

Nine YGPC

*9th Young Geotechnical Professionals Conference
11 - 14 July 2012, St Kilda, Victoria, Australia*

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9th Young Geotechnical Professionals Conference,
11 – 14 July 2012
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NUMERICAL ANALYSIS OF THE CYCLIC DEFORMATION BEHAVIOUR OF A CAISSON FOUNDATION ON A SUBMERGED ROCKFILL SLOPE

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ABSTRACT

Numerical analysis using the finite element method was implemented with the aim of modeling the effects of cyclic loading on a caisson foundation on a submerged rockfill slope. The influence of the cyclic loading on the rockfill slope and the consolidation of the soil underlying the rockwall are discussed in this study. PLAXIS 2D Version 2010 with Hardening Soil Model was used to take into account the cyclic loading of the rockfill slope, while Settle3D was used to model the global consolidation settlements of the underlying rockwall foundation.

The caissons analysed support a Spoil Barge Unloader (SBU) which is a temporary berthing facility constructed on the offshore face of a marine rockwall. The SBU structure comprises a pre-fabricated steel and concrete working platform supported at its front end by three steel, sand-infilled caissons, founded on a submerged level bench area excavated into the front face of the rockwall, which in turn is founded on stiff to very stiff overconsolidated Pleistocene deposits. The SBU supports an excavator and dump truck which is filled with spoil unloaded from a barge. Monitoring results of the SBU during the first 40 uses were compared with the combined numerical analysis results (PLAXIS 2D and Settle3D). The results show that the observed deformations are in general agreement with the calculation strategy adopted.

1. INTRODUCTION

Many marine and shoreline structures incorporate rockfill material, often for its relatively large particle size and durability for coastal protection. In port and wharf environments, rockfill structures may also be used to support loads for other structures. Existing published literature on the compressibility of rockfill indicates that it exhibits highly stress dependent, inelastic and non-linear stress-strain behavior (Marsal, 1967; Marachi et al, 1972; Duncan et al, 1980), which is consistent with the "Hardening Soil model" implemented in Plaxis 2D.

The Hardening Soil model is more advanced than the traditional Mohr-Coulomb material model in that soil stiffness is stress dependent (i.e. decreasing stiffness with deviatoric loading) as well as offering a distinction between primary loading and unloading / reloading. The Hardening Soil model can therefore model irrecoverable plastic deformations (Plaxis, 2010). The Hardening Soil model has been used in a number of research papers to successfully predict the deformation behaviour of high rockfill dams (Ozkuzukiran et al, 2006; Soroush and Aghaei Araei, 2006). In this present study, it is used to model cyclic loading of a caisson foundation on a submerged rockfill slope.

2. SPOIL BARGE UNLOADER AND MARINE ROCKWALL

SPOIL BARGE UNLOADER

The Spoil Barge Unloader (SBU) is an 11 m x 15 m relocatable berthing facility constructed on the face of a marine rockwall in Queensland, and is used to load ashore dredged spoil generated from maintenance dredging of nearby waterways. The general arrangement of the SBU is shown in Figure 1. The SBU essentially comprises a steel-framed working platform with precast concrete decking supported at its front end by three steel tubular caissons and supported at its rear end by an L-shaped abutment with stem and strip footing sections. The working platform is also tied to two mooring piles at the upstream and downstream ends of the platform. The SBU is accessed by a 50 t Moxy articulated dump truck and a 30 t excavator working in tandem to load ashore the dredged spoil from a maintenance dredging vessel moored to the front of the platform.

The caissons are around 6.0 m in height and founded on a local bench excavated into the front face of the rockwall 2.75 m below low water datum. The central caisson is 3.5 m in diameter, whilst both side caissons are 3.0 m in diameter. Each caisson is seated on a 250 mm thick circular pad of crushed rock. The basal 1.5 m of the caisson is backfilled with crushed rock and the remainder with sand which is densified *in situ* and then capped with a 300 mm thick concrete slab, on which the platform deck beams directly bear. The sand infill and crushed rock is separated by a geotextile to prevent sand leakage.

The rear concrete abutment includes a 1.3 m wide strip footing founded at shallow depth below the rockwall crest, as well as a stem section which is keyed into the rockwall.

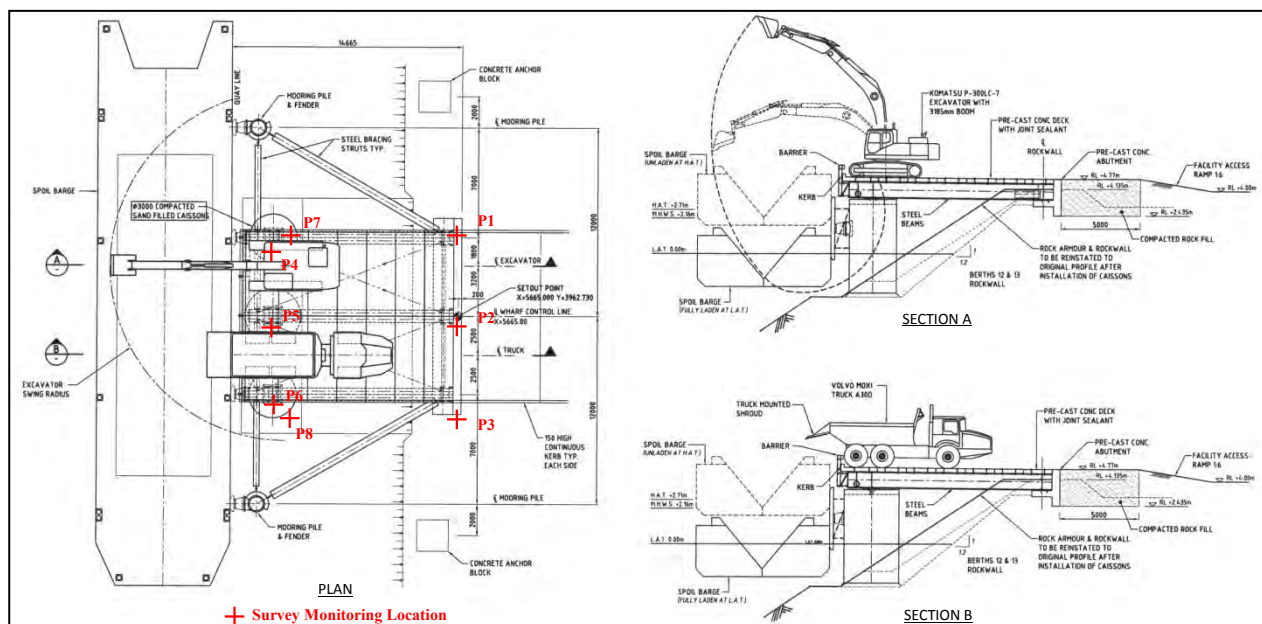


Figure 1: SBU general arrangement

The foundation working loads for the SBU structure are shown in Table 1 and include buoyancy effects for the submerged parts of the structure.

Table 1: SBU working loads

	Caisson Support Load (kN)		Rear Abutment Strip Footing Load (kN/m)
	Central	Side	
Dead Load	1332	931	51
Imposed Plant Load*	814	408	127
Total Live Load	2146	1339	178

Note: *Imposed Plant Load incorporates a dynamic load factor of 2.0 for dynamic loading effects.

It should be noted that the live load is cyclic in nature, involving the mobilisation and demobilisation of the Moxy and excavator with each use. The imposed plant loads include dynamic effects due to the dumping of spoil material into the Moxy tray as well as the berthing / mooring load of the dredge vessel.

Construction of the SBU facility was completed on 22 October 2010. Beginning on 10 November 2010 it was typically used around five to ten times per week.

ROCKWALL

The marine rockwall is located adjacent to a large waterway and its main purpose is as a retention structure for dredged reclamation materials. The section of rockwall which supports the SBU is trapezoidal in shape and was constructed between November 2008 and January 2009 by end-dumping of rockfill on to an underwater dredged bench (below a 1V:3H dredged slope) at around 13.5 m below low water datum. The rockwall founding bench is cut into alluvial Pleistocene deposits comprising 7 m of stiff to very stiff silty clay overlying medium dense to dense sand. The rockwall has a crest level 4.1 m above low water datum and a crest width of 5 m. The rockwall front face slope and rear face slope are 1V:1.5H and 1V:1.25H, respectively.

The rockfill material had been produced by quarry blasting, and then crushing to various smaller sizes in a crushing plant. Individual rock particles were sub-angular and sound. The rock material itself was a diorite containing accessory quartz. The particle size distribution curve for the rockfill material is shown in Figure 2.

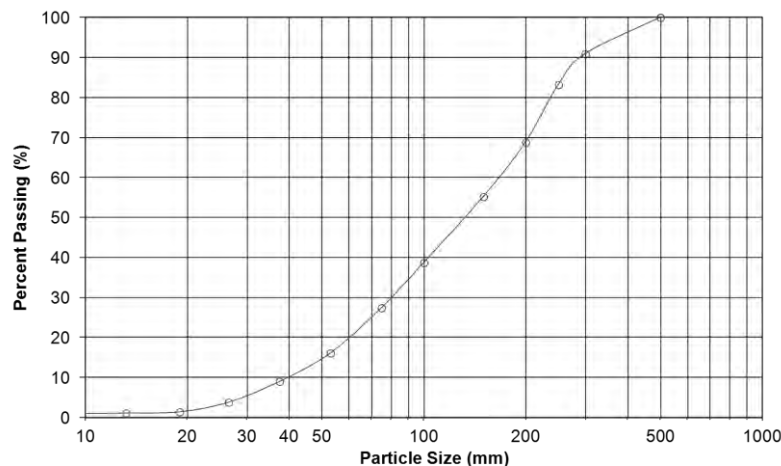


Figure 2: Particle size distribution for rockfill material used for the marine rockwall construction

3. SBU SETTLEMENT MONITORING

Considering that the SBU is a temporary facility and the support caissons are positioned near the front face of the rockwall which had been constructed underwater with no visual construction controls, it was proposed that settlement monitoring of the SBU be undertaken during the initial load cycles. Prior to loading of the structure, eight survey points were established on the SBU structure for monitoring the vertical movements of the platform. The survey monitoring point locations are shown in Figure 1 and are designated P1 to P8. The survey monitoring points P1 to P3 coincide with the SBU rear strip footing, whilst the survey monitoring points P4 to P8 coincide with the SBU front caissons.

Survey monitoring was initially done on a weekly basis, however the frequency increased to twice-weekly after three weeks due to concerns about the ongoing settlement of the front caissons.

The results of the survey monitoring are shown in Figure 3(a) which plots actual settlement vs. time and Figure 3(b) which plots actual settlement vs. cumulative load cycles. The SBU front caissons settled around 75 mm to 80 mm due to the first live load cycle and continued to settle at a rate of 1 mm to 2 mm per load cycle and without any sign of deceleration. After 40 load cycles the total settlement of the front caissons was 115 mm to 140 mm. The SBU rear footing settled initially around 40 mm to 60 mm, however did not appear to experience the excessive ongoing settlements associated with the front caissons. The rate of settlement of the rear footing was 4 mm per month.

Due to the magnitude of settlements occurring during the first 40 usages of the SBU, operation of the structure was suspended on 21 December 2010. Settlement monitoring continued over the subsequent month, with recorded settlements for the front caissons and the rear footing being 1 mm to 2 mm.

4. SBU CAISSON SETTLEMENT ANALYSES

The vertical deformation of the caissons measured by the *in situ* survey monitoring points is considered to be the sum of two different components of movement, namely:

1. Localised deformation of the rockwall due to cyclic nature of the SBU live load applications;
2. Ongoing global consolidation settlements of the underlying Pleistocene deposits on which the rockwall is founded.

Two separate analyses were therefore carried out in order to evaluate the respective contribution of each settlement component to the deformations, namely:

1. Finite element analyses to quantify rockwall deformation due to cyclic loading
2. Consolidation settlement analyses to quantify residual global consolidation settlements.

The methods and parameters adopted in the analyses are described in the following sections.

CYCLIC DEFORMATION MODELING FOR THE SBU CAISSONS

Numerical analysis using the Hardening Soil model (PLAXIS 2D version 2010) was carried out in order to model the deformation behavior of the SBU caisson foundation and the rear footing. The material parameters adopted in the analysis are shown in Table 3. Material parameters for the rockfill are based on past experience with similar materials as well as published references, whilst material parameters for the insitu soils are based on geotechnical investigation.

An important parameter for the Hardening Soil model is the reference stress for stiffness (p^{ref}), which is the minor principal stress (σ'_3). Within the marine rockwall the minor principal stress will increase with depth and therefore it was necessary to divide the rockwall into several horizontal sublayers and assign each a different p^{ref} equivalent to $K_0 \times \sigma_1$, where the initial coefficient of horizontal earth pressure at rest (K_0) has a value of 0.5.

The cyclic nature of the live load was simulated by repeating alternating dead load and then live load calculation phases. A total of 80 calculation phases were carried out (i.e. 40 cycles of loading / unloading).

Table 3: Finite element analysis (Plaxis) material models and parameters

Parameter	Symbol	Rockfill	Pleistocene Silty Clay	Pleistocene Sand	Unit
Material Model	<i>Model</i>	Hardening Soil Model	Mohr-Coulomb	Mohr-Coulomb	-
Type of Behaviour	<i>Type</i>	Drained	Drained	Drained	-
Unit Weight	γ	20	20	19	kN/m^3
Young's Modulus	E	-	15, 000	60, 000	kN/m^2
Secant Stiffness in Standard Drained Triaxial Test	E_{50}^{ref}	30, 000	-	-	kN/m^2
Secant Stiffness for Primary Oedometer Loading	E_{oed}^{ref}	39,000	-	-	kN/m^2
Unloading / Reloading Stiffness	E_{ur}^{ref}	180, 000	-	-	kN/m^2
Power for Stress-Level Dependency of Stiffness	m	0.5	-	-	-
Poisson's Ratio	ν	0.3	0.25	0.3	-
Cohesion	c_{ref}'	0.1	15	0.1	kN/m^2
Friction Angle	ϕ	40	30	36	<i>degrees</i>
Dilatancy Angle	ψ	10	0	0	<i>degrees</i>

GLOBAL CONSOLIDATION SETTLEMENT ANALYSES FOR THE MARINE ROCKWALL

Three-dimensional consolidation settlement analysis was undertaken (using Settle3D by Rocscience) in order to evaluate the residual ongoing consolidation settlement induced in the Pleistocene deposits underlying the rockwall due to the weight of the rockwall and the dead load of the SBU. Settle3D adopts a Boussinesq distribution to calculate the change in vertical stress due to external loads and vertical consolidation settlements are based on Terzaghi's one-dimensional consolidation equation (Rocscience, 2009). The material parameters adopted for the Settle3D analysis are shown in Table 4 and, where relevant, correspond to the finite element analysis material parameters.

Table 4: Consolidation settlement analysis (Settle 3D) material parameters

Parameter	Symbol	Rockfill	Pleistocene Silty Clay	Pleistocene Sand	Unit
Initial Overconsolidation Ratio	<i>OCR</i>	-	3	3	-
Normalised Compression Index	$Cc / (1+e_0)$	-	0.1	-	-
Young's Modulus	E	-	-	60, 000	kN/m^2
Poisson's Ratio	ν	-	0.25	0.3	-
Coefficient of Consolidation	c_v	-	10	-	m^2/yr

The settlement analyses take into account the stress history of the Pleistocene deposits by modeling the various construction activities (and associated time intervals) that occurred prior to the first usage of the SBU, including: dredging of the rockwall founding bench (November 2008); construction of the marine rockwall (November 2008 to January 2009); backfilling behind the rockwall (January 2009 to May 2009); excavation of the localised SBU caisson founding bench (September 2010); and construction of the SBU (22 October 2010).

5. RESULTS AND DISCUSSION

Predicted settlements using the combined results of PLAXIS and Settle3D analyses, and comparisons to the survey monitoring data are shown in Figure 3(a) and Figure 3(b). The results indicate that the predicted settlements are close to the measured data and fall within the upper and lower bounds of the survey monitoring data. It should be noted that the analysis results also incorporate the predicted immediate settlement of the 6 m high column of sand and crushed rock inside the tubular caissons, which was calculated separately (from elastic theory) to be 20 mm. A plot of the total displacement vectors for the front caissons and rear footing after 40 load / unload cycles are shown in Figure 3(c).

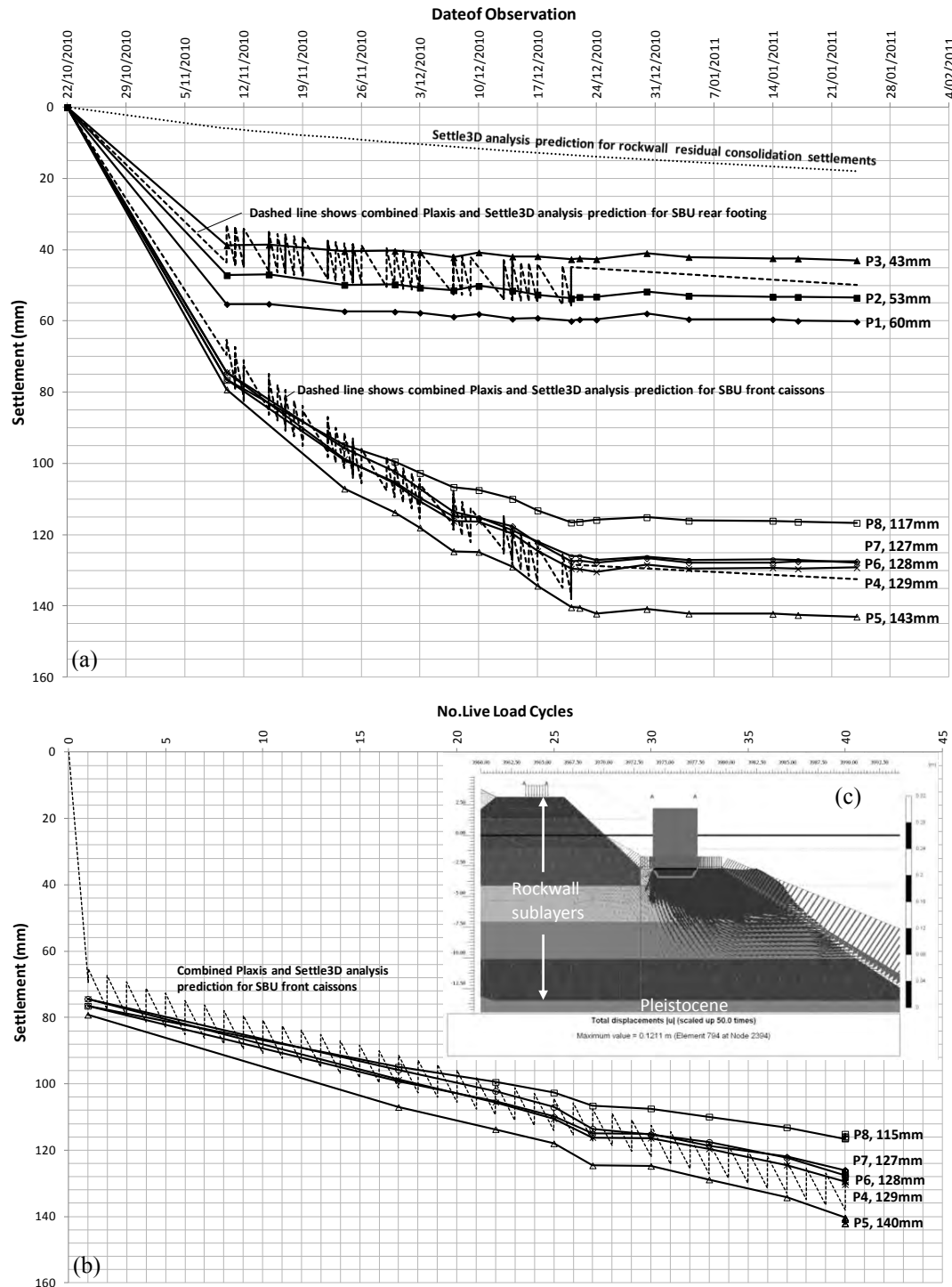


Figure 3

- (a) Predicted and monitored settlements vs. time
(b) Predicted and monitored settlement of caissons vs. cumulative load cycles
(c) Plaxis output plot showing total displacement vectors after 40 load / unload cycles

The results of the predicted settlements show that each load cycle of the SBU produces irrecoverable plastic deformation in the underlying rockwall, and that this deformation is more pronounced beneath the caissons than beneath the rear abutment footing. The results also show that when usage of the SBU was suspended on 21 December 2010 following the 40th load cycle, the measured ongoing settlements of the SBU (under dead load alone) correspond reasonably closely to the predicted consolidation settlements of the Pleistocene strata beneath the rockwall.

