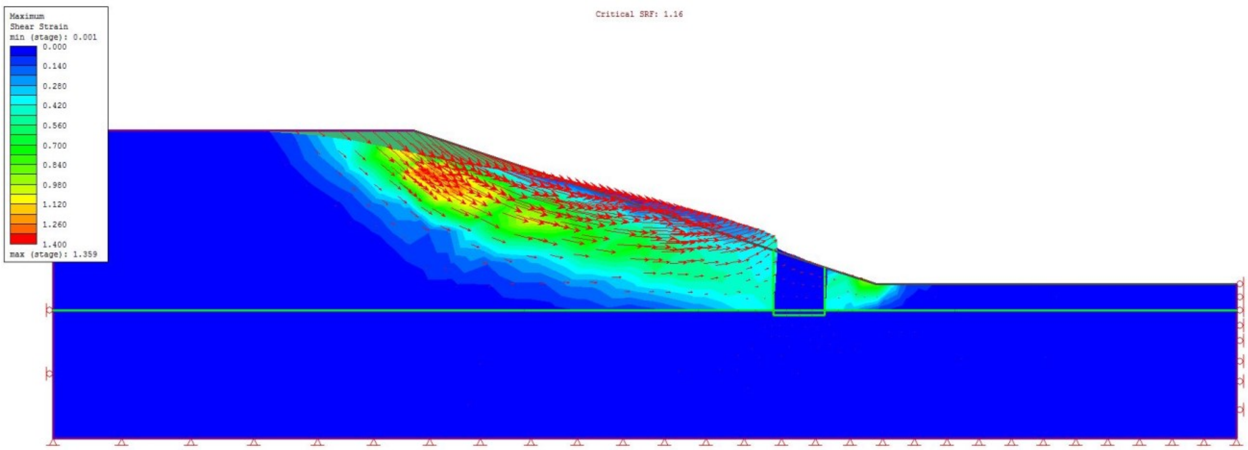


Figure 3.10: Depiction of a passive wedge overriding a granular shear key at the toe of a slide using the finite element method.

3.3.5 Trenched shear key dimensions recommended in literature

The two preceding sections discussed and outlined the analytical process of determining the size and stability of a granular shear key. While the results of this process are certainly needed, there remain practical limitations to what can be constructed. The following section discusses practical geometric limits and design recommendations for granular shear keys, collected from literature. The subject matter includes the angle of the trench walls, the depths of excavation, the distance to excavate below an identified slip surface, and typical ranges for the width of trenched shear keys. The recommendations for these dimensions are presented in Table 3.3 alongside their respective sources. The ranges are summarized in Figure 3.11 with the labelled dimensions shown on a schematic of a trenched shear key.

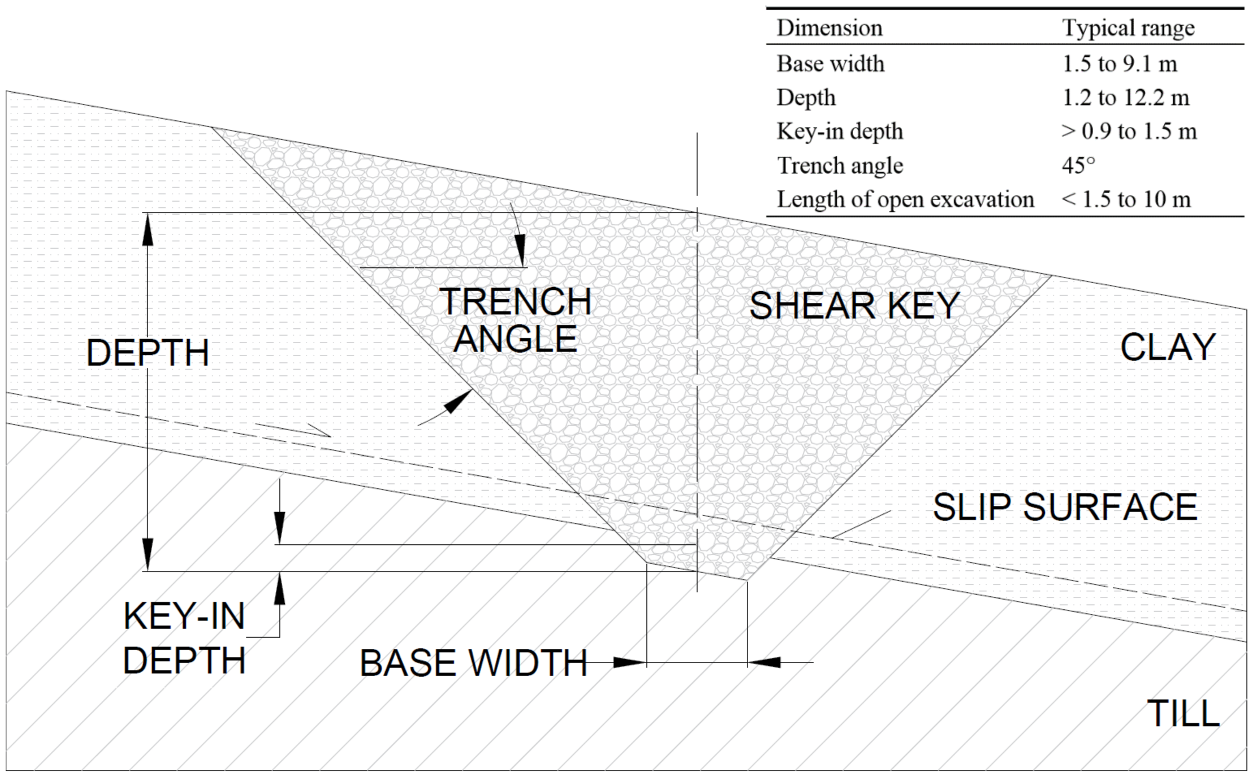
The slope of the trench walls is often designed to be as steep as possible, to maximize the normal stress acting along the base for a given volume of rockfill. This design consideration also reduces the surface footprint, excavation cost, and backfill cost. Nevertheless, the trench walls are usually sloped at about 45° or flatter (Cornforth, 2005; Proudfoot D. , 2016). This may be because of practical reasons, such as an aversion to risk or insufficient stand-up time for construction.

Per Denning (1994), the Oregon Department of Transportation limits their shear key excavations to a maximum depth of 40 feet (~12.2 m), citing increased construction difficulty and very massive volumes when permitted to go deeper. The California State Department of Transportation reports that shear keys tend to be excavated 4 to 10 feet (1.2 to 3.0 m) below ground surface (Caltrans, 2014). Cornforth (2005) states the most common application of shear keys for slope stabilization is where the slip zone is between 10 to 30 feet (3.0 to 9.1 m) deep in the section of the slope to undergo excavation. He warns that the added benefit of the rockfill can be minimal when adopting a shear key for slides shallower than 10 feet (3.0 m), and that there is even a risk of the slip surface daylighting upslope of the shear key. Cornforth (2005) recommends that rockfill extend at least 12 feet (3.7 m) above discrete slip surfaces. On the other hand, for slip surfaces greater than 30 feet (9.1 m) deep, it is the volume of the shear key that makes the project prohibitively expensive when compared to alternatives. He adds that deeper trenches also impose a greater risk of upslope instability during excavation.

Table 3.3: Summary of typical and suggested ranges of trenched shear key dimensions.

Dimension	Typical range	Reference
Base width	1.5 to 4.6 m	Caltrans (2014)
	4.6 to 9.1 m	Cornforth (2005)
	4 to 6 m	Yarechewski and Tallin (2003)
Depth	1.2 to 3.0 m	Caltrans (2014)
	3.0 to 9.1 m	Cornforth (2005)
	< 12.2 m	Denning (1994)
Key-in depth	> 0.9 to 1.5 m	Cornforth (2005)
	> 1.5 m	Denning (1994)
Trench angle	45°	Cornforth (2005), Proudfoot (2016)
Length of open excavation	< 6.1 m	Denning (1994)
	< 6 to 10 m	Proudfoot (2016)
	< 1.5 m	Sills and Fleming (1992)

Trenched shear keys are stated to be most effective where the structure can be excavated into a stronger material than the overlying soils (Caltrans, 2014). Cornforth (2005) recommends extending a shear key at least 3 to 5 feet (0.9 to 1.5 m) below the slip surface to prevent new surfaces from forming immediately below the shear key. Per Denning (1994), the Oregon Department of Transportation recommends that shear keys extend a minimum of 5’ (1.5 m) below the slip surface to help ensure that potential slip surfaces beneath the shear key are made adequately long and forced through stronger material. While these given ranges are useful starting points, Cornforth (2005) points out the key-in depth ultimately depends on the type of material into which the shear key is being keyed. For example, Cornforth remarks that the presence of hard bedrock could permit keying in by only several inches.



Dimension	Typical range
Base width	1.5 to 9.1 m
Depth	1.2 to 12.2 m
Key-in depth	> 0.9 to 1.5 m
Trench angle	45°
Length of open excavation	< 1.5 to 10 m

Figure 3.11: Schematic of a trenched shear key with labelled dimensions. Recommended ranges for the dimensions are shown in the inset table.

The width of a trenched shear key is usually reported in terms of the base of the excavation. Cornforth (2005) reports typical base widths range from 15 to 30 feet (4.6 to 9.1 m). Caltrans (2014) gives a typical range of 5 to 15 feet (1.5 to 4.6 m), although does not specify the location of this measurement.

Denning (1994) reports that the Oregon Department of Transportation suggests limiting the length of open trench to no more than 20 feet (6.1 m) at the base of the excavation. A practitioner in Alberta, Canada suggested a maximum open trench length of 6 to 10 m. This length should be adjusted based on site-specific conditions such that the risk of instability (local and global) is acceptable (Proudfoot D. , 2016). For the construction of a similar remedial technique using stone-fill trenches, the US Army Engineer District in Vicksburg limited the length of open trench at any time to less than 5 feet (1.5 m) (Sills & Fleming, 1992).

It should be noted that no recommendations were encountered regarding how far to extend a shear key beyond the lateral extents of a landslide. It is likely this is not of significant consequence since the shear key primarily relies on being excavated deeper than the slip surface in order to ensure it does not simply become part of the existing landslide.

Rockfill column shear key recommendations

With rockfill column shear keys, recommendations and typical ranges have been documented for the area replacement ratio, spacing, stress concentration factor, diameter, key-in distance and depth. These dimensions are illustrated in Figure 3.12 and the recommendations from literature are summarized in Table 3.4.

Table 3.4: Summary of recommended or typical ranges of rockfill column-specific dimensions and parameters.

Dimension	Typical range	Source
Diameter	0.7 to 1.3 m[2]	Barksdale and Bachus (1983)
	2 to 3 m	Yarechewski and Tallin (2003)
Depth	4 to 10 m	Barksdale and Bachus (1983)
	< 27.4 m	Cornforth (2005)
Key-in depth	0.6 to 0.9 m[3]	Cornforth (2005)
	1.8 to 3.0 m[4]	Cornforth (2005)
Area replacement ratio	0.15 to 0.35	Barksdale and Bachus (1983)
Spacing	2.1 to 2.4 m	Barksdale & Bachus (1983)
	1.8 to 3.7 m	Cornforth (2005)
Stress concentration factor	2 to 2.5	Barksdale & Bachus (1983)

Barksdale and Bachus (1983) report that the equilateral triangular pattern is more commonly used than the square pattern. They also recommend using a spacing greater than 1.5 m when the wet method (vibro-replacement using jetting water) is employed. This recommendation is likely to reduce the risk of interacting with adjacent columns or holes during the construction process.

Abdul Razaq (2007) compiled several recommendations to help ensure the effectiveness of rockfill columns. He states they must be of large diameter and be relatively stiff to generate adequately large stabilizing forces. He also found that to increase the factor of safety, the columns had to

[2] Recommendation is for columns intended to resist compressional loads in foundations.

[3] In the presence of hard bedrock.

[4] In the event the clay extends beyond the slip surface and bedrock or another competent unit is not nearby.

extend below a slip surface by a distance that was large enough to ensure the slip surface could not easily pass below the columns. Lastly, he found that rockfill columns should be placed near the centre of the slip surface in cross section to limit the ability of a slip surface to daylight up or downslope of the columns.

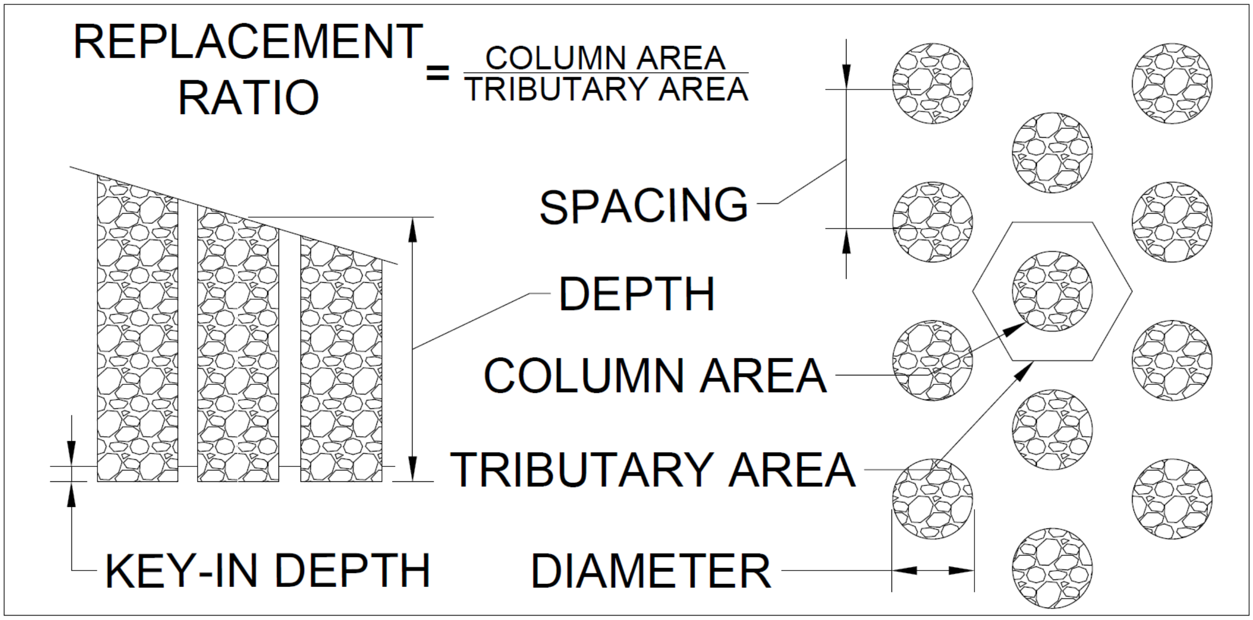


Figure 3.12: Key dimensions relating to rockfill column shear keys are labelled in cross-section (left) and plan view (right).

3.4 Constructing a Granular Shear Key

The construction of a shear key temporarily exposes the slope to additional risk. The trench that is excavated during construction reduces stability because the resisting force provided by the native soil is being removed. During the construction phase, considerations include site logistics, the time of year, temporary stability and the excavation sequence, and the placement of the backfill.

3.4.1 Site logistics

There exist several different techniques and approaches to constructing granular shear keys but common to each of them is the fact that the site must be accessible to the machinery being used to carry out the work. Denning (1994) recommends identifying flatter areas of the slide, which can be used as an access way for the rest of the slide. A suitable area for stockpiling the excavated material and the backfill prior to placement should also be identified at that time. This area must not be located directly upslope of the planned excavation so as not to further destabilize the slope.

3.4.2 Selecting a time of year for construction

Since the excavation of a shear key temporarily reduces the stability of a slide, it is recommended that construction take place during periods when the groundwater level is lowest so that the slope stability is at a seasonal high (Denning, 1994; Cornforth, 2007). Dewatering is also required if construction will extend into sands and silts below the water table (Cornforth, 2005). Per Proudfoot (2016), granular shear key construction in Alberta, Canada is ideally undertaken between the late summer to fall. The below-freezing temperatures experienced during winter in Alberta is known to make for poor compaction conditions. On the other hand, it is preferred to undertake shear key construction in the second half of winter in Winnipeg, Manitoba, Canada. This is due to the locally-controlled river levels having been lowered in the fall, increasing the area that is accessible along the riverbanks there. The ground is also frozen at that time, providing a firmer footing for machinery (Kenyon, 2016).

3.4.3 Temporary stability of the slope and trench during excavation

A common error in landslide remediation projects identified by Cornforth (2007) is allowing a contractor to remove support from a landslide for extended periods during construction. Either bracing of the trench or adopting a closely-sequenced excavation is often recommended to limit the length of time this support is removed (Abramson et al., 2002, Cornforth, 2005). A closely-sequenced excavation consists of excavating the trench in segments (see 'Length of open excavation' in Table 3.3) which are then backfilled before the trench is advanced. The closely-sequenced excavation technique is depicted schematically in Figure 3.13.

These techniques are adopted to minimize the reduction to slope stability while the shear key excavation is open. Nevertheless, the landslide is expected to accelerate while the excavation is taking place. Slope inclinometers can be used to monitor this deformation (Figure 3.14), although it is not uncommon for these instruments to be destroyed because of these deformations. It is also sometimes possible during the excavation for the slip surface to be observed, such as in Figure 3.15. This visual confirmation of the slip surface can be useful for design verification and can help to address one of the common errors identified by Cornforth (2007), which is the incorrect identification of the depth of sliding.

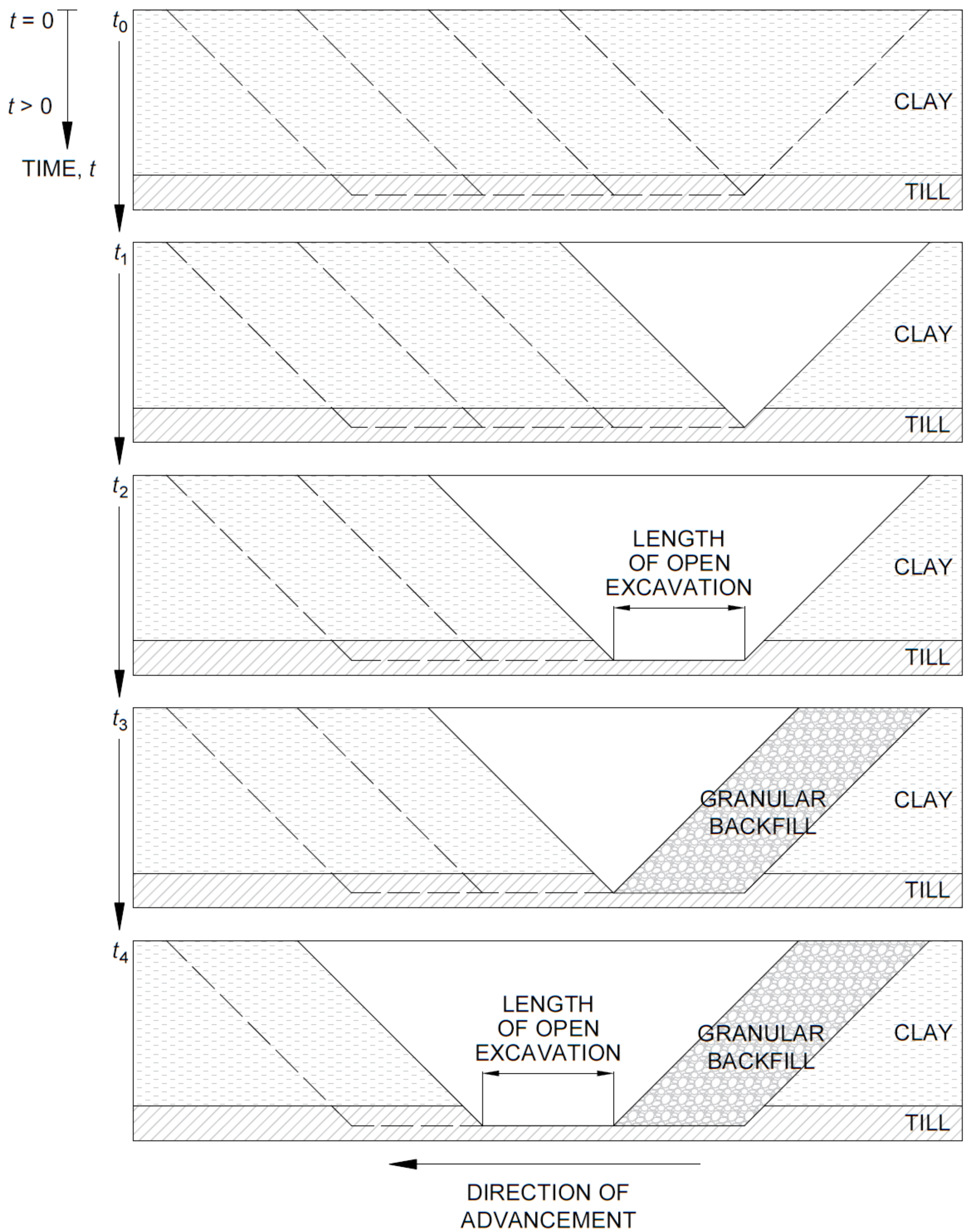


Figure 3.13: Schematic of a granular shear key cross-section, illustrating the closely-sequenced excavation method.

The length of open trench should be adjusted as necessary, depending on site conditions and observed slope movement during construction. During this period of increased risk, some organizations opt for shiftwork such that work can proceed nonstop until the full length of the shear key has been completed (Schutta, 2017). When this is not the case, it is recommended that the trench be backfilled by the end of the work day (Denning, 1994).

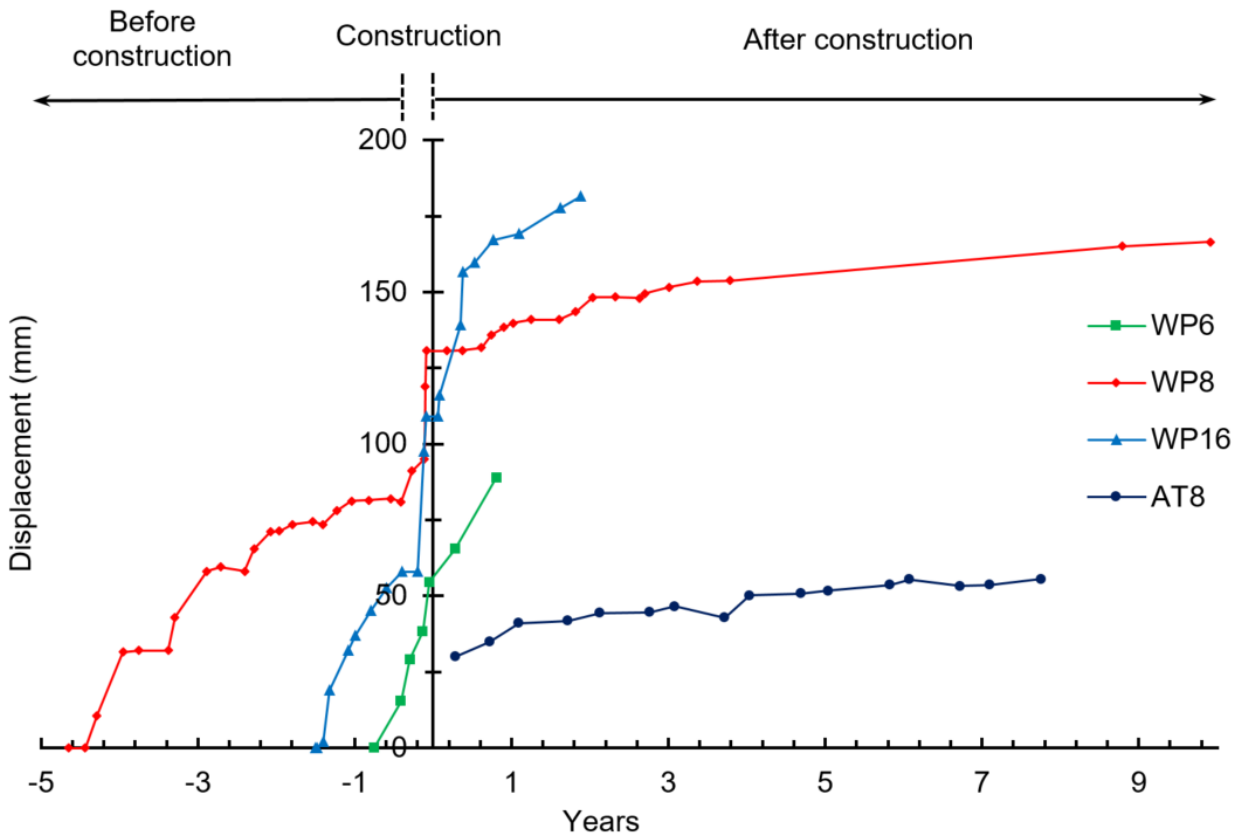


Figure 3.14: Slope inclinometer data shows the cumulative displacement measured at three different sites over time. In all three cases, the rate of displacement increases while construction is taking place.

For deep shear keys, a bench may have to be constructed to access the base of the excavation (Denning, 1994). The use of a bench reduces the time the full height of the trench is left free-standing and sloughing of the trench walls can be mitigated. This can also help reduce the destabilizing effect the excavation has on the slope itself.

Although less common, there have been cases where the slope was pre-stabilized by constructing supplementary earthworks or retaining structures prior to the construction of the granular shear key. In one case, a caisson wall was constructed to support the upslope side of the shear key excavation (Reuter & Kwasny, 2013). In another, a series of granular ribs was constructed upslope of the planned location for the shear key (Yarechewski & Tallin, 2003). Cornforth (2007) recommends implementing drainage both before and during construction, to enhance stability during this critical phase.

Figure 3.15: A slip surface can be seen during panelled construction of a granular shear key (McLachlan, 2016). The direction of sliding is indicated by a black arrow. Photo by Duncan McLachlan. Used with permission.

Finally, these temporary excavations are susceptible to typical excavation-related movements which are not necessarily associated with landslides. These can include movements from consolidation, movements at the excavation base in the form of base heave, or movements along the walls due to sloughing (Canadian Geotechnical Society, 2006). Shear key excavations are also susceptible to artesian groundwater pressure and exposure to weathering if left for too long (Cornforth, 2005). To help mitigate these hazards and to improve stability in general prior to construction, drainage measures can be implemented before the excavation of the shear key begins.

3.4.4 Backfill placement and compaction

When placing the granular backfill, it is typically set down in lifts which are then compacted (see Figure 3.16). This compaction increases the relative density of the backfill material, which enhances its shear strength by increasing the dilation that it undergoes during shear. By compacting the granular backfill, the normal effective stress also increases because of the corresponding increase in density.

The compaction of rockfill is increasingly being accomplished using a vibroflot, a long vibratory lance that is inserted into rockfill and then slowly extruded as compaction takes place (Barksdale & Bachus, 1983). This technique is especially used for rockfill columns.

Figure 3.16: Shear key backfill being compacted with a vibratory plate. Reproduced from Yarechewski and Tallin (2003), with permission.

3.4.5 Construction techniques unique to rockfill columns

In foundation reinforcement projects, vibro-replacement using jetting water (i.e. the wet process) or vibro-displacement without the use of jetting water (i.e. the dry process) are used to install rockfill columns. The construction of rockfill columns for landslide mitigation is typically achieved by means of a large-diameter auger to pre-bore the columns (see Figure 3.17). Barksdale and Bachus (1983) remark that this approach is common in the presence of stiff soils or stiff lenses. If the soil is prone to sloughing, a steel sleeve can be used to assist with construction or individual holes can be relocated (Yarechewski & Tallin, 2003). Once bored, the hole can be backfilled with granular material.

Figure 3.17: Pictured is a 2.13 m diameter auger in the process of lifting soil from a hole that is being excavated for a rockfill column. Photo by Hugh Gillen.

Barksdale and Bachus (1983) report that compaction of the granular backfill is typically accomplished using a vibratory probe. The vibratory probe was originally developed in 1935 for the compaction of granular, noncohesive soil. Contractors offering compaction using this instrument have various names for it but it has been referred to in the literature as a vibroflot or a poker. Lateral vibration is conveyed from the tip of the instrument. For large-diameter (~2.1 m) rockfill columns, the probe is typically advanced into the rockfill after it has been placed by means of jetting with large quantities of water under high pressure. The probe itself typically measures 12 to 18 inches (300 to 460 mm) in diameter (Barksdale & Bachus, 1983). Thiessen (2010) recommends compacting the top of rockfill columns with a hoe pack as the vibroflot does not compact very effectively at shallower depths.

Compaction testing of rockfill columns was conducted by UMA Engineering Ltd. in 2001 as reported by Thiessen et al. (2007). The testing was done using 100 mm down crushed limestone filling a 10-m deep, steel-sleeved hole. The density of the rockfill was found to increase from 1712 kg/m^3 to 2219 kg/m^3. However, without the use of a sleeve, it was found that a volume of rockfill 3% greater was required to fill the same hole to achieve that same density. The implication of this observation is that the cross-sectional area of the column is 3% greater than the drilled cross-sectional area for similar conditions, per Tallin (2001). In evaluating the stability of a reinforced slope, disregarding this expansion of the reinforced cross-sectional area would likely result in designs being slightly more conservative.

It was stated in §2.3.3 that the use of rockfill columns is sometimes favoured over trenched shear keys because of the reduced destabilizing effect the construction process has on the slope. While the size of the excavation at any given time is smaller for a rockfill column project than for a trenched shear key project, the time it takes to complete the process of excavating and backfilling should also be considered. Yarechewski and Tallin (2003) noted that the construction of trenched shear keys and ribs can be completed in nearly half the time that rockfill columns can be constructed. They also remark that the rockfill column construction process results in the clay between the columns becoming disturbed. Thiessen (2010) found this clay displaced as a continuum with the columns when subjected to loading under a test embankment, but shallow displacements greater in the clay than in the columns developed following construction. However, no conclusion was reached as to whether this process was related to the columns.

Lastly, field tests are sometimes performed to verify the strength parameters used in the stability analyses. When field testing for the angle of internal friction of rockfill columns, Barksdale and Bachus (1983) recommend using a double ring, direct shear test to prevent local bearing failure from occurring. When this type of failure is sustained, the test can yield lower strength values than can be achieved. They note that the test adds little value though, provided the construction was performed competently and adhered to accepted practices. It can also be substituted for large-scale triaxial testing. They do, however, recommend examining the general appearance, gradation and fouling of the columns, and performing several density tests.

3.5 Expected Timeframe for the Persistence of Slope Movement

The surface of a landslide can be expected to continue moving for 2 to 4 years after remediation, while the granular backfill shear strength becomes mobilized (Denning, 1994). In Winnipeg, Manitoba, Canada, granular shear keys were first adopted in the early 1990's, with initial estimates for movement to stop after 1 to 3 years. However, this was not widely observed, with changing stress states being cited as a likely reason for continued movements (Kenyon, 2016). These changing stress states can be attributed to fluctuations in the groundwater level, or the removal of material at the toe of a slide. A granular shear key will initially mobilize enough shear resistance to bring the slide to a state of equilibrium. If the shear resistance required for equilibrium increases, the shear key will continue to mobilize, resulting in strain. The time-dependent behaviour of the soils comprising the slope may also result in creep-related strain.

If the failed soil of the existing landslide mass has not been completely replaced, voids that formed in the active wedge prior to remediation may surface over time. These cracks will heal gradually as the soil rearranges itself, resulting in additional long-term deformations (Cornforth, 2005; Denning, 1994). Denning (1994) recommends these open surface cracks be filled during post-construction maintenance so water infiltration can be reduced.

Abdul Razaq (2007) cites a need to reduce the uncertainty with regards to the magnitude of deformation that must be sustained before shearing resistance has been sufficiently mobilized to resist landslide movements. He postulates the reason for this uncertainty is partly due to the reliance on limit equilibrium analyses for designing these remedial works. Barksdale & Bachus (1983) also state a need for more documentation of the performance of rockfill columns in the field. Thiessen (2010) performed a full-scale test of the short-term degree of improvement provided by rockfill columns subjected to a test embankment but movement was observed to be ongoing afterward. Long-term deformation behaviour of rockfill columns including the contribution from creep would likely be of great benefit to practitioners.

In comparing rockfill columns to trenched shear keys, Yarechewski & Tallin (2003) postulate that shear resistance might be able to mobilize at lower strains for trenched shear keys. This is due to the fact that the trench acts as a continuous unit as opposed to many isolated units, as is the case with rockfill columns. On the other hand, Abdul Razaq (2007) found after testing scale models in a large-scale direct shear box that rockfill ribs (with long axis oriented parallel to direction of shear) were able to mobilize shear resistance at significantly lower strains than a shear key.

3.6 Summary of Literature-Based Design Recommendations

In this chapter, the existing literature on granular shear key design was reviewed. Very few comprehensive guides were identified for the conventional trenched-style of granular shear key. Those located for rockfill columns tended to focus on compressional loading applications. The following is a step-by-step summary of the process recommended in the literature for granular shear key design, construction, and post-construction monitoring and maintenance. This process is depicted as a flowchart in Figure 3.18.

Phase 1: Establish a base model

- Step 1: Establish the slide geometry and the depth of the slip surface
    a. Slope inclinometers should be used and should be monitored through construction.
- Step 2: Establish the shear strength of the slip surface and ground water conditions
    a. Back-analysis is considered the best method for establishing the residual properties of the slip surface.
    b. Groundwater conditions can be established using piezometer readings but the seasonality of groundwater levels should be taken into consideration.

Phase 2: Select a backfill material

- Step 3: Source the available granular materials
    a. The ideal backfill material that is recommended for use in granular shear keys is a coarse grade of hard, strong and angular free-draining gravel.
- Step 4: Establish the shear strength of the backfill in compacted and loose conditions

Phase 3: Design to meet the target factor of safety

- Step 5: Establish the target factor of safety
    a. Typically, a factor of safety of 1.3 is targeted.
- Step 6: Select a position for the granular shear key
    a. Recommendations in the literature suggest a granular shear key is most effective when placed near the toe of a slide, where the slip surface is horizontal. Cost can be minimized by selecting a location where the slip surface is shallow.

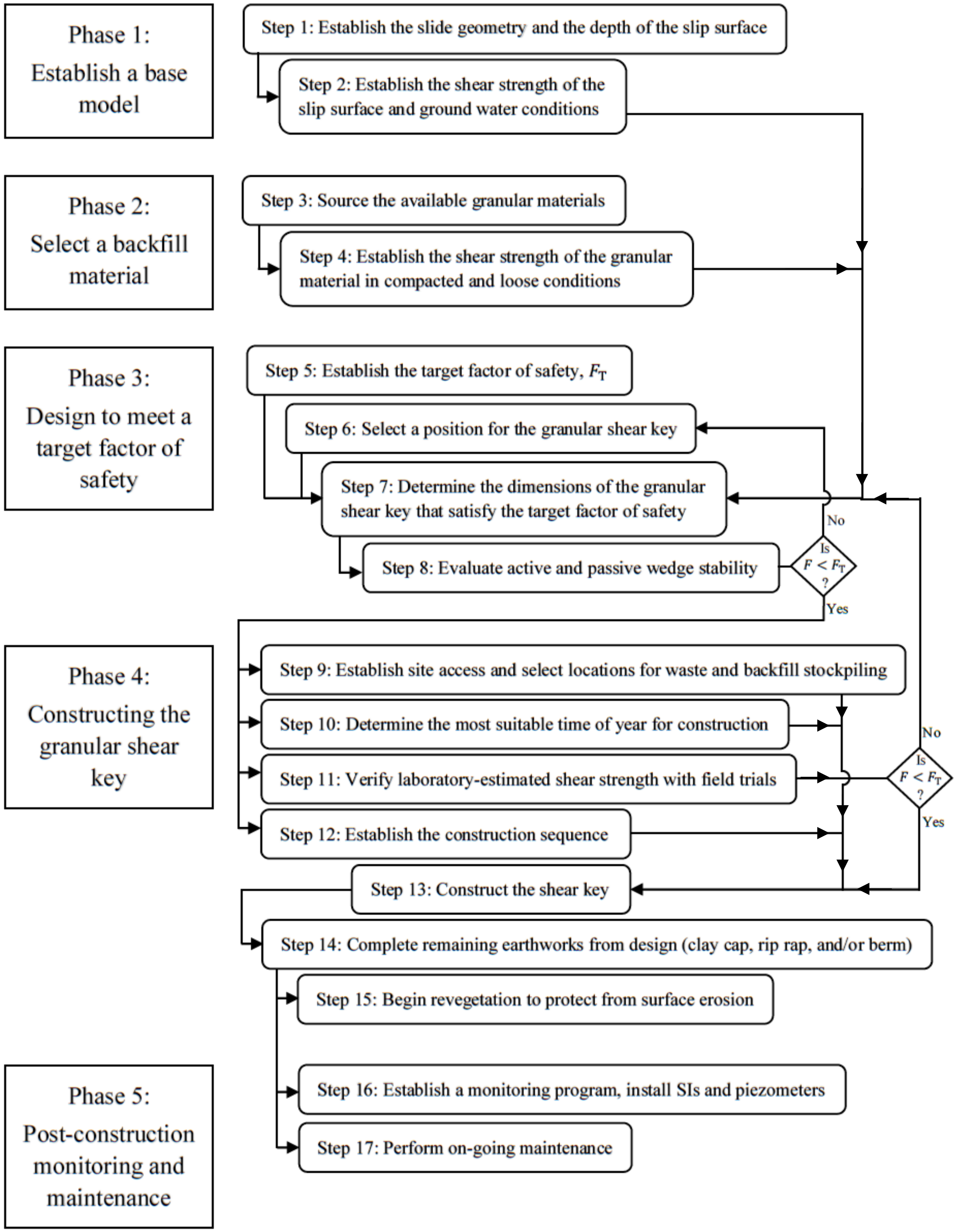


Figure 3.18: Flowchart depicting the principal phases of a granular shear key project and the steps involved in each phase.

Phase 3: Design to meet the target factor of safety (continued)

- Step 7: Determine the dimensions of the granular shear key that satisfy the target factor of safety
    a. The shear resistance requirements can be satisfied by increasing the normal effective stress above the slip surface, and by increasing the width of the shear key along the slip surface.
    b. When designing rockfill column shear keys, extra factors that must be considered are the area replacement ratio and the pattern that will be adopted. These factors affect the modelled properties of the materials used in the stability analyses.
    c. Typical dimensions are provided in Table 3.3 and Table 3.4.
- Step 8: Evaluate active and passive wedge stability
    a. If stability analyses have focused on a specified slip surface to this point, a grid search should be performed to evaluate stability upslope, downslope, and globally.

Phase 4: Constructing the granular shear key

- Step 9: Establish site access and select locations for waste and backfill stockpiling
- Step 10: Determine the most suitable time of year for construction, if non-emergency
- Step 11: Verify laboratory-estimated shear strength by conducting field trials
- Step 12: Establish the construction sequence
    a. Set up drainage and/or pre-benching if required.
    b. For closely-sequenced excavations, establish the maximum allowable length of open trench.
    c. For column construction, establish a drilling pattern that will not destabilize nearby holes or the slope.
    d. Establish rules for handling sloughing, construction delays, overnight, adverse weather, etc.
        - Backfill the excavation with the granular material in the event of uncontrolled events, until said events pass or are resolved.
        - The excavation should never be left open overnight; an unfinished panel can be backfilled loosely then re-excavated once work resumes.

    - It is advisable to establish shifts such that work can proceed for 24 hours a day until the shear key is completed.
    - Weather forecasts should be consulted so heavy rain can be avoided while construction is ongoing.

- Step 13: Construct the shear key
    a. Compact backfill in lifts as it is placed.
    b. Slope inclinometers should be monitored but often shear off during construction.
- Step 14: Complete remaining earthworks (clay cap, rip rap, and/or berm)
    a. Toe-armouring can mitigate the removal of material at the toe of the slide
    b. Earthworks rising above the existing grade must have been evaluated for stability
    c. Work is no longer time-sensitive from a stability perspective
- Step 15: Begin revegetation to protect from surface erosion
    a. Silt fences may be required until vegetation is sufficiently restored

Phase 5: Post-construction monitoring and maintenance

- Step 16: Establish a monitoring program and install slope inclinometers and piezometers
    a. The program should be designed to accommodate 1 to 4 years of movement, the typical timeframe for movement given by the literature
    b. SIs and piezometers upslope and downslope of the shear key to verify the shear key is performing as intended
    c. SIs outside of the slide, to verify movement does not go around the shear key
    d. SIs and piezometers within the shear key, to verify the slip surface has been intercepted, verify the dissipation of shear-induced pore pressure, and provide early warning if the shear key fills with groundwater
- Step 17: Perform on-going maintenance
    a. Repair or fill surface cracks to avoid unnecessarily prolonging movement and minimize surface water infiltration
    b. Verify subdrains are not blocked

4 CASE STUDY REVIEW OF SHEAR KEY DESIGN AND RESULTING PERFORMANCE

A comprehensive case study database was compiled as the first step toward establishing performance-based design guidelines for granular shear keys. This database was required to perform an empirical analysis of the relations between shear key performance, design, and site conditions. Altogether, 39 case studies were analyzed, having sufficient information to serve as a foundation for the planned analyses. The key information for these case studies is summarized in Table 4.1 and Table 4.2[5].

The case studies that were collected are mostly constrained to the Canadian Prairies, with only a few from the United States of America. These case studies were obtained through cooperation with several contributing parties. A select few case studies were also acquired from literature. The primary contributors to this research were Alberta Transportation, Canadian Pacific Rail, Canadian National Rail, and the City of Winnipeg. Additional data was obtained from geotechnical consultants involved with the design, construction and monitoring of the remedial works. These consultants are Thurber Engineering Ltd., Klohn Crippen Berger, Clifton Associates Ltd., Tetra Tech EBA, Amec Foster Wheeler, and KGS Group. The case study summaries are presented in Appendix A[5] and are indexed by calling codes outlined there. Any references made to specific case studies throughout this chapter will utilize this index system. There are two additional case study summaries presented in Appendix A for which insufficient data was available to include in the analyses. However, the background histories and descriptions of the slope conditions for these two additional sites provided valuable context for the conditions for which granular shear keys are designed to address.

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[5] One case study was omitted from the summary tables and Appendix A in order to protect confidential information.

Table 4.1: Summary of the general types of repairs, backfill, and landslide information for the granular shear key case studies that were collected.

			Repair type		Backfill information			Landslide information						
						Compaction								
Index	Site name	Approximate location	Trench or Columns	Additional	Backfill type	Y/N	Method	Landslide type	Sheared material	Estimated volume (m^3)	Width (m)	Height (m)	Natural slope (°)	Residual friction angle (°)
AT1	PH68: Hwy 743:02, Bogey Slide	North of Peace River, AB	Trench	Slope replacement	Des. 6 Cl. 80 granular fill	Y	"In lifts"	Rotational	Lacustrine silty clay (CH)	1.3×10^3	45	4.8	16	nd
AT2	PH10: Eureka River Site #2	South of Worsley, MB	Columns	Buttress	Pitrun gravel	nd	nd	Planar	CH lacustrine clays and silt	1.1×10^5	100	16	12.5	nd
AT4	PH16: Hwy 88:18 Ft. Vermillion Culvert	Fort Vermillion, AB	Trench	None	nd	Y	nd	Rotational	CI to CH lacustrine clay, slickensided	1.3×10^3	60	12	45	nd
AT5	GP24: Hwy 725:02 Hamelin Creek Slide	North of Blueberry Mountain, AB	Columns	Berm	40 mm minus crushed gravel	Y	Drop weight	Rotational	Clay fill	2.2×10^4	50	24	18.5	11
AT6	PH31: Judah Hill - Michelin Slides	South of Peace River, AB	Trench	Multiple others	Shredded tire	nd	nd	Soil slope deformation	Ancient landslide terrain	nd	60	nd	nd	nd
AT7	GP02: Hwy 40:38 Kakwa Slide	North of Grande Cache, AB	Trench	Slope replacement	Well-graded pit run gravel	Y	"In lifts"	Soil creep	nd	nd	50	10	nd	nd
AT8	NC59: Hwy 43:16 Little Paddle River Slide	Mayerthorpe, AB	Columns	Berm	2" crushed gravel	Y	Vibroflot	Compound	High plastic clay	nd	120	13	14	14
AT9	Little Smoky River	Between Fairview, AB and Peace River, AB	Columns	None	Pit run gravel	Y	Vibrated H-pile	Planar	Weathered clay shale	2.7×10^4	65	12.5	18	14
AT10	S07: Hwy 549:02 W. of Millarville	West of Millarville, AB	Trench	None	Pit run gravel, des. 6 cl. 125	Y	nd	Planar	Clay, clay fill	5×10^2	110	5.5	9	11
AT11	GP10: S. of Sturgeon Lake	West of Valleyview, AB	Trench	Buttress	Granular material	nd	nd	nd	Embankment overlying soft clay deposits	nd	nd	2.5	14	nd
AT12	PH4: W. of Fairview	West of Fairview, AB	Trench	Unknown	Crushed limestone	nd	nd	Rotational	Alluvial silty clay	1.1×10^5	200	12	14	nd

Table 4.1 (continued)

			Repair type		Backfill information			Landslide information						
						Compaction								
Index	Site name	Approximate location	Trench or Columns	Additional	Backfill type	Y/N	Method	Landslide type	Sheared material	Estimated volume (m^3)	Width (m)	Height (m)	Natural slope (°)	Residual friction angle (°)
CN1	CN Sangudo Bridge Mile 74.7	Mayerthorpe, AB	Trench	Buttress	Granular fill	Y	Non-vibro	Planar	Lacustrine high plastic clay	7.2×10^2	20	9.5	15	20
CP1	Harrowby Hills, MB Mile 86.70	South of Harrowby, MB	Trench	None	Sandy	N	na	Planar	Weathered clay shale	nd	nd	12	13	nd
CP1	Harrowby Hills, MB Mile 86.75	South of Harrowby, MB	Trench	None	Sandy	N	na	Planar	Weathered clay shale	1.4×10^4	70	12	13	16.7
CP1	Harrowby Hills, MB Mile 86.80	South of Harrowby, MB	Trench	None	Sandy	N	na	Planar	Weathered clay shale	1.9×10^4	81	12	13	13
CP1	Harrowby Hills, MB Station 16+575	South of Harrowby, MB	Trench	None	Sandy	N	na	Planar	Weathered clay shale	nd	nd	12	13	nd
CP1	Harrowby Hills, MB Station 16+725 (West Shear Key)	South of Harrowby, MB	Trench	None	Sandy	N	na	Planar	Weathered clay shale	nd	nd	12	13	nd
CP2	Carrington Sub Mile 294.0	West of Fargo, ND	Trench	Buttress	Pit run gravel	N	na	Planar	Weathered clay shale	1.8×10^4	135	15	26.5	9.5
CP4	Emerson Sub Mile 61.57	Emerson, MB	Trench	Berm	Ballast reject	nd	nd	nd	Embankment	nd	50	nd	nd	nd
CP5	Lanigan Sub Mile 26.35	Northwest of Regina, SK	Trench	Berm	nd	nd	nd	nd	nd	nd	47	2.7	26.5	nd
CP6	Lloydminster Sub Mile 78.3	South of Lloydminster, SK	Trench	Berm	Pit run gravel	nd	nd	Compound	Fill, clay shale	1.9×10^4	100	11	16	13.4
EX1	Bear Creek	Grande Prairie, AB	Trench	None	Gravel	Y	"In lifts"	Planar	Lacustrine silt and clay	4.0×10^3	105	4.9	13	nd
EX2	Bender's Park, SD (bottom)	Lead, SD	Trench	Caisson wall	Rockfill	N	na	Planar	Clayey silt/silty clay	1.7×10^5	185	29	7	10
EX2	Bender's Park, SD (top)	Lead, SD	Trench	None	Rockfill	N	na	Planar			184	25	7	nd
EX3	Hagg Lake	West of Portland, OR	Trench	Buttress	nd	nd	nd	nd	Silty clay	1.4×10^4	85	18	nd	nd
EX4	Orange County	South of Irvine, CA	Trench	Berm	nd	nd	nd	nd	nd	4.7×10^5	213	67	26.5	nd
WP1	1, 7 & 11 Evergreen Place	Winnipeg, MB	Both	None	Crushed limestone	N	na	Planar	Clay (CH)	6.0×10^3	150	2.5	16	nd
WP2	99 & 141 Wellington Crescent	Winnipeg, MB	Both	None	Crushed limestone	Y	Vibroflot	Planar	Alluvial clay	2.7×10^4	180	6.7	10	nd
WP3	Ave de la Cathedrale Outfall	Winnipeg, MB	Columns	None	Crushed limestone	Y	Vibroflot	Planar	Silty clay (CH)	1.3×10^4	65	9	9	nd

Table 4.1 (continued)

Index	Site name	Approximate location	Repair type: Trench or Columns	Repair type: Additional	Backfill information: Backfill type	Compaction: Y/N	Compaction: Method	Landslide information: Landslide type	Sheared material	Estimated volume (m^3)	Width (m)	Height (m)	Natural slope (°)	Residual friction angle (°)
WP4	Byng Place Outfall 915-2013	Winnipeg, MB	Both	None	Crushed limestone	Y	Vibroflot	nd	Clay	1.1×10^4	55	11.5	nd	nd
WP5	Churchill Drive Park 936-2010	Winnipeg, MB	Trench	None	Crushed limestone	Y	Vibro-compacted	Planar	Silty clay (CH)	5.4×10^4	450	12.1	16.5	nd
WP6	Fort Rouge Park 876-2007	Winnipeg, MB	Trench	None	Crushed limestone	Y	nd	Planar	Lacustrine clay	6.5×10^3	87	10	12	11
WP7	Hawthorne FPS	Winnipeg, MB	Columns	None	Crushed limestone	N	na	Planar	Lacustrine silty clay (CH)	1.2×10^4	38	18	24	nd
WP8	Mager Drive	Winnipeg, MB	Trench	Granular ribs	Crushed limestone	Y	Vibratory plate	Compound	Riverbank	1.2×10^3	7	10	10	8
WP9	Oakcrest Place	Winnipeg, MB	Trench	None	Crushed limestone	Y	Vibroflot	nd	Lacustrine clay	nd	nd	10	14.7	11
WP 10	Omand's Creek Outfalls	Winnipeg, MB	Trench	Granular ribs	Crushed limestone	Y	nd	Rotational	Siltstone and claystone	nd	nd	4.7	17	10
WP 11	Pembina-Ducharme Culvert	Winnipeg, MB	Trench	None	Crushed limestone	Y	"In lifts"	nd	CH lacustrine clay, slickensided	1.4×10^3	25	5	9.5	nd
WP 12	Radcliffe Road Outfall	Winnipeg, MB	Columns	None	Crushed limestone	Y	Vibroflot	Planar	Clay	9.8×10^3	80	7	12	nd
WP 13	Rue Despins Outfall	Winnipeg, MB	Columns	None	Crushed limestone	Y	Vibroflot	Planar	Silty clay (CI)	6.7×10^3	25	13	18.5	nd
WP 14	Rue Dumoulin Outfall 646-2003	Winnipeg, MB	Trench	None	Crushed limestone	Y	nd	Rotational	CH glacio-lacustrine clay	6.5×10^3	35	9.3	10	nd
WP 15	Rue la Verendrye Outfall	Winnipeg, MB	Columns	None	Crushed limestone	N	na	Planar	Alluvial silty clay	9.3×10^3	50	14	15.5	nd
WP 16	Seine River Siphon	Winnipeg, MB	Columns	None	Crushed limestone	Y	Vibrated H-pile	Rotational	Lacustrine silty clay	nd	nd	nd	nd	nd

nd no data was recorded

na not applicable

Table 4.2: Summary of repair information for the granular shear key case studies that were collected.

		Repair information							Trench information		Column information				
													Spacing (m)		
Index	Site name	Effective depth (m)	Excavation depth (m)	Height of granular (m)	Height of cap/berm (m)	Length of repair (m)	Distance from scarp	Effective width (m)	Base width (m)	Angle from horiz.(°)	Diameter (m)	# of rows	Cross-slope	Up-slope	Area replacement ratio
AT1	PH68: Hwy 743:02 Bogey Slide	2	5.6	5.3	0.3	42	64%	9.2	2	45	na	na	na	na	na
AT2	PH10: Eureka River Site #2	7	12	12	nd	100	nd	1.0	na	na	0.75	4	2.0	2.0	11%
AT4	PH16: Hwy 88:18 Ft. Vermillion Culvert	6.8	6	6.4	4.5	40	nd	26.2	18	45	na	na	na	Na	na
AT5	GP24: Hwy 725:02 Hamelin Creek Slide	7	7	7	3	47.5	90%	8.0	na	na	1.2	18	2.5	2.2	21%
AT6	PH31: Judah Hill - Michelin Slides	nd	nd	nd	nd	50-70	nd	nd	nd	nd	na	na	na	na	na
AT7	GP02: Hwy 40:38 Kakwa Slide	nd	5.5	5.5	nd	nd	nd	nd	nd	nd	na	na	na	na	na
AT8	NC59: Hwy 43:16 Little Paddle River Slide	10.5	8.5	9	3	185	55%	1.4	na	na	1.82	3	5.6	3.0	10%
AT9	Little Smoky River	5.2	5.6	4.8	0.6	65	54%	7.4	na	na	2.44	4	2.5	3.1	43%
AT10	S07: Hwy 549:02 W. of Millarville	3	6.1	5	1.1	110	nd	12.2	3	34	na	na	na	na	na
AT11	GP10: S. of Sturgeon Lake	nd	nd	nd	nd	nd	nd	nd	nd	nd	na	na	na	na	na
AT12	PH4: W. of Fairview	nd	nd	nd	nd	200	49%	nd	nd	nd	na	na	na	na	na

Table 4.2 (continued)

		Repair information							Trench information		Column information				
													Spacing (m)		
Index	Site name	Effective depth (m)	Excavation depth (m)	Height of granular (m)	Height of cap/berm (m)	Length of repair (m)	Distance from scarp	Effective width (m)	Base width (m)	Angle from horiz.(°)	Diameter (m)	# of rows	Cross-slope	Up-slope	Area replacement ratio
CN1	CN Sangudo Bridge Mile 74.7	1.8	4	3	1	20	na	5.0	5	90	na	na	na	na	na
CP1	Harrowby Hills, MB Mile 86.70	4	11	8	3	268	57%	12.0	6.7	69	na	na	na	na	na
CP1	Harrowby Hills, MB Mile 86.75	4.1	13	11	2	105	57%	21.9	13	63	na	na	na	na	na
CP1	Harrowby Hills, MB Mile 86.80	4	13	11	2	95	57%	14.1	7.5	70	na	na	na	na	na
CP1	Harrowby Hills, MB Station 16+575	4	6	5	1	65	57%	9.0	7.9	74	na	na	na	na	na
CP1	Harrowby Hills, MB Station 16+725	4	6	5	1	94	57%	8.8	7.4	71	na	na	na	na	na
CP2	Carrington Sub Mile 294.0	6.1	9.1	7.6	1.5	109	nd	3.3	1.5	73	na	na	na	na	na
CP4	Emerson Sub Mile 61.57	nd	2	2	nd	50	nd	nd	nd	nd	na	na	na	na	na
CP5	Lanigan Sub Mile 26.35	nd	1	1.5	nd	47	nd	nd	1	90	na	na	na	na	na
CP6	Lloydminster Sub Mile 78.3	8.7	10	10	0	85	nd	4.3	3	63	na	na	na	na	na
EX1	Bear Creek	3.5	5	5	0	105	48%	3.8	1.5	53	na	na	na	na	na
EX2	Bender's Park, SD (bottom)	9	16.7	13.4	3.3	162	81%	11.5	7.4	75	na	na	na	na	na
EX2	Bender's Park, SD (top)	6	15	10	5	162	12%	32.0	5	34	na	na	na	na	na
EX3	Hagg Lake	7.3	nd	nd	nd	85	nd	nd	nd	nd	na	na	na	na	na
EX4	Orange County	8	9.1	9.1	nd	213	nd	17.2	15	45	na	na	na	na	na
WP1	1, 7 & 11 Evergreen Place	3.5	5.5	5	0.5	144	67%	4.2	3	73	2.1	Spot installation			
WP2	99 & 141 Wellington Crescent	7.4	6.6	6	0.6	60.5	78%	2.9	3.2	81	2.1	4	3.4	2.7	35%
WP3	Ave de la Cathedrale Outfall	6.1	9.6	9	0.6	66	74%	4.5	na	na	2.1	4	3.1	2.3	42%

Table 4.2 (continued)

		Repair information							Trench information		Column information				
													Spacing (m)		
Index	Site name	Effective depth (m)	Excavation depth (m)	Height of granular (m)	Height of cap/berm (m)	Length of repair (m)	Distance from scarp	Effective width (m)	Base width (m)	Angle from horiz.(°)	Diameter (m)	# of rows	Cross-slope	Up-slope	Area replacement ratio
WP4	Byng Place Outfall 915-2013	6.7	7.9	7.3	0.6	30	65%	2.6	2	76	2.13	2	3.3	2.9	38%
WP5	Churchill Drive Park 936-2010	5.6	5.9	4.7	1.2	450	69%	3.7	3.5	76	na	na	na	na	na
WP6	Fort Rouge Park 876-2007	3.7	6	5.4	0.6	50	54%	5.2	4	76	na	na	na	na	na
WP7	Hawthorne FPS	3.5	4.1	2.9	1.2	38	26%	4.2	na	na	2.1	4	3.3	2.7	37%
WP8	Mager Drive	3.3	7.8	7	0.8	87	69%	13.0	4	45	na	na	na	na	na
WP9	Oakcrest Place	5.2	6.3	4.7	1.6	25	48%	3.3	2.5	76	na	na	na	na	na
WP10	Omand's Creek Outfalls	2.5	5	5	0	38	nd	3.5	1	63	na	na	na	na	na
WP11	Pembina-Ducharme Culvert	4.4	3.2	4	1.5	18	nd	nd	4	90	na	na	na	na	na
WP12	Radcliffe Road Outfall	6.1	7.3	6.1	1.2	61	nd	2.2	na	na	2.1	2	3.1	2.7	42%
WP13	Rue Despins Outfall	10.5	13.6	13	0.6	43	62%	4.5	na	na	2.1	4	3.1	2.5	42%
WP14	Rue Dumoulin Outfall 646-2003	6.2	7.9	5.8	2.1	30	58%	6.8	6	76	na	na	na	na	na
WP15	Rue la Verendrye Outfall	6.7	8.9	7.7	1.2	51	89%	3.1	nd	nd	2.1	3	3.4	2.8	35%
WP16	Seine River Siphon	7	7.4	5.4	2	nd	81%	6.3	nd	nd	2.1	6	3.3	2.9	37%

nd no data was recorded
na not applicable

4.1 Construction of a Case Study Database

The case studies were compiled if the following information was available:

i) A granular shear key or a rockfill column repair must have been designed for the site.

ii) Basic design specifications must be available for the granular shear key or rockfill columns.

iii) A site-specific description of the slope and stratigraphy must be available.

iv) Some degree of monitoring data must be available, whether qualitative or quantitative.

It was later found that monitoring data was quite sparse, so this criterion was relaxed. The following criteria were also considered assets but, to avoid severely limiting the number of sites that could be considered, they were not strictly required:

i) A site history detailing landslide triggers, site importance, and other miscellaneous facts.

ii) Laboratory and field tests for soil parameters including moisture content, Atterberg limits, soil strength (undrained and/or drained), etc.

iii) Back-analyzed landslide soil properties; notably, the residual strength of the failure surface.

iv) Design drawings in plan view and in cross section, ideally as-built.

Once case studies were acquired, a summary was written to preserve the context of the remedial work. Each summary contains information collected on the site background, remediation, performance of the repairs, and lessons learned. Summaries are followed by a 1-page recap of site statistics, which includes basic landslide information such as the sliding mechanism, trigger, soil type, slide dimensions, and shear key dimensions. Where available, the statistics are followed by supplementary information which includes type logs, cross sections, and/or plan maps. All case study summaries are compiled in Appendix A.

Reported values were used to populate the database fields wherever possible. However, inconsistencies in reporting and unreported information had a considerable impact on the completeness of the database. These gaps were supplemented by taking measurements from cross sections and plan maps.

It was recognized that compiling values deriving from a variety of sources could introduce error. To minimize this potential error, the methodology for obtaining these values was kept consistent. This methodology is explained as analyses using measured or calculated data arise. For a few of the sites, both reported values and illustrations were available. These sites were used to verify that the error between the measured and reported values was small. This potential error was further deemed acceptable because of how the reported values were being recorded. For example, field instructions often specify that the shear key excavation must extend a minimum distance into a competent unit. The actual depth of a shear key may therefore vary in the order of a meter or more depending on the nature of the geological units. An estimate of the average or range of depths for the shear key might be reported in this case. Compared to these average values, the potential error from measuring or calculating missing information was deemed acceptable.

4.1.1 Statistics from the case study database

Once the database had been constructed, some statistics were generated to describe shear key projects. These statistics can be compared to the ranges suggested by literature, cited in Chapter 3.

Trenched shear keys

The statistics shown in Table 4.3 are for the trenched shear key projects that were collected. The dimensions are shown schematically in Figure 3.11 and are generally consistent with the ranges cited from the literature in Table 3.3. The greatest discrepancies are for the key-in depth and the trench angle.

The key-in depths for the case studies were shorter on average than the minimum key-in depth suggested by the literature. This discrepancy may be explained by the bias toward adopting shear keys at sites where a competent unit can be keyed into, since they are well-suited to such sites. This was the situation that was reported for most of the case studies. Thus, the key-in depth would have been justifiably reduced and this is reflected in the statistics. Furthermore, the key-in depth was not found reported in many of the documents that were collected. Measuring this dimension from cross sections was considered, but the key-in depth is small in proportion to the total height of the shear key. For this reason, the decision was made to not rely on measurements made for this dimension. Thus, the sample size for key-in depths is small and the statistics may be biased toward the sites for which reported values were obtained.

Table 4.3: Statistics for the trenched shear key case studies, presented next to the recommended ranges from the literature.

Dimension	Trenched shear key case studies			Typical range from literature
	Minimum	Average	Maximum	
Base width (m)	1.0	5.5	18.0	1.5 to 9.1
Depth (m)	2.0	7.1	13.4	1.2 to 12.2
Key-in depth (m)	0.3	0.8	3.0	> 0.9 to 1.5
Trench angle (°)	34	63	90	45
Length of open excavation (m)	2.0	4.4	7.0	< 1.5 to 10

The trench angle from the case studies was on average 18° steeper than the typical trench angle of 45° cited from the literature. Many of the case studies were from the City of Winnipeg, where shear key projects are typically completed in the winter when the ground is frozen. The stand-up time for the shear key excavations would therefore be prolonged, allowing for steeper trenches to be excavated.

Rockfill columns

The statistics generated for the rockfill column case studies are presented in Table 4.4, where they are compared to the ranges cited from the literature (originally given in Table 3.4) for the dimensions illustrated in Figure 3.12. The first parameter for which there is a discrepancy between the literature and the database is the column diameter. The average column diameter for the case studies is almost twice as large as the cited diameters. This contrast is because the rockfill columns in the case studies were exclusively constructed for landslide remediation and would need to be wide to provide adequate shearing resistance. The diameters cited from the literature are for rockfill columns intended to resist compressional loading, in foundation applications. The center-to-center spacing between the columns in the database is near the upper end of the ranges reported in the literature. This is a direct result of the larger column diameters used in landslide mitigation projects.

The replacement ratios for the rockfill column case studies are also toward the upper range suggested by the literature. Again, this is likely a reflection of these rockfill columns being used for landslide mitigation. Replacing a greater amount of the native soil with rockfill increases the shear force generated by the columns.

Some of the database statistics, however, are consistent with the values given by the literature. One of these parameters is the depth to which the columns are constructed. This consistency may be because similar equipment and construction challenges exist for rockfill columns, regardless of whether they are intended to resist lateral or vertical loads.

Another parameter that is consistent between the database and the literature is the key-in depth. The key-in depths are consistent with the values given for when hard bedrock is present. This appears to be reasonable since most of the rockfill column case studies reported the presence of competent clay shales or till beneath the slip surface.

Table 4.4: Statistics for the geometry of rockfill columns used as shear keys.

Dimension	Rockfill column shear key case studies Minimum	Average	Maximum	Typical range from literature
Diameter (m)	0.8	1.9	2.1	0.7 to 3.0
Depth (m)	4	7.9	12	4.0 to 27.4
Key-in depth (m)	0.3	0.6	0.9	0.6 to 3.0
Replacement ratio (%)	10	31	42	15% to 35%
Spacing (m)	2	3.3	5.6	1.8 to 3.7

Comparison between trenched shear key and rockfill column statistics

By comparing the statistics shown in Table 4.3 and Table 4.4, several observations can be made. First, the average depths for trenched shear keys and rockfill columns are quite similar. Under closer scrutiny though, the rockfill column projects appear to go to greater depths more consistently than trenched shear keys. This is evidenced by rockfill column projects having a smaller standard deviation, a greater median and a greater minimum depth than trenched shear key projects. The average depth to the slip surface for rockfill column projects is also about 50% greater than that for trenched shear keys. This finding is consistent with the literature, wherein it is suggested rockfill columns are more suitable where deeper slip surfaces have been identified. Looking at key-in depths, the two techniques are similar once again. The trenched shear keys in the database appear to occasionally key-in much deeper than rockfill columns though, as evidenced by a comparison of the maximum key-in depths. A possible explanation is that the soils into which these shear keys were being excavated were expected to weaken over time. This may be a reasonable assumption since, as stated previously, the trenched shear keys tend to be shallower than rockfill columns.

4.1.2 Evaluating shear key performance from slope inclinometers

The objective of producing performance-based design guidelines necessitated the collection and logging of performance indicators from each of the case studies. It should first be clarified that the use of the word *performance* to describe the deformation exhibited at each of these sites is in no way intended to describe the adequacy of the designs. The definition of satisfactory performance is unique to each site and entirely depends on the tolerance for deformation, which can vary considerably. Rather, the amount of deformation was compared on a relative basis only to identify trends that could be used to more reliably and accurately predict future deformations for new designs.

With transportation infrastructure, such as railways or highways, the most obvious indicator of granular shear key performance is damage to the railway or road in the form of cracks or alignment issues. Evaluating damage is sometimes the only way to gauge the performance of a granular shear key, since monitoring programs cannot always be justified where the consequences of these movements are low. For this work, some sites were limited to maintenance reports that included qualitative descriptions of cracking or bulging, while others contained quantified displacements measured using slope inclinometers (SI). The analyses that were performed exclusively focused on the quantitative data that was available.

The number and locations of SIs used to monitor displacement at each site was also highly variable. Some sites were monitored for a short period before construction began, using a single SI. Others were instrumented with many SIs located all over a slope. Wherever possible, data was taken from SIs located upslope of the shear key and midway across the slide for the sake of consistency.

SI data was often acquired in the form of printed or scanned plots. These plots were converted to usable digital point data using DigitizeIt Version 2.2.2 (Bormann, 2001). In this software, figure axes were designated as scales and individual data points were hand-selected. The software determined the value of the points based on their relative position with respect to the scales that had been set. The error introduced by this methodology was dictated by the precision with which the points and the axes were selected. The plots were magnified so this error could be minimized. Where traces on the original SI plots overlapped, the margin of error was potentially greater. This was reasoned to be inconsequential to the analysis since overlapping only occurred where the rate of sliding was near zero.

The types of displacement data recorded in the database were the total displacement and the rate of displacement. Only the magnitude of these measurements was recorded. The exact direction of the movement was treated as being of relatively minor importance since a preliminary check found it was consistently in the general direction of the shear key (downslope). Rates were calculated as the change in the total displacement over a time interval. Where the total displacement was not available but rates were reported, the reported values were used instead. For most of the analyses, the annual rate of movement was used.

The displacement data that was used for the analyses was taken from the depth where discrete movement was taking place, as observed on the SI logs. Discrete movements were more easily identified by plotting the incremental displacement with depth. An example of this is shown in Figure 4.1. Where discrete movement was not exhibited (see Figure 4.2), the maximum displacement was recorded instead; this was typically at the surface. The potential limitations and error introduced by this inconsistency in selecting displacement data were evaluated by comparing this methodology against two alternatives.

The alternatives were to consistently record the maximum displacement, or to consistently record the surface displacement. To simply record the maximum displacement for slides where a discrete plane of movement was observed might include surface movements that were not caused by the slide block itself. This would be considering movements from two different mechanisms, with the second mechanism being anything from surface erosion to the freezing and thawing of the near-surface soils. Furthermore, many shear keys were capped with clay which was observed to slide above the shear key in some instances. This movement influenced the maximum displacement but did not reflect the performance of the shear key itself. To record the surface displacement (not necessarily the maximum displacement) might introduce error too. Some SI logs showed total displacement profiles with downslope movement at depth but none near the surface. In these cases, the logs are potentially indicating that the SI or the sliding mass is tilting, as can occur for rotational slides. The displacement at surface would therefore be a poor indicator of the performance of the shear key.

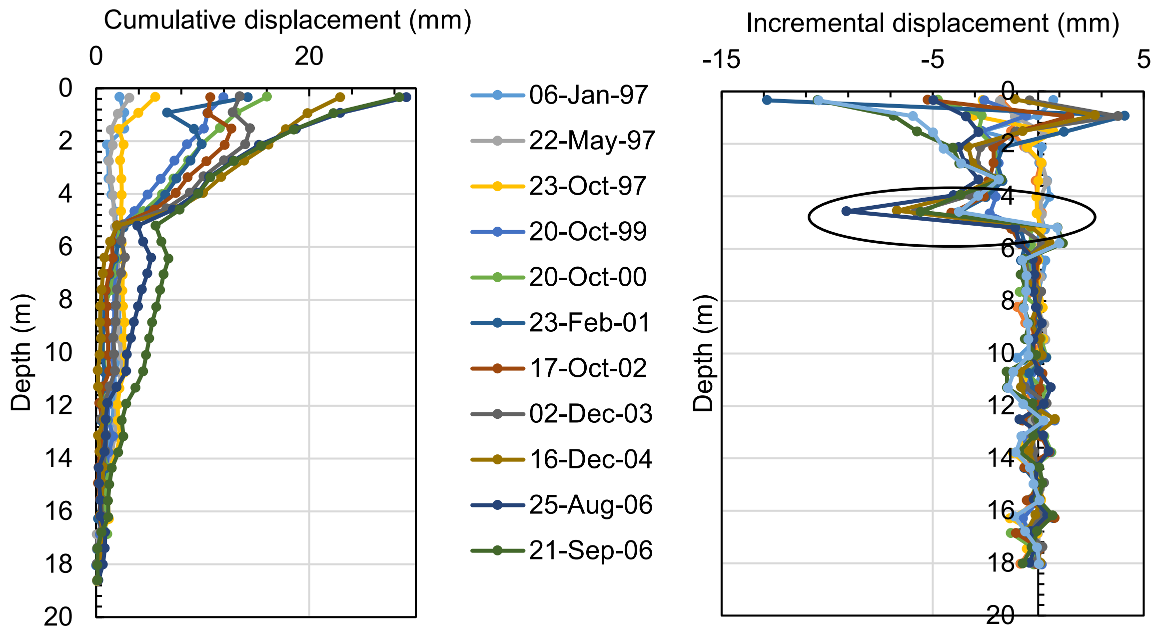


Figure 4.1: Cumulative and incremental displacement plotted with depth for slope inclinometer data. The discrete sliding zone is circled in black on the incremental displacement plot. The surface displacement corresponds to movement in the clay cap.

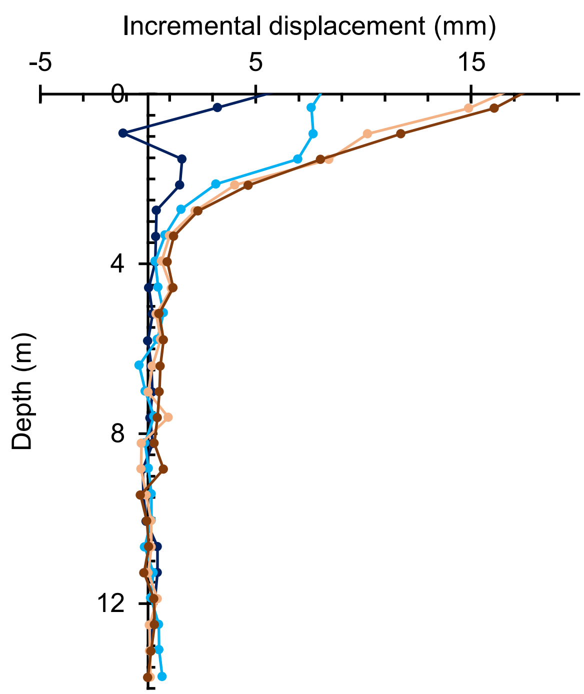


Figure 4.2: Incremental displacement rates for a slide that did not exhibit a discrete zone of movement.

In consideration of these two alternatives, the methodology that was adopted was reasoned to be the best option because of the focus on capturing the primary slide mechanism. The data collected using this method was also expected to be the most reflective of the performance of the shear key.

Additional error was likely introduced because of the inconsistency in the locations of the SIs consulted for each slide, which was dictated by availability and could not be tightly controlled. Thus, the error that was potentially introduced by the methodology that was adopted was judged to be reasonable and was not expected to inhibit the identification of general trends should they exist.

4.1.3 Creating data subsets for analyses

The variety of conditions encompassed by the case studies was expected to obscure possible trends in the data. To overcome this challenge, smaller subsets were constructed from the database. Some of these subsets were created using histograms. For example, to investigate the impact of changes in the effective depth, a histogram was created for the effective width. Subsets were then created using the sites falling within each bin. This resulted in subsets being formed where the effective width was essentially held constant.

Some other subsets were created for conditions or materials that were expected to have an impact on the effectiveness of granular shear keys. These subsets were sometimes used in combination with other subsets, to identify multiple relations in a single plot. It was found that some relations became very apparent when the data were sorted by these additional conditions at the same time as having certain dimensions constrained, as mentioned above.

4.2 Shear Key Design in Practice and Its Relation to Slide Movement

In this section, the expectations and results are presented for the analyses that were performed on the granular shear key database. The section is structured such that results are presented then discussed before proceeding to the next analyses. Where applicable, these results are presented alongside modelled results which were produced using Slide 7.020, a 2D LE software created by Rocscience (2016). Examples of sites that demonstrated the identified relationships particularly well are also provided. Lastly, the number of data points featured in each analysis varies depending on the number of case histories for which the data required for those analyses had been obtained.

4.2.1 The effect of shear key location on slide movement

Shear keys are almost always located near the toe of a slope. This satisfies the objective of increasing shear resistance, without increasing the driving forces acting on the slide system. Another reason for placement near the toe of a slide is that toe erosion is frequently cited as a triggering mechanism. Toe armouring measures, such as the placement of rip-rap (see Figure 4.3), are often performed as part of shear key construction in these cases. Placement toward the toe of the landslide is also favoured since slip surfaces tend to be shallowest there and the repair volume can be minimized (Figure 4.4).

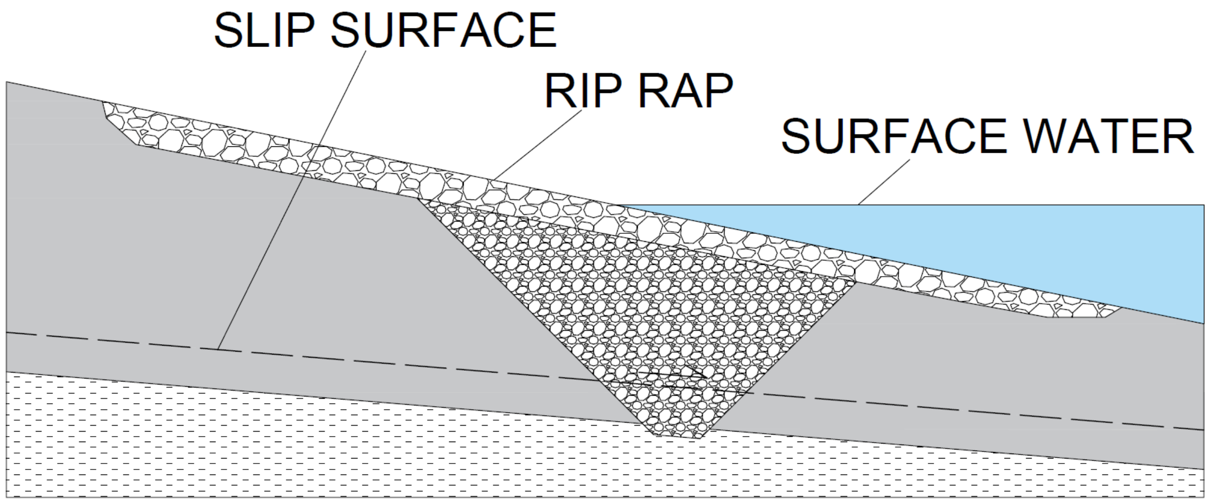


Figure 4.3: A granular shear key schematic in cross section showing rip rap placed at surface to protect from erosive toe conditions brought on by water at surface.

The granular backfill used in shear keys is a frictional material though, so shallower shear keys will be able to provide less resistance. Thus, practitioners must find a balance between maximizing the shear resistance that can be provided while also minimizing the costs and the destabilizing effect associated with a larger excavation.

There are few exceptions to this tendency toward placing shear keys near the toe. In one case (AT6), a lightweight shear key was considered for construction in the upper portion of the slide. It was reasoned that the use of lightweight fill would reduce the driving force on the downslope portion of the slide because the unit weight of the fill was less than the soil that was being replaced. This would enhance the downslope stability. The shear key would also resist the portion of the slide on its upslope side, enhancing upslope stability. As mentioned in Chapter 3, the shear key can also act as a groundwater interceptor trench so this placement far upslope of the toe might also improve the groundwater conditions from a stability perspective.

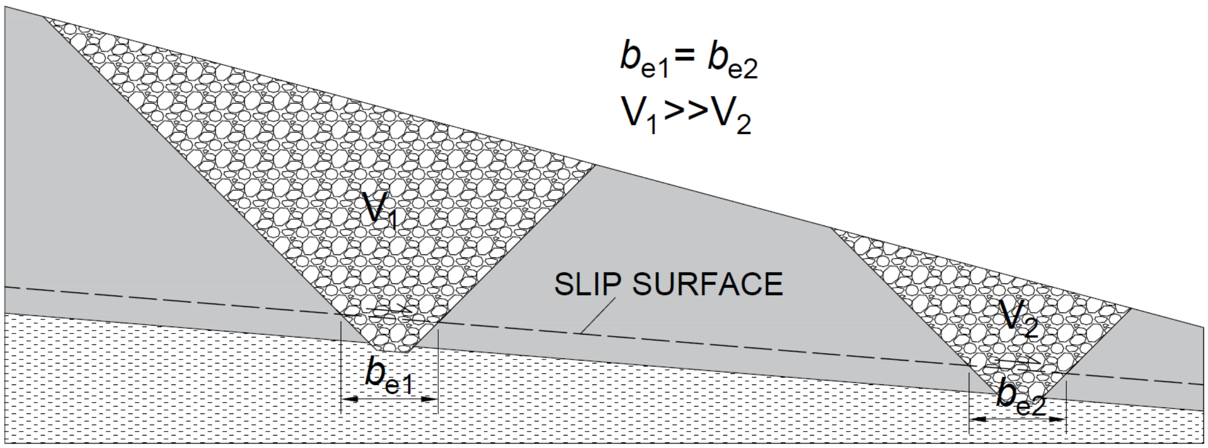


Figure 4.4: Two granular shear keys in cross section designed with the same effective width, b_e, but intercepting a slip surface at different depths. The deeper shear key requires a much greater volume, V, than the shallow shear key.

For the reasons given above and discussed in Chapter 3, the placement of a granular shear key can potentially affect its performance. This concept was investigated by comparing the placement of different shear keys to the resulting performances. First, the relative location of the shear key had to be quantified in a way that would permit comparison between case histories. A unitless dimension for the position of the shear key, P_{SK}, was defined as

$$P_{\mathrm{SK}} = \frac{L_{\mathrm{SK}}}{L_{\mathrm{H}}} \times 100\% \tag{4.1}$$

where L_{SK} is the horizontal distance between the head scarp of a slide and the centerline of the shear key in cross section, and L_{H} is the horizontal length of the slide. These dimensions are illustrated schematically in Figure 4.5.

In the literature, it was recommended that shear keys be constructed where the slip surface was roughly horizontal. It was not possible to accurately determine the angle of the slip surface at the location of each shear key with the data that was available though.

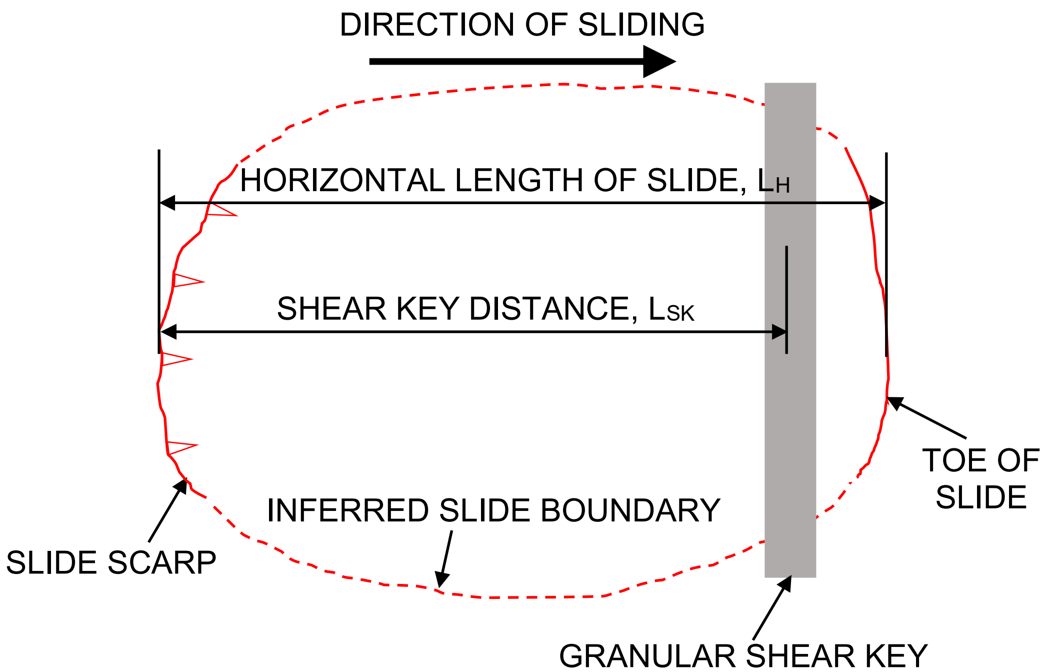


Figure 4.5: Schematic plan view illustration of a landslide and a granular shear key. The distances used to quantify the relative location of the shear key with respect to the slide are shown.

The first analysis consisted of plotting all the case studies where the percentage reduction to the rate of sliding one year after construction, R_1, was known. This measure of performance was defined as

$$R_1 = \frac{\Delta_0 - \Delta_1}{\Delta_0} \times 100\% \tag{4.2}$$

where Δ_0 is the pre-construction rate of sliding and Δ_1 is the rate of sliding one year after construction was completed. Δ_0 and Δ_1 must have the same units, typically mm/year.

The result of this analysis is shown in Figure 4.6. From this preliminary result, it appears the closer a trenched shear key was constructed to the toe (i.e. as P_{SK} approaches 100%), the greater the value of R_1. Sites where there was a column and trench combination appeared to follow an entirely different trend, with R_1 decreasing as P_{SK} increased. To isolate the contribution of the shear keys from the potential contributions of other aspects of the repairs, the dataset was filtered to exclude case histories involving berms, or where significant regrading and major changes to the groundwater regime had been reported. However, only one trenched shear key data point remained after this filter was applied, so the result was inconclusive.

Typically, post-construction movements are expected to take place over several years. For this reason, the 1-year timeframe that was analyzed may have been too short to be a reliable indicator of performance.

The next analysis investigated the effect P_{SK} had on performance during the first five years after construction. The percentage reduction in the rate of sliding after each year, R_{t}, was defined as

$$R_{\mathrm{t}} = \frac{\Delta_0 - \Delta_{\mathrm{t}}}{\Delta_0} \times 100\% \tag{4.3}$$

where Δ_{t} is the rate of sliding t years after construction was completed.

The result was plotted in Figure 4.7. Separate plots were produced for rockfill column shear keys (a) and trenched shear keys (b). The value of P_{SK} for each of the case histories that are presented is shown in Table 4.5.

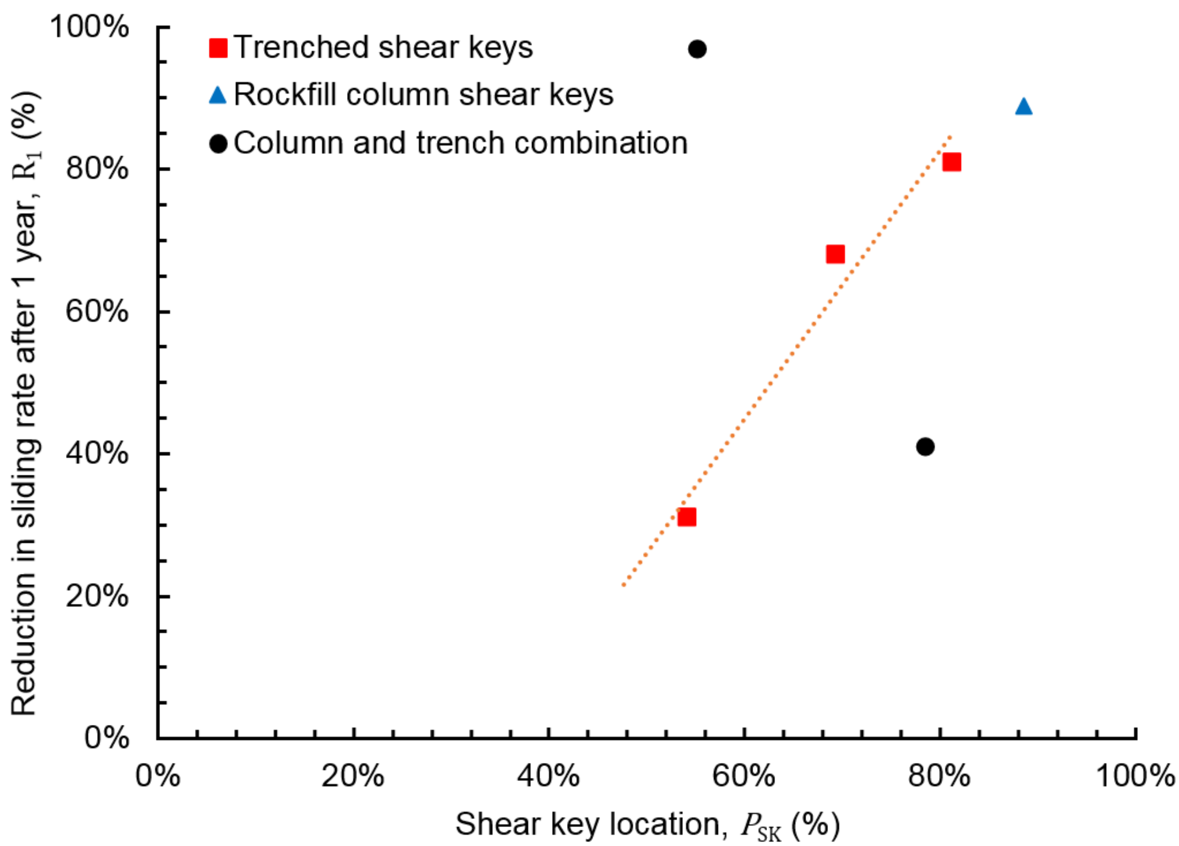


Figure 4.6: The percentage reduction in the rate of sliding one year after construction, plotted against the location of the shear key.

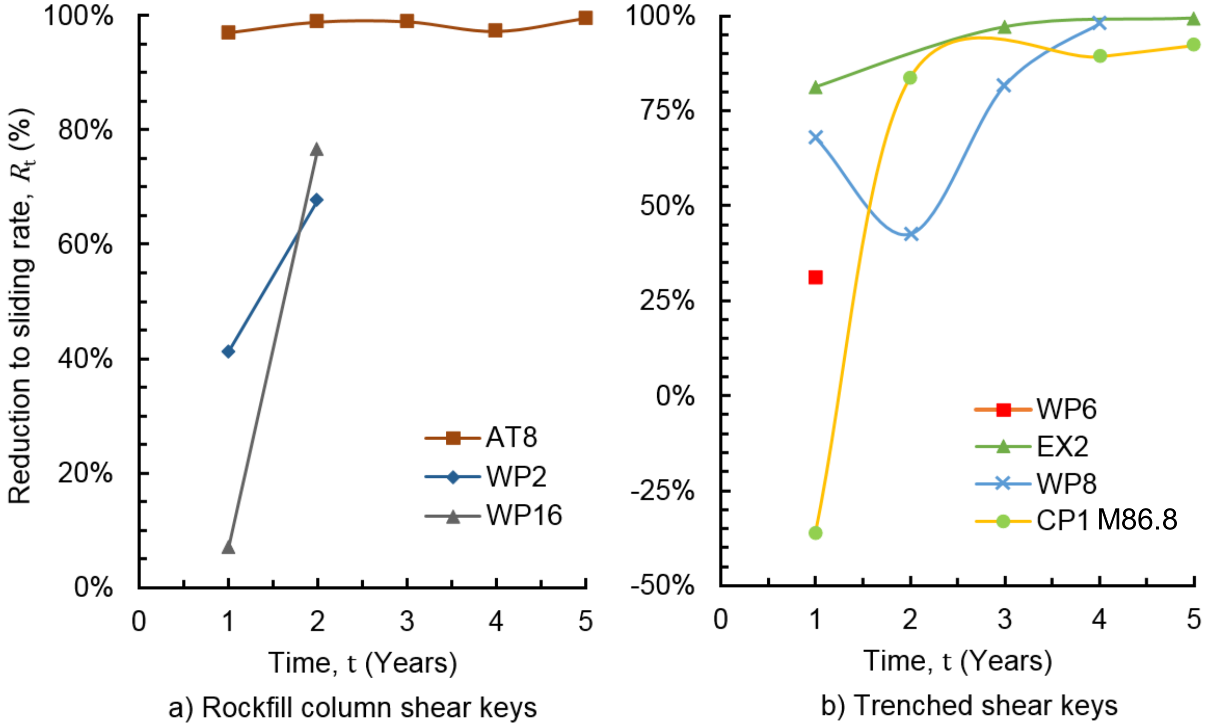


Figure 4.7: The percentage reduction in sliding rates, plotted over time for rockfill column shear keys (a) and trenched shear keys (b).