Thus, the results of the analysis highlight a key distinction between the settlement behaviour of the front caissons compared to the rear abutment footing during use of the SBU. The settlement behaviour of the caissons tends to be localised and related to the cyclic nature of the SBU live load. Settlements are cumulative for each load application. In contrast, the settlement behaviour of the rear abutment footing is significantly less and is reflective of the time-dependent global consolidation settlements of the rockwall foundation.

The reason for the accumulation of irrecoverable plastic strains in the rockfill underlying the caissons with each progressive load cycle is largely due to the caisson foundation's proximity to the face of the rockwall slope, which is approximately 2 m distance. The displacement vectors in Figure 3(c) show that in the rockfill beneath the caisson and near the face of the rockwall front slope, a large proportion (ranging from about 50% to 90%) of the total displacement is horizontal (i.e. towards the slope face). This horizontal displacement results in a reduction of confining pressure (which is analogous to σ'_3) and therefore a reduction in reloading stiffness as shown in Equation (1) which governs the calculation of reloading stiffness in the Hardening Soil model. (Note that σ'_3 is negative for compression). The decrease in stiffness results in an increase in vertical and horizontal displacement which accumulates with each load cycle.

$$E_{ur} = E_{ur}^{ref} \left(\frac{c \cos \varphi - \sigma'_3 \sin \varphi}{c \cos \varphi + p^{ref} \sin \varphi} \right)^m \quad (1)$$

The results of the analysis indicate that simulating the complex settlement behaviour of the caissons requires an advanced constitutive model, such as the Hardening Soil model, which incorporates stress dependent stiffness and also distinguishes between primary loading and reloading/unloading, combined with cyclic loading calculation phases. It would not be able to be accurately modeled using the traditional Mohr-Coulomb model or with monotonic loading.

6. CONCLUSIONS

In this study, the settlement behaviour of a caisson foundation undergoing cyclic loading on a submerged rockfill slope was analysed using finite element analysis and the Hardening Soil model and compared with vertical movements measured from *in situ* survey monitoring points. The predicted settlements obtained from the finite element analysis are found to be in general agreement with the results of the survey monitoring.

This case study indicates that the Hardening Soil model can satisfactorily predict the deformation characteristics of rockfill slopes undergoing loading and unloading cycles with some confidence.

7. ACKNOWLEDGEMENTS

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USE OF FRICTION SLABS AS THE PREFERRED WHARF TIE BACK SOLUTION AT PAGO PAGO AND AVATIU PORT

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ABSTRACT

Within the greater Pacific area, ports are generally a key infrastructure asset for many island nations. Two ports which have recently had new wharf structure designs undertaken include the Avatiu Port in the Cook Islands and Pago Pago in American Samoa. Both ports handle a combination of container freight and passenger ferries or cruise ships.

The proposed wharf structures at both these ports differ in form and underlying geology, however both sites incorporated pavement slabs as the preferred structural tie back system to the respective wharf structures. Both sites also had existing wharf structures which were needed to be reconstructed. In addition, the existing wharf at Pago Pago is not currently designed to cater for seismic events.

This paper briefly discusses the challenges in re-developing port facilities and the geological conditions, constraints, risks and other key considerations which attributed to the adoption of friction slabs as the preferred tied back system at both Avatiu Port and Pago Pago.

1. INTRODUCTION

Ports within the greater Pacific region are generally a key infrastructure asset for island nations, allowing for import and export of goods as well as regional trade and transport between outer islands. More recently they have provided the opportunity for increased exposure to the tourism market through cruise ship visits.

Two ports within the Pacific which have recently looked at upgrading their facilities are at Pago Pago in American Samoa and Avatiu Port in the Cook Islands. Both ports handle a combination of container and other supply ships, passenger ferries or cruise ships. The reconstruction design of the main wharf at Avatiu Port and the service wharf at Pago Pago was undertaken by Beca International Ltd and Beca Pacific Inc., respectively. Both designs incorporated friction slabs as the preferred tie back solution. This paper discusses the challenges of re-developing existing port sites and the considerations in adopting the, general layout and design features of friction slabs.

2. PORT UPGRADE CHALLENGES

Upgrading or reconstruction of existing port facilities is often faced with many construction and geotechnical challenges. The two sites are constructed over differing ground conditions from deep layers of soft compressible deposits to hard coral. Reclamation fill can be uncontrolled and highly variable consisting of unknown dumped materials, mass concrete, boulders, gravels, sands, silts and clays, with some placed as hydraulic fill. The water table is often shallow and tidal. Slopes and bunds that occur above and below the water level are usually protected by rip rap. In addition these sites usually have a varied history with a number of past building types and layouts with existing and abandoned foundations, pavements and services. These ground conditions can make simple excavation and piling operations slow and difficult or even impossible.

Wharf structures usually have high lateral load demands, such as bollard mooring loads, fender reactions from ship berthing and large lateral soil loads due to high retained heights and equipment/cargo stacking surcharges. Earthquake actions on high retaining walls further complicate matters. Due to these lateral loads wharf structures often require tie backs or other support systems.

As ports are key infrastructure assets, complete closure of an existing working wharf for reconstruction is usually considered unviable and access and use of at least a portion of the wharf must be maintained throughout the work program. Therefore ease of construction and construction staging can be a key consideration during design.

3. DESIGN LOADS

The design loads can be high due the extent of retained heights and use of heavy plant on the surface and stacking and storage facilities. Typical design loads adopted for the new wharf structures at Avatui Port and Pago Pago consisted of:

- a) Active soil pressure on the wharf
- b) Water table/tidal lag
- c) Live loads (surface load 30-40kPa or 96t Axle)
- d) Bollard mooring loads of 80t (Avatiu Port) and 25t (Pago Pago)
- e) Fender berthing reactions
- f) 12kPa surcharge for future raising of the wharf to allow for sea level rise (Avatiu Port only)
- g) Single emergency/storm cruise ship bollard- 200t load (Pago Pago only)
- h) Seismic actions, soil pressure and structure inertia (Pago Pago only)

4. TIE BACK OPTIONS

With high lateral loads cantilevered wall or wharfs solutions often have insufficient capacity to provide economical designs to resist these forces. Therefore, some form of tie back system is usually adopted.

To provide tie back support a number of typical systems include:

- Tie back with concrete deadman
- Tie back with sheet pile or king pile wall
- Inclined grouted anchors

With high lateral loads the use of a discrete or shallow deadman anchoring system is unlikely to have sufficient design capacity, therefore longer and deeper excavations are required. Excavations within variable reclamation fill are at risk of encountering large obstructions and of collapse due to high water levels.

Driving of sheet piles or king piles into variable reclaimed fill may encounter obstructions within the fill, resulting in laterally loaded sheets and piles pulling up short of the design embedment.

While advancing small diameter drill holes through obstructions is more achievable; a large number of anchors would be required to achieve the load demands. Where ground conditions are soft or loose there is a risk of hole collapse and insufficient bond capacity. In gravel deposits hole collapse is also a risk along with voids which can cause grout loss.

The use of specialist equipment and expertise can reduce many of these risks. However where sites are remote, there is generally very limited local expertise so specialist skill needs to be brought in at a premium cost. In light of the above, innovative solutions were sought to minimise these risks.

5. FRICTION SLAB

The use of a friction slab was considered an innovative solution that limited the depth and extent of likely excavation. It utilised pavements already within the scope of works to provide restraint to the wharf structure. It is also a system that can be undertaken without introducing another construction type to the scope of works.

The principle on which a friction slab provides support is simple. By tying a number of concrete slabs together in series with a central layer of reinforcement a single mass spread over a large area can be formed. To mobilise this mass the base friction needs to be overcome. This friction provides the resisting support to the wharf structure. The base friction was calculated on the same principle as a sliding block and that used for base friction on gravity walls were the horizontal sliding resistance is equal to:

$$Fr = \sigma v' \alpha \tan \phi'$$

Where: $\sigma v'$ = Effective over burden pressure

α = Friction coefficient

ϕ' = Effective soil friction between layers

An advantage of utilising base friction is that as additional load is applied to the pavement additional friction resistance is developed increasing the horizontal load capacity.

5.1 FRICTION SLAB DESIGN

During the design of the friction slab, it was generally found that load cases where there is no vertical load on the slabs in combination with high lateral loads were the most critical. Live loads on the slab tended to provide additional weight to “clamp” the friction slab.

The friction capacity of the slab needs to be checked at a number of depths, that is, at the base of the slabs, between underlying pavement layers and at the interface with the subgrade. While there may be a reduction in friction angle between subsequent layers, the overburden mass is increasing to overcome the reduced friction angle, therefore the most critical layer is dependent on the thickness, unit weight, water level and friction angle.

To increase the ultimate capacity of the slab system, downstand rows were included. The rows were spaced to avoid interaction between them and this limited the length of required friction slab. For working loads only; base friction was relied upon.

For vertical pavement loads, being an evenly distributed live load and design axle loads, the concrete slabs were designed as an unreinforced concrete pavement with shear keys on the transverse joints (joints running perpendicular to wharf face) and dowels used on longitudinal joints (joints running parallel to wharf face). Centrally placed continuous reinforcement is provided across transverse joints to tie slabs together.

By distributed loads over a large area and providing resistance by means of friction, friction slabs provide a stiff response to loading. Therefore a high stiffness was applied in retaining wall design models, this resulted in small displacements and higher tie demands than that of other tie back systems. This provides good restraint and limits the demands within the retaining wall.

As the pavement is relied upon to develop suitable friction to support the wharf structure; slabs cannot be readily cut or removed without equivalent restraint being provided. This requires certain aspects of the project to be carefully considered along with future proofing, for example of items as drainage and services. The installation of portable or light structures can generally be accommodated easily due to the pavement having design capacity in the order of 30-40kPa.

Differential settlement between the wharf structure and the friction slab pavement was accommodated by designing the first pavement slab behind the wharf to fully span.

Overall this system provides a robust and durable design which minimised the risks of working with variable ground conditions and was innovative by utilising existing pavement slabs to provide the required support to the wharf.

6. CASE STUDY 1 - AVATIU PORT

6.1 LOCATION AND GEOLOGY

Avatiu Port is located on the island of Rarotonga within the Cook Islands. The island of Rarotonga comprises an extinct volcano rising over 5000m from the sea floor, formed by two phases of volcanism some 2.3-1.6 million years ago (Ma) and 1.4-1.1 Ma. The volcano is surrounded by a coral reef, cut through in places by alluvial channels formed by streams running down the volcano (Thompson et al, 1998).

The site is located on the coral reef over one of these channels, formed by the Avatiu River, which has since been diverted to the east of the site. Ground conditions consist of varying reclamation fill overlying variable coral deposits. The investigation at the northern end of the existing wharf generally encountered 5m of massive coral or re-cemented coral gravel and cobbles, overlying approximately 5m of coral cobbles and boulders. At the southern end un-cemented coral gravels and cobbles were encountered over the upper 5m. Alluvial deposits were also encountered at depth at the southern end, below the toe of the main wharf piles.

6.2 EXISTING STRUCTURE

The existing wharf was constructed in the 1980's and consisted of two sections, a southern and northern end with a kink in the wharf alignment between these two sections, providing a total berth length of 180m. The structure consisted of precast concrete sheet piles with near surface tie backs. The sheets were driven and considered to have founded a minimal distance into the coral. Reconstruction of the wharf was considered necessary to provide a deeper berth and lengthen the berth by providing a straight alignment. In addition pavements behind the existing wharf did not meet the current or future proposed use and also needed to be replaced.

6.3 NEW STRUCTURE

The new wharf comprises a single row of driven steel piles placed in front of the existing wall over the southern section and behind the existing wall over the northern section. The wharf level is +2.1m RL with a -8.0m RL dredge level (including 0.5m over dredge allowance). This 10.1m vertical wharf wall consists of 762mm diameter steel tubular “king piles” at 2.0m centres with AZ profile intermediate sheets. The tube piles were designed to provide sufficient capacity for the wall demands with the sheet piles necessary to contain the reclamation materials. A typical Section is provided in Figure 1 Below:

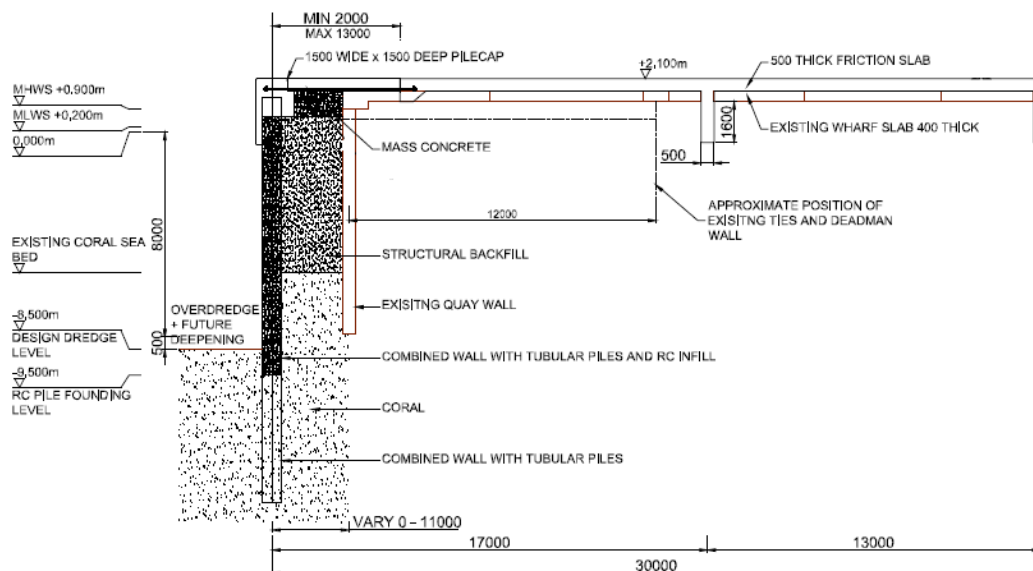


Figure 1: General arrangement of the Avatiu Port Wharf (southern section)

To reduce soil pressures on the southern wall, the backfill material between the existing and new wall would be stabilised by the addition of Portland cement. This was to reduce lateral loads and therefore reduce the size of the required wall. This also reduced the expected on going settlements below the concrete pavement. However, to limit wall displacements, to reduce the embedment depth to within hard material layers to avoid a softer layer at depth and to further reduce the wall section, a tie back support was considered necessary to provide an efficient and economical solution.

6.4 USE OF FRICTION SLAB

With existing pavement slabs over the site and variable fill and coral gravels and a high water table underlying the site excavation or piling to construct a deadman system was likely to be slow or difficult, requiring additional heavy plant or an extended construction program. Hole collapse and grout loss was also considered a high risk with ground anchors. The area behind the wharf was to be paved with a new concrete pavement to allow for higher wheel/axle loads. Using the new pavement as a friction slab reduced the scope of the piling works in hard ground conditions. In addition two rows of downstands were added to the pavement to combine the mass of the new and existing pavements together increasing the horizontal capacity of the tie back system.

7 CASE STUDY 2 - PAGO PAGO

7.1 LOCATION AND GEOLOGY

Pago Pago is located within the islands of American Samoa. These islands are part of a volcanic archipelago that rises thousands of feet from the Pacific Ocean floor as a series of broad shield volcanoes, formed as the Pacific Plate moved over the stationary Samoa hotspot. Alternating lava flows and tuff, volcanic breccia, lapilli and cinder layers from these volcanoes were ejected in the various stages of activities. After volcanism ceased about 1.28 million years ago weathering and erosion occurred from exposure to tropical climate, thus producing the topography seen today (McCue, 1998).

The site consists of reclamation fill between 4-5m thick and includes concrete, basalt gravel, sands and silts. The reclamation fill has been placed over approximately 11.5–28m of marine sediments. The marine sediments consist of coral and basalt gravels and boulders mixed with soils eroded from the land mass, being sandy clayey silts.

Layers of completely weathered to highly weathered gravelly basalt between 6–22.5m thick underlie the recent marine sediments. This material was found to consist of sandy clayey silts, basalt gravels and boulders. The underlying bed rock consists of highly to slightly weathered basalt at varying depth up to 60m below the surface of the reclamation.

7.2 EXISTING STRUCTURE

The southern and shallower end of the wharf comprises a gravity quay wall (likely to consist of mass concrete) and is approximately 30m in length. The northern end of the wharf consists of a piled deck with a retaining wall on the landward side and is approximately 80m in length. Over a short section of the northern wharf, the rear piles support a wharf shed; movement of the wharf has resulted in separation of the shed walls between the wharf and the adjacent ground. The use of tie backs for the existing wharf is unknown. The existing wharf is considered to be at the end of its useable life due to the degree of deterioration of the structure, and requiring full replacement.