Table 4.5: Summary of the percentage reductions in the sliding rates over time.

			R_t (%)				
Case history	Repair	P_{SK} (%)	Year 1	Year 2	Year 3	Year 4	Year 5
AT8	Columns	55.1	96.9	98.8	98.9	97.1	99.5
WP2	Columns	78.3	41.1	67.7	nd	nd	nd
WP16	Columns	88.6	6.9	76.7	nd	nd	nd
WP6	Trench	54.0	31.2	nd	nd	nd	nd
CP1 M86.8	Trench	57.1	-36.1	83.7	nd	89.3	92.3
WP8	Trench	69.3	68.1	42.7	81.9	98.1	nd
EX2	Trench	81.0	81.2	nd	97.2	nd	99.5

nd: No data

Overall, the trends that had previously been observed were seen again. Plot (a) of Figure 4.7 showed that rockfill columns constructed closer to the toe of a slide had a delayed response in reducing the rate of sliding relative to pre-construction rates. The opposite was expressed in Plot (b) of Figure 4.7, with trenched shear keys instead achieving a greater reduction to the rate of sliding the closer they were to the toe of the slide.

The relation observed between the performance of shear keys and their relative location within a slide may be related to several aspects of their design. First, rockfill columns are typically designed to resist shearing at greater depths than trenched shear keys. This is because the volume of the excavation for rockfill columns is relatively insensitive to depth compared to trenched shear keys. The improved performance for rockfill columns constructed further from the toe might be explained by the increase in the effective depth, since the slip surface tends to be deeper further upslope. On the other hand, trenched shear keys are typically placed at the toe of the slide to minimize the excavation volume. For a given excavation volume, the effective width could be increased as the depth was decreased. Thus, the improved performance of trenched shear keys constructed closer to the toe of the slide may be a result of wider designs. This is investigated in the next section.

The results may have also been influenced by differences in the pre-construction sliding rates between sites. For sites experiencing the highest pre-construction rates of sliding, a discrete reduction in the rate of sliding would equate to the smallest percentage reductions. To account for this possibility, the relative location of the shear key was plotted against the time it took for the rate of sliding to decrease below 1 mm/year, consistently.

The results are shown in Figure 4.8 and they suggest a potential optimum location for a shear key at a P_{SK} value of 68%. Even at this location though, the shear keys took anywhere from 0.5 to 4 years to reach a rate of 1 mm/year. This optimum location may provide some support for the recommendation to construct shear keys at the point where the slip surface runs horizontally, for enhanced long-term performance.

Only 11 case histories are plotted in Figure 4.8. This is because displacement data was not available for all case histories, and not all case histories which did have displacement data reached a sliding rate of 1 mm/year. The shear key location, P_{SK}, was known for most of the sites though so the maximum and minimum values were labelled to add context to the points that were plotted. The average shear key location and time to reach a sliding rate of 1 mm/year were also plotted as a point, with error bars indicating one standard deviation, σ, in either direction.

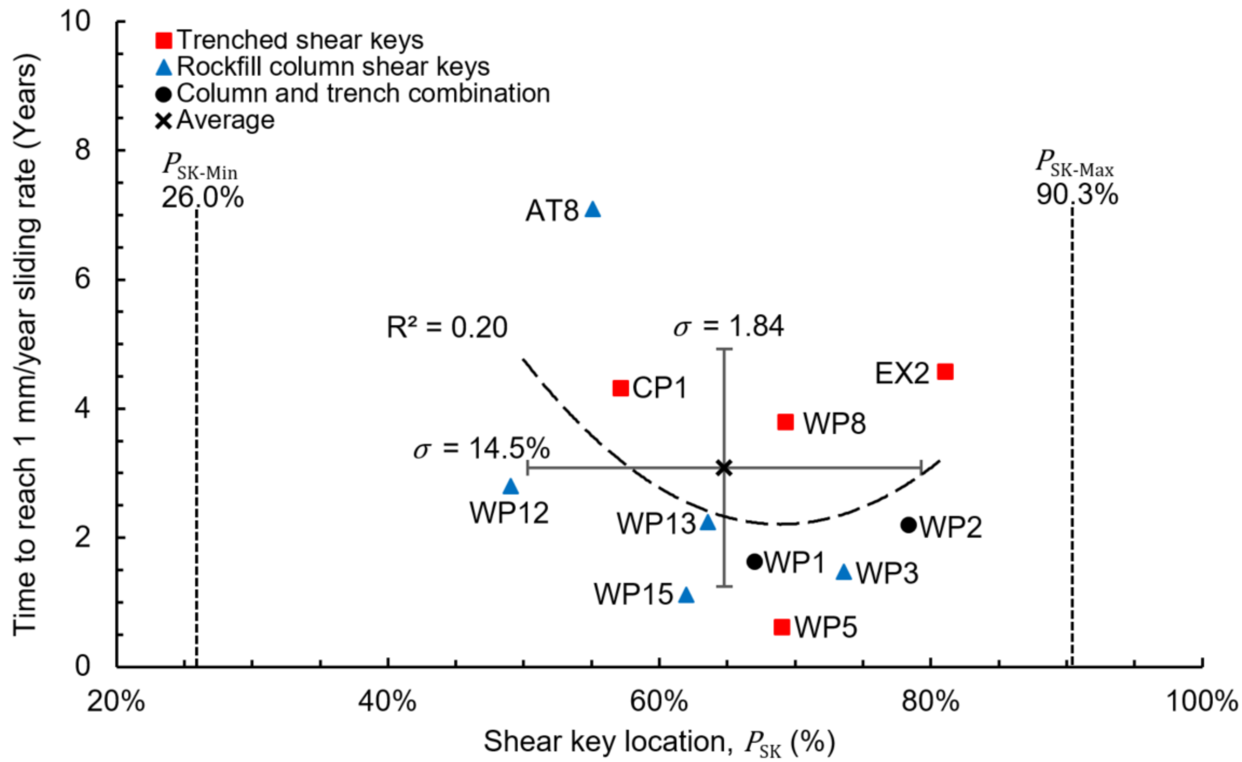


Figure 4.8: The time in years required for the rate of sliding to fall below 1 mm/year, plotted against the location of the shear key. A quadratic fit is shown as a dashed line. Case histories for which a sliding rate of 1 mm/year was not achieved or for which displacement data was not available are not plotted, so the maximum and minimum values of P_{SK} are labelled. The average for all case histories is also shown, with error bars representing one standard deviation, σ, in both the x and the y directions.

4.2.2 Determining shear key size for resistance and performance requirements

The following section presents the analyses performed to investigate how the dimensions of a shear key can be modified to satisfy different shear resistance requirements, and how those changes affect performance. The dimensions that were the primary focus of these analyses were the effective width and the effective depth. Performance was evaluated in terms of the strain that was exhibited, and the length of time it took for that strain to develop.

Satisfying shear resistance requirements

A series of analyses were performed to search for trends involving shear key width and depth. In Chapter 3, increasing the effective width and the effective depth were both said to increase the shear resistance of a shear key. In §4.2.1, it was postulated that shallower shear keys may tend to be designed wider, as compensation for the smaller effective depth.

To investigate the relation between shear key width and depth, new subsets were created from the database. Histograms were used to query the effective depth to identify reasonable bins from which to create these subsets. A bin size of 1 m provided a reasonable balance between encompassing an adequate number of points and constraining the effective depth to a tight range. This resulted in subsets being created where the effective depth was relatively constant, while the effective width varied.

Next, a way in which to represent the required shear resistance needed to be selected. The required shear resistance is directly related to the driving force so the size of the slides was selected as a proxy. Another proxy for the driving force would be the inclination of the natural slope normalized to the residual friction angle of the soil. It was reasoned that practitioners would find more value in having a comparison between the sizes of landslides and the sizes of the shear keys designed to remediate them. To simulate the fact that shear keys are designed in cross section, the cross-sectional area was used to represent the landslide size. The cross-sectional area was calculated as one half of the product of the midslope slip surface depth and the slide length at surface, as follows:

$$A_{\text{Cross section}} = \frac{1}{2}(D_{\text{Midslope}} \cdot L) \tag{4.4}$$

where $A_{\text{Cross section}}$ is the cross-sectional area of the slide, D_{Midslope} is the midslope slip surface depth, and L is the slide length at surface.

These dimensions are shown in Figure 4.9. D_{Midslope} is depicted as being a third of the horizontal length of the slide away from the scarp, but it should be noted that the actual location of the measurement for this dimension was highly variable due to it being measured from the nearest available SI.

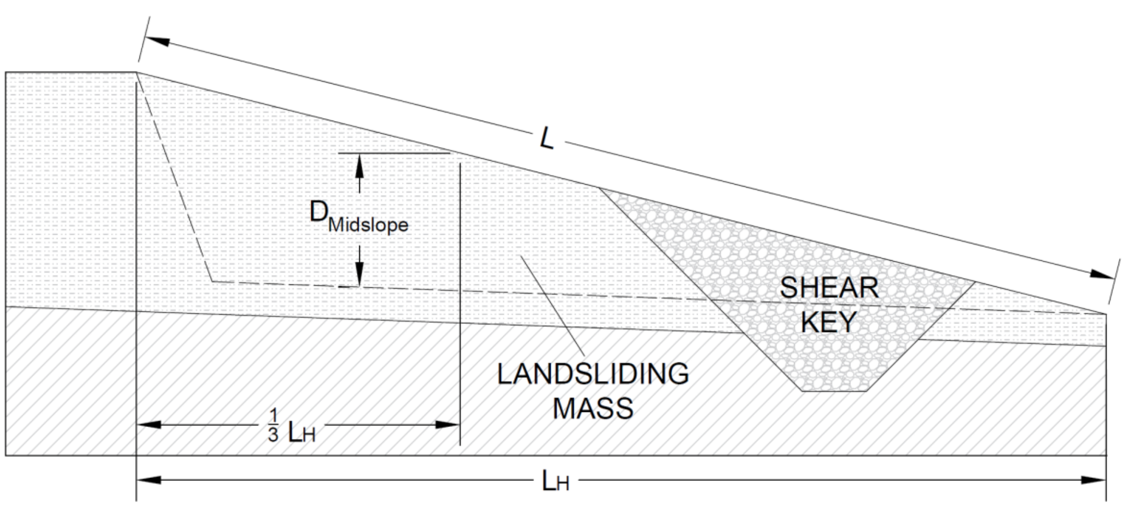


Figure 4.9: The dimensions used to estimate the cross-sectional area of a landslide.

The cross-sectional area was calculated in this way for consistency and to also account for the likelihood that only a single SI might be available with which to evaluate the depth of sliding. The slide length at surface can also be estimated or measured with reasonable ease.

Plotting the cross-sectional area of the slides against the effective width of the shear keys designed for those slides (see Figure 4.10), several relations were observed. First, positive linear relations were exhibited by each effective depth subset, confirming that wider shear keys are designed to stabilize larger slides. It was also found that as the effective depth decreased, the effective width increased in greater proportion to the slide cross-sectional area. This indicates that for shallower slip surfaces, shear keys are designed to be wider, confirming what was postulated in §4.2.1.

Three outliers were present in the data plotted in Figure 4.10. The three sites in the order they are labelled in the figure are WP14, CP1, and AT4. WP14 does not appear to outlie significantly and has an effective width of 6.2 m, which is close to the 6-m-threshold separating the two data subsets. The CP1 data point is one of two shear keys plotted for that site and is the only one that outlies. The shear key represented by this point appears to be too wide compared to the rest of its subset, but its effective width of 4.1 m also places it relatively close to the 4-m-threshold between subsets. Furthermore, this shear key is directly under railway tracks whereas the other is located downslope of the tracks. This shear key may have been designed with a greater width so none of the native soil would be left below the track, risking differential settlement. The third outlier, AT4, is for a shear key stabilizing a slope in which a large-diameter culvert is present. The increased width may have been selected to compensate for a low tolerance for deformation, lest the culvert sustain damage.

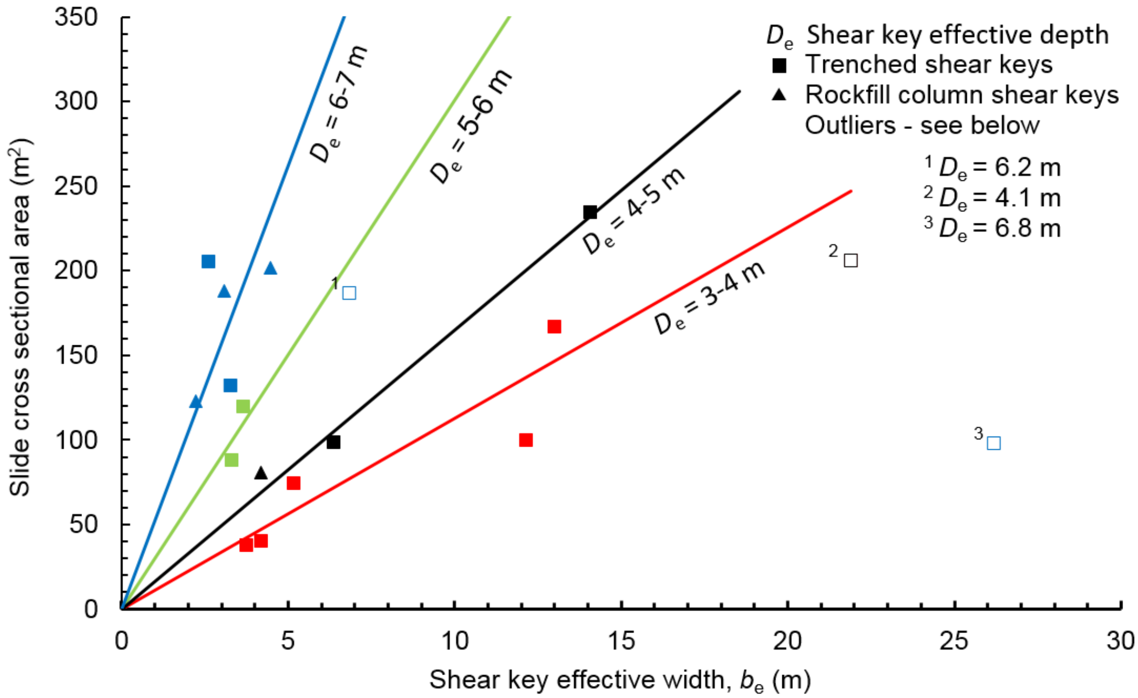


Figure 4.10: The cross sectional area of each landslide was plotted against the effective width of the shear key designed to mitigate it. The data was subdivided into smaller subsets based on 1-m-wide ranges for the effective depth.

Following the results of this analysis, the slide cross-sectional area was plotted against the product of the effective width and the effective depth (Figure 4.11). For clarity, the product of the effective width and the effective depth is referred to as the shear key cross-sectional area key for the remainder of the document. The sites that had previously been identified as outliers were omitted. It was expected that taking the product of these two dimensions would combine the two relations that had been identified previously; between the effective width and effective depth, and between the slide cross-sectional area and the shear key effective width. Both relations were roughly linear, so another linear relation was expected. A roughly linear trend at a slope of 4:1 emerged for shear key cross-sectional areas less than 60 m^2. For the full range of shear key cross-sectional areas encompassed in the case histories, the trend was exponential (Figure 4.12).

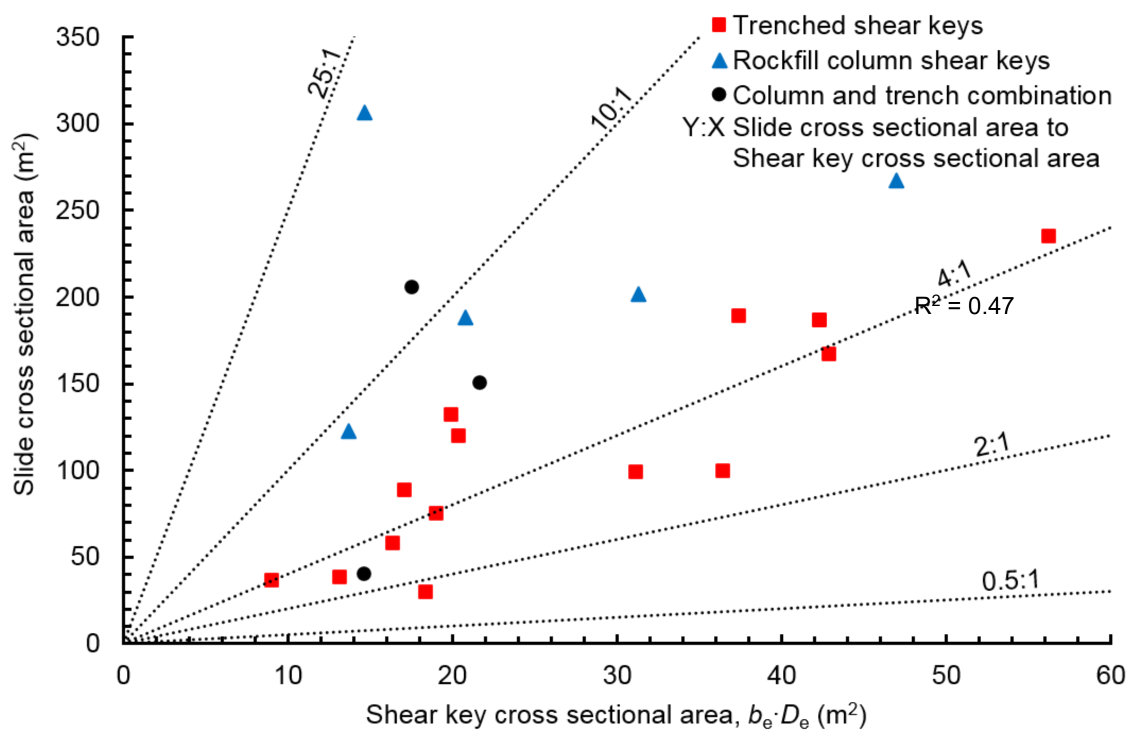


Figure 4.11: The cross-sectional area of the slides was plotted against the cross-sectional area of the shear keys designed to mitigate them. A roughly exponential trend can be seen in the trenched shear key data.

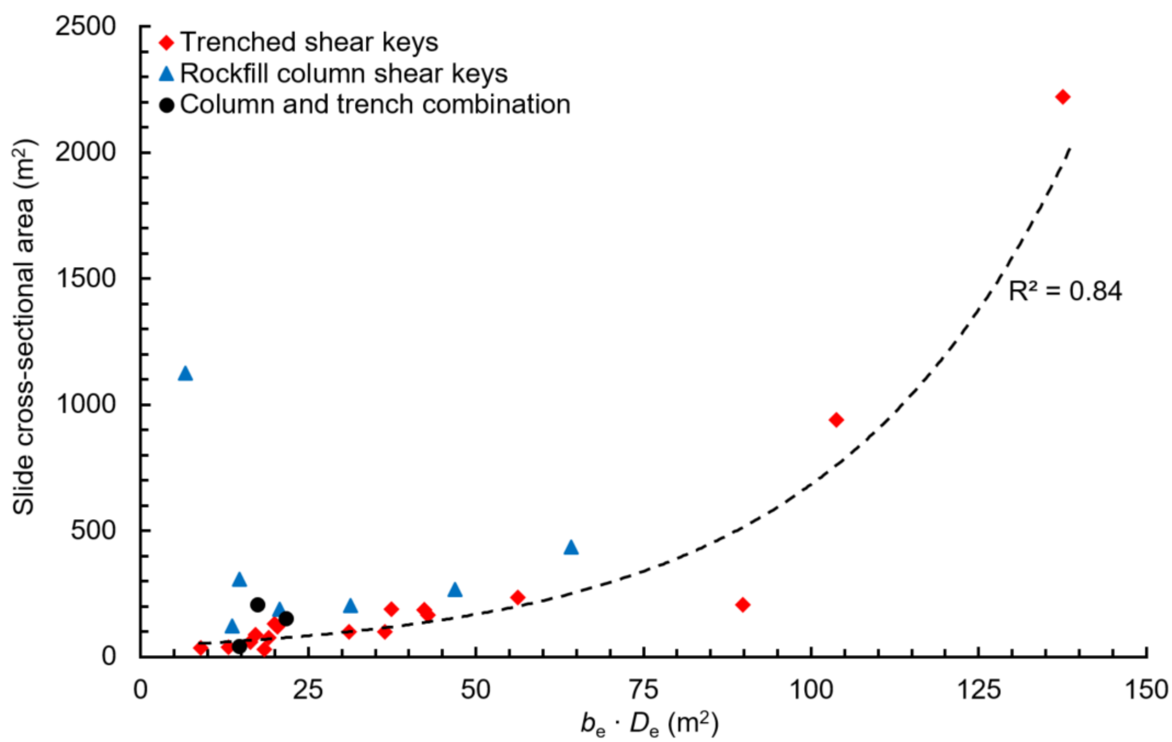


Figure 4.12: The slide cross-sectional areas plotted against the full range of shear key cross-sectional areas appears to follow an exponential curve.

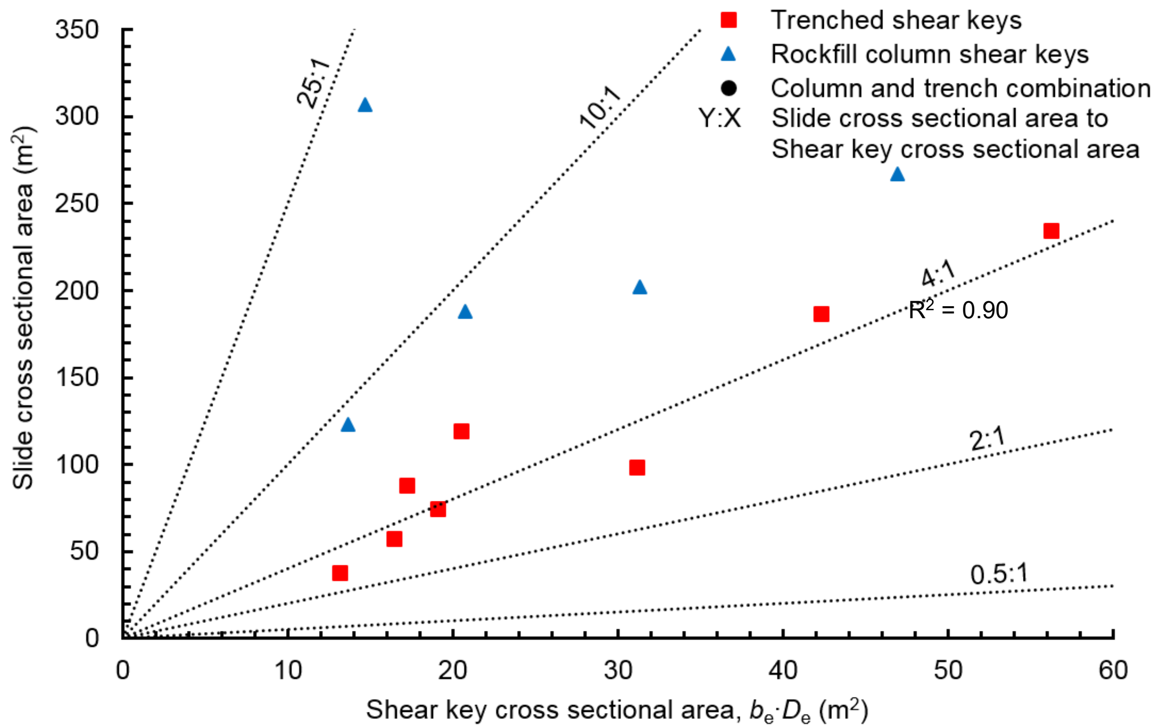


Figure 4.13: The slide cross-sectional areas plotted against the shear key cross-sectional areas below 60 m² for the filtered dataset.

A filter was then applied that excluded sites with berms or extensive regrading, and where significant slope drainage was relied upon in attaining the stability target. The filtered data can be seen plotted in Figure 4.13, where it appeared to follow a cleaner linear trend, particularly for trenched shear keys. The rockfill column sites appeared to follow another linear trend, translated slightly upwards. This translation is perhaps due to the consideration of soil arching between columns, which would permit more efficient use of rockfill by volume.

The results presented thus far do not distinguish the data by the factor of safety for which each site was designed to meet. By sorting the data by the factor of safety, the plot was expected to facilitate estimating the granular shear key dimensions that exceed the minimum shear resistance by a specified factor. The filtered data was sorted, which resulted in Figure 4.14 for trenched shear keys. The rockfill column shear keys for which the target factor of safety could be obtained were all designed to satisfy a factor of safety of 1.3, so this data is not presented.

In contrast with the expected result, the data for the trenched shear keys plotted in a relatively straight line along which the factor of safety decreased as the slide cross-sectional area increased. This can be interpreted as showing it becoming increasingly difficult to achieve higher factors of safety as the size of the slide increases. Factors influencing the magnitude of the target factor of safety discussed in § 2.4.1 (Chapter 2) included the size of the landslide. Lower factors of safety were suggested for larger landslides, and this is reflected in the data plotted in Figure 4.14.

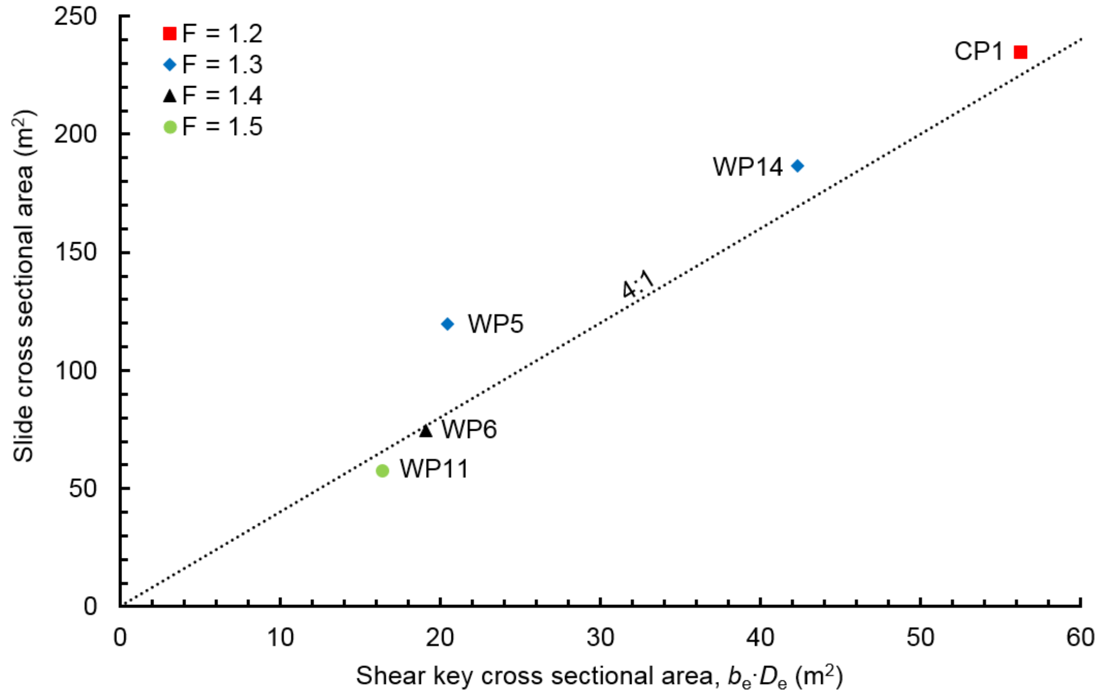


Figure 4.14: Landslide cross-sectional areas plotted against shear key cross-sectional areas sorted by the target factor of safety, F, which the corresponding shear keys were designed to satisfy.

The relationships between the size of the slides and the size of the shear keys were identified despite not accounting for the landslide soil or backfill properties. Disregarding the landslide soil properties likely did not obscure the relationships that were identified because the reported residual friction angles did not vary considerably; the average was 12.1° with a standard deviation of 3.2°. The variation in the reported backfill friction angles was also small, averaging 37.1° with a standard deviation of 2.3°. However, the length of the slip surface passing through the backfill typically constitutes a relatively small portion of the total length of the slip surface and thus has a comparably smaller impact on stability.

The plots thus far can be used to estimate the required shear key cross-sectional area to stabilize a slide of a certain size. The shear key dimensions have not yet been constrained in proportion to one another. To investigate the potential impact of favouring one dimension over the other (for example, a large shear key effective width rather than a large shear key effective depth), the shear key dimensions were plotted on their own against several performance indicators in the following section.

Satisfying performance requirements

Having established an apparent relationship between landslide size and shear key size, the next set of analyses focused on relating the effective width and the effective depth to performance. Several different performance indicators were selected for these analyses, to provide more comprehensive representation of the different stages of post-construction. The performance was evaluated in terms of the length of time over which movement took place, and the amount of strain that was exhibited. The performance indicators relating to time were:

- The percentage reduction in the sliding rate one year after construction (using Equation 4.2),
- The number of years it took for the rate of sliding to decrease below 16 mm/year (Varnes Class I – Extremely Slow), and
- The number of years it took for sliding to effectively stop (< 1 mm/year).

Shear strain, ε_{s}, was calculated as a percentage of the as-built shear key effective width, as

$$\varepsilon_{\mathrm{s}} = \frac{\Delta x}{b_{\mathrm{e}}} \times 100\% \tag{4.5}$$

where Δx is the displacement measured in a slope inclinometer.

The dimensions Δx and b_{e} are illustrated schematically in Figure 4.15. The strain was calculated for when the rate of sliding decreased below 16 mm/year and again for when it decreased below 1 mm/year.

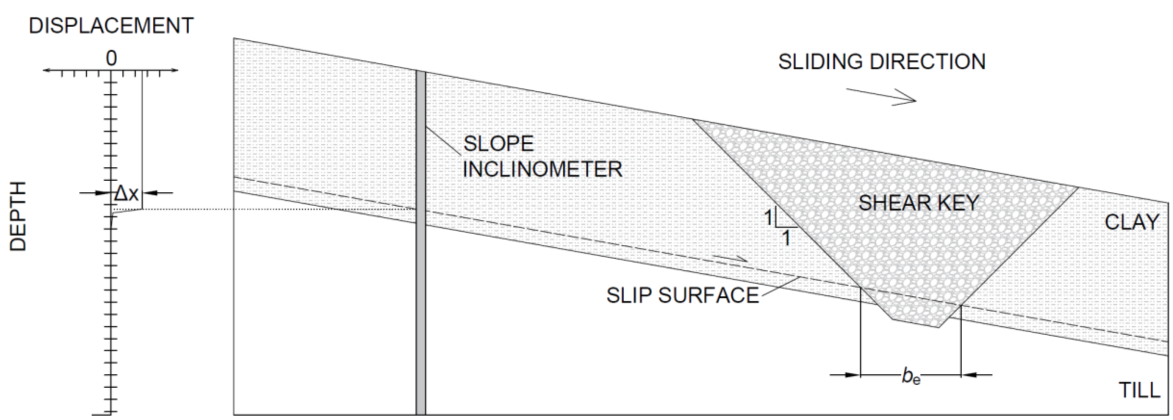


Figure 4.15: Strain was defined as the displacement, Δx, measured from slope inclinometer data normalized to the shear key effective width, b_e.

The shear key cross-sectional area was plotted against these performance indicators first. No clear relationships could be identified for the percentage reduction in the sliding rates one year after construction. The longer-term plots seemed to show that as the shear key cross-sectional area increased, so did the time required for sliding rates to decrease below 16 mm/year and 1 mm/year, respectively. The complete dataset was plotted first, to verify the presence of a general trend. Once this was verified, the filtered dataset was plotted to further refine that trend. The two plots in Figure 4.16 were created using the complete dataset, while Figure 4.17 was created using the filtered dataset. The results suggest larger shear keys require more time to stabilize.

For these analyses, the different landslides that were plotted were sliding at various rates before the granular shear keys were constructed. To address this variability, the times were normalized to the pre-construction rates of sliding. The result was highly scattered though and no trend could be identified. This suggests the pre-construction rates do not have an impact on the times required for the shear keys to reduce the rate of sliding below the two velocity thresholds that were selected.

The next analyses aimed to investigate the individual effects each of these dimensions have on performance. The complete dataset was used for this since it appeared to mirror the trends identified in the filtered dataset but the greater sample size facilitated the identification of these trends. From the results, it appeared an increase in the effective width resulted in the granular shear keys taking longer to reduce the rate of sliding (see Figure 4.18). The effective depth did not appear to have any effect (Figure 4.19).

It has been well-established that increasing applied normal stresses result in decreasing friction angles due to reduced dilatancy during shear (Bolton, 1986; Charles & Watts, 1980; Rowe, 1962). Thus, it was expected that the strain required to mobilize the shear resistance required for equilibrium would increase with normal stress. This was confirmed in the plots presented in Figure 4.20, where the effective depth was plotted against the strain-related performance indicators. No relationship was identified when the effective depth was plotted against strain.

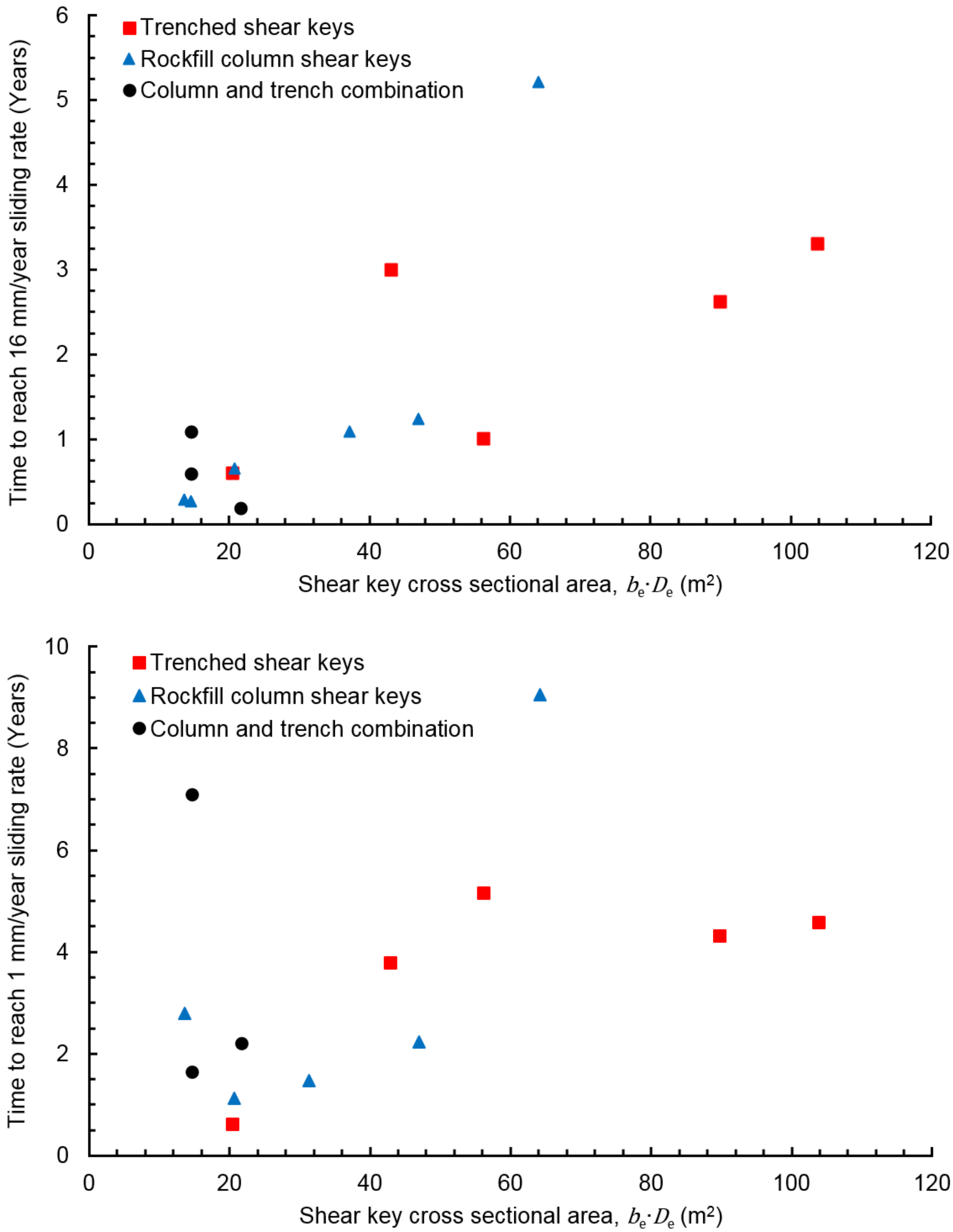
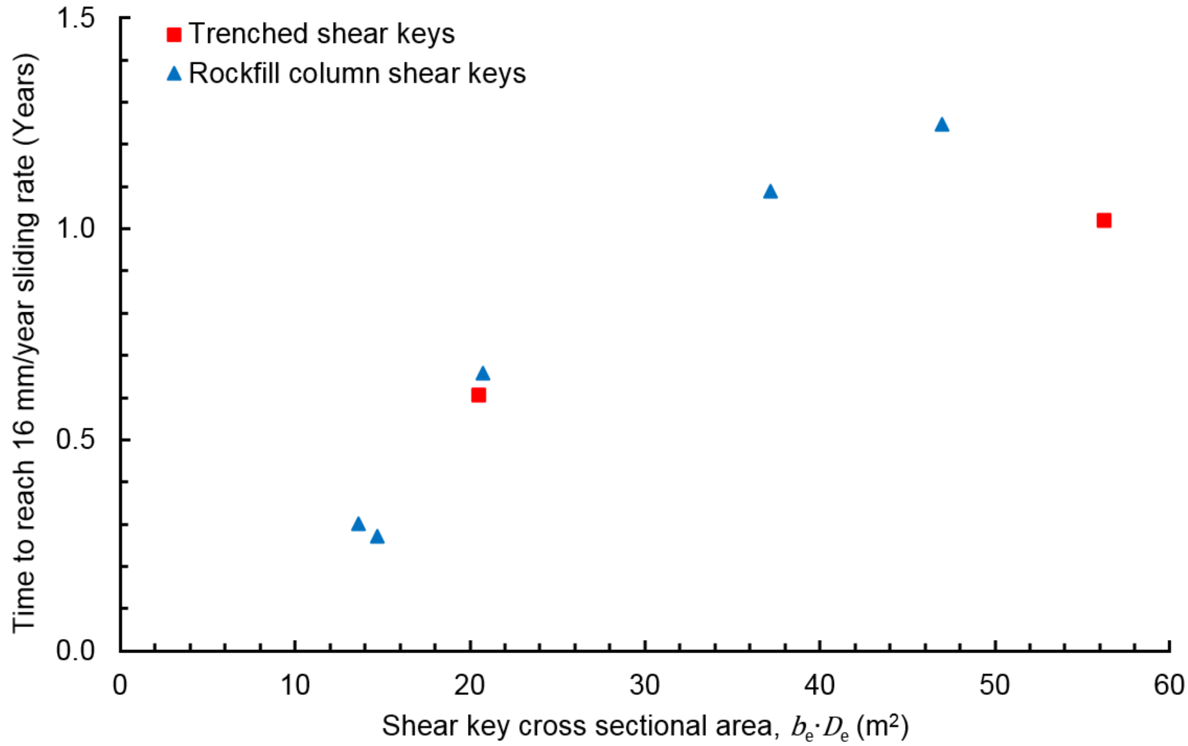


Figure 4.16: Shear key cross-sectional areas plotted against the time required for the corresponding granular shear keys to reduce the rate of sliding to below 16 mm/year (top) and 1 mm/year (bottom).



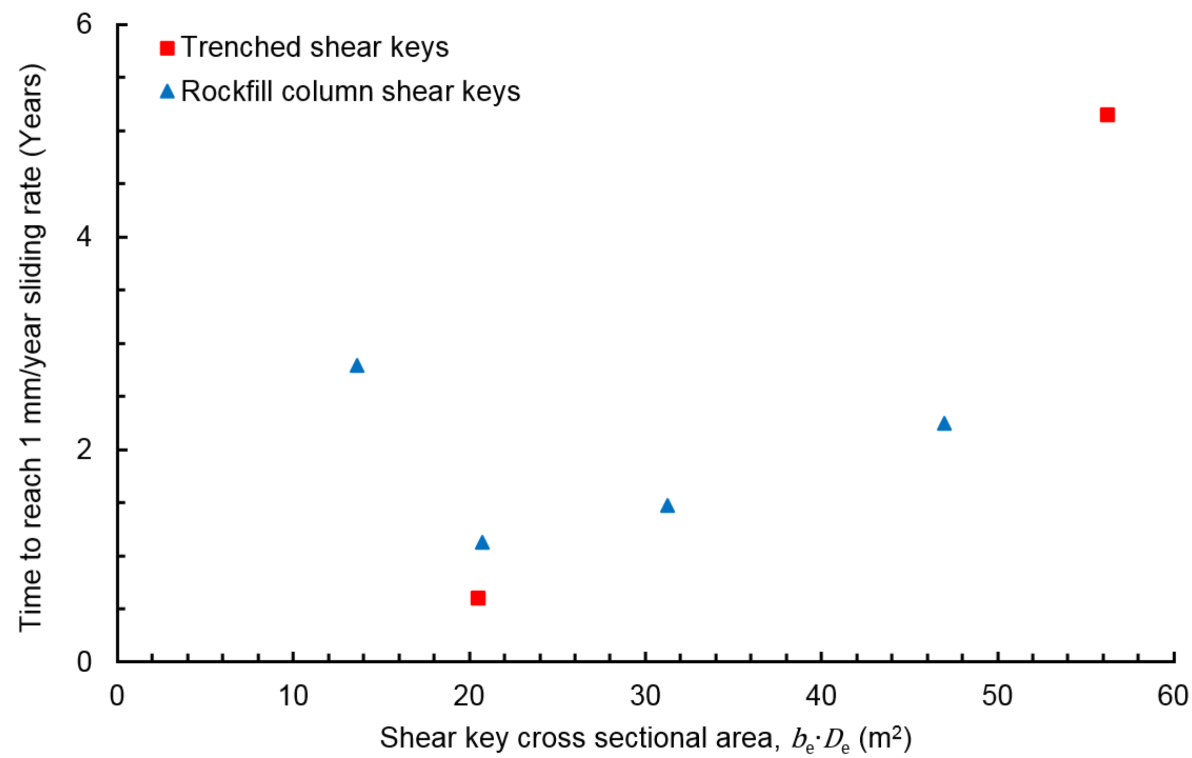


Figure 4.17: The time until the rate of sliding had decreased below 16 mm/year (top) and 1 mm/year (bottom) plotted against the product of the shear key dimensions, for the filtered dataset.

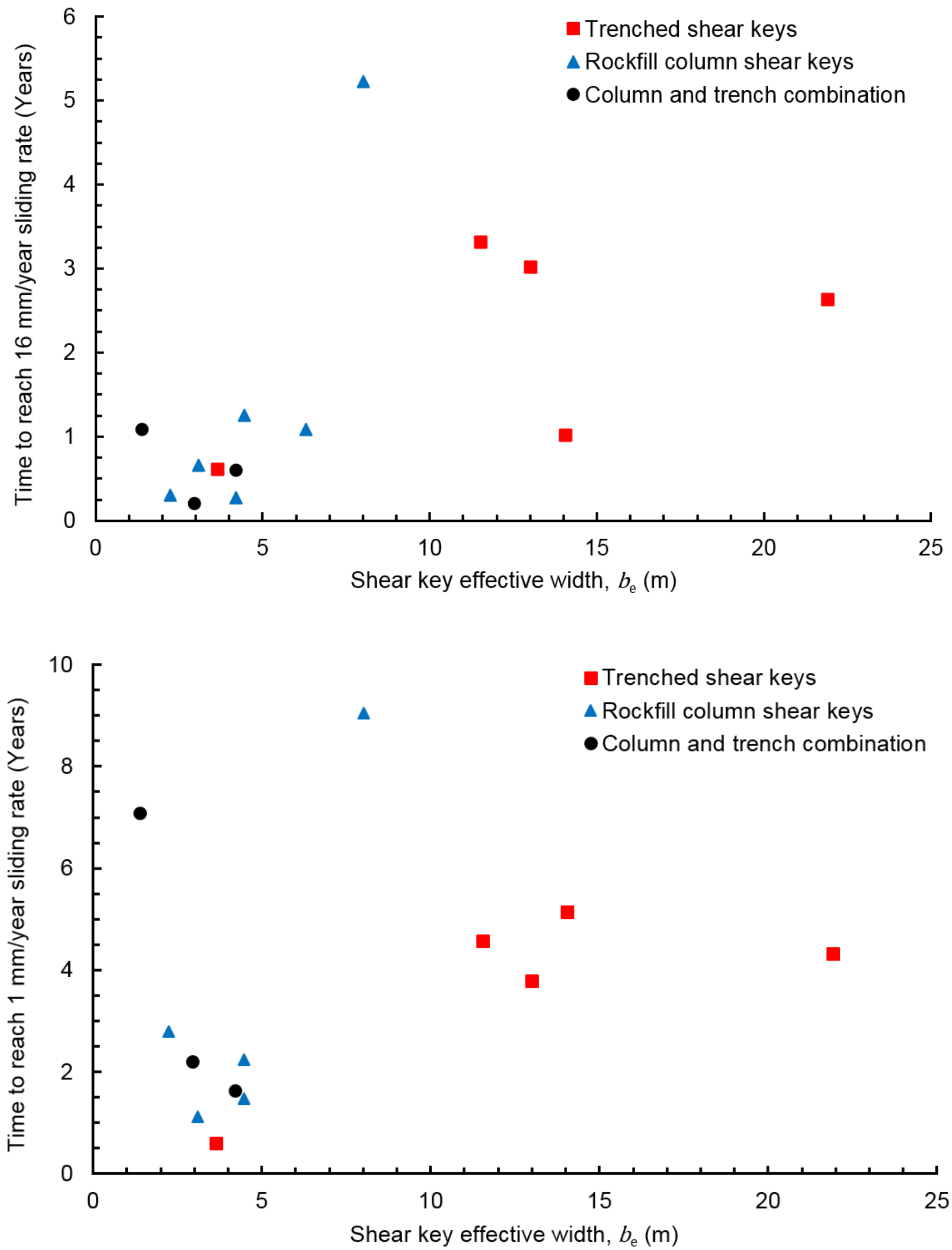


Figure 4.18: Plots showing a moderate correlation between an increase in the effective width and the time required for the shear key to stabilize.

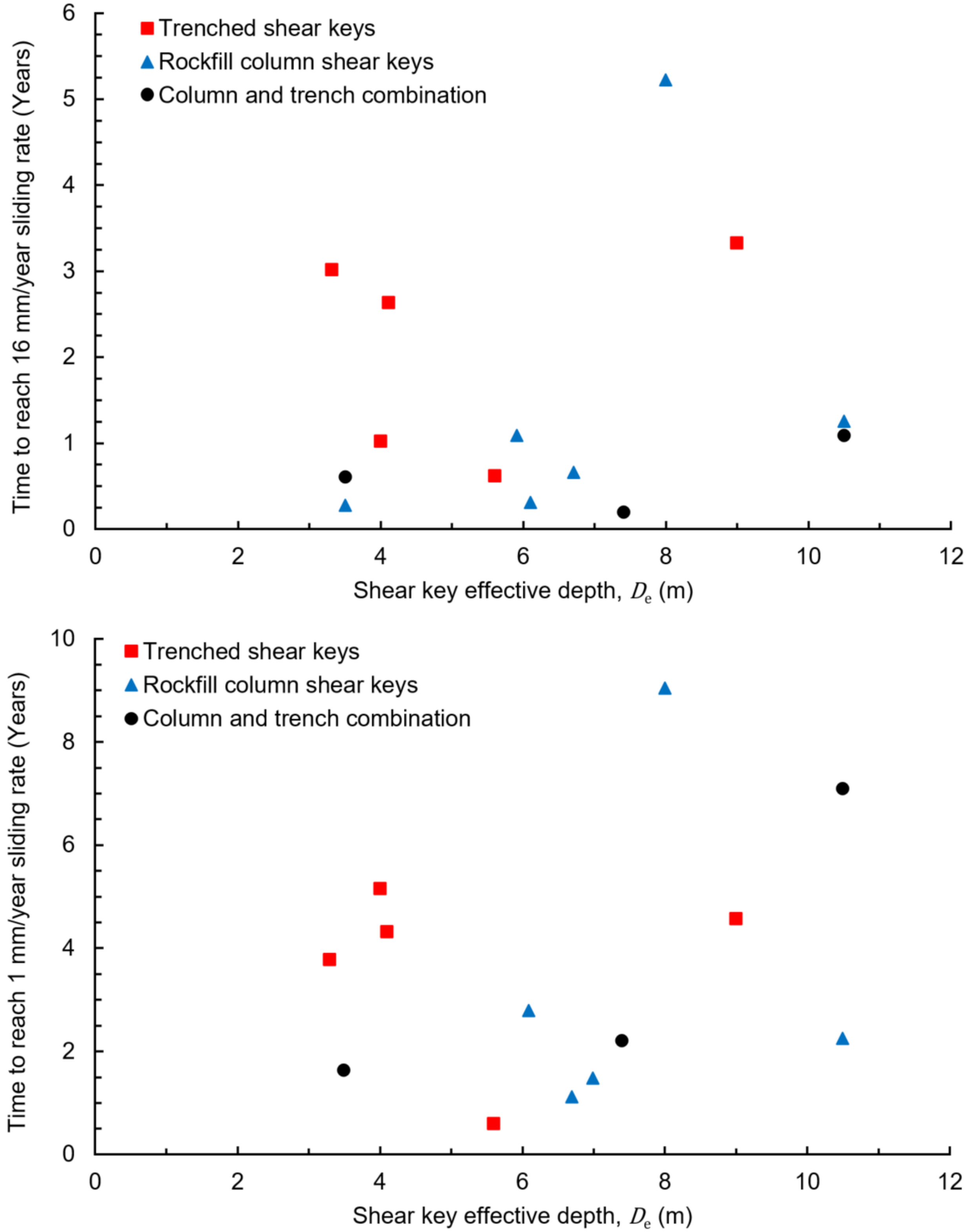


Figure 4.19: Plots showing no correlation between the effective depth and the time required for the shear key to stabilize.

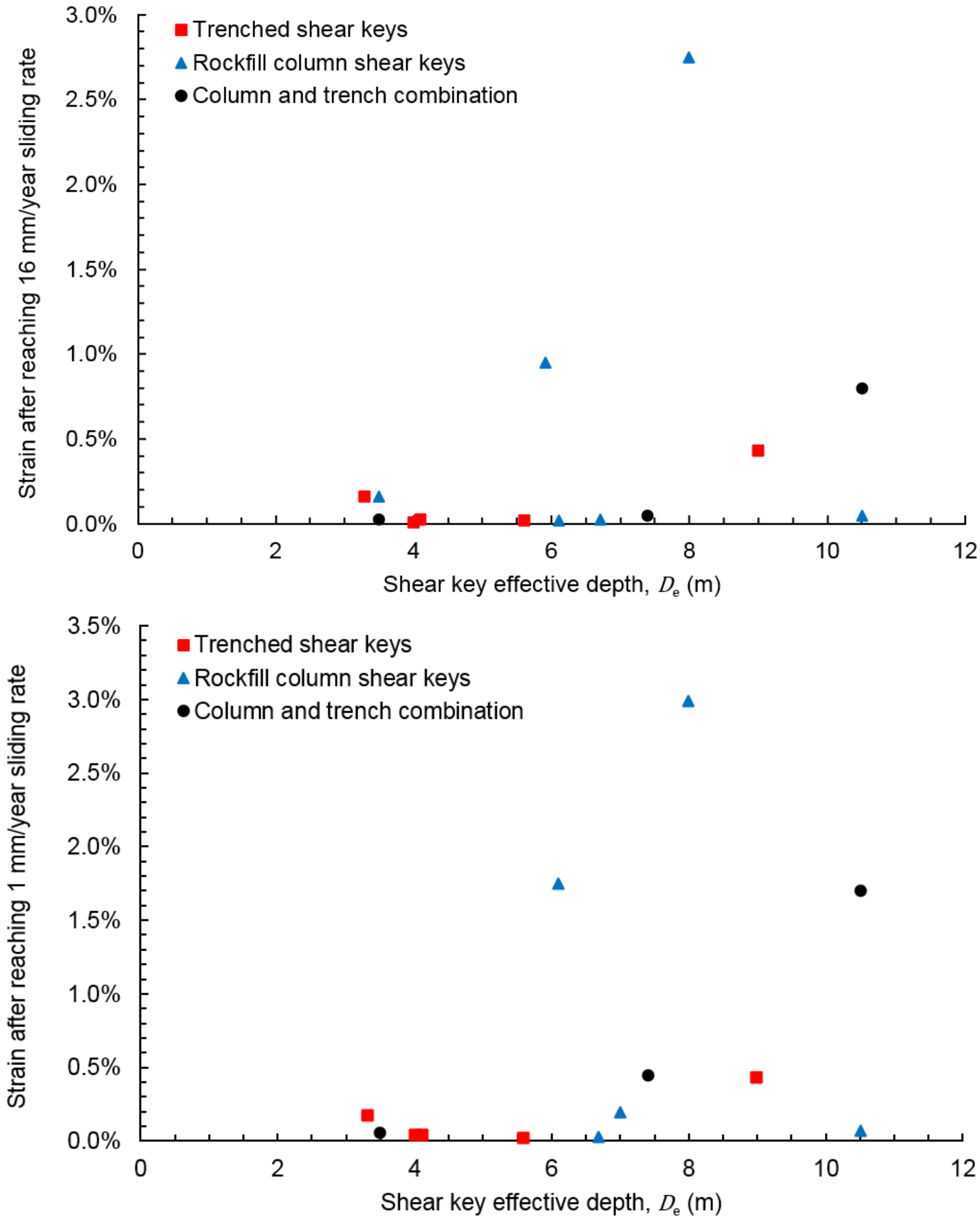


Figure 4.20: Plots showing a generally relation between greater effective depths and increased strain being sustained after the rate of sliding had decreased to velocity thresholds of 16 mm/year (top) and 1 mm/year (bottom).

The rate of sliding was typically observed decreasing over several years before steadying out at a final rate of sliding. Some sites did not reach the thresholds that had been set and were not included in the previous analyses. To consider these sites, the next analysis considered the strain that was experienced by the granular shear keys after the rate of sliding had become steady. The relation that had been observed in Figure 4.20, where deeper shear keys experienced greater strains, was observed once again in the results of this latter analysis (see Figure 4.21). To further refine this relationship, the plotted data was sorted by whether it had been reported that the backfill was compacted. This was expected to account for some of the differences in the stiffness and density of the different backfills. The sites where the backfill was compacted were expected to exhibit less strain than those with loose backfill, at an equivalent effective depth.

The plot of the percentage strain after a steady rate of sliding had been reached versus the shear key effective depth can be seen in Figure 4.21. For the sites with less than 0.5% strain, the compacted shear keys were designed with greater effective depths than the uncompacted shear keys but experienced roughly the same amount of strain. Strain was expected to increase with effective depth, so this observation may confirm that compaction does reduce strain.

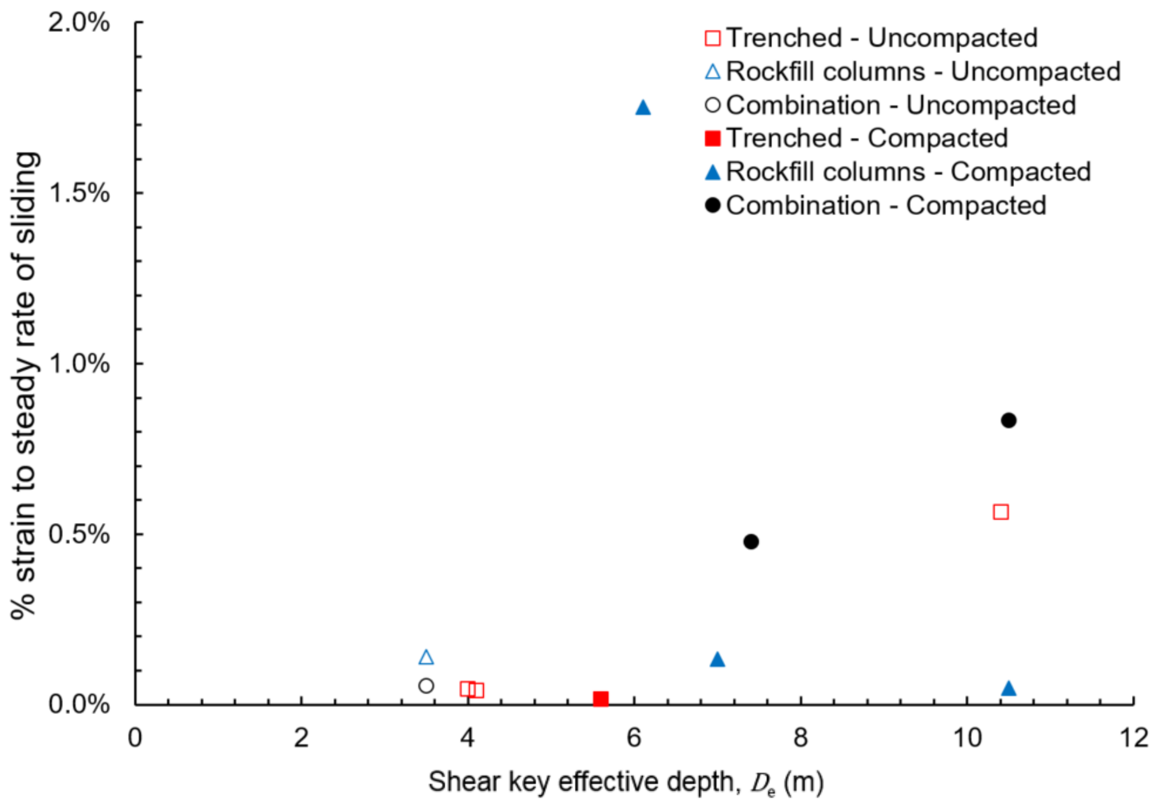


Figure 4.21: The strain exhibited after a steady rate of sliding had been reached, plotted against the granular shear key effective width. Sites where compaction was reported were scattered.

The uncompacted shear keys follow a positive linear trend. This may be coincidental due to the small number of points, but it is also concurrent with theory and previously observed trends. The sites where the backfill was compacted were highly scattered, possibly reflecting differences between the compaction methods that were used. Some variation was also expected from different materials being used. This was further investigated in the next section, which focused on the effects of the backfill materials that were used.

4.2.3 Observed effects of backfill material properties

The selection of an appropriate backfill material was discussed in Chapter 3. The backfill was expected to chiefly affect the size of the shear key needed to satisfy the stability target. It was also expected to have some influence on shear key performance. There are several different backfill-related factors that were investigated. They are the type of backfill used, the contrast between the friction angle of the backfill and of the landslide, the density and stiffness of the backfill, and the mobilization of shear strength.

The effect of different backfill friction angles

To investigate the effect different types of backfill have on shear key size, the landslide cross-sectional area was plotted against the shear key cross-sectional area once again (Figure 4.22). The data was then sorted by the different backfill materials that were used, rather than by the type of shear key technique that was used.

The backfill materials used in the case histories were sorted into three categories: crushed gravel, other gravel, and sand. Some were not specified but were nevertheless plotted as well, although they were kept separate. From Figure 4.22, granular shear keys backfilled with crushed gravel or other types of gravel, such as pit run, were smaller than those which used sand. The shear keys backfilled with gravel could not be differentiated despite the appreciable differences between crushed gravel and pit run gravel. This can be explained by examining the friction angles of these types of materials.

In Chapter 3, the contrast between the friction angle of the native soil and the backfill material was said to affect the degree of improvement. The size of the shear key required to stabilize a given size of slide would be expected to decrease as the backfill friction angle increased. The friction angle for the granular backfill was not available for most of the case studies, so this theory could not be investigated directly. Instead, the friction angles reported in the literature for these types of materials were examined. For the materials compiled in Table 2.7, the peak friction angles for SP, SW, GP and GW soils at 1 atm of confining pressure averaged 39.8°, 50.3°, 49.4°, and 49.3°, respectively. There is almost no different between the friction angles for the gravels, thereby explaining why the gravels could not be distinguished in Figure 4.22. The sand sourced for a shear key would be clean and uniform. The lower average friction angle for the SP soils likely explains why the shear keys backfilled with sand were distinct from the rest of the dataset.

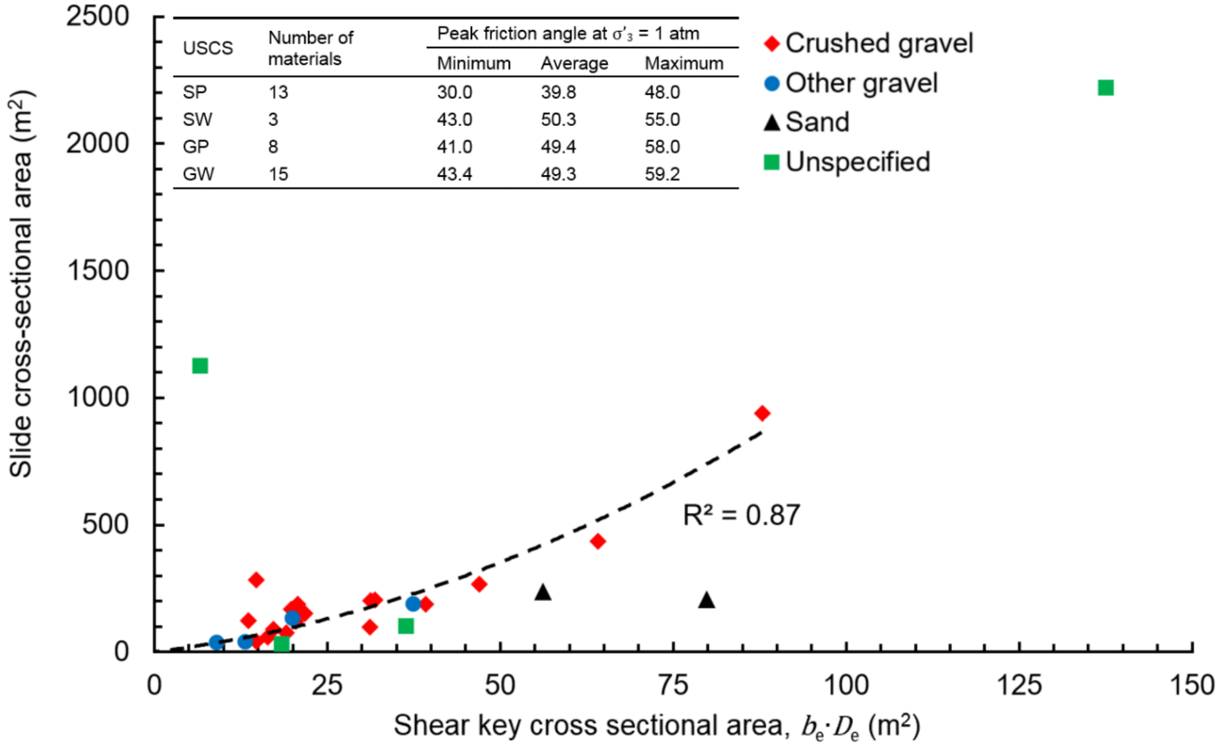


Figure 4.22: The slide cross-sectional area plotted against the shear key cross-sectional area, sorted by backfill material types.

The shear key cross-sectional area was also plotted against the residual friction angle that had been back-calculated for the slides (see Figure 4.23). The data was then sorted by backfill soil types. The results suggest landslides with similar landslide residual friction angles can be mitigated using smaller shear keys if gravel is used instead of sand. This probably reflects the contrast between the lower friction angle associated with the sand, and the greater friction angle for the gravel.

It can also be seen that for a given backfill soil type, the shear key size increases as the landslide residual friction angle increases. This trend is consistent with what would be expected using theory. If the backfill friction angle is held constant but the residual friction angle of the landslide increases, the contrast between the two decreases. To compensate, the size of the shear key must increase.

In Figure 4.23, one outlier was identified (circled). The outlier was EX2 and the shear key appears to have been designed much larger than the rest of the case studies. At this site, a small subdivision was located directly on top of the landslide and a mall was located immediately below the toe. A very low tolerance for movement may have justified the size of this shear key.

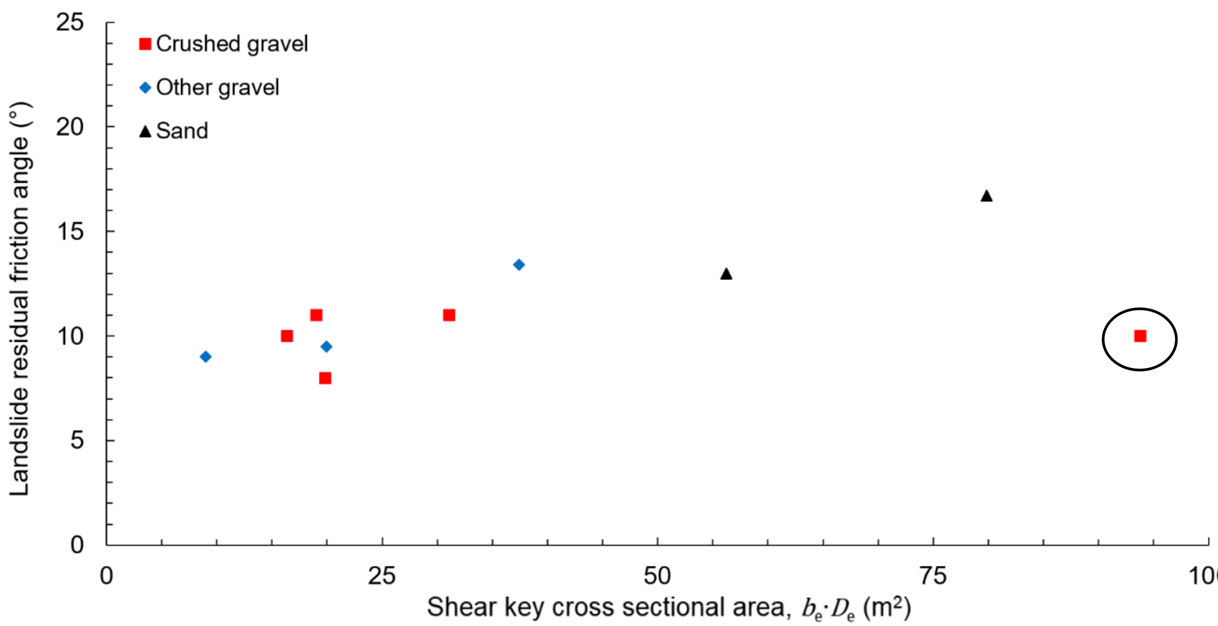


Figure 4.23: The back-calculated residual friction angles for landslides plotted against the product of the shear key dimensions. For landslides with roughly equivalent residual friction angles, gravel shear keys were smaller than those backfilled with sand.

The relationships identified between the residual friction angle of the landslide and the size of the shear key were also investigated using a LE analysis. The friction angle for the granular backfill was set at 45° and cohesion was assumed to be zero. The modelled slope was drawn at 3H:1V and was assigned a single material. The slope was placed above a competent material which was assigned infinite strength. The slip surface was specified to produce a wedge-shaped planar slide. For the slide, cohesion was set to zero and different friction angles were tested. For each friction angle, an idealized shear key was added to the system. The shear key was centered such that its location and the effective depth remained constant. The effective width was then modified until different target factors of safety were satisfied. Figure 4.24 shows the results for when factors of safety of 1.2 and 1.3 were targeted. The results appear to follow an exponential relationship and are generally consistent with what was observed in Figure 4.23.

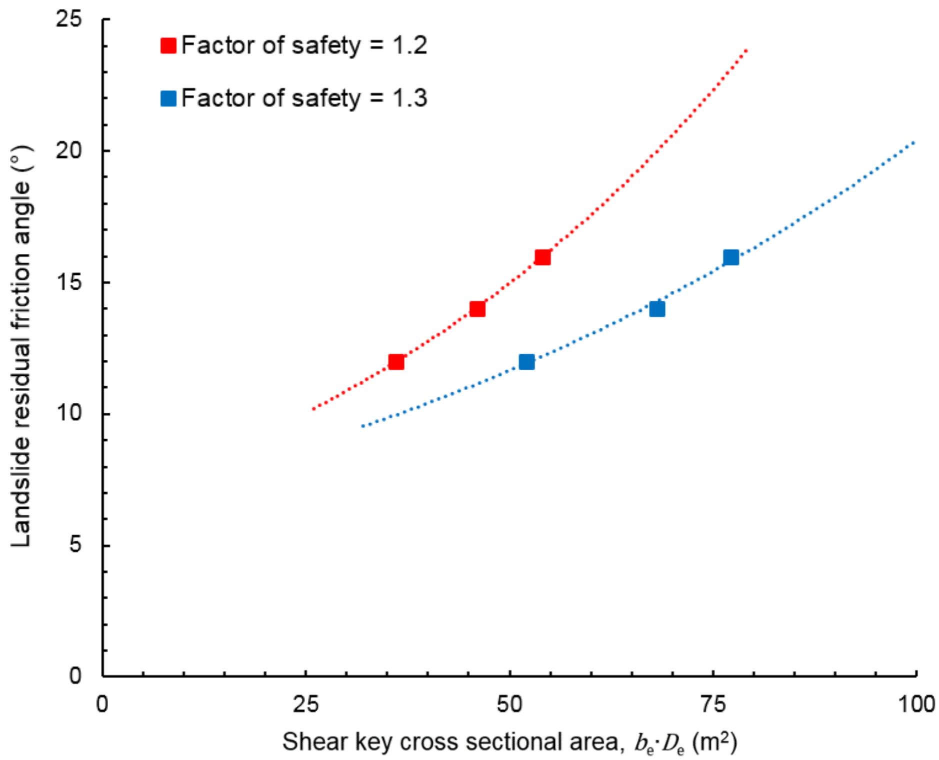


Figure 4.24: An exponential relationship was identified between the size of the shear key and the residual friction angle of the landslide. Increasing the target factor of safety resulted in larger shear keys being required, as expected.

The effects of relative density and stiffness

Another factor that affects performance is the relative density and stiffness of the backfill material. These parameters were expected to control the amount of strain it took to mobilize the shear resistance required for equilibrium. Whether the backfill was compacted or not would directly affect these parameters. Some variability in how well the backfill was compacted was expected between compaction methods, so the data was sorted by these methods. Statistics were generated for the time in years for the rate of sliding to decrease below 16 mm/year (shown in Table 4.6). The same was done for the time in years it took for the rate of sliding to decrease below 1 mm/year (not shown), for the percentage decrease in the rate of sliding one year after construction (Table 4.7), for the strain after a sliding rate of 16 mm/year had been reached (Table 4.8), and for the strain after the rate of sliding had become relatively steady (Table 4.9).

Table 4.6: The number of years it took for shear keys to slow the rate of sliding to below 16 mm/year was recorded and used to calculate basic statistics for comparing between different compaction methods.

Compaction method	Number of case histories	Years for the rate of sliding to reach 16 mm/year		
		Minimum	Mean	Maximum
Not compacted	6	0.27	1.42	3.32
Compacted	8*	0.20	1.60	5.22
Compacted**	7	0.20	1.08	3.01
Vibroflot	*4*	*0.20*	*0.71*	*1.25*
Other vibration	*3*	*0.61*	*1.57*	*3.01*
Drop weight	*1*	*5.22*	*5.22*	*5.22*

** The compaction method was unspecified for one of the case histories in this sample.*

*** Does not include Drop weight category.*

Table 4.7: Statistics for the percentage decrease in the rate of sliding one year after construction, for different compaction methods. A positive value indicates the rate of sliding decreased compared to pre-construction rates.

Compaction method	Number of case histories	Decrease in sliding rate 1 year post-construction (%)		
		Minimum	Mean	Maximum
Not compacted	2	-36	23	81
Compacted	6*	-29	50	97
Vibroflot	*2*	*41*	*69*	*97*
Other vibration	*2*	*68*	*79*	*89*

** The compaction method was unspecified for two of the case histories in this sample.*

Table 4.8: The shear strain after the rate of sliding had decreased to below 16 mm/year, used to compare the influence different compaction methods had on performance.

Compaction method	Number of case histories	Strain after reaching 16 mm/year rate (%)		
		Minimum	Mean	Maximum
Not compacted	6	0.01	0.14	0.55
Compacted	8*	0.01	0.57	2.74
Compacted*	7	0.01	0.26	0.95
Vibroflot	*4*	*0.01*	*0.13*	*0.43*
Other vibration	*3*	*0.02*	*0.44*	*0.95*
Drop weight	*1*	*2.74*	*2.74*	*2.74*

** The compaction method was unspecified for one of the case histories in this sample.*

*** Does not include Drop weight category.*

Table 4.9: The shear strain at the point in time when the slide had stopped decelerating, calculated as a percentage of the as-built effective width of the shear key, was used to compare the effect on performance of different compaction methods.

Compaction method	Number of case histories	Strain after steady sliding rate reached (%)		
		Minimum	Mean	Maximum
Not compacted	5	0.04	0.17	0.57
Compacted	7	0.02	0.52	1.75
Vibroflot	*5*	*0.05*	*0.65*	*1.75*
Other vibration	*2*	*0.02*	*0.20*	*0.39*

From Table 4.6 it can be seen that compaction with a Vibroflot reduced the time required for sliding rates to decrease compared to the time it took for the uncompacted shear keys. For the mean length of time to reach a sliding rate of 16 mm/year, compaction using a Vibroflot reduced the time by almost 50% compared to uncompacted shear keys. For slides to reach a rate of 1 mm/year, compaction by vibration reduced the mean time by 14% relative to uncompacted shear keys. The non-vibratory means of compaction resulted in longer times. Where the performance of the compacted shear keys was poorer than the uncompacted cases, it is uncertain whether a denser state was actually achieved in the backfill. The drop weight resulted in the longest time by far. It is possible that poor control over where the compactive effort was applied using this method could have resulted in the in-situ soil surrounding the excavation to become highly disturbed. This would be expected to weaken the soil and could result in prolonged movements.

It can be seen from Table 4.7 that compaction resulted in more significant reductions in the rate of sliding one year after construction than when the backfill was left uncompacted. The case histories where any means of compaction was utilized experienced on average a 50% reduction to the rate of sliding, whereas those left uncompacted averaged only 23%. This suggests the shear resistance provided by the shear keys is mobilizing much faster when the backfill has been compacted.

Both Table 4.8 and Table 4.9 examined the strain that was exhibited after the rate of sliding had decreased below a specific threshold. Compaction appears to have resulted in the shear keys undergoing greater magnitudes of strain than when the backfill was left uncompacted. This result goes against what would be expected from theory, whereby an increase in the relative density should result in less strain being required to mobilize the required shear resistance. There are several possible explanations for these results. First, the act of compacting the backfill may have resulted in vibration causing the surrounding, in-situ soils to weaken. This could potentially cause more deformation to be required to compensate for the reduced composite shear strength. To verify this, in-situ shear strength tests (vane shear, for example) would be required from both before and after construction to see if the shear strength had been considerably affected. Another possible explanation is that these vibratory compaction methods resulted in particle crushing, reducing the friction angles that could be achieved by the backfill. A final possibility is that by increasing the relative density, the confining stress increased, also reducing the friction angle. Without additional test results and field verification though, these postulations cannot be confirmed.

The mobilization of shear strength and its potential relation to creep

Creep behaviour is observed in many landslides after shear key construction. To explore a possible explanation for this, the shear strength mobilization curves for the dense and loose crushed limestone rockfill specimens tested by Abdul Razaq (2007) were analyzed. The dense specimen test data was plotted then a linear fit, shown in Figure 4.25, and a power curve fit, shown in Figure 4.26, were applied. The same was done on the loose specimen test data (Figure 4.27 and Figure 4.28).

The unit weights, relative densities, the D_{10}, D_{30} and D_{60} sizes, grading coefficients, and USCS for the test specimens are summarized in Table 4.10.

Table 4.10: Summary of the material properties for the crushed limestone rockfill specimens tested by Abdul Razaq (2007).

Specimen	Unit weight (kN/m^3)	Relative density (%)	Particle sizes (mm)			Grading coefficients		USCS
			D_{10}	D_{30}	D_{60}	C_c	C_u	
"Loose"	15.0	< 15						
"Dense"	19.1	> 90	3.7	10	16.6	1.6	4.5	GW

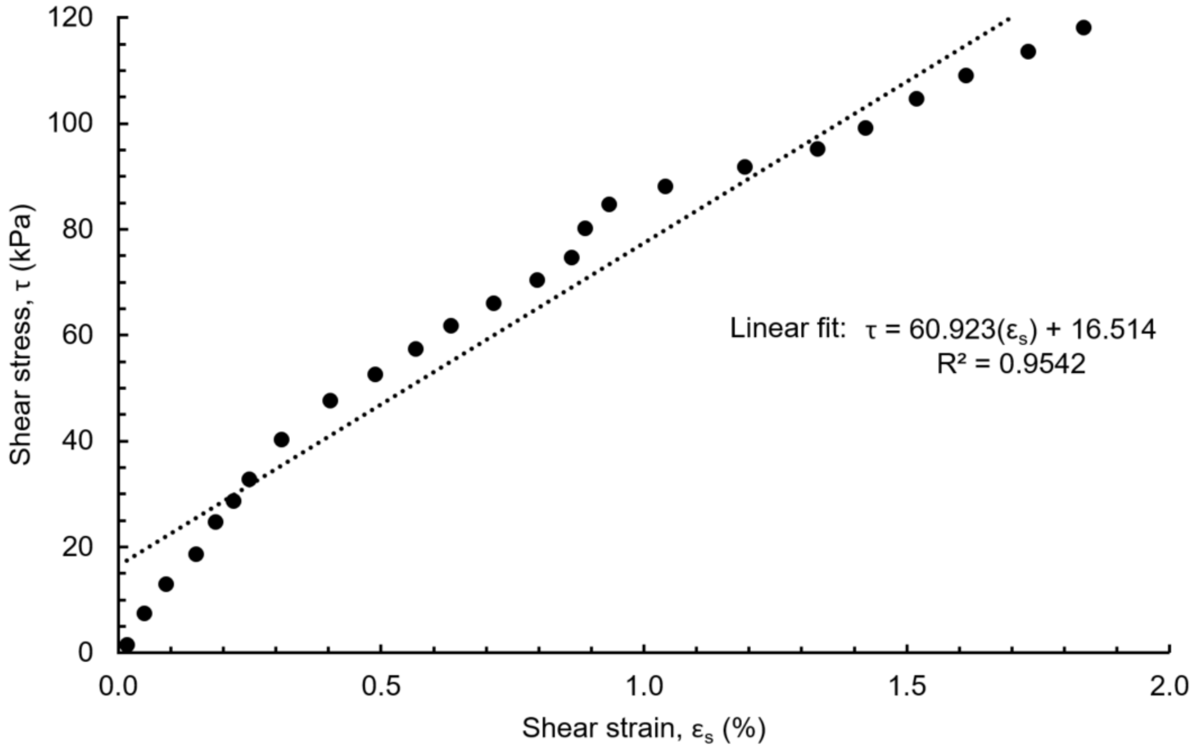


Figure 4.25: Linear fit applied to dense crushed limestone rockfill stress-strain data reported by Abdul Razaq (2007).

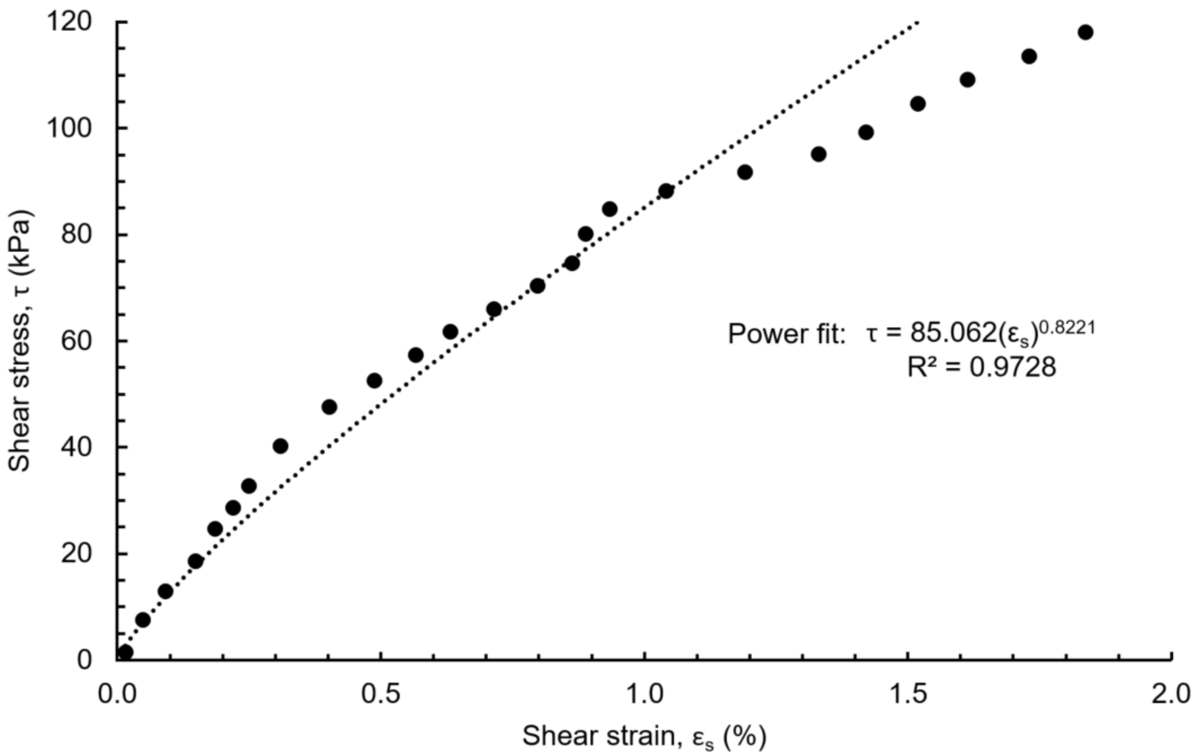


Figure 4.26: Power curve fit applied to dense crushed limestone rockfill stress-strain data reported by Abdul Razaq (2007).

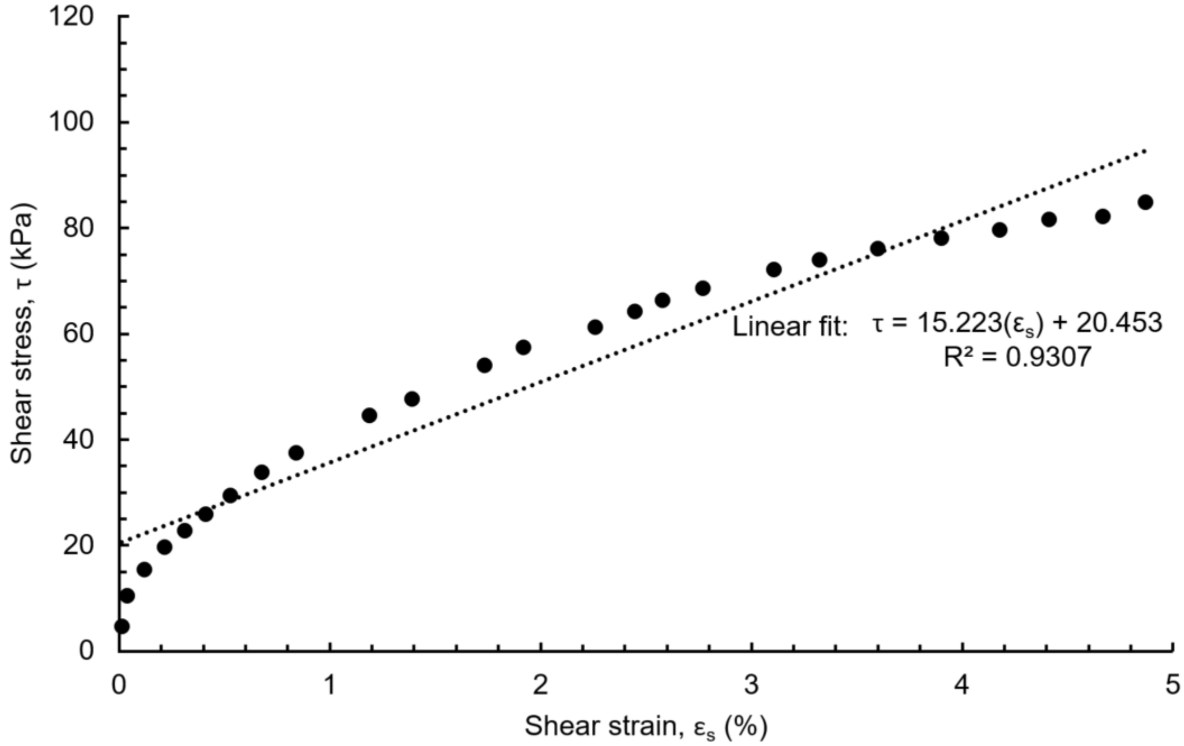


Figure 4.27: Linear fit applied to loose crushed limestone rockfill stress-strain data reported by Abdul Razaq (2007).

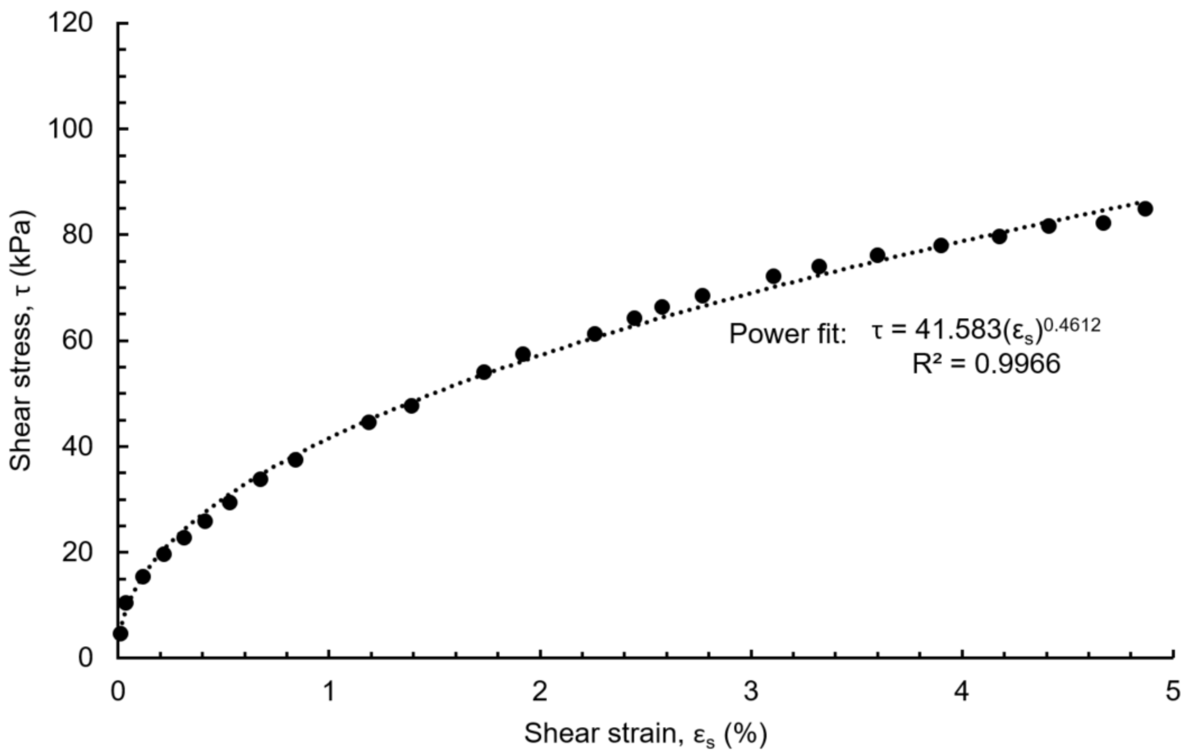


Figure 4.28: Power curve fit applied to loose crushed limestone rockfill stress-strain data reported by Abdul Razaq (2007).