7.3 PROPOSED NEW STRUCTURE

The proposed wharf will be approximately 110m long and 6.0m wide and allow for a future dredge depth of approximately -6.0m CD. The outer piles were aligned on the seaward side of the existing wharf with the finished deck level (+1.8m CD) above the existing wharf to allow the new structure to span over the existing one to minimise the extent of demolition required. Construction of the new wharf comprises 2 rows of 600mm diameter tubular piles at approximately 10m centres longitudinally and 5.0m apart, driven to a depth of approximately 55m. A new AZ sheet pile land side retaining wall would be constructed along the full length of the new wharf and tied into the widened capping beam. A retained height of approximately 5.0m is allowed for with a 1.0V to 2.0H slope in front of the retaining wall beneath the wharf deck. A pavement behind the wharf is also to be provided. A typical Section is provided in Figure 2 below:

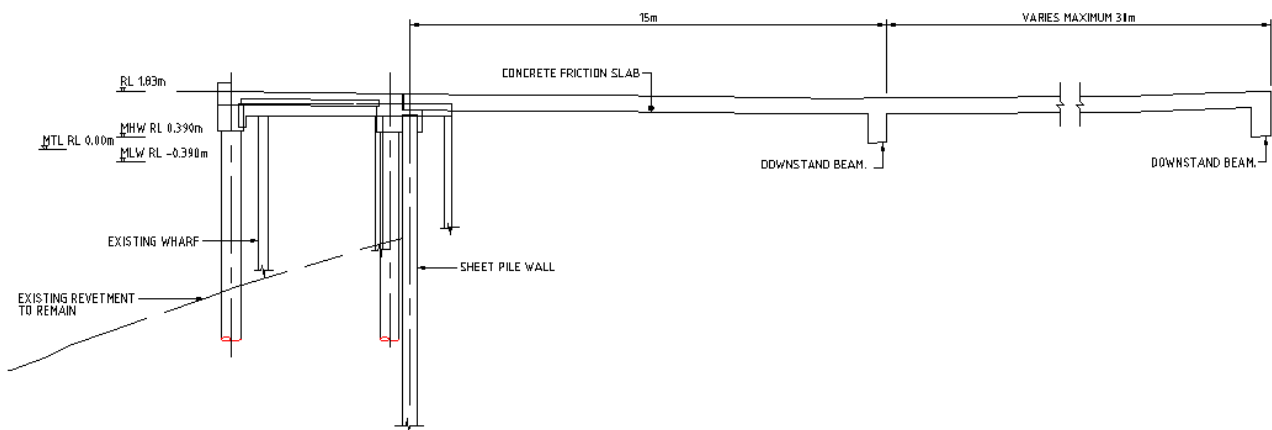


Figure 2: General arrangement of proposed Pago Pago Service Wharf reconstruction

Due to the long slender nature of the piles and high horizontal loading been applied to the structure from soil pressures, bollard loads and seismic loading; tie back support was required to limit deflections and to achieve an economic and efficient solution.

7.4 USE OF FRICTION SLAB

A friction slab was adopted over other tie back systems due to a high water level increasing the difficulty in which an open excavation could be undertaken for a deep concrete strip deadman system. The frequency of expected obstructions was also considered too high for a sheet pile deadman system to be installed efficiently. The depth to a suitably hard founding stratum was considered too deep for ground anchors to be a practical solution.

The area behind the wharf was to be paved with a heavy duty pavement. By construction this type of pavement, a good service life could be achieved while also utilising this as the tie back system for the wharf structure. However, the area to be paved was quite irregular, approximately triangular in shape. This meant at some locations there was an insufficient length of slab directly behind the wharf to carry the ultimate design loads. To overcome this full reinforcement was provided in both directions outside the central core slabs to spread load laterally. In addition the perimeter edge beam of the pavement was adapted to aid in the spread of load over the frictions slabs and serve as a downstand to reduce the required slab length; a second downstand was also included in the design to provide additional load capacity to the system.

8. CONCLUSIONS

The reconstruction of existing wharf infrastructure has many challenges. Reclamations often consist of highly variable fill, obstructions and are often low lying. These ground conditions can make deep excavations, pile driving or anchor installation difficult. During the preliminary and detailed design phases of the reconstruction of the Avatiu Port Wharf and Pago Pago Service Wharf, friction slabs were considered as an innovative solution.

The use of friction slabs utilises the new pavement to provide suitable lateral restraint to the wharf with minimal additional works. The construction methodology limits the extent of excavation required on the sites and provides a robust and economical solution to support the different wharf structures.

Friction slabs are likely to be a viable solution where high capacity pavements are being provided and resistance to high lateral loads is required.

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CASE STUDY - GEOTECHNICAL DESIGN FOR THE SOUTH ROAD SUPERWAY

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ABSTRACT

The South Road Superway project involves the upgrade of a 4.8km section of a major arterial road to expressway standard. The works include the construction of 2.8km of elevated bridge structure with six approach embankments. This paper focuses on geotechnical design considerations for the northern bridge approach embankments which are to be constructed over the Holocene coastal marine sediments of the St Kilda Formation. Due to the presence of shallow groundwater and potential acid sulphate soils, the bridge approach embankments were designed to be supported by a geotextile reinforced granular platform over the soft, potentially liquefiable, soils. This strategy increases the embankment shear resistance and also minimises the amount of spoil generated in treating any unsuitable materials (i.e. soft soils). To limit settlement at one of the bridge abutment locations, ground improvement in the form of grouted displacement columns was required. Embankment movements greater than those specified in the Project Specific Technical Requirements, would not only affect the long-term pavement performance but also impose greater additional loads on the piles supporting the bridge abutments.

1. INTRODUCTION

The South Road Superway project in the north-western suburbs of Adelaide, South Australia involves the construction of several embankments and structures over estuarine soils. The subsurface conditions in the northern interchange area, near the intersection of South Road and Port River Expressway, comprises a surficial fill associated with land development underlain by firm and loose estuarine soils.

Without specific foundation treatments, the bridge approach embankments and retaining walls would have been prone to instability and excessive settlement. The geotechnical challenges required solutions for embankments built on compressible soils to achieve strict post-construction and differential settlement criteria.

This paper outlines the South Road Superway project and the design methodology for the bridge approach embankments in the northern interchange area including the design and numerical modeling of Controlled Modulus Columns (CMCs).

2. PROJECT OVERVIEW

The \$842 million South Road Superway project is the largest and most complex road project in South Australia's history to date. The Urban Superway Joint Venture (comprising John Holland, Macmahon Contractors and Leed Engineering) was awarded the task of designing and constructing the 4.8km upgrade of South Road to expressway standard between its intersection with the Port River Expressway and Regency Road. GHD, SMEC and IBT are the main design consultants. The upgrade works comprise extensive earthworks, drainage upgrades, service relocations, large reinforced soil structure walls (RSW), a soil nail wall, concrete cantilever walls, ground improvement works and the construction of a 2.8km elevated bridge structure launched using the precast balanced cantilever method (Figure 1).



Figure 1: Construction of the South Road Superway is scheduled to be completed by the end of 2013

3. GEOTECHNICAL CONDITIONS

Extensive geotechnical investigations, including 192 boreholes, 106 electronic friction cone penetration tests (CPTs), 24 groundwater monitoring wells, 28 dilatometer tests (DMTs) and an extensive suite of laboratory testing has been undertaken to characterise subsurface conditions along the South Road Superway alignment.

Subsurface conditions at the northern interchange are typified by an Estuarine Muds and Sands (EMS) soil profile consisting of soils of the St Kilda and Glanville Formations overlying Hindmarsh Clay. Covering the natural soil profile is typically about 2m of fill materials associated with land reclamation and transport developments such as the existing South Road and Port River Expressway. Much of the fill outside of the transport corridors is waste material comprising mixed soils with large concrete blocks, bricks, plastic, steel and asphalt. Groundwater lies approximately at mean sea level (about 2 – 3m below surface level).

The St Kilda formation deposits within the project area consist of sedimentary materials derived from estuarine and back barrier lagoonal environments of deposition. These Holocene sediments vary from silts, clays and muds to shelly sands. Some isolated regions of peat have been known to occur within the formation but were not encountered during the site investigation for the South Road Superway. It is widely reported (Cann and Gostin 1985) that materials of the St Kilda formation are soft, compressible, hyper saline and potential acid sulphate soils. The depth of St Kilda formation soils within the project area was generally limited to 6m below existing surface level (typically 4m thickness in total).

Figure 2 displays the encountered soil types, as well as the undrained shear strength and overconsolidation ratio profiles in the northern interchange area determined via interpretation of representative CPT data (Robertson 2010). The CPT material interpretations were assessed against adjacent borehole logs with satisfying results. Likewise, undrained shear strengths determined via unconsolidated undrained (UU) triaxial testing and insitu vane shear tests were assessed against the Robertson (2010) correlations to determine a project specific empirical cone factor N_{kt} .

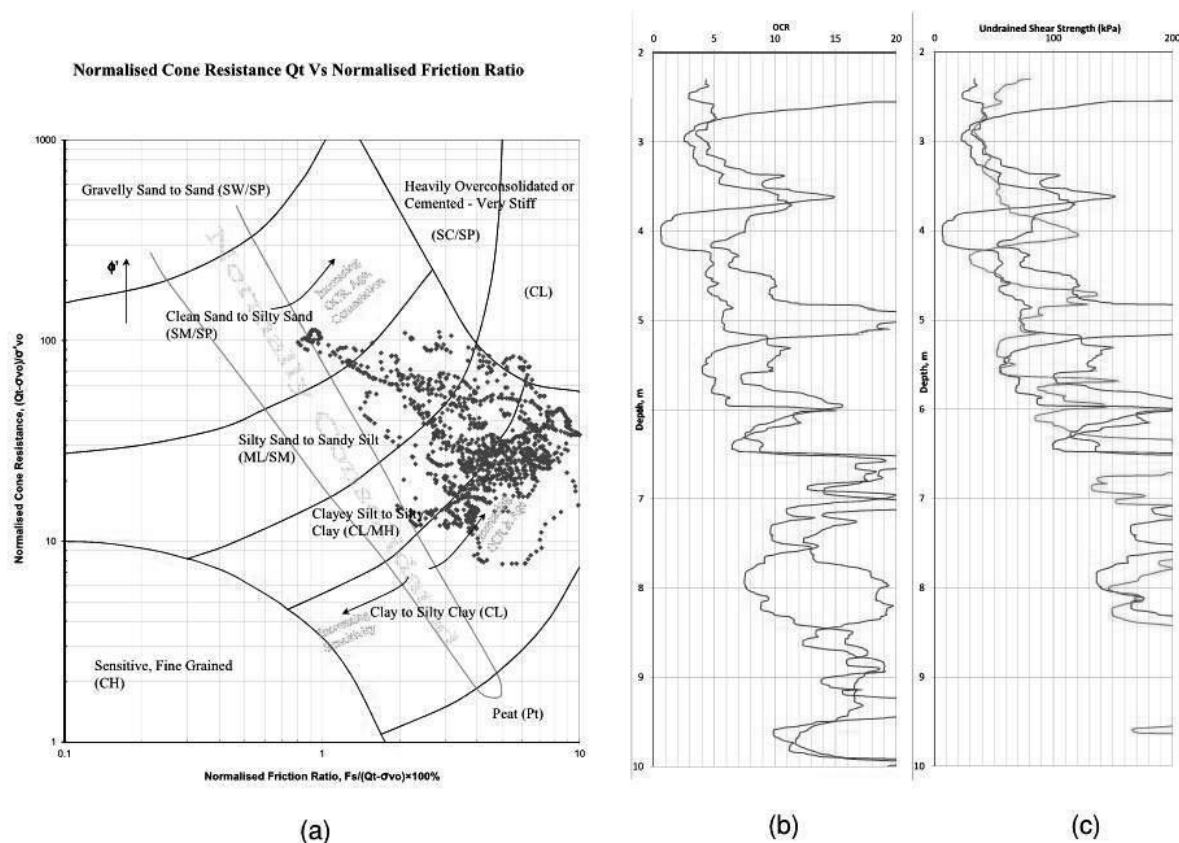


Figure 2: Typical CPT plots in the northern area of works

Underlying the St Kilda formation soils in the northern interchange area for the South Road Superway is the Hindmarsh Clay. Soils of the Hindmarsh Clay are typically stiff to hard in consistency, often fissured and are overconsolidated from desiccation. Pile foundations for the bridge structure were founded within the Hindmarsh Clay. The base of the Hindmarsh Clay was not determined during the investigation but has been reported to extend to depths greater than 60m.

4. GEOTECHNICAL DESIGN CONSIDERATIONS

4.1 LIQUEFACTION ASSESSMENT

The normally consolidated soils of the St Kilda formation have been acknowledged as having a high potential for liquefying during a large earthquake event (Poulos et al, 1996). Although only two cases of liquefaction have been documented in Australia (Beachport 1897 and Warnambool 1903), these events indicate that the effects of liquefaction in Australian soils can be significant (Mitchell and Moore, 2004).

The liquefaction potential of the soils in the Superway project area were assessed using the Bray et al (2004) criteria, the deterministic CPT methodologies outlined by Youd et al (2001) and the probabilistic method of Moss et al (2006). Standard penetration test (SPT) data was reviewed but not relied upon due to reported concerns relating to the repeatability and reliability of SPT assessments (Mitchell and Moore, 2004).

The Bray et al (2004) criteria for liquefaction susceptibility of fine-grained soils use plasticity index and the liquid limit to moisture content ratio as the primary indicator of a potentially liquefiable soil. Based on this assessment no recovered samples were considered susceptible (Figure 3). In addition, assessment based on the deterministic methods of Youd et al (2001) yielded factors of safety in excess of one throughout the alignment for the design earthquake event (PGA = 0.143). The probability of liquefaction occurring for the design earthquake event based on the Moss et al (2008) procedure was also considered acceptably low.

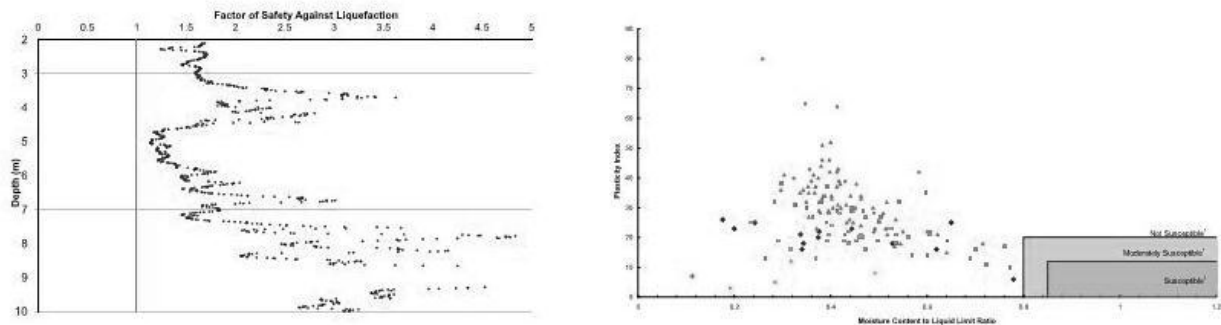


Figure 3: Susceptibility assessment using the criteria of Youd et al (2001) and Bray et al (2004) revealed the risk of liquefaction was acceptably low

4.2 EMBANKMENT AND RETAINING WALL SETTLEMENT AND STABILITY

Figure 4 shows the general arrangement of the northern interchange area for the South Road Superway. The main alignment bridge approach embankment is to be constructed alongside and above the existing embankment. Extensive foundation treatments comprising deep removal and replacement undertaken during construction of the existing embankment, in addition to relaxed differential settlement requirements, ensured foundation treatments in this area solely comprised the use of high strength geotextile and a preload period of six months to minimise post construction settlements at the embankment/approach slab interface to 30mm. The settlement criteria were relaxed in this area as the arrangement shown in Figure 4 represents the interim arrangement prior to the construction of an additional future elevated structure.

The northern entry ramp (east) is to be constructed on the existing Port River Expressway to South Road turning lane. Whilst settlement predictions were within the specified criteria of the Project Scope and Technical Requirements (PSTR), stability requirements could only be met with the use of high strength geotextile at the base of the embankment/retaining wall.

The northern exit ramp (west) is to be constructed on former industrial land. Stability and settlement analyses indicated that without foundation treatment the stability and settlement requirements of the PSTR could not be met. The objectives of the bridge approach treatment were to:

- Minimise differential settlements to 0.1% and post-construction settlement at the approach slab/embankment interface to 10mm.
- Increase shear stability of the RSW to achieve a minimum factor of safety of 1.5 under permanent loading and 1.35 under seismic loading with 75% of the peak ground acceleration and 50% traffic surcharge (RTA R57).

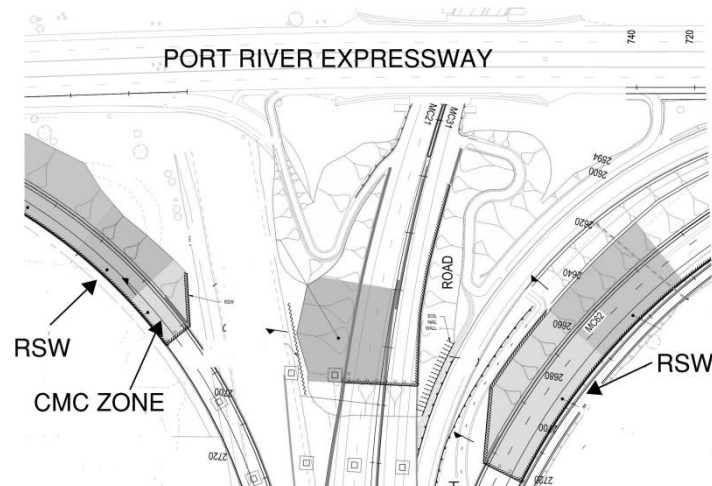


Figure 4: General arrangement plan for the northern interchange area of the South Road Superway

4.2.1 Bridge Approach Treatment

During the tender design for the project a number of ground treatment options were considered including remove and replace, stone columns and wick drains with surcharge. Controlled modulus columns (CMCs) were preferred for their speed of construction, greater certainty in predicted settlements, reduced spoil generation, reduced fill requirements and compatibility with the proposed piling rig for the bridge substructure.

The principle of CMCs is to form semi rigid vertical inclusions in the soil to improve the global stiffness and strength of weak ground. Reverse flight augers displace soil laterally with virtually no spoil or vibration, thus increasing the lateral stress at the soil/column interface and practically eliminating the need for off-site spoil removal or treatment of naturally occurring potential acid sulphate soils. Unlike piled structures, the CMC method of ground improvement does not bypass the compressible layers and support the entire load on piles but rather the CMCs improve the soil globally and reduce its deformability.

In order to arrive at the optimum solution, the CMC modeling followed the approach outlined in Masse et al (2004). This involved analysis at the level of an individual column followed by analysis of the overall system using a PLAXIS 3D model. The axisymmetric PLAXIS model allows for an iterative and optimized design of the installation grid, column diameter, load transfer platform (LTP) and grout strength. The PLAXIS 2D and 3D models allowed for assessment of ground movement, CMC structural integrity, assessment of tension forces in the high strength geotextile within the LTP and assessment of additional stresses imposed on abutment piles due to the placement of fill. A PLAXIS 3D model was undertaken to assess the complex soil structure interaction between the CMCs, high strength geotextile, bridge abutment piles and bridge abutment headstock. The full-height bridge abutment headstock was 'tied back' with reinforced soil straps to reduce the lateral forces taken by the bridge abutment piles.

Figure 5 shows the arrangement of the bridge approach treatment which reduces post construction settlement at the interface of the piled bridge abutment and non-piled embankment. The following systems of treatments have been incorporated into the design:

- 450mm diameter CMCs installed in a triangular grid at 2.5m centre to centre spacing with grout of unconfined compressive strength = 15 MPa at 28 days.
- Zone 1 comprised three rows of full length CMCs embedded in the very stiff Hindmarsh Clay. Zone 2 represents a transition zone where the length of each row is reduced linearly to ensure the differential settlement criteria of 0.1% for pavements and reinforced soil walls would be met. Transition zones may be achieved by either one or a combination of increasing the pile spacing, changing the pile length or pile diameter.
- A load transfer platform (LTP), or geotextile reinforced mattress, was incorporated above the CMCs to support and distribute embankment load to the CMCs and surrounding ground. The LTP consists of 2 layers of 400 kN/m high strength geotextile laid orthogonally to each other separated by select granular fill.
- To reduce the tensile stress on the high strength geotextile, column caps were included. These are unreinforced and essentially a thickening of the CMC that can easily be installed during withdrawal of the CMC auger.

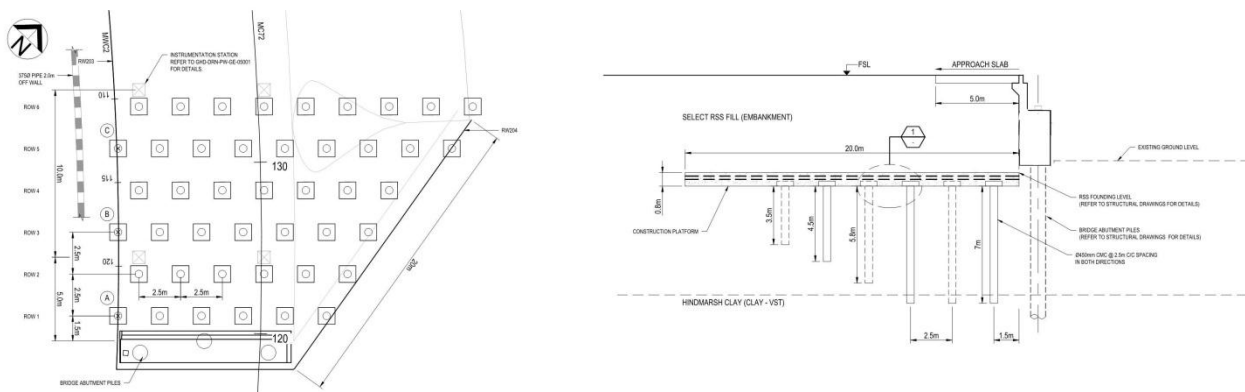


Figure 5: Plan and elevation view of the foundation treatment at the Northern Exit Ramp of the South Road Superway

The PLAXIS model was used to assess the structural integrity of the CMCs and bridge abutment piles. The CMCs were modelled using basic material mechanics. The two dimensional stresses σ_{xx} , σ_{yy} and τ_{xy} were converted to principal stresses and compared to the allowable compressive stress and allowable tensile stress of the grout to ensure the structural integrity of the CMCs. The bridge abutment piles were modelled using pile elements with the additional stresses due to embankment movements input into the bridge foundation design.

The tensile stresses of the dual 400 kN/m high-strength geotextile layers within the LTP were also determined through PLAXIS analysis. As reported by Hewitt et al (2004) the tensile force predicted using the procedures outlined in BS8006 (2010) were much larger than those determined via numerical modeling.

5. CONCLUSIONS

The case study presented in this paper describes the geotechnical considerations and foundation treatment design for the northern interchange area of the South Road Superway project. The northern embankments are to be constructed on compressible soils of the St Kilda formation. Liquefaction assessment based on CPT and material characteristics showed that the risk of liquefaction was acceptably low for the design earthquake event.

The use of CMCs is described as a means to control differential settlement at the elevated structure/earth embankment interface to ensure compliance with the project specific technical requirements. This method enables greater certainty in achieving the design settlement criteria over other forms of ground improvement whilst also minimising the requirements for imported fill, reducing construction time and also limiting the amount of potential acid sulfate spoil.

6. ACKNOWLEDGEMENTS

The author wishes to extend his thanks to the Urban Superway Joint Venture and GHD for permission to present this paper.

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PERFORMANCE DRIVEN SLOPE MANAGEMENT, DESIGN AND OPTIMISATION AT BROCKMAN OPERATION, RIO TINTO IRON ORE

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ABSTRACT

Geotechnical design is characterised by a large number of uncertainties, even if industry best practice is followed during the design process. Regulatory and corporate standards require appropriate geotechnical design and ongoing slope management through to closure. Rio Tinto Iron Ore (RTIO) have addressed this challenge at the Brockman Operation by monitoring actual slope performance and verifying design assumptions by ongoing mapping of pit walls during mine design implementation.

A case history is presented to illustrate the benefits of:

- early detection of differences in actual vs. expected slope conditions (allowing time to develop and present remediation options to management) and
- continued verification of actual slope conditions with mine progression (improved understanding of slope conditions and behaviour leading to an opportunity for increased ore recovery);

These processes have lead to improved outcomes in terms of safety, design reliability and protection of the business plan.

1. INTRODUCTION

RTIO operates in over 260 individual open pits across 14 operations in the Pilbara Region of Western Australia. Geotechnical design and operational slope management require continuous geotechnical assessment in order to effectively manage slope instability risks with optimal economic outcomes. Due to the aforementioned scale of the RTIO mining operation, it is impractical to physically map and assess all geological and geotechnical parameters on all slopes. Instead, a risk based approach was developed to focus efforts on high risk slopes whilst still checking lower risk areas for unforeseen hazards (de Graaf et al 2011).

Brockman Syncline 2 (BS2) is an open pit mining area at the Brockman Operation that produces a high quality ore which is crucial for blending to form the salable iron ore product.

Mining at BS2 had historically been in shallow above water table pits where nominal geotechnical drilling and subsequent design rigour was required. However, in more recent years, the discovery of mineralization at depth had resulted in deeper pit designs (Pit 6) requiring significant drilling and rigorous design. Due to adverse topography and active mining in Pit 6, limited geotechnical drilling was undertaken during the feasibility stages in 2007.

In order to adequately manage the risk presented by mining deeper with limited geotechnical data, the slopes at Pit 6 were initially monitored using manually surveyed prisms. Additionally, pit wall mapping was undertaken during various stages of mining to verify slope conditions and design assumptions.

2. PILBARA ENGINEERING GEOLOGY

The most common mode of slope instability in the Pilbara Region is structurally controlled sliding on adversely orientated bedding surfaces. Bedding surfaces frequently interact with large scale structures (e.g. faults, persistent joints) to form release or complex slip surfaces. Such instabilities can be observed at various scales in both natural and excavated Pilbara slopes as illustrated in Figure 1.

The iron orebodies in the Pilbara Region are found within a strongly anisotropic rock mass which comprises of weak shale beds inter-bedded with very strong Banded Iron Formation (BIF) within the Hamersley Group. At the pit scale, the persistence of bedding is effectively infinite, ensuing in potential sliding instability wherever it adversely dips out of the slope. Developing a good understanding of structural geology is a fundamental aspect of Pilbara slope design. Unforeseen variations in the shear strength of bedding or the structural geology model can lead to substantial instability.

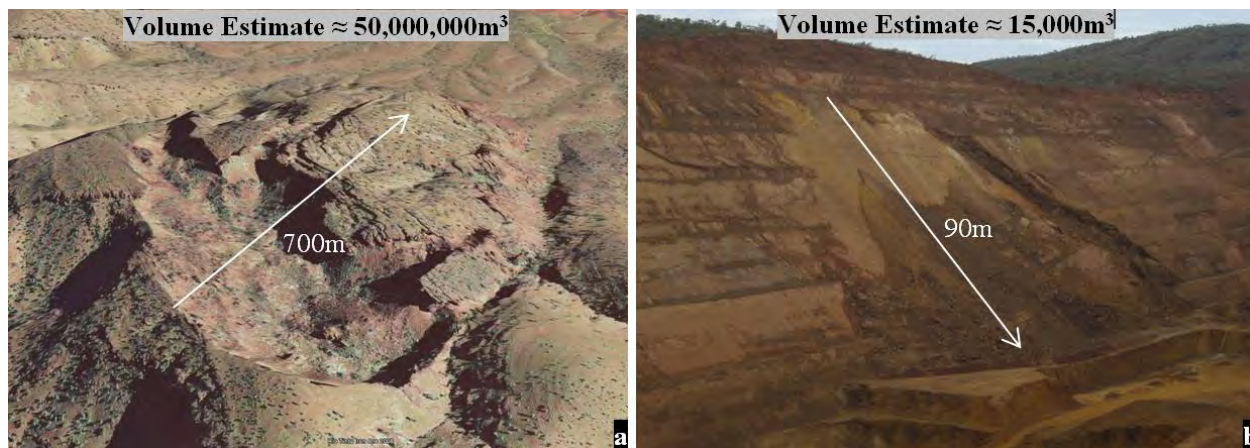


Figure 1: a. Natural slope sliding along McRae Shale (MCS); b. Bedding sliding example from RTIO operation

3. PROJECT HISTORY

The geotechnical design history of the southern slope of Pit 6 at Brockman Operations has been presented in this section to demonstrate the benefits of monitoring slope performance and verifying design assumptions during mine design implementation to improve safety, design reliability and protect the business plan.

3.1 Initial Stages of Mining

Based on the 2007 feasibility study, the slope design geometry comprised of 190m ultimate depth with 20m high, 65° batters and 8m wide berms (giving an inter-ramp angle of 48°).

During the initial stages of mining from 2007-2009, outcrop and pit wall mapping was completed as a part of a pit extension study. A subsequent geotechnical review in November 2009 uncovered potential stability concerns associated with less favorable (than initially expected) bedding orientations on the southern slope of Pit 6 (RTIO, 2009). Two-dimensional limit equilibrium analysis during this review determined the factor of safety (FoS) in the southern slope was approximately one (meta-stable). The area presented in Figure 2 was consequently highlighted as a high risk area subject to ongoing assessment and re-evaluation.

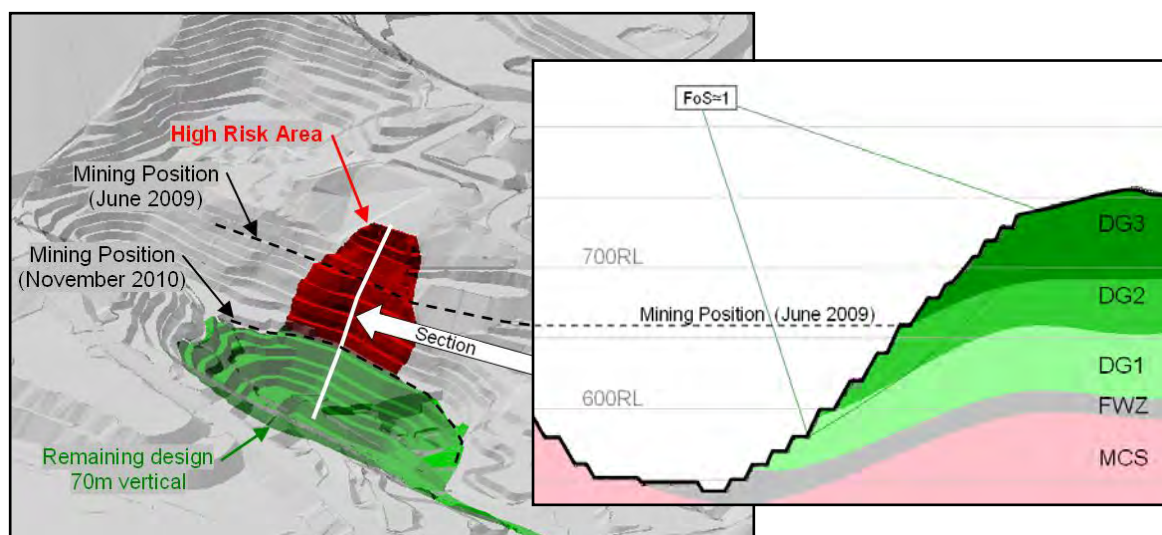


Figure 2: Southern slope high risk area and section with 2009 structural geology

Following the 2009 work, review of prism monitoring observations showed a general slow creep response in the area of concern and short periods of acceleration following blasting and excavation of trim shots. Due to the relatively shallow slope height at the time, this result was considered confirmation of the high risk status for the slope. Prism density was increased, as was the frequency of monitoring (from monthly to weekly), and visual inspections frequency also increased.

3.2 Slope Design Reconciliation with Mine Progression (Design Change 1)

A formal geotechnical reconciliation process was introduced for the southern slope of Pit 6 in January 2010. The intention of the process was to verify and/or update slope design assumptions including the structural geology model, rock mass and defect conditions, the hydrogeological model and actual slope performance.

In February 2010, mapping carried out on the 630RL berm confirmed the findings of the 2009 review and instigated a substantial redesign comprising the removal of the haulage ramp from traversing beneath the risk area at the base of the ultimate pit design. Additionally, batters below the 580RL were reduced from 20m high to 10m to mitigate uncertainties relating to unfavorable bedding orientations in the structural geology model and to provide flexibility for potential future design changes.

3.3 Signs of Instability

As part of the reconciliation work throughout 2010, slope performance reviews including the review of prism monitoring data confirmed a constant creep trend since prism installation. Increased rates of displacement (or accelerations) were consistently correlated with the blasting and excavation of trim shots along the southern slope as mining progressed to the 610RL. Shallow tension cracks were also observed less than 50m behind the slope crest.

3.4 Detailed Geotechnical Investigation and Slope Decoupling (Design Change 2)

A stability assessment of the southern slope was completed in September 2010 using all available oriented drill holes, window and traverse mapping, slope design reconciliation data, laboratory testing (UCS and direct shear tests) and an updated structural geology model. Kinematic, limit equilibrium and finite element analysis methods were applied to construct stability models which consider the mining schedule and planned bench progression. Slope performance interpretations from visual observations to prism deformation data were used to calibrate the stability models. The stability models demonstrated that the factor of safety for the current slope (above 600RL) did not meet RTIO slope design acceptance criteria due to a combination of unfavorable structural geology and poorer than initially expected rock mass quality (Figure 3a). Moreover, the models indicated that meta-stable slope conditions ($FoS \approx 1$) were likely to be encountered when excavations reached the 580RL, with the potential mode of instability involving sliding along shale bedding within DG2 and rock mass breakout. It was concluded that a remedial design change would be necessary to mitigate the risk of multiple batter instability in the order of 80m high over 200m length of slope (RTIO, 2010).

Three remediation options for the southern slope were provided to mine management:

1. Cutback to unload the upper part of the slope. Cutback dimensions: 60m high over 200m length of slope with a minimum width of 40m would significantly delay to the mining schedule and increase waste movement.
2. Decouple the slope on the 600RL by introducing a 20m wide berm. The slope above the 600RL would remain marginally stable and the associated risks managed through improved slope monitoring and improved hazard awareness. Operational impacts included some ore loss and increased monitoring costs.
3. No design change and managing risks while mining the final 70m. The stability of the slope would remain marginal and the potential instability volume would increase with every mined bench. The potential for ore loss and impact on the mine schedule would remain and significant increases in monitoring would be required (i.e. highest risk option – safety and economic, in the event of large scale instability).

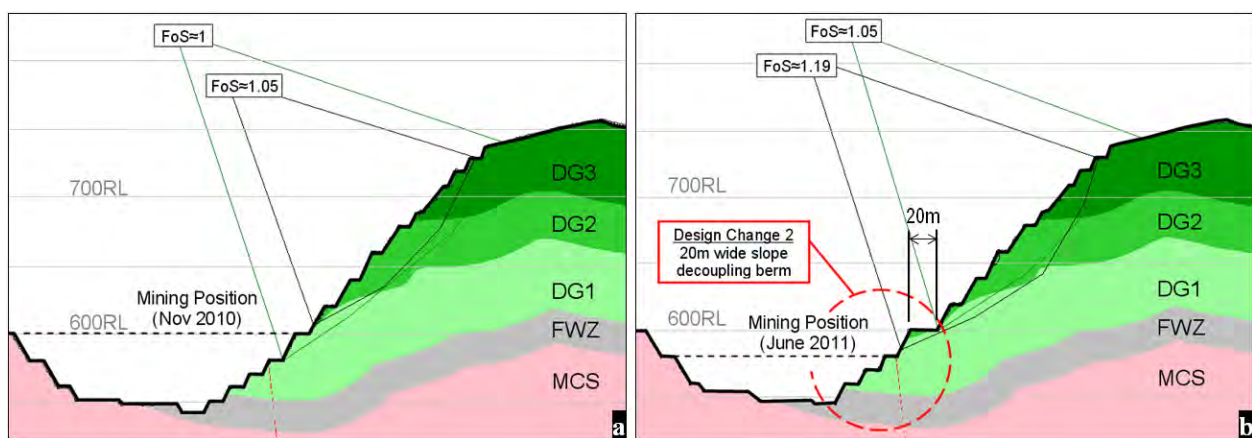


Figure 3: a. Stability model showing unacceptable $FoS \approx 1.05$ on 600RL and meta-stable conditions $FoS \approx 1$ on 580RL; b. Stability model with 20m wide slope decoupling berm on 600RL showing stable conditions, $FoS \approx 1.19$.

Considering the risks with the aforementioned options, Option 2 was selected by mine management as the best balance between economic impact and the acceptable level of business risk (with no deterioration in safety requirements). This change was implemented in November 2010 and coincided with the upgrade of the slope monitoring systems. The upgrade included automated prism monitoring (hourly readings) and slope stability radar to characterize the response to mining. Increased hazard awareness education for mine personnel was completed and a TARP (trigger action response plan) was developed and implemented.