The mobilization of shear strength in the granular backfill was modeled by inputting a granular shear key into a slope in 2D LE (Figure 4.29). The model featured a 3H:1V slope which was given a defined slip surface with a residual friction angle of 12° and zero cohesion. This set-up yielded a factor of safety equal to unity. An idealized granular shear key was then introduced to the model. The shear key was given an effective width of 11.5 m, and a unit weight of 21 kN/m^3. An effective depth of 4 m was selected to match the confining stress of 100 kPa applied by Abdul Razaq (2007) for the direct shear tests from which the data was obtained. These parameters yielded a factor of safety of 1.3 when the granular backfill was modeled to be cohesionless and the friction angle was set to 45°. This friction angle was selected as it corresponded to a 5° reduction to the peak friction angle reached by the dense specimen. It was also judged to be a realistic value, falling within the range typically used in practice.

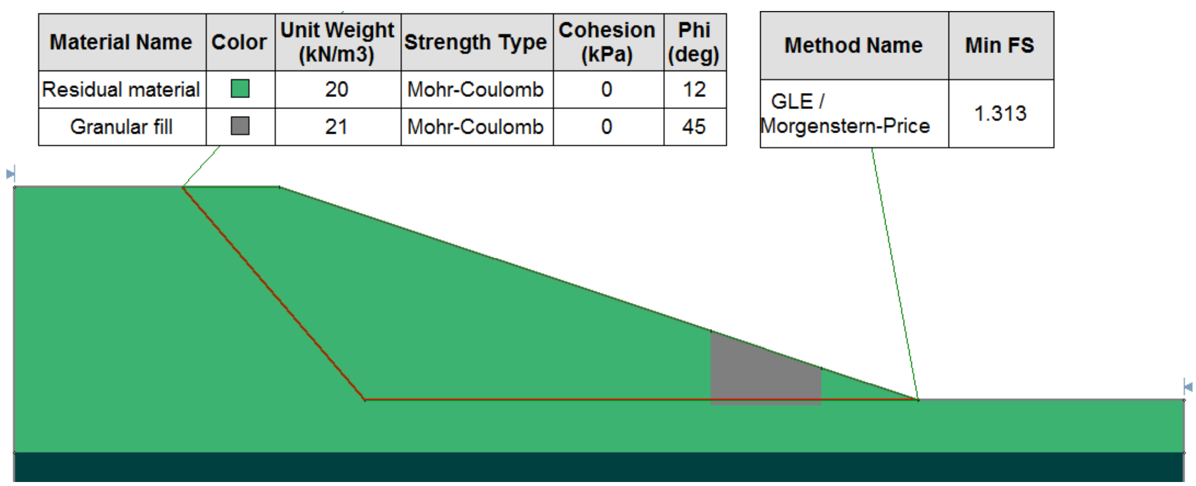


Figure 4.29: Modeled granular shear key using 2D LEM software, Slide 7.020.

Upon introducing the shear key to the model, the shear key was initially given null strength values. This can be likened to having an open excavation of infinite length. The factor of safety calculated for this condition was below unity (0.91), as expected. This should serve as support for adopting the closely-sequenced excavation technique and keeping the length of time during which the trench is open to a minimum.

The factor of safety was calculated using the strengths estimated with the linear fits and the power curve fits that were mentioned previously. The results are shown in Figure 4.30. They show that the factor of safety target of 1.3 was achieved after considerably less strain (almost 3%) using the strength parameters for the dense test specimen, compared to using the strength parameters for the loose specimen. This is consistent with the theory that was discussed in § 2.2.3. Furthermore, the difference between using the M-C (linear) model and the Power Curve model to estimate strength was apparent. By adopting the M-C model, the target factor of safety of 1.3 was achieved after a shear strain of 1.3%. Had the Power Curve model been used, the factor of safety after this amount of strain would have been calculated as being almost 1.4. This shows that the design that would have been produced using the M-C model would have been over-conservative and using the Power Curve model could have resulted in a more economic design.

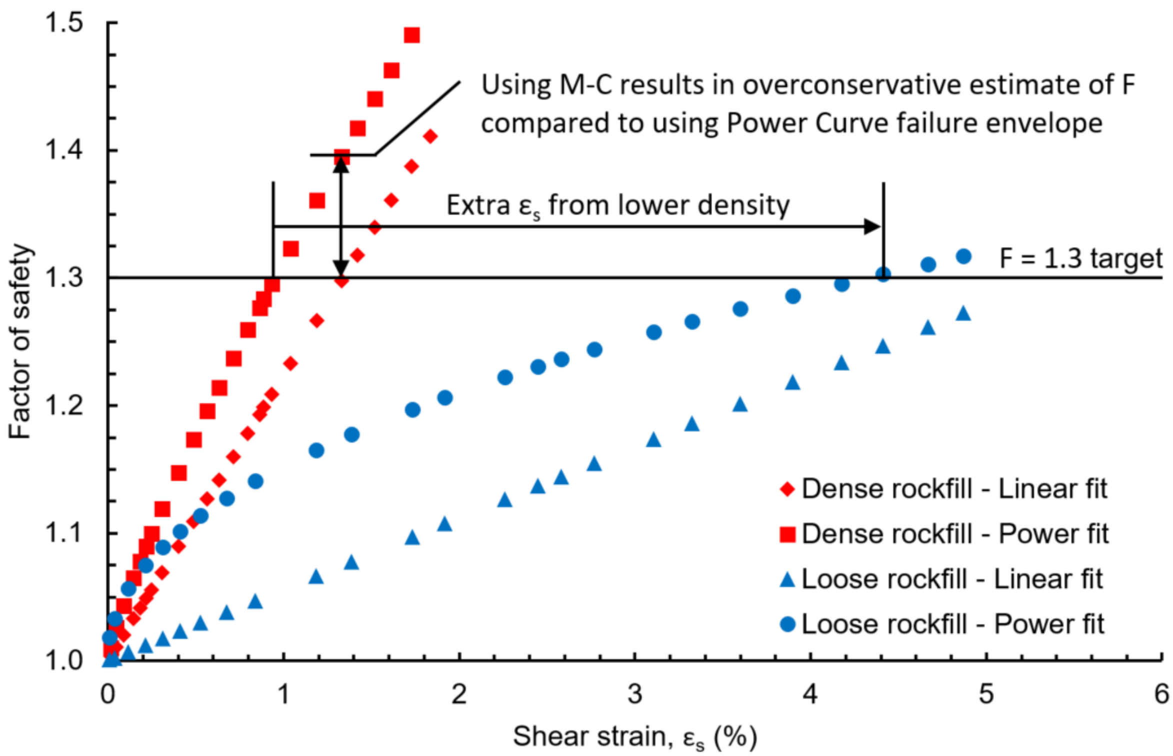


Figure 4.30: The factor of safety calculated for increasing degrees of mobilization as represented by the shear strain, for the dense and loose rockfill test data.

In theory, the shear key would only mobilize until a state of equilibrium was established, rather than until a factor of safety of 1.3 was achieved. Under dynamic site conditions, strain would develop each time a change in the site conditions necessitated an increase in the shear resistance provided by the granular shear key. For example, a flood event would increase pore pressures in a slope. Upon receding, the buttressing effect of the flood water would be relieved but high pore pressure could remain. If pore pressure remains high, effective stress will be reduced along with the shear resistance. This could potentially lead to the slope reverting to a marginally unstable state. In this scenario, movement would be sustained until sufficient shear resistance had mobilized in the shear key to restore the slope to a state of equilibrium. As the pore pressures continued to dissipate, the degree to which the shear key had mobilized would remain unchanged. An excellent example of a site that exhibited this scenario is WP8. The monthly rate of sliding was calculated using slope inclinometer data obtained from Yarechewski & Tallin (2003), and is shown in Figure 4.31.

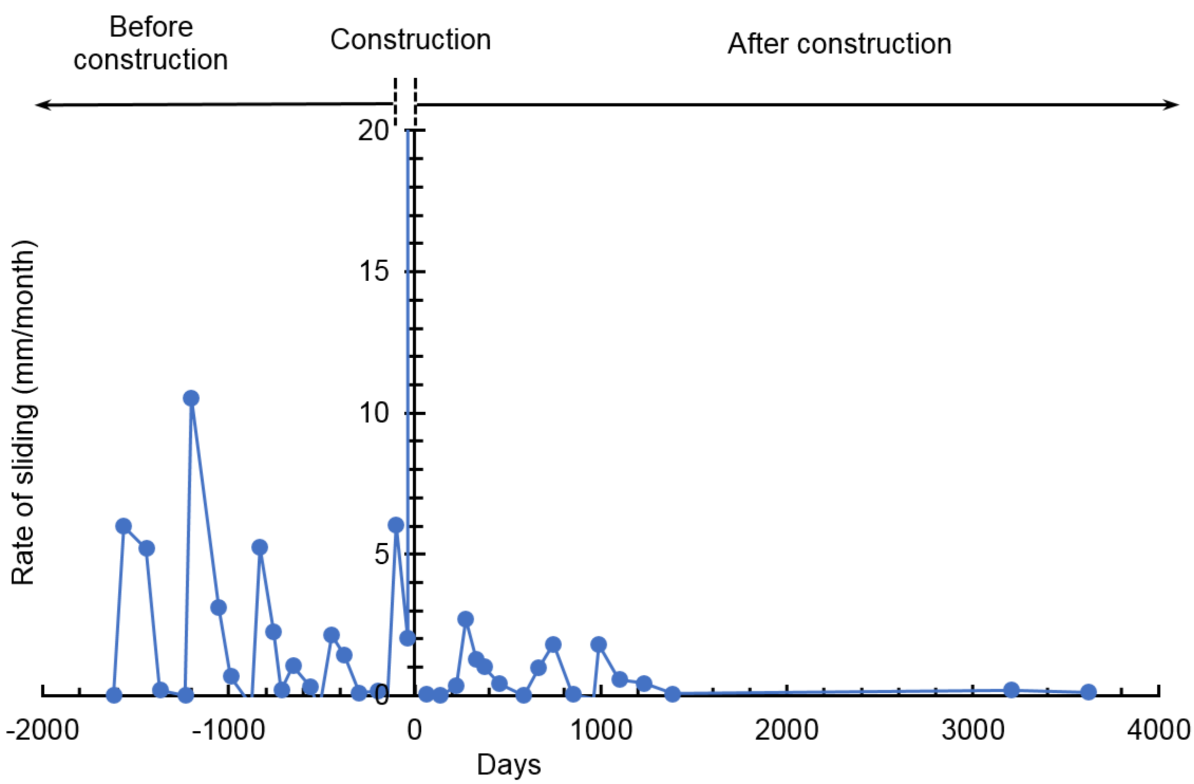


Figure 4.31: The monthly rate of sliding as calculated between measurements for WP8. Original cumulative displacement data obtained from Yarechewski & Tallin (2003). Used with permission.

The monthly rate of sliding for WP8 is plotted against the time in days relative to construction. Positive time values represent post-construction, whereas negative values represent the time preceding and leading into construction. The rate of sliding before construction fluctuated significantly, forming six regularly spaced peaks. These peaks likely coincided with the seasonal lowering of the river levels by the City of Winnipeg each fall. There is perhaps some additional influence from weather phenomena that temporarily affected the river levels too. The average time between peaks is approximately 8.9 months. As construction was commenced, the rate increased significantly. This increase coincided with the excavation of the granular shear key and is typical of these projects. Following construction, the monthly rates were significantly reduced and they became more consistent. The peaks that formed post-construction can be compared with the pre-construction peaks. These peaks are no longer as regular, and the average time between them increased to 15.2 months. They also decreased in magnitude with each subsequent peak. Following a 5-year gap in the monitoring data, the rate of sliding had become effectively null.

The peak rates following construction likely represent periods when the shear resistance provided by the native soil was at local minima. The shear resistance required of the granular shear key would thus increase, to maintain stability. To mobilize this extra shear resistance, movement is required. Until the shear resistance provided by the shear key meets the new requirement, the slope accelerates and these peaks are established.

After each peak is formed, the granular shear key mobilizes a little more. Thus, for a new peak to form, site conditions must result in a greater shear resistance requirement being established. If the threshold that has been established is subsequently surpassed by only a small amount, the time during which the slope is in an unstable state will be shorter. This process may explain why each successive peak is smaller. This explanation is supported by the concept of return periods. The likelihood and magnitude by which, for example, a flooding event will exceed the weakening effect of the previous flooding event should diminish with each passing event. The exception would be for more significant flooding events that would recur over a longer return-cycle. For WP8, the first post-construction peak occurred between October and November of 1993, corresponding well with about the time of the year the Red River is at a seasonal low. The second post-construction peak occurred in the spring of 1995, and appears to correspond with a small flood, per the historical information published on the City of Winnipeg website (City of Winnipeg, 2017). The third post-construction peak occurred around October of 1995 and likely corresponds once again to the lowering of the river level for winter. Thus, the theory and supporting information that has been presented may explain how the mobilization of shear resistance relates to temporary periods of slope movement.

4.2.4 Evaluating the impact of shear keys on groundwater

Granular shear keys are constructed using free-draining backfill, which is typically expected to promote drainage by intercepting the groundwater as it flows in the downslope direction. It was expected this would be reflected in a lowering of the groundwater table downslope of the shear keys following construction. The groundwater data that was collected was plotted in Figure 4.32.

The water level was calculated using the pore pressure readings acquired from vibrating wire piezometers, or as recorded in standpipe piezometers.

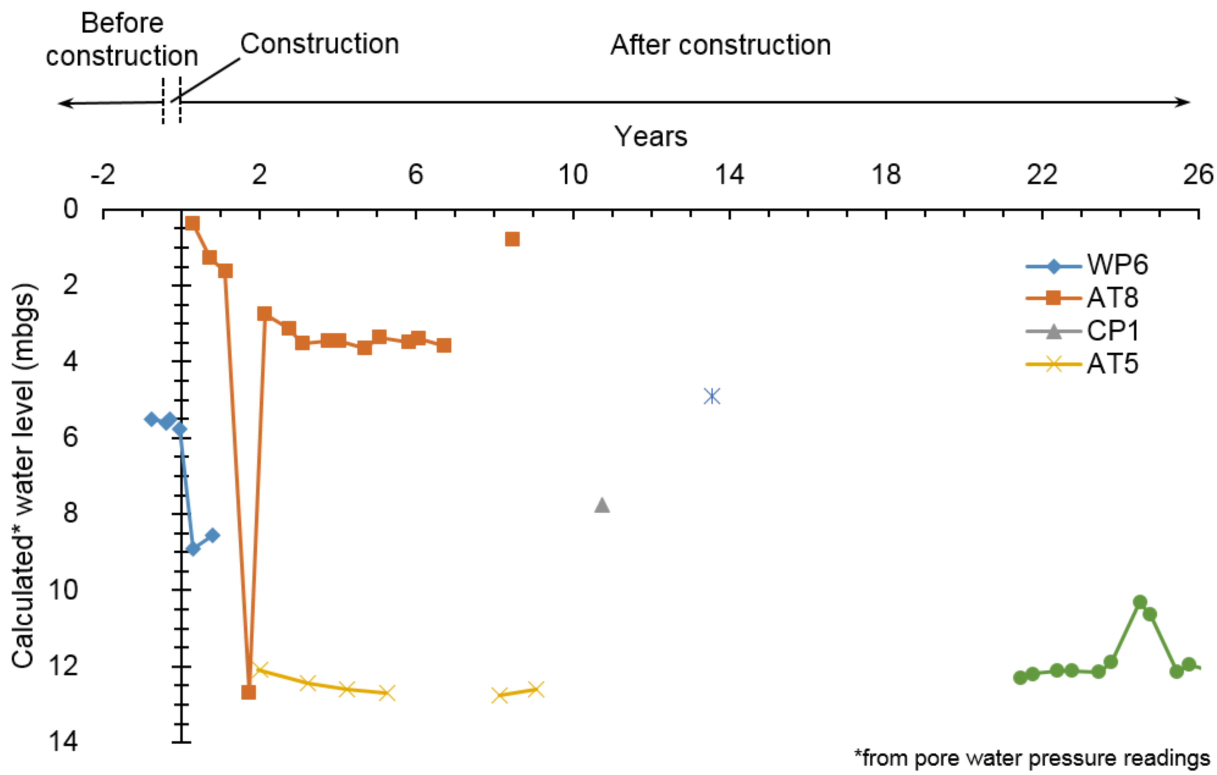


Figure 4.32: Groundwater levels after construction of granular shear keys.

A general decrease in the water level can be observed for the two sites with readings beginning around construction. Limited groundwater data was acquired though, so performance analyses were not carried out for the stabilizing effect these drops in the water levels might have had. However, the drop in the corresponding water levels in WP6 and AT8 can still be discussed in the context of each case study.

The shear key at WP6 was constructed approximately 6 m deep, with a key-in depth of 0.6 m into till. The groundwater level before and after construction appears to have been approximately 5.5 mbgs and 8.8 mbgs, respectively, corresponding to a drop of 3.3 m. The water level was relatively consistent across the two post-construction readings, despite one being from summer and the other being from winter. The piezometer was located approximately 27 m upslope of the shear key and the tip was near one of the potential slip surfaces that was analyzed. The post-construction water level appears to correspond with the elevation of the bottom of the river in the cross sections that were acquired. The tip location might suggest the readings are indicative of the pore pressure across the slip surface. By reducing movement, the shear key may have relieved excess pore pressure along it.

The piezometer referenced for AT8 is in line with the rockfill columns that were constructed there. A pore pressure drop equivalent to 3.5 m was recorded several years after construction. The main feature of this site was the construction of a new highway embankment, which would likely have induced an increase in excess pore pressures. The gradual drop in pressure over the years following construction seems to suggest the rockfill columns were effective in dissipating these excess pore pressures.

The observations discussed for these two case studies suggest shear keys may relieve slide-induced pore pressures. In doing so, the shear resistance along the slip surface may increase. The implications of this are that, if left unaccounted for in the model, LE-designed shear keys could be overly conservative.

4.2.5 Other design observations

In addition to the primary analyses discussed in the previous sections, some additional analyses were carried out for parameters or concepts that were not necessarily fundamental to all granular shear keys. These include the area replacement ratio and the clay caps constructed overtop of granular shear keys.

Rockfill column area replacement ratios

The area replacement ratio, A_{r}, is a design parameter related to the design of rockfill columns, as discussed in Chapter 3. Large-scale direct shear tests performed by Abdul Razaq (2007) indicated the area replacement ratio had no impact on the stiffness of the composite soil. However, it was found that greater shear resistance could be mobilized. This theory was verified using the case studies that featured rockfill columns.

The area replacement ratio was plotted against the time required for sliding rates to reach 16 mm/year, 1 mm/year, and to become steady. They were also plotted against the strain as a percentage of the as-built effective width of the rockfill columns. The effective width of the rockfill columns was calculated using the equivalent width method per Thiessen (2010), where,

$$b_{\mathrm{e}} = \frac{A_{\mathrm{r}}}{s} \tag{4.1}$$

As expected, per the findings of Abdul Razaq (2007), no correlation could be identified between the area replacement ratio and the strain, nor the time over which that strain developed. Furthermore, no relationships were identified between the cross-slope center-to-center column spacing, s, and performance either. Perhaps in part due to the prevalence of the triangular pattern, the spacing does not appear to have exceeded any sort of stability threshold in any of the case studies. Significant movement would have been taken as a possible indication of native soil squeezing between columns.

Movement in clay caps

A common feature of many granular shear keys is a clay cap. The clay cap is typically at least 0.6 m thick and can be thicker if it is part of a weighting berm. The purpose of the clay cap is to limit the infiltration of surface water into the granular shear key, whether for environmental reasons or to limit the fouling of the granular material. It is also not uncommon for geosynthetic liners to be placed along the interface between the clay cap and the granular material. In several of the case studies for which SI data was available, movement observed within the clay cap was investigated. The aim of this section is to identify, present and discuss these movements. Potential techniques to mitigate these movements are not presented; rather, the intent is to simply raise awareness of such issues. These near-surface movements may not always be expected since the focus during the design of granular shear keys tends to be further below ground.

Three sites featuring clay caps over granular shear keys were selected for further investigation. The first site is CP1, where there were two granular shear keys that were constructed and monitored. The first shear key was constructed in 1996 and is subsequently referred to as the North Shear Key. The second was constructed in 2002 and is subsequently referred to as the South Shear Key. The shear keys are approximately 25 m apart, so there is no overlap between them.

A SI named SI901 was identified and, from the information available, appears to have been installed adjacent to the North Shear Key on the upslope side and about halfway along its length. About 65 m north of SI901, a borehole log was acquired for BH834. This borehole is located about 20 m beyond the northern extent of the North Shear Key. The SI data from SI901 was digitized and replotted to show the cumulative and incremental displacement with depth (Figure 4.33). The interpreted stratigraphy for BH834 from Yong (2003) was superimposed between the two sets of SI data.

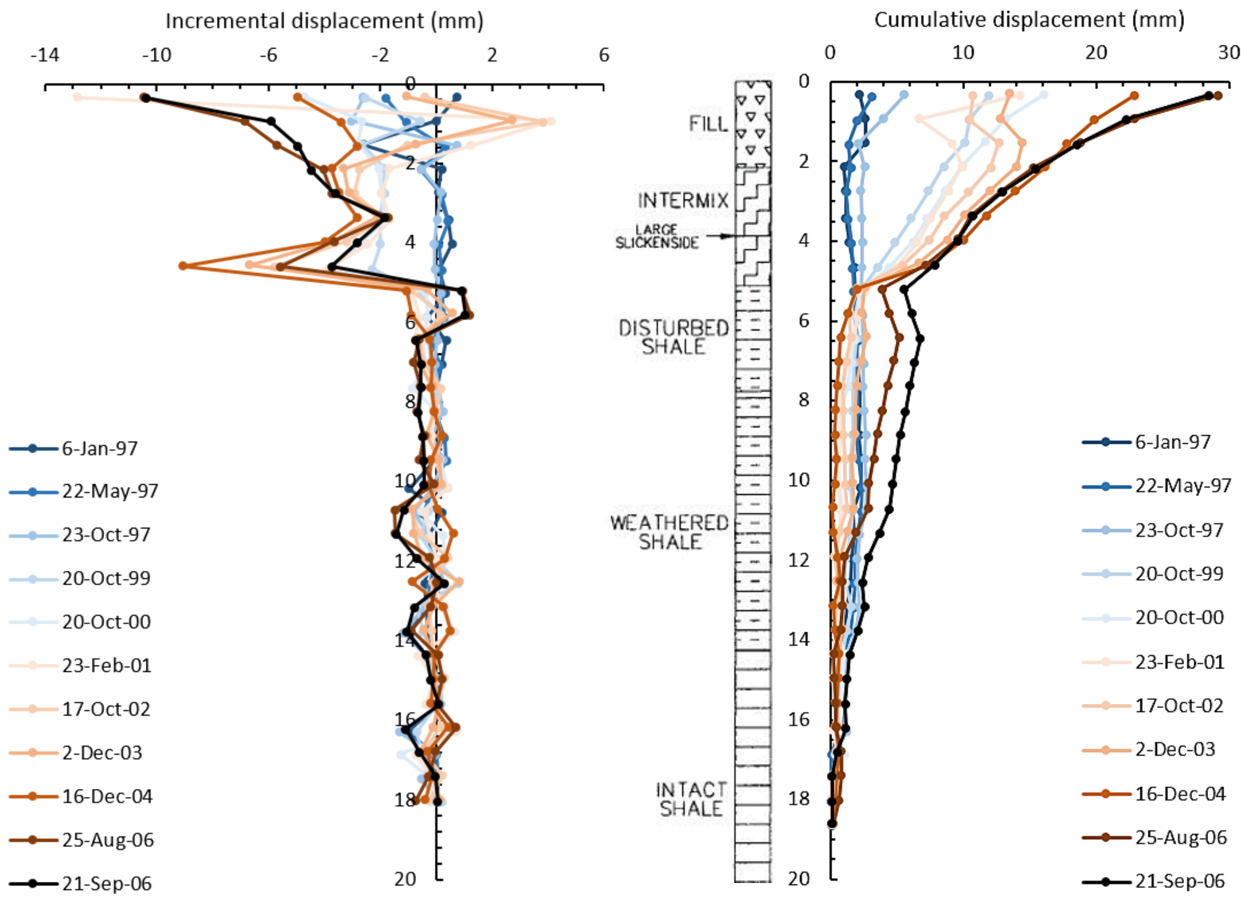


Figure 4.33: Incremental and cumulative displacements from SI901 between January 1997 and September 2006. Two zones of movement can be identified; one just below 4 m in depth and another around 1 m below surface. The borehole log in the middle is for BH834, located roughly in line with SI901 but 65 m out of section. Borehole log taken from Yong (2003), with permission.

Two distinct zones of movement are identifiable in the SI data: one around the same depth as a large slickenside identified in the borehole log, and another around 1 m below surface. The deeper movement likely corresponds to the slip surface which the granular shear key was designed to mitigate. Movement there can be seen occurring through the summer of 1997, but slows considerably over the next few years, as evidenced by the decreasing spread in the plotted cumulative displacements. The shallower movements do not appear to follow this trend though and correspond to the 2 m of fill shown at the top of the borehole log. This is the first example of movement occurring in the clay cap of the granular shear key.

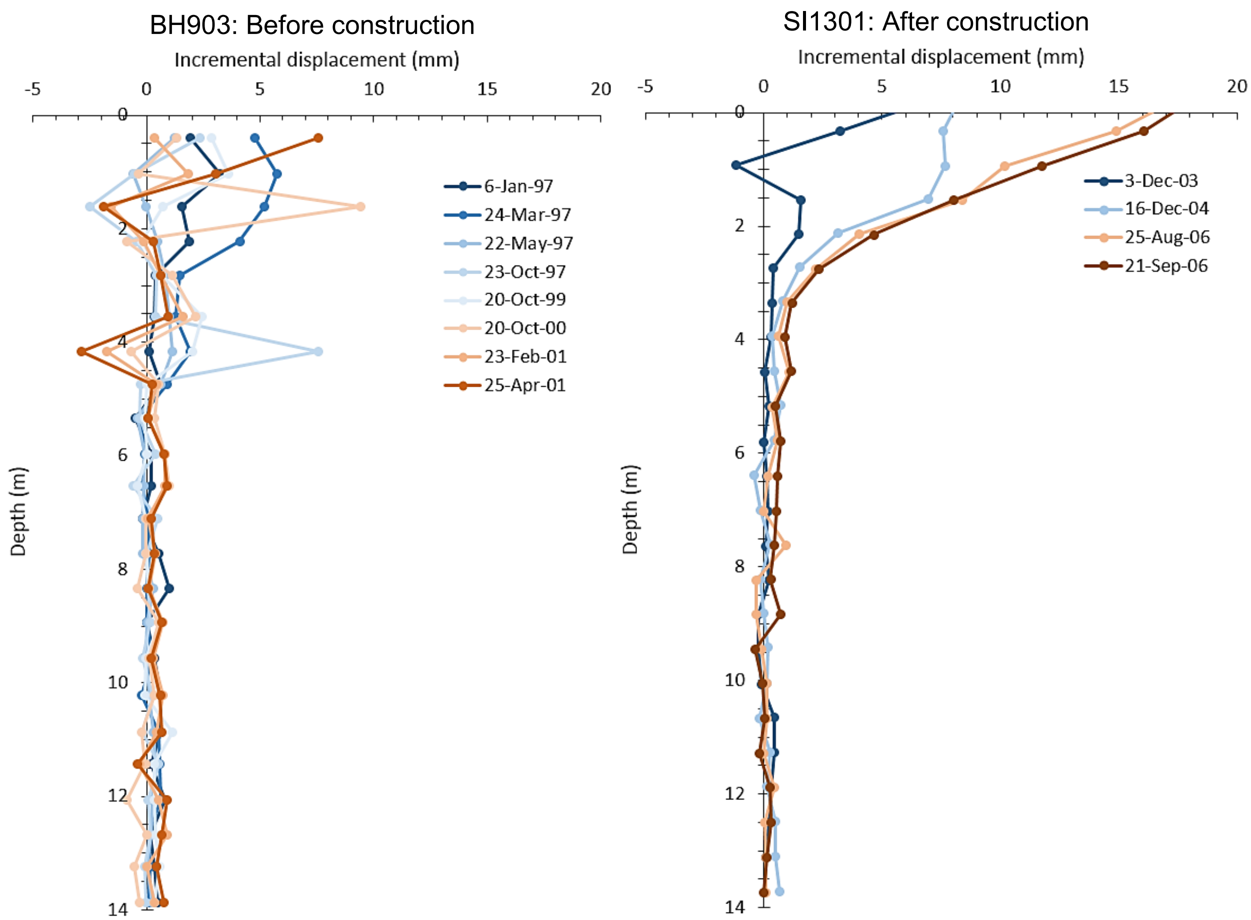


Figure 4.34: Incremental displacement data from BH903 (left) and SI1301 (right). Two zones of movement can be identified in BH903 but only the upper zone can be identified in SI1301.

Another example of clay cap movement can be seen in the South Shear Key at this site. SI data from before (BH903) and after (SI1301) construction was available for this shear key, and the SIs are both located within the footprint of the shear key. The incremental displacement was plotted for each, in Figure 4.34. In the data for BH903 (left-hand plot in Figure 4.34), two zones of movement can be identified, as was the case in SI901 for the North Shear Key. For SI1301 though, only the near-surface movement can be seen. This appears to be evidence that the South Shear Key was quite effective in eliminating movement along the deeper slip surface. The clay cap, which is approximately 2 m thick, corresponds well with the zone encompassed by the near-surface movements.

The cumulative displacements in BH903 and SI1301 were also plotted with depth, in Figure 4.35. From BH903 to SI1301, the majority of the movement at depth is seen being eliminated and movement near surface is seen becoming more pronounced.

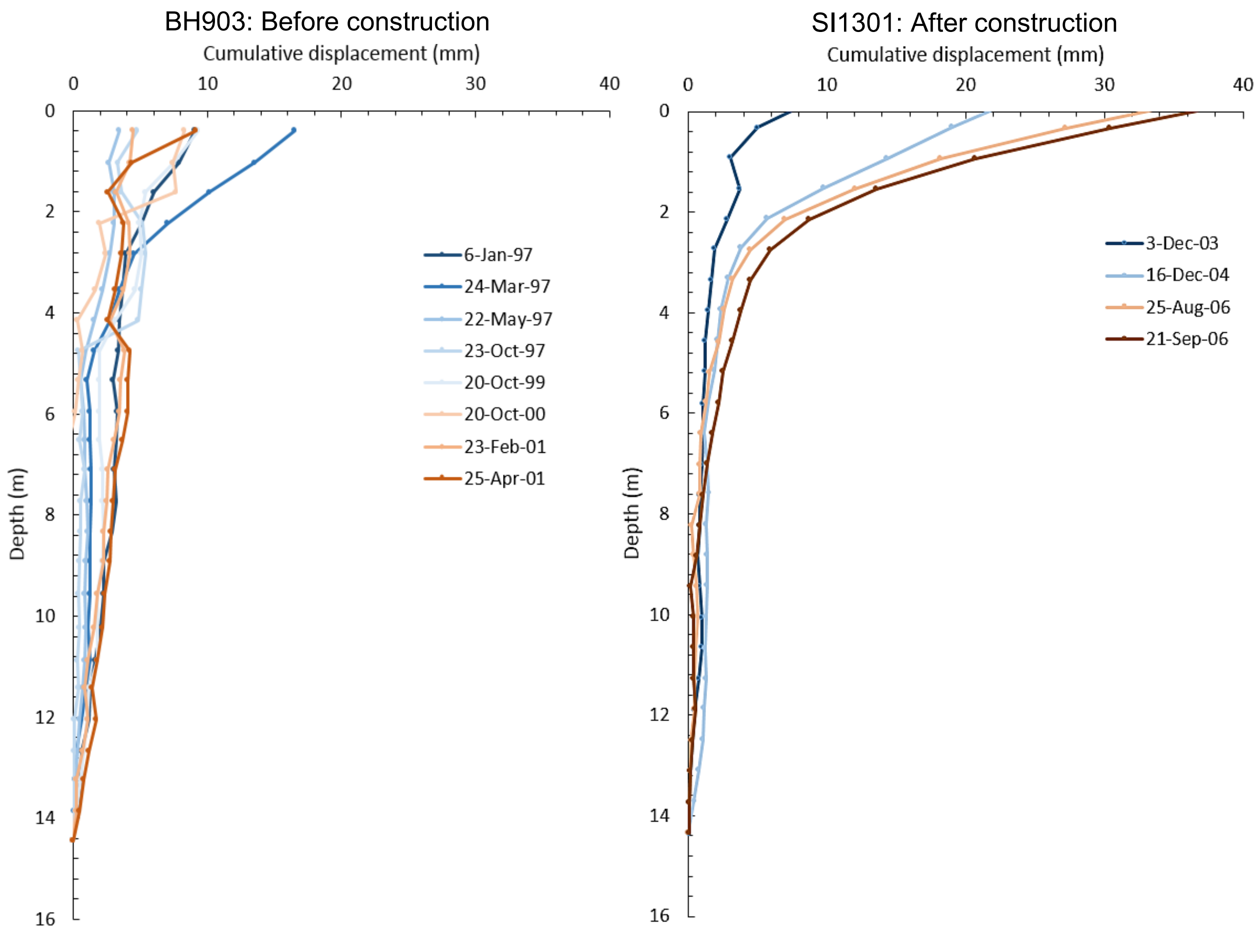


Figure 4.35: Cumulative displacement data from BH903 (left) and SI1301 (right). Movement in BH903 has been adjusted so it is relative to the same base depth as SI1301, at 14.4 m below surface.

A second example of a site with a clay cap over a granular shear key is WP3. This site did not exhibit movement in the clay cap. Data from two SIs was acquired for this site, and the incremental displacement was plotted then superimposed over a cross section of the site Figure 4.36. The axes are not labelled; the plots are only intended to illustrate the depths of the zones where movement was observed. This figure shows SI-01, upslope of two rows of rockfill columns, where a discrete slip surface is easily identifiable. This corresponds quite well with the tension cracks immediately upslope of this SI. SI-03 is located downslope of these two rows of rockfill columns. What was interpreted as being the same slip surface can be seen at an elevation approximately equal to a third of the way up from the base of the columns. The slip surface is no longer as discrete, after having had to pass through the columns. Clay caps are also clearly illustrated above the rockfill columns, but there is no clear movement occurring there. It is possible movement there was prevented because of the vertical offset between the two rows of columns, and the lateral offset into the plane from the pattern that was adopted.

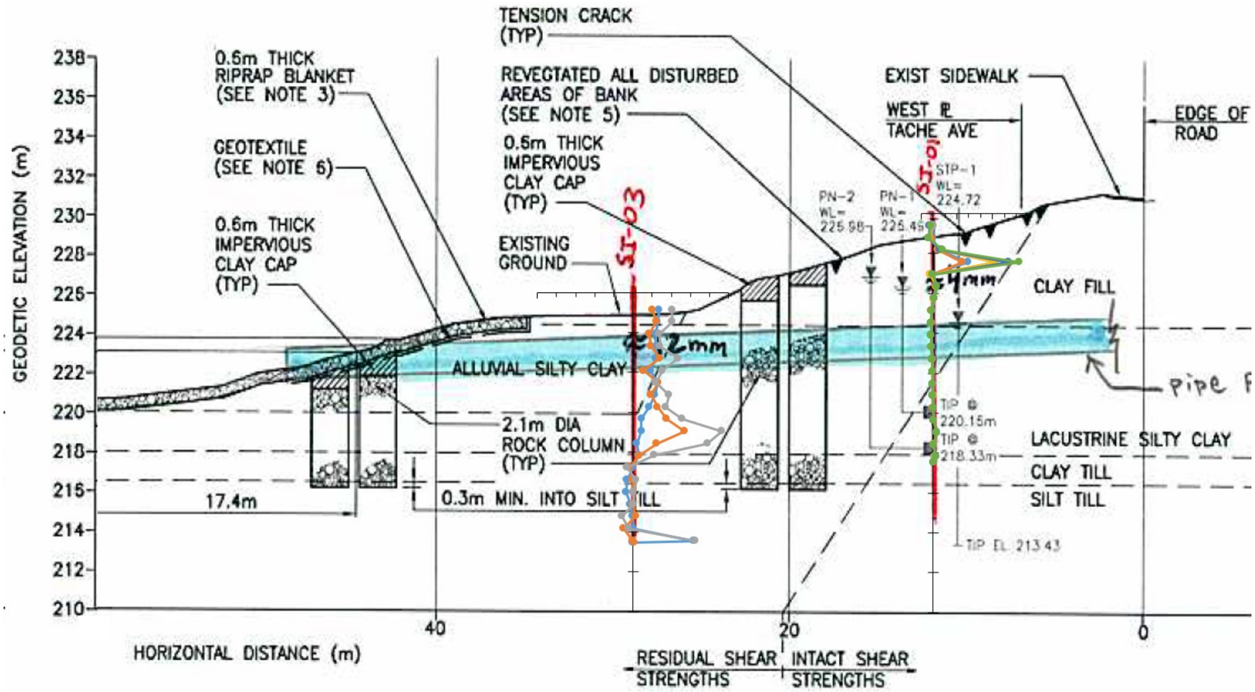


Figure 4.36: Cross section of the rockfill columns at WP3. The incremental displacement data for SI-03 and SI-01 was superimposed over the cross section. A discrete slip surface can be seen in SI-01 upslope of the rockfill columns and directly downslope to some tension cracks that were identified. SI-03 shows what was interpreted as being this same slip surface, but the surface is no longer as discrete, having crossed through the rockfill columns.

The final example of a site with clay caps constructed over granular shear keys is WP13. In this case, movement appears to have been reduced significantly at depth, to the point where it cannot be easily observed in the incremental displacement plots superimposed on the cross section in Figure 4.37. What is apparent though is the movement near surface, upslope and downslope of the upslope set of rockfill columns. In SI-3, the movement extends to a depth of approximately 2 m but becomes constrained to the upper 0.5 m in SI09-01. This can be interpreted as showing effective mitigation of movement by the rockfill columns, but surface movement persisting due to the presence of clay caps.

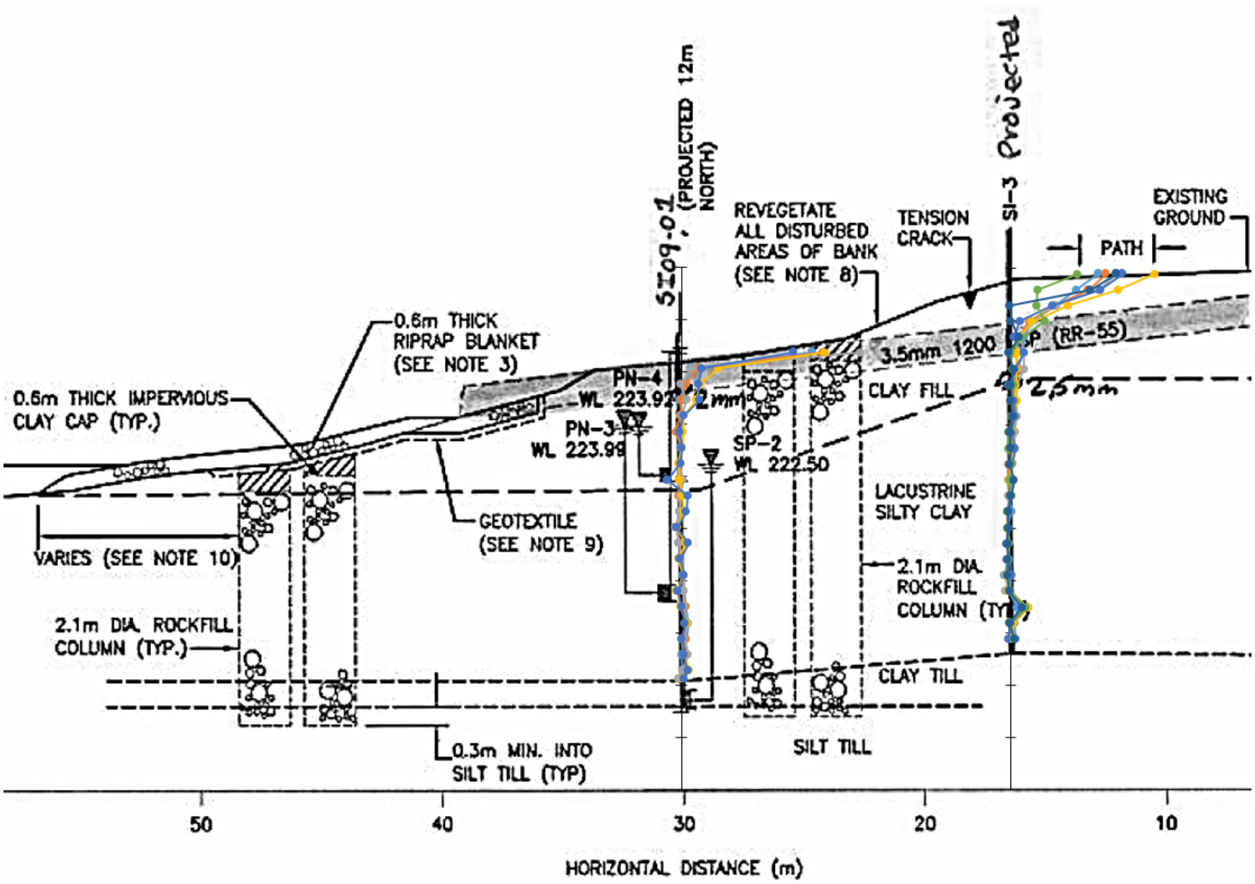


Figure 4.37: Incremental displacement plotted with depth for SI09-01 and SI-3 at WP13. The zone of movement appears to have been constrained to the same depth as the clay caps after passing from upslope to downslope of the rockfill columns.

A simple exercise was performed where the Mohr-Coulomb shear strength was plotted for a conceptual granular material and a clay (Figure 4.38). The granular material was given an effective unit weight of 21 kN/m^3, an effective friction angle of 40° and zero cohesion. The clay was given an effective unit weight of 17.3 kN/m3, based on the average from clays tested by Liang and Lovell (1983), an effective friction angle of 16.8° and an effective cohesion value of 30.4 kPa, both based on data from Matyas (1967) for Winnipeg Clay. The shear strength was plotted against depth rather than the normal effective stress, to show the depth at which the strength of the granular material surpasses that of the clay. With the initial properties, the crossover depth is approximately 2.4 m, signifying that a clay cap of a thickness less than or equal to this would possess greater shear strength than the granular material. In this case, it would be appropriate to exclude a typical 2-m-thick clay cap from stability analyses since the granular material would yield the more conservative result. However, it would be reasonable to expect some degree of softening or weakening in the clay cap, from exposure to surface conditions. A 25% reduction was applied to the cohesion value for the clay and its shear strength was replotted. The weakened clay shear strength envelope translated downward, resulting in a 0.6 m reduction in the crossover depth, to 1.8 m. The implications of this are that for a 2-m-thick clay cap, the bottom 0.1 m of clay would now be weaker than the granular material. This introduces a more conservative scenario than that which was analyzed during design, if the clay cap had been excluded. This simple exercise illustrates a possible explanation for the movement observed in some of the clay caps. Of course, there are many additional complexities that could be contributing to these movements, including unsaturated soil mechanics, the freeze-thaw cycle, seasonal flooding, and vegetation.

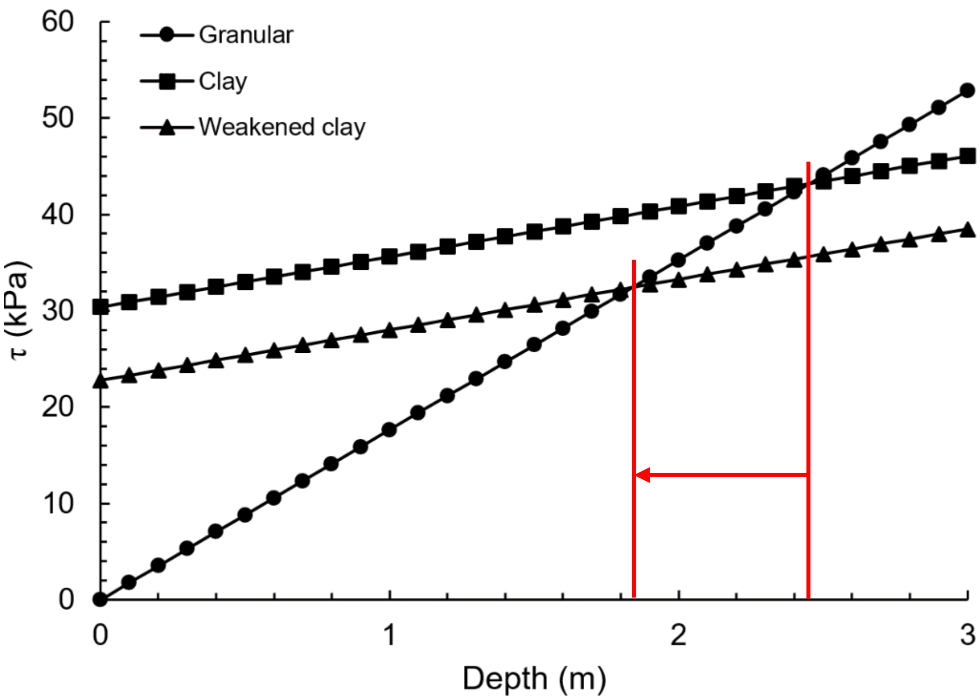


Figure 4.38: Mohr-Coulomb strength envelopes for a granular material, a clay, and the same clay after cohesion was reduced by 25%. Shear strength is plotted against depth to show how the crossover depth changes after the cohesion loss.

The three examples that have been presented each show cases where clap caps have been constructed over granular shear keys. Two of the cases suggest movement is persisting near surface and passing through these clay caps. The other case shows the presence of clay caps is not necessarily always accompanied by these surface movements. While an extensive analysis into the potential causes was not performed, these cases should provide an appreciation for the potential for near-surface movements when a clay cap is an expected component of a granular shear key project.

4.3 Case Study Review Conclusions

The design of granular shear keys and its potential impact on performance has been analyzed across a collection of 39 case studies. The construction of a database comprising these case studies has enabled the creation of a multitude of plots, verifying and providing quantifiable findings to supplement the design of granular shear keys. A statistical breakdown of certain key parameters has been carried out showing typical ranges of values observed across the database. These ranges provide valuable context for new projects considering granular shear keys as a solution to mitigate landslide movements.

Several design theories were tested by plotting different variables involved in the granular shear key design process, to verify how well these theories are reflected in actual design. These same variables were plotted against various performance indicators to attempt to supplement the currently resistance-based design practice. The findings of these analyses lay the framework for performance-based design of granular shear keys, and raise a need for additional studies.

The major conclusions from this chapter are summarized as follows:

1. The performance of granular shear keys appears to be related to where the shear key is placed within a slide.
    a. Trenched shear keys appear to achieve greater reductions in the rate of sliding one year after construction (R_1) the closer they are constructed to the toe of a slide (i.e. P_{SK} = 100%). This may reflect shallower depths of sliding near the toe permitting greater friction angles to be achieved due to increased dilatancy. Five-year performance (R_5) also appears to improve as P_{SK} nears 100%.
    b. The performance of rockfill columns is delayed in the first year after construction the closer they are constructed to the toe (i.e. lower R_1 as P_{SK} approaches 100%) but this is followed by a rapid reduction in the rate of sliding in the following years.
    c. After evaluating the relationship between the time in years shear keys took to effectively stop slope movements (1 mm/year or less) and their location (P_{SK}), an optimum location for long-term performance was identified at a P_{SK} value of 68%. Shear keys at this location still took between 0.5 and 4 years to stabilize.

Case history examples:

Two trenched shear key sites with different placements and correspondingly different percentage rate reductions are WP6 and WP8. A rockfill column and a trenched shear key site with similar rate reductions but different placements relative to the slide scarp are AT8 and EX2 (both featured in Figure 4.7). Two examples of sites with shear keys placed near the optimum location in the slide are WP3 and WP8.

2. A relationship between the size of a landslide and the size of a shear key was identified.
    a. As the landslide cross-sectional area increases, so does the shear key cross-sectional area designed to remediate it.
    b. Rockfill column shear keys appeared to require smaller volumes of rockfill than trenched shear keys to remediate the same size of landslide. This was postulated to be the result of soil arching between columns.

Case history examples:

The relationship between the size of a landslide and the size of a shear key can be seen by comparing EX1 and WP14. The slide cross-sectional areas are 38 m^2 and 187 m^2, respectively. The shear key cross-sectional areas are 13.1 m^2 and 49.1 m^2, respectively. The rockfill column site WP15 and the trenched shear key site WP5 have similar cross-sectional areas of 22.4 m^2 and 20.4 m^2, respectively, but were designed to remediate landslides with very different cross-sectional areas of 188 m^2 and 120 m^2.

3. A relationship between the size of a shear key and its performance was identified.
    a. As the shear key effective width increased, so did the time it took for the rate of sliding to decrease below 16 mm/year and 1 mm/year.
    b. As the shear key effective depth increased, so did the strain that was exhibited.

Case history examples:

The influence of effective width on stabilization times can be seen in WP12 and WP16, with WP16 being wider (6.3 m) and taking longer (1.09 years) than WP12 (2.2 m, 0.30 years) to slow to a rate of 16 mm/year. An example of the influence of effective depth on strain can be seen with EX2 and WP8. EX2 experienced 0.44% strain after reaching a rate of 1 mm/year with an effective depth of 9.0 m. WP8 experienced 0.18% strain with an effective depth of 3.3 m.

4. The backfill material used in granular shear keys plays a significant role in the design and performance of granular shear keys.
    a. When the backfill material was gravel, smaller shear keys were constructed than when sand was used for a landslide of an equal size. An evaluation of friction angles that were compiled suggests the difference in the size of the shear keys is because gravel tends to have a higher friction angle than sand.
    b. As the residual friction angle of the landslide increases, the size of the shear key must also increase. This is because the difference between the landslide and the backfill friction angles becomes smaller.
5. Compaction can decrease the time required for a shear key to reduce the rate of sliding and decrease the strain that it must undergo.
    a. The compaction of backfill using a Vibroflot resulted in a decrease in the time required for the rate of sliding to decrease below 16 mm/year and 1 mm/year, and reduced strain compared to cases where the backfill was not compacted. It also resulted in greater reductions in the rate of sliding one year after construction.
    b. Other compaction methods did not share this result, possibly because a denser state was not achieved. Compaction with a drop weight appears to have produced the longest time to stabilize and the most strain. This may be due to poor control over where the compactive effort was applied, resulting in a weakening of the in-situ soils surrounding the excavation.
    c. A comparison between the shear strength estimated for a dense rockfill (D_R > 90%) and a loose rockfill (D_R < 15%) indicated that a shear key would need to undergo considerably less strain (a reduction of 3.5%) to mobilize the required shear strength to satisfy a factor of safety of 1.3 if the rockfill was dense.
6. Modelling shear strength using the Power Curve model can improve the economy of designs.
    a. The Power Curve model predicted greater strength than the Mohr-Coulomb model, leading to higher estimates of the factor of safety at the same magnitude of strain.

7. A shear key will undergo mobilization in response to changing stress conditions brought on by peak weather events.
    a. An example (WP8) was provided as a possible model for explaining the cyclical behaviour of granular shear keys at riverbank locations, in response to fluctuating water levels. Peak sliding rates may correspond to events where a new minimum shear resistance is temporarily established in the landslide. Successive events resulted in progressively smaller sliding rates until sufficient shear strength had become mobilized to prevent future movements.
8. Movement observed in clay caps was discussed using three examples: CP1, WP3, and WP13.
    a. A simple exercise wherein the Mohr-Coulomb shear strength envelopes were plotted against depth showed that a loss of cohesion in the clay cap could result in scenarios where the clay cap is weaker than the granular material. This suggests stability analyses which include the clay cap may be warranted.

5 CONCLUSIONS

The design of granular shear keys was investigated by reviewing published guidelines and analyzing a collection of 38 case studies primarily from across the Canadian Prairies. To perform the analyses, a large database was compiled from the documents gathered for each case study. The analyses were used to verify and, where possible, quantify the concepts applied to the design of granular shear keys. The following chapter provides suggested guidelines for the design of these structures which reference the findings from the previous chapters. The chapter concludes with suggestions for future areas of research that would broaden the applicability of these guidelines and enhance the given recommendations.

5.1 Recommendations for Design of Granular Shear Keys

The design of granular shear keys is most often accomplished using LE analyses, wherein a factor of safety is calculated by satisfying force and moment equilibrium. The granular backfill used in shear keys must deform for shear resistance to mobilize. Current practice can be supplemented with the empirical relationships identified from the review of different designs and the ensuing slope deformations. The incorporation of these findings could potentially lead to improved and more consistent performance without dramatically increasing design effort.

The granular shear key design recommendations deriving from this work are presented in the following subsections. They are organized into the principal phases of a slope remediation project, from preliminary design to closure. The recommendations are intended to be supplemental to current practice and so, this section does not touch on all aspects of a landslide remediation project.

5.1.1 Granular properties

Compaction

Throughout this review, it was found that compaction was either not pursued or not reported in almost half of the case histories. From the discussion in Chapter 2 on the shear strength of granular backfill, and from the results of Chapter 4, it should be evident that compaction has clear benefits and should be undertaken in all granular shear keys. However, the means by which compaction is achieved should be carefully considered. Compaction should be controlled using standardized test methods such as the standard or modified Proctor tests.

Nonlinear failure criterion

Currently, it is typical to use the Mohr-Coulomb model for estimating the shear strength of the granular backfill used in shear keys. A discussion on the limitations of this linear model was given in Chapter 2, where it was concluded that this method is over-conservative. In the shallow environments in which granular shear keys are being constructed, dilation plays a significant role in the shear strength of the materials that are being used. The Power Curve model was suggested as a more realistic method of estimating the shear strength. Modern limit equilibrium software is compatible with this model and adopting it should not pose any significant new challenges. However, when using the Power Curve model to estimate shear strength, it is advisable to increase the target factor of safety to compensate for the fact that there is a dramatic drop in strength if the granular shear key experiences shear strain such that the peak strength is exceeded. It is noteworthy that the strain that a shear key must undergo to succumb to this drop in strength is typically far greater than that which the shear keys in this study experienced before stopping. For example, the large-scale direct shear tests on crushed limestone performed by Abdul Razaq (2007) reached shear strains in excess of 5%. For a granular shear key designed with an effective width of 5 m, this would equate to 250 mm of displacement. The total displacement experienced by those case histories for which displacement data was obtained is summarized in Table 5.1. The average displacement was 43.7 mm, and the maximum displacement was 195 mm.

5.1.2 Stability analysis

It is unreasonable to expect LE analyses to stop being used for the design of granular shear keys. This approach has proven adequate in that it is quick, simple, and catastrophic failures involving granular shear keys are rare. A reasonable starting point for the location can be taken from Figure 4.8 in § 4.2.1. Next, an initial estimate for the dimensions of the granular shear key can be taken from the sizing charts presented in § 4.2.2. The estimated dimensions can then be modified based on the findings in § 4.2.3, depending on the type of backfill, the back-analyzed friction angle of the slide, and whether it is expected the backfill will be compacted. A detailed design can then be determined by performing LE analyses. The software that is used also obviates the need to perform separate analyses for active and passive wedges, provided the surface search parameters are not overly restrictive.

Following the LE analyses, the geometry of the proposed granular shear key should be cross-referenced with the design-performance plots that were presented in § 4.2.2. This performance-based approach to design should be used to enhance current practice by providing estimates for the timeframe and magnitude of post-construction slope movements. Once consulted, the design should be revised as necessary and the LE models should be updated to reflect these changes. It is worth noting, the use of the M-C constitutive model is standard practice and this is not expected to change. The design-performance plots were created using real observed deformations, so this may compensate for the potential shortfalls of using the M-C constitutive model to represent the granular backfill.

It is not typical to include clay caps in the stability analyses for granular shear keys. Under peak strength conditions, the clay tends to be stronger than the granular material near surface due to the low confining stress. However, when cohesion loss is considered, the clay may become weaker than the granular material at depths that fall within the thickness of the clay cap. Thus, if a clay cap is to be constructed overtop of the granular shear key, the stability analyses should consider the inclusion of the clay cap using reduced strength parameters. This step may aid with the selection of a clay cap thickness that minimizes the risk of sliding.

Shear key constructability

Some aspects of the granular shear key design touch on the constructability of the structure. The first is the stand-up time of the soil, which will control the angle of the trench walls. This will also influence whether a steel casing will be required during rockfill column construction. The trench wall angle can be designed using principles that apply to the short-term stability of unsupported excavations. Adopting a closely-sequenced excavation sequence can permit steeper trenches than what might be suggested using 2D LE analyses. This is because the moving excavation slot is bounded by four sides rather than the two that are considered in the 2D analyses. However, a 3D analysis should be performed before justifying steeper trench walls and an acceptable short-term factor of safety should still be satisfied. If a 3D analysis is performed, the length of the open trench at any time should be determined. Otherwise, the lengths provided in § 3.4.5 should be consulted and adjusted as necessary. Benching the trench excavation is also an option that was successfully exercised in several of the case studies that were collected.

One of the advantages of granular shear keys is that the slip surface depth can be verified in the field during the excavation. The depth of the granular shear key can then be modified to suit this updated information. An estimate for the potential impact of these changes can quickly be made using the plots which included the dimensions of the shear key, presented in § 4.2.2.

Target factor of safety

The target factor of safety for granular shear key design in Canada is 1.3. A discussion was presented in Chapter 2 on the selection of an appropriate factor of safety. Granular shear keys tend to be adopted for the remediation of relatively small slides (the average volume for this compilation was 40 x 10^3 m^3). These slides are also typically Very Slow; the slides compiled for this study averaged 113.9 mm/year before remediation. The properties of the granular materials that are used as backfill are well known, and landslides are different from slope stability projects in that the factor of safety at the onset of failure is known to be at unity. Thus, a factor of safety of 1.3 may be appropriate for the typical granular shear key project, but factors of safety as low as 1.15 should be considered where appropriate. When adopting the Power Curve model, factors of safety should be increased (1.50) to accommodate for the considerable drop in shear strength post-failure at low confining stresses.

5.1.3 Role of groundwater

A high groundwater table was identified as a contributor to instability in many of the case studies. The high groundwater table was sometimes caused by natural springs, and can be accompanied by surface seepage. High groundwater tables, pore pressure and artesian conditions should be identified prior to construction and the risk of base heave, sloughing and other types of failure should be considered. In addition to the risk to the excavation, long-term drainage measures should be incorporated into the granular shear key design if any of these conditions are identified. Even in the absence of significant water conditions, insufficient drainage resulting in the saturation of the granular shear key can result in a bathtub effect. This has contributed to prolonged slope deformations in some cases (ex: AT11) due to the introduction of a large wetting area surrounding the shear key. When additional drainage measures are planned, they are typically constructed prior to the excavation of the shear key. This is done to enhance stability since the excavation of the trench reduces stability.

5.1.4 Reductions in the rate of movement

The purpose of a granular shear key is to promote stability and ultimately, to reduce the rate of sliding. Displacement data was obtained for 21 of the 38 case histories that were compiled. These are summarized below, in Table 5.1. This data shows that granular shear keys are typically very effective. For the study sample, the average rate of sliding before remediation was 113.9 mm/year, which corresponds to Varnes Class 2 – Very Slow. According to Hungr et al. (2014), this is high enough of a rate to require maintenance. After remediation with a granular shear key, the average rate of sliding dropped to 6.3 mm/year, or Varnes Class 1 – Extremely Slow. At this rate, Hungr et al. state that no maintenance is required.

5.1.5 Post-construction displacements

A major limitation of limit equilibrium analyses is that movement is not considered in the model. Predictions for the cumulative displacement and the timeframe over which it will occur can be made using the findings in § 4.2.1, the second half of § 4.2.2, and the discussion on the mobilization of shear resistance in § 4.2.3.

Nevertheless, post-construction monitoring is still recommended. The cracking or deflection observed in infrastructure such as highways or railroads can be used to determine whether movement is still taking place. A quantitative measure of the deformations provides the additional benefit of allowing for the change in the rate of deformation to be calculated. The data can then be used to refine predictions for the total time that the granular shear key will require to reach a state in which a steady rate of sliding is achieved.

If the granular shear key does not appear to be reducing slope movements to rates that are manageable or acceptable, additional measures may be deemed necessary. One option is to construct additional trenches or rockfill columns. At CP1, miniature trenched shear keys have been installed where required. At WP1, individual rockfill columns have been installed to supplement the pre-existing columns and fill gaps.

Lastly, a maintenance program should be established to ensure any drains that are present do not become clogged, and to fill surface cracks and limit the infiltration of water into the body of the slide.

Table 5.1: Summary of the rates of displacement that were compiled for granular shear keys.

Case history	Depth of sliding data (m)	Rate (mm/year)		Monitoring period (years)	Cumulative displacement (mm)
		Pre-remediation	Post-remediation		
AT2	7.0	nd	3.2	26.7	19.2
AT5	10.8	nd	9.3	10.7	195.0
AT6	3.0	nd	3.5	14.1	nd
AT8	7.3	540.0	3.0	8.3	55.4
CP1 M86.75	3.8	7.9	1.1	4.4	5.4
CP1 M86.80	4.0	11.5	0.6	9.8	9.6
CP2	6.2	nd	29.7	7.8	52.1
CP6	9.5	49.0	nd	0.0	nd
EX2	nd	365.0	0.2	6.4	51.4
WP1	10.4	nd	0.3	10.0	nd
WP2	5.1	4.8	1.0	9.8	nd
WP3	6.1	nd	2.1	3.9	nd
WP5	13.4	nd	0.2	1.4	11.9
WP6	3.7	65.0	44.7	0.8	34.4
WP7	3.1	nd	3.6	7.9	nd
WP8	nd	38.0	1.8	10.0	35.8
WP12	6.1	nd	0.2	2.8	39.1
WP13	12.8	nd	0.2	5.1	nd
WP14	5.2	1.5	1.9	0.5	24.1
WP15	6.7	nd	2.8	5.6	nd
WP16	5.9	56.0	15.9	2.0	72.5
Average	6.8	113.9	6.3	7.0	43.7

5.2 Additional Considerations

Granular shear keys are exposed to a broad range of site conditions. There are sometimes special conditions that pose challenges to the success of these structures. The following section aims to address some of these additional factors based on the successes or shortcomings observed in the case studies that were collected.

5.2.1 Changes in site conditions over time

In general, a granular shear key is designed to enhance slope stability for a slope under a set of existing conditions. Once the granular backfill has mobilized to satisfy the required shear resistance, the rate of sliding may steady out or drop to zero. If site conditions change, additional movement should be expected. Examples include adverse environmental conditions, such as significant flooding events, and changes to load restrictions for highways or railroads. If the granular material can accommodate these increases to the shear resistance required of it, deformation will persist until this new threshold is met.

It is also sometimes deemed necessary to target both shallow and deep sections of a slide. In these cases, the use of rockfill columns and conventional shear keys has been justified. Rockfill columns have been found to be a more economical approach to stabilizing deep-seated slides due to the prohibitive size of a shear key for equivalent depths. However, the more specialized equipment (particularly large-diameter augers and vibratory lances) involved in rockfill column construction can make shallow depths less economic than conventional trenched shear keys.

5.2.2 Seepage and wet conditions

Subdrains and weeping tiles are usually included in granular shear key designs, to provide a conduit for discharging the water that can build up in the free-draining granular backfill. Drains are particularly common of repairs where seepage had been identified as one of the triggers for instability. The drains are installed along the base of the shear key to ensure the structure does not become saturated. To enhance drainage, the base of the shear key itself is sometimes excavated to drain freely toward the downstream slope. One construction report cited a grade of 2% toward the buttress downslope of the key. The outfalls for any drainage system should be carefully considered. In some instances, drainage outfalls redirected water onto other parts of a slope only to later result in new instabilities in those parts.

In cases where the granular shear key is excavated into unsaturated soils, delaying the installation of drainage measures can result in long-lasting movement and/or settlement. This is because of the bathtub effect i.e. the creation of a deeper zone of wetting from a shear key becoming saturated due to insufficient drainage. Another potential source of long-term deformations due to a shear key becoming saturated is the presence of sodic soils.

While proper drainage has been found to be critical in many instances, it is not always necessary to fully drain a shear key. A shear key was constructed at one site where a spring had been identified as the trigger for slide movement. The shear key performed successfully despite not being designed to drain fully. In this case, the shear key had been constructed below the water table where the surrounding soil was already saturated. The artesian pore pressure from the spring was relieved by the shear key but the structure did not introduce a new zone of wetting. In another series of cases, the City of Winnipeg regularly installs shear keys between the regulated summer river level and the winter river water level of the Red and the Assiniboine Rivers, showing that shear keys can perform well in these saturated environments. If the shear key is constructed below the natural water table, drainage may not be a concern since the surrounding soil has already been wetted.

The free-draining nature of the backfill can also make shear keys a favourable option in situations where water levels routinely change, whether naturally or artificially. The free-draining backfill permits water levels to change without a significant build-up in pore pressures, which would negatively affect stability. In these environments, it is necessary to account for the buoyant weight of the backfill.

5.2.3 Erosive toe conditions

For a slope where the toe is slowly being eroded, a granular shear key would continue to deform until the shear resistance provided by the removed portion of the toe had been fully replaced. This concept may explain the persistence of movement in the form of creep in many of the case studies where toe erosion is potentially occurring. A study on the persistence of movement or lack thereof when granular shear keys are accompanied by toe armouring, such as when they are topped with rip rap, would shed light on this matter.

It is recommended that, in the absence of plans for toe armouring, granular shear keys intended for sites with these conditions be designed to compensate for the portion of the toe that may be removed by erosion.

5.3 Subjects for Future Research and Advancement

The subjects touched upon in this research were selected with a focus on the fundamental characteristics of these structures, such as size, placement, and backfill. There remain many other aspects of granular shear key design that would benefit from additional study. The following section introduces a few of those subjects and briefly outlines the potential benefits that could arise from studying them.

5.3.1 Instrumented field study and measuring as-built strength

A full-scale instrumented field study would be an excellent opportunity to address a variety of subjects at once. The availability of monitoring data posed a significant challenge during this research. It would be beneficial to have displacement and pore pressure data from several points upslope, downslope, and within a granular shear key. This information would be particularly useful if it were acquired from before, during, and after construction.

Figure 5.1: A steel sleeve used to help maintain hole stability during the construction of rockfill columns. Photo by Hugh Gillen.

Displacement and pore pressure data from before construction would be useful for establishing a baseline against which all following measurements could be compared. An advantage that is often cited for rockfill columns over the conventional trenched-style of shear key is that there is more control over construction-related movements. Yarechewski and Tallin (2003) cited several options including the use of steel sleeves (see Figure 5.1) to case drills holes with a high risk of sloughing, and being able to move drill hole locations to avoid less favourable conditions. Construction-related movements are also sometimes mitigated by constructing caissons or granular ribs prior to the granular shear key excavation. The benefits of each of these techniques could be verified and quantified with monitoring data from the construction period. A challenge would be posed by the considerable acceleration in the rate of slope deformation during construction though. These deformations can shear slope inclinometers, rendering them inoperable until replaced. An alternative would be to take continuous measurements of the slope using remote sensing techniques. This would be a viable approach provided the slope is not regraded extensively.

Currently, it is assumed the shape and angle of the slip surface does not change between entering and exiting the granular material. It would not be unreasonable to expect the slip surface to deviate from this path though. Multiple measuring points across the granular shear key would provide insight on the development and propagation of the slip surface within these structures.

Another benefit of having multiple measuring points both upslope and downslope of the granular shear key would be to verify whether movement in clay caps is related to the formation of active or passive wedges.

5.3.2 Documenting as-built shear strength

A major limitation to this work was a lack of as-built strength measurements for granular shear keys. This may be due to the nature of granular shear key projects, which tend to be small repairs or performed as emergency measures without lengthy periods of planning. The ability to gauge the performance of these granular shear keys would be greatly improved if field tests were performed and the data included in completion reports for these projects.

In particular, field data pertaining to the as-built shear strength, the degree of compaction, and the shear strength of the soil surrounding the excavations both before and after construction would address some of the questions surrounding the data that was presented in Table 4.8 and Table 4.9.

5.3.3 Reliability-based design

There is potential for reliability-based design theory to enhance the design of granular shear keys, by improving the economy of designs. However, on a case-by-case basis, this concept may not yet be practical for this design setting. The calculation of coefficients of variability for the many variables that are used as inputs in the design analyses would require substantial time and money to be spent during the site investigation phase. The reality is that many shear keys are constructed as part of what is classified as emergency or unplanned work due to their suitability to small- and medium-sized slides and relatively rapid constructability.

A detailed study using a small number of case studies where sufficient data is available for a reliability analysis to be carried out could help to answer whether there is an appreciable benefit from incorporating reliability concepts in granular shear key design.

5.4 Summary

The design of granular shear keys is typically accomplished using LE stability analyses, and this has been largely successful, as evidenced by the significant reductions in the rate of sliding following construction. It is expected there is potential to enhance these designs by taking advantage of the benefits of compacting the backfill, and employing nonlinear shear strength models in stability analyses. The empirical relationships identified using a collection of 39 case studies may enhance the long-term performance of these structures by providing insights into the dimensions needed to satisfy shear resistance requirements and the factors that affect the length of time over which movements will continue, and the strain these structures will undergo.

Granular shear keys can be found in a variety of environments, which sometimes requires additional site-specific considerations. Lessons learned from the case study review were presented to aid in addressing several scenarios. These scenarios were for changing site conditions over time, seepage or very wet conditions, and when erosion at the toe of the slide is likely.

Finally, the performance of granular shear keys could be enhanced with a better understanding of several subjects not touched upon by this research. An instrumented field study and additional documentation from the field might provide data that could be used to address some of the questions that remain and improve on the findings of this work.

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APPENDIX A: CASE STUDY SUMMARIES

Each case study that was collected for this research was summarized to retain the context of the remedial work that was performed. The following appendix contains each case study summary organized based on the organization affiliated with the site, whether as an owner or as a contractor in the rare case that the owner could not be identified. Table A-1 lists the sites as they are presented along with a calling code which was used to refer to sites in the text of this document.

Table A-1: Index of the case study summaries for the sites that were studied.

Calling code	Site name	Organization	Location	Page
AT1	Bogey Slide	Alberta Transportation	Alberta	A-13
AT2	Eureka River	Alberta Transportation	Alberta	A-19
AT4	Fort Vermillion Culvert	Alberta Transportation	Alberta	A-25
AT5	Hamelin Creek	Alberta Transportation	Alberta	A-34
AT6	Judah Hill – Michelin Slide	Alberta Transportation	Alberta	A-41
AT7	Kakwa River	Alberta Transportation	Alberta	A-47
AT8	Little Paddle River	Alberta Transportation	Alberta	A-52
AT9	Little Smoky River	Alberta Transportation	Alberta	A-58
AT10	West of Millarville	Alberta Transportation	Alberta	A-63
AT11	South of Sturgeon Lake	Alberta Transportation	Alberta	A-70
AT12	West of Fairview	Alberta Transportation	Alberta	A-75
CN1	Sangudo Bridge	CNR	Alberta	A-82
CP1	Bredenbury	CPR	Manitoba	A-84
CP2	Carrington	CPR	North Dakota	A-90
CP4	Emerson	CPR	Manitoba	A-97
CP5	Lanigan	CPR	Saskatchewan	A-101
CP6	Lloydminster	CPR	Saskatchewan	A-104
EX1	Bear Creek Village Condo	GES Geotech Inc.	Alberta	A-109
EX2	Bender's Park	City of Lead	South Dakota	A-112

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Calling code	Site name	Organization	Location	Page
EX3	Hagg Lake	Unknown	Oregon	A-117
EX4	San Joaquin Hills	Caltrans	California	A-120
WP1	1, 7 & 11 Evergreen Place	City of Winnipeg	Manitoba	A-123
WP2	99 & 141 Wellington Crescent	City of Winnipeg	Manitoba	A-131
WP3	Avenue de la Cathedrale	City of Winnipeg	Manitoba	A-137
WP4	Byng Place	City of Winnipeg	Manitoba	A-143
WP5	Churchill Drive	City of Winnipeg	Manitoba	A-149
WP6	Fort Rouge Park	City of Winnipeg	Manitoba	A-155
WP7	Hawthorne FPS	City of Winnipeg	Manitoba	A-162
WP8	Mager Drive	City of Winnipeg	Manitoba	A-169
WP9	Oakcrest Place	City of Winnipeg	Manitoba	A-174
WP10	Omand's Creek	City of Winnipeg	Manitoba	A-180
WP11	Pembina-Ducharme Culvert	City of Winnipeg	Manitoba	A-184
WP12	Radcliffe Road	City of Winnipeg	Manitoba	A-187
WP13	Rue Despins	City of Winnipeg	Manitoba	A-193
WP14	Rue Dumoulin	City of Winnipeg	Manitoba	A-200
WP15	Rue la Verendrye	City of Winnipeg	Manitoba	A-206
WP16	Seine River	City of Winnipeg	Manitoba	A-213

Case Study Summary Breakdown

The case study summaries that are presented here each contain background information on the site, details of the remediation, a summary of the performance of the shear key if reported, and, where applicable, some lessons that can be taken away from studying the site.

Details from the case study are also summarized. The Details Section is intended to provide basic statistics about the site, touching on the landslide and the shear key. Descriptions are given in Table A-2 for those fields which require further clarification. Fields left empty indicate that the information could not be located or adequately verified.

Table A-2: Descriptions for the fields contained in the Details Section of the case study summaries.

Field		Description
Landslide information	Landslide type	The landslide type in accordance with the descriptions provided in Hungr et al. (2013).
	Depth of movement	The depth of sliding of the landslide. Where this depth could be interpreted from multiple SIs, the depth from the SI nearest to the shear key is presented.
	Landslide velocity class	The landslide velocity class of the landslide prior to remediation, in accordance with the classes given in Cruden and Varnes (1996).
	Most recent rate	The rate of sliding from the latest data that was obtained for the site.
Monitoring information	Movement first reported	The earliest mention of slope movements that could be found in the documentation that was obtained for this review.
	Last inspection	The most recent inspection for which a report could be obtained.
Repair information	Repair type	The type of repair that was performed. In the interest of keeping the Details Section to a single page, the following shortened terms were used: “Shear key”: Trenched shear key “Columns”: Rockfill/stone column shear key

A Note on Accuracy of Information

Every effort was made to document the information contained in these summaries as accurately as possible. Due to the nature of site investigations and on-going maintenance programs, discrepancies may exist between the most recent information available and that contained in these summaries. The information is based on the reports that could be obtained and may not be fully representative of the sites and repairs in question. Inaccuracies may also exist where it was necessary to interpret data or information.

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List of Figures

Alberta Transportation Sites

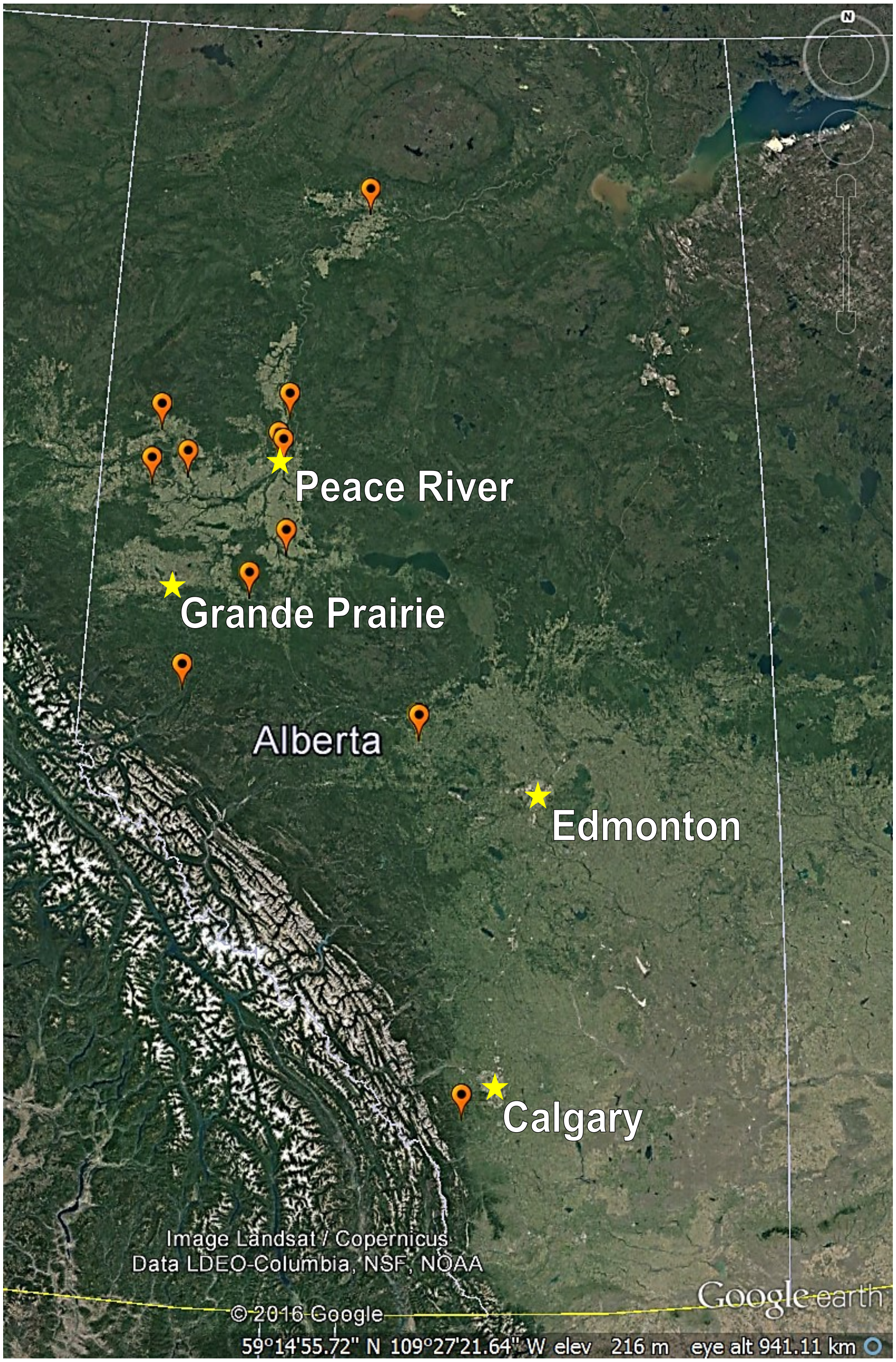


Figure A-1: Map of Alberta with Alberta Transportation granular shear key case study sites flagged. Major cities are labelled and marked with yellow stars. Image credit: Google earth V 7.1.5.1557. (December 30, 2016). Alberta, Canada. 59° 14' 55.72"N, 109° 27' 21.64"W, Eye alt 941.11 km. LDEO Columbia, NSF, NOAA. Google 2016. [April 18, 2017].

City of Winnipeg Sites

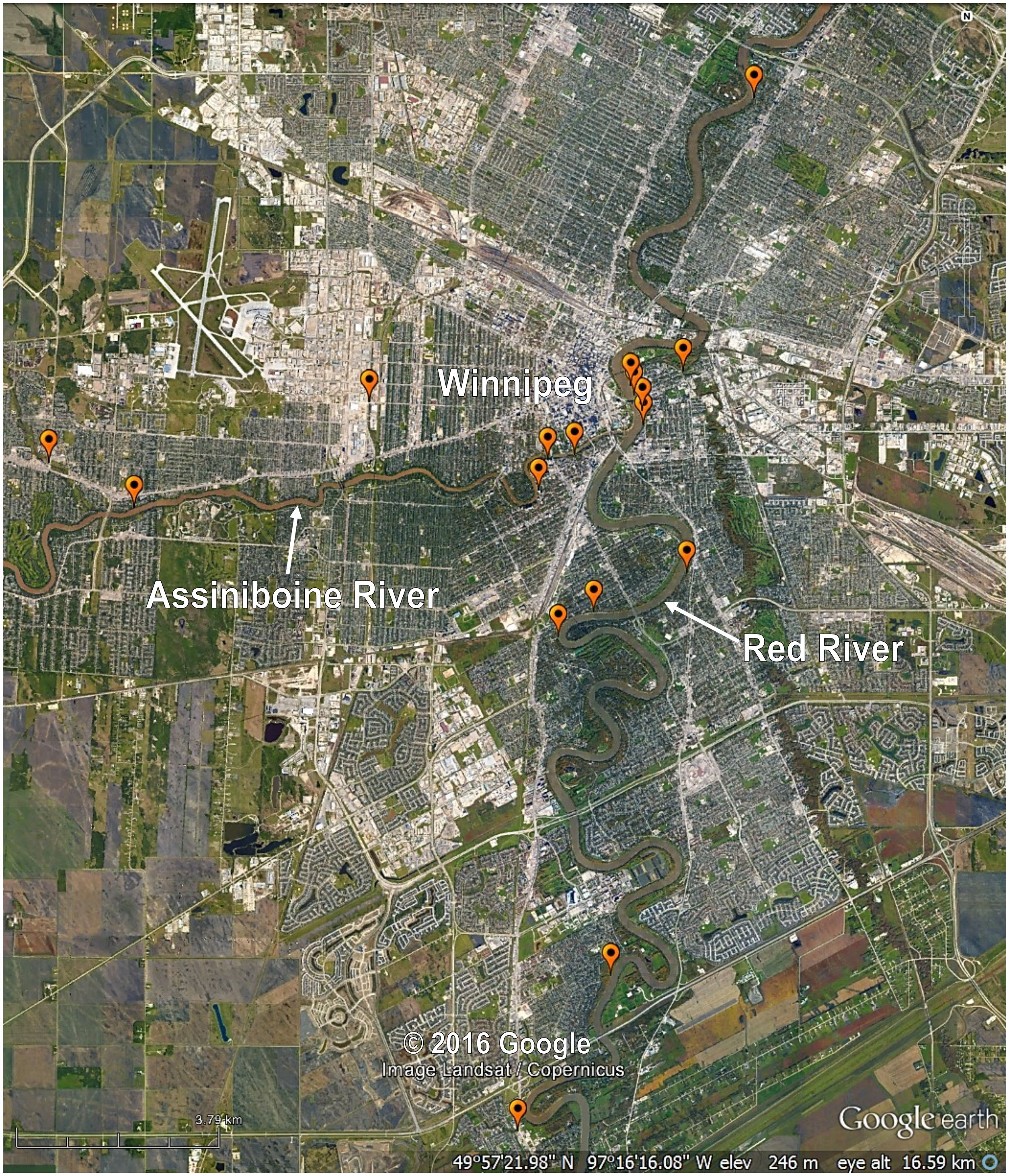


Figure A-2: Map of Winnipeg, Manitoba with the City of Winnipeg granular shear key case study sites flagged. The two major rivers running through the city are labelled. Image credit: Google earth V 7.1.5.1557. (August 24, 2015). Winnipeg, Manitoba, Canada. 49° 57' 21.98"N, 97° 16' 16.08"W, Eye alt 16.59 km. Landsat / Copernicus. Google 2016. [April 18, 2017].

AT1 – *PH68 Bogey Slide*

Hwy 743:02, KM 3.6; North of Peace River

Case Study Summary

Background

Highway 743:02, located near Peace River, Alberta, was paved in 1988/1989. Grade widening and profile improvement was completed at that time. The highway received an overlay of 170 mm of asphalt in 2000 (Alberta Transportation and Thurber Engineering Ltd., 2011).

Movement of the Bogey Slide was first observed in 2004/2005 along Highway 743. The slope movement was affecting the northbound lane (east side) of the highway so remediation was considered. In 2010, the highway was patched at the location of the slide. Heavy rain on July 27, 2011 resulted in a large degree of movement so the highway patched again on the same day, once the rain subsided. On a call-out inspection on August 2, 2011, it was reported that the highway was being patched almost every week, with cracks reappearing due to slide movements. Observations at the time included pavement cracks accompanied by vertical drops and slow seepage coming from the side slope on the east side of the highway. A local golf course owner is also said to have reported a water spring in an embankment on the south side of a nearby culvert. Recommendations from the August 2, 2011 call-out inspection included reconstructing the northbound lane of the highway with granular fill and a subdrain, and/or extending the culvert and constructing a toe berm. The drilling of a test hole and a test pit was recommended to allow for the remedial designs to be completed (Alberta Transportation and Thurber Engineering Ltd., 2011).

Remediation

Repairs were made to the Bogey Slide in fall 2011. The repairs consisted of excavating the slide mass and replacing it with granular fill. A granular shear key located at the toe of the slide was included in the repairs (Thurber Engineering Ltd., 2011).

The following is a description of the repairs which can be seen in Figure A-3. The granular shear key is 2 m wide at its base and 2 m high. It was constructed at depth, below the reconstructed embankment, with its base located approximately 6 m below surface. The shear key was set back approximately 4.5 m upslope of the slide toe. The upslope trench wall is vertical and the downslope trench wall is sloped at 1H:1V. A gravel buttress lies on top of the shear key. The buttress is approximately 2 m high and its slopes are also constructed at 1H:1V. The upslope wall of the buttress is set back 2 m from the shear key, but the downslope wall of the buttress joins the shear key to the toe of the slope. Above the buttress lies the reconstructed slope, which slopes at 4H:1V and rises 5.4 m from the toe to the crest. The slope surface is topped with 230 mm of common clay which in turn underlies 70 mm of topsoil. The upslope wall of the excavation marking the limit of the replacement fill is set back 10 m from the top of the buttress. The upslope wall of the excavation rises to the crest of the slope through a series of 5 benches, each 1 m deep and 1 m high, topped by the highway (Thurber Engineering Ltd., 2011).

A drain is located at the base of the upslope wall of the reconstructed slope excavation. The drain is a 150-mm perforated galvanized corrugated steel pipe contained within a 0.4 m x 0.4 m square cross section of washed gravel filter material in turn surrounded by non-woven geotextile. The base of the reconstructed slope excavation has a 2% grade in the direction of the buttress (Thurber Engineering Ltd., 2011).

These repairs are somewhat typical except for the following details. First, the gravel shear key excavation walls are typically sloped at 1H:1V. This was not the case for the upslope side of the shear key, with a vertical wall being excavated. While not as typical as 1H:1V slopes, vertically excavated trenches are preferable when soil conditions are such that stand-up times are sufficient for construction to proceed safely. A vertical excavation reduces the volume of soil that must be excavated and subsequently hauled and replaced. Second, the drain incorporated in this design was placed at the back of the slope as opposed to within the shear key (Thurber Engineering Ltd., 2011). The designer was interviewed and indicated this was due to the groundwater table being located above the top of the shear key. The drain was placed to intercept seepage from a spring that had been identified, to prevent the reconstructed slope from becoming saturated. Since the shear key was constructed in soil that already existed in saturated in-situ conditions, it was not deemed necessary to drain the shear key (Proudfoot D. , 2016).

Performance of Repairs

An inspection was completed on August 2, 2011. This inspection preceded the gravel shear key repairs detailed above. At that time, the highway was reported to be undergoing almost weekly asphalt patching but that cracks continued to reappear. An Alberta Transportation risk score of 60 was assigned to the site (Alberta Transportation and Thurber Engineering Ltd., 2011).

In an inspection report dated May 30, 2013, it was reported that the Bogey Slide was repaired in the fall of 2011 and the highway was repaved in 2012. No seepage and only minor cracks in the asphalt were observed at that time. No signs of movement were observed and an Alberta Transportation risk score of 16 was assigned. However, an additional slide named the Double Bogey Slide was reported. This new slide was located south of the original site (Alberta Transportation and Thurber Engineering Ltd., 2013). As of the 2015 inspection, repairs for the Double Bogey Slide had not been performed but were being discussed. The Bogey Slide was assigned an Alberta Transportation risk score of 16 at the time of the report (June 10, 2015) (Alberta Transportation and Thurber Engineering Ltd., 2015).

Lessons Learned

The trigger for the slide movement appears to have been a spring identified by the owner of the adjacent golf course, but the slide was also found to be influenced by heavy rain fall (Alberta Transportation and Thurber Engineering Ltd., 2011). This source of instability appears to have been satisfactorily addressed by the slope repairs that were undertaken. This case study showed that a shear key can be constructed below the groundwater table, and that it does not necessarily have to be drained if that is the case.

South of the Bogey Slide, a new slide, dubbed the Double Bogey Slide, occurred in 2012/2013. This slide occurred in the cut slope of a borrow pit pond and was believed to be caused by a combination of the cyclic filling and draining of the pond, a high groundwater table, and weathering and progressive loss of cohesion in the clay cut slopes. One of the two remedial options recommended for this slide involves the reconstruction of the slope using geogrid reinforced gravel and a gravel shear key at the base of the excavation. The authors of the report state this option would allow the repairs to stay within the existing right-of-way for the highway (Alberta Transportation and Thurber Engineering Ltd., 2013). Given the success of the Bogey Slide repairs, this option seems feasible.