3.5 Continuous Monitoring and Reduced Mining Rate

With the introduction of the slope decoupling berm, mining progressed to the 590RL in January 2011 and to the 580RL later in May 2011. The most critical area identified in the 2010 work had been successfully mined through. Coincidentally the Pit 6 mining rate was reduced, with increased development in adjacent mining areas.

The reduced mining rate provided adequate time for further slope design reconciliation in the newly exposed benches and revealed:

- Significant increase in confidence surrounding the recently updated 2010 structural geology model;
- Improved rock mass conditions in DG2 and DG1 (higher than expected GSI) in lower benches.

During this time, the slope was being monitored and no signs of instability were detected:

- Slope stability radar was operational and did not detect any significant deformation;
- Prism monitoring showed minimal creep deformation, in the order of 1mm per month (i.e. no further accelerations had been observed in 2011 as illustrated in Figure 4);
- No further extension was observed along the tension cracks behind the slope crest.

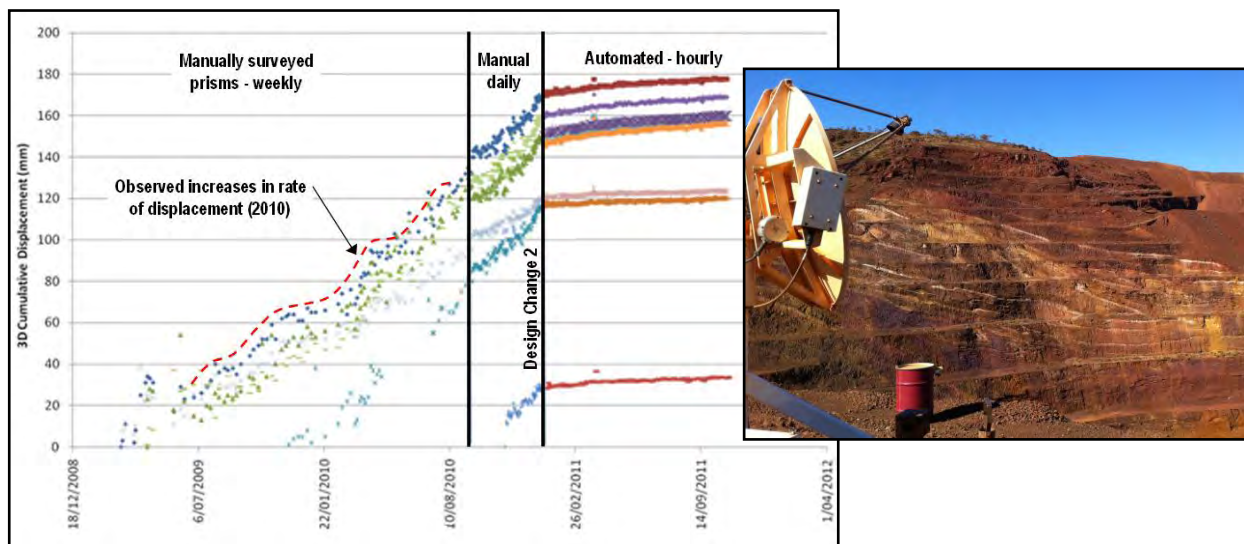


Figure 4: Sample of prism displacements and photograph of radar monitoring southern slope

During the initial six months of automated prism monitoring, prism alarms associated to deformation and velocity thresholds were established. The prism alarms were linked from the Brockman Operation to pit controllers in the Perth Remote Operations Centre to enable 24 hour monitoring coverage. Pit evacuation plans and procedures (TARP) were developed in the event of prism alarm activation or system malfunction.

3.6 Radar Removal and Geotechnical Review (Design Change 3)

The slope stability radar was removed from the site in September 2011 following a significant period of monitoring which indicated that the slope had stabilized (radar was relocated to another RTIO Pilbara Operation with an elevated slope instability risk at the time).

A comprehensive review of the geotechnical model was undertaken following reconciliation findings, confirming the structural geology model and the detection of improved rock mass conditions in the lower benches (RTIO, 2011). Shear strength parameters were revised and a notable increase in rock mass strength in the order of 15% was observed in the lower benches. Stability analysis was carried out using two dimensional limit equilibrium models and indicated that both the lower and overall slopes were stable with adequate factors of safety.

Slope optimisation options were also investigated as a part of the 2011 review and determined that 140,000t of additional high grade ore could be extracted simply by increasing the batter height back to 20m in the lower part of the slope (i.e. steepening the lower slope below 600RL for the final 40m of vertical depth).

Stability analyses were carried out using the updated geotechnical model and indicated that steepening would not significantly impact the stability of the lower and overall slopes. The condition of the upper slope (above 600RL) condition remained unaffected as illustrated in Figure 5. A series of sensitivity assessments were undertaken to increase confidence in the stability models of the lower and overall slopes with changes in structural geology and shear strengths.

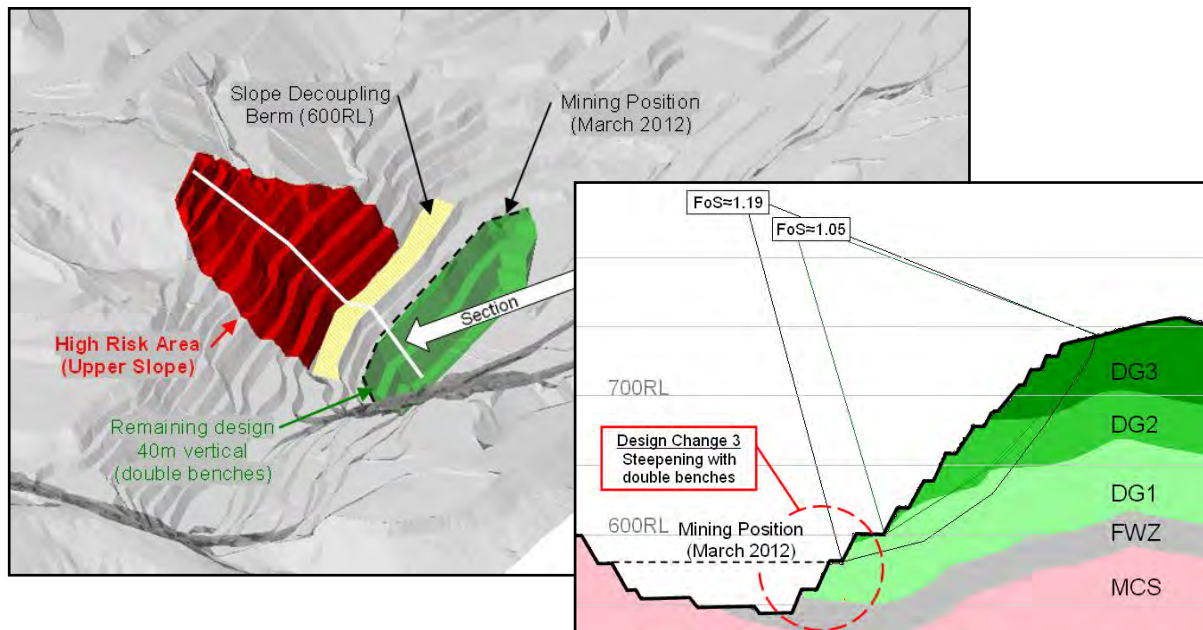


Figure 5: Upper slope high risk area, slope decoupling berm and section with 2010 structural geology model and double benches in the lower slope (Design Change 3)

The design change was implemented and is being monitored with no indications of destabilization to date. The risk presented by the upper slope continues to be managed with hazard awareness, and pit evacuation plans and procedures associated with alarmed prism monitoring.

Slope design reconciliation is planned as benches progress further below the southern slope and flexibility in the design permits further changes in the event that poorer than expected conditions or slope destabilization are encountered.

4. CONCLUSIONS

The most accurate or reliable geotechnical model is typically only achieved when mining is complete or is nearing completion. Verifying design assumptions by ongoing pit wall mapping and monitoring slope performance during implementation are paramount to implementing effective geotechnical slope management and achieving optimized slope designs.

The rigorous slope design reconciliation process has been successfully implemented across the higher risk areas within the 14 RTIO Pilbara Operations, and has been instrumental in early identification and management of potential geotechnical design issues. This process of ongoing refinement of slope design parameters, has enabled plans to be developed, presented to management and implemented involving slope remediation (where actual conditions were less favourable than expected) and slope design optimization (where more favorable than expected conditions were verified) over the past two years.

5. ACKNOWLEDGEMENTS

The author wishes to acknowledge to contribution of RTIO colleagues past and present in the development of the slope design reconciliation process and its implementations across the Pilbara operations.

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THERMO-MECHANICAL BEHAVIOUR OF KAOLIN CLAY

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ABSTRACT

This paper presents the findings of a laboratory investigation on the thermo-mechanical behaviour of kaolin clay. The tests were performed on reconstituted samples of kaolin using a modified triaxial apparatus capable of testing samples at elevated temperatures. The range of temperatures that were investigated was between 20 and 60°C. Several different temperature loading patterns were used to gain insights into the influence of temperature and temperature history on the strength and stiffness properties of kaolin clay. The study revealed that increasing the temperature of the soil caused changes to the shear strength and elastic modulus during shearing. The modulus was generally observed to increase with increasing temperature. The shear strength was observed to increase when the temperature was raised from 20 to 40°C and observed to decrease or remain unchanged when the temperature was raised from 40 to 60°C. The failure modes of the samples was observed to be different in samples tested at 60°C compared to samples tested at 20 and 40°C.

1. INTRODUCTION

Understanding the thermo-mechanical behaviour of soils is important in the design of many thermally active ground structures. For example, oil and gas pipelines (Slegel and Davis, 1977), underground power cables (Campbell and Bristow, 2009), geothermal energy foundations (Brandl, 2006) and clay barriers for nuclear waste disposal facilities (Tang and Cui, 2006) all induce temperature changes on the ground surround these structures.

Recently concerns over the adverse effects of greenhouse gas emissions from the use of traditional energy resources such as fossil fuels have resulted in a global push towards alternative, more energy efficient technologies. Shallow geothermal energy and in particular energy foundations is one technology that has been receiving considerable amount of interest. Energy foundations interact with the subsurface through pipe networks placed in building foundation elements such as piles, allowing heat energy to be extracted from or rejected into the ground. The underground pipe networks are used in conjunction with a heat pump to provide energy efficient space heating and cooling in buildings.

Although the use of energy piles is becoming more widespread, little is known about the influence of temperature on the mechanical performance of energy piles. In particular, the influence of temperature induced changes to soil stiffness and settlements of energy foundations are not yet fully understood. The majority of studies investigating the thermo-mechanical behaviour of soils have been undertaken at temperatures much greater than the operating temperature of energy foundations. In addition, conflicting experimental results have been obtained on the influence of temperature on the strength of soils.

This study presents the results of an experimental investigation into the effects of the thermo-mechanical behaviour of kaolin clay at temperatures of 20, 40 and 60°C. The effect of temperature and temperature history on the shear strength and stiffness of kaolin was investigated. A modified triaxial apparatus capable of handling temperatures of above 60°C was used in this study. This paper discusses the experimental setup, sample preparation and results of the temperature controlled triaxial tests. In addition, before presenting the results of the study a review of the literature is discussed and summarised.

2. BACKGROUND

Research into the effect of temperature on the mechanical behaviour of soils dates back to the 1930s. One of the earliest studies was undertaken by Gary (1936) who investigated the influence of temperature on the consolidation properties of soils. More recently there have been numerous investigations into the influence of temperature on the shear strength of soils, particularly clay. Many of the studies have been undertaken at temperatures of between 20 and 100°C specifically for the design of nuclear waste disposal facilities.

Despite the large number of investigations undertaken, the shear strength behaviour of saturated clays at elevated temperature is still not clear. Investigations by Ghahremannejad (2003); Hueckel and Baldi (1990); Lingnau et al. (1995); and Moritz (1995) all found that increasing the temperature reduced the shear strength of the soil. On the other hand, Abuel-Naga (2006); Cekerevac and Laloui (2004); Graham et al. (2001); Houston et al. (1985); and Kintiwattanakul et al. (1995) all found that the shear strength of the soil increased with increasing temperature. Kuntiwattanakul et al. (1995) explained some of the variations reported in the literature after conducting their own

study on samples of kaolin clay at temperatures of 20 and 90°C. They conducted undrained triaxial tests on both normally consolidated (NC) and over consolidated (OC) samples under both drained and undrained heating conditions. Kuntiwattanakul et al. found that the shear strength increased with increasing temperature under drained heating and decreased under undrained heating for NC samples. For OC samples they observed no significant changes in shear strength. Through this study Kuntiwattanakul et al. were able to resolve some of the differences reported in the literature. They reported that the decrease in shear strength observed by Murayama (1969) and Sherif and Burrous (1969) probably arose from undrained heating. However, they were not able to explain the differences between increasing and decreasing strength observed in the literature for undrained triaxial tests with drained heating paths.

Normally consolidated samples subjected to undrained heating and undrained triaxial tests were reported to decrease in all cases (De Bruyn and Thimus, 1996; Kuntiwattanakul et al., 1995; Moritz, 1995). Kuntiwattanakul et al. attributed this to a build up of excess pore pressure within the sample due to the expansion of the water particles, which caused a reduction in the effective stress of the sample and as a result a reduction in the strength and stiffness of the soil. The influence of temperature on samples where drained heating was performed is however much less clear. When drained heating was followed by drained triaxial tests the strength was reported to increase (Abuel-Naga, 2006; Cekerevac and Laloui, 2004). When drained heating preceded undrained triaxial tests an increase, decrease or no change in strength have been reported. Hueckel et al. (2009) conducted a review of previous research in an attempt to explain these differences. They concluded that there was no unique solution in the temperature range of 4 to 120°C and that the influence of temperature on strength was material specific and dependent on the history of the thermo-mechanical loading.

Overconsolidated samples tested by Cekerevac and Laloui (2004) showed an increase in strength at higher temperatures however the increase was slightly smaller than in normally consolidated samples. Kuntiwattanakul et al. (1995) on the other hand found the OC samples showed no change in strength with increasing temperature compared to NC samples which showed an increase. Hueckel and Baldi (1990) also tested OC samples and found a reduction in shear strength with increasing temperature. In these cases OC samples appeared less likely to cause an increase in strength with increasing temperature compared to NC samples.

A number of studies have also investigated the influence of cycling the temperature of soil. Single temperature cycles were performed by Abuel-Naga (2006) from 20 to 90 and back to 20°C prior to shearing. Abuel-Naga (2006) found that temperature cycling increased the shear strength of the soil by a similar magnitude as samples tested at elevated temperatures. However, the maximum shear strength was observed to occur at lower strains in temperature cycled samples. Kuntiwattanakul et al. (1995) also performed tests on samples subjected to thermal cycles to 90°C. They found the samples showed no increase in strength however they did observe an increase in stiffness. Similarly, Burghignoli et al. (2000) reported no change in strength and an increase in stiffness for soils subjected to temperature cycles.

The stiffness of soil has also been observed to be influenced by temperature. In most studies the stiffness of the soils was shown to increase if shear strength increased or decreased if shear strength decreased at elevated temperatures. However, in studies that reported no change in shear strength by Burghignoli et al. (2000) and Kuntiwattanakul et al. (1995) the stiffness was observed to increase with increasing temperature. Kuntiwattanakul et al. (1995) found that the magnitude of increase decreased as the overconsolidation ratio of the soil increased.

In summary, the literature shows that whilst under undrained conditions, clays subjected to heating followed by shearing causes a reduction of strength and stiffness. However, there are still uncertainties surrounding the influence of temperature on the undrained shear strength of samples subjected to drained heating. More experimental work is required in this field to help understand the mechanism influencing the strength of clay subjected to drained heating conditions followed by undrained shearing.

3. EXPERIMENTAL SETUP

The high temperature triaxial apparatus used in this study consisted of a standard triaxial apparatus which was modified to apply elevated temperatures to the soil specimen. The standard triaxial setup was a commercially available triaxial apparatus manufactured by Geocomp Corporation. The setup of the triaxial apparatus consisted of a triaxial cell and a loading frame equipped with a load cell and displacement transducer to measure the vertical load and displacement of the sample. Two flow pumps were connected to the triaxial cell and controlled the sample and cell pressure. The flow pumps and loading frame were connected to a central computer that recorded and controlled the load, displacement and pressure of the system through a computer program.

The temperature of the soil specimens was elevated by heating the triaxial cell with an electronic heating element that encapsulated the triaxial cell. The heating jacket was placed over the aluminium outer cylinder of the triaxial cell and connected to a temperature control box. A type K thermocouple was inserted between the heating jacket and aluminium cell to regulate the temperature of the cell. A calibration test was performed on a trial sample with thermocouples

inserted into the cell to measure the water temperature and the temperature of the sample. The temperature inside the cell was about 0.5°C lower at 40°C and 1.5°C lower at 60°C this was allowed for during testing. The schematic of the triaxial cell can be seen in Figure 1. The cell was used to test specimens of 63 mm in diameter and 126 mm in length.

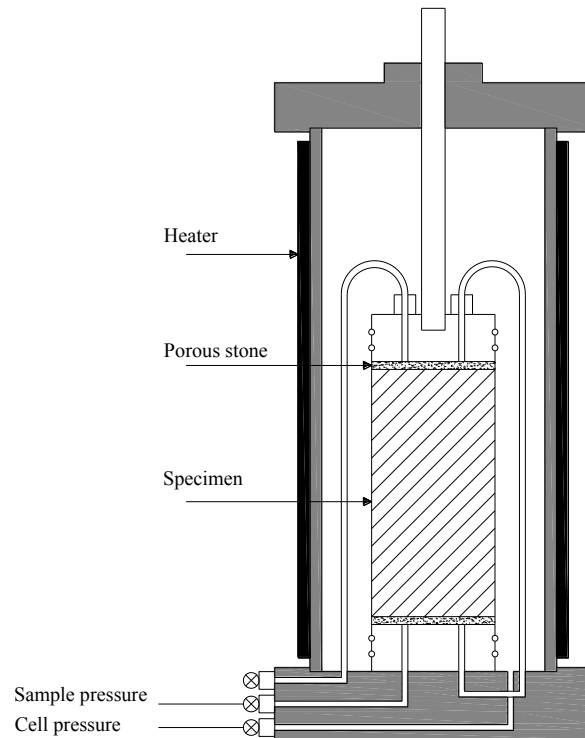


Figure 1: Schematic drawing of triaxial cell

3.1 SAMPLE PREPARATION

Samples of remoulded kaolin were prepared by consolidating a block of kaolin slurry. The soil was consolidated in a steel mould 160 mm wide, 560 mm long and 300 mm deep. The kaolin was prepared in a slurry form with a moisture content of approximately 100% which is equal to twice its liquid limit. The bottom of the mould contained small drainage holes to allow drainage at the base of the block. The sides of the mould were covered with petroleum jelly to facilitate the removal of the block after consolidation. A porous plate and piece of geotextile was then placed at the bottom of the block to allow drainage at the base. The slurry was then poured into the mould to a height of approximately 270 mm and a geotextile and porous plate were placed on top of the slurry. Steel blocks were then placed on top of the porous plate to apply load. The slurry was initially consolidated using dead weights to a pressure of 20 kPa. The consolidation process was monitored daily to ensure even consolidation across the length of the block and ensure that the top and bottom of the kaolin block remained saturated. After the block was consolidated to 20 kPa with dead weights a loading jack was used to consolidate the sample to 200 kPa. The entire consolidation process took approximately 25 days after which the sample was extruded then wrapped in plastic and placed in a curing room at 100% humidity until samples were tested.