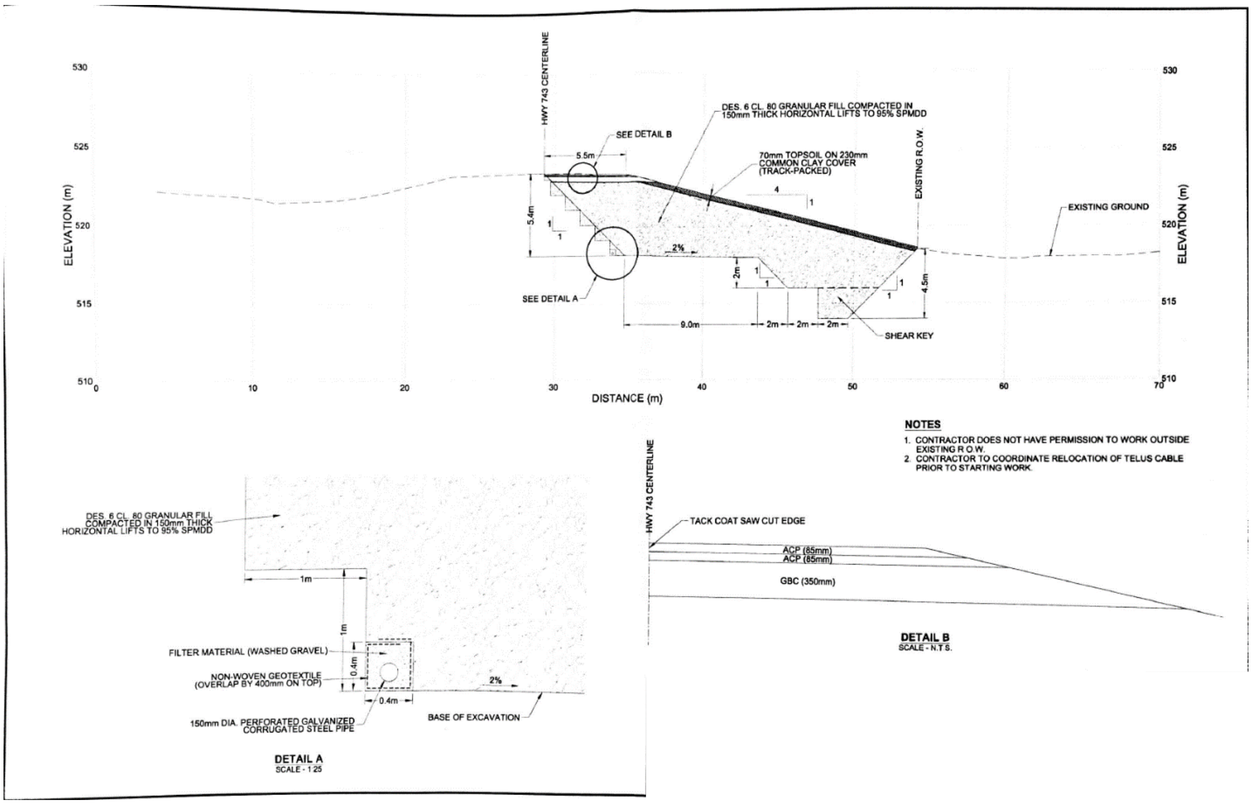


Figure A-3: Cross section of the Bogey Slide shear key and reconstructed slope (Thurber Engineering Ltd., 2011). Reproduced with permission from Alberta Transportation.

Case Study Details

Landslide Information

Landslide type:	Rotational slide	*Pre-support rate (mm/yr):*	
Trigger:	Seepage	*Landslide velocity class:*	
Depth of movement (m):	~2.9	*Most recent rate (mm/yr):*	N/A
Sheared material:	Embankment fill and native soil		

Landslide Dimensions

Width (m):	45	*Length (m):*	20.6
Height (m):	4.8	*Slope (°):*	16

Monitoring Information

Movement first reported:	2004 – 2005	*Last inspected:*	May 30, 2013
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level	~1.7 m	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	Yes	*Surface water type:*	Stream/Creek

Repair Specifications

Repair type:	Buttress & Shear key	*Repair date:*	January 1, 2011
Base width (m):	2	*Trench slope ratio (H:V):*	1:1
Length (m):	42	*Drainage (yes/no):*	Yes
Granular height (m):	2	*Backfill:*	
Overburden (m):	3.3 (granular) + 0.3 m (fill)		

Overburden material(s): Granular fill topped with 70 mm topsoil on 230 mm common clay cover

Additional Site Information

Cross Section

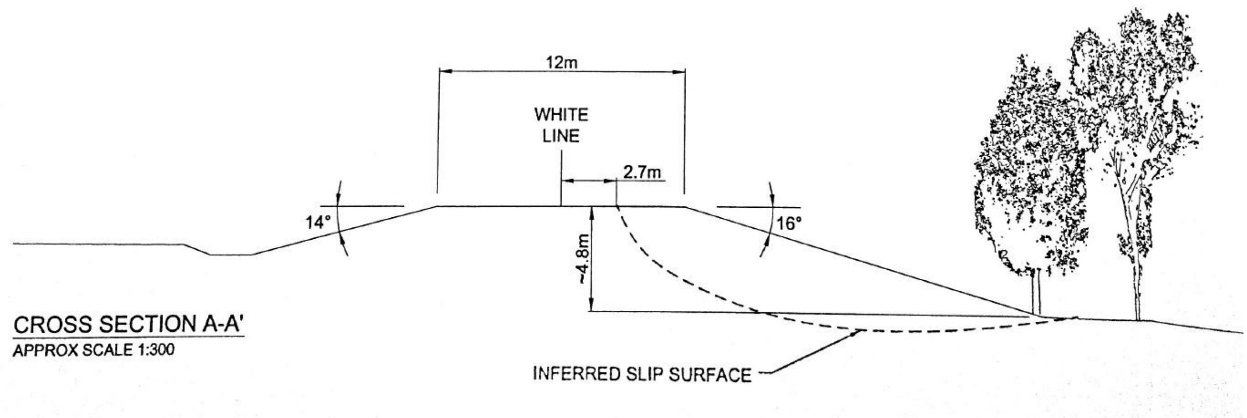


Figure A-4: Cross section of inferred slip surface for the Bogey Slide (Alberta Transportation and Thurber Engineering Ltd., 2011). Reproduced with permission from Alberta Transportation.

Plan Map

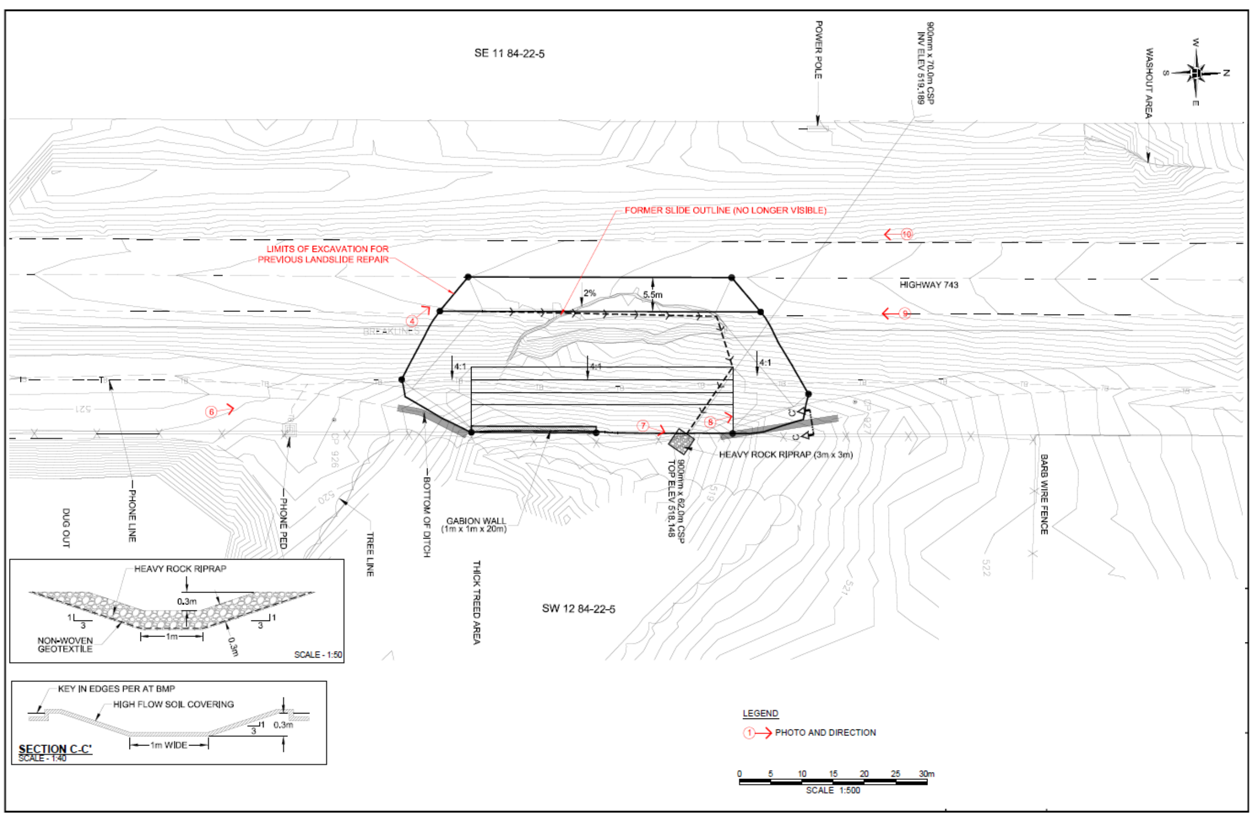


Figure A-5: Plan view of the slope reconstruction and shear key (Alberta Transportation and Thurber Engineering Ltd., 2013). Reproduced with permission from Alberta Transportation.

AT2 – *PH10 Eureka River Site #2*

Hwy 726:02, KM 10.85 & 10.6 (8 km South of Worsley)

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by Thurber Engineering Ltd. in *Alberta Transportation Landslide Risk Assessment, Peace Region (Peace River-High Level Area), Site PH 10, (Old PH10, Sites#1@km 10.85 & #2@km 10.6* (2014).

Background

This landslide occurred in June 1988 in the Eureka River Valley, on a slope below Highway 726:02 inclined at roughly 12 degrees or 4.5H:1V. The slide was measured to be 100 m wide and 180 m long. The river valley is approximately 30 m high here. The failure itself was found to be relatively shallow, resting at only 4 m below the road and only 2 m below surface closer to the toe. The toe was well developed and extended above the native peat, blocking the river.

The soil comprising the slide was found to be predominantly high plastic, slickensided lacustrine clay. Clay fill and discontinuous peat up to 1 m thick were identified overlying the clay in some areas. A single borehole drilled in 2008 identified 16 m of high plastic lacustrine clay. Clay till was found below it, to a depth of 20 m. Two inclinometer test holes drilled in 1998 found gravel and clay fill up to 2 m thick lying above the lacustrine clay. In 2002, slope inclinometers indicated movement was occurring between 9-16 m below surface.

Remediation

Remediation initially consisted of 212 750-mm-diameter stone columns with lime, spaced on a 2-m grid and extending to a depth of 12 m beneath the road, between Station 10+740 and 10+800. It was reported that designs for a shear key, toe berm, drainage trenches, a gravel basal blanket, and lightweight fill (sawdust) were submitted but that it is unclear whether they were ever actually implemented. It appears the remediation of this slope was investigated significantly at one point, considering the complexity of the remedial measures designed for the site. For the shallow sliding depth identified after the 1988 failure, it is unclear why stone columns were adopted since this type of remediation technique is more typical of deeper-seated failures. Information gathered in 2002 indicated that sliding was occurring between 9-16 m below surface though, so it is possible that this was observed in 1988 but was not recorded in the reports that were available.

Additional remediation was proposed in 2004 after new slope inclinometer data indicated the slope was experiencing a deep-seated failure. Short term measures included patching and monitoring, and armouring and re-aligning the river to reduce toe erosion. Long term measures consisted of a road re-alignment in combination with backslope flattening, re-routing a powerline, and the consideration of either a pile wall, river crossing, or toe berm. A shear key or slope replacement were not considered due to the deep-seated nature of the slide. Subdrains were also not considered due to the low permeability of the in-situ clays.

Performance of Repairs

After the initial remediation, five standpipe piezometers were installed. Additional settlement was reported in 1997. Likely in response to this, it was recorded that the slope was instrumented with two inclinometers in 1998 but it was reported that the information gathered from these was insignificant. The road was repaved in 1999 but an additional 100 mm of settlement was observed in 2002. Three SIs and three pneumatic piezometers were installed at that time, downhill of the road. Movement was identified at depths of 9-16 m.

In 2003, settlement and cracks in a settlement bowl spanning 40 m in width were reported. Extensive slumping uphill of the road, cracks and scarps located between the road and the river, and small scale sliding adjacent to the river were also reported. An inspection in June, 2004 found a crack had developed between Sites #1 and #2 (km 10.75).

In 2008, the crack identified in 2004 had been found to have extended to 40 m in length. The possibility that a single large slide block encompassing Sites #1 and #2 might be developing was considered. At that time, one slope inclinometer and one pneumatic piezometer were installed. Only one of the slope inclinometers from 1998 and one from 2002 were found still operational. One of the piezometers from 2002 was still operational too.

Lessons Learned

While the slide was reported to have been shallow, remediation was initially pursued using rockfill columns which are more typical of deeper-seated failures. Sliding was eventually identified at depth but the columns were still found to be insufficient. The description of the stone columns appears to indicate they were installed below the road. It is possible the stone columns would have performed better had they been installed closer to the bottom third of the slide, as is typical of other landslide remediation projects. The same is true of trenched granular shear keys and in fact, placement near the scarp of a slide is sometimes avoided so as not to increase the driving forces behind the slide.

Case Study Details

Landslide Information

Landslide type:	Planar slide	*Pre-support rate (mm/yr):*	Unknown
Trigger:	Construction	*Landslide velocity class:*	1 - Extremely Slow
Depth of movement (m):	3, 9, 13, 14.5	*Most recent rate (mm/yr):*	< 3
Sheared material:	Slickensided CH clay		

Landslide Dimensions

Width (m):	150	*Length (m):*	180
Height (m):	30	*Slope (°):*	12

Monitoring Information

Movement first reported:	June 1, 1988	*Last inspected:*	October 1, 2008
SIs installed:	6	*SIs active last inspection:*	3
Piezos installed:	9	*Piezos active last inspection:*	2

Groundwater Information

Groundwater level (mbgl):	2 – 4.5	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	Yes	*Surface water type:*	River

Repair Specifications

Repair type:	Buttress & Columns	*Repair date:*	February 1, 1988
Column diam. (m):	0.75	*Rows:*	
Length (m):		*Drainage (yes/no):*	No
Layout:	Square	*Spacing (up x across [m]):*	2 x 2
Granular height (m):	12	*Backfill:*	Lime added
Overburden (m):			
Overburden material(s):			

Additional Site Information

Stratigraphy

PH10 Eureka River Site #2				
Case study location:	Hwy 726:02, KM 10.85 & 10.6 (8 km South of Worsley)			
Depth (m)	**Soil descriptions (See Thurber Engineering Ltd. 2009)**	**Type**	**Source**	**Structure**
0.0 - 16.0	2008: One test hole; drilled by Thurber; located between Site #1 and #2; indicated high plastic lacustrine clay to 16 m in depth *1988: Six test holes; drilled by ATU; located across Site #2; indicated presence of discontinuous clay fill over discontinuous peat spanning up to 1 m in thickness, all overlying lacustrine clay reaching 15.7 m in depth* *1998: Two inclinometer test holes; drilled by Thurber; located at Site #2; indicated presence of gravel, possible clay fill spanning 2 m in thickness, all overlying lacustrine clay extending to 15.7 m in depth*		L	
			L	
			L	
			L	
			L	
			L	
			L	
			L	
			L	
			L	
			L	
			L	
			L	
			L	
			L	
			L	
16.0 - 20.0	Single 2008 test hole by Thurber: Clay till to 20 m depth		G	
			G	
			G	
			G	

Legend					
Type		**Source**		**Structure**	
Organics/Topsoil		Fill	F	Slickensided	
Clay		Fluvial	R	None	
Silt		Lacustrine	L		
Sand		Marine	M	**Groundwater (in Type)**	
Till		Glacial	G	Seepage	s
Stone/rock		Assumed	*Italicized*	GWT	-------------
Interbedded		Unknown	?		

Figure A-6: Schematic representation of the stratigraphy at Eureka River Site #2.

Cross Section

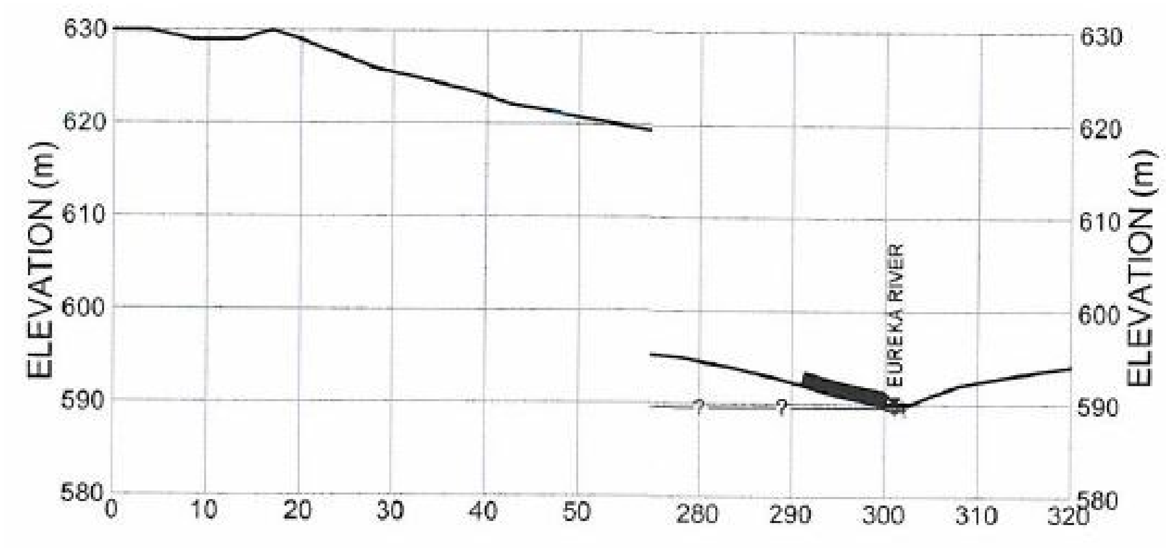


Figure A-7: Partial cross section for Eureka River Site 2 (Thurber Engineering Ltd., 2014). Reproduced with permission from Alberta Transportation.

Plan Map

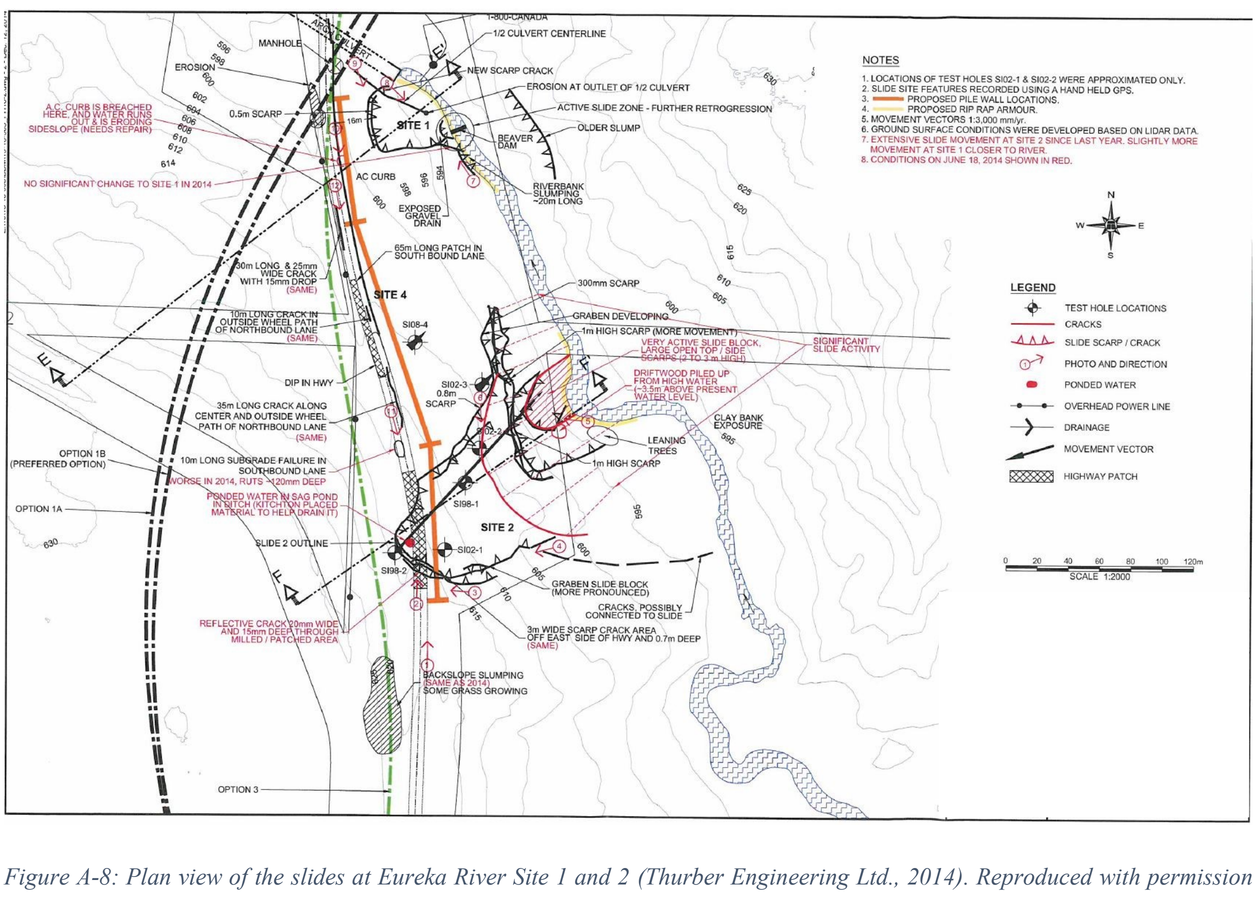


Figure A-8: Plan view of the slides at Eureka River Site 1 and 2 (Thurber Engineering Ltd., 2014). Reproduced with permission from Alberta Transportation.

AT4 – *PH16 Fort Vermillion Culvert*

Hwy 88:18, 3 km West of Fort Vermillion

Case Study Summary

Background

Approximately 3 km west of Fort Vermillion, a watercourse culvert was being constructed across Highway 88. The excavation was to be sloped at 1H:1V with a 4-m-wide bench level with the culvert crown, and additional benches every 5 m above the crown or as needed to maintain stability. Silt layers were observed in the eastern slope of the excavation so the slope was excavated at a shallower angle as a precaution. The western slope, however, was excavated as planned (1H:1V) (Amec Foster Wheeler, 2003).

The western slope failed on September 17, 2003. The failure began at the road surface, marked by approximately 4 m of vertical movement, and extended to the base of the excavation. The failed slope measured 12 m high. Movement is believed to have occurred first in the northern part of the slope, followed by the southern part. The trigger is thought to have been high pore pressures from construction (Amec Foster Wheeler, 2003).

On October 17, 2003, the invert of the culvert near the outlet was observed to have risen approximately 0.9 m. The culvert was also found to have deformed after the western slope failure. The downstream slope was being constructed to a slope of 3H:1V consistent with the final designs at the time (Amec Foster Wheeler, 2003).

A site visit on October 21, 2003 revealed the natural slopes downstream of the culvert were approximately 2H:1V or steeper. The visit also discovered recent (within 10 years) signs of instability including slump blocks. Lacustrine clays and silts were identified in the excavation. The silts were exposed at the upper part of the eastern slope and it was reported that they had also been observed in the western slope. The natural soils comprising the lower part of the slope were classified as high plastic clays (Amec Foster Wheeler, 2003).

During the October 21, 2003 visit, observers identified cracking in the fill placed on either side of the culvert. The western slide movement was judged to be toward the northeast and the eastern slide movement was judged to be toward the northwest. However, the damage to the culvert was attributed solely to the western slide movement. The overall slope of the fill on the west side of the culvert was measured to be between 4.3H:1V and the slope was measured to be approximately 6 m high at this time. The slip surface was defined by test pits at the toe and midslope, and the slide scarp (Amec Foster Wheeler, 2003).

In the excavated area, adjacent to the culvert, an upward shear thrust was observed in the underlying clay and the thrust was reported to have experienced 200 mm of movement overnight. A test pit adjacent to the culvert, excavated to locate the shear plane, revealed massive, high plastic, inorganic clay lacking varves or silt layers. Water was observed seeping into discontinuities believed to belong to the shear zone during this excavation. After leaving the test pit open overnight, the slip surface experienced an additional 120 mm of movement. The slip surface was described as appearing horizontal and sitting at approximately 255.0 masl, level with the culvert inlet elevation at the outlet. The slip surface is believed to have reached an elevation of 254.5 masl further into the slide mass, which would be below the culvert elevation (Amec Foster Wheeler, 2003).

Three test pits, including the one mentioned above, were excavated and encountered clay fill overlying lacustrine silty clay, all over clay till. The reported soil descriptions are included in the Stratigraphy, below. The slip surface was noted to have been identified in two of the three test pits due to significant movement of the test pit walls (Amec Foster Wheeler, 2003).

Remediation

The initial failure in September, 2003 was handled by excavating most of the failed material but it is reported the basal slip surface was likely not removed. Construction resumed, with the culvert ending up being backfilled to a height of 2-3 m above the crown. The distortions observed on October 17, 2003 resulted in the removal of the fill placed adjacent to the culvert. This action was taken to limit further damage to the culvert (Amec Foster Wheeler, 2003).

It was decided that the existing culvert would have to be lowered to the design invert and that the ultimate downstream slope of the completed structure would be 3H:1V for long term stability. Remediation was required to be carried out in a timely manner to avoid the onset of the cold season. Finally, remediation had to protect the culvert from high horizontal loads that could result in distortion. The remedial measures that were considered and the advantages and disadvantages are discussed below (Amec Foster Wheeler, 2003).

Flattening the downstream slope was expected to satisfy long-term stability. However, slope instability during construction and high horizontal loading to the culvert in the long term were risks associated with this solution. An extension of the culvert would also be necessary, which could result in delays (Amec Foster Wheeler, 2003).

The excavation of the slide mass was expected to effectively address the failed soil mass. This solution was unfavourable for the large excavation volume involved and the risk of sliding during the excavation. The construction of a shear key was favourable because the weak clays at the slide toe would be replaced with gravel, allowing for a significantly smaller excavation than a full replacement of the slide mass. The stability of the slide during construction could be managed through design and sequencing the excavation. The shear key was also expected to effectively manage horizontal loading on the culvert. The concerns for this solution included the need for additional soil data and stability analyses to determine the depth of the shear key, and the temporary risk to stability during the excavation (Amec Foster Wheeler, 2003).

A pile wall would share the same benefits provided by a shear key and would not pose as high of a risk to stability as a shear key excavation would. However, this solution would be expensive and would involve delays because it would require equipment that was not already present (Amec Foster Wheeler, 2003).

For the advantages and disadvantages presented above, it was ultimately decided that a shear key would be adopted. The proposed shear key was designed to allow for the previously designed slope of 3H:1V to be completed. A back-analysis performed on the failed western slope determined the slip surface required a friction angle of 19 degrees to yield a factor of safety of 1.0 (Amec Foster Wheeler, 2003).

The target factor of safety for the downstream slope was 1.3. This target was found to be unachievable with the original fill so a higher quality fill, modeled with a friction angle of 28 degrees and a pore pressure parameter of 0.1, was brought in. These required properties were found to be achievable with a silty, low to medium plastic, clay fill compacted to 95% standard Proctor maximum dry density at near optimum moisture content. To address the slide trigger, which had been reasoned to be high pore pressure, a recommendation was made to incorporate sand drainage blankets in the fill (Amec Foster Wheeler, 2003).

A temporary factor of safety of 1.1 was desired for the shear key excavation, to prevent damage to the culvert. A 2D slope stability analysis indicated that a 5H:1V excavation would be necessary to achieve this target. The designers were aware that a sequenced excavation would result in the trench wall being supported by the adjacent sides of the excavation, such that the trench wall could be steeper than 5H:1V. The trench walls ended up being sloped at 1H:1V from the base of the excavation to a height of 2 m above the base of the clay fill. The trench slopes above that level were designed to slope at 2H:1V. Any fill more than 2 m above the crown of the culvert was required to be removed (Amec Foster Wheeler, 2003).

It was decided that excavation segments would be 5-10 m wide at the base and proceed from west to east. The completed shear key was designed to have a base width of 18 m and a base length of 40 m, which was the maximum allowable length that could be permitted within the width of the ravine while still maintaining a maximum of 1H:1V trench slopes. The structure was to be keyed in to the till, which would place it about 4 m deep at the culvert outlet since the till was encountered at 251 masl there (Amec Foster Wheeler, 2003).

The final shear key design also incorporated drains to address the high degree of seepage observed previously in the silt layers within the lacustrine clay. Drainage blankets (constructed with sand or gravel) were to be placed at either end of the shear key to intercept seepage coming from the sides of the ravine. The shear key was also designed to drain freely towards the downstream slope (Amec Foster Wheeler, 2003).

The decisions and final designs documented above represent a typical shear key construction project. This structure was designed to resist shear movement and to enhance drainage, which are typical functions of shear keys. However, these structures are typically designed to resist shear movement coming from a direction perpendicular to the trend of the shear key. In this case, the shear key was intended to resist shear forces coming from either end of the trench in addition to those applied by the embankment. The placement of the shear key near the toe of the embankment is typical though, as is the geometry of the structure (Amec Foster Wheeler, 2003).

Performance of Repairs

The repairs detailed in the previous section were completed in the Fall of 2003. The performance of the gravel shear key repairs at the Fort Vermillion Culvert appears to be satisfactory. Two reports were available following the repairs, from site visits made in 2005 and 2006. In these reports, the only signs of damage were erosion gullies in the slopes, where vegetation had not yet recovered. Erosion gullies in the east slope, observed on June 20, 2005 (Thurber Engineering Ltd., 2006), were repaired by the May 9, 2006 visit (Thurber Engineering Ltd., 2006). New minor erosion gullies had, however, occurred in that slope again. These new gullies were located 20 m north of the previous ones. Observers also noted seepage and a wet area near the toe of the slope (Thurber Engineering Ltd., 2006).

In the culvert, silt had accumulated. On the south slope, near the culvert outlet, recent erosion gullies were observed. At the culvert inlet, soil erosion above the rip rap level had increased between June 2005 and May 2006. A silt fence located above the culvert outlet was found damaged. The south-east drainage ditch lined with geotextile and rip rap was noted as not showing any sign of malfunctions (Thurber Engineering Ltd., 2006).

Overall, the repairs were deemed to be successful and only basic maintenance costs were incurred. The erosion that was observed was attributed to the fact that the slopes are long and steep. A long-term solution involved the flattening of the east slope and constructing three flat benches across it to intercept and redirect surface runoff. This solution was only provided as an option should the need arise (Thurber Engineering Ltd., 2006).

Lessons Learned

The most significant lesson from this case study was that a gravel shear key can be an effective solution to resisting shear forces coming not only from the direction perpendicular to the trench but also parallel with it. Also noteworthy is the direct acknowledgement of the 3D edge effects in the trench excavation, upon which designers relied to excavate slopes steeper than what the 2D analysis had suggested were feasible.

Case Study Details

Landslide Information

Landslide type:	Rotational slide	*Pre-support rate (mm/yr):*	
Trigger:	Construction	*Landslide velocity class:*	
Depth of movement (m):		*Most recent rate (mm/yr):*	
Sheared material:	Slickensided lacustrine silty clay		

Landslide Dimensions

Width (m):	60	*Length (m):*	32
Height (m):	12	*Slope (°):*	45

Monitoring Information

Movement first reported:	September 17, 2003	*Last inspected:*	May 9, 2006
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	0	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	Yes	*Surface water type:*	Culvert

Repair Specifications

Repair type:	Shear key	*Repair date:*	October 24, 2003
Base width (m):	18	*Trench slope ratio (H:V):*	1:1
Length (m):	40	*Drainage (yes/no):*	Yes
Granular height (m):	6.4	*Backfill:*	Pitrun, 95% SP
Overburden (m):	4.5		
Overburden material(s):	Clay fill		

Additional Site Information

Stratigraphy

PH16 Fort Vermillion Culvert				
Case study location:	Hwy88:18, 3 km West of Fort Vermillion			
Depth (m)	**Soil description (for approximately mid-slope)**	**Type**	**Source**	**Structure**
0.0 - 4.0	Clay fill; soft and wet; derived from lacustrine silty clay; 4-5% wet of optimum MC; optimum MC = 18%; Phi' = 25, r_u = 0.25		F	
			F	
			F	
			F	
4.0 - 11.0	Lacustrine silty clay; medium to high plastic clay layers with silt lenses; numerous slickensides in location of shear movement; LL 45, PL 20, MC 33.6; UU Su 40 kPa; seepage, B = 0.5	s	L	
		s	L	
		s	L	
		s	L	
		s	L	
		s	L	
		s	L	
		s	L	
11.0 - 17.0+	Clay till; silty clay with some sand and a trace of gravel; firm to hard; pink-grey in colour; considered competent		G	
			G	
			G	
			G	
			G	
			G	

Legend					
Type		**Source**		**Structure**	
Organics/Topsoil		Fill	F	Slickensided	
Clay		Fluvial	R	None	
Silt		Lacustrine	L		
Sand		Marine	M	**Groundwater (in Type)**	
Till		Glacial	G	Seepage	s
Stone/rock		Assumed	*Italicized*	GWT	
Interbedded		Unknown	?		

Figure A-9: Schematic representation of the stratigraphy at the Fort Vermillion Culvert site.

Cross Section

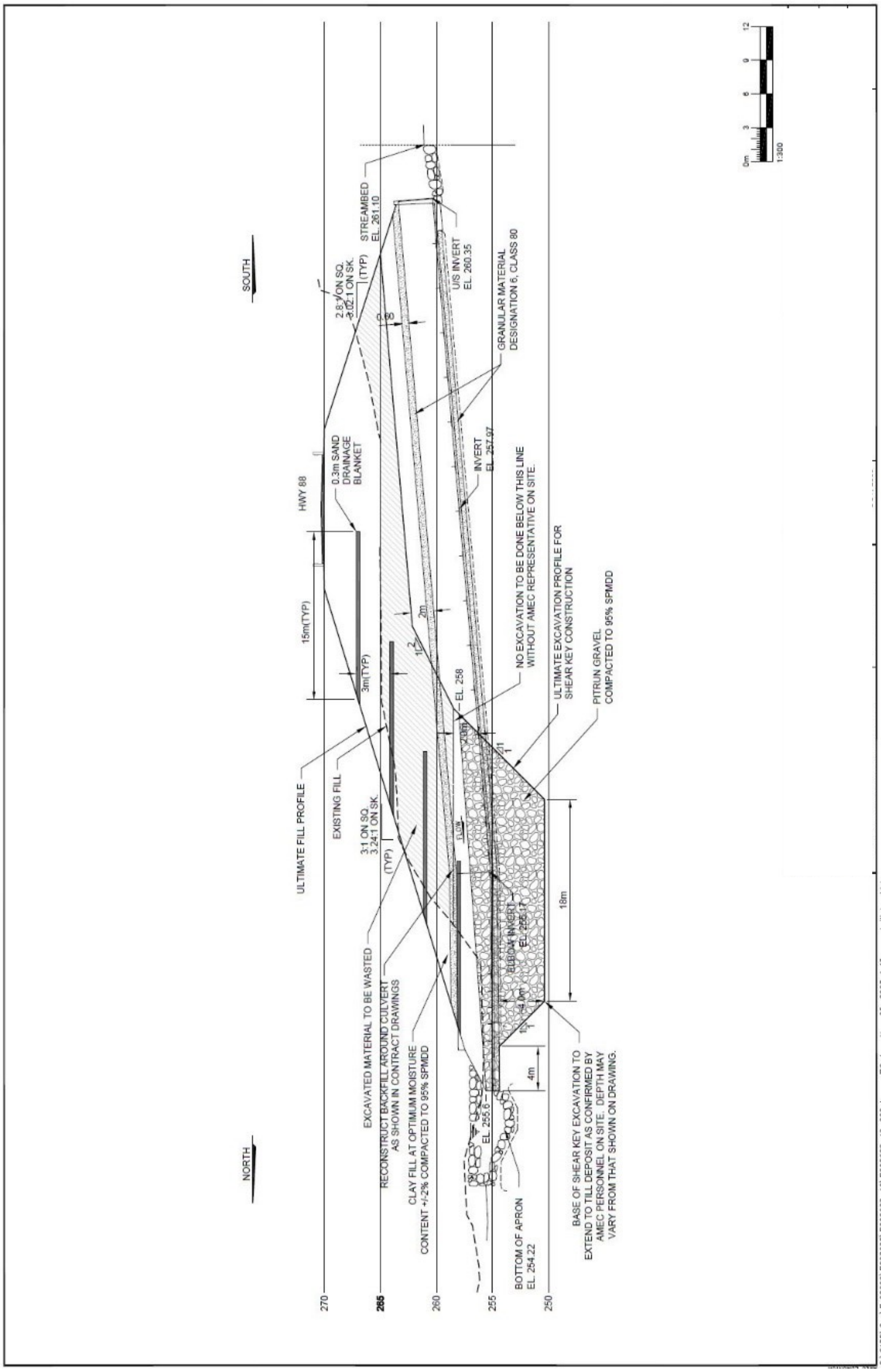


Figure A-10: Cross section of the repairs performed at the Fort Vermillion Culvert site (Amec Foster Wheeler, 2003). Reproduced with permission from Alberta Transportation.

Plan Map

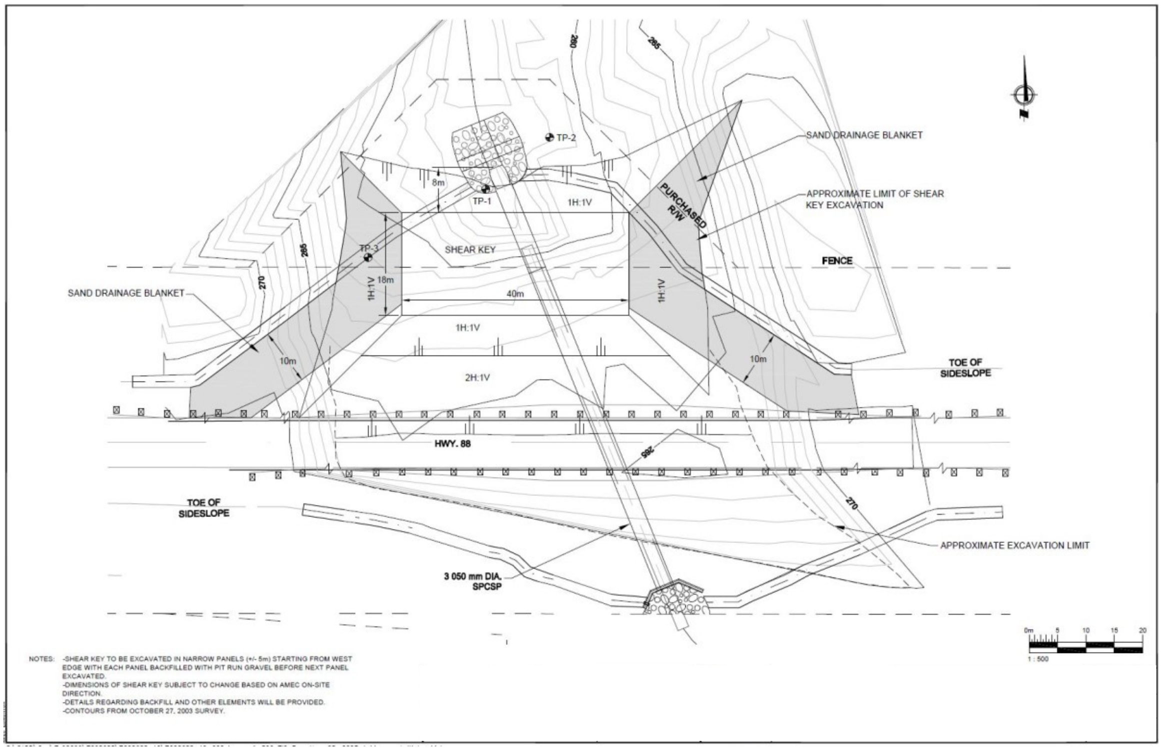


Figure A-11: Plan map of the repairs at the Fort Vermillion Culvert site (Amec Foster Wheeler, 2003). Reproduced with permission from Alberta Transportation.

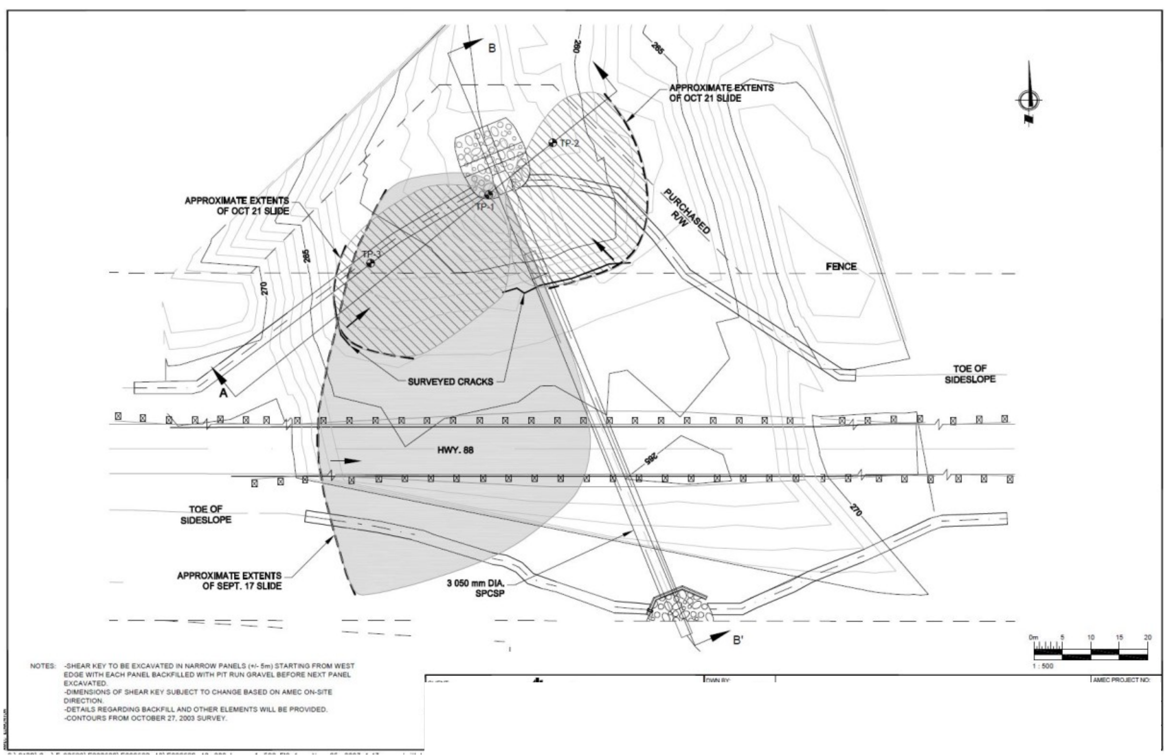


Figure A-12: Plan view of the slides at the Fort Vermillion Culvert site (Amec Foster Wheeler, 2003). Reproduced with permission from Alberta Transportation.

AT5 – *GP24 Hamelin Creek*

Hwy 725:02; UTM: N 6,207,357 E 361,697

Case Study Summary

Background

The Hamelin Creek site experienced several slides because of construction activity involving a new arch culvert in 2002-2003. Three causes were identified: the stockpiling of fill on the valley slopes, the use of frozen fill in the construction of the embankment above the new arch culvert, and the realignment of Hamelin Creek and its tributary. The realignment of Hamelin Creek involved lowering the channel's gradeline by 5 m which resulted in a subcut of the toe of the slope (Karl Engineering Consultants Ltd., 2009).

The initial instabilities were attributed to the stockpiling of fill on the valley slopes. Construction resumed by using staged earthwork toe berming techniques. Staging was coordinated with the staged construction of the new arch culvert. These operations extended through the winter though. Consequently, the stockpile fill material used for the fill embankment froze. The embankment suffered a sliding failure toward the end of construction and the failure was attributed to the frozen fill being incompetent due to its snow and ice content. This sliding was also found to have damaged the new culvert. Stone columns were installed in September 2003 to address these new instabilities. Additional settlement and lateral spreading of the fill was anticipated afterward, as the shear resistance of the stone columns mobilized, pore pressure from the frozen fill dissipated, and long term consolidation reached completion (Karl Engineering Consultants Ltd., 2009).

Finally, the channel was realigned after construction was finished. The valley slopes were destabilized yet again as the toe was steepened and undercut. These processes resulted in slumping at the toe of the fill around the culvert. Minor bank failures were also observed along the channel (Karl Engineering Consultants Ltd., 2009).

Remediation

The primary remediation technique used at the Hamelin Creek site was the toe berming of the slope to finish construction. However, this appears to have been a temporary measure and the first long-term remedial work that was completed was the installation of stone columns once the frozen fill failed. The stone columns permitted construction crews to leave the lower portion of the fill (up to the top of the culvert). The columns were extended to the bottom of this lower portion and topped with a basal drainage blanket. Using stone columns permitted the frozen fill that remained to drain, while also partially replacing it without excavating to the bottom again and potentially disturbing the culvert (Karl Engineering Consultants Ltd., 2009).

Stone columns for slope stabilization are typically used when deep shear movement is encountered. In this case, they were used the stabilize the slope through partial soil replacement and by providing a conduit for drainage. This special case is due to the known presence of the frozen fill placed earlier in the project. Stone columns are also favoured when the full excavation required for a shear key is not practical due to the potential destabilizing effect. This was true of the conditions at this site, particularly due to the presence of the already damaged culvert.

Performance of Repairs

The complexity of the conditions and repairs at the Hamelin Creek site justified abundant instrumentation. A report filed in April 2009 indicated that since the end of 2003, when construction was completed, movement rates had been reduced significantly. Pore pressures had also declined significantly. However, the authors stated additional settlement and lateral spreading was still expected. These movements were anticipated to necessitate future pavement patching. At that time (April 2009), the site was assessed an Alberta Transportation risk score of 36 (Karl Engineering Consultants Ltd., 2009).

The most recent report that was available was from May 28, 2014. This visit indicated sliding activity was persisting in the form of lateral spreading and creep, although rates were minimal. The Alberta Transportation risk score had decreased to 8, due to the probability factor having decreased substantially since 2009 (Alberta Transportation and Karl Engineering Consultants Ltd., 2014).

While the above-mentioned movements continue, the site was considered to have been successfully remediated due to the significant decrease in movement rates. However, typical shear key and stone column repairs are expected to essentially halt movement after 3-5 years. This site seems to be experiencing movement beyond this timeframe, perhaps due to the length of time associated with the dissipation of pore pressure as well as the length of time required for the modified channel to mature.

Lessons Learned

While several issues were encountered throughout the construction history of this site, the remediation was ultimately judged to be successful. Stone columns were shown to be an effective method for accelerating the dissipation of pore pressure and facilitating drainage. They were also demonstrated to be effective in resisting lateral loads and preventing additional landsliding. Detailed slope inclinometer records are expected to be beneficial to characterizing the mobilization of shear resistance in the stone columns. This site was also the subject of a conference paper entitled *Stabilization of a Highway Embankment Fill Over an Arch Culvert Using Stone Columns* by Tweedie et al. (2004).

Case Study Details

Landslide Information

Landslide type:	Rotational slide	*Pre-support rate (mm/yr):*	
Trigger:	Construction	*Landslide velocity class:*	1 - Extremely Slow
Depth of movement (m):	6 – 17.5	*Most recent rate (mm/yr):*	5 – 7
Sheared material:	Clay fill (Partially frozen)		

Landslide Dimensions

Width (m):	300	*Length (m):*	76
Height (m):	24	*Slope (°):*	18.4

Monitoring Information

Movement first reported:	May 24, 2003	*Last inspected:*	May 28, 2014
SIs installed:	10 – 12	*SIs active last inspection:*	8
Piezos installed:	20	*Piezos active last inspection:*	8

Groundwater Information

Groundwater level (mbgl):		*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	No	*Surface water type:*	Stream/Creek

Repair Specifications

Repair type:	Columns	*Repair date:*	September 1, 2003
Column diam. (m):	1.2	*Rows:*	17
Length (m):	47.5	*Drainage (yes/no):*	No
Layout:	Triangle	*Spacing (up x across [m]):*	2.2 x 2.5
Granular height (m):	6 – 8	*Backfill:*	
Overburden (m):	0		
Overburden material(s):	None		

Additional Site Information

Stratigraphy

GP24 Hamelin Creek

Case study location: Hwy 725:02; UTM: N 6,207,357 E 361,697

Depth (m)	Soil description	Type	Source	Structure
0.0 - 8.0	Clay fill		F	
			F	
			F	
			F	
			F	
			F	
			F	
			F	
8.0 - 13.0	Partially frozen clay fill (replaced with clay fill after remediation)		F	
			F	
			F	
			F	
			F	
13.0 - 18.5	Clay till		G	
			G	
			G	
			G	
			G	
			G	
18.5 - 20.5	Clay shale		*M*	
			M	
20.5 - 23.0	Siltstone		*M*	
			M	
			M	
23.0 - 23.5	Clay shale and Clay silt		*M*	
23.5 - 24.0	Siltstone		*M*	
24.0 +	Clay shale		*M*	

Legend

Type		Source		Structure	
Organics/Topsoil		Fill	F	Slickensided	
Clay		Fluvial	R	None	
Silt		Lacustrine	L		
Sand		Marine	M	**Groundwater (in Type)**	
Till		Glacial	G	Seepage	s
Stone/rock		Assumed	*Italicized*	GWT	-------------
Interbedded		Unknown	?		

Figure A-13: Schematic representation of the stratigraphy at the Hamelin Creek site.

Cross Section

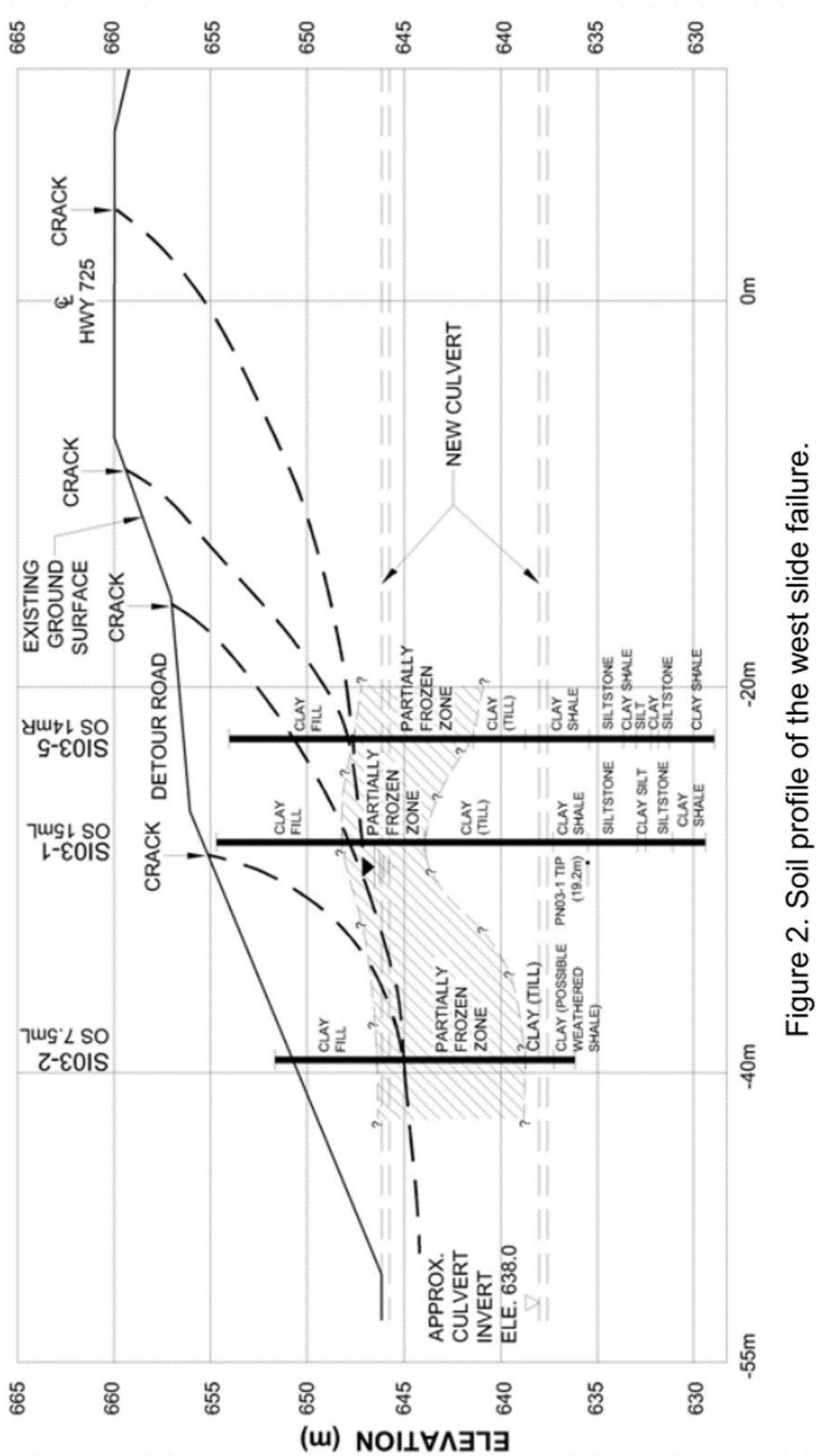


Figure A-14: Cross section showing the inferred slip surfaces for the west slide at the Hamelin Creek site. Reproduced from Tweedie et al. (2004), with permission.

Plan Map

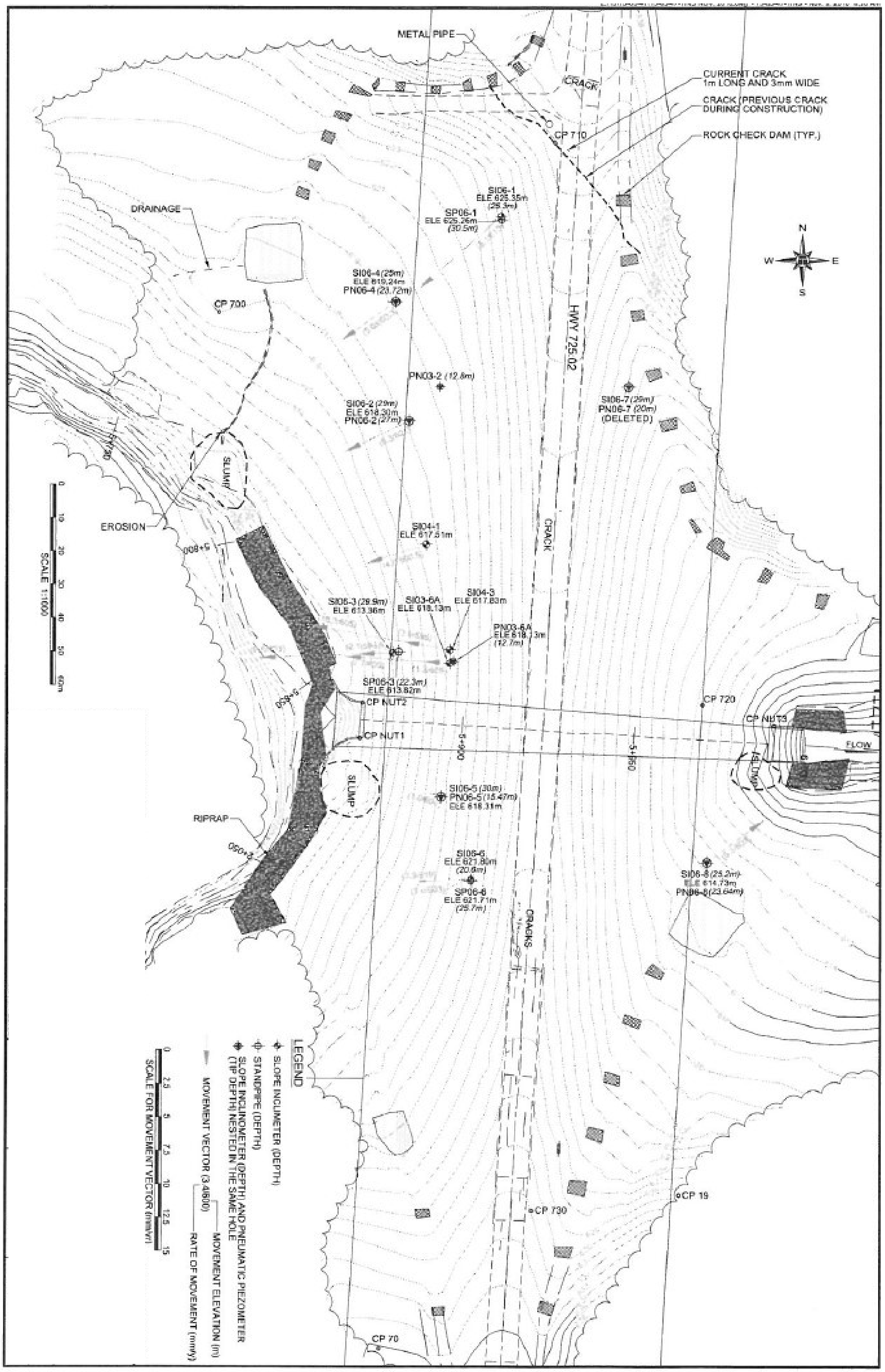


Figure A-15: Plan view of the Hamelin Creek site, showing the culvert, highway and SIs (Thurber Engineering Ltd., 2010). Reproduced with permission from Alberta Transportation.

AT6 – *PH31 Judah Hill – Michelin Slide*

Hwy 744:04, KM 57.8

Case Study Summary

Background

This stretch of Highway 744:04 runs in a north-south direction across ancient landslide terrain located between the Heart River to the east and the Peace River to the west. Mapping in the region identified 14 slides and 7 geographical sections over only a 2.5 km long section. The road was paved in 1984 (Kjelland et al., 2009) and was reported to have annual average daily traffic of 570 vehicles for the year 2012 (Alberta Transportation and Thurber Engineering Ltd., 2013).

The Michelin Slide is one of the slides located along Judah Hill. Slide issues were first reported for the Michelin Slide in 1986 (Kjelland et al., 2009). In the summer of 1997, a 50-70 m wide slide occurred at KM 57.8. Repairs were made in May 1999 but movement continued in the backscarp and at the north end of the site, between KM 57.8 and the repairs belonging to the Makeout Slide nearby. New movement was also detected along the east side of the highway, toward the Heart River. It was postulated that a larger slide complex was developing, spanning 500 m in width and extending toward the Peace River. The Heart River Valley slide backscarp measured 120 m in width and was reported to follow the gas line right-of-way crossing the site. Judah Hill has continued to exhibit instabilities in more recent times. In May 2013, the Sunshine Landslide failed at KM 58.3. Cracking in the road near the Michelin Slide was also reported at that time (Alberta Transportation and Thurber Engineering Ltd., 2013).

The groundwater table was identified at depths ranging from 10-19 m in one location and between 6.7-12.6 m at another location in the slide (Alberta Transportation and Thurber Engineering Ltd., 2013). A drilling program in the area logged sediments to a depth of 165 m before hitting bedrock. The first 20 m were described as a sequence of bedded silts, clays and sands from Glacial Lake Peace. Slickensided surfaces were observed running horizontally or at low angles. The following 60 m contained a diamicton of silt tills and debris flows. This unit also contained slickensided surfaces at high angles near the interface with the overlying Glacial Lake Peace deposits. Rhythmically bedded silts, clays and sands were identified for the 75 m. These sediments contained slickensided surfaces like those in the upper 20 m sequence, as well as a 25-m thick sand unit. This unit is also suspected for several the slides in the region. Finally, a 10-m thick sequence of sand, silt and gravel was identified overlying the bedrock. The bedrock was classified as being part of the Shaftesbury Formation Cretaceous marine shales (Kjelland et al., 2009).

Remediation

Conditions along Judah Hill are variable and have resulted in numerous remediation measures being implemented in the area over time. In May 1999, the 50-70 m wide slide from the summer of 1997 was repaired with a shear key, toe buttress and light-weight shredded tire fill. The use of shredded tire fill prompted the naming of the slide as the Michelin Slide (Alberta Transportation and Thurber Engineering Ltd., 2013).

Per a journal article by Kjelland et al. (2009), the Michelin Slide and the Makeout Slide both received shear keys as part of repairs. The Makeout Slide was reported to have received a toe berm with layers of geogrid in 2005, which incorporated a shear key.

Worth noting is that the nearby Fence Slide and the Makeout Slide were both reported to have had as many as 490 stone columns installed in 1988. The exact number of stone columns at each respective site was unspecified. The columns were said to be 760 mm in diameter, extending 6.7 m deep and installed in a 3-row grid with 2 m spacing. They were installed along the downslope side of the road, which was taken to mean the west side. The road was then realigned after the columns proved ineffective (Kjelland et al., 2009).

The Michelin Slide has a history of repairs that includes the use of drainage measures, road realignments, retaining wall installations and shear key construction. Drainage measures were typically implemented concurrently with additional remedial works. Water transfer from the uphill ditch to the downslope side of the road was attempted with the intention of keeping discharge away from unstable zones. However, this was not successful, with seven instabilities occurring after the road realignment near the Michelin Slide due to changes in the hydraulic regime (Kjelland et al., 2009).

After drainage measures were found to be ineffective, earthworks solutions consisting of slide excavation and rebuilding of the roadway with granular fill were attempted. Slope grading was anticipated to negatively impact the surrounding slopes so a lightweight fill was selected for use at the Michelin Slide. This fill was composed of lightweight tire shreds. The repairs were considered permanent and stable (Kjelland et al., 2009).

The north side of the Michelin Slide was repaired in 1997 using 91 concrete piles 760 mm in diameter, 20-25 m long and anchored with 24 m long anchors with 12 m grouted sections. The reason for this different approach was that grading was anticipated to accelerate movement downslope. The pile wall was successful except it was noted by Kjelland et al. (2009) that downslope movement has brought into question for how long this success would last. This is due to the passive confinement pressure gradually being reduced because of this movement.

The costs for remediation were summarized and included $800 000 spent on the repair and monitoring of CNR, Fence and Michelin Slides as of 1994. An additional $2 million was spent on the pile wall at the Michelin Slide in 1997, which includes the berm and tire fill at the adjacent slide (Kjelland et al., 2009).

Performance of Repairs

The 1988 stone column repairs at the nearby Fence and Makeout Slides did not prove to be effective, nor did the road realignment. The repairs completed in May 1999 do not appear to have halted movement, instead resulting in additional backscarp movement. Additional movements and landslides nearby and on the opposite site of the ridge on which the highway is situated indicate that the entire area is unstable (Kjelland et al., 2009).

Several slope inclinometers were installed at the Michelin Slide. One slope inclinometer, SI98-10i, recorded 56.1 mm of movement since being installed in 1998. Over the same period, deformation rates were measured as high as 12.5 mm/year and a general trend of increasing rates was recognized. An incremental rate of 3.9 mm was recorded between fall 2008 and June 2013. Another inclinometer, SI94-43i, was installed in 1994 about 450 m downslope of the road and 100 m below the level of the road. It recorded an incremental rate of 3 mm at a depth of 25-28 m between fall 2011 and June 2012 (Alberta Transportation and Thurber Engineering Ltd., 2013).

Six slope inclinometers were installed in March 2010: SI10-4, SI10-5, SI10-6, SI10-7, SI10-8 and SI10-9. The first three (SI10-4 to 6) were installed in the Michelin Slide scarp, at the crest of the Heart River slope on the east side of the highway. SI10-4 did not record any movement, SI10-5 recorded 226 mm in the upper 12 m toward the Heart River, and SI10-6 recorded 199 mm of movement in the upper 6 m toward the Heart River. SI10-5 sheared at a depth of 2.1 m (Alberta Transportation and Thurber Engineering Ltd., 2013).

SI10-7 to 9 were installed along the crest of the Michelin Slide on the west side of the road. SI10-7 recorded 11 mm of movement between depths of 3-8 m, and 5 mm of movement between depths of 9-12 m. SI10-8 recorded 22 mm of movement between depths of 14-18 m. SI10-9 recorded 3 mm of movement at depths of 6-8 m and 12 mm at depths of 12-14 m (Alberta Transportation and Thurber Engineering Ltd., 2013).

Lessons Learned

The Michelin Slides are an example of a complex system that continued to move after an equally complex history of repairs. Notable lessons from this case study include the observation that additional instabilities were caused by drainage outfalls redirecting water onto other parts of the slope. Another lesson from this study was that some of the repairs, particularly the pile wall, were anticipated to become ineffective as the downslope soil moved away and reduced passive confining pressure. Lastly, the use of lightweight fills proved to be somewhat effective in reducing driving forces and contributing positively to the overall stability of the slope.

Case Study Details

Landslide Information

Landslide type:	Planar slide	*Pre-support rate (mm/yr):*	
Trigger:		*Landslide velocity class:*	1 - Extremely Slow
Depth of movement (m):	0 – 12, 25 – 28	*Most recent rate (mm/yr):*	3.0 – 3.9
Sheared material:	Glaciolacustrine sediments		

Landslide Dimensions

Width (m):	50-70	*Length (m):*	
Height (m):		*Slope (°):*	

Monitoring Information

Movement first reported:	January 1, 1988	*Last inspected:*	June 3, 2013
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	6.7-19	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	River

Repair Specifications

Repair type:	Buttress & Shear key	*Repair date:*	1988, May 1999+
Base width (m):		*Trench slope ratio (H:V):*	
Length (m):		*Drainage (yes/no):*	Yes
Granular height (m):		*Backfill:*	Shredded tire
Overburden (m):			
Overburden material(s):			

Additional Site Information

Stratigraphy

PH31 Judah Hill - Michelin Slides				
Case study location:	Hwy 744:04, South of Peace River			
Depth (m)	**Soil description**	**Type**	**Source**	**Structure**
0.0 to 20.0	Bedded silts, clay and sands derived from Glacial Lake Peace; slickensided surfaces observed running horizontally at low angles		L	
			L	
			L	
			L	
20.0 to 80.0	Diamicton of silt tills and debris flows; possess high angle slickensides at interface with deposits lying above		G	
			G	
			G	
			G	
			G	
			G	
			G	
			G	
			G	
			G	
			G	
			G	
80.0 to 155.0	Rhythmically bedded silts, clays and sands; horizontal to low angle slickensides present; suspected of the cause of numerous slides in this region; contains a 25 m thick sand unit		?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
155.0 to 165.0	Sequence of sand, silt and gravel		?	
			?	
BEDROCK: Shaftesbury Formation Cretaceous marine shale				

Legend

Type		Source		Structure	
Organics/Topsoil		Fill	F	Slickensided	
Clay		Fluvial	R	None	
Silt		Lacustrine	L		
Sand		Marine	M	**Groundwater (in Type)**	
Till		Glacial	G	Seepage	s
Stone/rock		Assumed	*Italicized*	GWT	
Interbedded		Unknown	?		

Figure A-16: Schematic representation of the stratigraphy at the Judah Hill site.

AT7 – *GP2 Kakwa River*

Hwy 40:38 (20 km South of Kakwa River)

Case Study Summary

Background

The earliest available report for this slide is from June 12, 2001. Observations made by assessment crews summarize the movement as being erosion and seepage related, with deep shear movement not being apparent. A view of the slide can be seen in Figure A-16, below. The headscarp of the slide was noted to be regressing, with 0.5-1.0 m of regression observed since the previous year's inspection. At this point in time, there was approximately 1.0 m of setback distance between the headscarp and the guardrail for the highway, which crews estimated would allow between 1-2 years before remediation would be required. Conceptual designs for remedial works had been submitted in 1998 and comprised a granular drainage blanket and earth works. The decision following this site visit was to continue monitoring the slide and to remediate when the headscarp began to encroach on the roadway shoulder (Alberta Transportation and EBA, 2001).

By June 25, 2003, the headscarp had regressed to within 0.5 m of the guardrail, which crews remarked was nearing a critical distance to affect the highway. The width of the anticipated slide reconstruction was estimated to be 50 m at that time. Surveying placed the highway elevation at 496 masl. The ground level was surveyed to be at 486-487 masl for the first level bench of the excavation and the toe of the slope was cited to fall within the footprint of this bench. The remediated slope was designed to have a slope of 2.5H:1V. Testpitting at this time confirmed a bedrock elevation of 480-486 masl, which would require 4-7 m of overburden to be excavated to reach the base of the remedial structure. Remediation was anticipated to take place within one year of this visit, following the 1998 designs and making use of a government gravel pit 14 km away for soil disposal. Granular backfill for the slope reconstruction was stated to be well-graded pit run, which was to be compacted in 0.3 m lifts. The hauling, processing and placement of the granular fill and hauling of excavated material was estimated at that time to be the major cost component of the project (Alberta Transportation and EBA, 2003).

Figure A-17: Picture of the slide looking south, in the direction of Grande Cache (Alberta Transportation and EBA, 2001). Reproduced with permission from Alberta Transportation.

Remediation

In November 2004, the slope was reconstructed. Detailed designs and a tender package for the remediation had been submitted in February of 2004. The revised cost estimate was $450 000, versus the previous estimate of $330 000.The slope ended up being reconstructed at a slope of 3H:1V, with an erosion protection blanket over the slope face. Two outlet subsurface drains were noted to be installed as follows: the first was constructed at a lower elevation, and the second was installed higher for redundancy. The first drain was observed to yield continuous flow, while the second was found to be dry. These drains were installed along the base of the granular toe key (Alberta Transportation and EBA, 2005).

The repairs appeared to be functioning well as of May 5, 2005 and no shear movement was apparent. By June 21, 2006, re-vegetation of the slope surface had begun and the drains were found to be working still. Seepage from the lower drain remained constant. The upper subdrain was found to contain trapped water (Alberta Transportation and Karl Engineering Consultants Ltd., 2006).

The repairs for this slide appear to be consistent with standard practice. The failed mass of the slope was removed and reconstructed with gravel at a typical slope angle. A toe key was added to prevent future slides in the native soil underlying the reconstructed slope. Drains were incorporated to ensure the reconstructed slope would not become saturated, especially since seepage had been identified as one of the original triggers of slope movement. Re-vegetation of the slope surface is typical and is often used to reduce the risk of surface erosion as well as for aesthetic reasons.

Performance of Repairs

The repairs appear to have been successful in achieving the primary purpose of preventing additional movement. Furthermore, the added drainage appears to have addressed seepage in an effective way and continued to function as of the last available report in June of 2006. Notes were found indicating the Spring 2005 Instrumentation Monitoring Report contained current instrumentation data and details of the interpretations, but this report was not located. It is therefore unknown how quickly the repairs took effect. However, no comments were made in the available reports indicating cracking had been observed in the road surface (Alberta Transportation and Karl Engineering Consultants Ltd., 2006).

Lessons Learned

This case study serves as a good example of the successful remediation of a slide through the identification of the sources responsible for initiating movement and subsequently addressing them with repairs that efficiently reduced the effects of those sources.

Case Study Details

Landslide Information

Landslide type:	Soil creep	*Pre-support rate (mm/yr):*	
Trigger:	Seepage & Toe erosion	*Landslide velocity class:*	
Depth of movement (m):	Shallow	*Most recent rate (mm/yr):*	Negligible
Sheared material:	Surficial soils		

Landslide Dimensions

Width (m):	50	*Length (m):*	
Height (m):	10	*Slope (°):*	

Monitoring Information

Movement first reported:	January 1, 1998	*Last inspected:*	June 21, 2006
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):		*Surface water at toe (yes/no):*	
Seepage detected (yes/no):	Yes	*Surface water type:*	

Repair Specifications

Repair type:	Shear key	*Repair date:*	November 1, 2004
Base width (m):		*Trench slope ratio (H:V):*	
Length (m):		*Drainage (yes/no):*	Yes
Granular height (m):		*Backfill:*	
Overburden (m):			
Overburden material(s):			

Additional Site Information

Plan map

0 400
SCALE (metres)

Figure 1
1993AerialPhotograph

Figure A-18: Aerial photograph of the Kakwa River site (Alberta Transportation and EBA, 2005). Reproduced with permission from Alberta Transportation.

AT8 – *NC59 Little Paddle River*

Hwy 43:16, KM 41.2-41.5, 3.5 km NW of Mayerthorpe, Alberta

Case Study Summary

Background

Landslide activity at the Little Paddle River site was first observed in 1995. The landslide spanned a width of 75 m (Thurber Engineering Ltd., 2009). In 2004, the highway was divided and two lanes were added. This construction activity reactivated sliding over a span of 120 m (Thurber Engineering Ltd., 2007). The Little Paddle River runs parallel to the highway embankment, approximately 80 m away from the eastbound lanes and 120 m from the westbound lanes as measured from plans of the site. A geotechnical investigation in 2005 revealed the sliding was taking place in high plastic clay, just above the underlying clay till. The movement was attributed to high pore pressures in the clay because of the placement of fill for the new highway (Thurber Engineering Ltd., 2009).

In 2007, a 60-70 m slump occurred along the northeast side of the Little Paddle River channel east of the Little Paddle River bridge. The slump was attributed to the excavation of a diversion channel for a tributary of the Little Paddle River during the highway construction activity in 2004 (Thurber Engineering Ltd., 2009).

Remediation

Following the 2004 reactivation of sliding activity, subexcavation and reconstruction was attempted but movement persisted. After the geotechnical investigation in 2005, recommendations were made and repairs were completed in 2006. The repairs consisted of the partial excavation of the fill, the installation of wick drains, stone columns, and a small toe berm. Paving was completed in 2007. The 2007 slump was repaired by flattening the slope (Thurber Engineering Ltd., 2009).

The repairs to the original slide are typical where a competent material has been identified at depth (i.e. the clay till) and when pore pressure has been identified as a problem. Rockfill columns and wick drains both permit the accelerated dissipation of pore pressure. Flattening of the slope that failed in 2007 is also a typical repair when subcutting and oversteepened slopes are identified as the trigger, and when the surface footprint is permissible. When the right of way does not permit slope flattening, the use of rip rap and riverbank armouring may be employed instead, to reduce erosion.

Performance of Repairs

The slope that was flattened in 2007 after experiencing a slump failure was observed in 2008. No distress was identified and vegetation appeared to have re-established itself on the slope (Thurber Engineering Ltd., 2009). The annual inspection in 2008 identified cracking in the new pavement though. The original highway had also experienced minor cracking. Further monitoring was recommended after the 2008 inspection (Alberta Transportation and Thurber Engineering Ltd., 2008).

The cracking observed in the highway in 2008 is a typical result of stone column repairs. It is well known that movement can persist for 3-5 years after construction, as the shear resistance of the system is mobilized. Instrumentation monitoring reports up to Spring 2014 were available for this site. The rate of movement was observed to decrease significantly between 2006 and 2008. As of 2014, movement appeared to be in a state of creep, with the rate of movement ranging from 1-3 mm/year (Alberta Transportation and Golder Associates, 2015). Fluctuations in the rate could possibly be related to seasonal groundwater variation. Overall, this range of movement appears to be concurrent with what is typical and the remediation seems to have been successful. The trigger for landslide movement at this site had been identified as high pore pressure, so the installation of wick drains and the added drainage conduit provided by the rockfill columns seem to have addressed that.

Lessons Learned

In this case study, it was shown that rockfill columns could be used to successfully stabilize a landslide and to aid in the dissipation of excess pore pressure. The case was typical in that the rate of movement was observed to rapidly decrease for the first several years, followed by a long period of creep.

The slope flattening for the 2007 slump failure showed that this is an effective way of increasing stability when river erosion and subcutting has been identified as the source of instability. It is likely the quick regrowth of the surface vegetation also had a positive influence on the stability of the slope.

Case Study Details

Landslide Information

Landslide type:	Compound slide	*Pre-support rate (mm/yr):*	112
Trigger: Construction & Channel realignment		*Landslide velocity class:* 2 - Very Slow	
Depth of movement (m):	7.1-12, 2.9-4.7	*Most recent rate (mm/yr):*	1-3
Sheared material:	CH Clay		

Landslide Dimensions

Width (m):	120	*Length (m):*	114.5
Height (m):	11.5	*Slope (°):*	11.4

Monitoring Information

Movement first reported:	1995, 2004	*Last inspected:*	Spring 2014
SIs installed:	8	*SIs active last inspection:*	1
Piezos installed:	14	*Piezos active last inspection:*	1

Groundwater Information

Groundwater level (mbgl):	<5	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	No	*Surface water type:*	River

Repair Specifications

Repair type:	Buttress & Columns	*Repair date:*	September 1, 2006
Column diam. (m):	1.8	*Rows:*	3
Length (m):	185	*Drainage (yes/no):*	Yes
Layout:	Triangle	*Spacing (up x across [m]):*	3 x 2.8
Granular height (m):	9	*Backfill:*	
Overburden (m):	2.5-3.5		
Overburden material(s):	"Common (clay) fill" and sand drainage blanket		

Additional Site Information

Stratigraphy

NC59 Little Paddle River				
Case study location:	Hwy 43:16, KM 41.2-41.5			
Depth (m)	**Soil description**	**Type**	**Source**	**Structure**
0.0 - 3.3	Clay fill		F	
			F	
			F	
			?	
3.3 - 6.1	Clay (CI) with sand layers		?	
			?	
6.1 - 10.3	Clay (CH)		?	
			?	
			?	
			?	
10.3 +	Clay (till)		G	

Legend					
Type		**Source**		**Structure**	
Organics/Topsoil		Fill	F	Slickensided	
Clay		Fluvial	R	None	
Silt		Lacustrine	L		
Sand		Marine	M	**Groundwater (in Type)**	
Till		Glacial	G	Seepage	s
Stone/rock		Assumed	*Italicized*	GWT	
Interbedded		Unknown	?		

Figure A-19: Schematic representation of the stratigraphy at the Little Paddle River site.

Cross Section

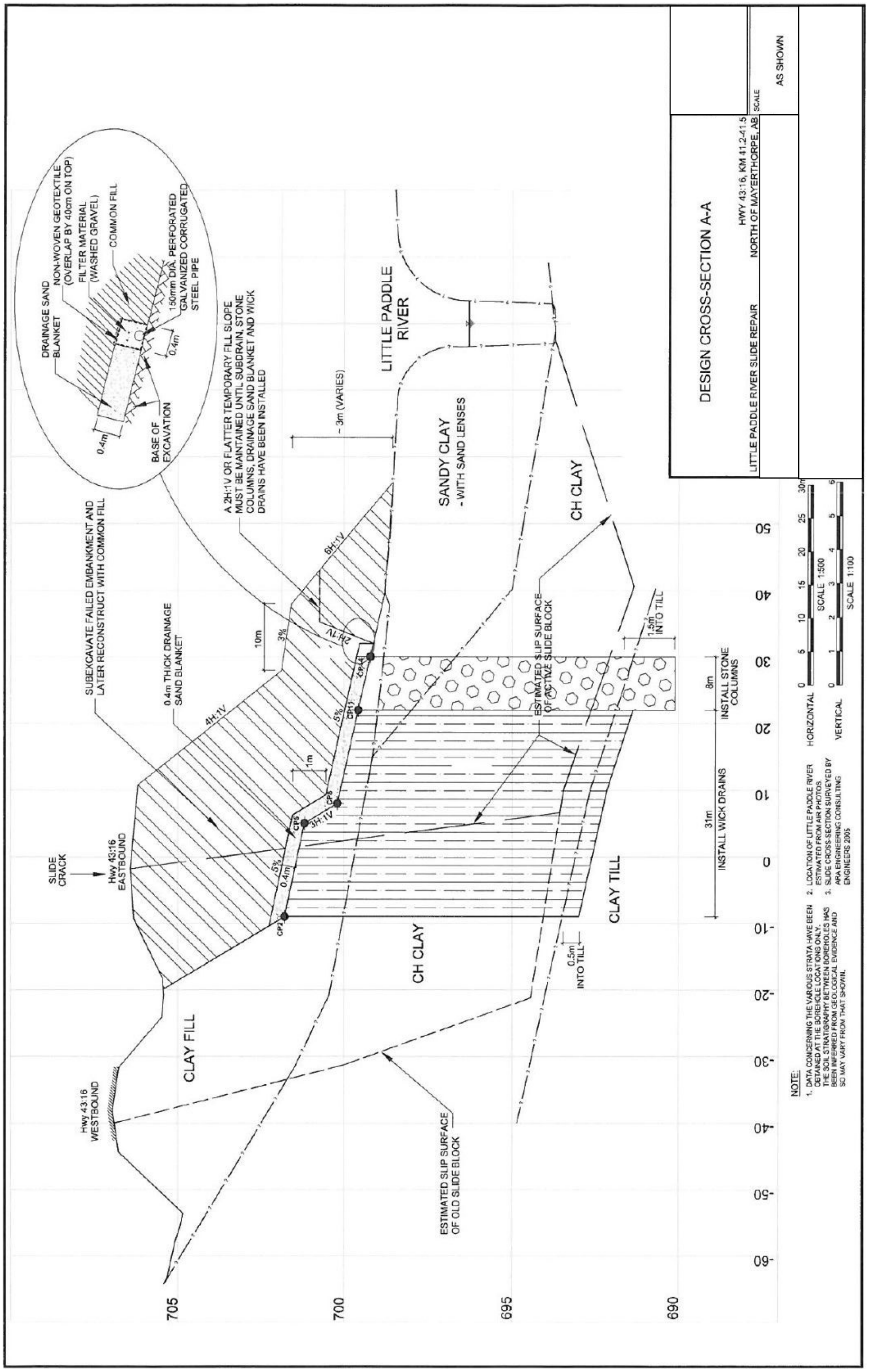


Figure A-20: Cross section of the repairs at the Little Paddle River site (Thurber Engineering Ltd., 2006). Reproduced with permission from Alberta Transportation.

Plan Map

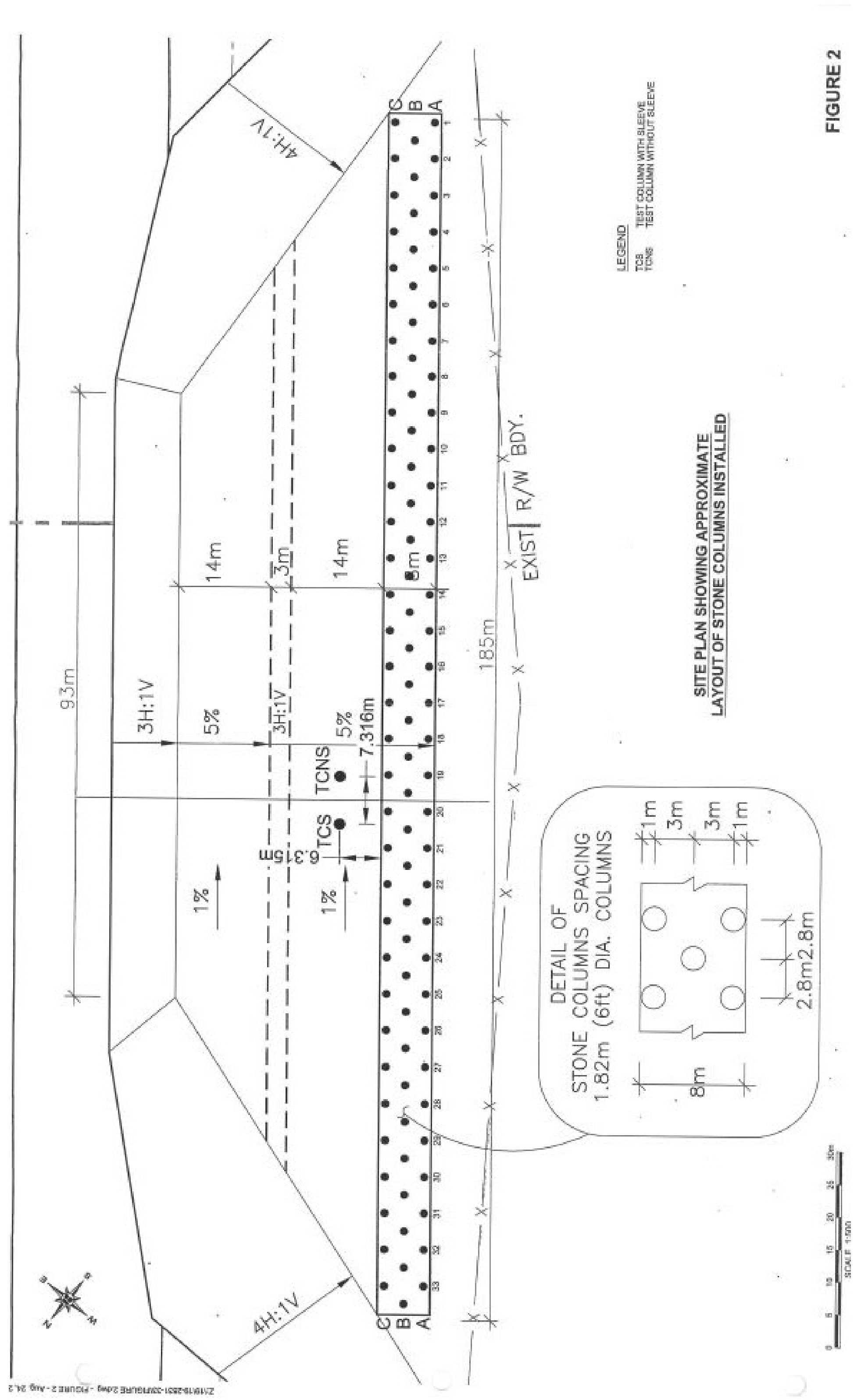


Figure A-21: Plan map of the repairs at the Little Paddle River site (Thurber Engineering Ltd., 2006). Reproduced with permission from Alberta Transportation.

AT9 – *Little Smoky River*

Hwy 49:12, beneath west abutment of bridge crossing Little Smoky River in Alberta

Case Study Summary

Background

The subject of this case study is a slide along the Little Smoky River. It is one of several slides which are collectively known as the Little Smoky Slides. The Little Smoky Slides have been the subject of several studies (Morgan et al., 2014; Mansour, 2009; Mansour et al., 2009; Mansour et al., 2008; Skirrow et al., 2005; Thomson & Hayley, 1975). Highway 49 runs through this area and crosses the Little Smoky River via a bridge that was completed in 1957. The river valley is 90 m deep and the average slope is 7° (Mansour et al., 2009). Soon after the bridge was completed, movements affecting the southwest abutment and pier were observed. These movements have continued since then. Mansour et al. (2009) states the rate of movement has been recorded as high as 100 mm/year. To accommodate these movements, "support on rollers" was adopted for the southwest abutment, but the bridge deck has still had to be extended several times (Karl Engineering Consultants Ltd., 2008). While slides have also been observed in the north and south slopes (Mansour et al., 2009), the focus of this case study is on this slide affecting the southwest abutment and pier of the bridge.

The slide mechanism has been assessed to be a series of translational blocks with retrogressive movement being triggered by erosion of the river banks (Karl Engineering Consultants Ltd., 2008). The site was noted as being located along an outside bend of the Little Smoky River (Alberta Transportation and Amec Foster Wheeler, 2015). The main slip surface was identified at approximately the same elevation as the river bed (Karl Engineering Consultants Ltd., 2008). Site cross sections indicate the slope is comprised of about 2.5 m of sand/silt/till at surface, underlain by re-worked clay shale bedrock to an elevation of 481.5 masl just upslope of the upper bridge pier, down to 480 masl downslope of the lower bridge pier (by the river). A thin (0.5 to 2.0 m) layer of gravel was identified below the reworked clay shale, thickening toward the toe of the slide. Intact bedrock underlies this gravel layer (Amec Foster Wheeler, 2016).

Surface grading was pursued to drain low areas of ponding water, likely produced from graben troughs. However, precipitation was not believed to be playing a major role in triggering movements (Karl Engineering Consultants Ltd., 2008). From 2007 until the end of 2008, pore pressures and movements were monitored with a series of piezometers and slope inclinometers. Periods of high groundwater levels were observed in response to accelerate movements of the slide system. At the time of the 2008 annual inspection, the site was assessed an Alberta Transportation risk score of 88 (Karl Engineering Consultants Ltd., 2008). By the 2015 annual inspection, this score had risen to 96. Remedial options including stabilization of the localized slumping near the toe, controlling lateral river migration, and improving flood resiliency along the toe and river bank were being considered at that time (Alberta Transportation and Amec Foster Wheeler, 2015).

Remediation

The toe of the slide below the southwest abutment was remediated in January 2017 by installing stone columns. The decision to adopt this technique was based on a strong correlation between pore pressure responses measured in the slope in response to river level changes and residual creep in the reworked clay shale. The columns are expected to relieve the pore pressures while also increasing the shear resistance along the slip surface.

The columns were installed in four rows, spanning a length of approximately 65 m. Each column was 2.44 m in diameter, installed with a center-to-center spacing of 2.5 m across the slope and 3.1 m in the upslope direction. The columns were installed to an average depth of about 5.6 m below the original grade, and were backfilled with 3.5-4.5 m of granular followed by approximately 1.5 m of gravel fill and a 0.6 m clay cap. The granular backfill was vibro-compacted in two lifts using an H-Pile. The slip surface was intercepted at an approximate depth of 5.2 m. The columns were keyed into the layer of gravel identified at depth. A subsurface drain was installed at the upslope heel of the gravel fill overlying the columns, immediately above the uppermost row. Toe armouring measures were also installed to address the previously identified trigger, and consisted for the most part of heavy rock riprap enclosed by geotextile.

Performance of Repairs

It is currently unknown how well the repairs are performing due to the very recent installation.

Case Study Details

Landslide Information

Landslide type:	Compound slide	*Pre-support rate (mm/yr):*	100
Trigger:	Construction	*Landslide velocity class:*	2 - Very Slow
Depth of movement (m):	5.2	*Most recent rate (mm/yr):*	
Sheared material:	Re-worked clay shale		

Landslide Dimensions

Width (m):	65	*Length (m):*	
Height (m):	12.5	*Slope (°):*	7-18 (at toe)

Monitoring Information

Movement first reported:	1957	*Last inspected:*	January 18, 2017
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	2.5	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	No	*Surface water type:*	River

Repair Specifications

Repair type:	Columns	*Repair date:*	January 18, 2017
Column diam. (m):	2.1	*Rows:*	4
Length (m):	65	*Drainage (yes/no):*	Yes
Layout:	Triangle	*Spacing (up x across [m]):*	3.1 x 2.5
Granular height (m):	4.8	*Backfill:*	Modified Des. 2 Class 25 granular
Overburden (m):	0.6		
Overburden material(s):	Clay cap		

Additional Site Information

Cross Section

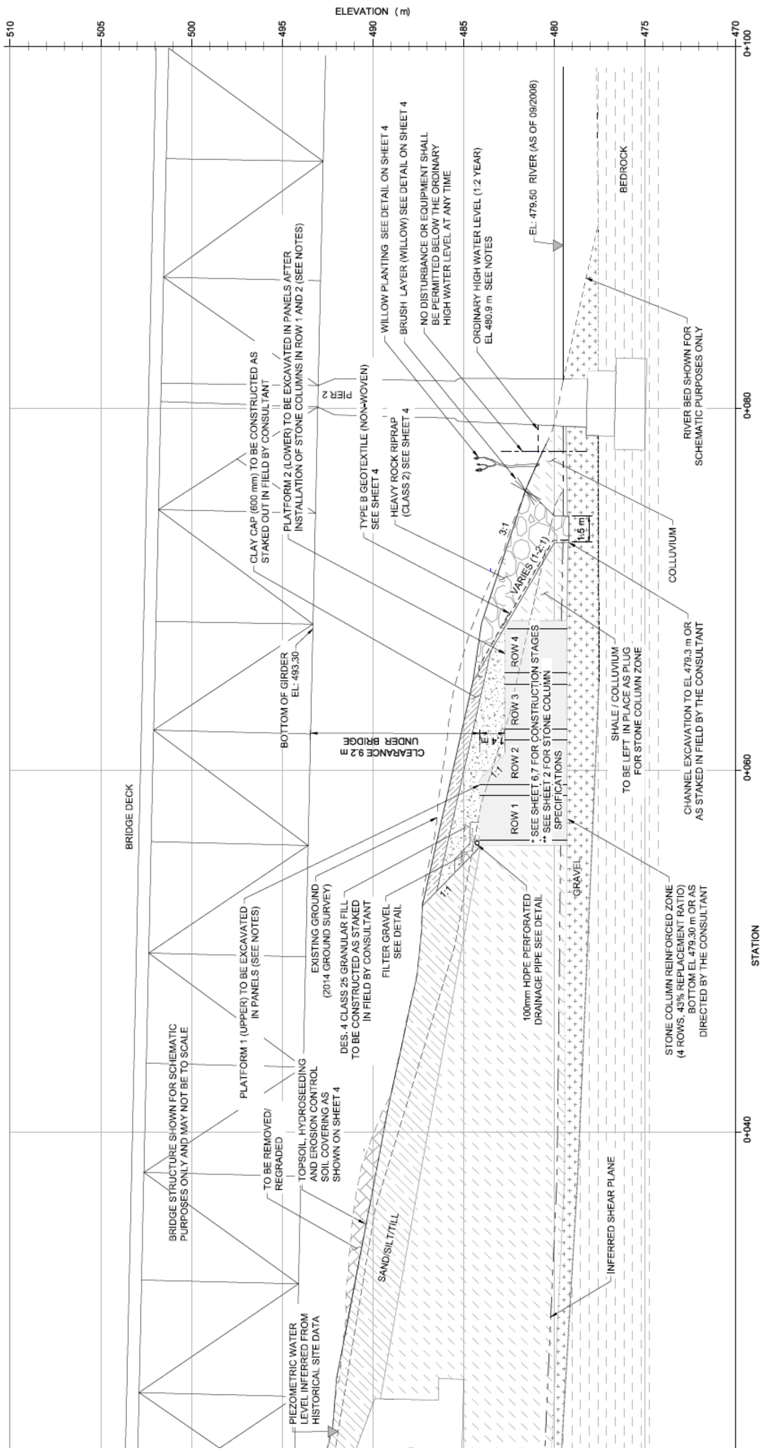


Figure A-22: Cross section of the stone column repair for AT9. Reproduced from Amec Foster Wheeler (2016) with permission from Alberta Transportation.

Plan Map

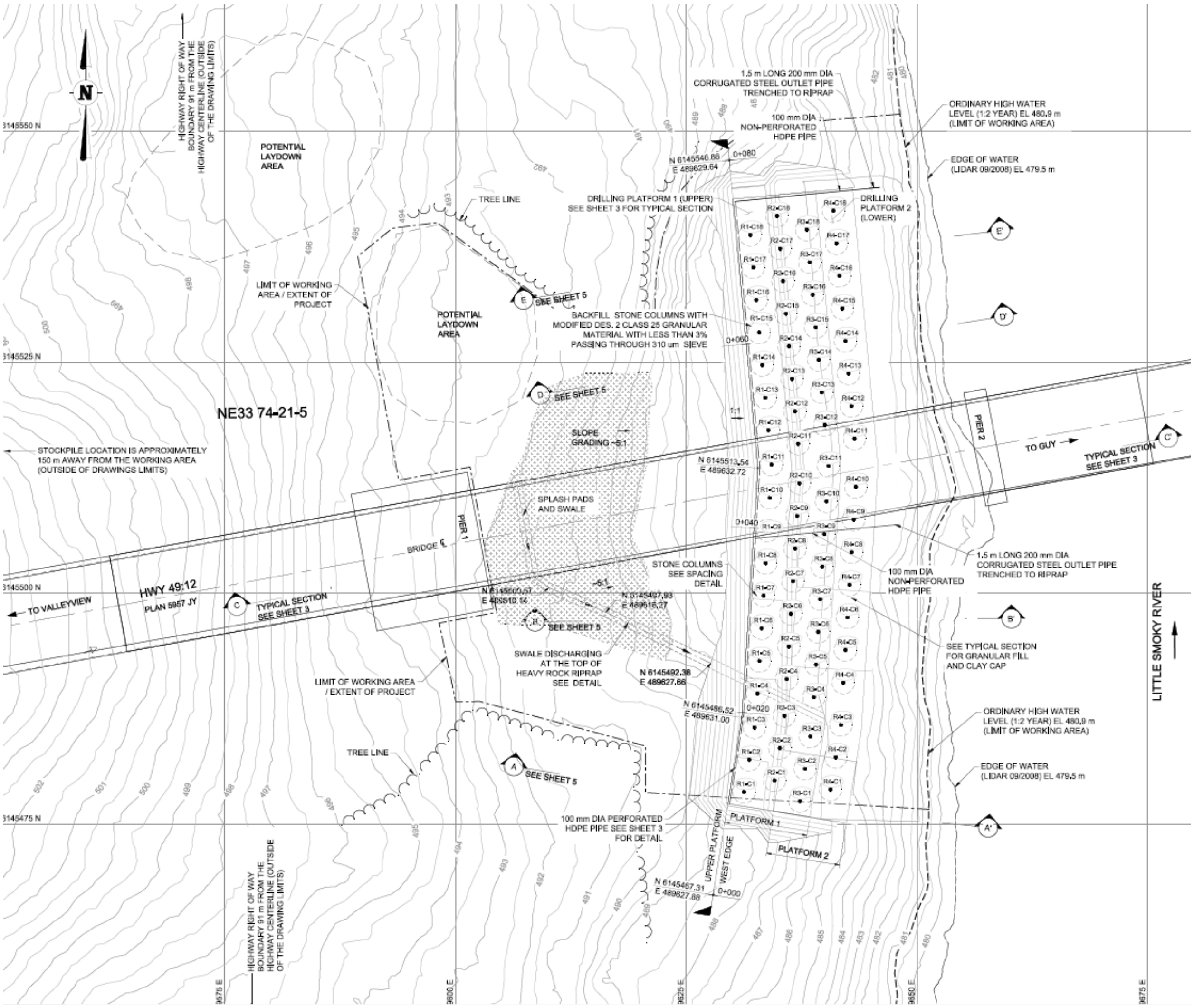


Figure A-23: Plan map of AT9 stone column repair layout. Reproduced from Amec Foster Wheeler (2016) with permission from Alberta Transportation.

AT10 – *S7: West of Millarville*

Hwy 549:02, 1 km west of SH762 Junction, Alberta

Case Study Summary

Background

This site is located along Highway 549:02, approximately 1.5 km west of the SH762 junction. The highway here runs east-west across a hill sloping down to the north, and is founded on a 5-6 m high embankment. The embankment was constructed as a transition from the cut section west of the failure zone to the lower areas to the east (Amec Foster Wheeler, 2005).

A rotational earth slide in the north slope of the embankment was reported in the spring of 1987. The slide was contained between Sta. 3+780 and 3+820 and resulted in the upper half of the slope face dropping downward. The fill comprising the lower half of the slope was observed bulging outwards. Sinkholes and depressions were also observed above a culvert located at Sta. 3+816, and upslope of the horizontal drain outlets located between Sta. 3+755 and 3+780 (Amec Foster Wheeler, 2005).

The failure was attributed to several factors. First, the embankment had been saturated and the GWT had been elevated from slope runoff coming from the south. Discharge coming from the horizontal drains also contributed to the raised GWT. Additional contributing factors included a weak organic layer on the natural slope face under the embankment and the poor weathering characteristics of the sandy shale rock fill that had been used for the embankment (Amec Foster Wheeler, 2005).

A series of boreholes drilled in March 2001 showed there was clay fill in the road embankment and it was underlain by high plastic clay, likely of glaciolacustrine origin. High plastic clay till was found below the fill. Organics 0.5 m thick were also identified at the base of the road fill in one of the boreholes (Amec Foster Wheeler, 2005).

Remediation

Initial remediation works were undertaken between August and November 1988 and consisted of excavating the slide mass and underlying weak organics to expose a competent base. This base was benched and a 0.3 m thick gravel drainage blanket was laid out. The embankment was then reconstructed using suitable material. To address the surface water issues previously identified as a trigger for the slide, the surface drainage was improved adjacent to the north toe of the embankment and the outlets of the horizontal drains were extended. A subsurface drain was also installed beneath the upslope ditch to intercept groundwater and transfer it safely to the downslope side. The road was repaved in October 2000 to repair cracks and dips that had appeared east of the original slide. The road was repaved again in the fall of 2001 and July 2003 (Amec Foster Wheeler, 2005).

Designs for remedial measures were submitted per a report dated June 10, 2002. The remedial measures include two shear keys and slope flattening (Amec Foster Wheeler, 2002). A report from May 25, 2004 indicated that tendering for these repairs was anticipated for late 2004 (Amec Foster Wheeler, 2004). This was confirmed in a report dated June 27, 2005, which reported that the shear keys and slope flattening repairs had been completed in the fall of 2004. The shear keys are approximately located between Sta. 5+400 and 5+460 and their construction included outlets for a weeping tile drain. The slope flattening took place roughly near Sta. 5+320 (Amec Foster Wheeler, 2005). Repaving occurred again following the June 2005 inspection (Amec Foster Wheeler, 2006), and after a May 2006 inspection (Amec Foster Wheeler, 2007).

The regular maintenance work reported after the installation of the shear keys is typical, with movement anticipated for 3-5 years following construction in most cases.

Performance of Repairs

Cracks and dips appeared in the road east of the original slide, so the road was repaved in October 2000. An inspection performed on May 28, 2001 revealed that crescent-shaped cracks had become evident since then, indicating the zone of instability was still active. This was confirmed by a pair of slope inclinometers installed in March 2001 (Amec Foster Wheeler, 2001). BH-1, located in the downslope, recorded 100 mm of displacement between March 2001 and October 2001 at a depth of 5.7 m before shearing sometime between October 2001 and May 2002. BH-2, located in the downslope lane of the road, recorded 12 mm of movement between March and July 2001 along a shear plane 3 m deep (Amec Foster Wheeler, 2005). A dip area west of this main zone of instability had been repaved as well but no distress was evident. The authors of the report noted at that time that further maintenance/repairs were anticipated until proper remediation was pursued (Amec Foster Wheeler, 2001).

A report from May 29, 2002 indicated ongoing movement was detected but that designs for remedial measures had been submitted for shear keys and slope flattening. Instability between Sta. 5+420 and 5+460 was observed, with the report describing cracking west of this main instability zone. The previous report had referred to this as a dip area. The cracking had occurred since the repaving in fall 2001. West of the main zone of instability, in the dip area, a significant crack was observed around Sta. 5+260. Since only a dip had been present in previous visits, the report offers the possibility that the zone was accelerating. The crack could be traced down the north slope, near a culvert. A probability factor of 13 and consequence factor of 5 were assigned, for a total risk rating of 65 in the Alberta Transportation risk rating system (Amec Foster Wheeler, 2002).

Per a report dated May 25, 2004, recent cracking and vertical settlement in the westbound lane (downslope side) and a portion of the eastbound lane were observed between Sta. 5+420 and 5+460. This cracking and settlement was new as of the overlay in July 2003. Recent cracking was also observed to the west, near Sta. 5+320. A probability factor of 13 and consequence factor of 5 were assigned, maintaining a total risk rating of 65 (Amec Foster Wheeler, 2004).

Repairs consisting of the construction of two shear keys and slope flattening were completed in the fall of 2004. An inspection from June 27, 2005 reported that no cracking or settlement was observed. However, cracking and settlement was observed near Sta. 5+320 (west of the shear key). The aperture and downdrop of these cracks measured 25-50 mm. It was noted as being the same cracking reported in 2004 but the magnitude had increased. This was adjacent to the area where slope flattening had been performed to increase stability. A hump and heave were observed in the eastbound lane, east of this cracking and closer to the shear keys. This was noted to possibly indicate movement was occurring towards the northeast and/or the south. It was also reported that the outlets of the weeping tile drains installed at the base of the shear keys could not be located. The outlets were supposed to daylight near the toe of the slope west of the 1200 mm culvert. The report suggested they were likely buried by soil washed down the slope during heavy rains in June 2005. For west of the shear keys (Sta. 5+320), a probability factor of 10 and a consequence factor of 3 were assigned giving a total risk rating of 30 (Amec Foster Wheeler, 2005).

As of an inspection report dated May 30, 2006, there were still no signs of cracking or settlement between Sta. 5+400 and 5+460, where the shear keys are located. Minor cracking and settlement west of the shear keys (Sta. 5+320) was evident despite repaving after the June 2005 inspection.

The cracking was not as significant as had been observed in June 2005 though, potentially indicating movement was slowing. A probability factor of 8 and a consequence factor of 3 were assigned, for a total risk rating of 24 (Amec Foster Wheeler, 2006).

An inspection on June 18, 2007 reported a well-defined diagonal crack across the entire width of the highway around Sta. 5+280 and 5+290. East of this crack, a downdrop of 30-50 mm was measured relative to the west side. The report stated the orientation and degree of downdrop suggested it formed the west or southwest flank of a landslide crossing the highway. Similar cracking had been mentioned in the 2005/2006 inspections but the road had been repaved since then, so this was new. A single crack was also found running across the shear keys but it was not accompanied by any vertical displacement. As of the 2007 report, the outlets for the shear keys had still not been located. Horizontal drains were suggested as a possible solution but the risk of penetrating the geotextile lining the base of the shear keys was acknowledged. A probability factor of 10 and a consequence factor of 3 were assigned, for a total risk rating of 30 (Amec Foster Wheeler, 2007).

The final report that was available was dated June 19, 2008. Cracking west of the shear key was reported once again but the authors noted the magnitude appeared to have reduced significantly since 2007. At this point in time, the shear keys were evaluated as having been effective and that minor cracking would continue to be repaired. A probability factor of 6 and a consequence factor of 1 were assigned, giving a total risk rating of 6; a significant decrease from previous years. Annual site assessments were discontinued from thereon (Amec Foster Wheeler, 2008).

In considering the report details summarize above, the shear keys appear to have been successful. It is not certain whether the subdrain outlets were ever located again but this does not seem to have had a significant impact on the performance of the shear keys. The magnitude of cracking was reported to decrease over a span of approximately 4 years, which is within the typical timeframe of 3-5 years for movement to stabilize following granular shear key construction.

Lessons Learned

Movements continued for approximately 4 years after the granular shear keys were constructed. The weeping tile outlets were not reported to have been located again after construction, and may remain buried. Despite this issue, the shear keys were declared successful.

Case Study Details

Landslide Information

Landslide type:	Planar slide	*Pre-support rate (mm/yr):*	
Trigger:	Saturation & Weak layer	*Landslide velocity class:*	2 - Very Slow
Depth of movement (m):	3.0-5.7	*Most recent rate (mm/yr):*	None near key
Sheared material:	Sandy shale rock fill, organics, CH clay		

Landslide Dimensions

Width (m):	40-60	*Length (m):*	
Height (m):	5-6	*Slope (°):*	~12

Monitoring Information

Movement first reported:	Spring of 1987	*Last inspected:*	June 19, 2008
SIs installed:	2	*SIs active last inspection:*	0
Piezos installed:	2	*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	Near surface	*Surface water at toe (yes/no):*	No
Seepage detected (yes/no):	Yes	*Surface water type:*	

Repair Specifications (Upper ; Lower)

Repair type:	Shear key	*Repair date:*	Nov '88 & Fall '04
Base width (m):	U: 3 ; L: 3	*Trench slope ratio (H:V):*	1.5 : 1.0
Length (m):	U: 45 ; L: 65	*Drainage (yes/no):*	Yes
Granular height (m):	U: 6 ; L: 4	*Backfill:*	95% SPMDD
Overburden (m):	U: 0.6 ; L: 1.5		
Overburden material(s):	Native clay compacted to 95% SPMDD		

Additional Site Information

Cross Section

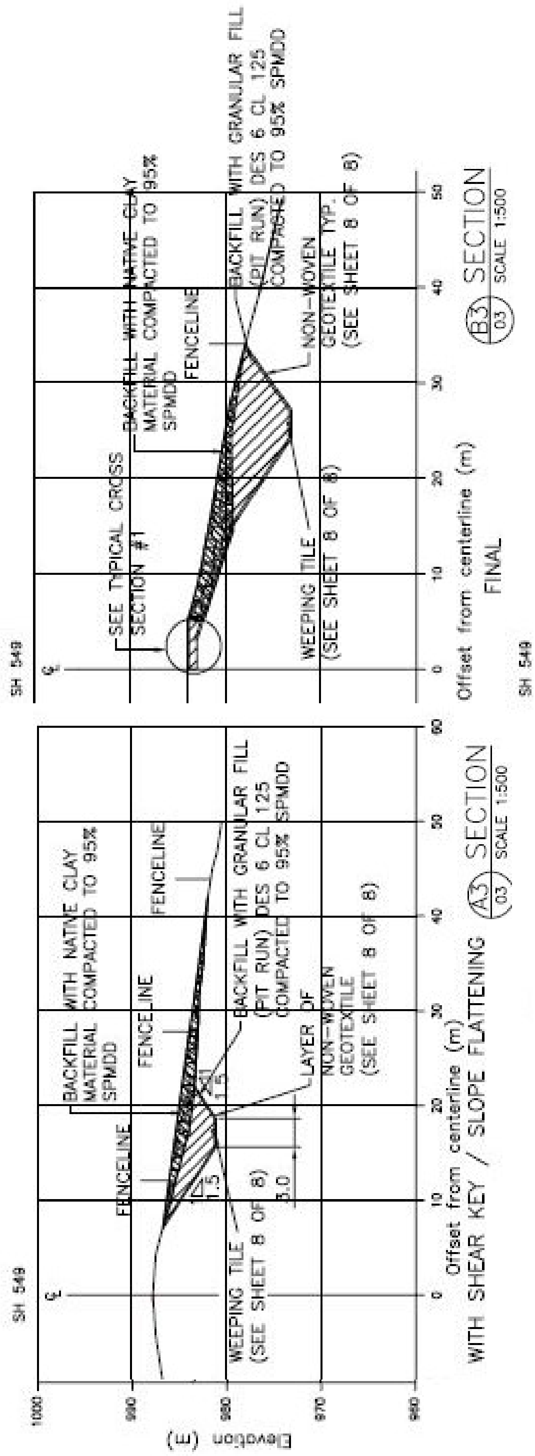


Figure A-24: Cross sections A3 and B3 (see Plan Map) for the West of Millarville site (Amec Foster Wheeler, 2002). Reproduced with permission from Alberta Transportation.

Plan Map

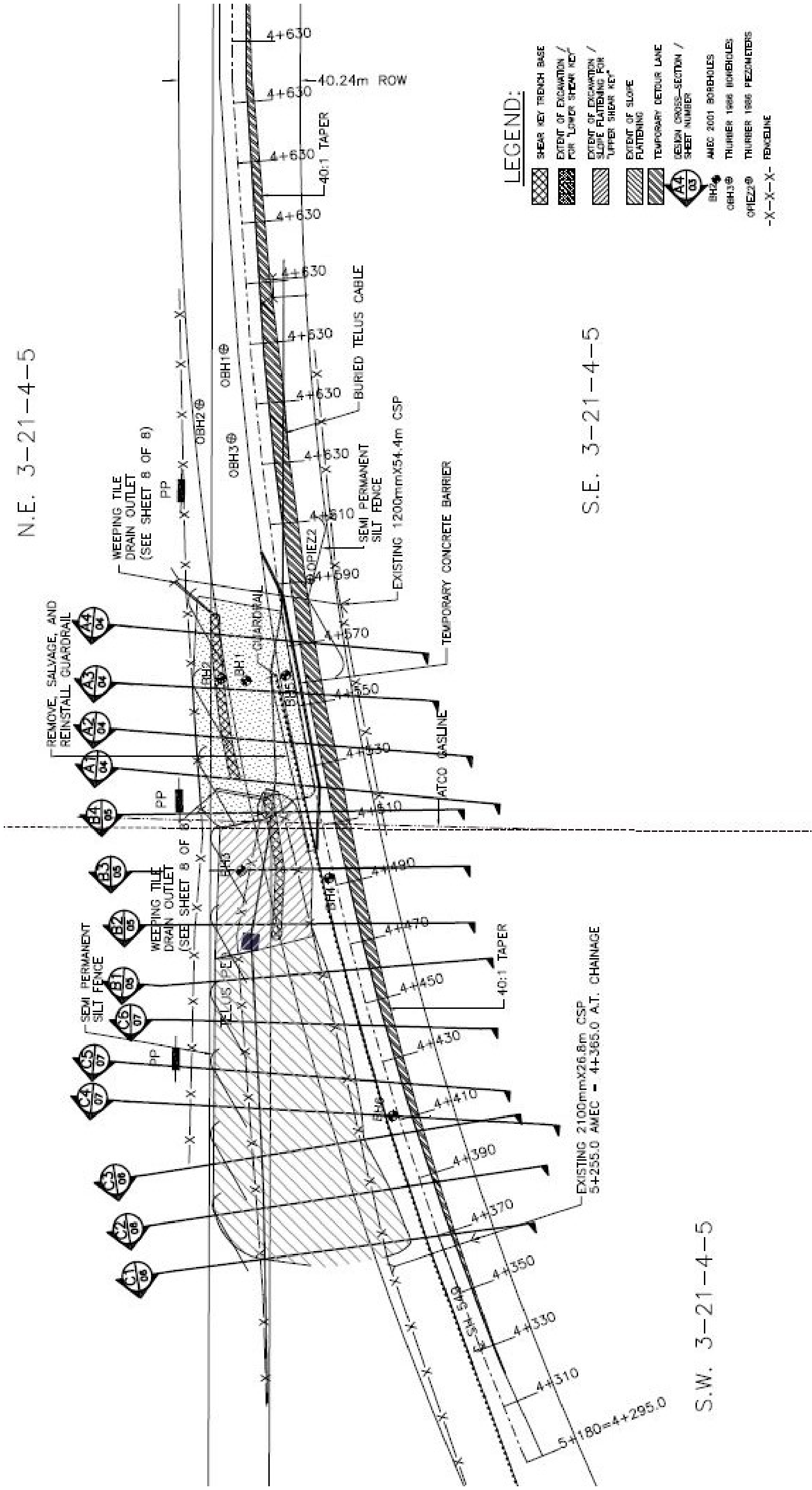


Figure A-25: Plan map of the repairs and highway at the West of Millarville site (Amec Foster Wheeler, 2002). Reproduced with permission from Alberta Transportation.

AT11 – *GP10 South of Sturgeon Lake*

Hwy 43:06, 25 km west of Valleyview, Alberta

Case Study Summary

Background

The highway at this site is located on a fill embankment overlying soft clay deposits (Tetra Tech EBA, 2004). The first available report for this site, from June 11, 2001, notes that the slope was remediated in May 2001. The remediation involved reconstructing the slope with granular material and installing a granular toe key (Tetra Tech EBA, 2001). The toe key was reported to have included a geotextile underlay (Karl Engineering Consultants Ltd., 2007). The embankment height and slope are uncertain, with one report stating the embankment was 1-2 m high, with a slope of 5H:1V (Tetra Tech EBA, 2003) and another report citing a height of 2-3 m and a slope of 4H:1V (Tetra Tech EBA, 2005).

Site investigation revealed a high groundwater table, which was said to be accounted for in the design of the remedial works. The source of the groundwater was later believed to be a local spring, after observing seepage coming from the toe key. Following reconstruction, it was decided that long-term performance would be monitored and assessed through detailed recordings of road patching events and any observations of distress (Tetra Tech EBA, 2001).

Remediation

In May 2001, the slope was reconstructed with granular material and a granular toe key was installed (Tetra Tech EBA, 2001). Seepage was later observed coming from the toe key and was addressed via the installation of a weeping tile drainage design implemented in early Fall 2001 (Tetra Tech EBA, 2002).

Performance of Repairs

Following the repairs in May 2001, groundwater was observed seeping from the west edge of the toe key and slope (Tetra Tech EBA, 2001). In early Fall 2001, a weeping tile drainage design was implemented. No further deterioration was observed for the cracks already present during a June 2002 inspection (Tetra Tech EBA, 2002).

A June 24, 2003 inspection revealed that a new crack had appeared along the shoulder of the road, accompanied by a drop of 25 mm. This crack was determined to have developed sometime after the June 2002 inspection. The granular toe key being saturated was given as a possible cause since seepage had been identified previously (Tetra Tech EBA, 2003).

A May 3, 2005 report noted that cracking and settlement that had been observed since 2002 appeared to have stabilized and possibly stopped over the previous year. Groundwater flow from the weeping tile was stated to have been continuous for the past 4-5 years. It was recommended at that time to continue with visual monitoring over the next 1-2 years (Tetra Tech EBA, 2005).

The minor settlement and pavement cracking was observed over a period of 3-4 years after the toe key was installed in 2001. Thus, yearly maintenance was required. A report from June 25, 2007, noted that a reflective crack that could be seen in the pavement had not manifested for the previous 2-3 years (Karl Engineering Consultants Ltd., 2007). The weeping tile was still discharging water as of June 19, 2006 (Karl Engineering Consultants Ltd., 2006). Substantial decreases in slide movement and pavement settlement were noted for the period spanning 2004-2007. At the time of the June 2007 report, it was said that the site appeared to have stabilized sometime between 4-6 years after the installation of the toe key (Karl Engineering Consultants Ltd., 2007).

Lessons Learned

This slide was originally repaired without appropriate drainage measures. Once seepage was observed, a weeping tile was installed but movement and settlement persisted for the next 3-4 years. It was concluded that a possible reason for this was the bathtub effect. The bathtub effect is the result of a free-draining backfill becoming saturated and subsequently creating a zone of deeper wetting (Karl Engineering Consultants Ltd., 2007). This zone of deeper wetting would have been located within the soft clay deposits that had been identified. The bathtub effect is known to typically result in some amount of deformation. In this case, the soft clay deposits may have softened because of the exposure to the deeper wetting zone.

Figure A-26: Continuous flow coming from the outfall pipe draining the granular toe key (Tetra Tech EBA, 2002). Reproduced with permission from Alberta Transportation.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:	Seepage	*Landslide velocity class:*	
Depth of movement (m):		*Most recent rate (mm/yr):*	
Sheared material:	Soft clay		

Landslide Dimensions

Width (m):		*Length (m):*	
Height (m):		*Slope (°):*	

Monitoring Information

Movement first reported:	Before 2001	*Last inspected:*	June 25, 2007
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	High	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	Yes	*Surface water type:*	N/A

Repair Specifications

Repair type:	Shear key	*Repair date:*	May 2001
Base width (m):		*Trench slope ratio (H:V):*	
Length (m):		*Drainage (yes/no):*	Yes
Granular height (m):		*Backfill:*	
Overburden (m):			
Overburden material(s):			

Additional Site Information

Plan map

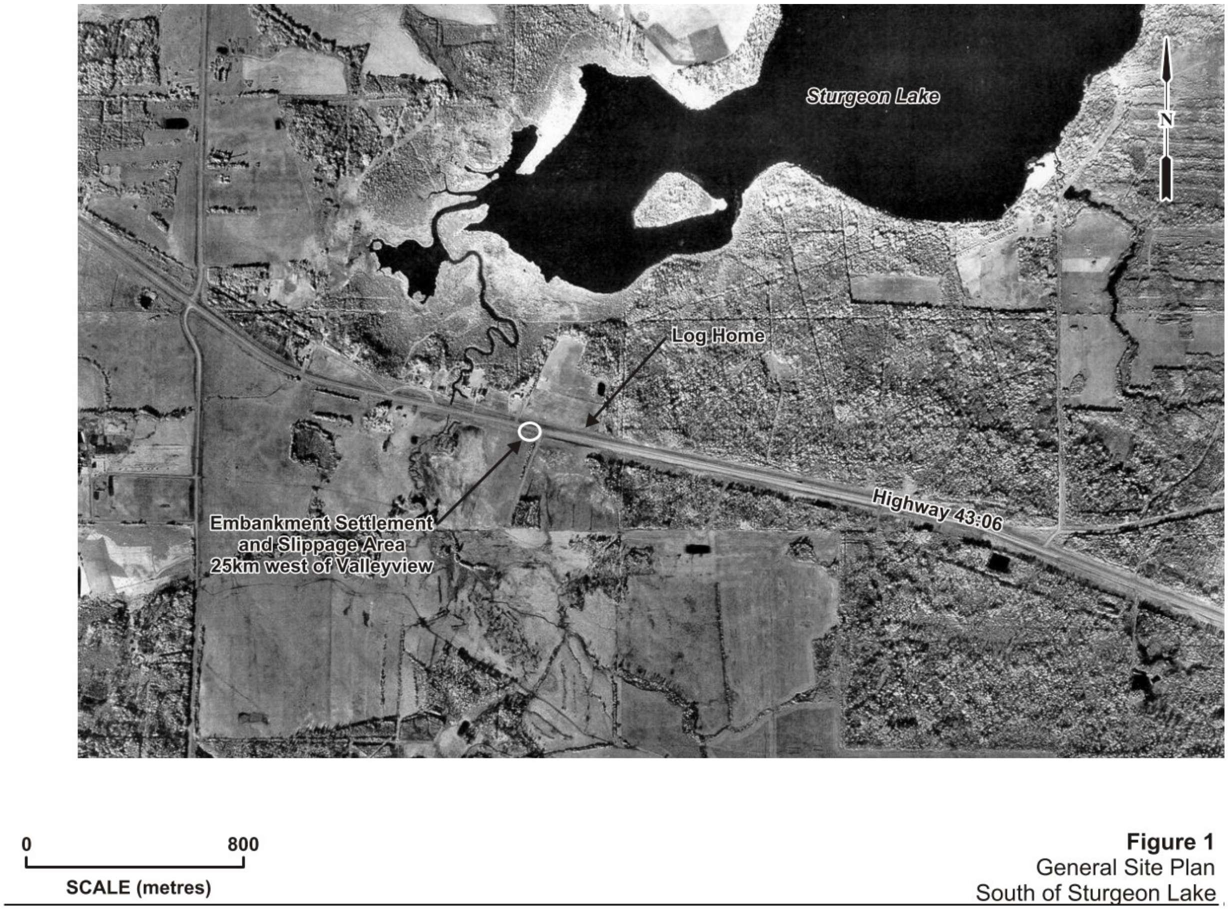


Figure A-27: Aerial photograph showing the South of Sturgeon Lake site (Tetra Tech EBA, 2002). Reproduced with permission from Alberta Transportation.

AT12 – *PH4 West of Fairview*

Hwy 682:02, Station 10+900; Tributary valley of Hines Creek in Alberta

Case Study Summary

Background

The road constructed at Station 10+900 of the PH4 West of Fairview site sits on an embankment 12 m high inclined at 4H:1V. The slope also contains an intermediate bench that is 15 m wide in cross section. Sliding was found to have initiated in 1984 after cracks were observed in the pavement. The slide has been measured to be about 200 m wide (Alberta Transportation, 2003).

The soil was classified as predominantly medium to highly plastic, lacustrine clay after a borehole investigation in 1985. Slickensides were also observed. Movement was identified at a depth of about 15-20 m, based in the native clay strata. Movement is believed to have been triggered by a combination of weak in-situ soils, the placement of fill for the highway, a high groundwater table, and seepage into the slide coming from a low spot in the ditch located at the slide scarp (Thurber Engineering Ltd., 2004).

Remediation

In 1984/1985, a Big O subdrain was installed in the upslope ditch and an initial flow of 2 gpm was recorded. Additional remediation measures pursued at that time were the injection of lime/flyash into boreholes on a 3-m grid and the installation of five horizontal drains. The horizontal drains were all installed from a single focal point located on a bench on the north side of the highway (Thurber Engineering Ltd., 2004). In 2000, a French drain for the upslope ditch and a new culvert were installed at the site (Alberta Transportation, 2001).

Designs for horizontal drains and a shear key were completed in 2000-01 (Alberta Transportation, 2002). In a report dated July 4, 2001, it was recommended that these remedial measures be implemented (Alberta Transportation, 2001). However, boreholes drilled during a geotechnical investigation revealed there were few high permeability target zones for the drains rendering this solution ineffective. Stability analysis indicated that the shear key would have to be deep and wide and would cost approximately $1 000 000. Engineers acknowledged the potential for additional costs due to the risky nature of excavating near the toe of a slide (Alberta Transportation, 2002).

Due to the high cost, risk and/or ineffectiveness of the two solutions presented above, alternatives were considered. In 2002, reverting the road to gravel was considered due to minimal amounts of movement exhibited since 1985. It is understood that a gravel road would handle deformations better than a paved road and would be less costly to repair/regrade (Alberta Transportation, 2002).

In a November 4, 2004 report, it was noted that a toe berm did not appear feasible because of the geometry of the existing structures and the presence of a creek at the toe of the slope. The option to re-align the road upslope of its current route was considered too. The road would either need to be re-aligned 30 m upslope to avoid the slide, or short sections still affected by the slide would need to be supported with a shallow pile wall. Recommendations going forward were to install more slope inclinometers, additional surveys and a slope stability analysis to assess the proposed solutions (Thurber Engineering Ltd., 2004).

Performance of Repairs

In 1985, five slope inclinometers and three piezometers were installed. A report from 2004 indicated these had all sheared off. An evaluation in 1999 found cracking and settlement in the road. The investigators also identified a low area that had formed in the ditch. Water was noted to be ponding at the location of the slide because of the low area in the ditch (Thurber Engineering Ltd., 2004). In 2000, the scarp uphill of the road was visible and that side shears could be seen running through the road, per a report dated May 9, 2002 (Alberta Transportation, 2002).

The toe was said to be observable during visits in 2001 and 2002, while the scarp was not as it had been obscured by work that had been undertaken. Movement at the time (May 9, 2002) was stated to have been relatively small, measuring less than 0.5 m (Alberta Transportation, 2002). By June 18, 2003, the scarp was visible once again (Alberta Transportation, 2003).

In July 2004, new or enlarged scarps, cracks and heaved or rutted areas were all identified. Two dips were spotted on the west side of the slide, and a large dip was spotted on the east side of the slide. Crews found cracks bounding the landslide; just beyond the dipping areas on each side of the slide, cracks were found crossing the highway, and a third crack was identified running along the south ditch. Minor cracks were identified east of the scarp too. At that time, it was said the slide was still moving and possibly spreading north (Thurber Engineering Ltd., 2004). At the time of this writing, it is unknown whether a shear key ever ended up being constructed.

Lessons Learned

This site is a good example of a case when a shear key may not have been installed despite being considered for many years. Several factors must be in place to make the adoption of a shear key favourable. In this case, the high cost and potential risk of destabilizing the slope outweighed the potential benefits posed by a shear key. Furthermore, the slide was noted to have moved relatively little.

Case Study Details

Landslide Information

Landslide type:	Rotational/Compound (?)	*Pre-support rate (mm/yr):*	"Moderate"
Trigger: Fill placement, high GWT, and seepage		*Landslide velocity class:*	2 - Very Slow
Depth of movement (m):	15-20	*Most recent rate (mm/yr):*	Steady
Sheared material:	Native CI to CH lacustrine clay (slickensided)		

Landslide Dimensions

Width (m):	200	*Length (m):*	64.1
Height (m):	12	*Slope (°):*	14

Monitoring Information

Movement first reported:	January 1, 1984	*Last inspected:*	November 4, 2004
SIs installed:	5	*SIs active last inspection:*	0
Piezos installed:	3	*Piezos active last inspection:*	0

Groundwater Information

Groundwater level (mbgl):	High	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	Yes	*Surface water type:*	Stream/Creek

Repair Specifications (Designed – Not Constructed)

Repair type:	Drains only	*Repair date:*	January 1, 1985
Base width (m):	Wide	*Trench slope ratio (H:V):*	
Length (m):	~200	*Drainage (yes/no):*	Yes
Granular height (m):	Deep	*Backfill:*	
Overburden (m):			
Overburden material(s):			

Additional Site Information

Stratigraphy

PH4 West of Fairview (Based on Test Hole 3)				
Case study location:	Hwy 682:02, Station 10+900; Tributary valley of Hines Creek			
Depth (m)	**Soil description**	**Type**	**Source**	**Structure**
0.0 - 12.0	Silty clay; slickensided at 6 m; slide plane at 10 m inclined to 30 degrees from vertical (according to cross section; based on scale, it is believed this is a mistake and that it should be from horizontal); contains rupture surface in cross section for upper half of slide		?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
			?	
12.0 - 21.0	Clay till; contains rupture surface in cross section within lower half of slide; rupture surface on cross section annotated as being at 12.0 m and drawn as being in till; using cross section scale places rupture surface at interface between silty clay and clay till		G	
			G	
			G	
			G	
			G	
			G	
			G	
			G	
			G	
21.0 - 24.0	Silty clay; bottom of hole at 24.0 m		?	
			?	
			?	

Legend					
Type		**Source**		**Structure**	
Organics/Topsoil		Fill	F	Slickensided	
Clay		Fluvial	R	None	
Silt		Lacustrine	L		
Sand		Marine	M	**Groundwater (in Type)**	
Till		Glacial	G	Seepage	s
Stone/rock		Assumed	*Italicized*	GWT	
Interbedded		Unknown	?		

Figure A-28: Schematic representation of the stratigraphy at the West of Fairview site.

Cross Section

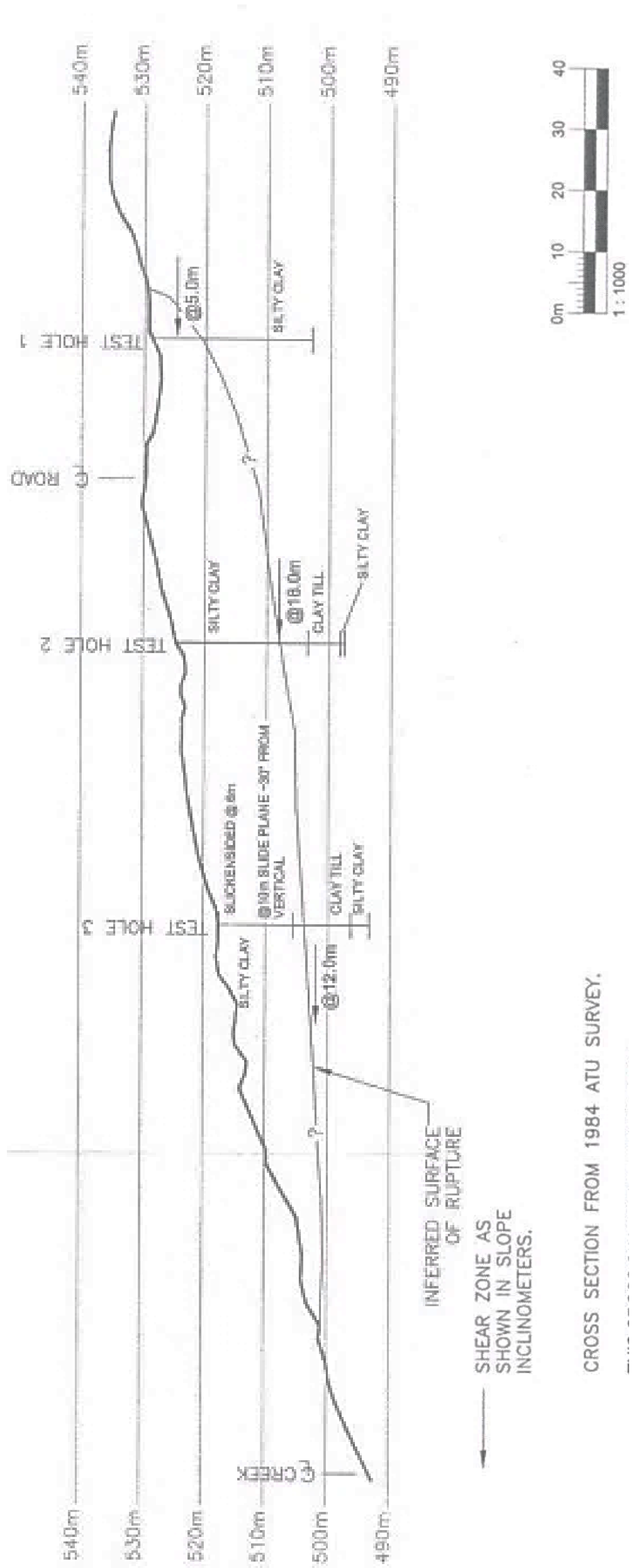


Figure A-29: Cross section showing the inferred slip surface at the West of Fairview site (Thurber Engineering Ltd., 2004). Reproduced with permission from Alberta Transportation.

Plan Map

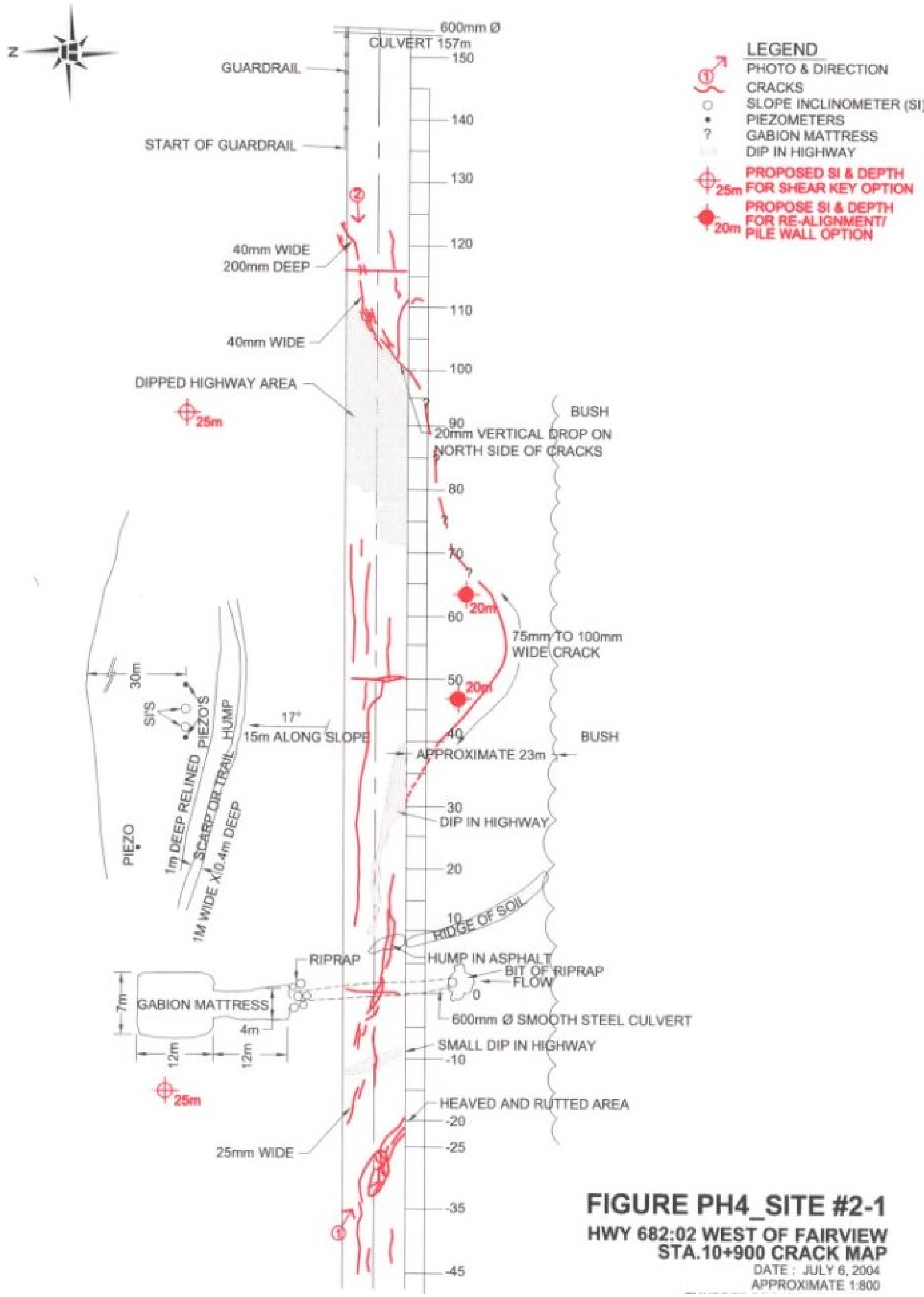


Figure A-30: Plan map of the West of Fairview site showing observed surface features with respect to the highway (Thurber Engineering Ltd., 2004). Reproduced with permission from Alberta Transportation.

CN1 – *Sangudo Bridge M 74.7*

1.1 km from center of Mayerthorpe, Alberta

Case Study Summary

For the following case study, the entirety of the information that is presented was provided by a geotechnical consultant in an unpublished report (2016) prepared for Canadian National Railway.

Background

A fire on April 26, 2016 resulted in the destruction of a rail bridge at Mile 74.7 of the CN Sangudo Subdivision. The bridge crossing spanned a small river from east to west. Emergency reconstruction included the construction of several granular shear keys within the new embankment.

Surficial clays were identified along the base of the valley. They were reported as consisting of alluvial floodplain deposits, interbedded with silts and sands. Along the valley slopes, the clays were reported as being more lacustrine. CPT profiles were used to create a geologic model for the site. The model consisted of a thin layer of fill overlying fluvial sediments, followed by outwash materials. These were all underlain by till and rafted clayshale.

Remediation

Preliminary design for the new embankment consisted of a 7-m width at the rail base and 2H:1V slopes down to ground level. Shear keys along the east and west embankments were to be 5 m wide and 4 m deep. A 5-m wide stabilization berm was also design for the north and south side of the east embankment to bring the overall slope to 4H:1V. These designs were later revised as results from the site investigation and instrumentation were acquired and slope stability modelling was performed. The final design involved shear keys only at the end corners of the west and east embankments.

Performance of Repairs

Monitoring data was only available for the first three weeks after construction was completed. While the data showed the repairs appeared to be performing well, it is difficult to compare this performance with other case histories since a pre-existing landslide was not present at this site.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:	Construction	*Landslide velocity class:*	
Depth of movement (m):	1.8	*Most recent rate (mm/yr):*	
Sheared material:	Lacustrine clays		

Landslide Dimensions

Width (m):	20	*Length (m):*	
Height (m):	9.5	*Slope (^{o}):*	15-27

Monitoring Information

Movement first reported:	April 30, 206	*Last inspected:*	May 24, 2016
SIs installed:	4	*SIs active last inspection:*	4
Piezos installed:	9	*Piezos active last inspection:*	9

Groundwater Information

Groundwater level (mbgl):		*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	No	*Surface water type:*	River

Repair Specifications

Repair type:	Buttress & Shear key	*Repair date:*	May 12, 2016
Base width (m):	5	*Trench slope ratio (H:V):*	2:1
Length (m):	20	*Drainage (yes/no):*	
Granular height (m):	3	*Backfill:*	Pit run
Overburden (m):	1		
Overburden material(s):	Clay		

CP1 – *Bredenbury M 86.75-86.80*

6 km east of the Saskatchewan/Manitoba border, near Russell, Manitoba

Case Study Summary

Background

The Canadian Pacific Railway follows the Assiniboine River valley, gradually climbing up the slope from Millwood, Manitoba to Harrowby, Manitoba. The railway was built in the late 1800s and crosses several ancient landslides. Field investigations determined a 70-m and a 90-m section of the tracks at Mile 86.8 and Mile 86.75, respectively, are currently being affected by shallow ground movement as well as deep-seated movement. The slopes along this section are inclined at 13° overall and run coincident with an outside bend of a meander in the river. The shallow movements were found to be occurring 3-10 m below surface in a portion of the Cretaceous Pierre Shale Formation that has weakened from weathering and exposure to moisture. The deep-seated movement is associated with a translational slump block sliding along a discrete plane in the high plasticity clay shale. These movement types are typical of the Cretaceous Pierre Shale (Yong et al., 2003).

The Cretaceous Pierre Shale Formation is described as a clay shale. Clay shales are characterized as possessing properties belonging to both soils and rock. They typically exhibit strength loss with time too. In North America, the clay shale bedrock sequence is known to include a mix of continental clastics, beds of coal, and bentonite. These clay shales have become known for presenting difficult conditions for engineering (Yong et al., 2003).

Mile 86.8 and 86.75 lie within an area that was judged to be the least stable during an investigation in 1986 and 1987. Instability in this area was attributed to poor surface drainage and river erosion at the toe. Remediation was pursued in 1988 in the form of rip rap along the river bank and swale ditches above and below the track to improve surface drainage (Yong et al., 2003).

It was reported that a major subgrade instability took place in 1995 because of a wet spring and early summer. The ground was measured moving 50 mm per week in early May and accelerating to 350 mm per week by late June and July. Movement was identified downslope of the track at Mile 86.8, while sloughing was detected upslope of the track (Yong et al., 2003).

In October of 1996, a 100 m long granular shear key was constructed immediately downslope of the track at Mile 86.8. In December of 1996, vertical and horizontal movements measuring 50-80 mm were identified along a 45 m stretch of track at Mile 86.75. Slope inclinometers checked in October of 1999 indicated the shear key at Mile 86.8 had been effective, but the tension cracks were migrating to the east. Additional reconnaissance in 2000 discovered substantial movement on the downslope side of the track. A slide scarp measuring 6 m high was reported at Mile 86.75 and a scarp 2.5 m high was reported at Mile 86.8. A bulge in the riverbank was visible as well (Yong et al., 2003).

In early spring 2002, 250 m of track was realigned 5 m upslope and in late May 2002, a second shear key was constructed. The excavation permitted observations to be made in the weathered zone of the shale bedrock. It was described as blocky and possessing orthogonal joint sets. A horizontal iron-stained plane was identified and believed to coincide with the shear plane, but this was not confirmed. The excavation was noted as being dry and the trench walls were found to remain stable without external support for at least two days (Yong et al., 2003).

A thesis written by Salina Yong (2003) documented the geology and history of the site. The sliding mechanisms were investigated and Yong concluded there were both shallow- and deep-seated mechanisms at play. Overall, slide movement was found to be dominated by the lower slope which was succumbing to deep-seated translational movements.

Remediation

Maintenance work since the early 1980's included culvert replacements, track lifts and realignments, and daily track patrols in the spring. The first major remedial works are reported to have been performed in 1988. The work included placing rip rap at the toe of the slide, along the river bank, and improving surface drainage upslope and downslope of the tracks using swale ditches (Yong et al., 2003).

Following the major subgrade instability in 1995, a 100 m long granular shear key was constructed immediately downslope of the track at Mile 86.8 in October of 1996. The purpose of this shear key was to isolate the slope failure downslope of the track from everything upslope of it. Periodic re-grading of the downslope was also undertaken to close open tension cracks, to minimize surface infiltration in the area (Yong et al., 2003).

In response to the instabilities identified in the reconnaissance program performed in 2000, a 250 m stretch of track was temporarily relocated 5 m upslope in early spring of 2002 in preparation for construction of a granular shear key. The surface was also regraded at that time to help control surface runoff and seepage. The granular shear key was constructed in late May 2002, directly under the original track location. The shear key was reported to be 12 m deep at its north end, and the trench walls were inclined at 70 degrees. A benched excavation was adopted for the project (Yong et al., 2003).

Performance of Repairs

In the wake of the shear key construction in 1996, tension cracks with vertical displacements of 600-900 mm formed between 18-60 m downslope of the track. The location of the cracks was later found to be the same as historical cracks and their formation was attributed to prolonged wet conditions in 1996. Slope inclinometer data from October of 1999 suggested the shear key at Mile 86.8 had been effective. However, tension cracks were found to be migrating to the east (Yong et al., 2003).

Lessons Learned

The Harrowby Hills Slides by Yong et al. (2003) clearly outlines the importance of understanding the slide process in the development of landslide remedial measures. Two independent sliding masses were identified, one being shallow- and the other being deep-seated. The shear keys were installed between the two, at the toe of the shallow sliding mass, and at least the first shear key appears to have been effective.

Case Study Details

Landslide Information

Landslide type:	Planar slide	*Pre-support rate (mm/yr):*	520.3
Trigger:	Weathering & Progressive failure	*Landslide velocity class:*	3 - Slow
Depth of movement (m):	3 – 10	*Most recent rate (mm/yr):*	
Sheared material:	Weathered clay shale		

Landslide Dimensions

Width (m):		*Length (m):*	
Height (m):		*Slope (°):*	

Monitoring Information

Movement first reported:	Early 1980s	*Last inspected:*	Late May 2002
SIs installed:	11	*SIs active last inspection:*	
Piezos installed:	33	*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):		*Surface water at toe (yes/no):*	
Seepage detected (yes/no):		*Surface water type:*	

Repair Specifications (Phase I ; Phase II)

Repair type:	Shear key	*Repair date:*	October 1996 and Spring 2002
Base width (m):		*Trench slope ratio (H:V):*	1 : 2.75
Length (m):	I: 100 ; II:	*Drainage (yes/no):*	
Granular height (m):	I: ; II: 12	*Backfill:*	
Overburden (m):			
Overburden material(s):			

Additional Site Information

Stratigraphy

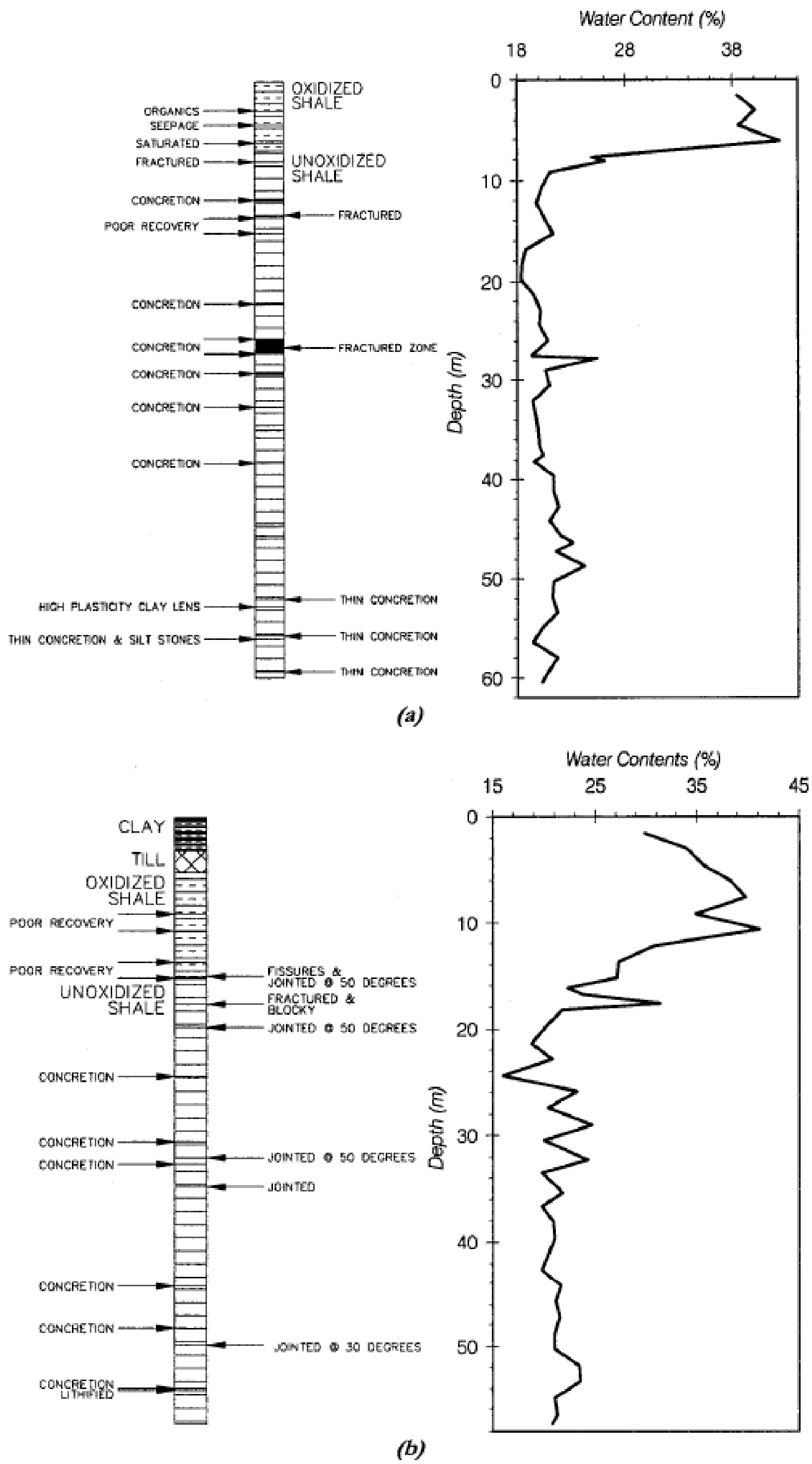


Figure 4-14: *Water content profiles of (a) SI 1108 and (b) SI 1109.*

Figure A-31: Water content profiles and borehole logs for SI 1108 and SI 1109. Reproduced from Yong (2003)*, with permission.*

Cross Section

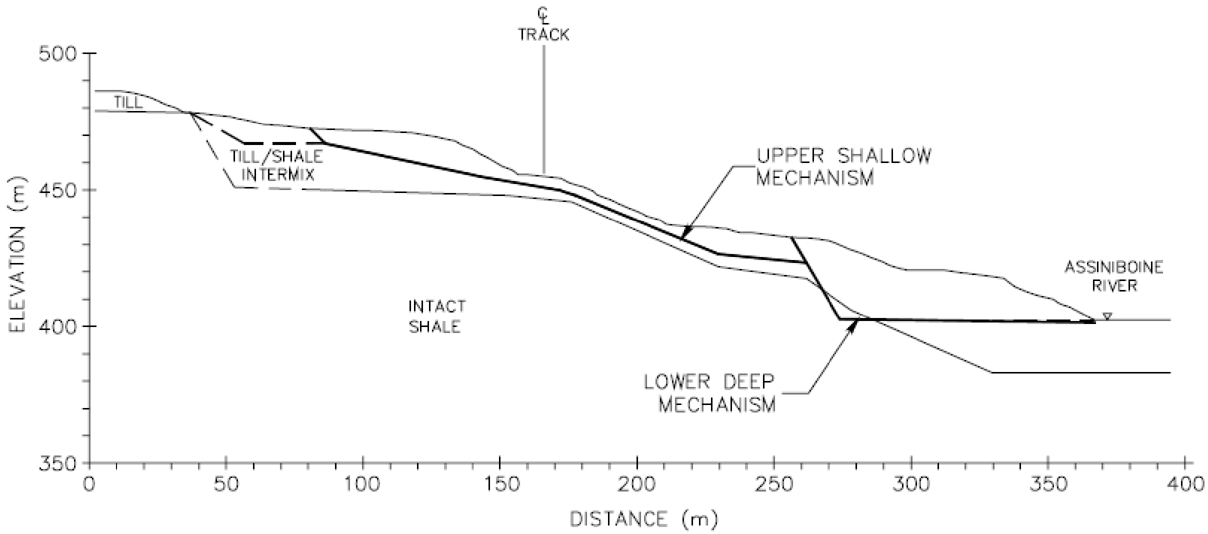
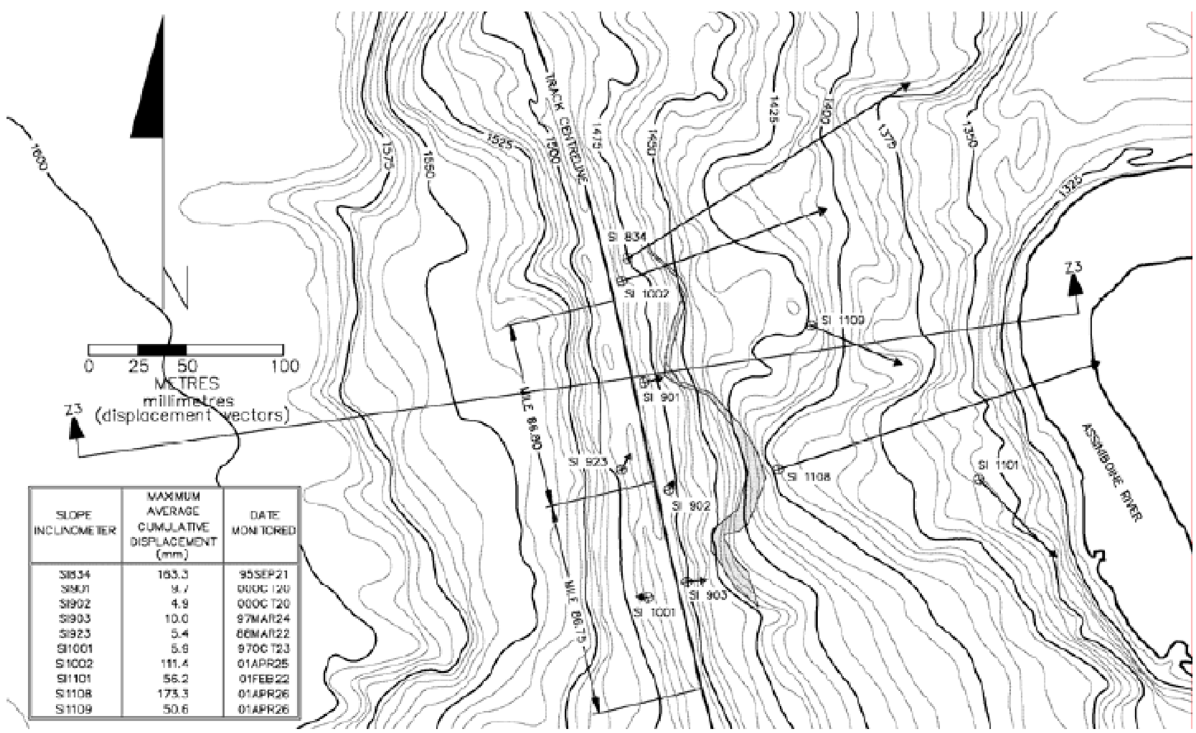


Figure A-32: Cross section of the CP1 site showing the interpreted sliding mechanism. Reproduced from Yong (2003)*, with permission.*

Plan Map



SLOPE INCLINOMETER	MAXIMUM AVERAGE CUMULATIVE DISPLACEMENT (mm)	DATE MONITORED
SI834	163.3	95SEP21
SI901	9.7	00OCT20
SI902	4.9	00OCT20
SI903	10.0	97MAR24
SI923	5.4	86MAR22
SI1001	5.9	97OCT23
SI1002	111.4	01APR25
SI1101	56.2	01FEB22
SI1108	173.3	01APR26
SI1109	50.6	01APR26

Figure A-33: Plan map of the CP1 site showing SI displacement vectors. Reproduced from Yong (2003)*, with permission.*

CP2 – *Carrington M 294.0*

12 km northwest of Valley City, North Dakota, USA

Case Study Summary

Background

A single railway track trending north-west to south-east is located at this site, about 50 m west of the Sheyenne River. The river was reported to show pronounced evidence of meandering nearby. The track is founded on a slope dipping perpendicular to the track toward the east, in the direction of the river. The slope rises roughly 40 m from the elevation of the river banks. About 15 m downslope (east), a 3-4 m wide berm runs parallel to the track (Golder Associates, 2012).

A 135-m long section of this track was known to have experienced historic movements. On-going movement was also noted as of June 28, 2012, particularly in the spring and then decreasing through the summer and fall. Most movement at the site had been associated with prolonged wet periods. Tension cracks at the top of the slope and a bulge at the toe of the slope imposing on the Sheyenne River had been observed via satellite imagery. The affected section of track required regular observation as well as maintenance to restore the alignment and grade. As a result of these instabilities, the operational rail speed here was reported to have been significantly reduced down to 16 km/h (Golder Associates, 2012).

In 1998, Clifton Associates Ltd. was contracted to conduct a geotechnical investigation. The investigation included drilling 5 boreholes labeled BH201 to BH205 to depths of 10-12 m below surface. Standard penetration tests (SPT) were completed in BH201, BH203 and BH205 at 1.5 m depth intervals. Slope inclinometers were installed to a depth of 15.2 m in BH201 and BH203 but a malfunction with the readout unit was reported to have occurred after they were initialized so no meaningful data was retrieved. No further attempts to acquire readings were made. Standpipe piezometers were installed in each of BH202, BH204 and BH205, with screened section between 9-12 m below surface. The groundwater table was found to be approximately 5-7 m deep (Golder Associates, 2012).

The geotechnical investigation determined the soil stratigraphy at the site comprised a 2.3 m thick layer of till overlying 3.7 m of clay (described as ‘weathered or residual shale’), followed by intact shale bedrock down to about 12 m, which was the maximum depth of investigation (Golder Associates, 2012).

An additional borehole drilling and instrumentation installation program was implemented between August 9, 2010 and August 20, 2010. While it was planned to drill 3 new boreholes, only 2 were completed (BH401 and BH403). They were drilled to depths of approximately 30 m and SPTs were completed at 1.5 m depth intervals down to 7.5 m below grade in BH401 and to 10.5 m below grade in BH403. Both boreholes were outfitted with slope inclinometers and a nested set of vibrating wire piezometers. A soil log for BH401 indicated a sequence of silty clay (CL), shale, and an intermediate layer of sand (SP) (Golder Associates, 2012).

Remediation

As part of the geotechnical investigation by Clifton in 1998, a back analysis was completed for the existing slope and possible remedial measures were investigated. A train load was included in the models, as well as a high groundwater table, which led to the calculation of a factor of safety of 0.95-1.05 for the existing conditions. The target factor of safety for remedial measures was approximately 1.2, equating to a 25% increase in stability. It was recommended that the slope be flattened to 2H:1V, that a 4.5 m wide berm be constructed downslope of the rail embankment, that a shear key with a base width of 1.5 m and a key-in depth of 0.6 m into the intact shale be constructed at the toe of the proposed berm, and that a series of granular trench drains be constructed below the track with 12.2 m spacing center-to-center (Golder Associates, 2012).

These remedial measures were implemented between September 29, 1998 and October 28, 1998 per a 1999 construction summary report issued by Clifton (Golder Associates, 2012). The shear key was excavated in 20 ft sections, from west to east, to maintain the stability of both the trench and the track. It was noted that the west end of the shear key was excavated into dark grey unoxidized till which was encountered at a depth of 15 ft. The maximum excavation depth was reported to be approximately 25 ft at this end. The east end of the shear key was keyed into intact clay shale at a depth of about 30 ft. Backfilling was carried out immediately after the excavation was complete using a total of 3500 yd^3 of pit run gravel for the entire trench. The toe berm was constructed using the material excavated from the shear key trench (Clifton Associates Ltd., 2000).

During the construction of the shear key, several important observations were made. On October 2, 1998, the central portion of the shear key was being excavated. A 1 ft thick layer of dark grey clay shale was encountered at a depth of 17 ft, with the layer appearing to increase in thickness toward the east. It was reported that significant seepage was observed at the contact between this clay shale layer and the overlying grey till. The trench excavation proceeded to a depth of 20 ft, to key-in a minimum of 2 ft below the clay shale layer. The report also notes that the trench was partially backfilled with granular material when two trains needed to pass. Following this event, track movement measuring approximately 6 inches to the north was observed above the excavation. The track was realigned and confirmed to be in satisfactory condition, then monitored for the next few days. No additional movement was reported for this section (Clifton Associates Ltd., 2000).

Construction was then halted between October 5-9, 1998 due to inclement weather on site. Approximately 5-6 inches of rain was estimated to have fallen between October 4-6, 1998. Additional seepage coming from a fine sand layer about 10 ft deep was noted when excavation resumed. Upon reaching the required excavation depth, track movement was observed measuring 6 inches to the north over a 20 ft section coinciding with the location of the excavation. The trench was keyed 3 ft into the clay shale before being backfilled with granular material, and then the track was leveled and realigned. The trench was completed by October 13, 1998 (Clifton Associates Ltd., 2000).

An additional investigation into the feasibility of applying cutter soil mixing technology on the site was carried out in 2012. The properties listed below (Table A-3) for the existing materials on the site were used for the modeling completed for that investigation (Golder Associates, 2012).

Table A-3: Summary of the material properties used for the stability analyses for CP2.

Material	Unit Weight (kN/m^3)	Angle of Internal Friction (°)
(CI) sandy SILTY CLAY (Till)	20	30
(CH) CLAY (Residual Shale), residual strength	19	9.5
(CH) CLAY (Residual Shale), peak strength	20	25
SHALE	Impenetrable	N/A

The 1998 repairs that were described above are typical. Toe berms are often employed to increase the normal load in the shear key to achieve greater shear resistance. However, when drainage measures are to be implemented, they are typically constructed first. In this case, the drainage trenches were not constructed until the shear key and toe berm were constructed. Finally, it is believed the granular backfill for the shear key was placed uncompacted.

Performance of Repairs

Baseline readings for the slope inclinometers in BH401 and BH403 were established on August 20-21, 2010. The first readings that followed were acquired on September 9, 2010. An attempt to take another reading on June 6, 2011 was unsuccessful as the inclinometers were found to have sheared since the previous reading. In both BH401 and BH403, 6 mm of movement was recorded in the A-axis direction. For BH401, that movement was found to have occurred in the upper 7.5 m whereas in BH403, the movement had occurred in the upper 9.5 m of soil. An abrupt plane of shear movement was identified at the contact between the till and the residual shale in each borehole (Golder Associates, 2012). Between the completion of the initial repairs on October 28, 1998 and the movement detected in the slope inclinometers in 2010, it can be concluded that the remediation did not perform as expected.

An investigation was carried out between April 16, 2012 and April 21, 2012 in support of a proposal to utilize cutter soil mixing technology to further stabilize the slope. An additional 5 boreholes (G12-01 to G12-05) were drilled and instrumented. The boreholes were drilled to depths of 15-25 m, permitting rock coring in two of the holes (G12-02 and G12-03), SPTs at selected depth intervals, and undisturbed Shelby tube sampling at selected depths. The Shelby tube sampling was not successful though due to the presence of coarse gravel and cobbles which damaged the sampling tool (Golder Associates, 2012).

SlopeAccelArray (SAA) inclinometers equipped with on-site data logging and remote collection capabilities were installed to depths of 15 m (intact shale) in G12-02 and G12-03. Standpipe piezometers were installed in G12-01, G12-04 and G12-05. Laboratory testing was carried out on selected soil and bedrock samples, which included water content testing, Atterberg Limit tests, grain size analyses, direct shear tests, and uniaxial compressive strength testing and point load index testing on bedrock. An external laboratory was consulted for additional tests, one of which indicated a sample from G12-05 contained sodic soil (Golder Associates, 2012). This finding may indicate the clay at this site could be susceptible to excessive swelling, build-up of pore pressure, and weakening through dispersion when wetted.

The SAA inclinometer monitoring reports indicated movement of 11 mm in G12-02 and 4 mm in G12-03 as of May 31, 2012 (Golder Associates, 2012).

Lessons Learned

Based on the inclinometer data acquired 12 and 14 years after the initial repairs, it can be posited that the repairs did not perform as well as anticipated. The investigation in 2012 revealed the possible presence of sodic soils. Potentially inadequate drainage of the shear key could have had negative consequences especially when this condition is taken into consideration. This is a possible explanation for how the repairs performed but this cannot necessarily be confirmed with the data that was available.

Case Study Details

Landslide Information

Landslide type:	Planar slide	*Pre-support rate (mm/yr):*	Unknown
Trigger:	Weathering	*Landslide velocity class:*	
Depth of movement (m):	6.1	*Most recent rate (mm/yr):*	
Sheared material:	Weathered clay shale		

Landslide Dimensions

Width (m):	135	*Length (m):*	
Height (m):	15	*Slope (°):*	26.5

Monitoring Information

Movement first reported:	Pre-1998	*Last inspected:*	April 21, 2012
SIs installed:	6	*SIs active last inspection:*	2
Piezos installed:	8	*Piezos active last inspection:*	3

Groundwater Information

Groundwater level (mbgl):	5-7	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	No	*Surface water type:*	River

Repair Specifications

Repair type:	Buttress & Shear key	*Repair date:*	October 28, 1998
Base width (m):	1.5	*Trench slope ratio (H:V):*	0.3:1
Length (m):	109	*Drainage (yes/no):*	No
Granular height (m):	6.1-9.1	*Backfill:*	Uncompacted
Overburden (m):	1.45		
Overburden material(s):	Clay fill (toe berm)		

Additional Site Information

Stratigraphy

CPR Carrington Mile 294.0

Case study location: 12 km northwest of Valley City, North Dakota, USA

Depth (m)	Soil description (From 1998 Geotechnical Investigation)	Type	Source	Structure
0.0 to 2.3	(CI) sandy SILTY CLAY (Till)		G	
			G	
2.3 to 6.0	(CH) Weathered/residual clay shale	s	M	
			M	
		s	M	
			M	
6.0 to 12.0	(CH) Intact clay shale bedrock		M	
			M	
			M	
			M	
			M	
			M	

Legend

Type		Source		Structure	
Organics/Topsoil		Fill	F	Slickensided	
Clay		Fluvial	R	None	
Silt		Lacustrine	L		
Sand		Marine	M	**Groundwater (in Type)**	
Till		Glacial	G	Seepage	s
Stone/rock		Assumed	*Italicized*	GWT	
Interbedded		Unknown	?		

Figure A-34: Schematic interpretation of the stratigraphy described for CP2.

CP4 – *Emerson M 61.57*

2.4 km northeast of Emerson, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by Canadian Pacific Railway in an unpublished report (2002).

Background

A single track trending northeast-southwest just outside of the town of Emerson, Manitoba crosses over Joe Creek at Mile 61.57 of the CPR Emerson Subdivision at an approximate intersection angle of 30°. Joe Creek is a small waterway flowing to the north where it joins the Red River on its east bank. The rail crossing is about 1 km south of the point where Joe Creek meets the Red River, and about 0.8 km east of the Red River. The Red River also flows roughly toward the north in this location.

On the south side of the rail crossing, the approach embankment experienced instability in the direction of Joe Creek (east). The failing section spanned 50 m in length, beginning about 50 m south of the bridge and extending to 100 m south of the bridge. Until 2002, CPR appears to have managed the instability by adding ballast. The result was a severely over-thickened ballast section on the east side of the track there.

Remediation

In September of 2002, a discussion to carry out stabilization work took place.

Soil parameters were to be determined through back analysis of the existing embankment, likely by assuming a FOS of 1.0. It was reported that no subsurface information was collected, so it was understood at the time that additional stabilization might be required should the remediation perform inadequately. It was also noted that the work should only be completed during prolonged dry conditions due to the possibility of an embankment failure should a toe excavation proceed in wet conditions.

A combined shear key and toe berm was proposed to stabilize the embankment. Two options were considered for implementing this solution. The first, which was noted as being preferred, was to use free draining granular material for both the shear key and the toe berm. This would enhance the drainage of the system as well. A total volume of 1250 m^3 of granular was estimated to be required for this option.

The second option was to use free draining granular material sourced from ballast reject in order to construct a 1 m thick under-drain. The rest of the berm would be constructed using compacted general fill, which would be acquired from the shear key excavation. The shear key would then require a total volume of 525 m^3 of granular.

It was then proposed that the shear key excavation be extended north along the base of the excavation up to the point where the base of the 2-m deep excavation daylighted in the Joe River valley. By backfilling the excavation with free draining granular material, the designers expected the trench would provide a conduit to drain the toe of the slope and base of the shear key. The designers calculated that an average section area for the trench was about 4 m^2. A length of 20 m was quoted for this trench so it would therefore require 80 m^3 of granular.

Finally, a cut-off trench drain was proposed to intercept flow along the base of the ballast and sub-ballast. The proposed trench would be 1.5 m deep and 0.75 m wide, and would daylight on the east side of the embankment so as not to potentially destabilize the west embankment slope. The length of the trench would be about 15 m, requiring a volume of 17 m^3 of granular.

The goal of the work was initially to increase the factor of safety by 50% using relatively conservative assumptions. However, a CPR representative proposed that a 25% increase in the factor of safety would be sufficient for this location. The solution detailed above is consistent with the initially proposed increase of 50%. To achieve a 25% increase, it was proposed that either a shear key with slope flattening or a stand-alone toe berm be selected. A shear key with drainage measures was stated to be the preferred option provided uncertainty still existed regarding whether ground movement would be halted.

The proposed repairs are consistent with typical designs from a general overview perspective. The nature of the slide was not described in the available reports and so it is not possible to comment on whether the proposed designs were appropriate.

Performance of Repairs

No information was located on whether the repairs were ever implemented, nor how they performed.

Lessons Learned

It can be seen in this case study that drainage was prioritized compared to increasing the normal load acting on a shear key (in the form of a toe berm). This is likely due to the cost effectiveness of this option. The amount of material that must be moved for the construction of a toe berm would be roughly equivalent to that of the shear key itself and hauling constitutes one of the major costs in such projects.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:		*Landslide velocity class:*	
Depth of movement (m):		*Most recent rate (mm/yr):*	
Sheared material:			

Landslide Dimensions

Width (m):	50	*Length (m):*	
Height (m):		*Slope (°):*	

Monitoring Information

Movement first reported:		*Last inspected:*	
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):		*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	Stream/Creek

Repair Specifications

Repair type:	Shear key	*Repair date:*	
Base width (m):		*Trench slope ratio (H:V):*	
Length (m):	50+	*Drainage (yes/no):*	Yes
Granular height (m):	2	*Backfill:*	
Overburden (m):			
Overburden material(s):	Toe berm (excavated material, compacted general fill)		

CP5 – *Lanigan M 26.35*

Case study location: East shore of Last Mountain Lake, 25 km NW of Regina, Saskatchewan

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by Clifton Associates Ltd. in an unpublished report (2003).

Background

The slope below a 55-m long section of CPR track at Mile 26.35 of the Lanigan Subdivision was found to be susceptible to toe erosion and slumping toward the lake. Remedial work was anticipated to improve the stability of the track bed while also reducing the chance of soil slumping into the lake.

The slope at this location was described as being steep, with a slope of 1.8H:1V. The rail is partway up this slope, restricting possible remedial options. A perched groundwater table was identified within the track subgrade, likely explaining the high pore water pressures that were present.

Remediation

Plans for remediation were reported but it is uncertain whether remediation was ever pursued. The planned slope remedial work involved regrading the entire slope below the track, construction of erosion protection at the toe and the installation of 6 subsurface drains on the immediate downslope side of the tracks. A detailed construction sequence mentioned the installation of a silt fence at the base of the slope above where rip rap was to be placed. The upper slope was to be regraded to 2H:1V and a bench was to be constructed with regraded soil to provide site access for the excavators to be used in drain construction. The bench was then to be removed and the rest of the slope was to be regraded to 2H:1V, with excess spoil to be placed immediately upslope of the rip rap. A second silt fence was to be placed at the shore line, followed by the excavation of a rip rap key trench.

For the rip rap key trench, the topsoil was to be stripped and then geotextile, bedding sand and rip rap were to be placed. The trench was to span 1 m by 1 m across 50 m of lakeshore. A final regrading of the site was to be completed afterward, followed by seeding and fertilizing of the slope and spoil. The remainder of the rip rap section was to consist of a geotextile covered by 150 mm of granular fill and 600 mm of rip rap. It was reported that the geotextile would be anchored in a 300 mm by 300 mm trench.

Six subsurface granular trench drains were to be constructed at a 10-m spacing in order to drain the perched water table. Each drain was to be constructed by excavating of a 1.0 m wide and 5.5 m deep trench, placing a perforated PVC pipe at the bottom, backfilling with granular material and topping with 0.5 m of common fill. The pipes were to daylight on the slopes below the trenches. The vegetation was anticipated to be sufficient to handle the flows, which were expected to be minimal. All work was expected to be carried out using typical equipment consisting of an excavator, small bulldozer and front end loaders.

This is the first case study where any mention of bedding sand has been included. The bedding sand is used to protect the geotextile from tearing due to direct contact with the coarse rip rap. Another component of these repairs is the use of silt fences. They were to be placed to control lake siltation during and after construction, up until a vegetative layer could be established.

Performance of Repairs

Four slope inclinometers were reported for this site: SI104, SI301, SI302, and SI303. Monitoring was reported as having been conducted on September 26, 2002. Plots of the cumulative and incremental deflections were produced and were not indicative of any significant slope movements.

Four piezometers were also installed: 301, 302A, 302B and 303. Piezometric levels at Mile 26.35 and Mile 26.18 were found to be 0.56-3.11 m greater than levels measured on June 7, 1996. Each piezometer was found to be dry on November 10, 1993. It is unknown how well the repairs performed, if they were ever implemented. As of February 3, 2013, they had still not been implemented.

Lessons Learned

The excess spoil placement appears to have been selected to provide a buttressing effect at the toe of the slope. This would likely reduce hauling costs especially for a site that is relatively inaccessible except by rail. The mention of bedding sand being used in the rip rap key trench is an important detail that has not been observed in other case studies comprising this research. Without bedding sand, the use of geotextiles may be largely negated due to direct contact with angular fill producing tears in the fabric during compaction or movement.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:	Toe erosion	*Landslide velocity class:*	
Depth of movement (m):		*Most recent rate (mm/yr):*	
Sheared material:			

Landslide Dimensions

Width (m):	55	*Length (m):*	
Height (m):		*Slope (°):*	29

Monitoring Information

Movement first reported:		*Last inspected:*	September 26, 2002
SIs installed:	4	*SIs active last inspection:*	4
Piezos installed:	4	*Piezos active last inspection:*	4

Groundwater Information

Groundwater level (mbgl):	Perched	*Surface water at toe:*	Yes
Seepage detected (yes/no):		*Surface water type:*	Lake

Repair Specifications

Repair type:	Buttress & Shear key	*Repair date:*	
Base width (m):	1	*Trench slope ratio (H:V):*	Vertical
Length (m):	50	*Drainage (yes/no):*	No
Granular height (m):	1	*Backfill:*	
Overburden (m):	0.75		
Overburden material(s):	Rip rap		

CP6 – *Lloydminster M 78.3*

6 km southeast of Lone Rock, Saskatchewan

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by Clifton Associates Ltd. in an unpublished report (2004).

Background

A section of single track runs along a side hill and across a small drainage channel at this site. The track sits on an 8-m high embankment. The valley wall here begins about 30 m below track grade and rises to 60 m above track grade. There is a 610-mm culvert on the north end of the fill section but the culvert inlet was blocked by a beaver dam at the time of a report filed in the spring of 2004. As a result, seepage from the valley wall had ponded upslope of the embankment.

During an inspection on October 17, 2002, cracking was observed in the upslope shoulder of the track. The downslope shoulder of the track was reported to have dropped by more than 1.5 m. The downslope direction and the direction of the movement were to the east. It was noted that lifting and the addition of ballast were frequently required at this location. A full field investigation was performed between October 30 and November 4, 2002. The investigation included test pitting, drilling, surveying and monitoring. Monitoring of the instrumentation was completed once again on June 10, 2003.

The slope inclinometer data was used to determine that a deep-seated failure had occurred within the bedrock because of high pore water pressure in the railway embankment and low shear strength in the bedrock. The depth of the failure was 13.3 m below the rail ties or 8.7 m below ground surface. This corresponded to the upper 0.6 m of the clay shale bedrock. Cumulative displacement of 29.4 mm was recorded roughly perpendicular to the track. An approximately 100 m long span of was being affected by the slope deformations.

The stratigraphy at the site was determined using boreholes, labelled BH101, BH102 and SI103. BH101 was located beneath the track and extended to a depth of exploration of 3.7 m. BH102 and SI103 were located on the flanks of the embankment and extended to a depth of exploration of 17.5 m. In BH101, a 0.9 m thick layer of ballast was found to overlie 1.5 m of sandy sub-ballast.

Below this was 0.3 m of cinders followed by silty clay fill down to a depth of 3.7 m. In BH102 and SI103, a 5.6-5.9 m thick layer of fill was found to overlie a 0.2-0.4 m thick layer of topsoil. Below this, a 2.1-3.2 m thick layer of clay, sand and till was identified, followed by clay shale bedrock down to a depth of 17.5 m.

The fill was described as predominantly soft to stiff, silty clay with traces to some sand. Some layers of gravel and silt were identified too. The topsoil, which was the ground surface before fill was placed, was reported to contain silt, clay and organics. The clay below the topsoil was described as oxidized, silty, firm, high plasticity clay. The sand was described as fine grained and oxidized. The till was described as a soft, oxidized to unoxidized, silty, low plasticity clay till. Lastly, the clay shale was said to be a silty clay with traces of fine grained sand, and was described as highly plastic, dark gray in colour, unoxidized and ranging from stiff to hard in consistency.

The groundwater conditions at the site were measured on November 4, 2002 and June 10, 2003 from standpipe piezometers located in BH102 and BH104. The groundwater table was found to lie at a depth of 4.2-4.4 m.

Remediation

The site was remediated between September 24, 2003 and October 17, 2003. Remediation consisted of constructing an 85-m long shear key along the east side of the tracks, installing three trench drains perpendicular to and intersecting the shear key, extending the existing 610 mm culvert outlet, and clearing the culvert inlet. The inlet had previously been blocked by a beaver dam.

It was reported that the drains were constructed first, to lower the groundwater table in preparation for the shear key excavation. For the drains, 150 mm "Big O" pipes were used and the trenches were backfilled with pea gravel before being capped with clay common fill. The shear key excavation then began on September 30, 2003. Pit run gravel was used for the backfill material in the shear key. The pit run gravel was required be free draining, with a grain size distribution of less than 5% passing the 71 μm sieve.

The designers opted to locate the shear key as far as possible down slope to minimize the depth of the excavation and to minimize any potential impact to the tracks. The track also remained in service throughout construction. This was done by utilizing track blocks then backfilling the excavation and stabilizing the shear key sideslopes before rail traffic arrived. The excavation began on the south end and proceeded to the north end by completing short panels to minimize the length of the unsupported trench walls. It was reported that the soil from between the embankment and the shear key was removed and stockpiled below the shear key to help further stabilize the trench walls.

Stability analyses were performed using the site stratigraphy described previously. A train load of 55 kPa was superimposed on the embankment and the water table from the June 10, 2003 monitoring was used for the analyses. The following is a summary of the soil properties used for the analysis (Table A-4). The clay and clay shale properties were determined using back analysis.

Table A-4: Summary of the soil properties used in the stability and design analyses.

Material	Unit Weight (kN/m^3)	Effective Cohesion, c' (kPa)	Effective Friction Angle (^{o})
Fill/Berm	20	5	24
Clay	18	3	13.4
Sand	20	0	30
Shear Key Granular	20	0	35
Clay Shale	19.9	3	13.4

The remedial design minimum target factor of safety was 1.30. The shear key was designed to be approximately 10 m deep to extend a minimum of 0.5 m below the shear surface. To key into intact shale though, the excavation depth beyond the shear zone often exceeded 0.5 m. The base was designed to be 3 m wide and the sideslopes were designed to be 0.5H:1V. The design called for the shear key to extend 14 m east of the CPR right-of-way for a length of 100 m, which required that the shear key extend onto the adjacent property. Soil conditions on the north end of the shear key were found to be better than anticipated which allowed for the length to be reduced to 85 m. A thin clay cover of common fill was used to cap the shear key.