Samples were collected from the block by pushing a 63 mm diameter tube (U63) into the kaolin block. The sample was then removed from the 63 mm tube with a hydraulic ram and trimmed to a height of between 126 -130 mm for testing. The dimensions of the sample were measured at several points and the average length and diameter established. The mass of the sample was measured and the off cuts were used to obtain the initial moisture content of the specimen.

4. EXPERIMENTAL RESULTS AND DISCUSSION

In this study the influence of temperature on the shear strength of kaolin clay was investigated on eight normally consolidated samples of kaolin. All samples were consolidated to a confining pressure of 200 kPa. The samples were tested at constant temperatures of 20 (room temperature), 40 and 60°C or after temperature cycles of 20-40-20, 20-60-20°C or multiple temperature cycles (sample K-8). Heating the samples was undertaken under drained conditions immediately following consolidation. Heating caused the samples to consolidate due to the expansion of the pore water in the samples. The volume change was not permanent and samples that underwent temperature cycles recovered approximately 80% of the volume contraction during cooling. The cell pressure was maintained at a constant pressure

throughout the heating phase. The specimens were then sheared to failure under undrained conditions. The results of the triaxial tests on the kaolin clay are summarised in Table 1.

Sample number	Temperature (°C)	Confining stress (kPa)	Maximum deviator stress (kPa)	Initial dry density (before heating) (kg/m ³)	Secant modulus (50% load) (MPa)
K - 1	20	200	116.0	1245	13.9
K - 2	40	200	128.6	1246	22.5
K - 3	60	200	112.5	1258	26.5
K - 4	20-40-20	200	125.8	1248	31.0
K - 5	20-60-20	200	112.6	1249	38.7
K - 6	60	200	127	1239	15.4
K - 7	60	200	122.3	1234	26.2
K - 8	20-40-20 x 4 cycles	200	125.1	1248	32.6

Table 1: Results of undrained triaxial tests on kaolin

The temperature induced change in shear strength of kaolin clay is illustrated in Figure 2 and Figure 3. Figure 2 presents the results from the tests undertaken at temperatures of 20 and 40°C. The shear strength of the kaolin was observed to increase when the temperature was increased from 20 to 40°C in all three samples. The magnitude of strength increase was slightly lower in samples that were subjected to temperature cycles and the peak stress was achieved at a lower strain in temperature cycled samples. No differences were observed between the samples subjected to one temperature cycle (K-4) and four temperature cycles (K-8).

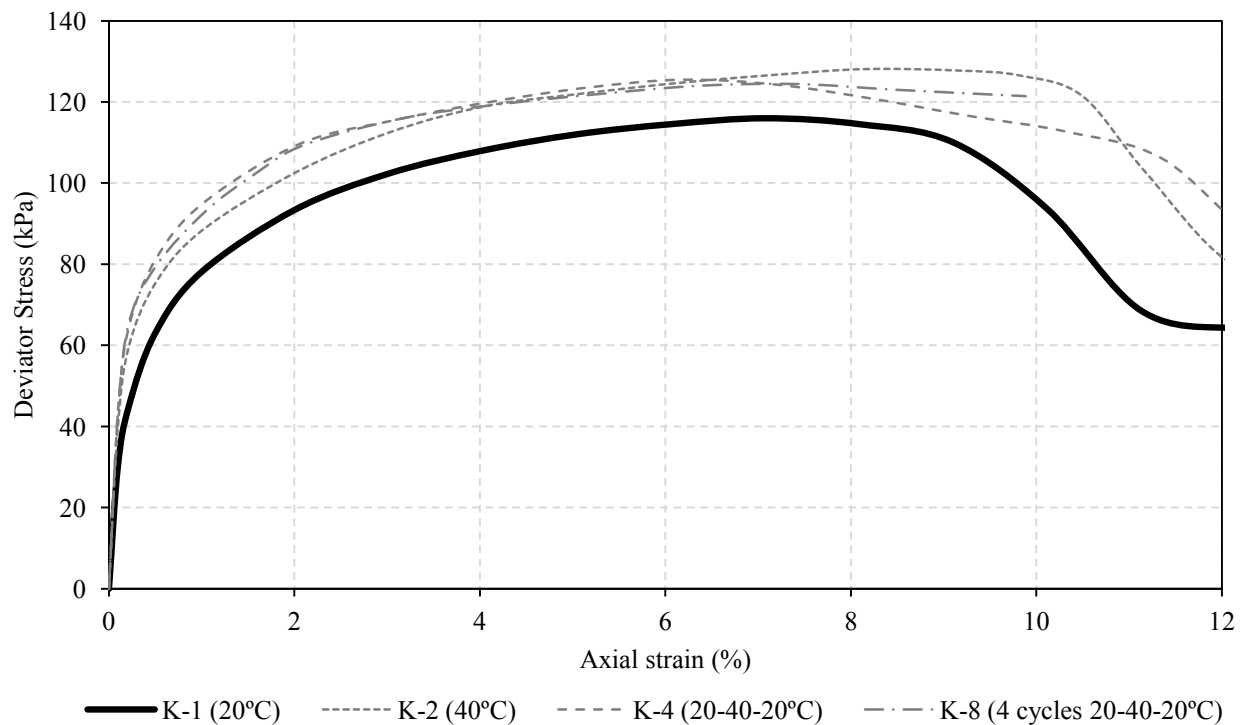


Figure 2: Stress strain plot for undrained triaxial tests at temperatures of 20 and 40°C

Figure 3 presents the results from the tests undertaken at temperatures of 20 and 60°C. The influence of temperature on the shear strength in these tests was much more complex. Unlike the tests taken at 40°C where all three tests followed an almost identical curve, three different curves were observed in samples subjected to temperatures of 60°C. Initially, tests K-3 and K-5 were undertaken, these tests both followed identical failure curves. They also failed at a significantly lower strain than samples tested at 20 and 40°C and at a lower load than the room temperature sample. The samples showed a different failure mode compared to samples tested at room temperature and 40°C. A single shear failure was observed through samples tested at 20 and 40°C (Figure 4 (a)). On the other hand, a dual shear failure with failure planes approximately perpendicular to one another was observed in samples K-3, K-5 and K-7 (Figure 4 (b)).

An additional two samples were tested at 60°C to validate the results of samples K-3 and K-5. Sample K-6 and K-7 were undertaken approximately 12 months after samples K-3 and K-5. The samples were tested under identical conditions using the same equipment and the specimens were sampled from the same kaolin block. Samples K-6 and K-7 both showed slightly different behavior when compare to the earlier samples and also when compared to each other. Unlike the initial tests at 60°C both samples showed an increase in strength compared to the sample tested at room temperature. However, the samples did not show an increase in strength compared to the samples tested at 40°C. Sample K-7 showed a dual shear failure similar to samples K-3 and K-5 while sample K-6 showed a slightly different failure mode with the development of two parallel shear planes as illustrated in Figure 4 (c).

The results from this study are unique in that both an increase and decrease in the shear strength of the soil was observed depending on the magnitude of the temperature. Different samples of kaolin subjected to a temperature of 60°C also exhibited different failure modes, failure strains and failure loads. A review of the literature found no reference to the inconsistencies observed in this study. Further research is needed to explain the differences occurring in samples tested at 60°C and to understand why the strength of the soil is increased when the temperature was elevated to 40°C and decreased or remained unchanged when the temperature was further increased to 60°C.

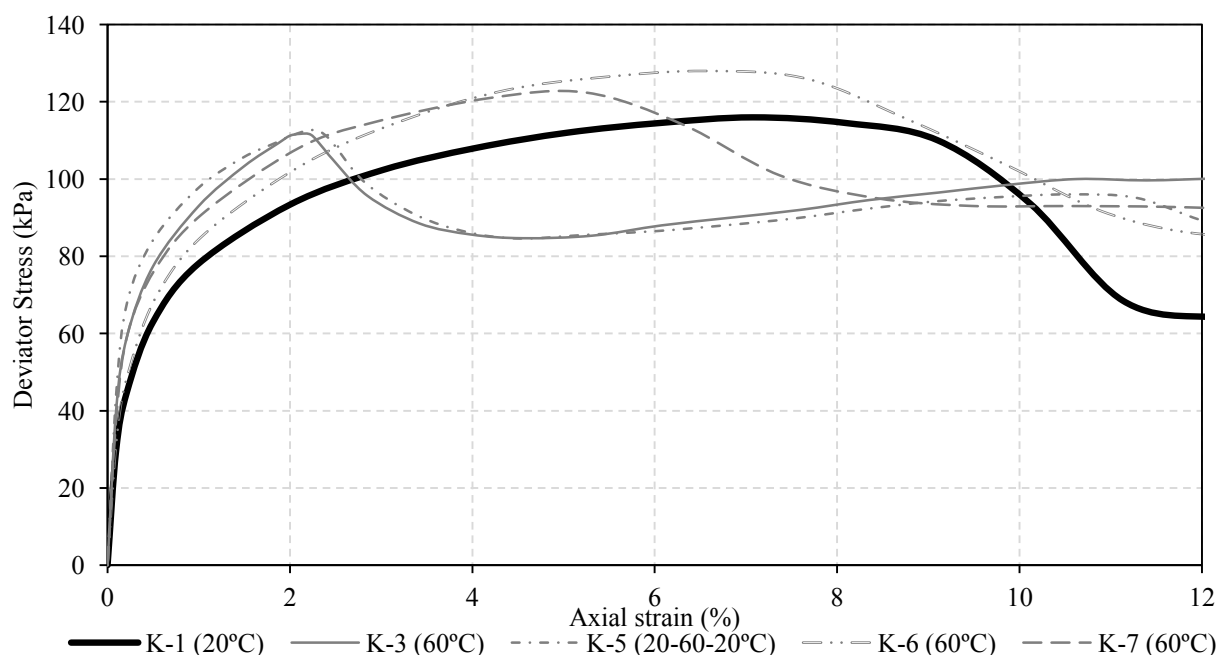


Figure 3: Stress strain plot for undrained triaxial tests at temperatures of 20 and 60°C

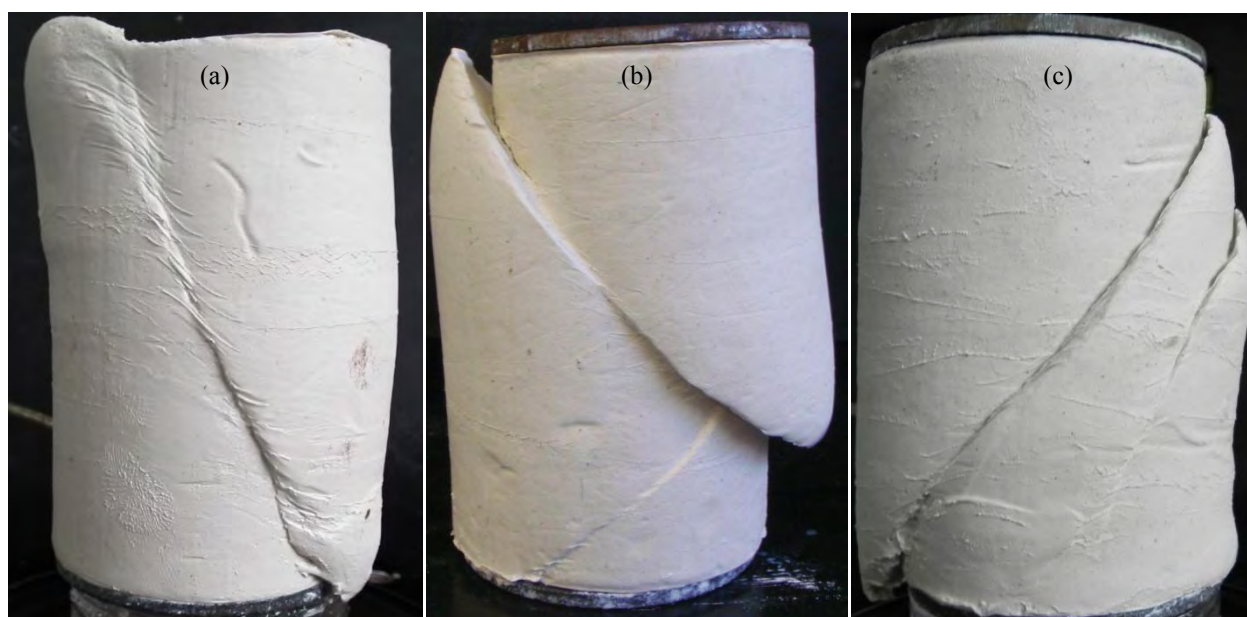


Figure 4: Shear failure of sample (a) K-1, K-2, K-4 and K-8 (b) K-3, K-5 and K-7 (c) K-6

5. CONCLUSIONS

The influence of temperature on the shear strength of eight normally consolidated samples of kaolin was experimentally investigated at various temperatures using a modified triaxial apparatus. The samples were tested at constant temperatures of 20, 40 and 60°C and after temperature cycles of 20-40-20, 20-60-20°C or multiple temperature cycles (sample K-8). The conclusions drawn from this study are summarised below:

- The shear strength was observed to increase when the temperature of the kaolin was increased from 20 to 40°C.
- Samples subjected to thermal cycles and then tested at room temperature showed a similar change in strength to samples tested at an elevated temperature of the same magnitude.
- No differences were observed between samples subjected to one temperature cycle and four temperature cycles.
- The shear strength was observed to decrease or remain unchanged when the temperature of the kaolin was increased from 40 to 60°C.
- Sample tested at a temperature of 60°C exhibited different failure strains and failure loads when tested under the same conditions.
- Sample tested at a temperature of 60°C showed a different failure mode to samples tested at 20 and 40°C.

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KARST GEOHAZARD – A CASE STUDY IN THE CENTRAL PILBARA

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ABSTRACT

A karst geohazard was identified in the vicinity of a proposed iron ore processing plant, in the Pilbara region of Western Australia. Subsidence sinkholes have been identified at the ground surface in the general area, and Tertiary calcrete and Archean dolomite units, prone to dissolution and cavity development, occur in the stratigraphy underlying the site.

Sterilisation drilling to ensure absence of iron ore at the plant site encountered cavities in 27 of 132 boreholes. A 3D subsurface model of the site was prepared based on the results of the drilling. A trial of five geophysical techniques, downhole laser telemetry and downhole videoing was conducted. The results correlated well with the stratigraphic model but locating cavities remains problematic. Drilling of 29 cored geotechnical boreholes targeting key plant infrastructure and geophysical anomalies was used to review the 3D ground model and determine rock mass characteristics. Published nomograms of cavity roof thickness, rock mass quality and engineering load have been utilised to assess subsidence risk and detailed construction recommendations have been provided to minimise this risk.

Investigations to date have provided substantial input to the proposed plant design and development of allowances should karst features be encountered during construction or operations. Works are ongoing to develop further understanding of the potential locations of specific karst features and for input into project risk management strategies.

1. INTRODUCTION

Karst is a geological feature caused by the dissolution of soluble carbonate rocks. The primary mechanisms of karst formation are rain and groundwater that dissolves particular gases in the atmosphere including carbon dioxide (CO₂) and other water soluble minerals in the earth's crust i.e. sulphides, resulting in the water becoming slightly acidic. The slightly acidic groundwater subsequently passes through the natural defects in rock and dissolves the soluble carbonate material. Over time, the dissolution is accelerated as the volume of water increases resulting in subsurface features including fissures and caves.

There are two types of carbonate rocks commonly found in the Central Pilbara of Western Australia; Tertiary age calcrete and Archean age dolomite both of which are widely known to be naturally affected by karst. Of the several types of karst formations that can affect surface topography, the main type which is relatively common in the Central Pilbara is sinkhole features. Typical sinkholes associated with dissolution of outcropping rock comprise Solution Sinkholes, Collapse Sinkholes and Caprock Sinkholes. Where cavities develop in soluble rock which is covered by soil, and the overlying soils are subsequently transported into the cavities, Dropout Sinkholes, Suffosion Sinkholes and Buried Sinkholes occur. Each of these types of Sinkhole is illustrated in Figure 1 (Bell, Culshaw & Waltham, p. 27).

A karst can pose a risk to the integrity of infrastructure, or in extreme cases, human safety. The proposed construction of an iron ore processing plant adjacent to an identified area of outcropping calcrete rock in the Central Pilbara required design engineers and the mining company* to better understand the potential for karst geohazard to affect development and operation of the plant.

2. CASE STUDY

Development at the new mine will include construction of process infrastructure such as crushers, conveyors, screening buildings, transfer stations, stackers and reclaimers and a train load out, and non processing infrastructure such as workshops and warehouses. The total area of the proposed site spans across over 5km² (500ha) most of which is located over low-lying areas susceptible to karst. A key component of the geotechnical investigation carried out for the proposed project was to locate, identify and understand the type of karst present within the vicinity of the proposed site and assess the potential risk of karst geohazard affecting the operation of the proposed processing plant. The investigation process was carried out over several stages.