The described repairs are consistent with typical shear key designs. The depth of the excavation lies on the greater end of the typical range of depths for shear key projects. While closely sequenced excavations are commonly adopted, it is less common for the transportation route to remain active during construction.

Performance of Repairs

During construction, a monitoring lath was used to observe deformations. Near vertical displacements could be seen in the track. Groundwater was also discharged from the rail bed.

No reports were available for the period after construction. It is assumed the repairs performed satisfactorily.

Lessons Learned

This case study provided a list of the contractor equipment that was used to carry out the project. A D5H Caterpillar Dozer, 25 Ton Caterpillar Rock Truck, 350BL Caterpillar Track Hoe, 744H John Deere Loader and a 955L Caterpillar Track Loader were used. It is understood that most of this equipment is considered standard for this type of project.

Case Study Details

Landslide Information

Landslide type:	Compound slide	*Pre-support rate (mm/yr):*	
Trigger:	Saturation & Weak layer	*Landslide velocity class:*	
Depth of movement (m):	8.7	*Most recent rate (mm/yr):*	
Sheared material:	Clay shale		

Landslide Dimensions

Width (m):	100	*Length (m):*	
Height (m):		*Slope (°):*	

Monitoring Information

Movement first reported:	October 17, 2002	*Last inspected:*	October 17, 2003
SIs installed:	1	*Piezos installed:*	2
SIs active last inspection:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	4.2 – 4.4	*Surface water at toe (yes/no):*	No
Seepage detected (yes/no):		*Surface water type:*	

Repair Specifications

Repair type:	Shear key	*Repair date:*	October 17, 2003
Base width (m):	3	*Trench slope ratio (H:V):*	0.5:1
Length (m):	85	*Drainage (yes/no):*	Yes
Granular height (m):	10	*Backfill:*	20 kN/m^3
Overburden (m):	Thin		
Overburden material(s):	Clay fill		

EX1 – *Bear Creek Village Condominiums*

Grande Prairie, Alberta

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by GES Geotech Inc. in *Slope Assessment & Stabilization Works, Bear Creek Village Condominiums, Grande Prairie, Alberta* (2016).

Background

Slope remediation work was carried out on a slope in Grande Prairie, Alberta by GES Geotech Inc. There is a condominium development sitting at the top of the east side of the Bear Creek valley. Construction of this development took place from 1997 to 1999.

Site investigation revealed two historical slide areas each spanning 105-120 m in width. The pre-established shear zones in the natural slopes at the site of the condominiums led to slope instability, with tension cracks being observed at surface and cracks forming in the concrete foundation walls.

A thorough geotechnical investigation was carried out. It revealed the slope consisted of banded lacustrine silt and clay overlying clay till at a depth of approximately 3.5-4.0 m at the location of the remedial measures. The lacustrine sediments were described as mostly stiff to very stiff. Some zones were identified where the clayey soil was soft and highly plastic and contained pre-sheared zones. Bedrock was identified at a depth of about 30 m. The groundwater table was indicated to be at roughly the same level as the surface of the clay till. The slope was depicted as being roughly 3H:1V.

Remediation

Following the geotechnical investigation, GES Geotech Inc. installed a shear key in the lower slope. The investigation and design process took place between May 2003 and 2010 so it is believed the shear key would have been installed shortly thereafter.

The shear key was keyed at least 1.5 m into the till. It is likely the shear zone extended to roughly just above the surface of the till. The GES report indicated that granular fill was used and that it was placed then compacted in lifts.

The repairs described in the report are typical of shear keys. Granular fill material was used, the slide was relatively shallow, and the shear key was keyed into a more competent material than the overlying sediments. However, the key-in depth of 1.5 m is toward the upper limit of typical key-in depths.

Performance of Repairs

There were no reported measures of performance in the report that was consulted.

Lessons Learned

The report cites compaction of the granular fill as being undertaken to mobilize its full strength. This is consistent with conclusions from other case studies and publications.

Case Study Details

Landslide Information

Landslide type:	Planar slide	*Pre-support rate (mm/yr):*	
Trigger:	Construction	*Landslide velocity class:*	
Depth of movement (m):	3.5	*Most recent rate (mm/yr):*	
Sheared material:	Lacustrine silt/clay		

Landslide Dimensions

Width (m):	105-120	*Length (m):*	15 (?)
Height (m):	5	*Slope (°):*	18.4 (estimate)

Monitoring Information

Movement first reported:	January 1, 2010	*Last inspected:*	
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	3.5-4.0	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	Stream/Creek

Repair Specifications

Repair type:	Shear key	*Repair date:*	January 1, 2010
Base width (m):	1.5	*Trench slope ratio (H:V):*	0.75:1
Length (m):	105 (assumed)	*Drainage (yes/no):*	Yes
Granular height (m):	5	*Backfill:*	
Overburden (m):	0		
Overburden material(s):	None		

EX2 – *Bender's Park Landslide*

Lead, South Dakota, United States of America

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by Reuter and Kwasny in *Stabilization of the Bender's Park Landslide, Lead, South Dakota* (2013).

Background

A slide at Bender's Park in Lead, South Dakota had been moving since the mid 1950s. The 350 000 m^3 slide spanning 2.6 hectares was exacerbated by a combination of heavy snowmelt and spring rain in 1994 so the decision was made to remediate the slide. Prior to remediation, movement was monitored and found to range from 0.003 to 1 mm/day. After a May 8-10, 1995 rainfall event, movement accelerated to more than 50 mm/day. The slide was 185 m wide and 174 m long, with relief of 29 m. A slope of 5-9° was present at the base of the hill, but a 6-m cut forming a slope of 45-50° had been made in the early 1920s at the toe. The cut permitted a well-defined, near-vertical shear zone to be observed. The shear zone was thin, spanning 50-75 mm, but highly polished slickensided surfaces were present. Back-analyses suggested a residual friction angle between 10-12° along the shear surface.

A distinct head scarp 0.1-0.3 m high was identified, whereas the flanks of the slide were poorly defined. The toe of the slide could be identified by damage consisting of bumps and cracks in the pavement, and uplifting of the lower floor slabs in the mall that was there.

The slope itself consisted of a surficial layer of soil fill or disturbed fill up to 3 m thick, a direct result of development. Beneath this was a major deposit of tan, buff, brown and light brown clayey silt and silty clay belonging to the White River Formation. White calcite nodules and the occasional schistosic clast were present. The soil had a liquid limit of 48-77 and a plastic index between 25-40. Under this unit was a red-brown silty clay containing numerous schistosic clasts especially just above the bedrock. X-ray diffraction showed that the White River Formation consisted predominantly of montmorillonite at this site.

The slip surface was predominantly located within the White River Formation, just above relatively dense soil for most of the slide and near the bedrock surface closer to the head of the landslide. The single failure surface was identified at a depth of 19 m within the main body toward the toe, beneath First Street. At this location, the White River Formation was found to exceed depths of 30 m. The bedrock was noted as being relatively shallow toward the head of the slide though, lying less than 1.5 m below the topsoil and a colluvium layer.

Due to lawn irrigation and breaks in the water/sewer lines running through the subdivision, water was an issue in the landslide. Further to these sources of water, the 24-hour rainfall in the area historically ranged from 25-50 mm. From May 8-10, 1995, 201 mm of rainfall was recorded. The average annual snowmelt was 614 mm with a snow water equivalent of 145 mm. In 1994, the average snow depth was 858 mm yielding a snow water equivalent of 221 mm. These conditions severely exacerbated the existing slide. The GWT was identified 7-12 m below ground, which was 6-14 m above the slide plane within the deepest section of the slide.

The mall at the toe of the slide had experienced cracking of slabs and walls shortly after construction, and lateral movements of more than 50 mm in some of the interior columns were recorded after movement in the spring of 1994. It was decided that remedial measures had become necessary, and were begun in the fall of 1995.

Remediation

To stabilize the slide, four options were considered. The first was to purchase all the properties in the affected subdivision so the entire landslide could be regraded. This was ruled out for being too disruptive to the community. The second option as to install deep subsurface drains or wells to lower the GWL, but the community favoured a passive approach instead. A third option involved the construction of a rock fill berm as a toe buttress, and the demolition of the mall. This was judged to be an unnecessarily large burden to the community though. Finally, ground anchors, stone columns or drilled shafts were considered. Ultimately, an alternative to use three deep, rock-filled shear trenches was selected instead.

The trenches were to be excavated below the failure plane, and located along the slide head and toe. The purpose was to increase shear resistance and enhance subsurface drainage. All parties involved were notified that movement would likely persist for an extended period of time before the slide halted entirely.

The design of the trenches assumed a friction angle of 40° for the rockfill and the target FOS was 1.4. The designers conducted analyses assuming the presence of an active failure wedge in the upslope shear trench and a passive failure wedge in the downslope shear trench. The drilled piers that ended up being installed along the upslope side of the shear trench located at the toe were not accounted for in the analysis.

Construction was expected to take months so concrete caissons were installed along the upslope side of the bottom trench to form a wall. The caissons were 1.2 m in diameter and spaced 3.1 m center-to-center. Installation took place in July 1995 to a depth of 17.8-18.9 m, which was 7-9 m below the shear plane of the landslide. It was expected soil arching could be relied upon so the caissons were not installed edge-to-edge. The total length of the wall was 162 m, and it was capped with a reinforced concrete grade beam so the caissons would be tied together and act as a single rigid wall. This wall was expected to provide the benefit of uniformly distributing loading to the shear trench that would later be installed immediately downslope.

The shear trench construction was sequenced such that a deep drainage trench was excavated near the toe first, followed by the shear trench along Montana Avenue near the mall, then the one along 3rd Street near the head of the landslide. A drainage trench was excavated along the left flank of the slide to connect the two shear trenches, permitting drainage from the whole landslide. The construction sequence called for the drainage trench near the toe first in order to intercept groundwater and relieve some of the driving forces while the shear trench near the toe was constructed. The shear trenches were constructed as moving slots, or in other words, maintained a closely sequenced excavation. The span of open trench at the bottom was limited to less than 7 m at any time, and the excavation was staged using a bench. The Aspen Street shear trench (near the mall and toe) had an 11 m right of way. The trench was excavated to a depth of 13.4 m to intercept the shear plane, and was completed in January 1996.

The backfill used in the shear trenches was blasted, angular waste mine rock from Homestake Mining Company, a local company operating only 2 km away. Only good quality, sound and durable rock was selected for use. The gravel was classified as being well-graded, ranging from 50 to over 500 mm in size and possessing minimal fines.

Most of the described repairs are typical of remedial works which incorporate shear keys. However, it is not typical to construct a shear key near the scarp of a slide. It is also not typical to install a caisson wall to work in conjunction with a shear key trench. It can reasonably be argued though that the size of the downslope shear key trench and the presence of several buildings both on the sliding mass and below it more than justify this additional stabilizing measure.

Performance of Repairs

The designers estimated it would take 3-5 years for significant reductions in the rate of movement. Once completed, the structures resulted in the rate of movement decreasing over 4 years as passive resistance was mobilized. As of 2002, no additional movement had been detected.

The upper portions of the piers installed toward the toe of the slide were observed to experience a gradual bending. Cumulative displacements were measured at the caissons and in the body of the slide after construction ceased. The rate of movement decreased until mid-1999, after which there was very little movement. The total movement over the 6-year period after construction was between 47-51 mm. Immediately downslope of the Aspen Street shear trench (nearest to the mall), no movement was detected, meaning the mobilization of the trench had not translated to movement downslope.

The reported performance of the repairs is consistent with other shear key projects. The total displacement over the 6-year period after construction is relatively high though. This can possibly be attributed to the fact that compaction of the rockfill was not mentioned in the construction procedures that were listed.

Lessons Learned

This project was unique in that a caisson wall was installed prior to excavation of the downslope shear key trench. The caisson wall helped stabilize the excavation and helped to distribute the loading from the sliding mass along the shear key (Reuter & Kwasny, 2013). This project also showed that a relatively large shear key can be successfully constructed using typical techniques, such as closely-sequenced excavations and benching.

Case Study Details

Landslide Information

Landslide type:	Compound slide	*Pre-support rate (mm/yr):*	1-365, 18250
Trigger:	Rain event	*Landslide velocity class:*	3 - Slow
Depth of movement (m):	~10 (Aspen St)	*Most recent rate (mm/yr):*	Negligible
Sheared material:	Brown and tan Clayey Silt and Silty Clay		

Landslide Dimensions

Width (m):	185	*Length (m):*	174
Height (m):	29	*Slope (°):*	5-9, 45-50 at toe

Monitoring Information

Movement first reported:	Mid-1950s	*Last inspected:*	Mid-2002
SIs installed:	2+	*SIs active last inspection:*	2+
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	5-7	*Surface water at toe (yes/no):*	No
Seepage detected (yes/no):		*Surface water type:*	N/A

Repair Specifications

Repair type:	Shear key	*Repair date:*	Completed January 1996
Base width (m):	<11	*Trench slope ratio (H:V):*	
Length (m):	162	*Drainage (yes/no):*	Yes
Granular height (m):	13.4	*Backfill:*	Blasted, angular waste rock
Overburden (m):	3.3		
Overburden material(s):	Compacted clay cap		

EX3 – *Hagg Lake Slide*

Hagg Lake, Oregon, United States of America

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by Cornforth and Fujitani in *Residual Strength of Volcanic Clay: Two Case Studies* (1991).

Background

A pair of landslides occurred at Hagg Lake, Oregon in December 1977 during a period of heavy rainfall. The slides occurred in what was identified as ancient landslide terrain. One of the landslides was remediated using a shallow key trench and a rockfill buttress at the base of the slope.

The slide was 85 m wide, 45 m long and 18 m high. The shear surface was identified and found to run very close to the interface between the highly weathered residual soil overburden and the underlying marine sedimentary bedrock belonging to the Yamhill Formation. The GWT was found to be a few feet above the shear zone, but an irrigation reservoir at the toe of the slide was drained at the time of the slide's occurrence. The soil was described as stiff to very stiff silty clay, with siltstone fragments and nodules which increased in quantity with depth. Also, worth noting were small amounts of wood observed in the shear zone, at a depth of 7.3 m, and basalt fragments at a depth of 3 m further downslope. These last observations are likely a result of the volcanic activity that deposited the soils.

Lab testing on the overburden soils indicated a plastic limit of 38 and a liquid limit of 60 at a depth of 5 m. Moisture content was measured at 41% with a clay fraction of 25% and a wet density of 1.84 g/cc. This clay fraction places the soil within the realm of turbulent shear, as proposed by the authors. The undrained shear strength was measured to be 72 kPa. Two direct shear tests were carried out on the overburden soil and suggest a residual friction angle of 33° and no cohesion. This was approximately equal to the peak parameters as well. Five consolidated-drained triaxial tests suggested a peak friction angle of 33° too.

Testing on samples from a depth of 7.9 m, from within the 3-4-foot-thick shear zone, was also undertaken. Seven stress reversals were performed during direct shear testing and yielded a residual friction angle of 22° and no cohesion. This was more consistent with the back-calculated residual friction angle of 23° and cohesion of 0 kPa for the slope.

Remediation

The slope was stabilized using a shallow key trench and rockfill buttress located at the toe of the slope. The outer slopes of the buttress were flattened and horizontal drains were installed to allow drainage to proceed faster as the irrigation reservoir is drawn down. The total cost of the remediation project was USD$300 000 in 1981.

Performance of Repairs

Cornforth and Fujitani (1991) did not present data or commentary pertaining to the performance of the repairs.

Lessons Learned

Cornforth and Fujitani (1991) have demonstrated that direct shear tests used to determine the residual strength properties of slide materials can yield a variety of results depending on the number of stress reversals performed.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:	Rain event	*Landslide velocity class:*	
Depth of movement (m):	7.3	*Most recent rate (mm/yr):*	
Sheared material:	Highly weathered residual soil		

Landslide Dimensions

Width (m):	85	*Length (m):*	45
Height (m):	18	*Slope (º):*	

Monitoring Information

Movement first reported:	December 1977	*Last inspected:*	
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level:	"few feet above shear zone"	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	Reservoir

Repair Specifications

Repair type:	Buttress & Shear key	*Repair date:*	1981
Base width (m):		*Trench slope ratio (H:V):*	
Length (m):		*Drainage (yes/no):*	Yes
Granular height (m):	"Shallow"	*Backfill:*	
Overburden (m):			
Overburden material(s):	Rockfill buttress		

EX4 – *San Joaquin Hills*

Orange County, California, United States of America

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by Vakili et al. in *Investigation of failure of a cut slope in soft rock* (1998).

Background

A slide occurred in a cut slope in soft rock in the San Joaquin Hills of Orange County, California in June 1995. The failure took place during construction and involved more than 0.5 million m^3 of material. The slope was designed to be 3H:1V (18°) and over 60 m high upon completion. The soft rock that failed consisted of interbedded siltstone and claystone. The slide was documented in detail, with Vakili et al. (1998) reporting a slide length of 555 m, a slope height of 67 m and a dip of 15-20° out of slope for the sheared siltstone bedding. An ancient landslide at the same location was discovered, spanning 213 m in width and reaching a depth of 30 m. The slide was classified as being translational in a direction parallel to the bedding. The lateral limits of the slide were noted to coincide with faults running through the site. Vakili et al. described the slip surface was a zone of sheared and brecciated clayey siltstone 2 m in thickness, possessing remolded clay seams. They did not specify the depth of this shear zone though, so the cited slide volume, width and length were used to estimate an average of 8 mbgl. Vakili et al. state the back analyses that were conducted showed an effective friction angle of 10° with cohesion of 5 kPa. Deeper boreholes discovered another identical shear surface 32 m below surface level near the toe of the slide.

Remediation

The slide was remediated by designing a buttress fill and shear key, with a new slope of 2H:1V rising 49 m high. This stabilization technique was selected as the most time and cost efficient solution that was considered. The new slope was found to require a shear key 24 m deep using Spencer's Method in a LEM analysis aiming for a FOS of 1.5. Heavy groundwater seepage was identified 11 m below the base of the slope as well, which would make construction more difficult. The as-built design was modified to be 38 m high and only required a shear key 9.1 m deep. This shear key was constructed 15 m wide at its base, with a drainage bench near the middle. The buttress was built 122 m into the slope to remove and replace a large portion of the bedding plane fault, and incorporated an extensive subdrainage system with the authors citing an anticipated increase in irrigation in the future. The head scarp of the slide was also cut to 1.5H:1V during this remediation project.

Construction of the shear key was accomplished by sequencing the excavation and backfilling of sections less than 100 m long at a time.

Performance of Repairs

Information pertaining to the performance of these repairs was not available.

Lessons Learned

In the context of the other case studies that were collected, this case study is most notable for the fact that a granular shear key was constructed to remediate a soft rock slope. The slopes studied for this work were all based in cohesive soils. Similar themes can be identified though, such as drainage being a considerable aspect of the work performed at this site. Fundamentally, the shear key was still excavated such that a shear surface was intercepted. The construction was performed in sections, but the length of these sections far exceeds the length of trench left open for projects in clays. This likely reflects the strength of the materials involved.

Also noteworthy is the size of the slide involved, which is significantly greater than most of the other slides that were studied.

Case Study Details

Landslide Information

Landslide type:	Planar slide	*Pre-support rate (mm/yr):*	
Trigger:	Construction	*Landslide velocity class:*	
Depth of movement (m):	8, 30-32	*Most recent rate (mm/yr):*	
Sheared material:	Interbedded siltstone and claystone		

Landslide Dimensions

Width (m):	213	*Length (m):*	555
Height (m):	67	*Slope (°):*	26.6

Monitoring Information

Movement first reported:	June 1995	*Last inspected:*	
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):		*Surface water at toe (yes/no):*	No
Seepage detected (yes/no):	Yes	*Surface water type:*	N/A

Repair Specifications

Repair type:	Buttress & Shear key	*Repair date:*	
Base width (m):	15	*Trench slope ratio (H:V):*	1:1
Length (m):	213	*Drainage (yes/no):*	Yes
Granular height (m):	9.1	*Backfill:*	
Overburden (m):			
Overburden material(s):	Buttress fill		

WP1 – *1, 7 & 11 Evergreen Place*

Evergreen Road, Winnipeg, MB

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by KGS Group in *Outfall Pipe Performance along Stabilized Riverbanks - Preliminary Assessment of Existing Sites in the City of Winnipeg* (2015).

Background

Located on the south bank of the Assiniboine River in Winnipeg, Manitoba, this site comprises three office buildings at the end of Evergreen Road and immediately to the west of the Osborne Street bridge.

The riverbank soil near surface for this site is composed of interlayered sand, silt and clay of alluvial origin. A thin, 0.15 m thick layer of lacustrine clay was identified directly underlying these deposits. Below this, a silt and clay till was identified at a depth of approximately 11 m below the top of the bank. The overall slope of the bank is 3.4H:1V but the lower bank appears to be much steeper for a few meters immediately above the regulated summer river level (RSRL).

Remediation

The site was the recipient of stabilization works which were completed in April of 2004 which consisted of a lower bank key trench and upper bank rockfill columns. It was reported that the granular backfill was not vibro-compacted.

As can be seen from the plan map of this site, the stabilization works were installed around existing infrastructure. The shear key was installed at the base of the steep portion of the lower bank, just below the RSRL and the outfall pipe outlets. It is approximately 3.0 m wide at its base and keyed 0.3 m into the till. The trench was excavated with sidewalls kept as vertical as possible under site conditions, but are depicted at an angle of 0.3H:1V for site drawings. Measured from site drawings, an approximately 0.5 m thick rockfill riprap blanket was placed over the shear key and extends about 8 m in the direction of the river. Riprap was also placed on the steep portion of the lower bank to regrade it to a slope of 1.5H:1V.

The rockfill columns were 2.1 m in diameter and are depicted in site drawings as being keyed into the till further than the shear key, but it is not specified how deep exactly. The rockfill columns are located immediately upslope of the top of the bank. A total of 6 columns were installed at 1 Evergreen Place, and 3 more were installed along the riverbank crest between 1 Evergreen Place and 7 Evergreen Place.

While the described repairs are typical of river bank stabilization projects, it is less common to have both installed together. It appears the rockfill columns were used to provide additional stability since it is solely the shear key that spans the entire site.

Performance of Repairs

Data from a 9.8-year long monitoring period was available for this site. Three slope inclinometers were installed, with one at each of 1, 7 & 11 Evergreen Place. SI-01, located downslope and north-northwest of 1 Evergreen Place, monitored between 224-225 masl and recorded 2.5 mm of displacement over the monitoring period. An average displacement rate of 0.62 mm/year was measured in the first year after remediation, followed by an average rate of 0.34 mm/year from Year 1-3, and an average rate of 0.11 mm/year from 3 years until the end of the monitoring period.

SI-02 was located downslope and north-northwest of 7 Evergreen Place and monitored between 224-225 masl. No displacement was observed in this slope inclinometer over the entire monitoring period.

SI-03 was installed downslope and approximately equidistant from 7 and 11 Evergreen Place. A total of 4 mm of displacement was measured in the zone of monitoring for this inclinometer, which was located between 226-227 masl. As with the trend observed in SI-01, the rate of displacement was found to decrease over the monitoring period. In the first year, an average rate of displacement of 1.24 mm/year was observed, followed by an average rate of 0.68 mm/year for the 1st to 3rd year, and then an average rate of 0.22 mm/year for the remainder of the monitoring period.

Flexible CSP-type pipes exist on site, with outlets at about 224 masl. The reduction in the rate of displacement over the monitoring period can be viewed as a success. The total displacement observed over that period can also be viewed positively, as movement remained minimal. SI-02 was located closest to the rockfill columns and experienced no movement.

Lessons Learned

Based on the slope inclinometer data, it appears the combined rockfill column-shear key system proved to be more successful than a shear key alone. However, the shear key on its own still appears to have performed well assuming the rate of movement across the entire slope was roughly equal prior to remediation. Whether the slightly better performance of the combined system can justify the cost should be considered for future similar projects.

It is very possible due to the presence of the thin lacustrine clay layer immediately above the till that deeper seated movement had been observed before remediation. In this case, the shear key would be well-justified. The columns may have been selected to slow surface movement. The reasoning can only be speculated upon with the data that was obtained.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:		*Landslide velocity class:*	1 - Extremely Slow
Depth of movement (m):	2-3	*Most recent rate (mm/yr):*	0.11-0.22
Sheared material:	Interbedded sand, silt and clay (alluvial origin)		

Landslide Dimensions

Width (m):	150 (?)	*Length (m):*	35 (?)
Height (m):	2.5 (?)	*Slope (°):*	16

Monitoring Information

Movement first reported:		*Last inspected:*	January 16, 2014
SIs installed:	3	*SIs active last inspection:*	3
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):		*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	River

Repair Specifications

Repair type:	Shear key and columns	*Repair date:*	January 30, 2004
Base width (m):	3	*Trench slope ratio (H:V):*	0.3:1 or more
Length (m):	145	*Drainage (yes/no):*	No
Granular height (m):	5	*Backfill:*	Unknown
Overburden (m):	0.5		
Overburden material(s):	Rockfill rip rap		

Additional Site Information

Stratigraphy

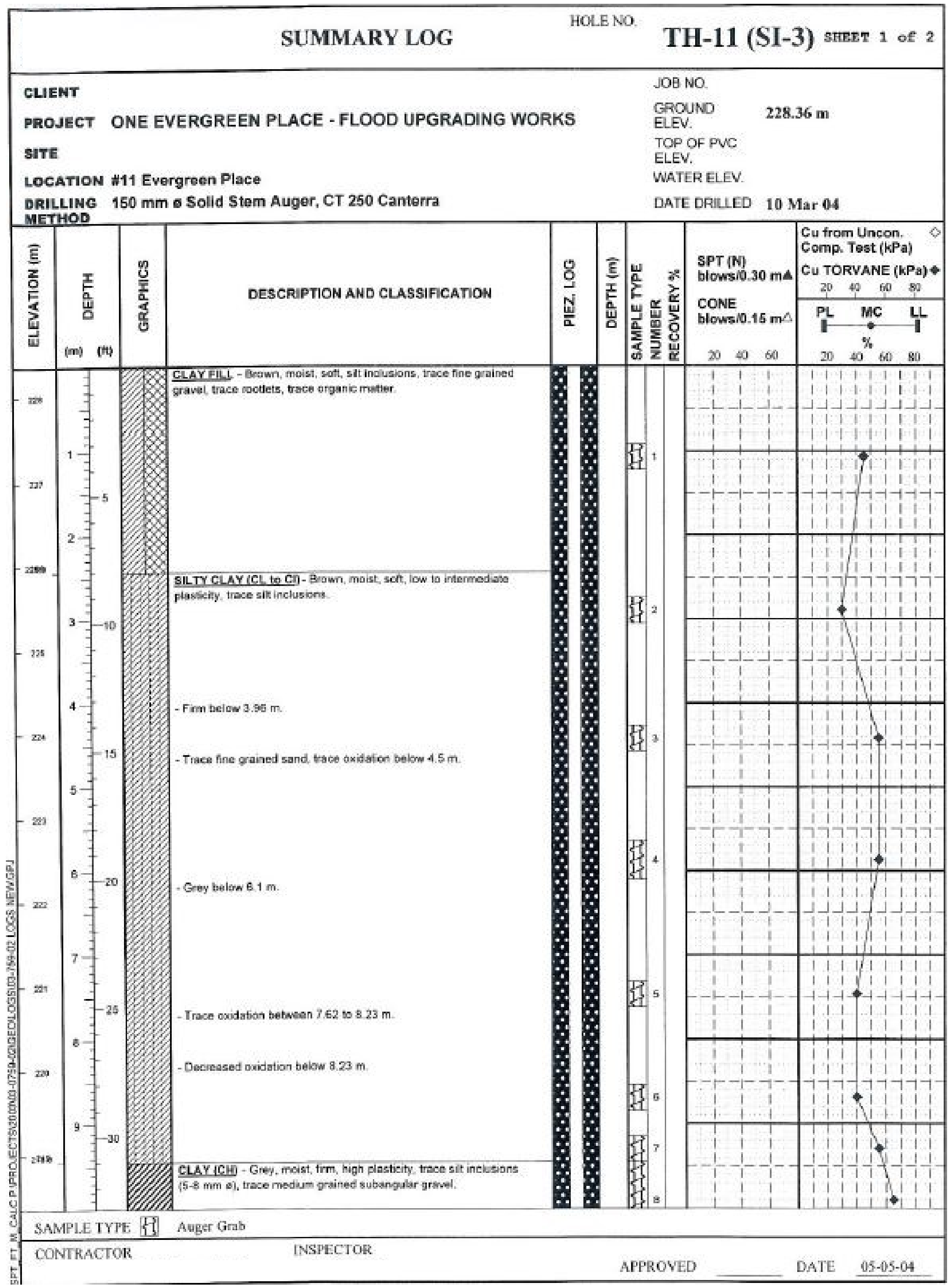
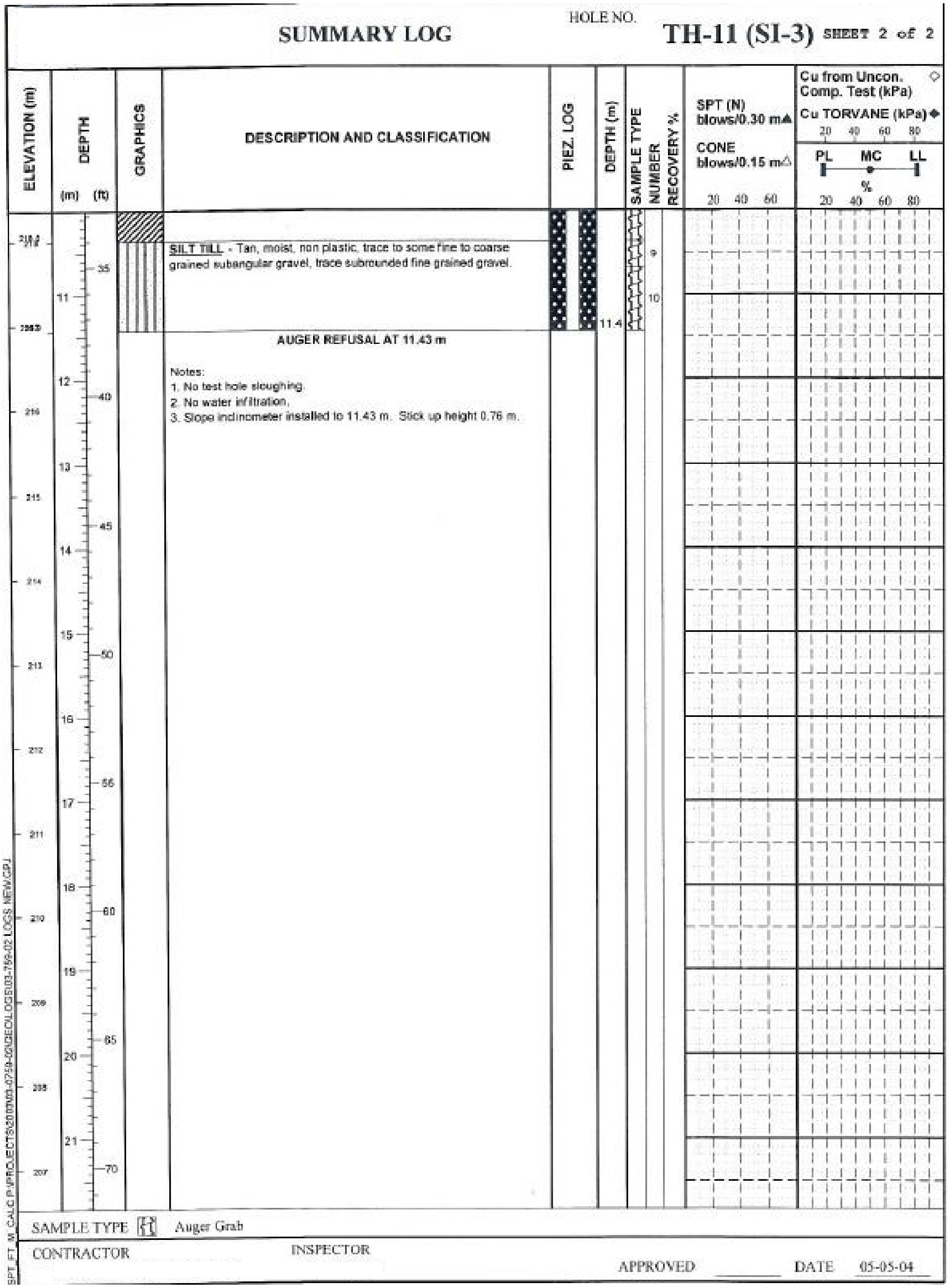


Figure A-35: Borehole log (Part 1/2) for WP1. Reproduced from KGS Group (2015), with permission.



SUMMARY LOG

HOLE NO. TH-11 (SI-3) SHEET 2 of 2

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	PIEZ. LOG	DEPTH (m)	SAMPLE TYPE	NUMBER	RECOVERY %	SPT (N) blows/0.30 m▲ CONE blows/0.15 m△	Cu from Uncon. Comp. Test (kPa) ◇ Cu TORVANE (kPa) ◆ PL MC LL %
			SILT TILL - Tan, moist, non plastic, trace to some fine to coarse grained subangular gravel, trace subrounded fine grained gravel.				9			
					11.4		10			
			AUGER REFUSAL AT 11.43 m							

Notes:
1. No test hole sloughing.
2. No water infiltration.
3. Slope inclinometer installed to 11.43 m. Stick up height 0.76 m.

SAMPLE TYPE Auger Grab

CONTRACTOR INSPECTOR APPROVED DATE 05-05-04

SPT_FT_M_CALC P:\PROJECTS\2003\03-0759-00\GEOLOGS\03-759-02 LOGS NEW.GPJ

Figure A-36: Borehole log (Part 2/2) for WP1. Reproduced from KGS Group (2015), with permission.

Cross Section

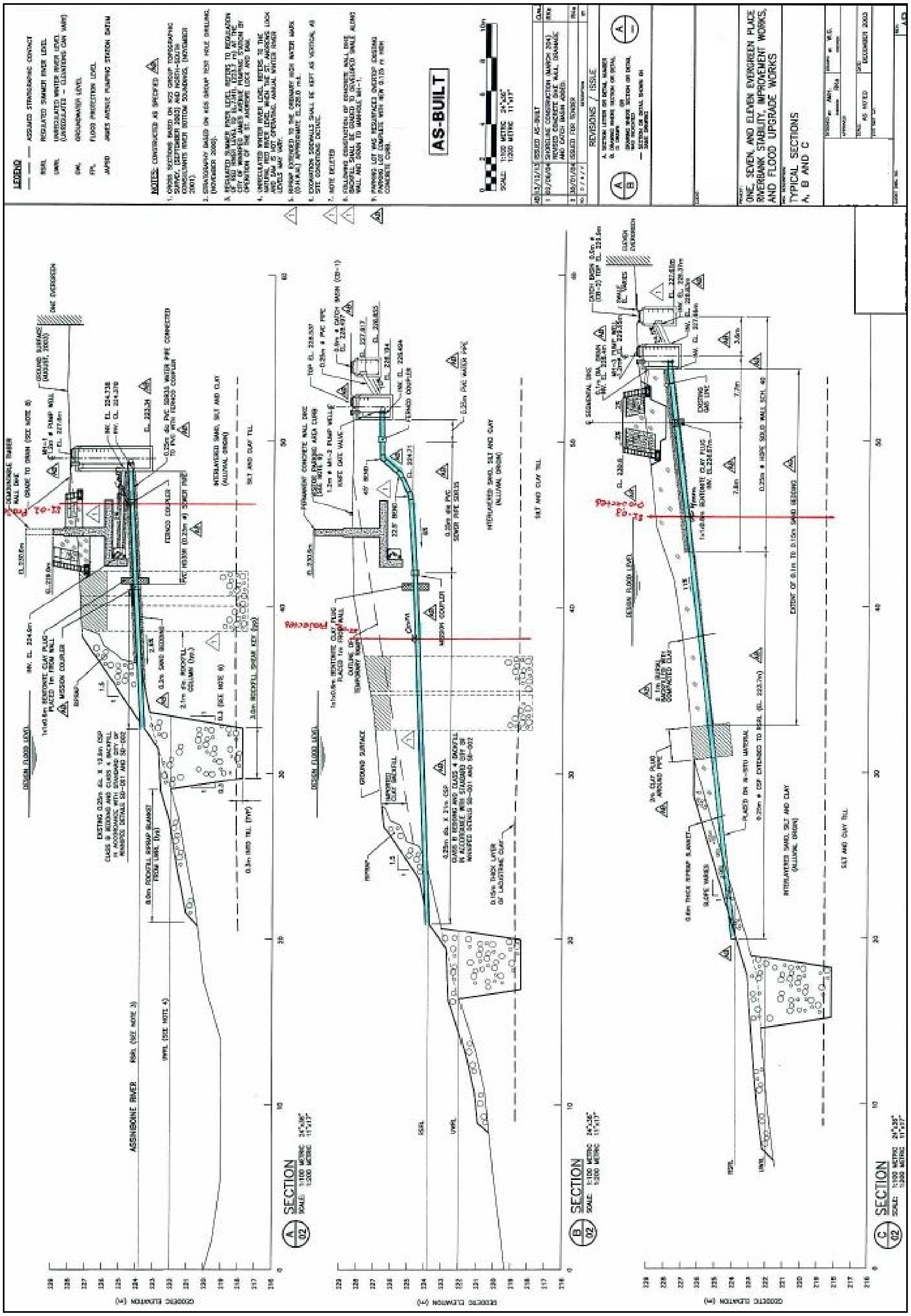


Figure A-37: Cross sections for WP1. Reproduced from KGS Group (2015), with permission.

Plan Map

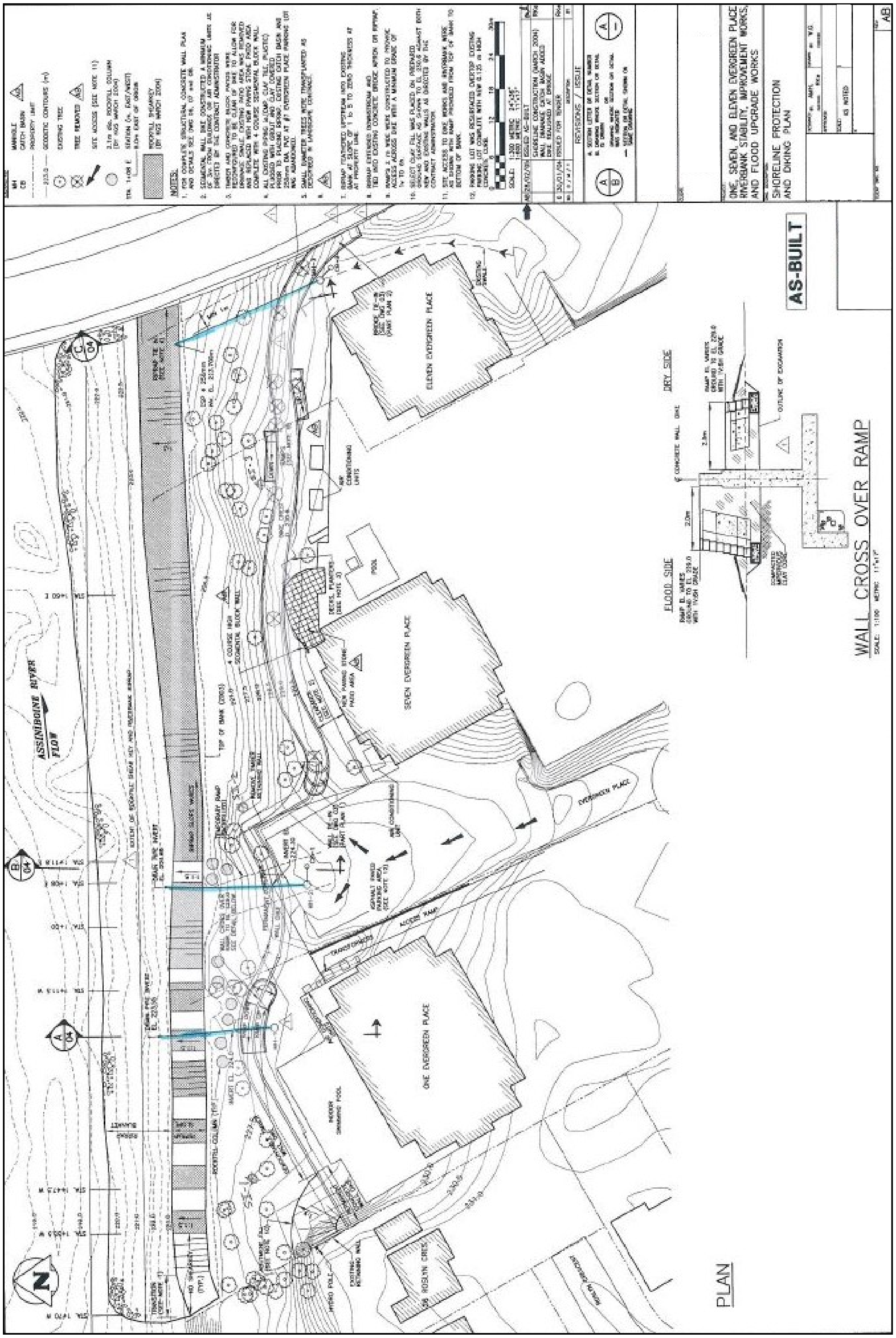


Figure A-38: Plan map for WP1. Reproduced from KGS Group (2015), with permission.

WP2 – *99 and 141 Wellington Crescent*

Winnipeg, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by KGS Group in *Outfall Pipe Performance along Stabilized Riverbanks - Preliminary Assessment of Existing Sites in the City of Winnipeg* (2015).

Background

The site is located along the riverbank adjacent to the Assiniboine River, next to buildings at 99 and 141 Wellington Crescent in Winnipeg, Manitoba. The riverbank here has an overall slope of 3.6H:1V and generally consists of alluvial sediments deposited over lacustrine deposits. A borehole log from the top of the riverbank, for SI-1, identifies clay fill in the upper 2 m. This is followed by 1 m of silty clay and then lacustrine clay to a depth of 7.5 m. Silty clay till was identified from 7.5-9.1 mbgl, below which silt till was encountered until refusal at 10.7 mbgl. There is also an outfall pipe at this site, for which the outlet invert is at an elevation of 223 masl.

Remediation

Remediation of the riverbank took place in April of 2004 and consisted of installing rockfill columns along the lower bank. The columns were reported to have been vibro-compacted. Examination of the plan maps available for this site shows extensive remediation was completed before the work in April 2004 too. Four rows of pre-existing rockfill columns are next to 99 Wellington Crescent, three rows of pre-existing rockfill columns and one pre-existing trenched rockfill shear key are next to 141 Wellington Crescent, and one pre-existing trenched rockfill shear keys is in line with 155 Wellington Crescent.

In the April 2004 remediation, two rows of rockfill columns were constructed immediately upslope of the trenched rockfill shear key aligned with 141 Wellington Crescent. Two more rows were constructed about 10 m upslope of those, located approximately midway up the slope. These two rows upslope were continued north such that they pass upslope of the three pre-existing rows of rockfill columns mentioned previously. They were then extended north past these pre-existing columns by about 15 m. Near the south side of 99 Wellington Crescent, four rows of rockfill columns were constructed along the edge of the water, filling the gap between the pre-existing columns next to this building and the pre-existing columns next to 141 Wellington Crescent. This is illustrated in the plan map included with this case study summary.

Performance of Repairs

The site was part of a preliminary study that was conducted to review and assess the measured performance of outfall pipes located on slopes that had been stabilized using trenched rockfill shear keys or rockfill columns in Winnipeg. The monitoring zones indicated below pertain to the zones of interest in the location of the slope inclinometers in the context of the outfall pipe performance study.

Four slope inclinometers were installed on the site and used to monitor post-construction slope deformations. Three of them, labeled SI-01 to SI-03, were monitored for a period of 4.7 years, while the last, SI-04, was monitored for 9.8 years. A summary for the four slope inclinometers is given below (Table A-5). It should be noted that the total displacement and the displacement rates are for the monitoring zones that are indicated.

Table A-5: Summary of the movement data corresponding to the slope inclinometers at WP2.

Slope inclinometer	Monitoring zone (masl)	Monitoring period (yrs)	Total displacement (mm)	Rate for Year 0-1 (mm/yr)	Rate for Year 1-3 (mm/yr)	Rate for Year 3+ (mm/yr)
SI-01	224-225	4.7	4	0.80	0.44	0.21
SI-02	222-223	4.7	5	1.72	0.94	0.46
SI-03	222-223	4.7	5	1.32	0.73	0.35
SI-04	221-222	9.8	18	4.40	2.41	0.77

From the study results, the rate of deformation can be seen decreasing over time. It can also be seen that displacement rates were relatively low for the entire monitoring period, with a maximum rate of 4.40 mm/yr and a maximum total displacement of 18 mm after almost 10 years.

Lessons Learned

Extensive repairs were performed on this slope in April 2004, in addition to those already present. This could suggest the pre-existing repairs may not have been performing adequately. Alternatively, the repairs may have been necessitated by a change in flood preparedness requirements. Without SI data from before April 2004, this cannot be verified.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:	Toe erosion	*Landslide velocity class:*	1 - Extremely Slow
Depth of movement (m):	1-3, 8	*Most recent rate (mm/yr):*	0.21-0.77
Sheared material:	Alluvial clay and Lacustrine clay		

Landslide Dimensions

Width (m):	180	*Length (m):*	38
Height (m):	6.7-10	*Slope (°):*	15-18

Monitoring Information

Movement first reported:	1985-93	*Last inspected:*	January 20, 2014
SIs installed:	7	*SIs active last inspection:*	4-7
Piezos installed:	1	*Piezos active last inspection:*	0-1

Groundwater Information

Groundwater level (mbgl):		*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	River

Shear Key / Column Specifications

Repair type:	Shear key & Columns	*Repair date:*	April 2004
Base width / Diameter (m):	3.2 / 2.1	*Trench slope ratio (H:V):*	~1:6
Length (m):	29.3 & 31.1, 180	*Drainage (yes/no):*	No
Granular height (m):	6, 5.3-10.6	*Backfill:*	Crushed rockfill
Overburden (m):	0.6, 1.2		
Overburden material(s):	Rip rap (shear key) and clay fill cap (columns)		

Additional Site Information

Stratigraphy

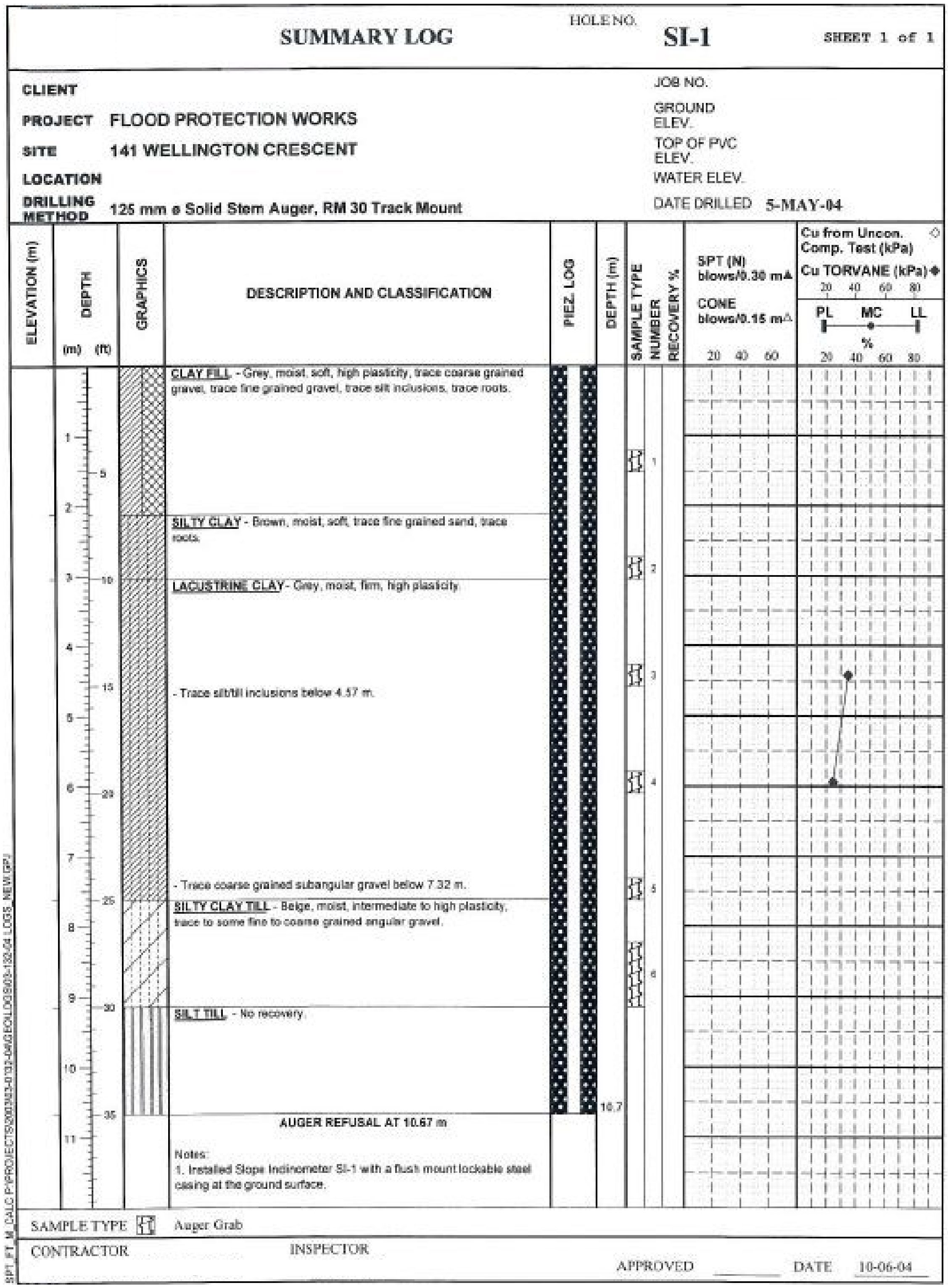


Figure A-39: Borehole log from WP2. Reproduced from KGS Group (2015)*, with permission.*

Cross Section

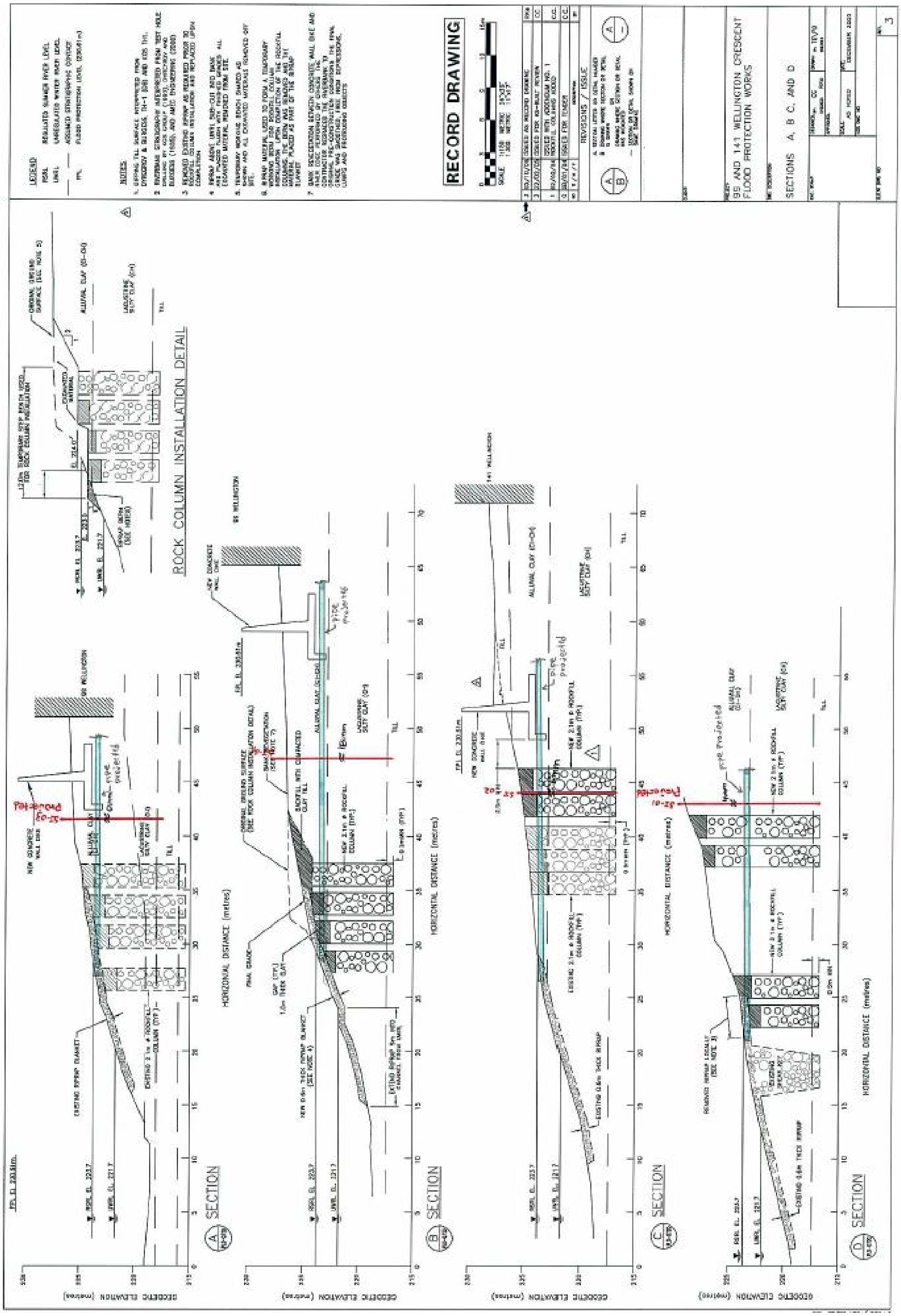


Figure A-40: Cross sections for WP2. Reproduced from KGS Group (2015)*, with permission.*

Plan Map

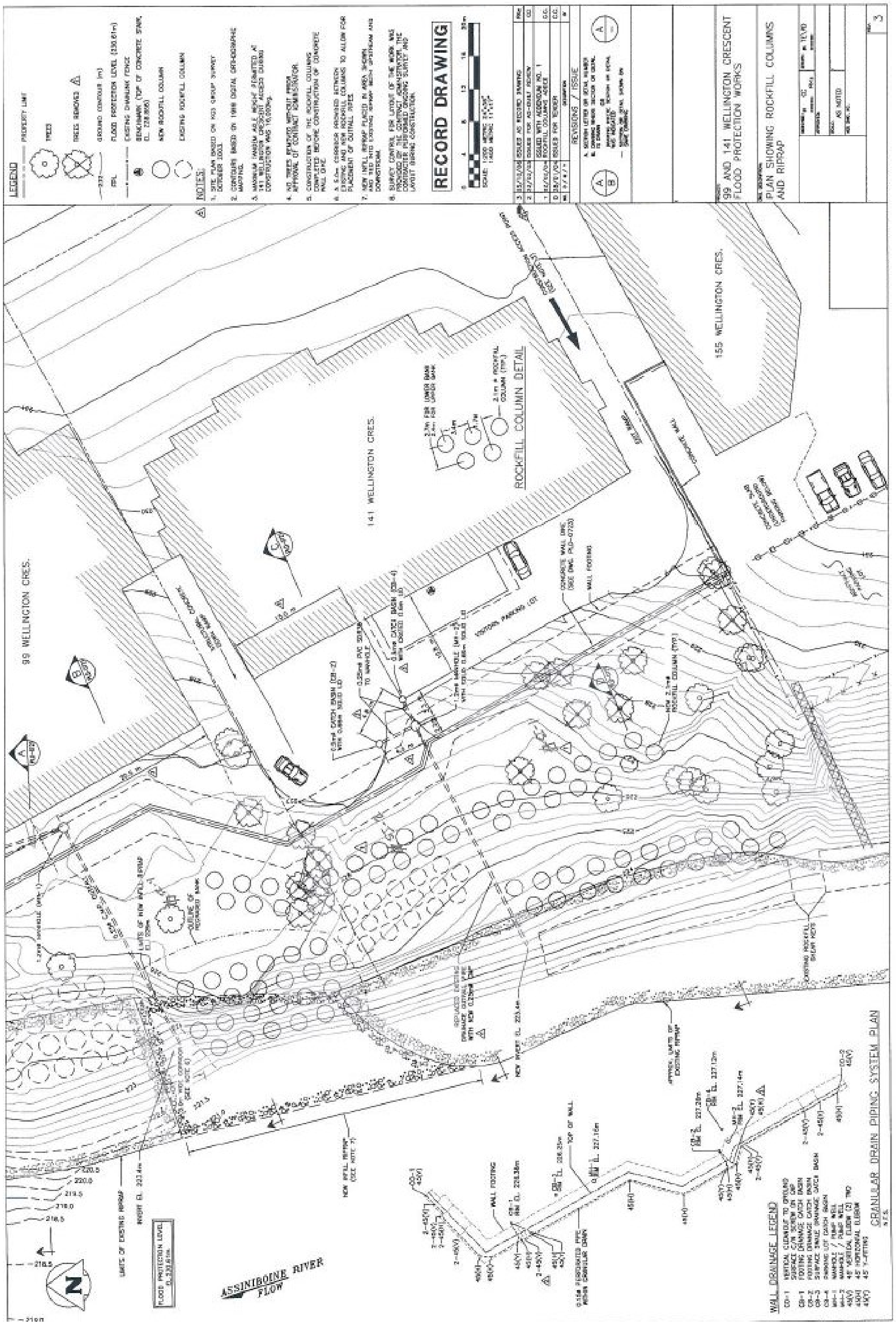


Figure A-41: Plan map for WP2. Reproduced from KGS Group (2015)*, with permission.*

WP3 – *Avenue de la Cathedrale Outfall*

Winnipeg, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by KGS Group in *Outfall Pipe Performance along Stabilized Riverbanks - Preliminary Assessment of Existing Sites in the City of Winnipeg* (2015).

Background

Avenue de la Cathedrale Outfall is along the Red River in Winnipeg, Manitoba. The slope at this site is composed of silty clay fill clayey silt fill overlying lacustrine clay, followed by till. A detailed log is included later in this case study summary. The slope is inclined at 5.4H:1V overall, with the till being identified at a depth of 13.1 m from the top of the riverbank. There is a pipe outlet at 222 masl at this site. Monitoring reports were focused on this pipe.

Remediation

Two rows of columns were installed in both the upper and lower bank. Reports suggest the project was completed either in March 2009 or on October 30, 2009; the correct date was not verified. Crushed rockfill was used as backfill, and it was vibro-compacted upon being placed. The columns were capped with 0.6 m of clay. Those in the lower bank were also topped with a 0.6-m-thick rip rap blanket to protect from the river.

Performance of Repairs

Two SIs were monitored at this site. The first, SI-01, was monitored for 2.1 years after construction. Monitoring was focused on 226-227 masl, for which 4 mm of movement was recorded. The rate in the first year was 2.63 mm/year, which then decreased to 1.75 mm/year for the period between 1 and 3 years after construction.

The second, SI-03, was monitored for 3.6 years. The monitoring zone was between 222-223 masl, in which 12 mm of movement was recorded. The rate of movement in the first year was 10.11 mm/year, followed by 5.55 mm/year for Year 1-3, and 3.29 mm/year for Year 3+. While these rates are greater than some similar sites in Winnipeg, it is difficult to comment on performance without a baseline from before construction.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:	Toe erosion	*Landslide velocity class:*	1 - Extremely Slow
Depth of movement (m):	7	*Most recent rate (mm/yr):*	3.29
Sheared material:	Clay fill, Lacustrine silty clay		

Landslide Dimensions

Width (m):	65	*Length (m):*	58
Height (m):	9	*Slope (°):*	10

Monitoring Information

Movement first reported:		*Last inspected:*	October 22, 2012
SIs installed:	2	*SIs active last inspection:*	1
Piezos installed:	1	*Piezos active last inspection:*	

Groundwater Information

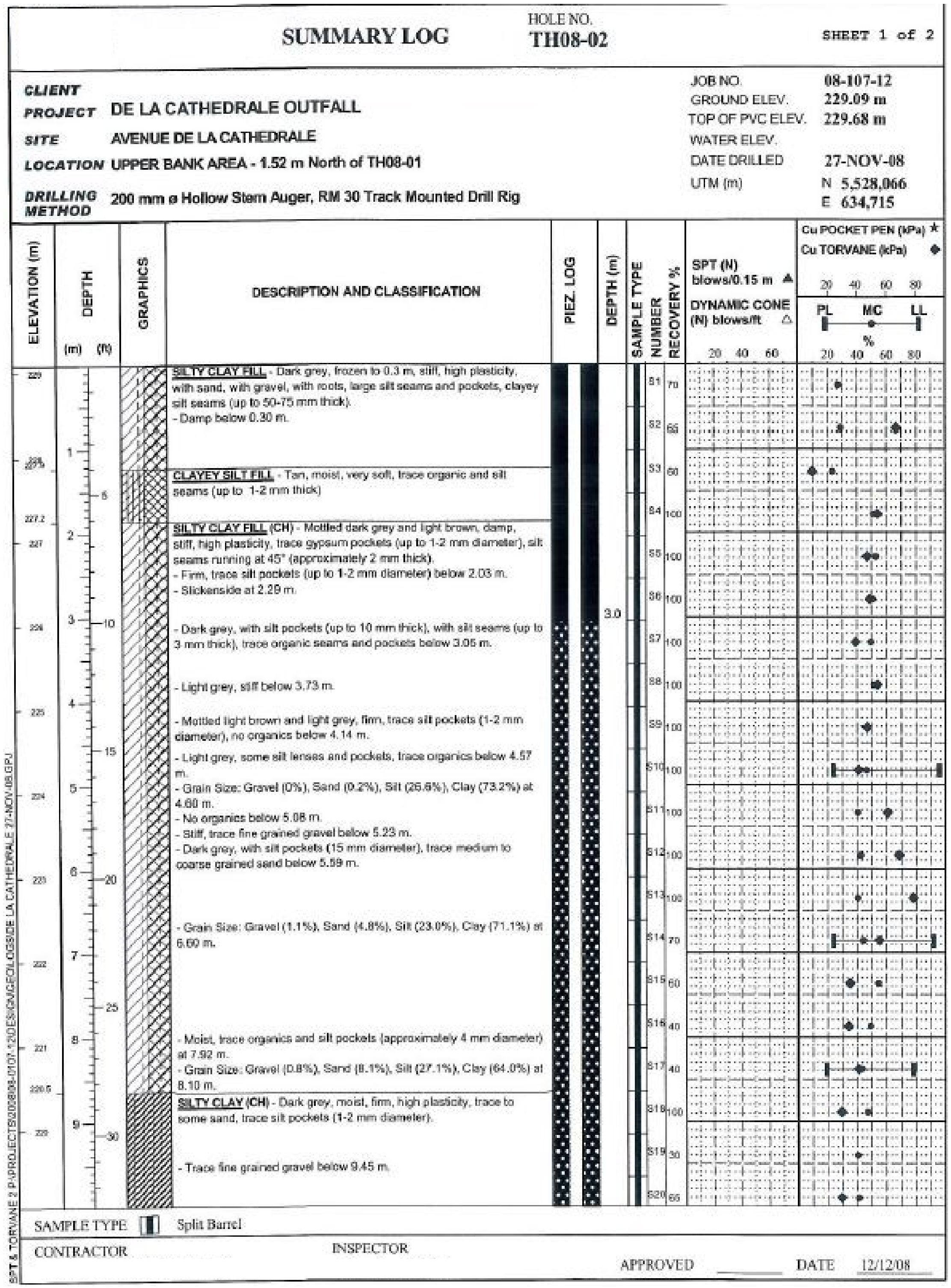
Groundwater level (mbgl):	0.3 (TH08-03)	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	River

Repair Specifications

Repair type:	Columns	*Repair date:*	December 16, 2008
Column diam. (m):	2.1	*Rows:*	2 and 2
Length (m):	~65	*Drainage (yes/no):*	No
Layout:	Triangle	*Spacing (up x across [m]):*	2.3 x 3.1
Granular height (m):	5-10.3	*Backfill:*	Crushed rockfill
Overburden (m):	0.6		
Overburden material(s):	Impervious clay cap		

Additional Site Information

Stratigraphy



SUMMARY LOG

HOLE NO. TH08-02

SHEET 1 of 2

CLIENT
PROJECT DE LA CATHEDRALE OUTFALL
SITE AVENUE DE LA CATHEDRALE
LOCATION UPPER BANK AREA - 1.52 m North of TH08-01
DRILLING METHOD 200 mm ø Hollow Stem Auger, RM 30 Track Mounted Drill Rig

JOB NO. 08-107-12
GROUND ELEV. 229.09 m
TOP OF PVC ELEV. 229.68 m
WATER ELEV.
DATE DRILLED 27-NOV-08
UTM (m) N 5,528,066 E 634,715

ELEVATION (m) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.15 m ▲ DYNAMIC CONE (N) blows/ft △ 20 40 60 | Cu POCKET PEN (kPa) ★ Cu TORVANE (kPa) ◆ 20 40 60 80 PL MC LL % 20 40 60 80

SILTY CLAY FILL - Dark grey, frozen to 0.3 m, stiff, high plasticity, with sand, with gravel, with roots, large silt seams and pockets, clayey silt seams (up to 50-75 mm thick).
- Damp below 0.30 m.

CLAYEY SILT FILL - Tan, moist, very soft, trace organic and silt seams (up to 1-2 mm thick)

SILTY CLAY FILL (CH) - Mottled dark grey and light brown, damp, stiff, high plasticity, trace gypsum pockets (up to 1-2 mm diameter), silt seams running at 45° (approximately 2 mm thick).
- Firm, trace silt pockets (up to 1-2 mm diameter) below 2.03 m.
- Slickenside at 2.29 m.
- Dark grey, with silt pockets (up to 10 mm thick), with silt seams (up to 3 mm thick), trace organic seams and pockets below 3.05 m.
- Light grey, stiff below 3.73 m.
- Mottled light brown and light grey, firm, trace silt pockets (1-2 mm diameter), no organics below 4.14 m.
- Light grey, some silt lenses and pockets, trace organics below 4.57 m.
- Grain Size: Gravel (0%), Sand (0.2%), Silt (26.6%), Clay (73.2%) at 4.60 m.
- No organics below 5.08 m.
- Stiff, trace fine grained gravel below 5.23 m.
- Dark grey, with silt pockets (15 mm diameter), trace medium to coarse grained sand below 5.59 m.
- Grain Size: Gravel (1.1%), Sand (4.8%), Silt (23.0%), Clay (71.1%) at 6.60 m.
- Moist, trace organics and silt pockets (approximately 4 mm diameter) at 7.92 m.
- Grain Size: Gravel (0.8%), Sand (8.1%), Silt (27.1%), Clay (64.0%) at 8.10 m.

SILTY CLAY (CH) - Dark grey, moist, firm, high plasticity, trace to some sand, trace silt pockets (1-2 mm diameter).
- Trace fine grained gravel below 9.45 m.

Sample	Recovery %
S1	70
S2	65
S3	60
S4	100
S5	100
S6	100
S7	100
S8	100
S9	100
S10	100
S11	100
S12	100
S13	100
S14	70
S15	60
S16	40
S17	40
S18	100
S19	30
S20	65

SAMPLE TYPE Split Barrel
CONTRACTOR INSPECTOR APPROVED DATE 12/12/08

Figure A-42: Borehole log (Part 1/2) for WP3. Reproduced from KGS Group (2015)*, with permission.*

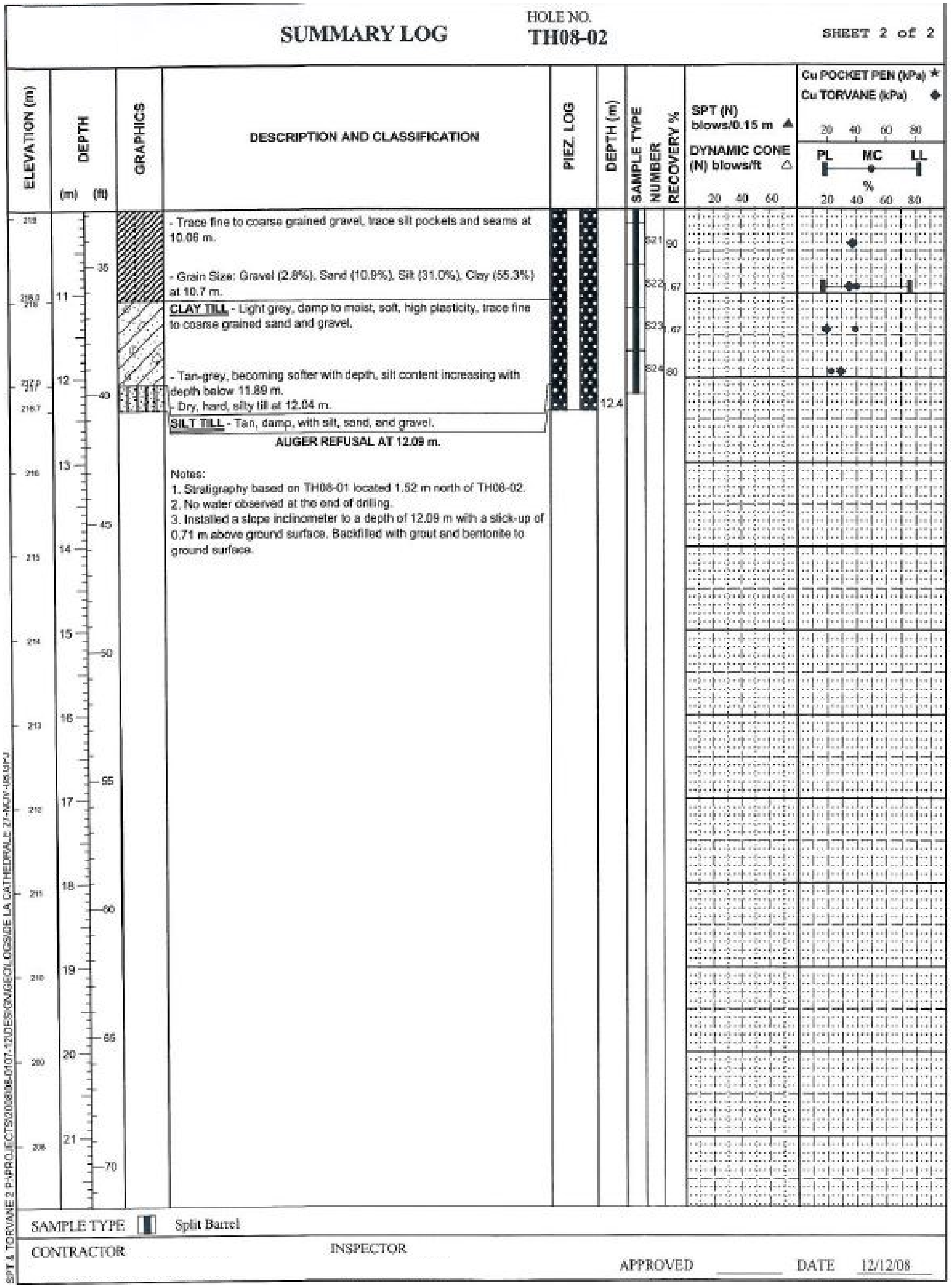


Figure A-43: Borehole log (Part 2/2) for WP3. Reproduced from KGS Group (2015)*, with permission.*

Cross Section

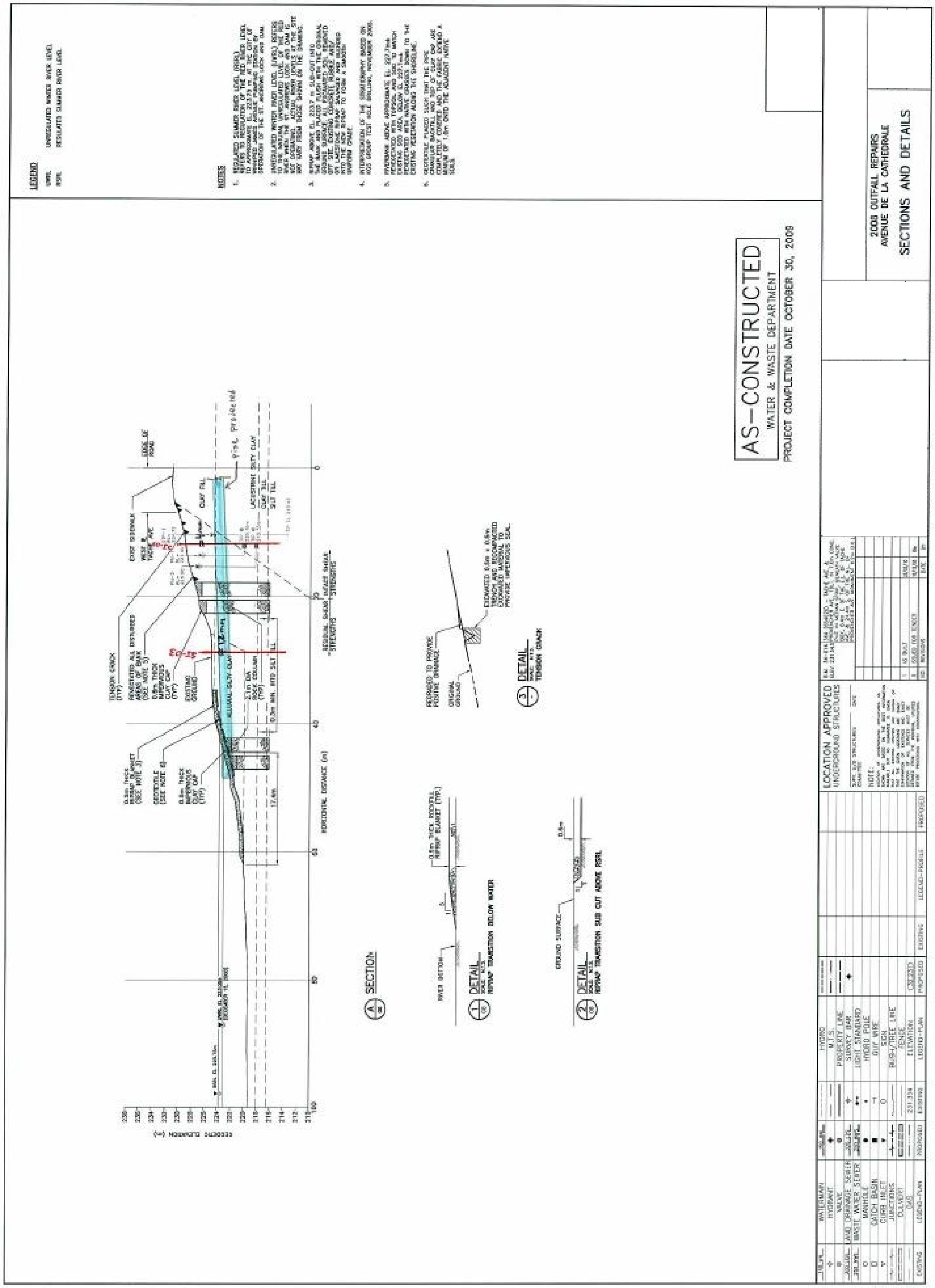


Figure A-44: Cross section for WP3. Reproduced from KGS Group (2015)*, with permission.*

Plan Map

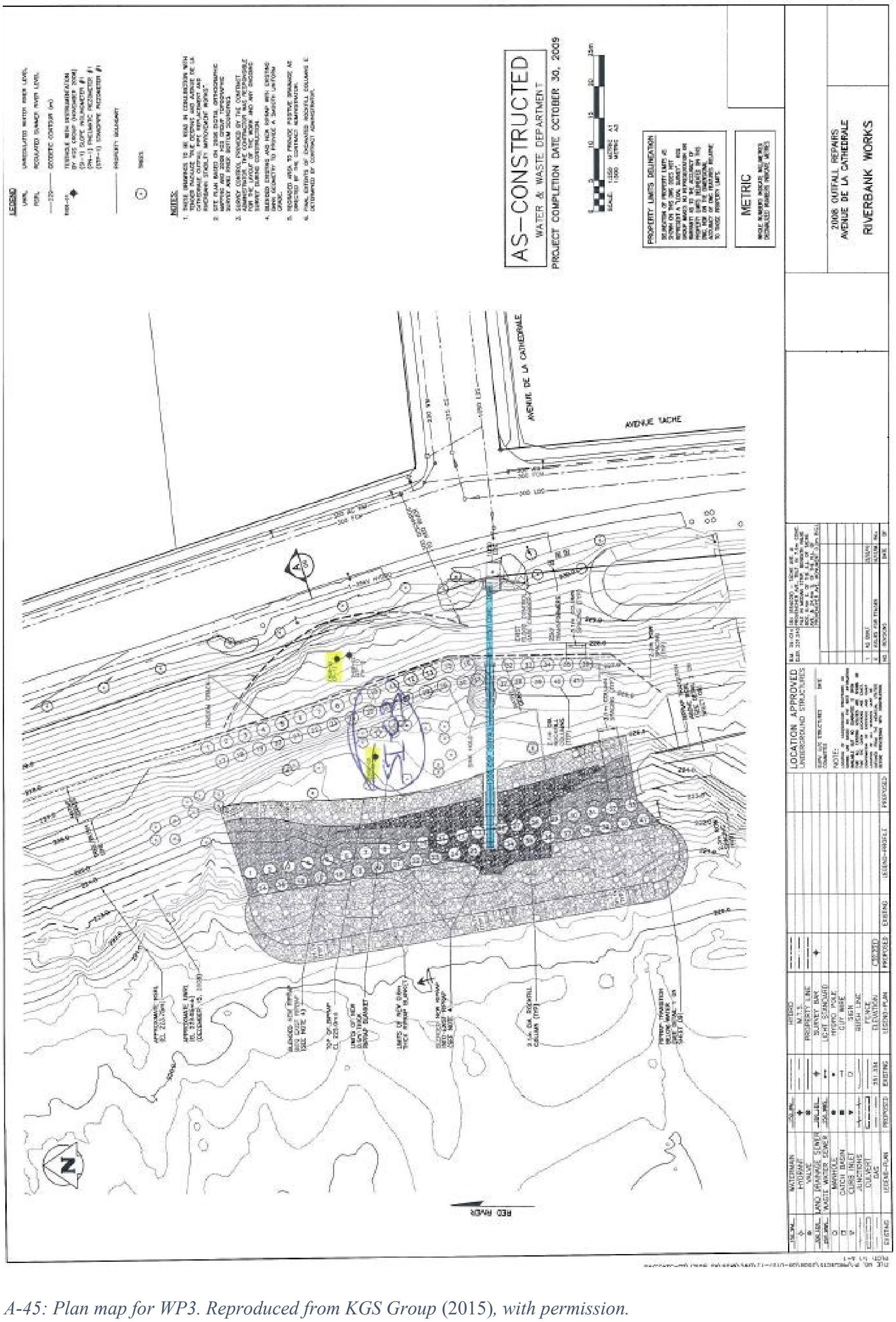


Figure A-45: Plan map for WP3. Reproduced from KGS Group (2015)*, with permission.*

WP4 – *Byng Place Outfall*

Near Riverside Drive and Calrossie Boulevard, Winnipeg, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by KGS Group in *Waterway Permit No. 111-2013, 46 Riverside Drive - Commonly Known as Byng Place Outfall in Toiler's Park, Riverbank Stabilization and Outfall Replacement - Letter of Completion* (2016).

Background

The Byng Place Outfall is in Toiler's Park in Winnipeg, Manitoba, along an outside bend of the Red River. The site was investigated in September and October of 2013 by KGS Group, in preparation for a new outfall gate chamber and pipe. The investigation did not reveal signs of slope instability. The slope at the location of the new outfall pipe was approximately 8H:1V, steepening to 2H:1V further upstream.

A drilling and soil sampling program was completed on October 4, 2013 by Paddock Drilling Ltd. and supervised by KGS Group. The stratigraphy interpreted by KGS Group consists of fill, followed by lacustrine silty clay (CH), then till. The fill ranges from 0.75-1.5 m in thickness and was dark brown, dry to damp, loose, and low to non-plastic. Some silty clay was identified in the fill, along with traces of limestone cobbles, glass, and organics.

Below the fill, silty clay was identified down to an elevation of 217.1-217.5 masl. The upper section of the lacustrine silty clay was oxidized and brown. The colour changed to grey around 225.0-227.0 masl. The clay was described as of high plasticity, moist, and firm but becoming softer with depth. The report describes silt pockets throughout this unit, with traces of sand and gravel at depth. A variety of tests were performed on the lacustrine clay. Estimates for the undrained shear strength using a field Torvane ranged from 15 kPa to 60 kPa, from bottom to top. The moisture content was measured ranging from 32- 58%. Atterberg Limit tests yielded plastic limits ranging from 15-22% and liquid limits ranging from 52-74%. The plasticity index ranged from 37-54%. The composition of the samples was as follows: 3-6% gravel, 6-14% sand, 24-35% silt, and 49-62% clay. The lacustrine clay was assigned $\varphi' = 14$ and $c' = 5$ kPa for stability analyses.

The deepest unit of the investigation was a silt till comprising 10-20% clay, sand and gravel. It was tan to grayish tan, damp to wet, loose to dense, and intermediate plasticity. The moisture content ranged from 9-16%.

Remediation

Remediation work was completed in July 2015. The remediation involved the construction of a rockfill trench shear key in two sections, and six rockfill columns arranged in two rows. The trench shear key was intended to improve the overall slope stability. The factor of safety was analyzed for two different sections (A and B), indicated on the plan map. The factor of safety for normal site conditions was estimated at 1.43 along Section A once the shear key was implemented. The rockfill columns were meant to improve stability in the upper bank, contributing to an estimated factor of safety of 1.36 overall and 1.66 in the upper bank, along Section B. These structures were completed November 7, 2014, per the plan map included in this summary. The trench shear key was 2 m wide at its base and the trench walls were excavated at 0.25H:1V. The trench was 30 m long and was keyed a minimum of 0.6 m into the underlying till. From south to north, the shear key base elevation went from 223.3 masl to 222.0 masl. The columns were 2.13 m in diameter and installed upslope of the southern section of the trench shear key. The columns were constructed with a 3.3 m center-to-center spacing, yielding a 1 m effective width. The trench and the columns were both backfilled with limestone rockfill which was vibro-compacted upon being placed. Both structures were then capped with 0.6 m of compacted clay.

Following the construction of the trench shear key and the rockfill columns, rockfill rip rap was placed along the riverbank. The rip rap blanket was 30 long, beginning about 3.5 m south of the southern extent of the trench shear key. The riverbank was covered starting at the rockfill columns and extending down the slope to approximately 10 m below the unregulated winter river level.

In addition to these repairs, the upper and middle bank areas were offloaded and regraded to improve bank stability and protect the outfall infrastructure.

Performance of Repairs

Data for one SI was obtained but the SI could not be located on the plan map. Without knowing the location of the SI, the performance of the repairs will not be discussed.

Two pneumatic piezometers were installed in the overburden clay and one standpipe piezometer was installed in the underlying till. The groundwater was measured on October 25, 2013. Within the clay, the groundwater level was between 225.9-226.9 masl, and within the till it was at 224.4 masl. A slight downward gradient from the clay to the till was interpreted from this data.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:		*Landslide velocity class:*	
Depth of movement (m):		*Most recent rate (mm/yr):*	
Sheared material:			

Landslide Dimensions

Width (m):		*Length (m):*	
Height (m):		*Slope (°):*	

Monitoring Information

Movement first reported:		*Last inspected:*	
SIs installed:	1	*SIs active last inspection:*	1
Piezos installed:	3	*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	0 – 2.1	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	No	*Surface water type:*	River

Repair Specifications

Repair type:	Shear key and columns	*Repair date:*	November 7, 2014
Base width / diameter (m):	2 / 2.13	*Trench slope ratio (H:V):*	1:4
Length (m):	30	*Drainage (yes/no):*	No
Granular height (m):	6.4	*Backfill:*	Vibro-compacted limestone rockfill
Overburden (m):	1.2		
Overburden material(s):	Compacted clay cap (0.6 m), rip rap (0.6 m)		

Additional Site Information

Cross section

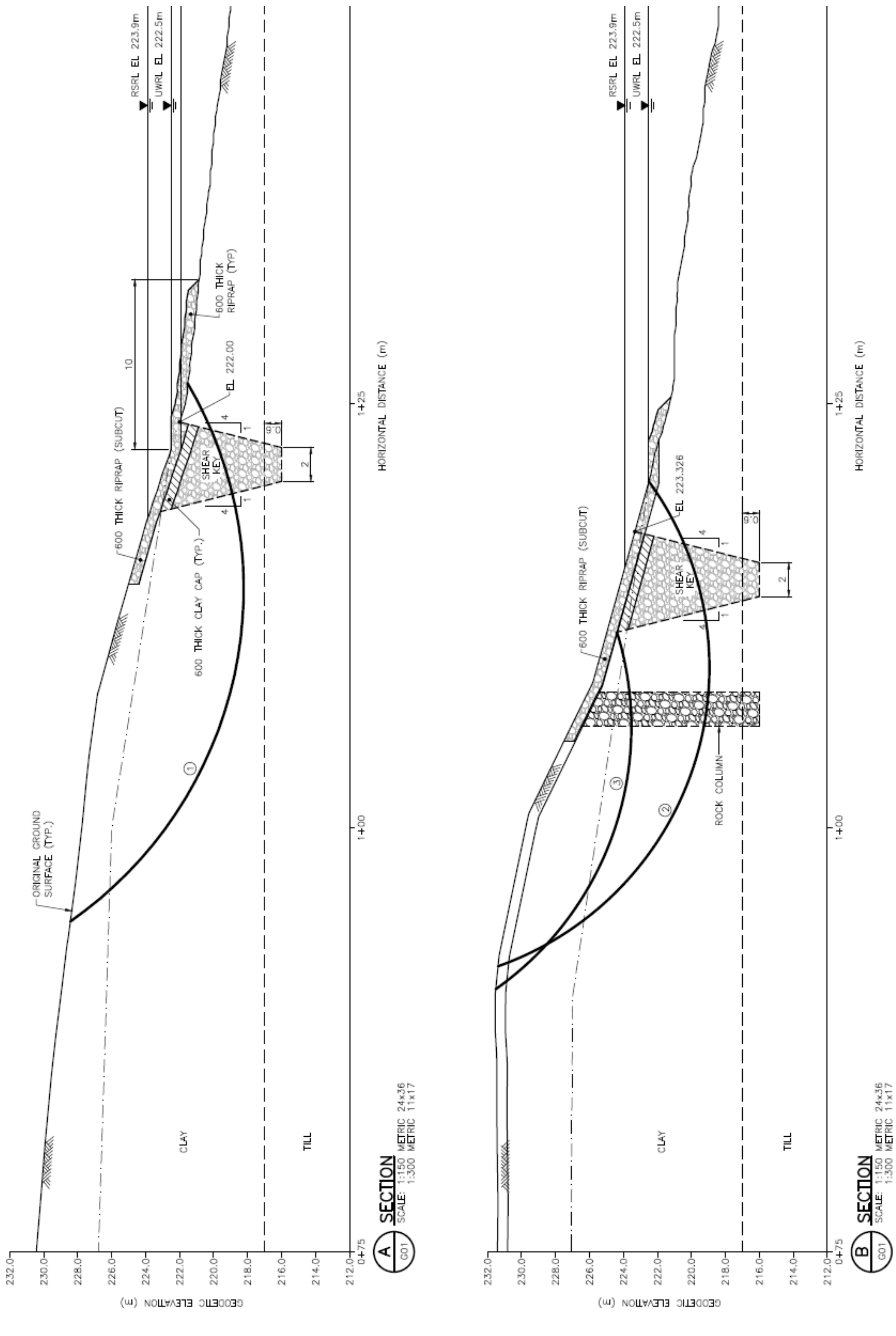


Figure A-46: Cross sections for WP4. Reproduced from KGS Group (2016)*, with permission.*

Plan map

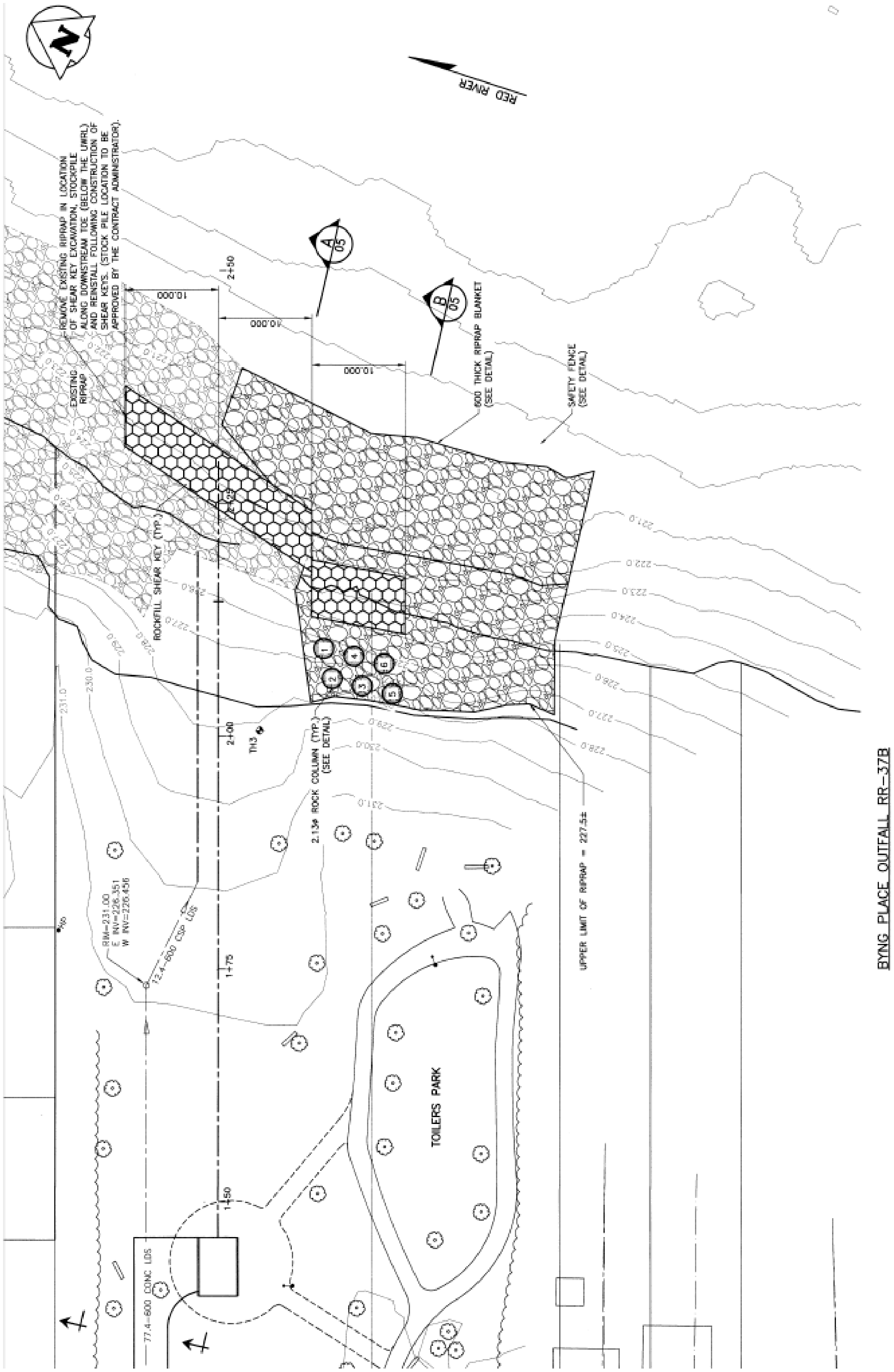


Figure A-47: Plan map for WP4. Reproduced from KGS Group (2016)*, with permission.*

WP5 – *Churchill Drive Park*

Winnipeg, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by KGS Group in *Outfall Pipe Performance along Stabilized Riverbanks - Preliminary Assessment of Existing Sites in the City of Winnipeg* (2015).