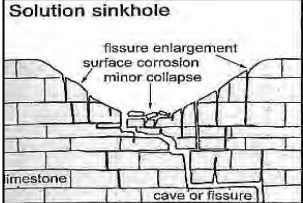
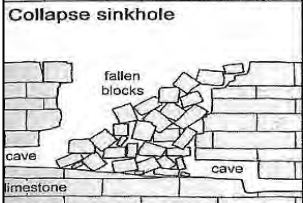
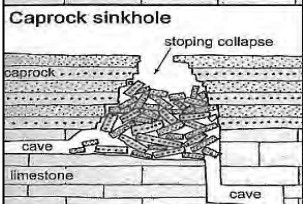
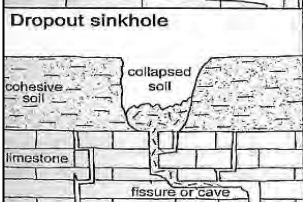
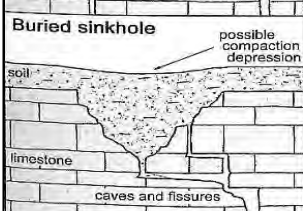
	<i>Formation process</i> <i>Host rock types</i> <i>Formation speed</i> <i>Typical max size</i> <i>Engineering hazard</i> <i>Other names in use</i>	Dissolutional lowering of surface Limestone, dolomite, gypsum, salt Stable landforms evolving over >20,000 years Up to 1,000 m across and 100 m deep Fissure and cave drains must exist beneath floor Dissolution s/h, cockpit, doline
	<i>Formation process</i> <i>Host rock types</i> <i>Formation speed</i> <i>Typical max size</i> <i>Engineering hazard</i> <i>Other names in use</i>	Rock roof failure into underlying cave Limestone, dolomite, gypsum, basalt Extremely rare, rapid failure events, into old cave Up to 300 m across and 100 m deep Unstable breakdown floor; failure of loaded cave roof Cave collapse s/h, cenote
	<i>Formation process</i> <i>Host rock types</i> <i>Formation speed</i> <i>Typical max size</i> <i>Engineering hazard</i> <i>Other names in use</i>	Failure of insoluble rock into cave in soluble rock below Any rock overlying limestone, dolomite, gypsum Rare failure events, evolve over >10,000 years Up to 300 m across and 100 m deep Unstable breakdown floor Subjacent collapse s/h, interstratal karst
	<i>Formation process</i> <i>Host rock types</i> <i>Formation speed</i> <i>Typical max size</i> <i>Engineering hazard</i> <i>Other names in use</i>	Soil collapse into soil void formed over bedrock fissure Cohesive soil overlying limestone, dolomite, gypsum In minutes, into soil void evolved over months or years Up to 50 m across and 10 m deep The main threat of instant failure in soil-covered karst Subsidence s/h, cover collapse s/h, alluvial s/h
	<i>Formation process</i> <i>Host rock types</i> <i>Formation speed</i> <i>Typical max size</i> <i>Engineering hazard</i> <i>Other names in use</i>	Down-washing of soil into fissures in bedrock Non-cohesive soil over limestone, dolomite, gypsum Subsiding over months or years Up to 50 m across and 10 m deep Slow destructive subsidence over years Subsidence s/h, cover subsidence s/h, alluvial s/h
	<i>Formation process</i> <i>Host rock types</i> <i>Formation speed</i> <i>Typical max size</i> <i>Engineering hazard</i> <i>Other names in use</i>	Sinkhole in rock, soil-filled after environmental change Rockhead depression in limestone, dolomite, gypsum Stable features of geology, evolved over >10,000 years Up to 300 m across and 100 m deep Local subsidence on soft fill surrounded by stable rock Filled s/h, compaction s/h, paleosinkhole

Figure 1: The six types of sinkholes.

2.1 BACKGROUND GEOLOGY

The Central Pilbara is located in the Pilbara Craton in the north west of Western Australia. The underlying bedrock associated with Iron Ore mining comprises rocks from the Hamersley Group, which occupy the Hamersley Basin. The bedrock dates from Late Archean to Early Proterozoic (2765 Ma to 2470 Ma) and generally comprises various units of banded iron formation (BIF), shale, chert and dolomite. The Hamersley Basin has been affected by several phases of folding at local and regional scales forming broad anticlines and synclines.

The proposed plant site is located in a broad valley on the southern limb of a breached anticline. The bedrock at the site dips at between 10° and 25° to the SSE. Following the breaching, weathering and denudation of the centre of the anticline, the overlying BIF and shale rocks have been eroded back, most likely by breakaway processes, to form resistant scarp slopes to the south and north of the plant site, while the centre of the anticline has been eroded down to the underlying Paraburdoo Dolomite Member.

The Paraburdoo Dolomite Member is the oldest rock found in the area and is expected to underlie the entire site. When fresh the dolomite occurs as a blue grey, crystalline rock, of high strength. Due to the carbonate content of dolomite, it is susceptible to being dissolved in slightly acidic groundwater, resulting in karst development.

The bedrock is covered by a stack of valley infill deposits. The surficial geology is dominated by materials formed and deposited under environmental conditions prevalent in the Tertiary Period. The near-surface materials which immediately underlie the proposed site comprise sandy, silty gravel formed by the weathering, erosion and deposition of the parent bedrock found on the surrounding hills. The material is generally red brown in colour and becomes iron cemented, usually within 1m to 3m of the ground surface (ferricrete).

The ferricrete is underlain by a large body of calcrete which outcrops at the surface to the north west of the proposed site. The calcrete likely formed as result of a evaporite deposit of calcium carbonate in a large ancient lake setting with ideal shallow, warm ($>26^{\circ}\text{C}$) and clean water conditions. Calcrete is the other known rock encountered at the proposed site that is susceptible to karst because of its carbonate content.

2.2 INVESTIGATION STAGES

2.2.1 Desktop Study and Field Mapping

An initial desktop study comprised analysis of available information including published geological maps, geology papers, aerial and satellite photographs and images of the local area. The aerial photographs overlayed with the preliminary plant design displayed the majority of the proposed plant site to be located over wash zones and inter-fluvial spurs. Outcropping calcrete adjacent to the proposed plant was observed. A large sinkhole feature up to ~90m across was identified within 200m of one of the proposed primary crushers. The feature was initially interpreted to be either a suffosion or a drop-out sinkhole. A site inspection was carried out and the large sinkhole appeared to have formed over a sustained period of time and no longer appeared to be active with an absence of fresh scarps. The upper side scarps were observed to be generally moderately steep ($\sim 25^{\circ}$) with steeper sections due to the weakly to well cemented nature of the gravels and ferricrete. The lower reaches of the sinkhole are gently sloping (5°) and contain mostly fine material and some large mature trees indicating the sinkhole is slowly infilling and becoming inactive. It was decided that further subsurface investigations were required in this area (refer section 2.2.5).

A smaller, younger and active sinkhole was encountered some 15km from the proposed site yet still in the local area and near to the calcrete (see Figure 2). The sinkhole is approximately 12m in width, up to 4m deep and displays relatively fresh scarps and characteristics of an active drop-out sinkhole. The side scarps are near vertical due to the weakly cemented ferricrete and calcrete. The sinkhole confirmed the initial assessment that the karst geohazard exists in the subject area.

The outcropping calcrete adjacent to the site showed little evidence of karst development i.e. solution, caprock or collapse sinkholes at the surface. Minor cavities less than 0.5m wide and up to 1m deep were rarely encountered.



Figure 2: Active drop-out sinkhole located some 15km from the proposed plant site.

Based on desk top studies and mapping, allowances to cater for karst features should they be encountered were recommended. Options for remediating karst comprised excavation and backfilling with various schemes proposed

depending on the scale and depth of any features encountered. Allowances were required with respect to the cost of foundation design and for any remedial works that may be required during construction.

2.2.2 Sterilisation Drilling

A sterilisation drilling programme within the proposed processing plant footprint was carried out to rule out other potential iron ore deposits. The programme involved the drilling of 132 boreholes using reverse circulation (RC) drilling technique. Where possible, drilling was observed by the geotechnical team and in all cases chips were logged to provide data for the development of a 3D model of the underlying geology. Records of where cavities were encountered were taken from the drillers logs. The drillers reported “easy drilling progress, no resistance or dropping of the drill rods” (cavities) in 27 of the boreholes, ranging in thickness from 1m to up to 14m with most cavities reported to be less than 2m and located between 6m and 93m below ground surface (bgs). Cavities were predominantly encountered within the lower calcrete and the deep dolomite units. However cavities were also encountered within the Tertiary gravel ferricrete and the bedrock shales, which have been attributed to migration or collapse of cavities from underlying calcrete or dolomite respectively.

The calcrete was found to underlie the majority of the footprint of the plant from 3m depth to 48m and averaging between 16m bgs at the top and 40m bgs at the base. The upper calcrete (top 10m to 15m) was found to be white and rich in calcium carbonate fizzing vigorously with weak hydrochloric acid while the lower calcrete was found to be enriched in siliceous chert and to have a pale brown colour. The majority of the cavities encountered during the sterilisation drilling programme were within the lower calcrete.

Approximately 15% of the cavities encountered during the sterilisation drilling were within the dolomite, with one cavity of 14m depth. The Paraburdoo Dolomite is a very thick unit, the base of which was not found during the sterilisation programme, but geological data on the area suggests that the unit might be up to 400m thick. The upper surface of the dolomite had been inferred from the sterilisation drilling programme. It indicates a varying surface between 50m and 170m depth but averaging 90m bgs beneath the plant site. A local sequence of folding brings dolomite closer to the ground surface in the north and east of the plant site.

2.2.3 Geophysics Trial

The sterilisation drilling programme identified cavities but their size, extent and spatial variation away from the borehole locations remained unknown. Critically, given the number of cavities encountered it is considered that there could be other, as yet unidentified cavities beneath the site. A specialist geophysics company was engaged to carry out a trial using five different geophysical methods at four sterilisation boreholes with known cavities of various sizes and depths and one control hole without a cavity. The objective of the trial was to assess whether if subsurface geophysical methods could be used to directly detect voids or to delineate areas with higher susceptibility to voids. The geophysical methods used in the trial were Ground Penetrating Radar (GPR), Electrical Resistivity Imaging (ERI), Seismic Refraction, Refraction Microtremor (ReMi), and Multi-Analysis of Surface Waves (MASW).

For the GPR, a 50MHz rugged antennae was used to collect data in three lines of approximately 350m in length at 2m spacing centred on each borehole surveyed. According to the geophysicist, GPR was ‘found to be ineffective at resolving features in the subsurface due to the electrically conductive nature of the local materials impairing signal penetration’.

For the ERI, one 360m line with 72 electrodes spaced 5m apart was utilised at each test site. The ERI surveys conducted resolved the electrical properties of the survey area up to a depth of approximately 40m. It was found that these ERI surveys did not directly detect any known voids; however the surveys were found to be effective at mapping different lithologies which aid identification of any layers susceptible to the presence of voids. The ERI data at all sites showed three units: a highly resistive unit at the surface overlying a less resistive unit, with a third unit at depth which is less resistive than the middle unit. At all sites, the middle and lower units corresponded with the same two calcrete layers logged in the boreholes. The varied ERI response of the lower unit supported the notion that the lower calcrete unit is probably of higher porosity which was consistent with the sterilisation drilling.

For seismic refraction one 144m line of refraction data was collected using a 48 channel linear array with a geophone spacing of 3m at each site. A manual slide hammer was used as a source. ReMi data was collected using the same receiver geometry as the seismic refraction. One 144m line of MASW data was acquired using a 24 channel linear receiver array with a geophone spacing of 3m.

The ReMi data provided a broad overview of S-wave velocity variations (which correlate with soil / rock stiffness) at the site to approximately 60m below surface while the MASW provided a more detailed overview of the S-wave velocity variations down to 25m depth. In general, the ReMi and MASW data did not show any significant lateral variations suggesting that the resistive near-surface features seen in the ERI were unlikely to be voids. Seismic refraction resolved a refractor at 10m to 15 m which was the top of the calcrete layer. There were no significant lateral

variations in the P-wave velocities at the site which supports the notion that it is unlikely that large voids occur near the surface at the tested locations.

2.2.4 Downhole Survey

Following the geophysics trial a cavity auto scanning laser system (C-ALS) program was carried out. Of the 27 sterilisation boreholes with cavities encountered 11 of the boreholes were unable to be tested due to collapse or the presence of groundwater. The remaining 16 boreholes were scanned, either providing a horizontal diameter or if the size permitted a full 3D scan of the cavity. The equipment was also fitted with a camera so that a variety of photographs and videos could be taken.

The investigations have generally shown that the cavities are not large open structures, but form more of a honeycombed rock mass. The majority of the cavities (80%) were found to be less than 1.5m in diameter. One cavity was found to be up to 9m long and the largest cavity was found to be approximately some 5m by 5m in dimensions. The cavities were found to be irregular in shape rather than spherical.

2.2.5 Diamond Drilling

To further assess and address the areas of concern, a programme of ground breaking investigations was proposed to obtain key geological and geotechnical information about the sub-surface conditions at the proposed mine plant site. These investigations included the drilling of 29 fully cored PQ3 sized diamond boreholes to depths of between 20m and 100m bgs. The principal objective of the diamond drilling in relation to the karst investigation was to characterise the materials in the stratigraphy below the site, in particular the presence and distribution of calcrete and dolomite. Anomalies identified from the ERI survey during the geophysics trial were also targeted. Groundwater levels were indicated by the drillers and these were correlated to the site groundwater model for input to ongoing karst geo-hazard assessment, particularly with respect to potential drawdown arising from mining operations. Samples were collected from the boreholes and a programme of laboratory testing was carried out to obtain geotechnical parameters to assist with rock mass rating classification and karst roof review.

A borehole was drilled adjacent to the large sinkhole feature and revealed that the rock mass quality of the shale underlying the calcrete was extremely poor, had no structure and contained infill silts and clays from the overlying surficial sediments. In addition several cavities up to 1m deep were encountered within the calcrete and a cavity up to 2.2m deep was encountered at 93m bgl within the Paraburdoo Dolomite. This suggests that the karst sinkhole had possibly developed by piping and or collapse of the overlying shale bedrock over time into fissures and caves within the underlying dolomite. The voids created within the collapsed shale bedrock encouraged cavity development within the calcrete and piping erosion within the non-cohesive surficial sediments caused the subsidence sinkhole at the surface. A second borehole drilled at the adjacent proposed primary crusher indicated a continuous stratigraphy with uniformly structured shale.

Fifty five percent (55%) of the remainder of the diamond drillholes encountered cavities ranging in height from 0.1m to 1.5m. The majority of the cavities were encountered within the lower cherty calcrete from 25m to 40m depth and displayed a honeycombed structure with micro-voids similar to that observed in the downhole photos/videos. Some boreholes encountered lengths of core up to 6.8m that had been heavily affected by honeycombing. The upper calcrete displayed a massive structure with little evidence of any cavity development which was consistent with the assumptions made from the sterilisation drilling and ERI geophysical interpretation.

The depth and extent of the honeycombed calcrete was generally consistent with the anomalies displayed in the ERI images with a less resistive layer generally shown from 25m to 40m depth. The drilling did not however, encounter any physical correlation with near surface anomalies from the ERI.

A review of the stability of materials overlying cavities using the Rock Mass Rating (RMR) approach was carried out. The RMR system (Bieniawski, 1989) uses uniaxial compressive strength (UCS) of rock material, rock quality designation (RQD), spacing of discontinuities, condition of discontinuities, orientation of discontinuities and groundwater conditions.

The UCS had been obtained from laboratory tests from each stratigraphic unit encountered. RQD had been assigned to each core run recovered as rock. For discontinuities each unit has been assigned constant values to simplify calculations for each borehole. Groundwater levels were obtained from the sterilisation and diamond boreholes along with data provided by a hydrogeology company working for the mining company. In general, groundwater is presently found below the base of the calcrete and above the top of the dolomite, approximately 50m to 70m below the existing ground surface.

The results of the RMR classification were compared to the published nomogram developed by Nottingham University. For any design load, selected factor of safety, estimated RMR and known or inferred cavity width, a safe roof thickness can be estimated from these nomogram (Figure 3).

In addition to use of the nomogram an informal guideline to the stability of the natural roof rock over a cavity which says that the ground should be stable if the thickness of rock is equal to or greater than its span was also considered. This excludes any thickness of soil cover or heavily fissured limestone at rock-head (Waltham and Fookes, 2003). This guideline is considered to be conservative.

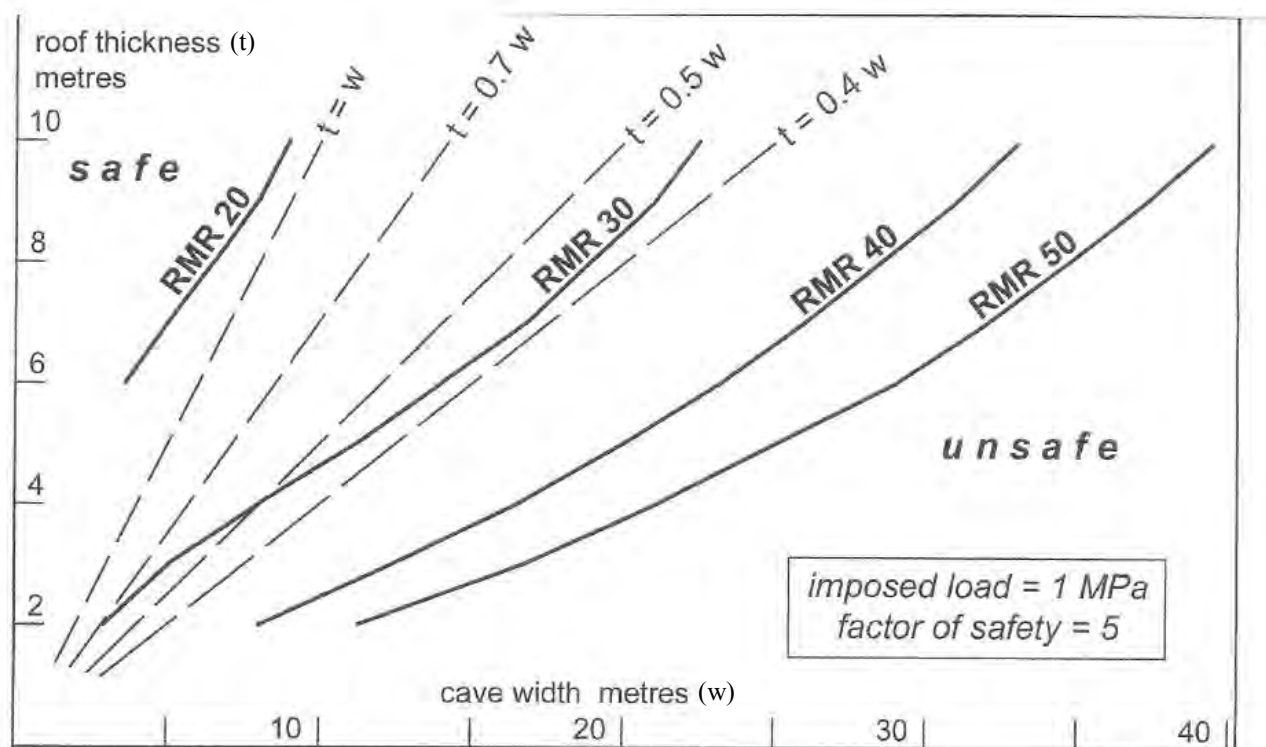


Figure 3: Nomogram that relates failure loads to cave width and roof thickness in ground of various rock mass ratings. Numerical models were generated with incremental loads applied to foundation pads, 1 m^2 , on the surface directly above the caves. The solid line represents situations of rock mass ratings with respect to cave width and roof thickness with a loading of 1 MN on the pad (1 MPa) leaving a factor of safety of less than 5, and is therefore considered unsafe. (From Waltham et al. 2005)

An averaged RMR had been calculated for the roof above each cavity, with the lowest RMR being 20 and the highest, 38. Both of these values indicate the rock mass (according to Beniaowski, 1989) is poor. Roof thickness varies between 7.15 m and 31.75 m. All encountered cavities appear to have safe roof thicknesses, assuming the cavities are equidimensional. The greatest span width encountered was assumed to be only 6.8 m wide. According to the nomogram in Figure 3, cavity roofs that exceed 14 m in thickness have been found to be stable with RMR 20, for span widths up to 15 m.