Background

Churchill Drive Park is located next to the Red River, in Winnipeg, Manitoba. The slope at this location is composed of lacustrine deposits. A borehole log for the site showed silty clay to a depth of about 2.75 m, underlain by 0.25 m of silt. This was followed by silty clay down to a depth of 9.7 m. Till is located below the silty clay. The slope was reported as being inclined at 4.4H:1V overall, and the till was measured at a depth of 14 m below the top of the bank.

Remediation

A trenched granular shear key was completed in the lower bank in March of 2012. The backfill was crushed rockfill and it was vibro-compacted upon being placed.

The shear key was keyed 0.6 m into the till, and was excavated with a base width of 3.5 m. The trench walls were noted to have been excavated as near to vertical as site conditions would allow, and are illustrated as being 0.25H:1V. The depth of the excavation was about 5.9 mbgl, and this was backfilled with an average of 4.7 m of rockfill. The shear key was topped with a 0.6-m-thick clay cap and 0.6-m-thick layer of rip rap, restoring the ground surface to the original grade.

Performance of Repairs

Movement data was reported for three SIs, each of which was monitored for 1.4 years following construction. SI-01 was located at the west end of the shear key and recorded 2 mm of movement. The rates reported for this SI were null. SI-02 was located at the east end of the shear key and 1 mm of movement was recorded. Rates of 0.71 mm/year and 0.16 mm/year were reported for the first year after construction, and the period afterward, respectively. SI-03 was located approximately halfway along the length of the shear key. No movement was reported for SI-03.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:	Toe erosion	*Landslide velocity class:*	1 - Extremely Slow
Depth of movement (m):	0, 5.6-6.8, 13.6-14.4	*Most recent rate (mm/yr):*	2.4
Sheared material:	Silty clay		

Landslide Dimensions

Width (m):	450	*Length (m):*	43
Height (m):	10.7-13.5	*Slope (°):*	12.8

Monitoring Information

Movement first reported:		*Last inspected:*	July 22, 2013
SIs installed:	5	*SIs active last inspection:*	3 (?)
Piezos installed:	10	*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	0	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	River

Repair Specifications

Repair type:	Shear key	*Repair date:*	March 2012
Base width (m):	3.5	*Trench slope ratio (H:V):*	0.25:1
Length (m):	450	*Drainage (yes/no):*	No
Granular height (m):	3.8-5.6	*Backfill:*	Crushed rockfill
Overburden (m):	1.2		
Overburden material(s):	Clay cap (0.6 m) and rip rap (0.6 m)		

Additional Site Information

Stratigraphy

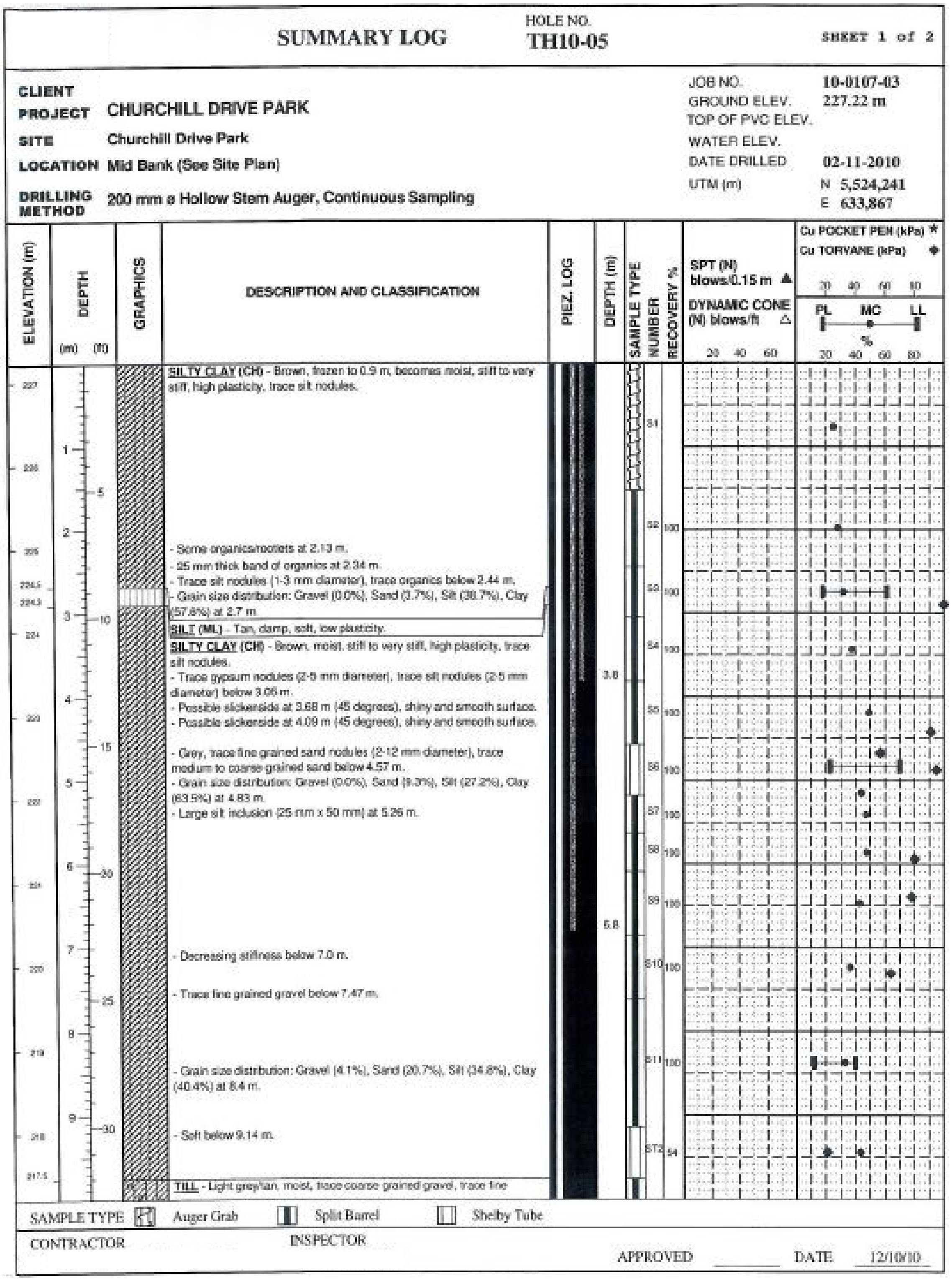


Figure A-48: Borehole log (Part 1/2). Reproduced from KGS Group (2015)*, with permission.*

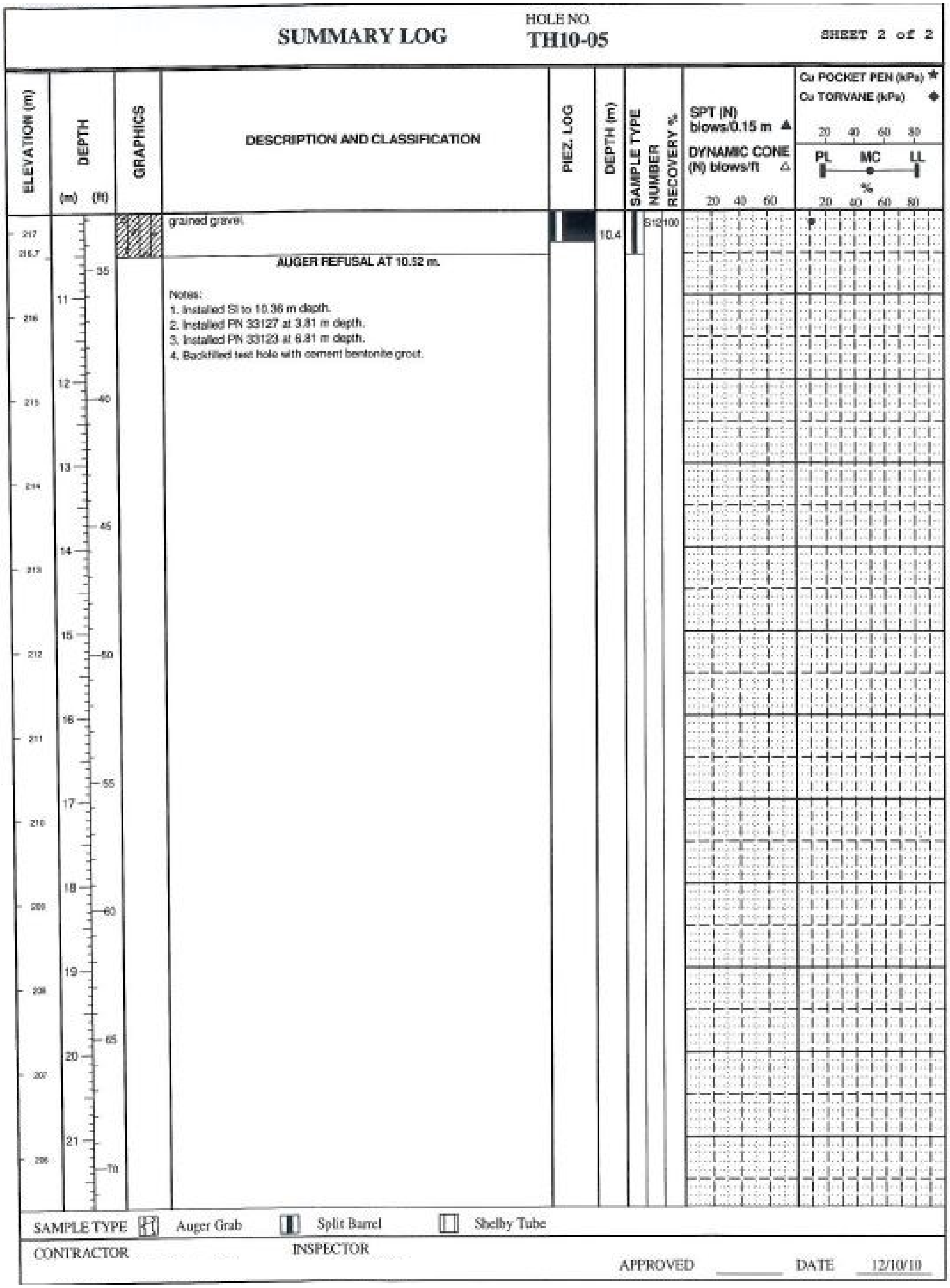


Figure A-49: Borehole log (Part 2/2). Reproduced from KGS Group (2015)*, with permission.*

Cross Section

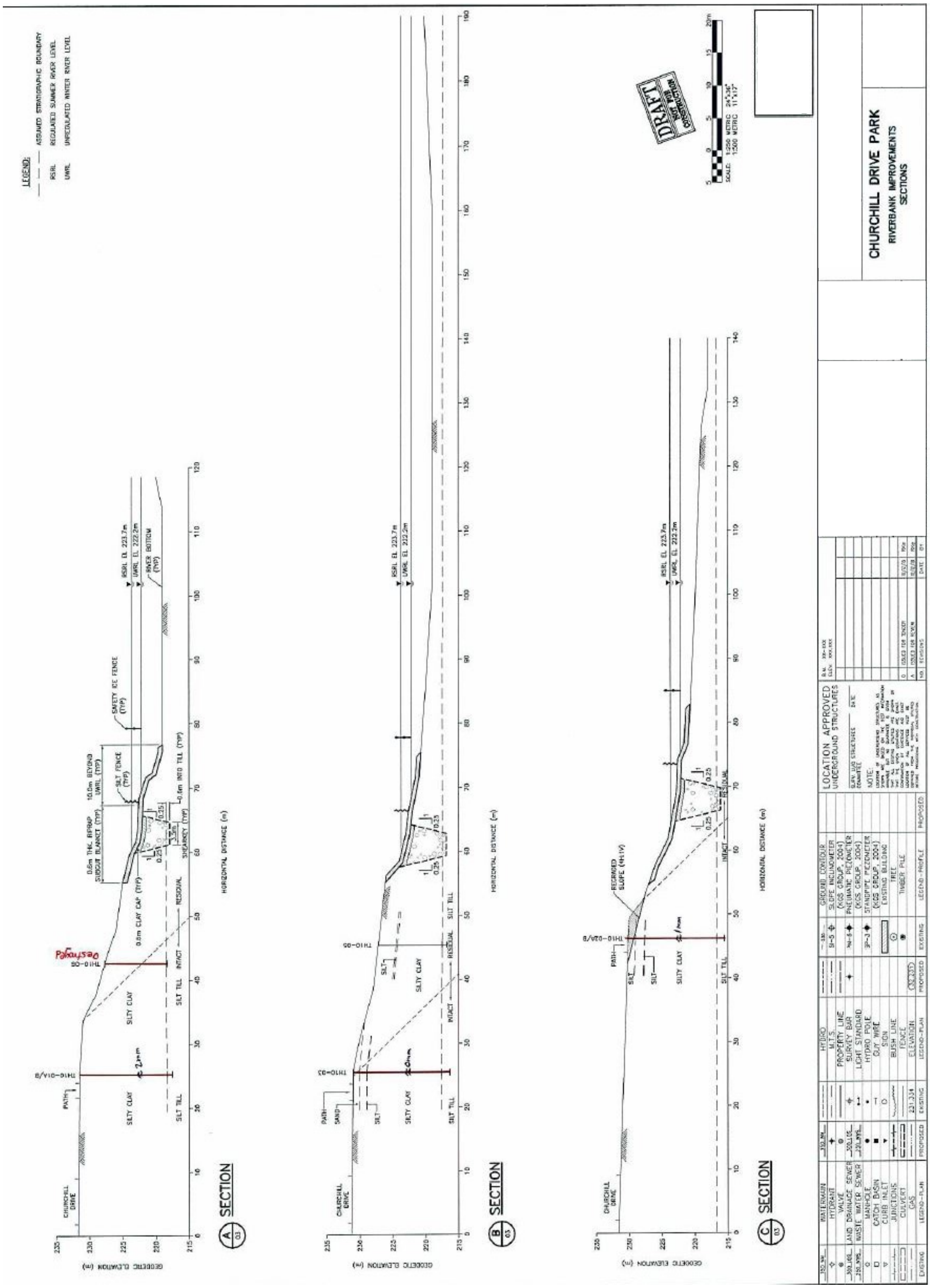


Figure A-50: Cross sections for WP5. Reproduced from KGS Group (2015)*, with permission.*

Plan Map

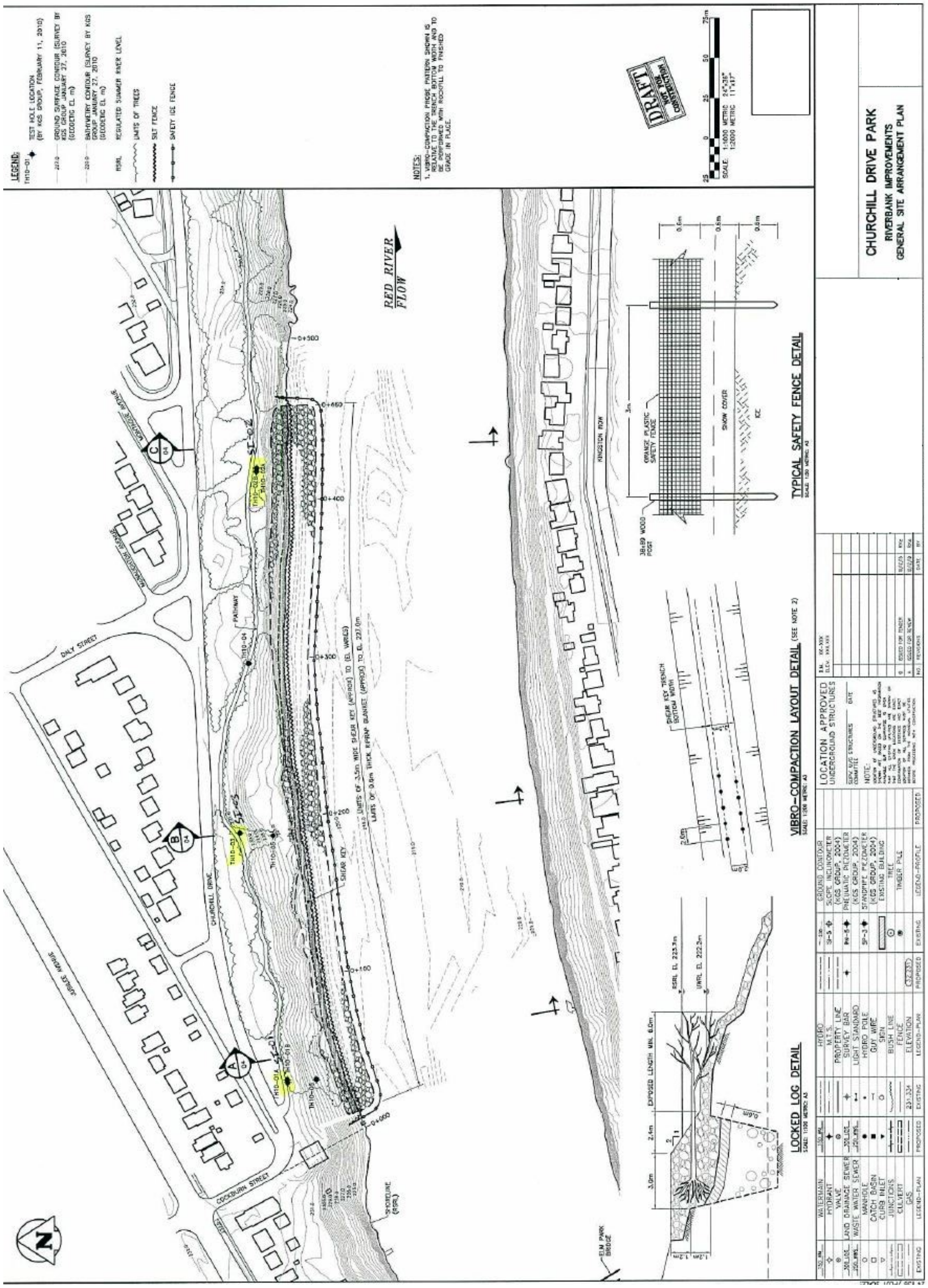


Figure A-51: Plan map for WP5. Reproduced from KGS Group (2015)*, with permission.*

WP6 – *Fort Rouge Park Outfall*

Winnipeg, MB

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by KGS Group in *Outfall Pipe Performance along Stabilized Riverbanks - Preliminary Assessment of Existing Sites in the City of Winnipeg* (2015).

Background

Fort Route Park is in Winnipeg, MB, along an outside bend of the Assiniboine River. In 2008, repairs were completed on the 2.4 m diameter outfall pipe located at the park. Riverbank stability improvements were also made at that time and included the construction of a granular shear key.

Prior to the remedial work, KGS Group assessed the recent history and performance of the riverbank. Air photo analysis revealed evidence of historic instabilities in the mid and upper banks of the slope. The upper bank was found to be creeping before any work began. The lower bank did not appear to have suffered any historic instabilities. However, in the short period preceding the outfall renewal significant movement in this part of the slope, measuring 51 mm over about 1.5 months and the failure of the outfall pipe necessitated stabilization efforts. The movement was classified as downslope creep movement, which was interpreted as being the equivalent of soil slope deformation in the Varnes classification.

The site stratigraphy consists of high plasticity clays which overly silt till. The lower bank clays were classified as being of alluvial origin near surface and of lacustrine origin further below. The upper bank soils were classified as lacustrine clays overlying a clay till layer roughly 0.6 m in thickness, followed by the silt till.

Riverbank instability at 7 Roslyn Road, a site 20 m upstream of Fort Rouge Park, was investigated by KGS Group in 2006. The investigation identified a failure plane within the lacustrine clay but the depth was cited as being 12 m in a report from March 4, 2008, and 4.1 m in a report from March 3, 2009. From the cross section of the site, it appears either interpretation could be correct depending on whether this depth was measured in the lower bank or in the mid bank.

Remediation

Construction of the remedial work designed by KGS Group took place in the winter of 2008, from January to the end of March. The work that was carried out included the construction of a shear key, placement of rip rap, minor riverbank regrading, the construction of a cast-in-place concrete weir, and the replacement of the outfall pipe. Site restoration and revegetation was carried out and completed by June 30, 2008.

The shear key was approximately 50 m long and ran along the riverbank. A 0.6-m-thick layer of rip rap was placed above it, spanning 55 m along the riverbank. The shear key was excavated about 6 m deep and 4 m wide at its base.

The designed remedial measures are typical of this type of structure. In proximity to a river, the use of rip rap is particularly typical since erosion is anticipated.

Performance of Repairs

The slope was instrumented to monitor groundwater levels and slope movement. One slope inclinometer, one pneumatic piezometer, and one standpipe piezometer were installed but were subsequently destroyed during construction of the remedial structures. They were replaced in May 2008 after construction was completed.

An overall reduction in the rate of movement was achieved, going from 65 mm/year before any remedial work to 33 mm/year by the time of the March 3, 2009 report. Since May 2008, approximately 2-3 mm of cumulative movement was recorded in the upper bank, and no movement was recorded in the lower bank. Located approximately 20 m upstream of Fort Rouge Park, 68 mm of movement was measured in the upper bank of 7 Roslyn Road. No signs of distress to the riverbank were observed during the 2008 monitoring period.

A reduction in the rate of movement by a factor of 2 is appropriate after a timeframe of 1 year following construction. While reports for the following years were not available, it is likely this rate of movement continued to decrease for a period of 2-3 years after the report date of March 3, 2009, before stabilizing.

Lessons Learned

The installation of a granular shear key at this site led to an increase in the stability of the slope. The rate of movement decreased drastically after only 1 year following construction, showing the remedial measures were effective. The shear key was installed in between the regulated summer river level and the winter river water level, indicating the support performed well in a saturated environment.

Case Study Details

Landslide Information

Landslide type:	Soil slope deformation	*Pre-support rate (mm/yr):*	65
Trigger:	Toe erosion	*Landslide velocity class:*	2 - Very Slow
Depth of movement (m):	4.1	*Most recent rate (mm/yr):*	33
Sheared material:	Lacustrine clay		

Landslide Dimensions

Width (m):	65	*Length (m):*	75
Height (m):	16.2	*Slope (°):*	8.7 – 11.3

Monitoring Information

Movement first reported:		*Last inspected:*	January 4, 2009
SIs installed:	4	*SIs active last inspection:*	2
Piezos installed:	4 PP, 4 SP	*Piezos active last inspection:*	2 PP, 2 SP

Groundwater Information

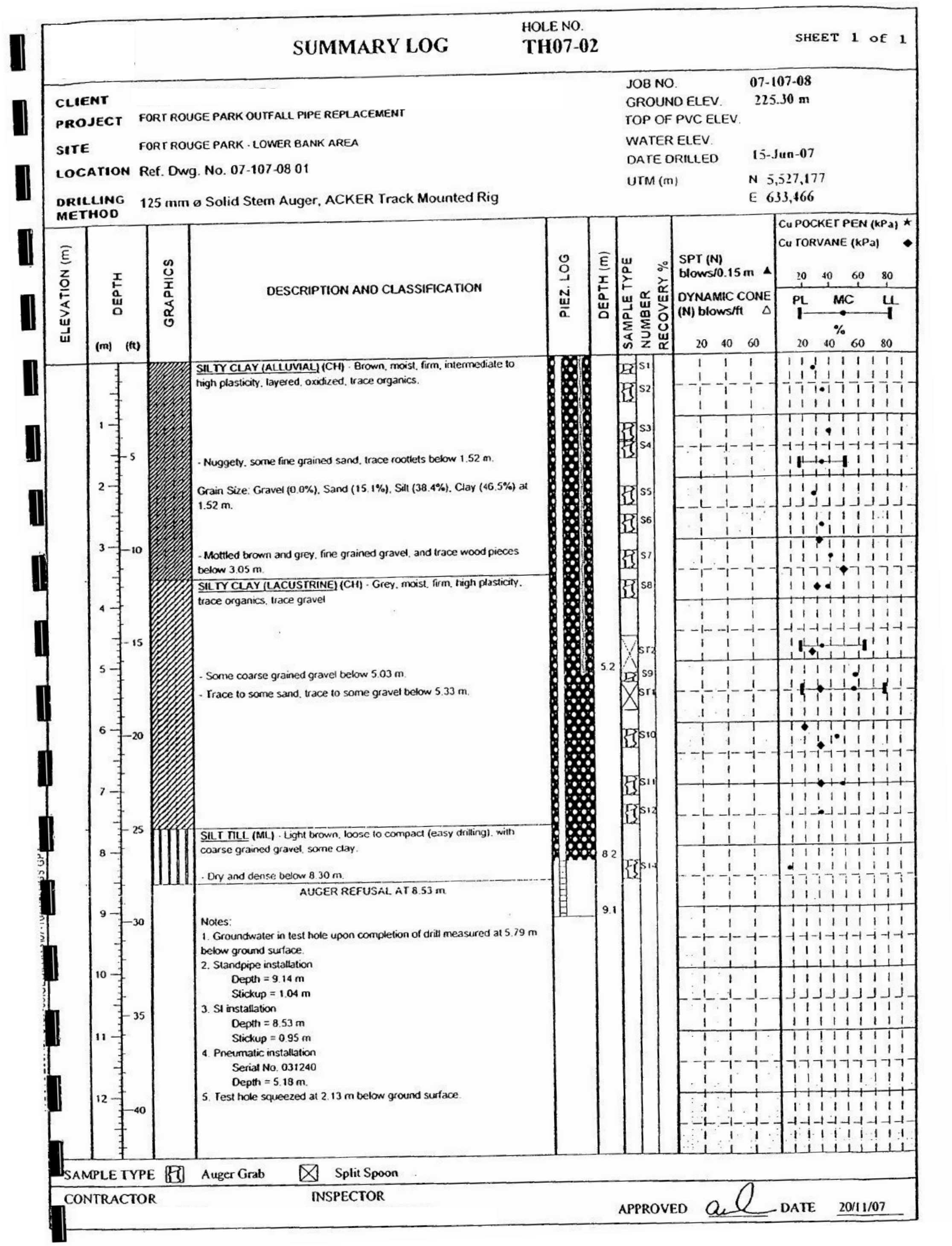
Groundwater level (masl):	219.1 – 225.3	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	Upward gradient (till)	*Surface water type:*	River

Repair Specifications

Repair type:	Shear key	*Repair date:*	March 2008
Base width (m):	4	*Trench slope ratio (H:V):*	4:1
Length (m):	50	*Drainage (yes/no):*	No
Granular height (m):	5.95	*Backfill:*	95% SPDD
Overburden (m):	0.6 (+ 0.6)	*Ground elevation (masl):*	227.2 – 228.0
Overburden material(s):	Rockfill rip rap blanket with possible clay cap underneath		

Additional Site Information

Stratigraphy



SUMMARY LOG

HOLE NO. TH07-02

SHEET 1 of 1

CLIENT

PROJECT FORT ROUGE PARK OUTFALL PIPE REPLACEMENT

SITE FORT ROUGE PARK - LOWER BANK AREA

LOCATION Ref. Dwg. No. 07-107-08 01

DRILLING METHOD 125 mm ø Solid Stem Auger, ACKER Track Mounted Rig

JOB NO. 07-107-08

GROUND ELEV. 225.30 m

TOP OF PVC ELEV.

WATER ELEV.

DATE DRILLED 15-Jun-07

UTM (m) N 5,527,177 E 633,466

ELEVATION (m) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.15 m ▲ DYNAMIC CONE (N) blows/ft △ | Cu POCKET PEN (kPa) ★ Cu TORVANE (kPa) ◆ PL MC LL %

SILTY CLAY (ALLUVIAL) (CH) - Brown, moist, firm, intermediate to high plasticity, layered, oxidized, trace organics.

- Nuggety, some fine grained sand, trace rootlets below 1.52 m.

Grain Size: Gravel (0.0%), Sand (15.1%), Silt (38.4%), Clay (46.5%) at 1.52 m.

- Mottled brown and grey, fine grained gravel, and trace wood pieces below 3.05 m.

SILTY CLAY (LACUSTRINE) (CH) - Grey, moist, firm, high plasticity, trace organics, trace gravel

- Some coarse grained gravel below 5.03 m.

- Trace to some sand, trace to some gravel below 5.33 m.

SILT TILL (ML) - Light brown, loose to compact (easy drilling), with coarse grained gravel, some clay.

- Dry and dense below 8.30 m.

AUGER REFUSAL AT 8.53 m

Notes:

1. Groundwater in test hole upon completion of drill measured at 5.79 m below ground surface.
2. Standpipe installation
   Depth = 9.14 m
   Stickup = 1.04 m
3. SI installation
   Depth = 8.53 m
   Stickup = 0.95 m
4. Pneumatic installation
   Serial No. 031240
   Depth = 5.18 m.
5. Test hole squeezed at 2.13 m below ground surface.

SAMPLE TYPE Auger Grab Split Spoon

CONTRACTOR INSPECTOR APPROVED DATE 20/11/07

Figure A-52: Borehole log for WP6. Reproduced from KGS Group (2015)*, with permission.*

Cross Section

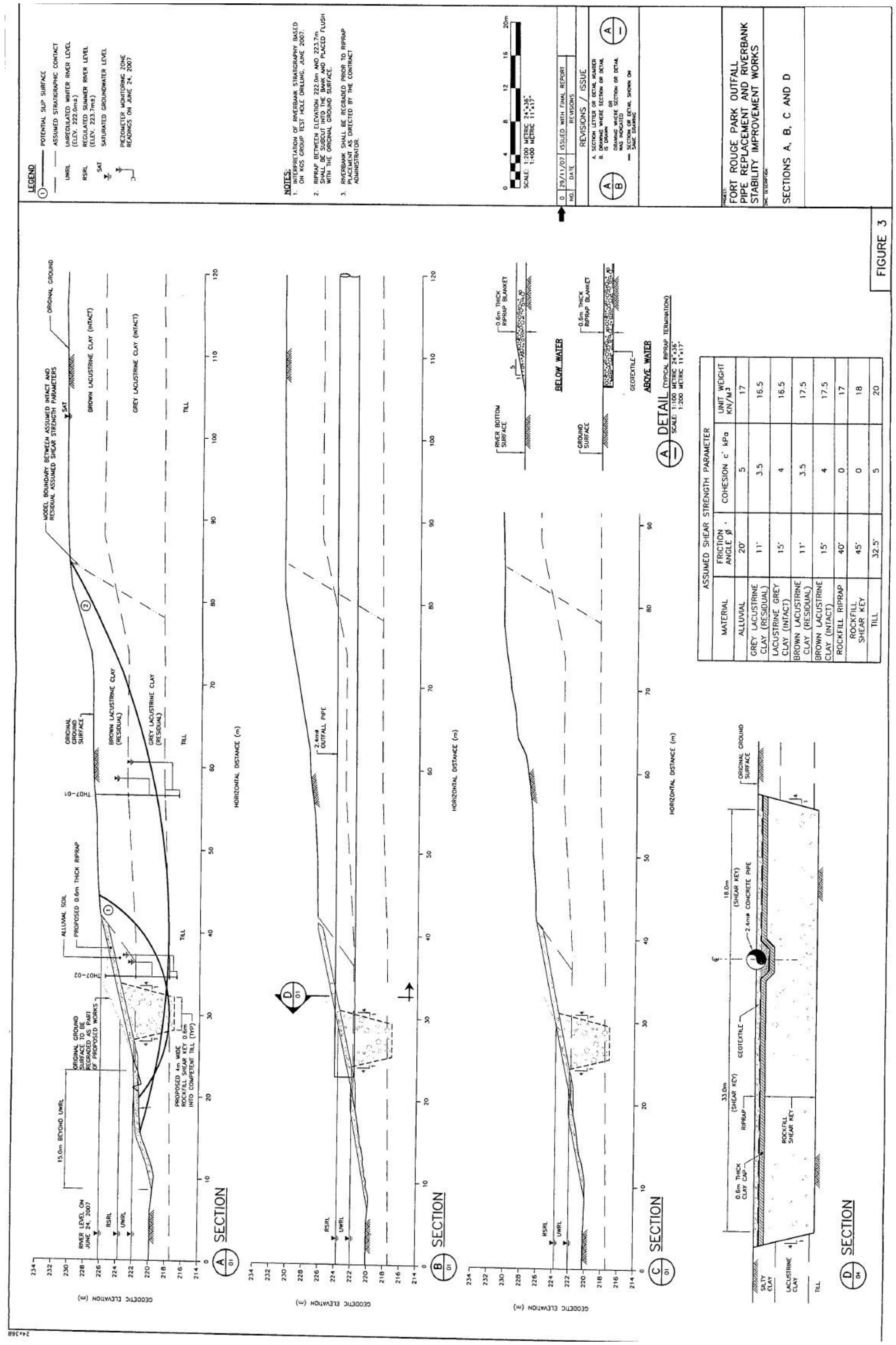


Figure A-53: Cross sections for WP6. Reproduced from KGS Group (2015)*, with permission.*

Plan Map

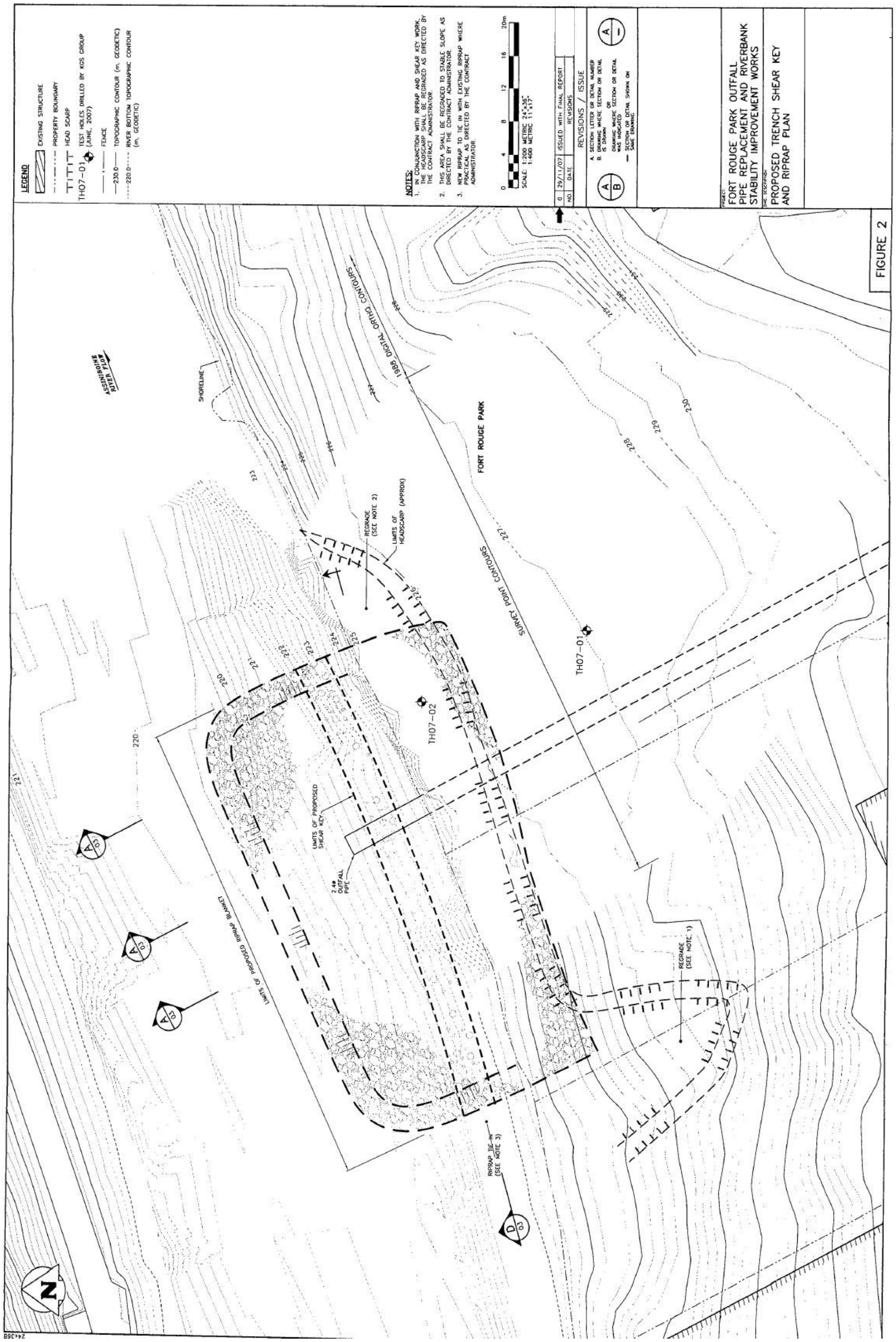


Figure A-54: Plan map for WP6. Reproduced from KGS Group (2015)*, with permission.*

WP7 – *Hawthorne FPS*

Winnipeg, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by KGS Group in *Outfall Pipe Performance along Stabilized Riverbanks - Preliminary Assessment of Existing Sites in the City of Winnipeg* (2015).

Background

Hawthorne FPS is located along the Red River in Winnipeg, Manitoba. The slope is composed of clay fill to a depth of 2.5 m, followed by a thin (0.25 m) layer of silty clay and a layer of silt (0.75 m). The rest of the bank is composed primarily of lacustrine clay, which starts at a depth of 3.5 m and extends down to a fractured limestone bedrock at a depth of 16.5 m. The overall slope is inclined at 5.7H:1V, and the depth of the bedrock from the top of the bank is 20.4 m. A pipe outlet is located at an elevation of 222 masl, which places it at the edge of the river.

Remediation

The slope was remediated starting in February of 2006, and was completed by July 20. Rockfill columns were installed in four rows along the lower bank, spanning 19 m in both directions coming from the outfall pipe. The slide scarp is illustrated immediately upslope of the columns, and the columns appear to span the entire region between the scarp and the river. Most of the columns were 2.1 m in diameter but many 1.05-m-diameter columns were also installed on the north side of the outfall pipe (see cross section). Rock fill was used to backfill the columns but it was not vibro-compacted. The columns were then topped with a 0.6-m-thick clay cap. A 0.6-m-thick layer of rip rap was also placed along the edge of the river, covering some of the columns.

Performance of Repairs

Two SIs were identified at this site: SI-05 and SI-6. Per the plan map that was acquired, SI-6 was destroyed. SI-05, located at the top of the riverbank, was monitored for 7.9 years following construction. A total of 15 mm of movement was recorded during that time. The rate in the first year was 3.78 mm/year, 2.02 mm/year for Year 1-3, and 0.75 mm/year for the period after three years. This deceleration suggests the repair was successful and performed as intended.

Lessons Learned

This site shows that smaller diameter columns can be used to the same effect as large diameter columns for slope stabilization purposes.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:	Toe erosion	*Landslide velocity class:*	1 - Extremely Slow
Depth of movement (m):	4, 14, 22 (?)	*Most recent rate (mm/yr):*	0.75
Sheared material:	Clay fill, Silt, Lacustrine clay		

Landslide Dimensions

Width (m):	38	*Length (m):*	44
Height (m):	18	*Slope (°):*	10

Monitoring Information

Movement first reported:		*Last inspected:*	December 18, 2013
SIs installed:	2	*SIs active last inspection:*	1
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	3.35	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	River

Repair Specifications

Repair type:	Columns	*Repair date:*	February 7, 2006
Column diam. (m):	2.1, (1.05)	*Rows:*	4
Length (m):	38	*Drainage (yes/no):*	No
Layout:	Triangle	*Spacing (up x across [m]):*	2.6 x 3.5, 1.7 x 2.2
Granular height (m):	15.5 – 17	*Backfill:*	
Overburden (m):	0.6		
Overburden material(s):	Clay cap		

Additional Site Information

Stratigraphy

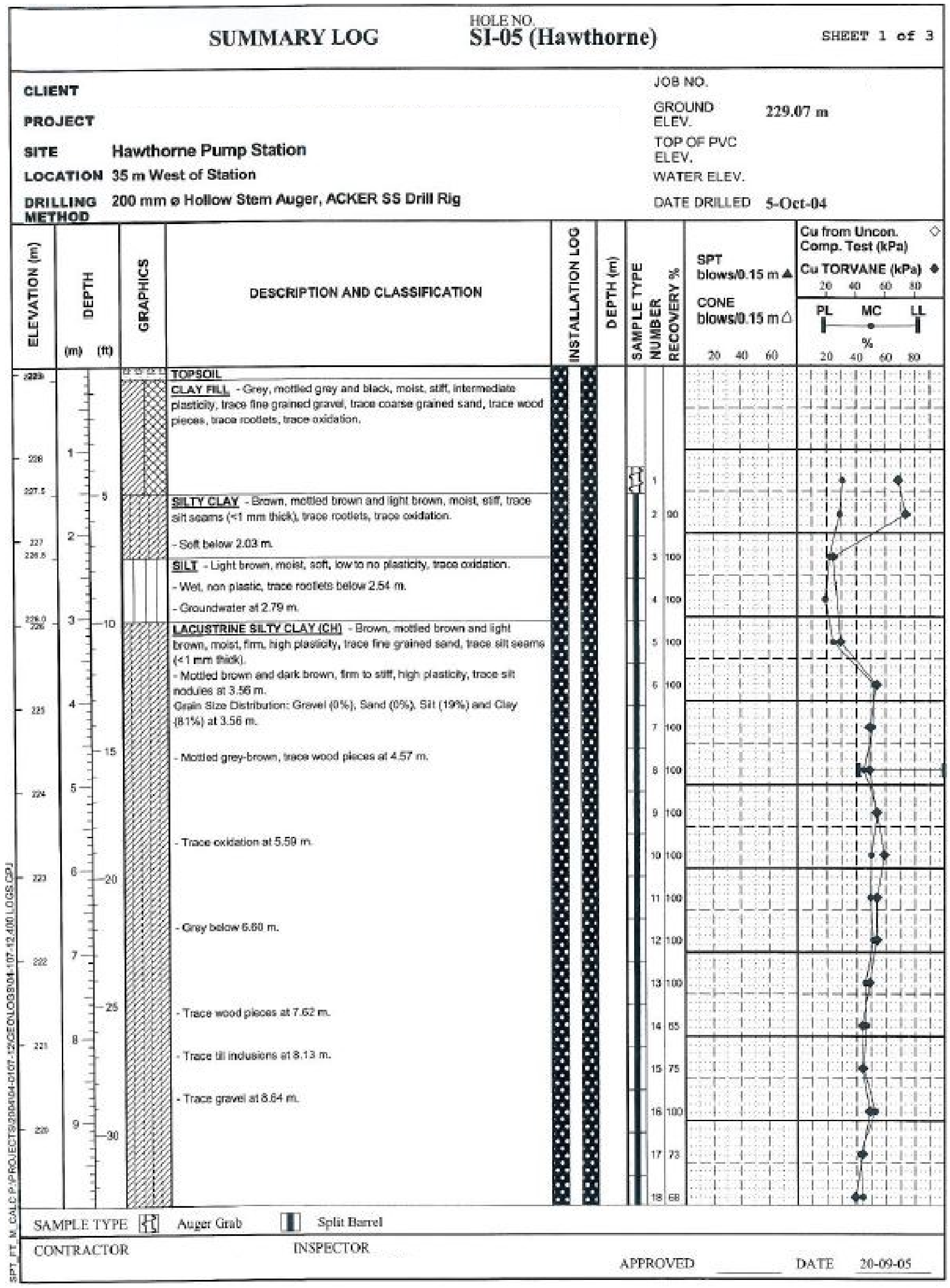


Figure A-55: Borehole log (Part 1/2) for WP7. Reproduced from KGS Group (2015)*, with permission.*

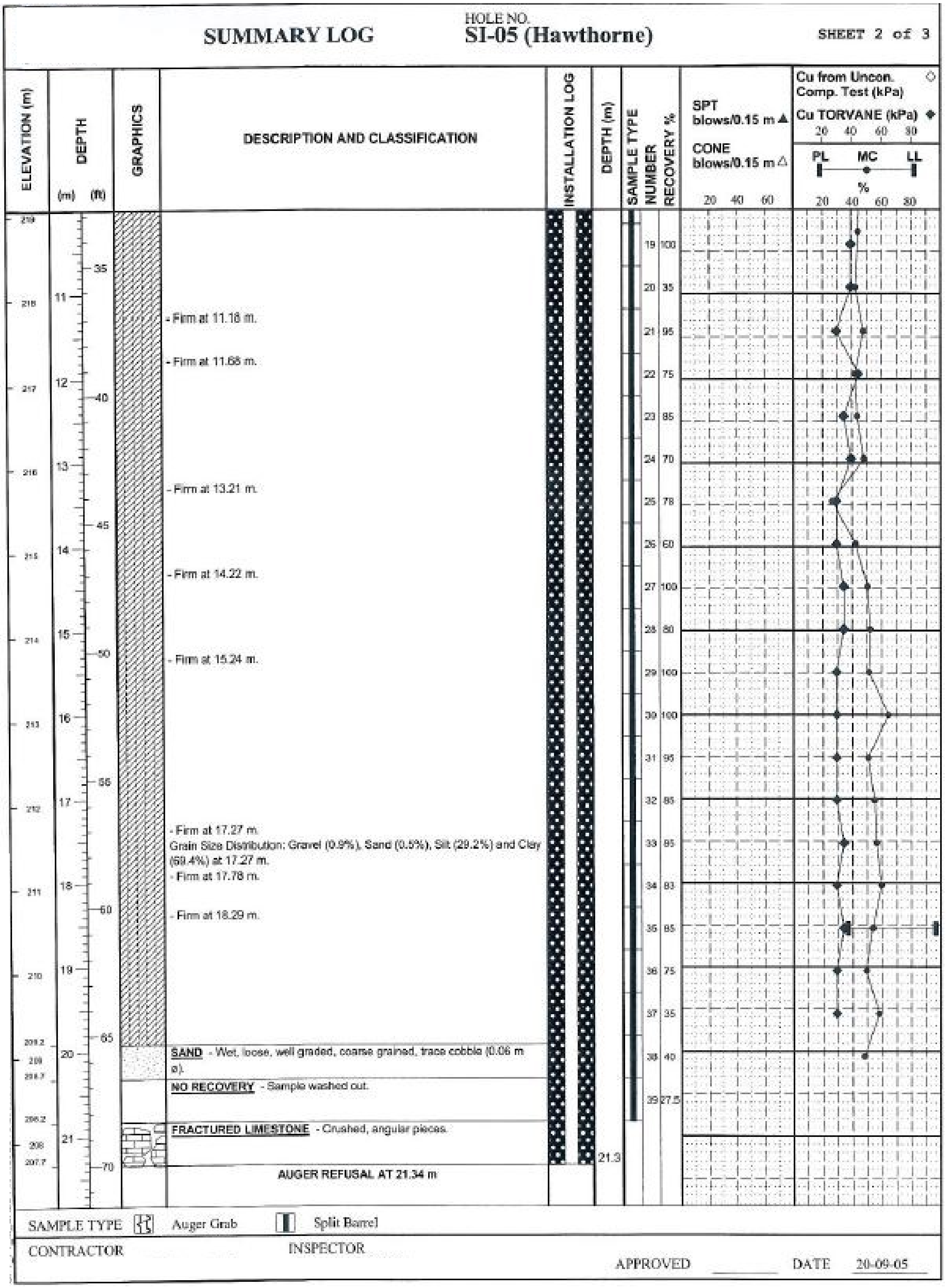


Figure A-56: Borehole log (Part 2/2) for WP7. Reproduced from KGS Group (2015)*, with permission.*

Cross Section

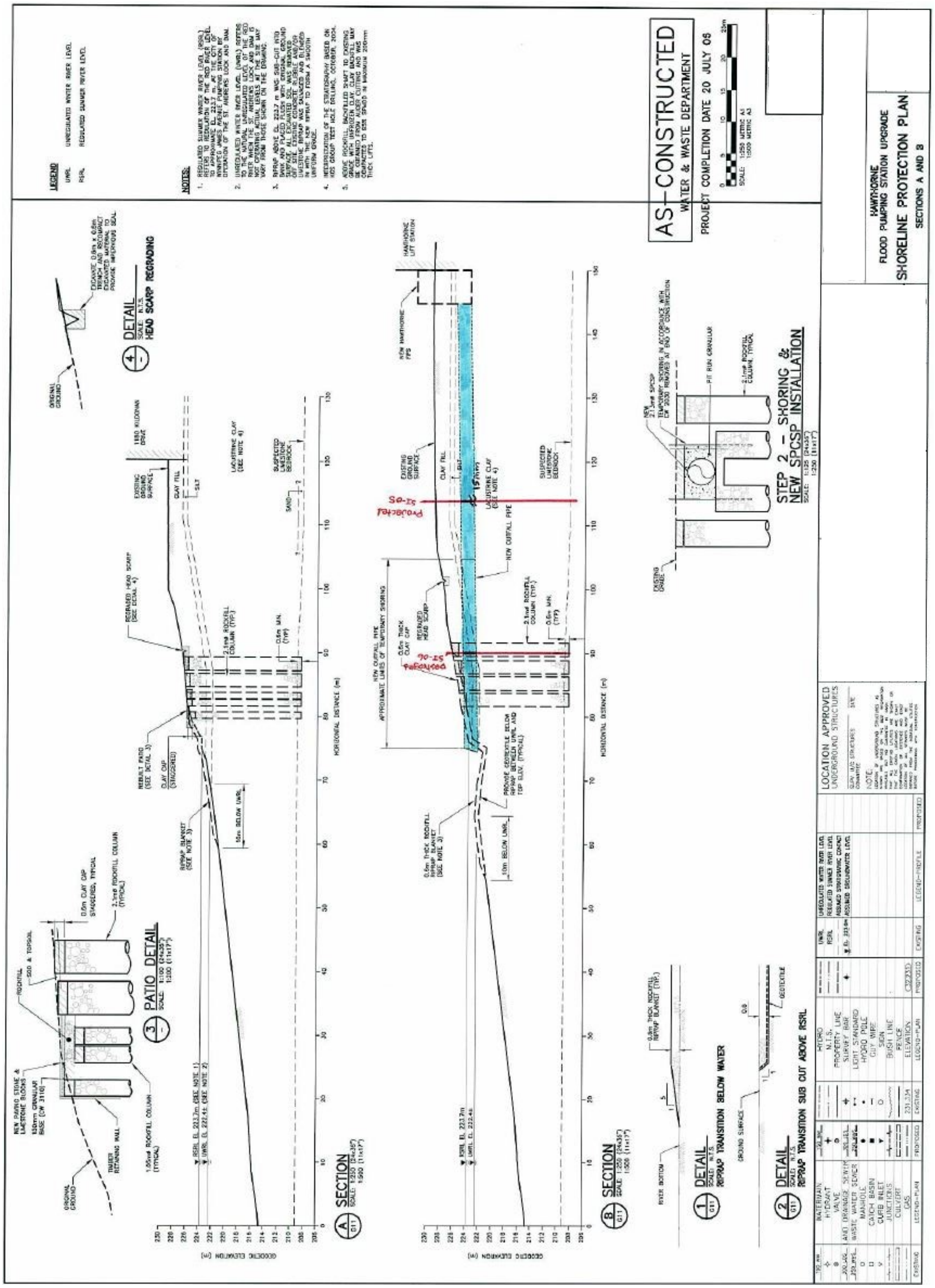


Figure A-57: Cross sections for WP7. Reproduced from KGS Group (2015)*, with permission.*

Plan Map

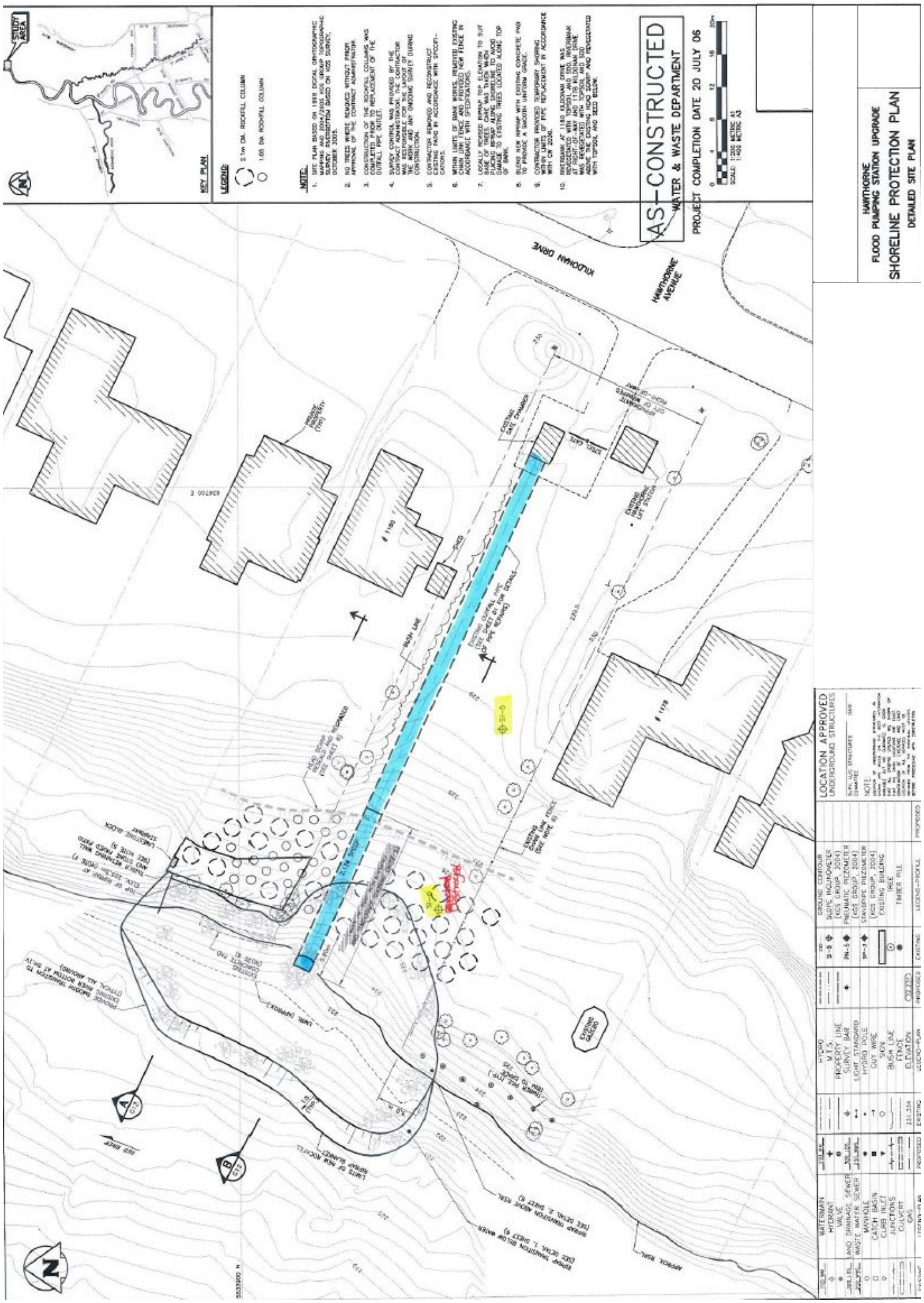


Figure A-58: Plan map for WP7. Reproduced from KGS Group (2015)*, with permission.*

WP8 – *Mager Drive Pumping Station*

Mager Drive West at St. Mary's Road, Winnipeg, MB

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by Yarechewski and Tallin in *Riverbank stabilization performance with rock-filled ribs/shear key and columns* (2003).

Background

The Mager Drive Pumping Station was one of two riverbank projects in Winnipeg that were the subject of a comparison between riverbank slope remedial measures typically adopted by the City of Winnipeg (Yarechewski & Tallin, 2003). The site is located on an outside bend of the Red River in Winnipeg, Manitoba near the intersection of Mager Drive West and St. Mary's Road.

The Mager Drive Pumping Station slope was remediated in 1993 using 30 stone-filled ribs and a granular shear key. The riverbank had been measured moving at a rate of 38 mm/year over the 4 years prior to remediation. The riverbank was reported to rise between 8-12 m above the bottom of the river channel and is composed of lacustrine clay. Dense till lies 6-8 m below the river channel. The slip surface was identified as being entirely in the lacustrine clay but extending very close to the till.

The residual shear strength of the lacustrine clay was back-calculated using limit equilibrium analysis, assuming a factor of safety of unity for the actively moving slope. The analysis yielded the following properties: $c' = 0\text{-}4$ kPa and $\varphi' = 8\text{-}12°$.

Remediation

The remediation of the Mager Drive Pumping Station consisted of 30 stone-filled ribs and a granular shear key. Crushed limestone from bedrock mines northwest of Winnipeg was used as the backfill material for both. The crushed limestone is well-graded angular rock with 100% finer than 100 mm and 5% finer than 0.075 mm. Large-scale direct shear tests were used to measure the shear strength of the material, giving an internal angle of friction of 54°.

The design of the remedial measures was achieved using limit equilibrium analysis and introducing the crushed limestone in the appropriate configurations. The crushed limestone was modeled with an internal angle of friction of 45°. The target factor of safety for the designs was 1.4.

Yarechewski and Tallin (2003) note that early construction of shear keys involved dumping the limestone backfill loose into the excavated trench. The authors remark that construction typically takes place in winter when river levels are lower, giving access to more of the riverbank, despite the difficulty of compacting frozen crushed limestone. A loose dry unit weight of 16 kN/m^3 is given for the frozen crushed limestone. The authors report that an angle of internal friction of 45° cannot be justified at this unit weight. Compaction using a vibratory plate was found to be able to increase the dry unit weight to 18.0-18.8 kN/m^3. This method was used on the shear key at the Mager Drive Pumping Station.

The ribs were constructed first to increase stability in anticipation of the shear key excavation. The ribs were 1.3 m wide, 8 m long, and were spaced 3 m apart, centre-to-centre. The shear key was 4 m wide and was placed at the toe of the slope. The shear key runs parallel to the Red River, while the ribs run perpendicular to the river. The ribs and the shear key were both excavated about 1 m into the dense till underlying the lacustrine clay comprising the riverbank. Rip rap was used to cover the slope after both support structures had been completed.

The use of stone-filled ribs in combination with granular shear keys is not very common, based on the case studies that have been compiled for this research. The use of stone-filled ribs seems to have been in lieu of a full excavation of the slope and replacement with gravel. This method, while uncommon, appears to have been well-suited to the site's proximity to the river as well as buildings on the street upslope. This method achieved partial stabilization of the slope without incurring significant risks to the slope during construction, which then in turn permitted the construction of the shear key. The placement and dimensions of the shear key are typical though, and the key-in depth for both the ribs and the shear key is typical.

Performance of Repairs

The post-construction monitoring program at the Mager Drive Pumping Station was reported to have been carried out for over 10 years, at the time of the publication by Yarechewski and Tallin (2003). Prior to remediation, movement was measured at 38 mm/year for 4 years. The riverbank was observed accelerating during construction of the remedial measures, reaching rates of 500-

1200 mm/year. This decreased to 6 mm/year after a few years, eventually reaching 1 mm/year after 10 years. A total displacement of 36 mm was measured over that 10-year period, compared to 90 mm measured over the 4 years prior to construction.

Given the significant decrease in the rate of movement, the repairs can be considered successful. From the displacement plotted against time in Yarechewski and Tallin (2003), the movement appears to have reached a state of creep after approximately 3 years. This timeframe is typical for primary movement to be halted by shear key remedial measures.

Lessons Learned

It was noted by Yarechewski and Tallin (2003) that the internal angle of friction of granular material is dependent on bulk unit weight. They also noted that the compaction of the crushed limestone to increase the unit weight would result in greater internal angles of friction, thus improving design economies and long-term performance.

This case study also serves as an example of an alternative to a full replacement buttress fill. The use of stone-filled ribs appears to have been an economic and effective remedial measure appropriate for the conditions encountered at this site.

Case Study Details

Landslide Information

Landslide type:	Compound slide	*Pre-support rate (mm/yr):*	38
Trigger:		*Landslide velocity class:*	2 - Very Slow
Depth of movement (m):	6 – 8	*Most recent rate (mm/yr):*	1
Sheared material:	Lacustrine clay		

Landslide Dimensions

Width (m):		*Length (m):*	
Height (m):	8 – 12	*Slope (°):*	8 – 12

Monitoring Information

Movement first reported:		*Last inspected:*	January 1, 2003
SIs installed:	2	*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):		*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	River

Repair Specifications

Repair type:	Shear key	*Repair date:*	January 1, 1993
Base width (m):	4	*Trench slope ratio (H:V):*	
Length (m):		*Drainage (yes/no):*	
Granular height (m):		*Backfill:*	18.0 – 18.8 kN/m^3
Overburden (m):			
Overburden material(s):	Rip-rap		

Additional Site Information

Cross Section

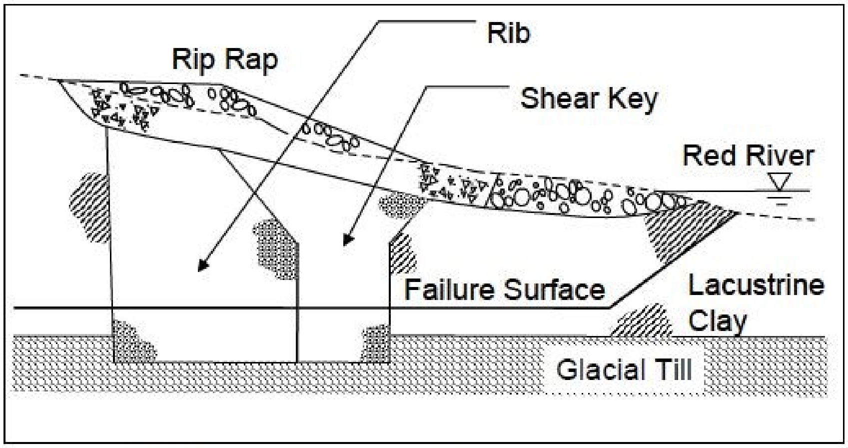


Figure 3. Mager Drive ribs/shear key section.

Figure A-59: Cross section of the Mager Drive Pumping Station shear key and ribs. Reproduced from Yarechewski and Tallin (2003)*, with permission.*

Plan Map

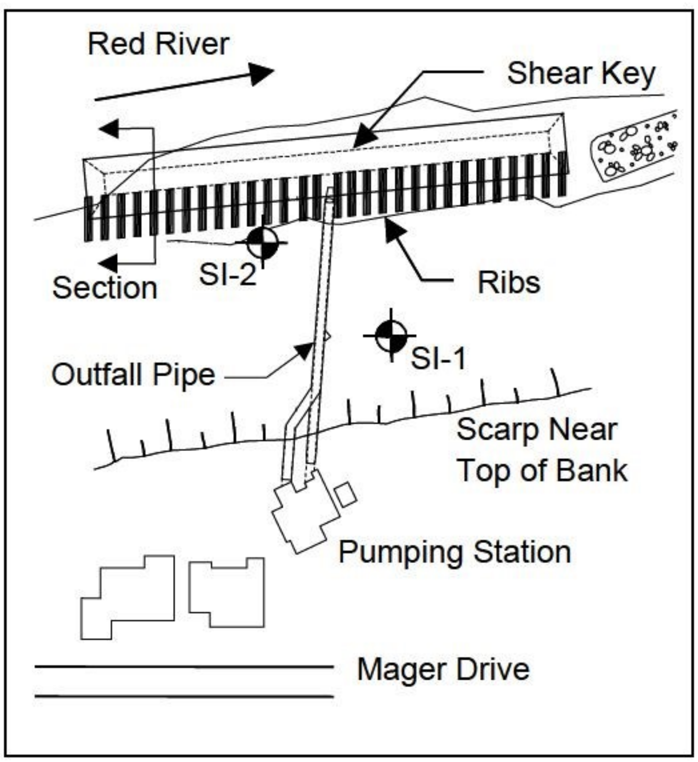


Figure 2. Mager Drive ribs/shear key plan.

Figure A-60: Cross section of the Mager Drive Pumping Station shear key and ribs. Reproduced from Yarechewski and Tallin (2003)*, with permission.*

WP9 – *Oakcrest Place Outfall*

Elm Park Road and Dunkirk Place, near Oakcrest Street, Winnipeg, MB

Case Study Summary

Background

This site is located along an outside bend of the Red River in Winnipeg, MB. In 1996, an outfall assessment rated the outfall structures at this location as having a failure condition (KGS Group, 2013).

The top of the bank is flat here, but slopes down at 1.1H:1V in the upper bank. After 4 m of elevation change, the bank transitions to a slope of 3.8H:1V for an additional 10.5 m of elevation change before becoming flat again. The slope is 6H:1V to the bottom of the river (KGS Group, 2013).

The slope is comprised of 1.1 m of silty clay fill at surface. This is underlain by alluvial silty clay to an elevation of 222.5 masl, followed by lacustrine silty clay to an elevation of 216.7 masl. Silt till was identified below the lacustrine silty clay (KGS Group, 2013).

Two pneumatic piezometers, one standpipe piezometer, and one slope inclinometer were installed in 2012. The groundwater was reported as ranging from 225.91 to 226.89 masl in the overlying clay, and 223.46 to 225.50 masl in the till. A downward hydraulic gradient was generally observed. No movement in the slope was reported throughout the one-year period between installation (October 2012) and October 2013 (KGS Group, 2013).

Stability analyses were performed on the slope. The lacustrine clay was modelled with intact strength parameters of $\varphi' = 15°$ and $c' = 4$ kPa, and residual parameters of $\varphi' = 11°$ and $c' = 3$ kPa. The alluvial clay was modelled with intact strength parameters of $\varphi' = 25°$ and $c' = 6$ kPa, and residual parameters of $\varphi' = 19°$ and $c' = 6$ kPa. A factor of safety near unity was calculated for a groundwater elevation of 226.3 masl and a regulated summer river level of 223.7 masl (KGS Group, 2013).

Remediation

The slope here was remediated with a 25-m-long rockfill trench shear key in 2016. The shear key was 2.5 m wide at its base and was constructed with 0.25H:1V side slopes. The shear key was keyed 0.6 m into the till and backfilled with limestone rockfill, which was then vibro-compacted. The shear key was topped with a 1-m-thick compacted clay cap followed by a 0.6-m-thick layer of rip rap. The mid and upper bank areas were also regraded to 3H:1V (KGS Group, 2016).

Performance of Repairs

Monitoring of bank movements was undertaken for a period lasting almost two years following the completion of construction in March 2014. A replacement slope inclinometer was installed at that time, in the upper bank. Deflection of up to 100 mm was measured at the ground surface on September 10, 2014. This deflection was attributed to an impact force on the SI casing though, and was not related to slope movements. No movements were observed below a depth of 2 m (KGS Group, 2016).

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:		*Landslide velocity class:*	
Depth of movement (m):		*Most recent rate (mm/yr):*	
Sheared material:	Lacustrine clay		

Landslide Dimensions

Width (m):	25	*Length (m):*	
Height (m):	10	*Slope (°):*	14.7

Monitoring Information

Movement first reported:	1996	*Last inspected:*	October 21, 2015
SIs installed:	2	*SIs active last inspection:*	1
Piezos installed:	3	*Piezos active last inspection:*	

Groundwater Information

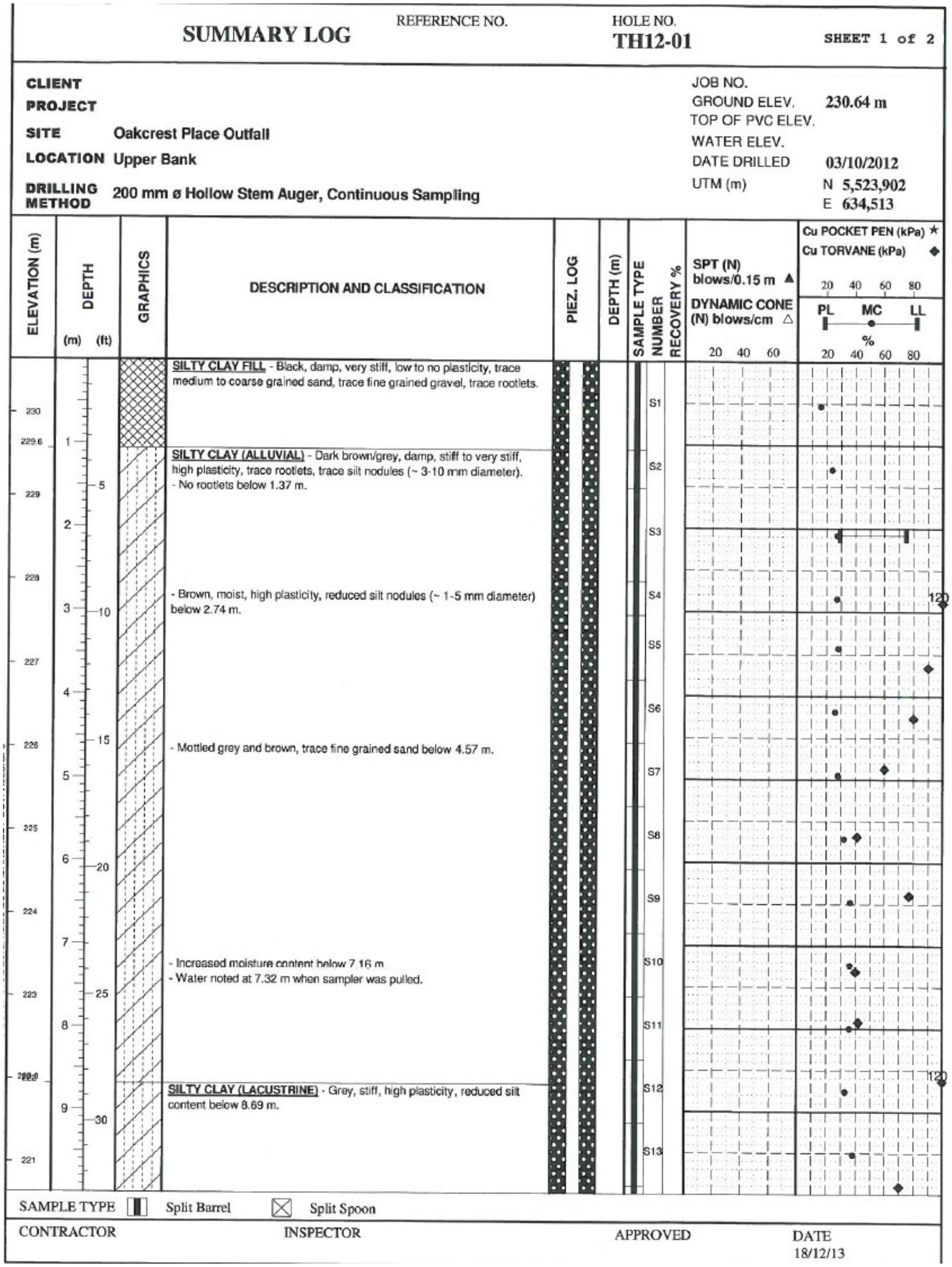
Groundwater level (mbgl):	0 – 4.2	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	No	*Surface water type:*	River

Repair Specifications

Repair type:	Shear key	*Repair date:*	March 2014
Base width (m):	2.5	*Trench slope ratio (H:V):*	0.25:1
Length (m):	25	*Drainage (yes/no):*	No
Granular height (m):	4.7	*Backfill:*	Crushed limestone
Overburden (m):	1.6		
Overburden material(s):	Clay cap and rip rap		

Additional Site Information

Stratigraphy


SUMMARY LOG   REFERENCE NO.   HOLE NO. TH12-01   SHEET 1 of 2

CLIENT
PROJECT
SITE Oakcrest Place Outfall
LOCATION Upper Bank
DRILLING METHOD 200 mm ø Hollow Stem Auger, Continuous Sampling

JOB NO.
GROUND ELEV. 230.64 m
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED 03/10/2012
UTM (m) N 5,523,902 E 634,513

ELEVATION (m) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.15 m ▲ DYNAMIC CONE (N) blows/cm △ 20 40 60 | Cu POCKET PEN (kPa) ★ Cu TORVANE (kPa) ◆ 20 40 60 80 PL MC LL % 20 40 60 80

SILTY CLAY FILL - Black, damp, very stiff, low to no plasticity, trace medium to coarse grained sand, trace fine grained gravel, trace rootlets.

SILTY CLAY (ALLUVIAL) - Dark brown/grey, damp, stiff to very stiff, high plasticity, trace rootlets, trace silt nodules (~ 3-10 mm diameter).
- No rootlets below 1.37 m.

- Brown, moist, high plasticity, reduced silt nodules (~ 1-5 mm diameter) below 2.74 m.

- Mottled grey and brown, trace fine grained sand below 4.57 m.

- Increased moisture content below 7.16 m.
- Water noted at 7.32 m when sampler was pulled.

SILTY CLAY (LACUSTRINE) - Grey, stiff, high plasticity, reduced silt content below 8.69 m.

S1 S2 S3 S4 S5 S6 S7 S8 S9 S10 S11 S12 S13

SAMPLE TYPE Split Barrel Split Spoon
CONTRACTOR   INSPECTOR   APPROVED   DATE 18/12/13

Figure A-61: Borehole log (Part 1/2) for WP9. Reproduced from KGS Group (2013) *with permission.*

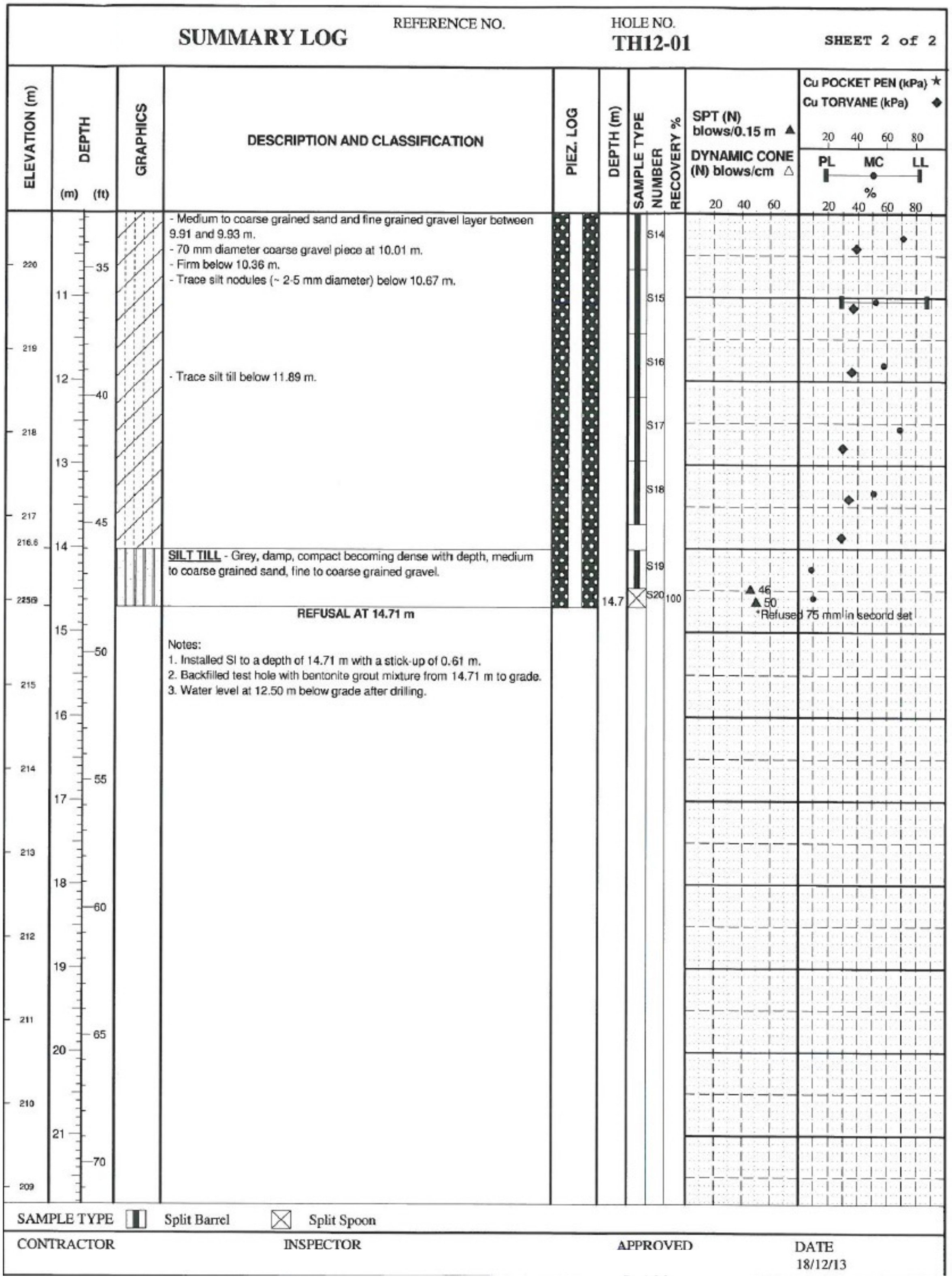


Figure A-62: Borehole log (Part 2/2) for WP9. Reproduced from KGS Group (2013) *with permission.*

Cross Section and Plan Map

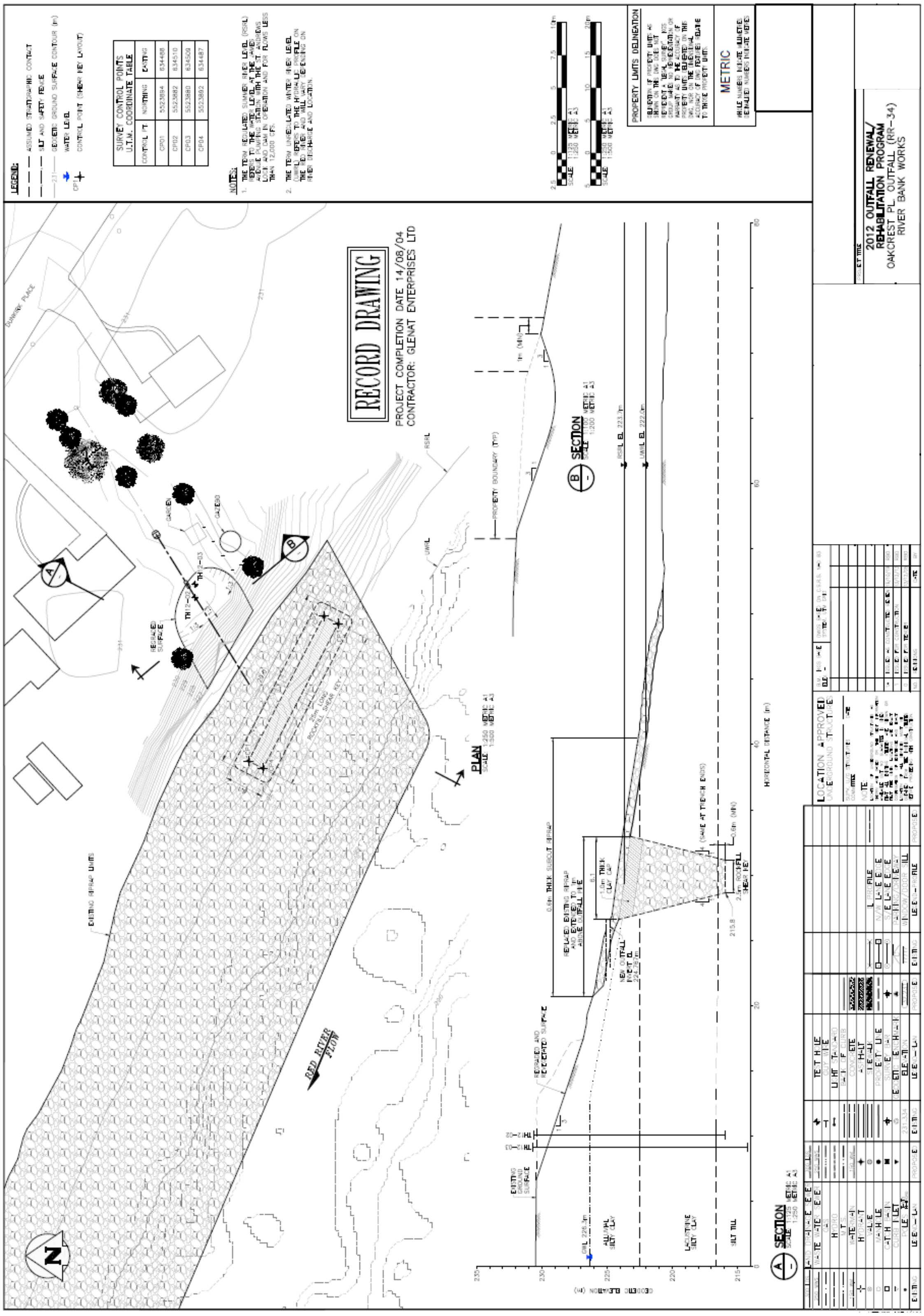


Figure A-63: Cross section and plan map for WP9. Reproduced from KGS Group (2016) *with permission.*

WP10 – *Omand’s Creek Outfalls*

Polo Park, Winnipeg, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by Trek Geotechnical in *Polo Park Infrastructure Upgrades Omand's Creek Outfalls - Geotechnical Investigation Report* (2014).

Background

Omand’s Creek is a channel that diverts water from Colony Creek into the Assiniboine River in Winnipeg, MB. Its banks have historically been affected by active erosion and instabilities but no such instabilities were observed during site investigations in 2014. Upgrades to the Omand’s Creek Outfalls were proposed in 2014 by the City of Winnipeg. The work that was planned for the outfall renewal involved slope steepening at the culvert crossing closest to the St. Matthews Avenue and Empress Street intersection. The St. Matthews Avenue crossing of Omand’s Creek had been constructed in 2011. A geotechnical investigation for that work was carried out in 2009 and involved the drilling of four test holes, a slope stability analysis, and recommendations for stabilization works. The data from the test holes drilled in 2009 was used for the design of the new outfalls at the St. Matthews Avenue crossing. The data set was supplemented with additional test holes drilled at the Ellice Avenue crossing one block north of the St. Matthews Avenue crossing.

The existing slope before remediation averaged an angle of 3.1H:1V and a factor of safety of 1.10 was calculated based on a stability analysis. The slope measured roughly 5 m high based on a simple cross section included in the 2014 proposal.

The soil stratigraphy was described as consisting of fill materials, followed by silt, then clay, all underlain by silt till. South of the St. Matthews Avenue crossing, two types of fill were reported. The first was a gravel fill with a moisture content of 22%, identified in a test hole located in the southeast bank. It was observed up to a depth of 1.64 m. The second type was a silty clay fill described as being brown and weathered, identified in a test hole located in the southwest bank. The silty clay fill was found to extend to depths of 1.54 m.

Silt was identified in several of the test holes beginning at a depth of 0.4-1.9 m. This layer varied between 0.6-1.4 m in thickness and was described as light brown, loose to compact, and having low to no plasticity. It was also described as being moist to wet, with moisture contents between 22-25%.

High plastic clay was found below the silt. It was described as mottled brown and grey in colour, and firm to stiff. It was found to become predominantly grey and softer with depth. Trace silt inclusions were also identified. The undrained shear strength was measured using unconfined compression, Torvane, and pocket penetrometer tests. The tests indicated values ranging between 25-60 kPa and an average of 41 kPa. The bulk unit weight was measured ranging between 16.7-17.5 kN/m^3 and averaging 17.1 kN/m^3. The clay was described as moist, with moisture contents ranging from 35-59%. Moisture content was found to increase with depth for the first metre before becoming relatively consistent. Testing also yielded a liquid limit of 80% and a plastic limit of 20%, giving a plasticity index of 60%.

The silt till was the deepest observed unit. Near the Ellice Avenue outfall, it was identified at a depth of 9.8 m and continued until refusal at 14.2 m depth. At the St. Matthews outfall, it was encountered at depths of 8.4 m and 7.9 m (225.0 masl and 221.6 masl) on the southeast and southwest banks, respectively. Descriptions for the Ellice Avenue test holes differed depending on the depth. For 9.8-12.2 m below ground, the till was brown in colour, containing trace sand and gravel. It was also described as being clayey, soft, and wet, with intermediate to high plasticity. From 12.2-14.2 m below ground, the till was reported to contain trace clay and become dense to very dense. It was also said to have become dry. The SPT blow count to advance 300 mm was 119 blows at a depth of 13.7 m.

At the Ellice Avenue outfall, the silty till moisture content was measured at 37% in the upper portion and between 8-11% for the lower portion. Sample from the test holes near St. Matthews Avenue yielded moisture contents ranging from 13-21%. Water inflow was reported at one of these test holes at a depth 2.1 m (227.9 masl), and sloughing was reported in the other at 4.6 m (226.2 masl).

Remediation

The remedial works were designed for a minimum factor of safety of 1.3. The designers considered a granular shear key and ribs to be the option that was most economic and most suitable to site conditions. The rockfill was modeled differently for the shear key and the ribs. This was because the rockfill properties for the ribs had to be representative of the weighted average of the rockfill and clay, based on the area replacement ratio in plan view. The shear key rockfill was assigned a unit weight of 21 kN/m^3, no cohesion, and a friction angle of 45°. The ribs were assigned a unit weight of 19 kN/m^3, cohesion of 2 kPa, and a friction angle of 30°. The high plastic clay was modeled with a unit weight of 16.5 kN/m^3, cohesion of 4 kPa, and a friction angle of 14°. The silt till was modeled with a unit weight of 21 kN/m^3, cohesion of 1 kPa, and a friction angle of 35°.

The granular shear key was found to yield the desired FOS (1.32) in the stability analysis. The shear key was designed with a 1 m base width and trench slopes of 1H:2V. The depth of the shear key was modified until the desired FOS was met, which was achieved by extending the shear key to an elevation of 225 masl. The slip surface was likely at an elevation of 227.5-228.0 masl.

A preceding report included in the 2014 report gave a recommendation for a shear key spanning 4 m at its base. This shear key design was expected to provide a FOS of 1.39 if founded at an elevation of 229.5 masl. For a length of 38 m, the shear key was designed with a volume of 2000 m^3 and would cost CAD$70 000 based on a cost of $40/m^3 of rockfill.

Several advantages were given in the 2014 report for the adoption of granular ribs. It is not known which remedial measures were adopted in the end. Both designs described in the report are typical of either method.

Performance of Repairs

The only report acquired for this site precedes the repairs. Thus, no indication of the performance of the repairs is available.

Lessons Learned

For the relatively shallow riverbank that was the subject of this case study, granular ribs and a granular shear key were both considered as stabilization options. While it was not possible to gauge the performance of whichever support was adopted, the proposed designs may offer some insight into the expected performance of both. To achieve the desired FOS, a shear key with a 1-m-wide base was required, assuming the groundwater table was at 2 m below surface near the crest of the roughly 5-m-high slope. This information can be compared to other case studies with slopes of similar dimensions.

Case Study Details

Landslide Information

Landslide type:	Rotational slide	*Pre-support rate (mm/yr):*	
Trigger:		*Landslide velocity class:*	
Depth of movement (m):		*Most recent rate (mm/yr):*	
Sheared material:	Brown and grey clay		

Landslide Dimensions

Width (m):		*Length (m):*	
Height (m):	5	*Slope (°):*	17.9

Monitoring Information

Movement first reported:		*Last inspected:*	
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	2	*Surface water at toe:*	Usually dry
Seepage detected (yes/no):		*Surface water type:*	Stream/Creek

Repair Specifications

Repair type:	Shear key	*Repair date:*	
Base width (m):	1	*Trench slope ratio (H:V):*	1:1
Length (m):		*Drainage (yes/no):*	
Granular height (m):		*Backfill:*	21 kN/m^3
Overburden (m):			
Overburden material(s):			

WP11 – *Pembina-Ducharme Culvert*

Winnipeg, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by AECOM in *Pembina-Ducharme Culvert Study Report - Appendix A: Geotechnical Report* (2011).

Background

At the Pembina-Ducharme Culvert Replacement project in Winnipeg, the objective was to bring the slope FOS to 1.5, produce a slope no steeper than 5H:1V, and reduce the length of the culvert as much as practical. Topsoil was identified to a depth of 0.3 m, consisting of black, dry to moist soil with trace organics. Below this was alluvial clay down to 225 masl. This clay was silty, with some sand, trace gravel and pockets of organics. The soil was brown to olive brown, firm to stiff and intermediate to high plasticity. Lacustrine clay was present below the alluvial clay and could be identified to 217 masl. The lacustrine clay was grey, homogeneous and highly plastic. Trace to some silt was present. This soil was generally moist, ranging from soft to firm in consistency. Finally, till was identified at an elevation of 216.7 masl and below. The till was predominantly silt, with variable amounts of sand, clay and gravel. It was light brown to brown in colour, wet, non-plastic, and dense to very dense. Back-analysis of the previous slope yielded the properties summarized in Table A-6.

Table A-6: Summary of the strength properties assigned to the materials present at the Pembina-Ducharme Culvert site.

Material	Unit weight (kg/m^3)	Friction angle (°)	Cohesion (kPa)
Clay (intact)	18	16	7
Clay (residual)	18	10	0
Gravel (shear key)	16.5	37	0
Gravel (2" down)	18	34	0

Remediation

The designers used Slope/W for the design analyses. A slope of 6H:1V was found to have a FOS nearest to unity, so this was selected as the slope for regrading with granular fill. To bring the FOS to 1.5, a MSE retaining wall, rock columns and a shear key were all considered. The shear key was selected for ease of construction within the existing 3H:1V slope. The MSE retaining wall would have required a deep foundation, while the rock columns were deemed too complicated and costly for such a project.

Optimization of the shear key focused on its dimensions and location. Shear key widths of 3 m and 4 m were considered at various locations along the slope. Global and local stability were investigated. The 4-m wide shear key founded at 224 masl and located 17 m from the crest of the slope was recommended from the results. On diagrams, the shear key is estimated to be 18 m long, 4 m deep and backfill topping out at 226.3 masl. The overlying regraded slope is approximately 830 m^2 in cross section, with a top elevation of 229.5 masl directly above the shear key.

The designers stated that the trench walls should be excavated near vertical if safe, and the backfill should consist of 150 mm down crushed limestone. The backfill should be placed in lifts using the bucket of an excavator. They also noted a possible need to undertake a staged excavation and to control groundwater. The designers also call for a compacted clay cap 600 mm thick in order to prevent surface water from infiltrating into the shear key. The clay was required to be placed in lifts no thicker than 200 mm and compacted to 95% of the standard proctor dry density. The slope specifications gave the option between using 150 mm or 50 mm down crushed limestone. Benching into the new slope was required to key in the fill. Finally, the shear key was required to be completed as the first step of the project.

For the excavation, the designers stated that cantilevered shoring could be undertaken to a depth of 4 m before bracing or tie backs would be necessary. They allowed or a FOS of 1.3 against base heave in the shear key, which would require the GWL be below 227 masl. The base stability required a FOS of 1.5, but was exceeded with a FOS of 2.8 using an undrained shear strength of 30 kPa and a unit weight of 18 kN/m^3 for the soil.

Performance of Repairs

No SI data was obtained for this site.

Lessons Learned

This case study serves as an example of the design process that is undertaken for granular shear keys. Several alternatives were considered and the reasoning for why a granular shear key was the best option was presented. The case study also touches on the specifications for the backfill material and the construction logistics.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:		*Landslide velocity class:*	
Depth of movement (m):	5	*Most recent rate (mm/yr):*	
Sheared material:	Silty alluvial clay		

Landslide Dimensions

Width (m):		*Length (m):*	
Height (m):		*Slope (°):*	

Monitoring Information

Movement first reported:		*Last inspected:*	
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	~2.5	*Surface water at toe (yes/no):*	
Seepage detected (yes/no):		*Surface water type:*	

Repair Specifications

Repair type:	Shear key	*Repair date:*	
Base width (m):	4	*Trench slope ratio (H:V):*	
Length (m):	18	*Drainage (yes/no):*	
Granular height (m):	2.3	*Backfill:*	Crushed limestone
Overburden (m):	3.2		
Overburden material(s):			

WP12 – *Radcliffe Road*

Winnipeg, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by KGS Group in *Outfall Pipe Performance along Stabilized Riverbanks - Preliminary Assessment of Existing Sites in the City of Winnipeg* (2015).

Background

The Radcliffe Road Outfall is located next to the Red River in Winnipeg, Manitoba. The slope is comprised of alluvial silty clay overlying a roughly 1-m-thick layer of clay till and another 1-m-thick layer of lacustrine clay. These deposits all overly a silt till at depth. The riverbank is sloped at an angle of 3.9H:1V overall. The till has been measured at a depth of 11.6 m below the top of the bank. A pipe outlet daylights at an elevation of 222 masl, placing it at the level of the river.

Remediation

Two rows of rockfill columns were constructed to remediate this slope. They were completed on March 20, 2010. The columns were installed in the lower bank, at the edge of the river. The backfill was reported to have been vibro-compacted. The columns were then topped with a 0.6-m-thick clay cap and 0.6 m of rip rap. The rip rap covered the extent of the rockfill columns and was extended down the bank to the existing river bottom. Geotextile was placed at the outfall pipe but does not appear to have been placed elsewhere though, per the cross-sections that were available.

Performance of Repairs

Monitoring data was available from a single SI located in the upper third of the slope, about 3.3 m downslope of the scarp. Monitoring of SI-01 took place over 2.8 years and focused on the elevation of the outfall pipes (222-223 masl). A total of 4 mm of movement was reported. The rate for the first year was 1.47 mm/year, and 1.54 mm/year for the period 1-3 years after construction.

A look at the SI data plotted for SI-01 suggested a discrete shear zone at 6.1 mbgl. A rate of 0.17 mm/year was calculated for this depth, using the last two measurements plotted for SI-01. The SI data plotted for this site was analyzed in detail and it appears a rapid deceleration took place over the first 110 days following construction. This would suggest the shear key performed as intended.

Case Study Details

Landslide Information

Landslide type:	Planar slide	*Pre-support rate (mm/yr):*	
Trigger:		*Landslide velocity class:*	1 - Extremely Slow
Depth of movement (m):	6.1	*Most recent rate (mm/yr):*	0.17
Sheared material:	Alluvial silty clay		

Landslide Dimensions

Width (m):	80	*Length (m):*	33
Height (m):	7	*Slope (°):*	12.3

Monitoring Information

Movement first reported:		*Last inspected:*	January 4, 2013
SIs installed:	1	*SIs active last inspection:*	1
Piezos installed:	1	*Piezos active last inspection:*	

Groundwater Information

Groundwater level (mbgl):	0	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	River

Repair Specifications

Repair type:	Columns	*Repair date:*	March 20, 2010
Column diam. (m):	2.1	*Rows:*	2
Length (m):	61	*Drainage (yes/no):*	No
Layout:	Triangle	*Spacing (up x across [m]):*	2.685 x 3.1
Granular height (m):	6.1	*Backfill:*	Vibro-compacted rockfill
Overburden (m):	1.2		
Overburden material(s):	Clay cap (0.6 m) and riprap (0.6 m)		

Additional Site Information

Stratigraphy

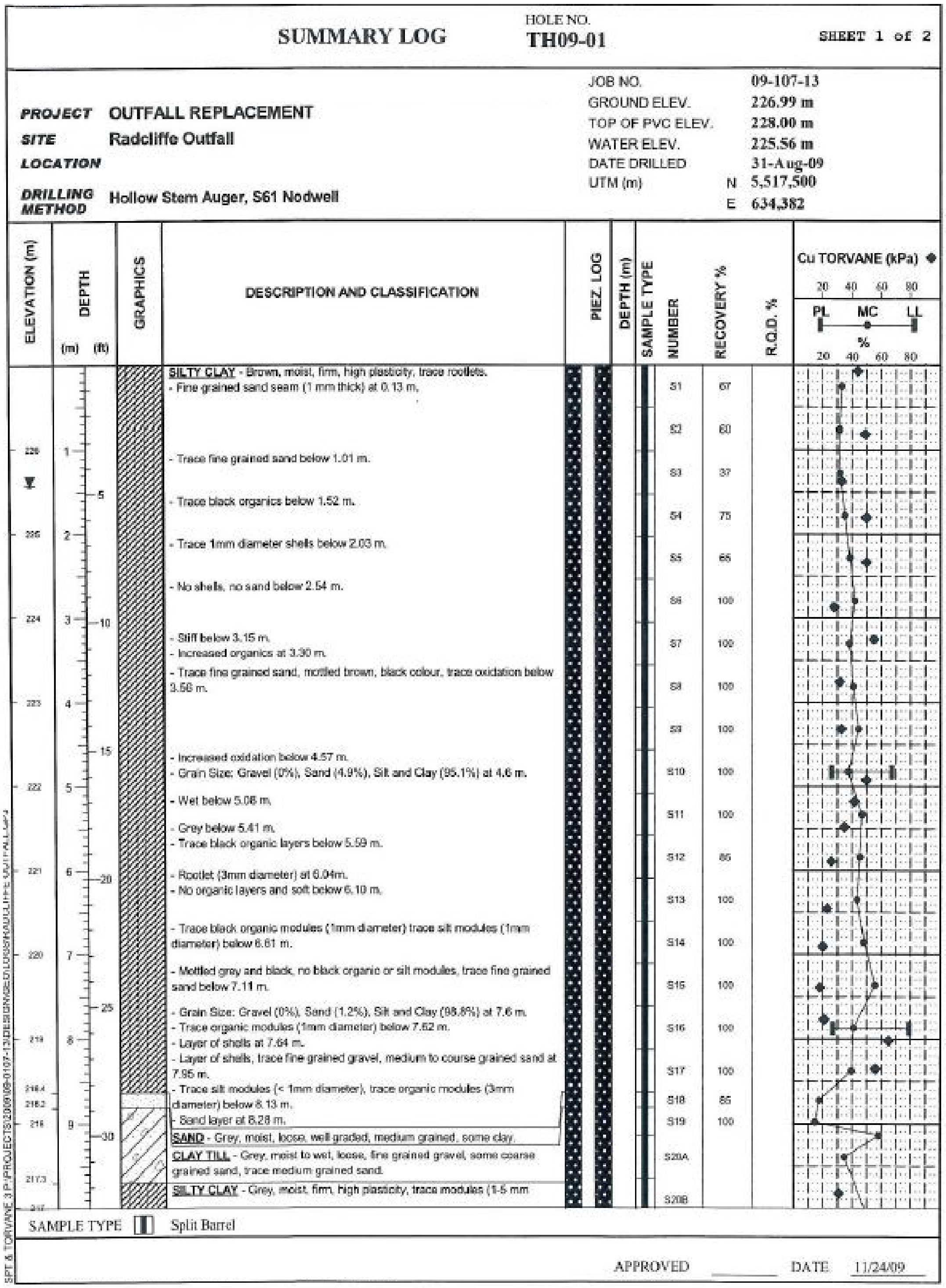


Figure A-64: Borehole log (Part 1/2) for the Radcliffe Road site. Reproduced from KGS Group (2015), *with permission.*

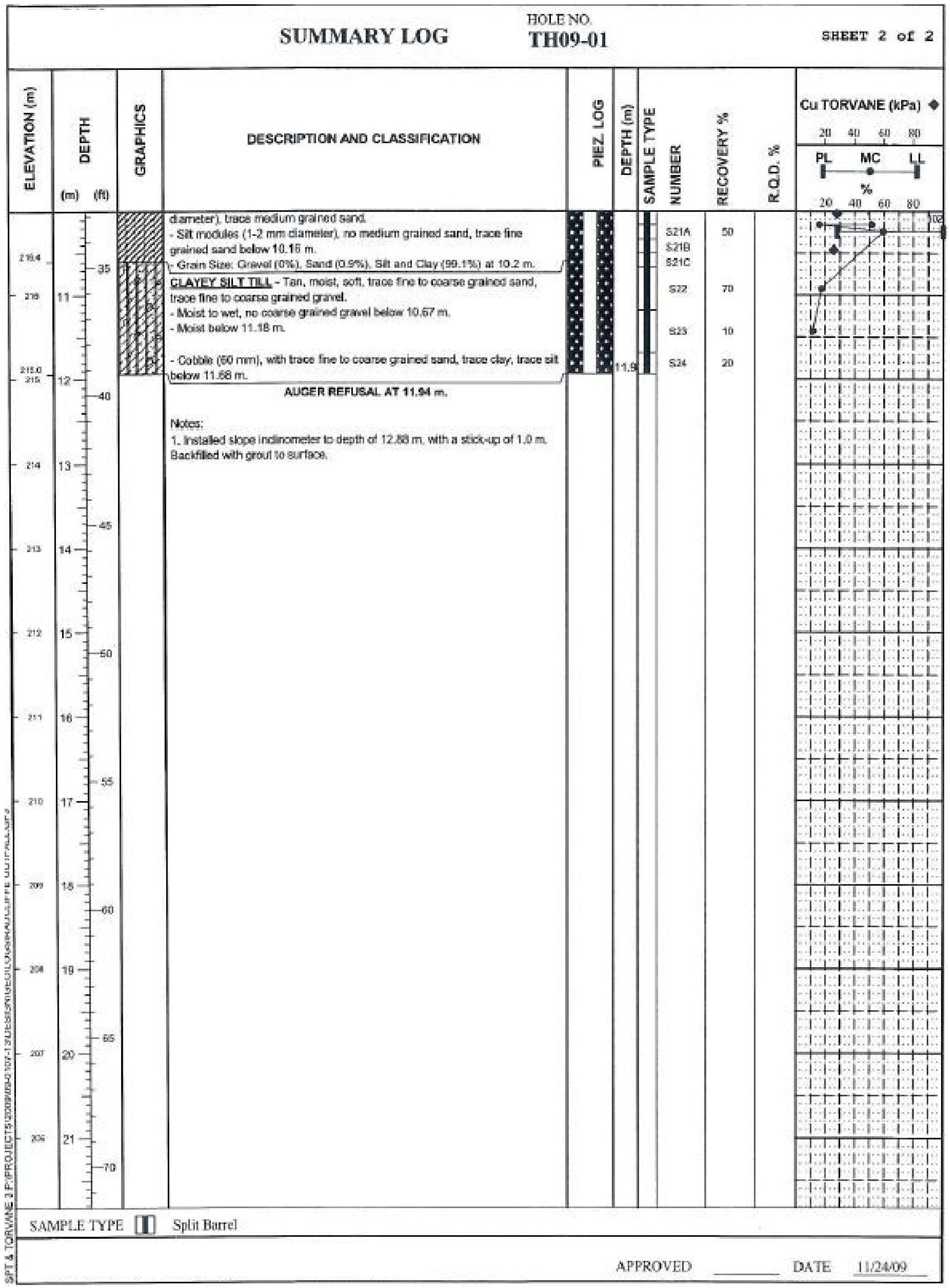


Figure A-65: Borehole log (Part 2/2) for the Radcliffe Road site. Reproduced from KGS Group (2015)*, with permission.*

Cross Section

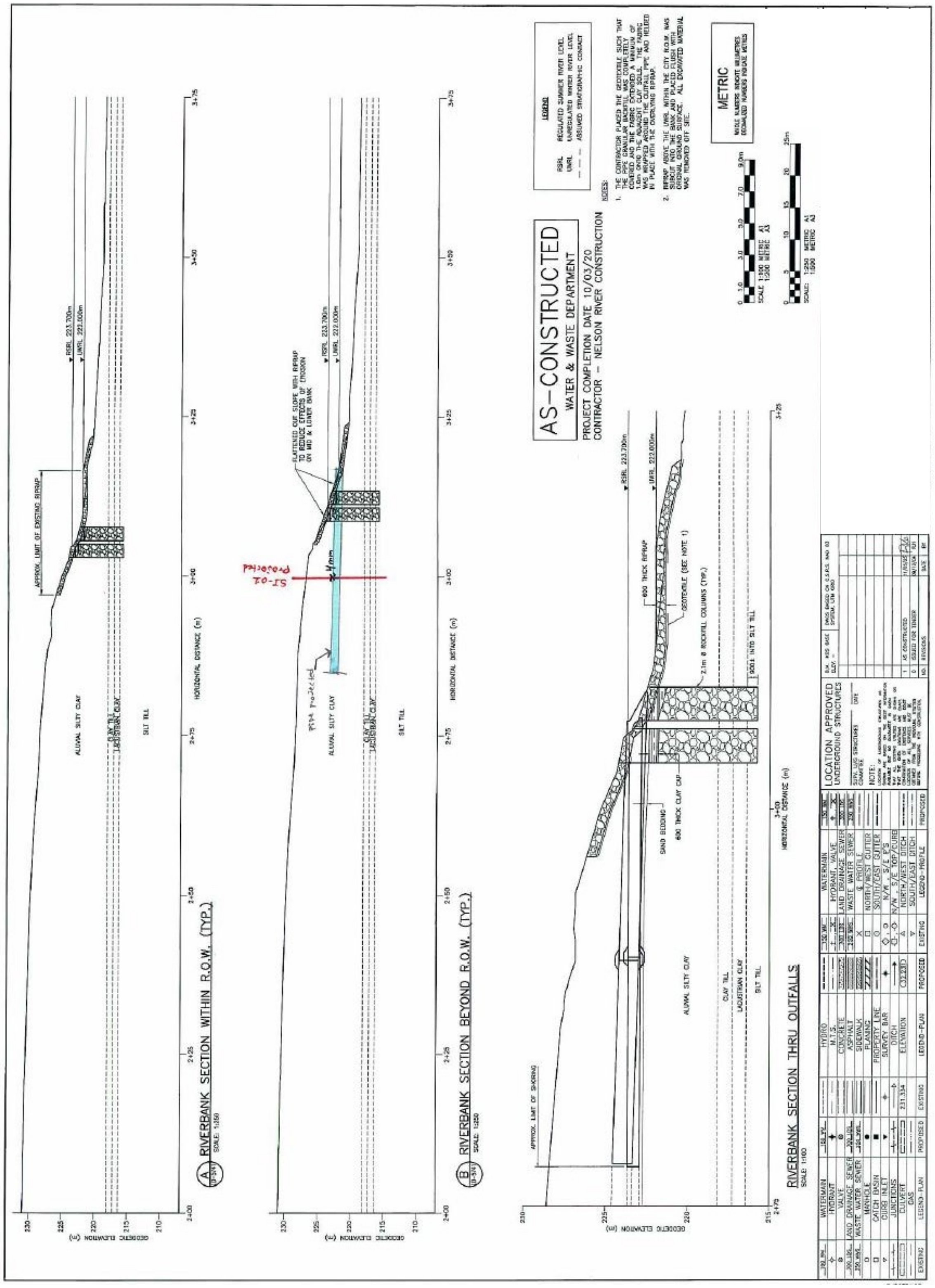


Figure A-66: Cross sections for the Radcliffe Road site. Reproduced from KGS Group (2015)*, with permission.*

Plan Map

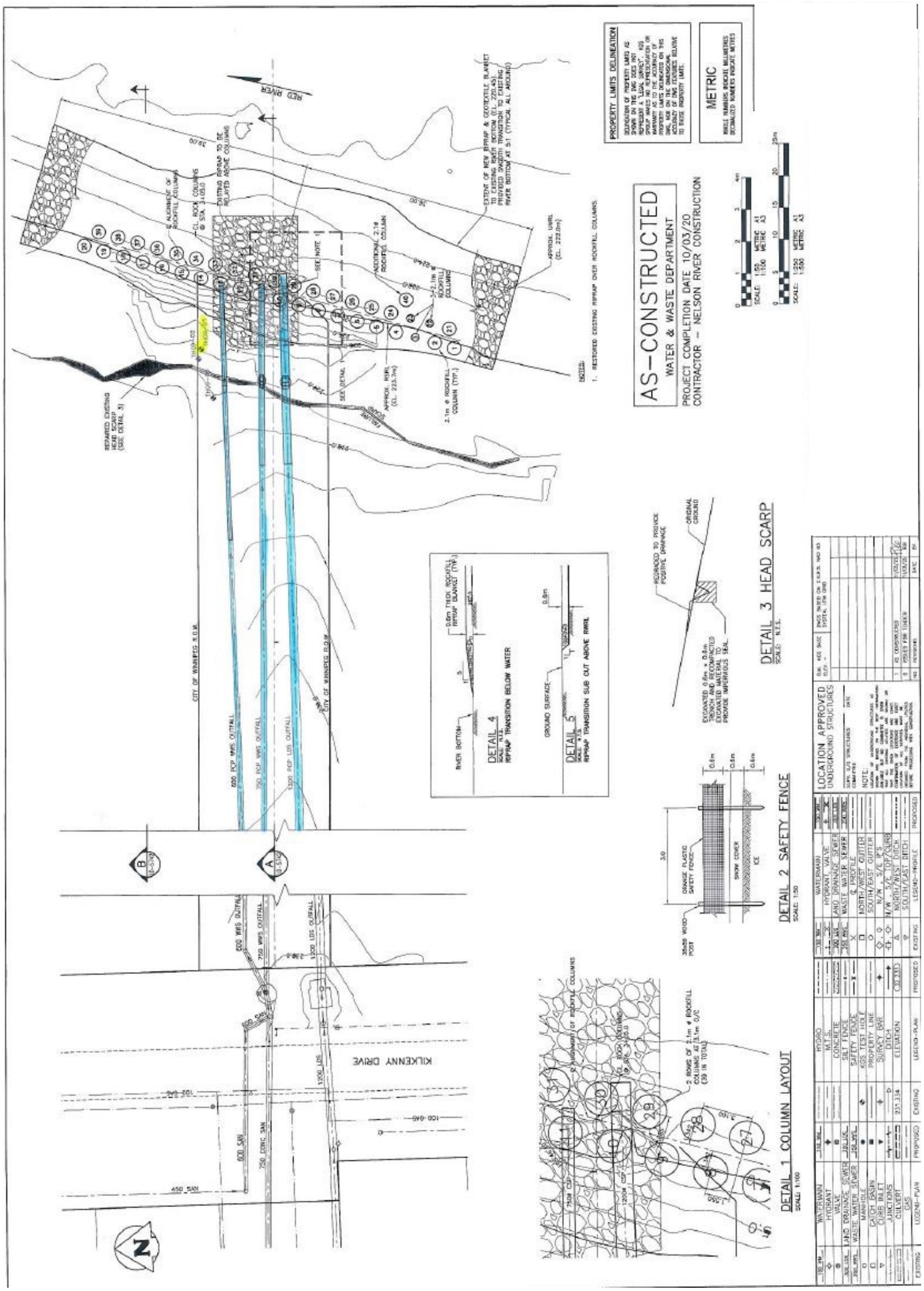


Figure A-67: Plan map of the repairs at the Radcliffe Road site. Reproduced from KGS Group (2015)*, with permission.*

WP13 – *Rue Despins*

Winnipeg, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by KGS Group in *Outfall Pipe Performance along Stabilized Riverbanks - Preliminary Assessment of Existing Sites in the City of Winnipeg* (2015).

Background

The Rue Despins site is located adjacent to the Red River in Winnipeg, Manitoba. The slope there is comprised of fill overlying lacustrine clay. Beneath this is a thin layer of clay till overlying a silt till. The slope is inclined at about 5H:1V overall. The top of the bank is about 14.7 m above the till. A pumping station is located above the slope and an outfall pipe outlet invert discharges at an elevation of 224 masl, roughly 10 m upslope of the river. Movement data was not available from before remediation.

Remediation

The slope was remediated in 2009, with the project finishing in March of that year. Two rows of rockfill columns were installed in the lower bank, and two more rows were installed in the upper bank. The backfill was reported to have been vibro-compacted upon being placed.

The installation of columns in both the upper and lower portions of this slope is not typical, although it is by no means uncommon. The till was approximately 9 m deep even at the edge of the river, so this site was ideal for rockfill columns. SIs installed in the mid and upper slope suggest movement was taking place at multiple depths, which may have influenced the decision to remediate both the upper and lower banks.

The columns were capped with 0.6 m of impervious clay. Those in the lower bank were then topped with a layer of geotextile and a 0.6 m thick rip rap blanket, to protect from river erosion and the discharge coming from the outfall.

Performance of Repairs

Data was available from two SIs at this site. SI-03 was in the upper bank, upslope of the rockfill columns. The focus of the monitoring program was on movement that could affect the outfall pipe, so movement in the zone lying between 225-226 masl was reported. This SI was monitored for 4.9 years, over which 2.5 mm of cumulative displacement was recorded. The monitoring period appears to have been exclusively from the post-construction period. In the first year after construction, a rate of 0.20 mm/year was reported. For the period spanning 1-3 years after construction, the rate had decreased to 0.11 mm/year. Finally, for 3+ years after construction, the rate had become almost negligible at 0.05 mm/year.

SI09-01 was located midslope, a few metres downslope of the upper bank rockfill columns but about 15 m upslope of the lower bank rockfill columns. This SI was monitored for 4.8 years, with a focus on the zone located 224-225 masl. A total of 3 mm of movement was reported, with a rate of 2.37 mm/year in the first year following construction. For Year 1-3, the rate had decreased to 1.30 mm/year, followed by 0.62 mm/year for the remainder of the monitoring program.

The decrease in the rate of movement in both SIs follows the expected trend, with the greatest decrease taking place in the earlier stages of monitoring.

Lessons Learned

This case study provides an example of a site where deep movement necessitated the installation of rockfill columns in both the upper and lower portions of the slope. This approach proved successful, resulting in decreasing rates of movement over the monitoring period.

Case Study Details

Landslide Information

Landslide type:	Rotational	*Pre-support rate (mm/yr):*	
Trigger:	Toe erosion	*Landslide velocity class:*	1 - Extremely Slow
Depth of movement (m):	9 – 15	*Most recent rate (mm/yr):*	0.05 – 0.62
Sheared material:	Clay fill and Lacustrine silty clay (CH)		

Landslide Dimensions

Width (m):	25	*Length (m):*	41
Height (m):	13	*Slope (°):*	11.3

Monitoring Information

Movement first reported:		*Last inspected:*	December 18, 2013
SIs installed:	3	*SIs active last inspection:*	
Piezos installed:	2	*Piezos active last inspection:*	

Groundwater Information

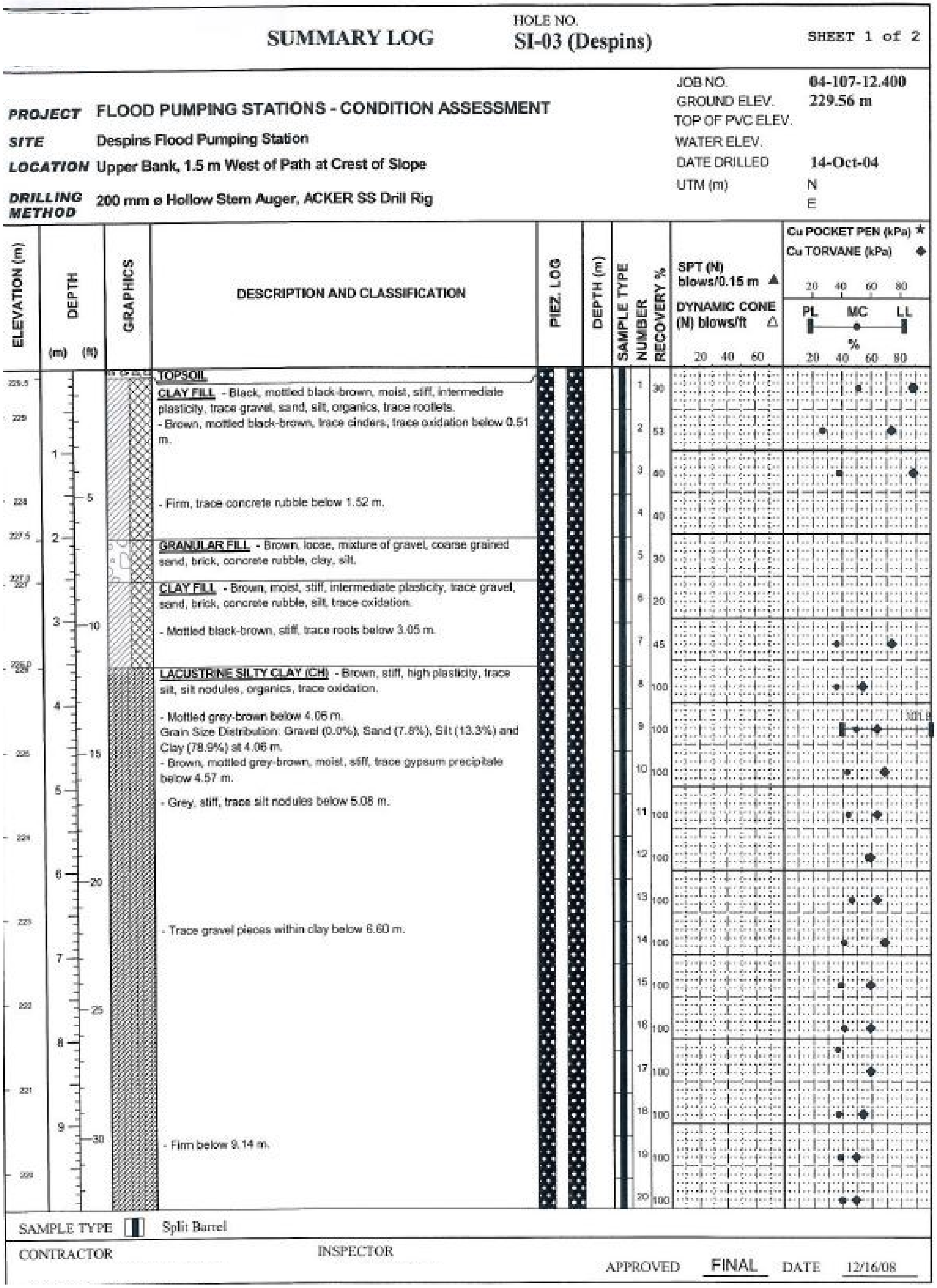
Groundwater level (mbgl):	2.5 – 3.6	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	River

Repair Specifications (Downslope / Upslope)

Repair type:	Columns	*Repair date:*	March 2009
Column diam. (m):	2.1	*Rows:*	2 / 2
Length (m):	42.5 / 40.5	*Drainage (yes/no):*	No
Layout:	Triangle	*Spacing (up x across [m]):*	2.5/2.7 x 3.1
Granular height (m):	8.7 – 13.4	*Backfill:*	Vibro-compacted rockfill
Overburden (m):	1.2		
Overburden material(s):	Impervious clay cap (0.6 m) and riprap (0.6 m) on lower bank, no rip-rap on upper bank		

Additional Site Information

Stratigraphy



SUMMARY LOG

HOLE NO. SI-03 (Despins)

SHEET 1 of 2

JOB NO. 04-107-12.400
GROUND ELEV. 229.56 m
TOP OF PVC ELEV.
WATER ELEV.
DATE DRILLED 14-Oct-04
UTM (m) N E

PROJECT FLOOD PUMPING STATIONS - CONDITION ASSESSMENT
SITE Despins Flood Pumping Station
LOCATION Upper Bank, 1.5 m West of Path at Crest of Slope
DRILLING METHOD 200 mm ø Hollow Stem Auger, ACKER SS Drill Rig

ELEVATION (m) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.15 m ▲ DYNAMIC CONE (N) blows/ft △ 20 40 60 | Cu POCKET PEN (kPa) ★ Cu TORVANE (kPa) ◆ 20 40 60 80 PL MC LL % 20 40 60 80

TOPSOIL

CLAY FILL - Black, mottled black-brown, moist, stiff, intermediate plasticity, trace gravel, sand, silt, organics, trace rootlets.
- Brown, mottled black-brown, trace cinders, trace oxidation below 0.51 m.

- Firm, trace concrete rubble below 1.52 m.

GRANULAR FILL - Brown, loose, mixture of gravel, coarse grained sand, brick, concrete rubble, clay, silt.

CLAY FILL - Brown, moist, stiff, intermediate plasticity, trace gravel, sand, brick, concrete rubble, silt, trace oxidation.

- Mottled black-brown, stiff, trace roots below 3.05 m.

LACUSTRINE SILTY CLAY (CH) - Brown, stiff, high plasticity, trace silt, silt nodules, organics, trace oxidation.

- Mottled grey-brown below 4.06 m.
Grain Size Distribution: Gravel (0.0%), Sand (7.8%), Silt (13.3%) and Clay (78.9%) at 4.06 m.
- Brown, mottled grey-brown, moist, stiff, trace gypsum precipitate below 4.57 m.

- Grey, stiff, trace silt nodules below 5.08 m.

- Trace gravel pieces within clay below 6.60 m.

- Firm below 9.14 m.

Number	Recovery %
1	30
2	53
3	40
4	40
5	30
6	20
7	45
8	100
9	100
10	100
11	100
12	100
13	100
14	100
15	100
16	100
17	100
18	100
19	100
20	100

SAMPLE TYPE Split Barrel

CONTRACTOR INSPECTOR

APPROVED FINAL DATE 12/16/08

Figure A-68: Borehole log (Part 1/2) for the Rue Despins site. Reproduced from KGS Group (2015)*, with permission.*

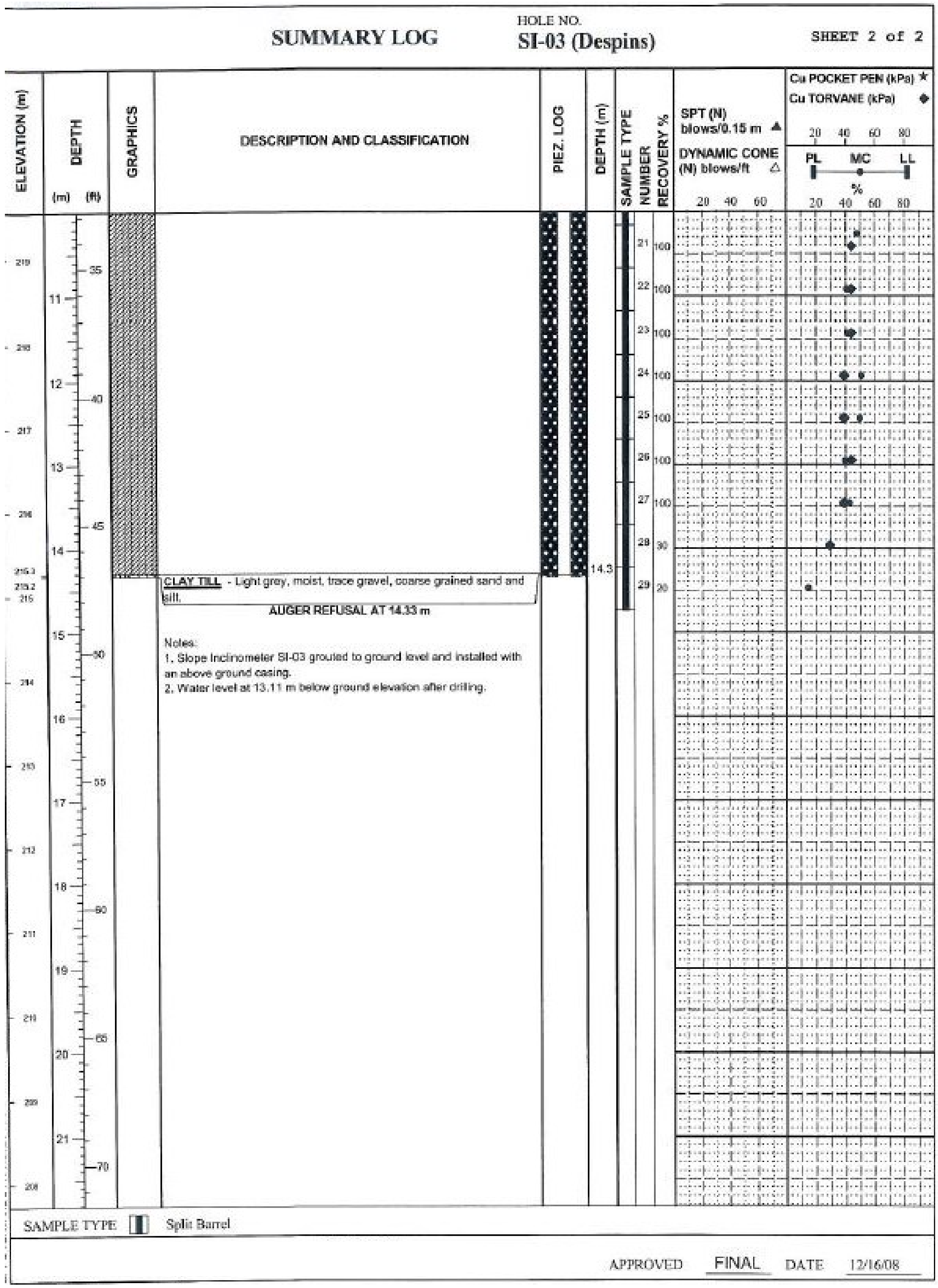


Figure A-69: Borehole log (Part 2/2) for the Rue Despins site. Reproduced from KGS Group (2015)*, with permission.*

Cross Section

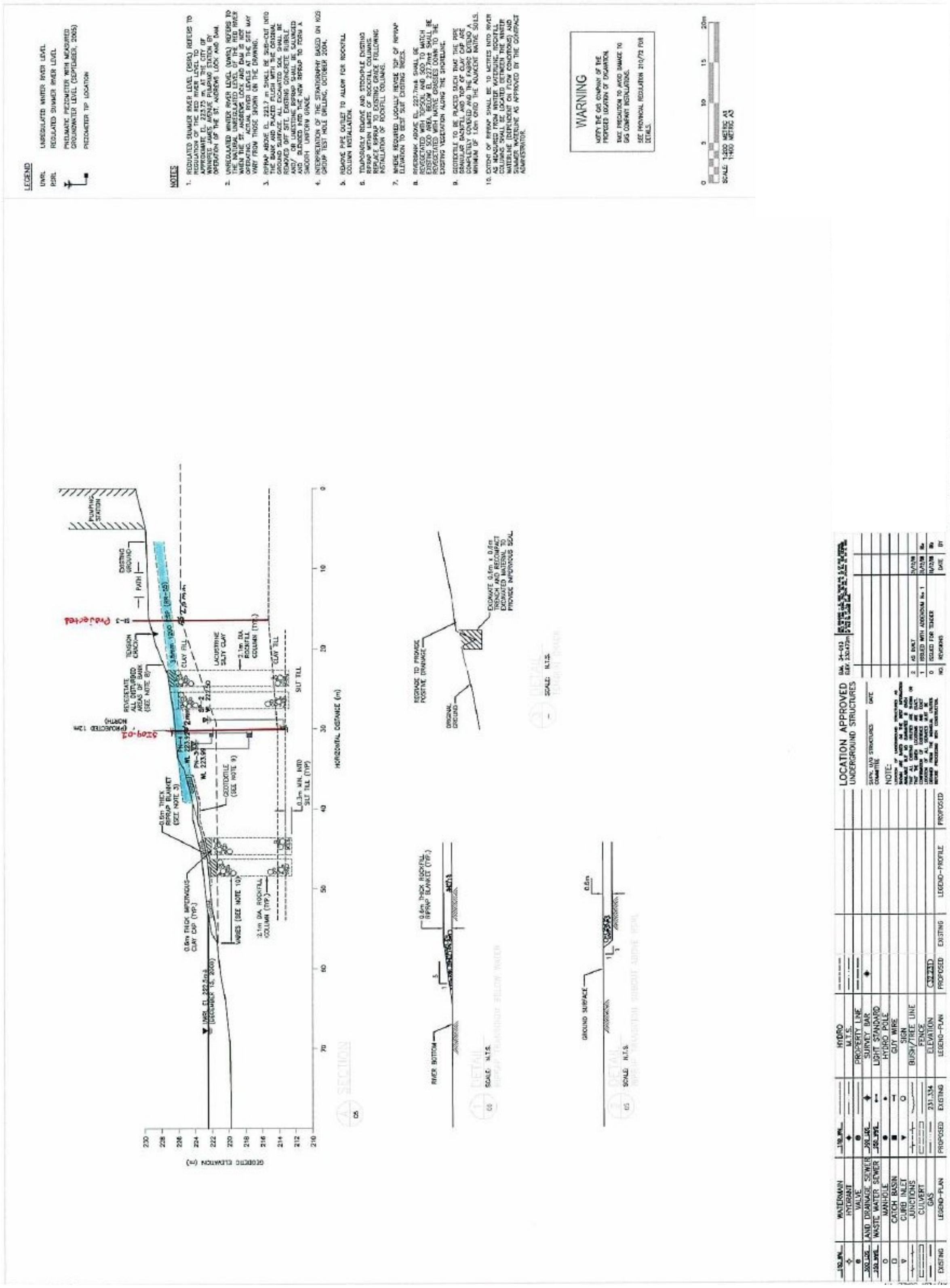


Figure A-70: Cross sections of the repairs at the Rue Despins site. Reproduced from KGS Group (2015)*, with permission.*

Plan Map

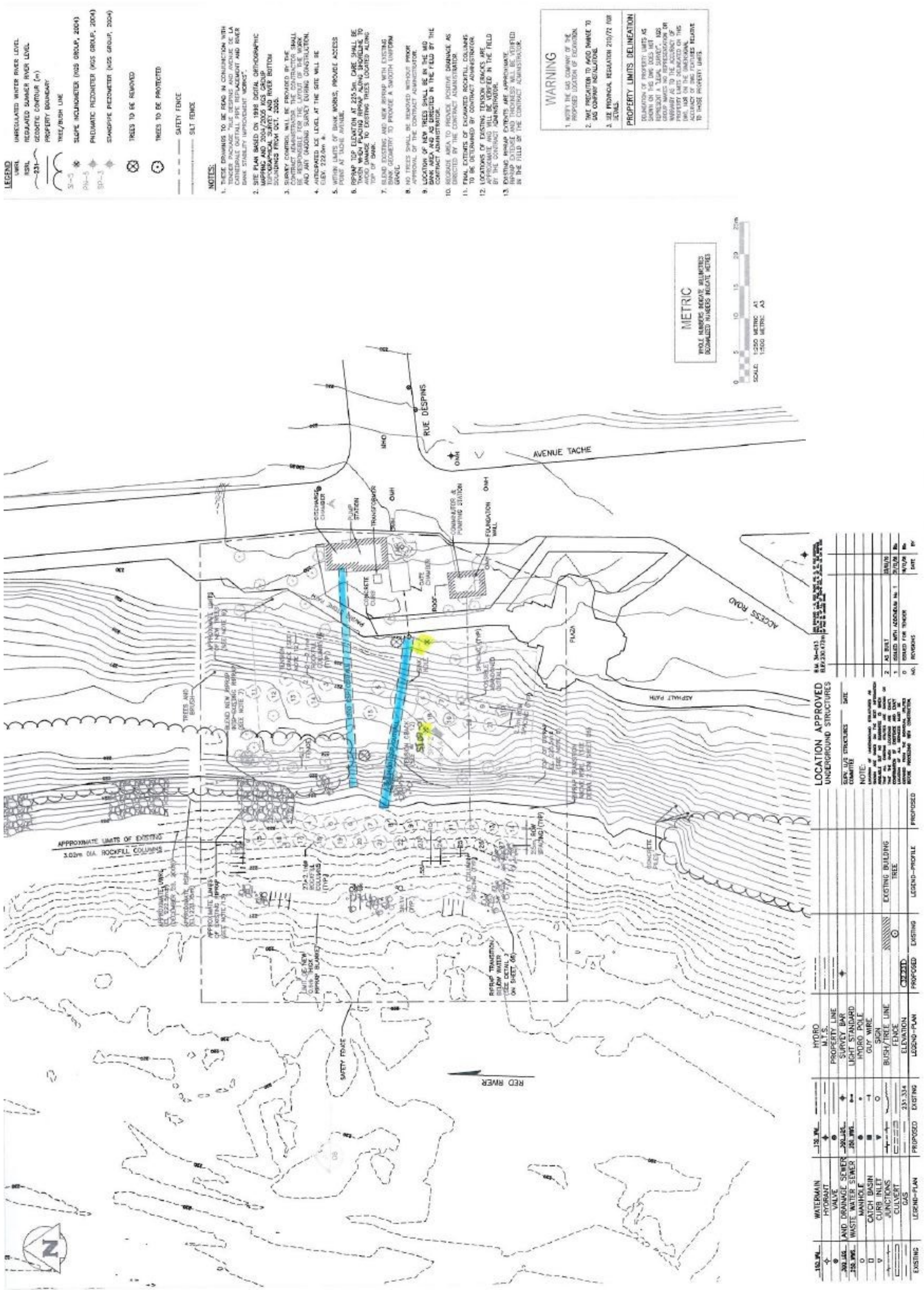


Figure A-71: Plan map of the repairs at the Rue Despins site. Reproduced from KGS Group (2015)*, with permission.*

WP14 – *Rue Dumoulin*

Dumoulin Avenue at the Red River, Winnipeg, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by KGS Group in *Dumoulin Outfall Repairs (RR-58)* (2004).

Background

Dumoulin Outfall is a City of Winnipeg site which underwent repairs involving a granular shear key in early 2004. The shear key construction was undertaken as part of the outfall replacement and riverbank stability improvement works that were performed on the east bank of the Red River at the Dumoulin Pumping Station. The work was performed per the City of Winnipeg Tender No. 646-2003.

A baseline for slope movement was established using a slope inclinometer, BH102-SI1, installed May 15, 2003 and initialized May 30, 2003. Movement appears to have been taking place in several zones; 1.6 mbgl, 5.2 mbgl, and between 8.5-9.5 mbgl. This SI was located approximately halfway up the slope, about 24 m upslope of the shear key. From initialization to October 25, 2003, approximately 4.9 mm of movement was recorded in the A+ direction. About 1.7 mm of movement was also recorded in the B- direction. The A+ direction was pointed 38 degrees in the downstream direction, relative to a direction perpendicular to the river. The B- direction was 90 degrees counter clockwise from the A+ direction. The resultant movement was calculated independently to be approximately 5.2 mm toward the river. The corresponding rate of movement was calculated to be 12.8 mm/year.

Data from only one SI was available, so it is difficult to classify the mechanism of this slide with certainty. Judging from the multiple discrete shear planes and the materials comprising the slope, the slope may be a rotational slide.

Remediation

The granular shear key designed for the Dumoulin Outfall repairs spanned 30 m along the riverbank and was installed between the summer water and winter ice levels of the Red River. From plan maps available for the site, it appears a 3.0-m-long gap, located 9.0 m from the south end of the repairs, was excluded from the shear key. This gap is roughly in line with one of the outfalls and a swale constructed for collecting discharge. The shear key was excavated 6 m wide at its base and was required to extend to a minimum depth of 0.6 m into the basal till. Approximately 2100 m^3 of rockfill was used to backfill the trench. Looking at the cross sections that were available for this repair, the shear key appears to be approximately 9.3 m deep. The trench walls are illustrated as being at a slope of 0.25H:1V, but a note indicates they were to be excavated as near to vertical as site conditions allowed.

In addition to the granular shear key, an erosion control blanket was constructed using 275 m^3 of rockfill, over the length of the shear key. The blanket was to be no more than 0.6 m thick and was to extend from the ordinary high water mark to approximately 8 m below the unregulated winter river level (221.8 masl). Rip rap was then to be placed along the riverbank above the unregulated winter river level. The rip rap was to be tapered such that the grades blended into the existing riverbank, to not impact erosion or stability.

Also, required in the tendered work was that any work on the lower bank was to be carried out during the winter months so that vehicular traffic and construction activity could be supported by the frozen ground.

The repairs described above can be considered typical of such projects for the City of Winnipeg. The granular shear key was located as close to the toe of the slope as was practical considering the presence of the river. The structure was keyed into the competent till layer below the riverbank surface soils. The use of an erosion control blanket and rip rap is also typical especially for sites near surface water bodies such as the Red River.

Performance of Repairs

Only an approximate date of completion is known. Post-repair rates of movement were calculated assuming the date the repairs were completed was March 22, 2004. From then until September 4, 2004, BH102-SI1 recorded approximately 29 mm of movement. It appears the rate of movement decreased rapidly over the first 6 months following construction. A rate of 0.5 mm/year was calculated using the last available data. This indicates the shear key was functioning successfully.

Lessons Learned

This repair is very typical of City of Winnipeg shear key projects. The repairs were successful and are an excellent example of routine work that has performed well.

Case Study Details

Landslide Information

Landslide type:	Rotational	*Pre-support rate (mm/yr):*	12.8
Trigger:		*Landslide velocity class:*	1 – Extremely Slow
Depth of movement (m):	1.6, 5.2, 8.3-9.5	*Most recent rate (mm/yr):*	0.5
Sheared material:	Clay fill, lacustrine clay		

Landslide Dimensions

Width (m):	35	*Length (m):*	52
Height (m):	9.3	*Slope (°):*	10.3

Monitoring Information

Movement first reported:		*Last inspected:*	September 4, 2004
SIs installed:	3	*SIs active last inspection:*	1
Piezos installed:	3	*Piezos active last inspection:*	1

Groundwater Information

Groundwater level (mbgl):	0	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):	No	*Surface water type:*	River

Repair Specifications

Repair type:	Shear key	*Repair date:*	2004
Base width (m):	6	*Trench slope ratio (H:V):*	0.25:1
Length (m):	30	*Drainage (yes/no):*	No
Granular height (m):	7.1	*Backfill:*	Rockfill
Overburden (m):	2.2		
Overburden material(s):	Clay cap (0.6 m), new rip rap (0.6 m), existing rip rap (~1 m)		

Additional Site Information

Stratigraphy (from BH102-SI1)

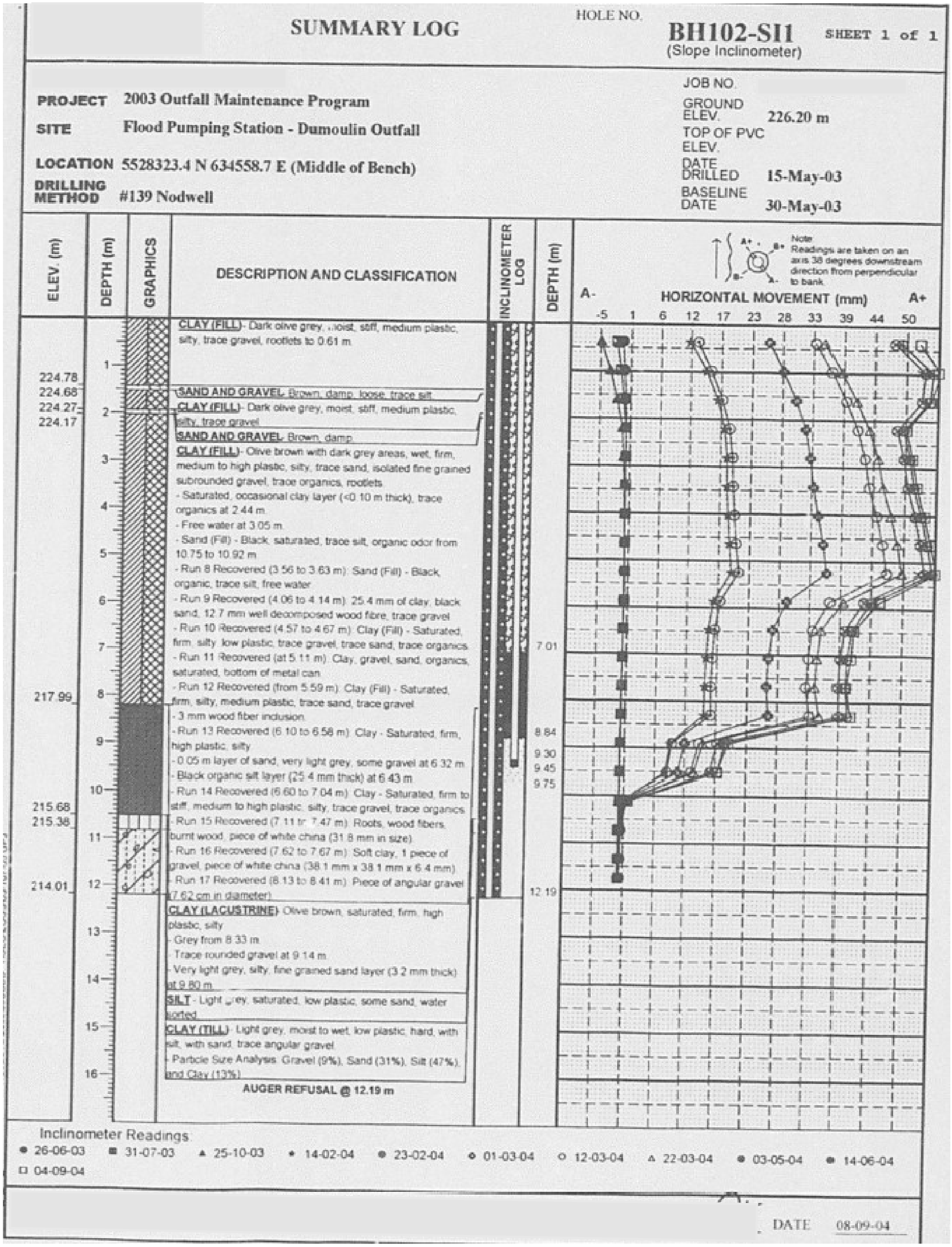


Figure A-72: Borehole log and SI data for the Rue Dumoulin site. Reproduced from KGS Group (2004)*, with permission.*

Cross section

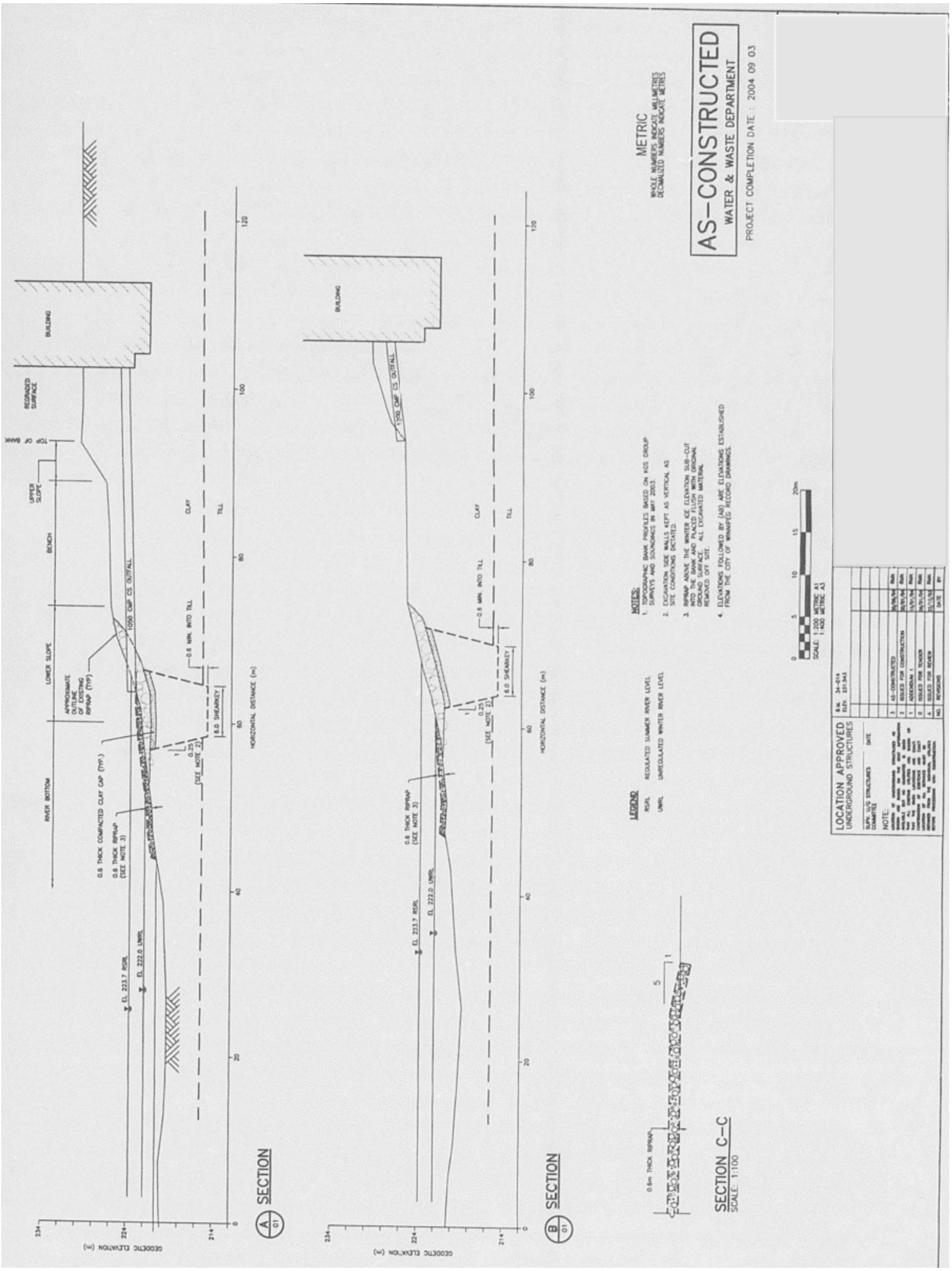


Figure A-73: Cross sections for the repairs at the Rue Dumoulin site. Reproduced from KGS Group (2004)*, with permission.*

Plan map

Figure A-74: Plan map of the repairs at the Rue Dumoulin site. Reproduced from KGS Group (2004)*, with permission.*

WP15 – *Rue la Verendrye Outfall*

Winnipeg, Manitoba

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by KGS Group in *Outfall Pipe Performance along Stabilized Riverbanks - Preliminary Assessment of Existing Sites in the City of Winnipeg* (2015).

Background

This site is located adjacent to the Red River in Winnipeg, Manitoba. The slope is comprised of alluvial silty clay overlying till. From the top of the riverbank, the till was 15.1 m deep. The slope was inclined at an angle of 4.4H:1V.

At this site, a pipe outlet invert is located at an elevation of 223 masl. This is roughly level with the regulated summer river level dictated by the City of Winnipeg. In site drawings, the pipe bedding and backfill details are stated to be unknown.

Remediation

The slope was remediated in March 2008 using three rows of rockfill columns. The project was completed on June 10, 2008. The columns were installed at the regulated summer river level, and were topped with 0.6-m-thick clay caps, followed by a 0.6-m-thick layer of rip rap. It was reported that the rockfill was not vibro-compacted. Rip rap was placed along the riverbank between the unregulated winter river level and the regulated summer river level (approximately 222 to 224 masl).

This repair is typical of rockfill column repairs by the City of Winnipeg. The choice to not vibro-compact the rockfill is not uncommon.

Performance of Repairs

Monitoring information was available for two SIs at this site: SI-01 and SI-02. Both SIs were monitored for 5.8 years after construction was finished. The monitoring program focused on movement between 223-224 masl, due to the presence of the outfall pipe at this elevation. In SI-01, no movement was recorded over the entire monitoring period. In SI-02, 12 mm of movement was recorded. In the first year, a rate of 3.92 mm/year was reported. For Year 1 to 3, the rate dropped to 2.15 mm/year. For 3 years and beyond, a rate of 0.92 mm/year was reported.

Judging from the reported data, the repairs appear to have been successful and performance is in line with other similar projects. A clear downward trend in the rate of movement can be noted.

Lessons Learned

This site is the last of 14 sites which comprised a study performed by KGS Group, based out of Winnipeg, Manitoba. The study compared displacement rates over time for sites that had received rockfill columns or trenched granular shear keys along the Red and the Assiniboine Rivers in Winnipeg. Overall, it appears the short-term performance (Year 1) of sites where the backfill had been vibro-compacted was better than those where vibro-compaction had not been pursued. However, the medium (Year 1 to 3) and long (Year 3+) term performance was identical for this study sample.

The performance of trenched shear keys and rockfill columns was also compared. Trenched shear keys performed better in the first year after construction, and more notably in the Year 1 to 3 period. Rockfill columns performed better than trenched shear keys though for the period exceeding three years.

Case Study Details

Landslide Information

Landslide type:		*Pre-support rate (mm/yr):*	
Trigger:	Toe erosion	*Landslide velocity class:*	1 - Extremely Slow
Depth of movement (m):	7 – 8, 12 – 12.5	*Most recent rate (mm/yr):*	0.92
Sheared material:	Alluvial Silty Clay		

Landslide Dimensions

Width (m):		*Length (m):*	
Height (m):	12.5	*Slope (°):*	12.8

Monitoring Information

Movement first reported:		*Last inspected:*	December 18, 2013
SIs installed:	2	*SIs active last inspection:*	2
Piezos installed:	2	*Piezos active last inspection:*	2

Groundwater Information

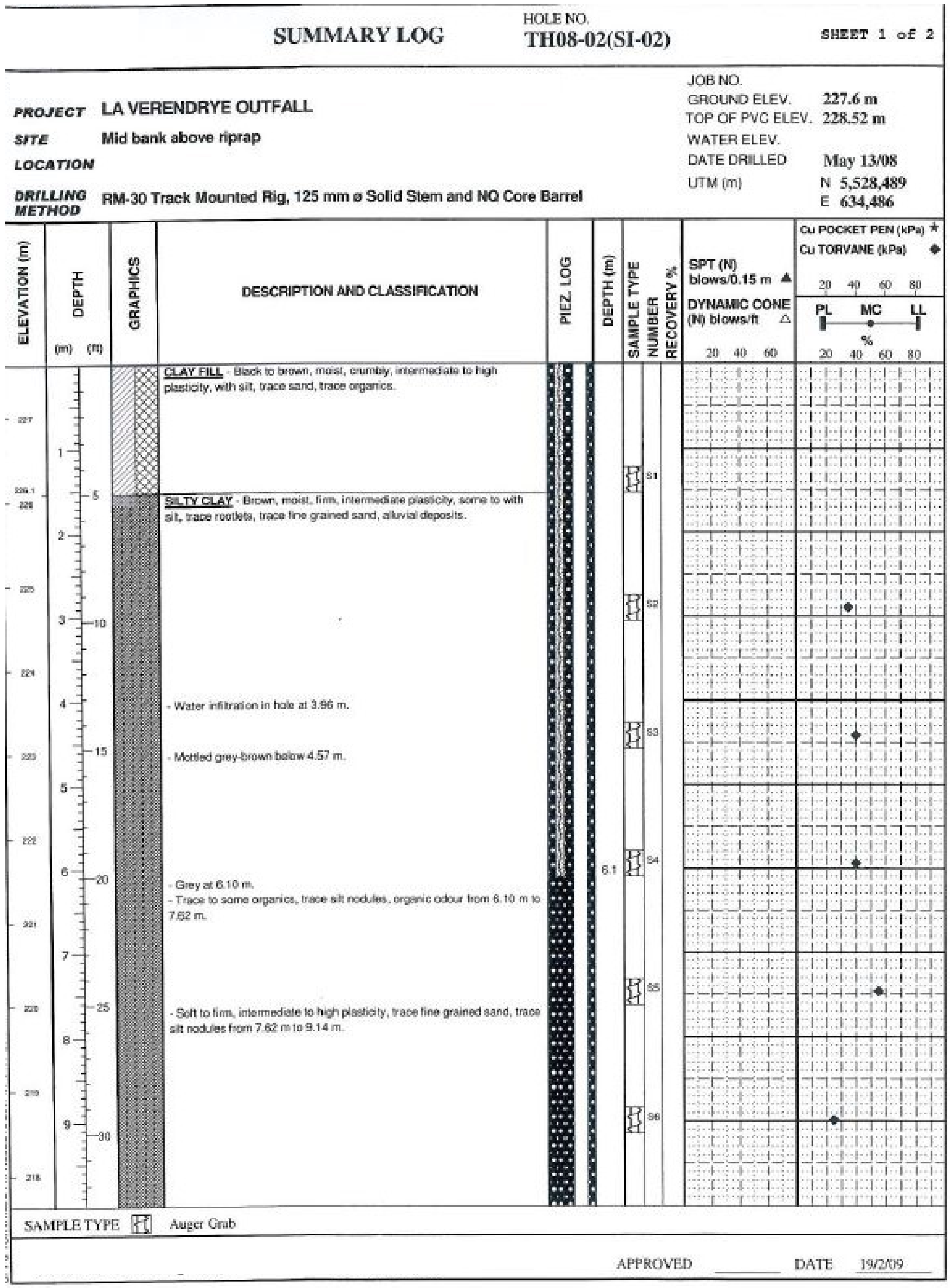
Groundwater level (mbgl):	4.0 – 4.9	*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	River

Repair Specifications

Repair type:	Columns	*Repair date:*	March 2008
Column diam. (m):	2.1	*Rows:*	3
Length (m):	49.6	*Drainage (yes/no):*	No
Layout:	Triangle	*Spacing (up x across [m]):*	2.79 x 3.36
Granular height (m):	7.5 – 8.0	*Backfill:*	
Overburden (m):	0.6, 0.6		
Overburden material(s):	Clay cap and riprap		

Additional Site Information

Stratigraphy


SUMMARY LOG

HOLE NO. TH08-02(SI-02)

SHEET 1 of 2

PROJECT LA VERENDRYE OUTFALL
SITE Mid bank above riprap
LOCATION
DRILLING METHOD RM-30 Track Mounted Rig, 125 mm ø Solid Stem and NQ Core Barrel

JOB NO.
GROUND ELEV. 227.6 m
TOP OF PVC ELEV. 228.52 m
WATER ELEV.
DATE DRILLED May 13/08
UTM (m) N 5,528,489 E 634,486

ELEVATION (m) | DEPTH (m) (ft) | GRAPHICS | DESCRIPTION AND CLASSIFICATION | PIEZ. LOG | DEPTH (m) | SAMPLE TYPE | NUMBER | RECOVERY % | SPT (N) blows/0.15 m ▲ DYNAMIC CONE (N) blows/ft △ 20 40 60 | Cu POCKET PEN (kPa) ★ Cu TORVANE (kPa) ◆ 20 40 60 80 PL MC LL % 20 40 60 80

CLAY FILL - Black to brown, moist, crumbly, intermediate to high plasticity, with silt, trace sand, trace organics.

SILTY CLAY - Brown, moist, firm, intermediate plasticity, some to with silt, trace rootlets, trace fine grained sand, alluvial deposits.

- Water infiltration in hole at 3.96 m.

- Mottled grey-brown below 4.57 m.

- Grey at 6.10 m.
- Trace to some organics, trace silt nodules, organic odour from 6.10 m to 7.62 m.

- Soft to firm, intermediate to high plasticity, trace fine grained sand, trace silt nodules from 7.62 m to 9.14 m.

6.1

S1 S2 S3 S4 S5 S6

SAMPLE TYPE Auger Grab

APPROVED ________ DATE 19/2/09

Figure A-75: Borehole log (Part 1/2) for the Rue de la Verendrye site. Reproduced from KGS Group (2015)*, with permission.*

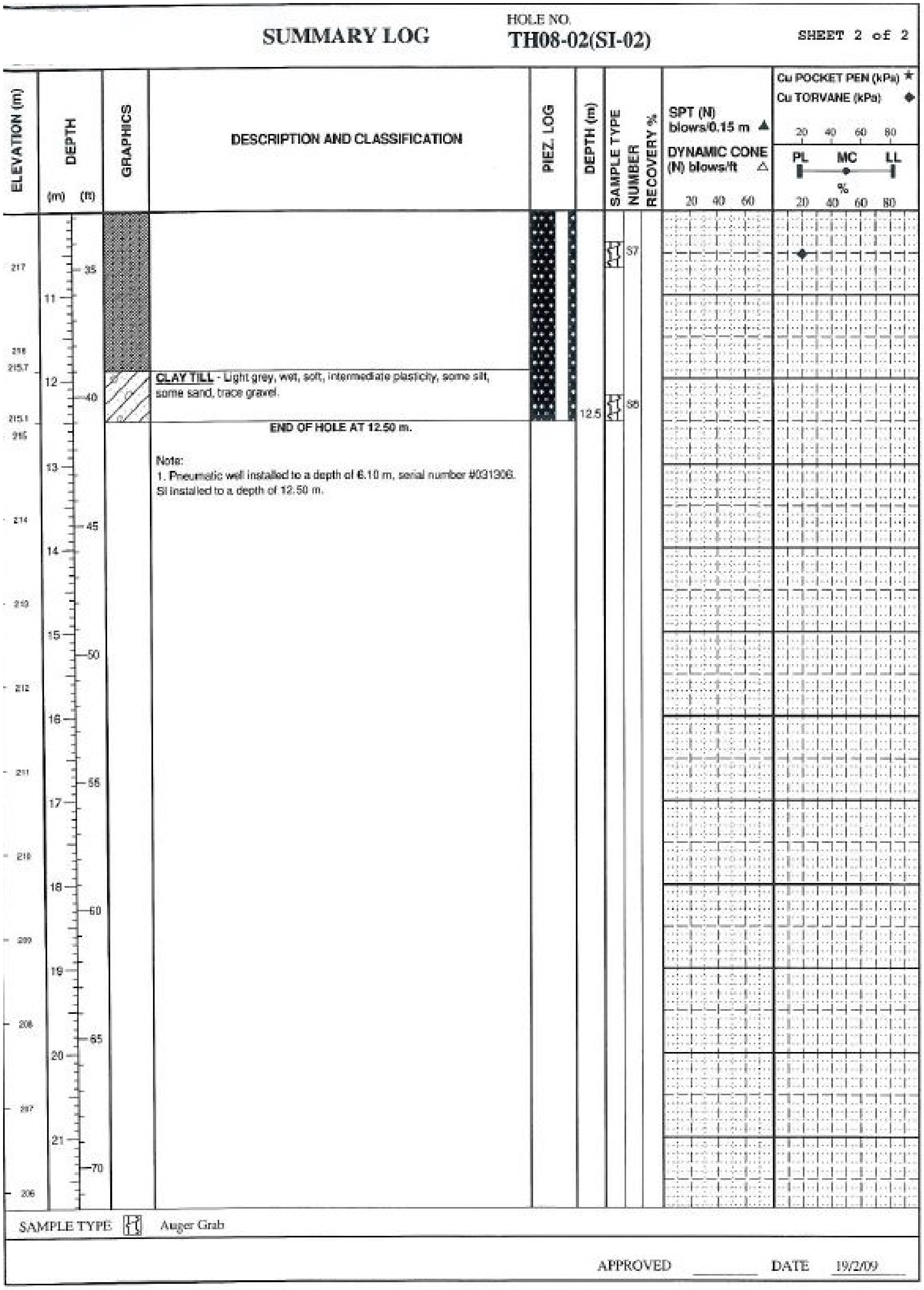


Figure A-76: Borehole log (Part 2/2) for the Rue de la Verendrye site. Reproduced from KGS Group (2015)*, with permission.*

Cross Section

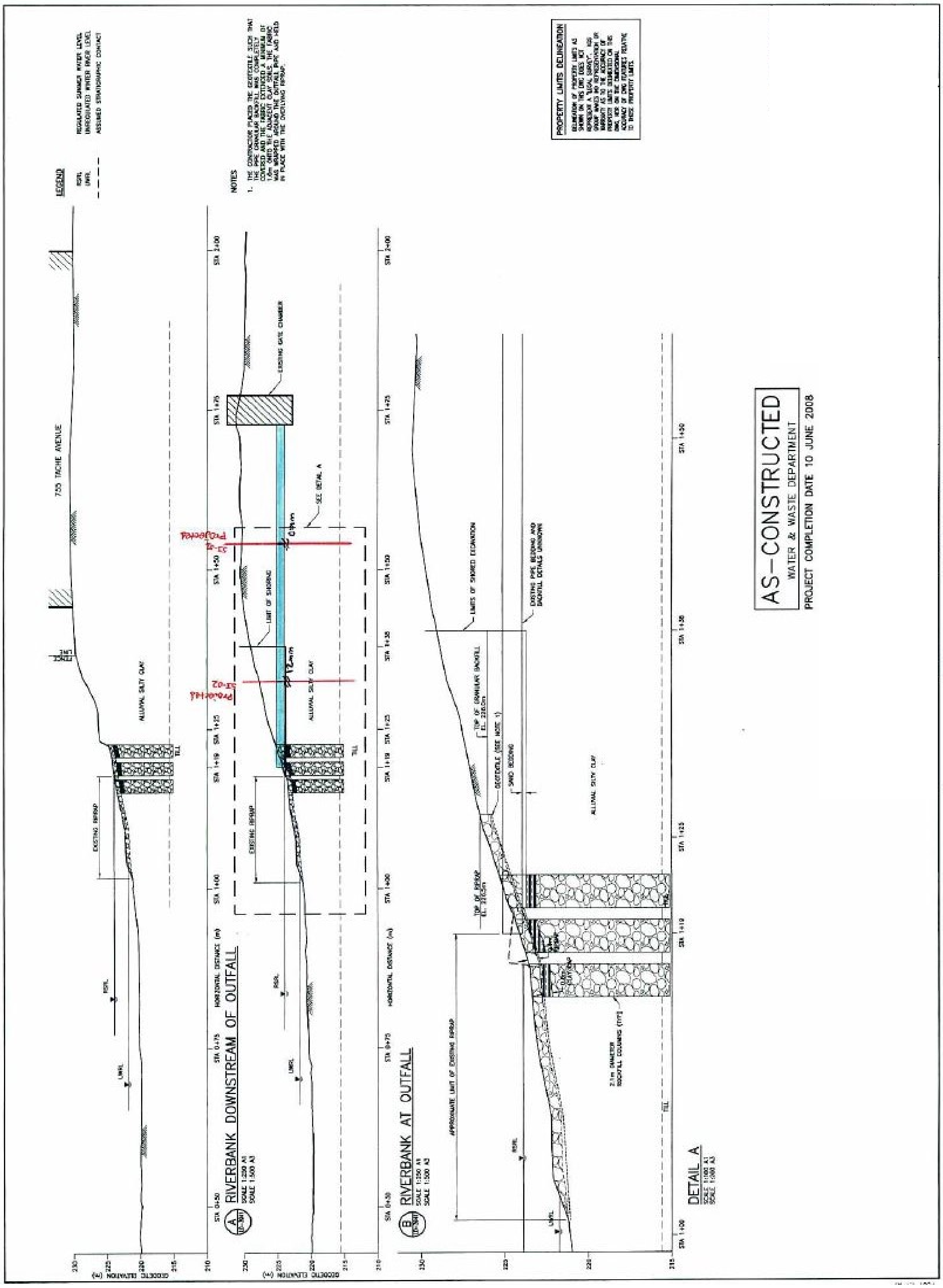


Figure A-77: Cross sections of the repairs at WP15. Reproduced from KGS Group (2015)*, with permission.*

Plan Map

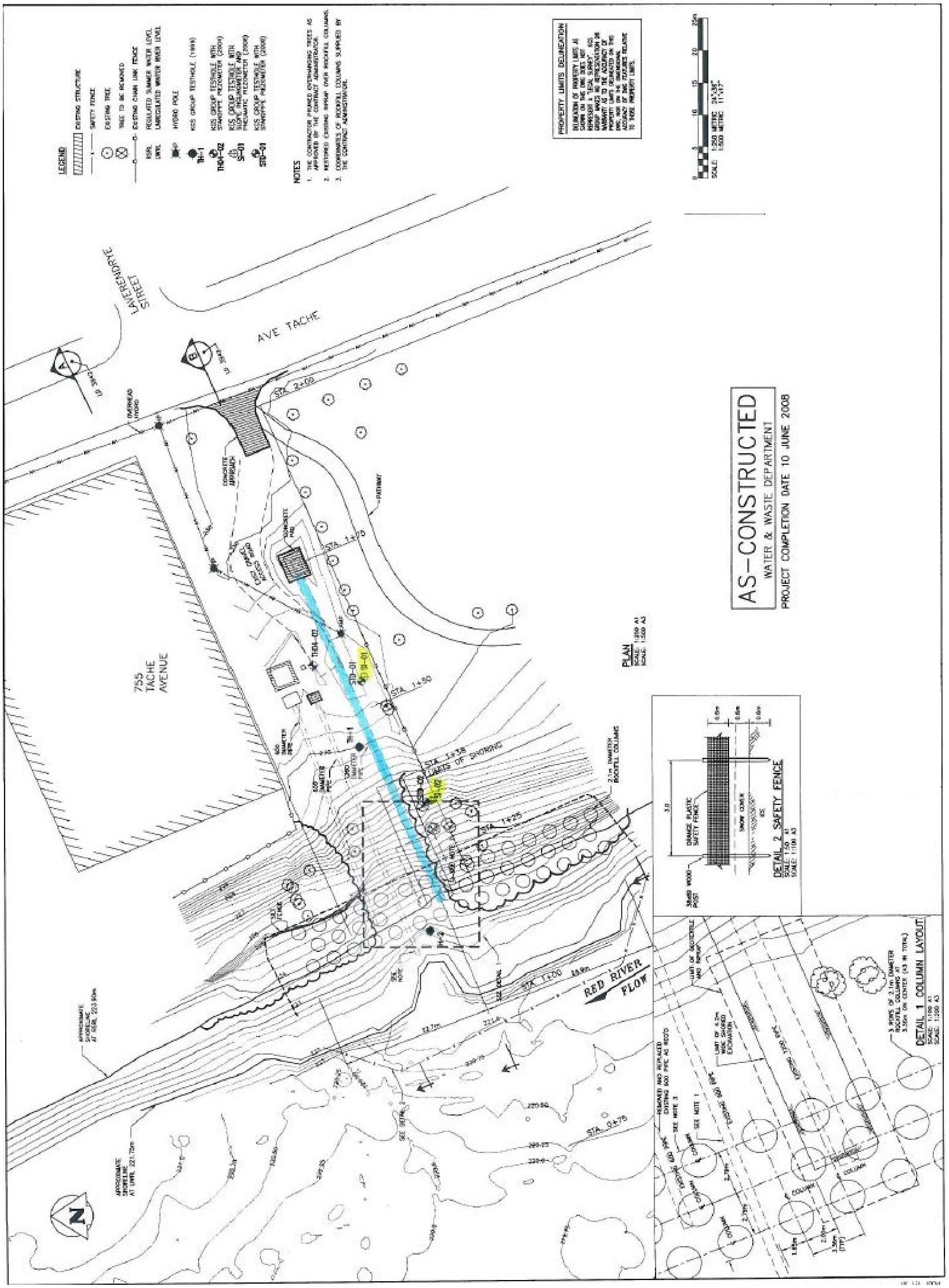


Figure A-78: Plan map of the repairs at WP15. Reproduced from KGS Group (2015)*, with permission.*

WP16 – *Seine River Siphon*

Winnipeg, MB

Case Study Summary

For the following case study, the entirety of the information that is presented was originally given by Yarechewski and Tallin in *Riverbank stabilization performance with rock-filled ribs/shear key and columns* (2003).

Background

The Branch I Aqueduct Seine River Siphon was one of two riverbank projects in Winnipeg that were the subject of a comparison between riverbank slope remedial measures typically adopted by the City of Winnipeg. The site is located next to the Lagimodière-Gaboury Historic Park along the Seine River, with the siphon running beneath it and across the channel.

The site has a long history. The Seine River Siphon was installed in July 1918 by means of an open trench across the river. Interlocking timber sheet piles were used to shore the excavation across the river channel, and timber piles with lagging were to shore the excavation where it extended up the riverbank. It was reported that the ground underwent significant movement during construction, leading to the decision to leave the timber shoring in place to provide additional support after construction was completed. A reinforced concrete mat and a timber mat were both reported to have been used to stiffen the pipe in certain spots too.

A 100 m stretch of riverbank at the site of the Seine River Siphon experienced instabilities in 2000, resulting in damage to infrastructure nearby but not the pipe. The site is located on an outside bend of the Seine River though so it was decided that preventative measures were required in anticipation of additional slope movement.

The riverbank at the site was described as consisting of lacustrine clay overlying dense till at a depth of 11-13 m. The slope was measured to rise 8-12 m above the river channel bottom and the failure surface was located within the lacustrine clay, running close to the till without extending into it. The residual shear strength of the lacustrine clay was investigated using laboratory tests and back analysis. The investigation yielded $\varphi' = 8\text{-}12°$ and $c' = 0\text{-}4$ kPa.

The Seine River Siphon slope was remediated between 2000-2001 using 286 rockfill columns. The riverbank had been monitored for a period of about one year before remediation. Monitoring continued for the 2.5 years leading up to the publication by Yarechewski and Tallin (2003). It is not known whether monitoring continued afterward.

Remediation

The design of remedial measures was achieved using limit equilibrium analysis, assuming a factor of safety equal to unity for the failed slope. A design factor of safety of 1.5 was targeted for the Seine River Siphon site. The use of a shear key was ruled out for this site due to the risk the associated excavation would pose to the pipe. Rockfill columns were selected as a viable alternative.

Remediation consisted of 286 rockfill columns, each spanning 2.1 m in diameter and extending 0.4 m into the till. Crushed limestone from bedrock mines northwest of Winnipeg was used as the backfill material. The crushed limestone is well-graded angular rock with 100% finer than 100 mm and 5% finer than 0.075 mm. Large-scale direct shear tests were used to measure the shear strength of the material, giving an internal angle of friction of 54°, but an angle of 45° was used for modeling.

Yarechewski and Tallin (2003) remark that the first compaction methods applied to the construction of rockfill columns used a large drop hammer. Improvements were necessitated when it became clear this method was both time-consuming and not very effective. A compaction tool consisting of a 15 m-long steel beam with four steel fins at the bottom was used for the Seine River Siphon project. The beam was vibrated with a vibratory pile-driving hammer at the top and it was found that dry densities of 17.5-18.8 kN/m^3 could be reached with frozen crushed limestone. In a loose state, the frozen crushed limestone dry density was approximately 16 kN/m^3.

Performance of Repairs

The Seine River Siphon site was monitored for 1 year prior to remediation, and at least 2.5 years after remediation was completed. Movement rates were measured at 30-42 mm/year in the year preceding remediation. Yarechewski and Tallin (2003) report that the rates accelerated to 70-2500 mm/year during construction. The first six months after construction was finished, rates reaching 130 mm/year were measured and total displacements of 40-58 mm were measured after the first full year. Significant improvements were observed in the second year, with rates ranging from 3-14 mm/year.

At the time of the publication by Yarechewski and Tallin (2003), the rock columns were reported to have experienced an average shear strain approaching 2%. The columns were found to have disrupted the typically abrupt displacement profile along the shear surface, instead forcing a more gradual profile.

Lessons Learned

It was noted by Yarechewski and Tallin (2003) that the internal angle of friction of granular material is dependent on bulk unit weight. They also noted that the compaction of the crushed limestone to increase the unit weight would result in greater internal angles of friction, thus improving design economies and long-term performance.

This case study also serves as an example of an alternative to shear keys in situations where the construction method cannot pose a risk to existing on-site infrastructure. The authors remark that this method is considered to provide more control over construction. Options for maintaining that control include the use of steel sleeves to case the drill holes when there is a high risk of sloughing, and moving drill hole locations to avoid less favourable conditions. Finally, the authors note that the rock column method is not limited by depth whereas the shear key method is.

Case Study Details

Landslide Information

Landslide type:	Rotational slide	*Pre-support rate (mm/yr):*	30 – 42
Trigger:	Toe erosion	*Landslide velocity class:*	2 - Very Slow
Depth of movement (m):	7	*Most recent rate (mm/yr):*	3 – 14
Sheared material:	Lacustrine clay		

Landslide Dimensions

Width (m):		*Length (m):*	
Height (m):	8 – 12	*Slope (°):*	8 – 12

Monitoring Information

Movement first reported:	January 1, 2000	*Last inspected:*	January 16, 2003
SIs installed:		*SIs active last inspection:*	
Piezos installed:		*Piezos active last inspection:*	

Groundwater Information

Groundwater:		*Surface water at toe (yes/no):*	Yes
Seepage detected (yes/no):		*Surface water type:*	River

Repair Specifications

Repair type:	Columns	*Repair date:*	January 1, 2001
Column diam. (m):	2.1	*Rows:*	
Length (m):		*Drainage (yes/no):*	
Layout:	Triangle	*Spacing (up x across [m]):*	
Granular height (m):	3 – 5	*Backfill:*	
Overburden (m):			
Overburden material(s):			

Additional Site Information

Cross Section

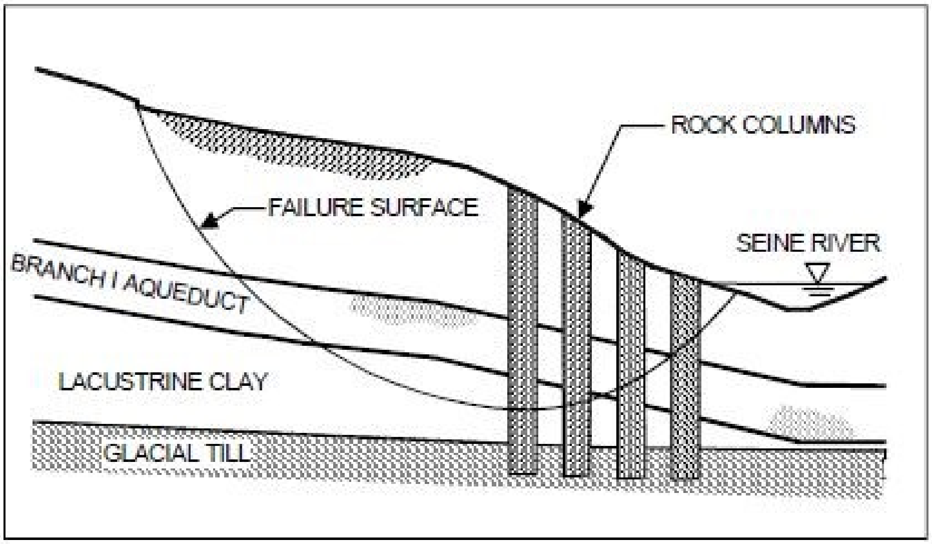


Figure 6. Branch I Aqueduct profile.

Figure A-79: Cross section of the Seine River Siphon site. Reproduced from Yarechewski and Tallin (2003)*, with permission.*

Plan Map

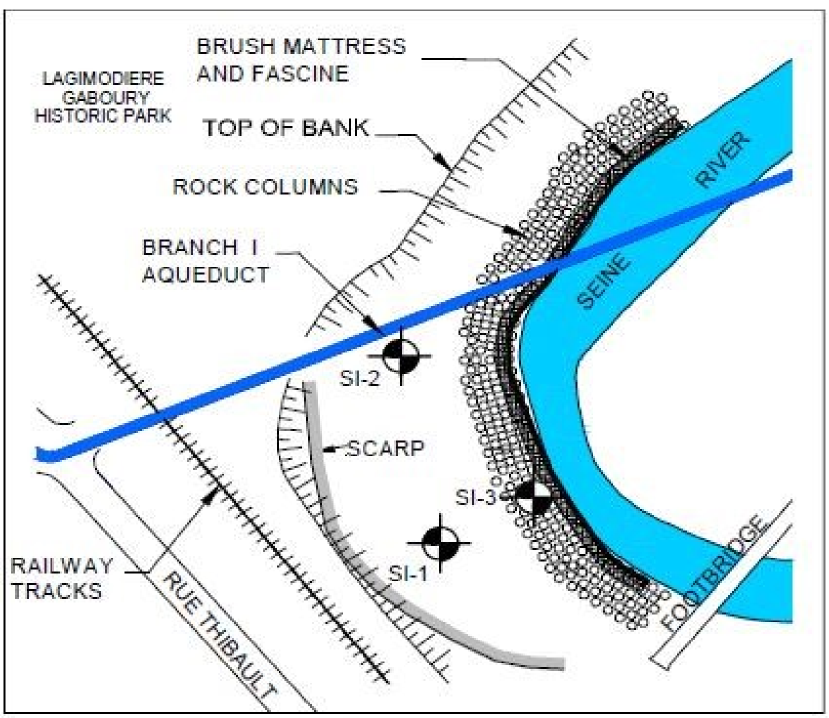


Figure 5. Branch I Aqueduct plan.

Figure A-80: Plan view of the Seine River Siphon site. Reproduced from Yarechewski and Tallin (2003)*, with permission.*

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