3. CONCLUSIONS

The potential for karst geohazard to affect a proposed iron ore processing plant in the Central Pilbara, WA was identified at the early stages of the geotechnical investigation at the site. Based on desk top studies and mapping, allowances to cater for karst features should they be encountered were recommended. Options for remediating karst comprised excavation and backfilling with various schemes proposed depending on the scale and depth of any features encountered. The allowances were required with respect to the cost of foundation design and for any remedial works that may be required during construction.

Following the preliminary works a comprehensive geotechnical investigation was carried out to determine and understand the extent and nature of the karst geohazard at the site. The investigations found that two types of carbonate rocks; calcrete and dolomite, contained cavities. Dolomite was generally encountered from 80m below ground surface and as shallow as 47m depth with cavities up to 14m depth. Calcrete was generally found to exist from 15m to 40m below ground surface and as shallow as 0.6m. The upper white calcrete (top 10m to 15m) was found to be of relatively

high strength, massive in structure and absent of any significant cavities. The lower brown cherty calcrete was found to be affected by minor to heavy honeycombing and cavities extending up to 6.8m deep were excavated.

Published nomograms relating cave width, roof thickness and rock mass quality provide a means of assessing cavity stability with respect to project development. Using rock mass ratings for the upper Calcrete only it appears safe roof thickness exists. The overlying ferricrete will also provide roof support but is not considered in the analyses.

Further investigation is required to determine the presence of any karst cavities in the upper 5 to 10m of the stratigraphy at the site. Such features have been encountered elsewhere in the region in similar conditions. Ongoing investigations including a second programme of Ground Penetrating Radar using different survey methodologies have been recommended to review the potential for such shallow cavities within the ferricrete beneath key plant infrastructure.

If no significant karst features are encountered in the second GPR investigation the risk of karst impacting on the proposed infrastructure would be considered to be relatively low so that changes to current design concepts are not required.

Continuous inspection will be required during construction to identify any evidence of karst features. The installation of monitoring systems to provide early warning of any subsidence that might occur beneath the infrastructure and allowances to remediate any movement if detected is also required. Careful attention to drainage such that water flows away from all foundation zones is required.

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A COMPARISON OF PREDICTED AND OBSERVED LIQUEFACTION-INDUCED SETTLEMENTS IN CHRISTCHURCH

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ABSTRACT

The geological model of a site is complex and three dimensional. As geotechnical engineers we develop the ground model by investigation, taking samples and measurements at discrete locations. Unlike other forms of engineering, where known quantities and parameters are used with recognised codes and methods, geotechnical engineering is a combination of art and science, where experience and judgment must be used to formulate workable solutions for our clients.

The upper soil profile in Christchurch comprises predominantly Holocene-aged, interbedded terrestrial and marine alluvial sediments with a relatively high groundwater table. This soil profile is susceptible to liquefaction when subjected to strong ground motions.

This paper explores the role of the geotechnical engineer in relation to our professional responsibility to make sense of the differences between predicted and observed liquefaction-induced settlements using sound engineering judgment.

This paper will examine a recent case study from Northlands Shopping Centre in Christchurch where we have compared predicted settlements with observed settlements resulting from the 4 September 2010 and 22 February 2011 earthquakes. In this case the predicted settlements differed from the observed settlements. There are many possible reasons for the differences we often encounter between the predicted and observed settlements. This paper discusses our rationale and how we have come to sensible geotechnical engineering conclusions and recommendations for our clients.

1. INTRODUCTION

Since the 4 September 2010 M7.1 earthquake, the Canterbury region has been shaken by over 10,500 well recorded aftershocks, including 40 over M5.0, two of M6.0 and one of M6.3. The most devastating event in the Canterbury aftershock sequence was the M6.3 earthquake on 22 February 2011. This was a particularly energetic event, located very close to Christchurch, at a shallow depth, and with very high ground accelerations (Holden 2011).

We are now in a unique position in Christchurch, where we have measured ground motions across the region. This enables us to carry out a “back analysis” for each event, as a sanity check to ensure our liquefaction-induced settlement calculations are comparable to observed and measured settlements on site.

2. GEOLOGICAL GROUND MODEL

A site’s geological model is complex and three dimensional. As geotechnical engineers we develop the model ground at a site via desk study and investigation, taking samples and measurements at discrete locations. This section provides a brief overview of the geological setting in Christchurch.

The Canterbury Plains are made up of a series of post-glacial alluvial fans, deposited by eastward flowing rivers from the foothills of the Southern Alps. The predominant surface geology in Christchurch is the Springston Formation, with historic lagoon, estuarine, swamp and dune field deposits (Christchurch Formation) forming the coastal fringe. Banks Peninsula, a volcanic complex at the south-eastern edge of Christchurch, was probably an offshore island until the late Quaternary Period (Brown & Weeber 1992).

Christchurch’s upper soil profile is predominantly Holocene aged, interbedded terrestrial and marine alluvial sediments with a relatively high groundwater table. This soil profile is susceptible to liquefaction when subjected to strong ground motions. The post-glacial upper soil profile is underlain by older (Pleistocene) alluvial gravels of the Burnham and Windwhistle Formations to the west of Christchurch, and Riccarton Gravel under the city and to the east. The underlying Riccarton Gravel Unit is usually encountered at an approximate depth of 10m in the west, to 40m in the eastern suburbs (Brown & Weeber 1992).

3. MEASURING GROUND ACCELERATIONS

Once we have established a ground model, we need to assess the liquefaction risk and likely ground deformations under the design earthquake scenarios of Serviceability Limit State (SLS) and Ultimate Limit State (ULS). Assessment of liquefaction risk is carried out with reference to New Zealand Geotechnical Society (NZGS) guidelines, and using best-practice methods, including Idriss and Boulanger 2008, and Robertson 2009. For the purpose of a liquefaction hazard assessment, earthquake scenarios are modeled using earthquake magnitude (M) and the peak ground acceleration (PGA).

We are in a unique position in Christchurch post 4 September 2010, whereby a dense network of GNS strong motion sensors across the region has recorded the ground motions for the Canterbury aftershock sequence.. Peak ground accelerations (PGAs) are archived in the New Zealand GNS strong-motion network database (GNS Science 2011).

Following liquefaction analysis we can compare our predicted liquefaction induced settlements for each of the recorded earthquake events, against actual measured settlements on site. This “sanity check” is an important step in verifying the methods used. The current provisional design parameters for liquefaction hazard assessment purposes, as at 27 April 2012 (for an IL2 structure and subsoil class D), (DBH 2012) are:

- SLS: M7.5 PGA 0.13g
- ULS: M7.5 PGA 0.35g

As a comparison to the SLS and ULS design parameters, typical ground motions experienced in the Christchurch CBD were:

- 4 September 2010: M7.1, PGA 0.15g to 0.20g
- 22 February 2011: M6.3, PGA 0.40 to 0.50g

In liquefaction hazard assessment the duration of shaking is represented by the earthquake magnitude. The earthquake demand on the land is represented by the Cyclic Stress Ratio (CSR), which is a function of earthquake magnitude and peak ground acceleration.

A comparison of the CSR profiles for these two events with the SLS and ULS events indicates that the 4 September 2010 and 22 February 2011 events “tested” the city to comparable if not slightly higher earthquake demands than the current SLS (September) and ULS (February) design scenarios for liquefaction hazard assessment.

4. PROFESSIONAL RESPONSIBILITIES

As professional engineers we are representing our client’s best interests and follow a code of ethics (IPENZ 2012).

Professional ethical values:

- Protection of life and safe guarding people
- Professional integrity and competence
- Commitment to community well-being
- Being aware of the environmental impacts of our work
- Sustaining engineering knowledge

The social and economic impacts of over-conservative engineering are of particular relevance in post-earthquake Christchurch. An overly conservative design may be prohibitive to build and or lead to excessive, unwarranted expenditure. An over-engineered or unrealistic engineering solution may result in the unnecessary closure of homes and businesses.

The calculation of liquefaction-induced settlement and lateral spreading is recognised as an inexact process. Sound geotechnical engineering judgment should be used when developing practicable solutions based on the output from analysis. There is a real risk that (apparently) user-friendly liquefaction analysis software can be used without the user understanding the methods employed or the relevance and sensitivity of the input parameters. This type of analyses can result in over (or sometimes under) predicted liquefaction induced settlements, with social and economic consequences.

Geotechnical engineers also have an ethical responsibility to inform building owners that for relatively little additional cost, additional earthquake resilience can be built into the foundations. Building a structure with increased resilience to future earthquake events will not only provide additional life safety, but is more environmentally sustainable, as the life span of the structure is potentially increased.

5. CASE STUDY: NORTHLANDS SHOPPING CENTRE

5.1 INTRODUCTION

Northland's Shopping Centre is a mall complex spanning an area of approx. 50,000m² in Papanui, in northern Christchurch. Coffey Geotechnics NZ Ltd was commissioned to investigate earthquake damage from the 22 February 2011 event. Northlands Shopping Centre was open and due to the closure of the Christchurch CBD and other shopping centres across the city, was providing valuable social and economic well-being, providing jobs and services.

5.2 GEOLOGICAL MODEL

The upper soil profile comprises alluvial sand and silt deposits, interbedded with marine sand, silt, clay and peat. Figure 1 below shows a typical CPTu record for the Northlands site.

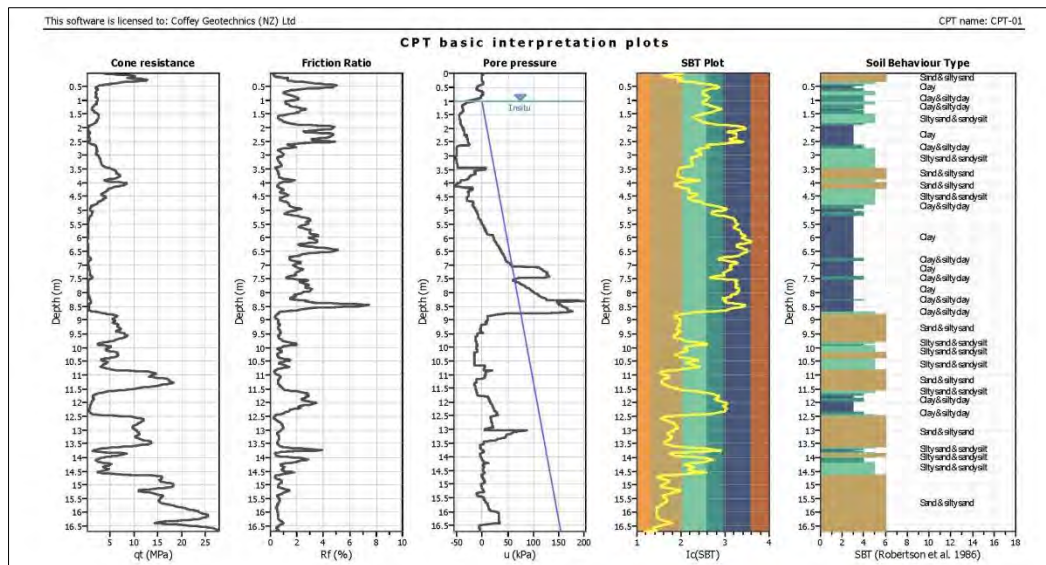


Figure 1: Typical CPTu Soil profile for Northlands

There is a notable abrupt transition in depth to dense gravel across the northern half of the site as shown below in Figure 2, which has significance for the site's liquefaction susceptibility. The soil units overlying the dense gravel are found to be liquefiable in all earthquake scenarios considered. The dense gravels are inferred to be the Riccarton Gravel Unit at the northern end of the site, and as a possible intermediate gravel layer at the southern end of the site. The dense gravel is considered as competent and non-liquefiable under the design earthquake scenarios.



Figure 2: Northlands site plan showing approx. depth to dense gravel

5.3 MEASURING GROUND ACCELERATIONS

One of the GNS strong motion network seismographs was located across the road (to the west) from Northlands, at Papanui High School. The peak horizontal ground accelerations PGAs for the 4 September 2010 and 22 February 2011 events as taken from the GNS strong motion database were (GNS Science 2011):

- 4 September 2010: M7.1, PGA 0.22g
- 22 February 2011: M6.3, PGA 0.25g

Using the PGAs (as shown above) from the Papanui High School Seismograph (PPHS), we carried out liquefaction induced settlement analyses for the 4 September 2010 and 22 February 2011 events.

Comparison of the Cyclic Stress Ratio (CSR) profiles for these two events indicated that the earthquake demand on the ground at the site was comparable. However, observations of damage did not compare well.

A comparison of predicted (based on the above PGAs) and observed liquefaction induced settlements are presented below in Table 1.

Earthquake event	Predicted ground surface settlement (mm)		Observed ground surface settlement (mm)	
	Southern half	Northern half	Southern half	Northern half
4-Sep-10	40-70mm	120-250mm	No observed settlement	25mm
22-Feb-11	30-50mm	100-230mm	<25mm	≈100mm

Table 1: A comparison of predicted and observed settlements at Northlands Mall

While the trend of increased settlement towards the northern half of the site was consistent, the magnitude of the predicted settlements was significantly higher than the observed values, especially for the 4 September event. While liquefaction may have been triggered in the 4 September event, the surface manifestations were minor.

Professional responsibilities dictated that before we could confidently report our findings we need to comment on these discrepancies. The 4 September 2010 and 22 February 2011 events had comparable ground motions. It was clear that the ground had behaved in markedly different ways for the two events. Just as it is important to get the geological model right, we must ensure the seismic parameters used at the site are also correct.

After a review of our methodologies and consultation with seismologists from the University of Canterbury and GNS Science, who have been researching the ground motions since 4 September 2010, we considered the following as contributing factors to the difference in observed and predicted liquefaction induced settlements:

- The geometric mean of the two horizontal components of the PGA record, better characterises the ground acceleration experienced at the site, rather than the PGA recorded;
- PGAs can be highly variable over short distances due to variable geological conditions in the upper 30m of ground;
- Actual duration of shaking for different Magnitude events; and
- The cumulative effects of smaller aftershock swarms.

Expert advice from seismologists from GNS Science and the University of Canterbury suggested the maximum PGA taken from the strong motion database, may be over conservative, and they advised that in engineering seismology, the geometric mean is generally considered to give a more characteristic representation of ground motion intensity (Bradley & Cubrinovski 2011).

The geometric mean PGAs for the Papanui High School strong motion recording site proved to be approximately 15% less than the maximum PGAs.

The variability of PGA over short distances can be shown with the example of the Christchurch Hospital and Botanic Gardens. The Christchurch Hospital seismograph measured a PGA of 0.35g on 22 February 2011, while approximately 900m away, the Botanic Gardens seismograph measured 0.48g for the same event. This is a 27% variance over 900m, roughly perpendicular to the 22 February source.

The Papanui High School Seismograph is located across the road from our site, however the Northlands site is approximately 500m long in a south-west to north-east direction, perpendicular to the 4 September source and along the same distance contour to the 22 February source.

A comparison of source effects for the two events is shown below (Bradley 2011; Holden 2011):

4 September 2010 M7.1:

- 11km deep, 40km west of Christchurch
- Very complex multi-fault rupture
- Greendale fault rupture directed east, towards Christchurch
- Hororata fault rupture directed south-west

22 February 2011 M6.3:

- 4km deep, 8km south-east of Christchurch (Heathcote Valley)
- High rupture velocity
- Fault rupture directed up and north-west, towards the city
- Maximum vertical PGA of 2.21g in Heathcote Valley

The 22 February 2011 earthquake had a high rupture velocity aimed directly upwards at the city. It was a very energetic earthquake with very high vertical ground accelerations (Bradley 2011; Holden 2011). The high vertical ground accelerations are likely to have contributed to increased liquefaction across the city in the 22 February 2011 event.

The duration of shaking is factored into liquefaction analyses by means of the earthquake magnitude. The Christchurch earthquakes have, however, shown to be characteristically high intensity relative to this magnitude. We have reviewed the time history plots for the Papanui High School seismograph and it appears that the February event was at or close to the mean PGA for a longer duration than the September event. This could explain the difference in observed settlements.

Using geometric mean PGAs for our liquefaction induced settlement analyses for the 4 September 2010 and 22 February 2011 events, and taking the variability of PGAs over short distances as discussed above, into consideration, we were able to report predicted liquefaction induced settlements as follows:

- 4 September 2010: 10mm to 30mm for the southern half and 20mm to 60mm for the northern half of the site.
- 22 February 2011: 10mm to 40mm for the southern half of the site and 50 to 110mm for the northern half of the site.

This result gave us the confidence in our methodology to report our predicted settlements for the design scenarios, but also made us aware of the variability of PGAs over short distances. A site specific ground motion assessment is now a standard part of our liquefaction hazard assessment.

While the revised predicted settlements for the two events were in the right “ball-park” they were still a little high when compared to the observed settlements on site. Another tool geotechnical engineers can use to predict the way in which liquefaction is manifest at the surface is the Liquefaction Potential Index or LPI. The Liquefaction Potential Index or LPI (Iwasaki et al. 1982; Li et al. 2006), measures the potential for liquefaction to cause damage at the ground surface. LPI is based on the depth to liquefiable material and the thickness of liquefiable layer (Li et al. 2006).

The LPI for 4 September 2010, and 22 February 2011 events at Northlands was Low, indicating a low potential for liquefaction to cause damage at the surface. This is related to the depth and thickness of the liquefiable layers, suggesting that the non-liquefiable “crust” will mitigate the damage manifest at the surface. This could help to explain the difference between the predicted and observed settlements on site at Northlands Shopping Centre.

6. CONCLUSIONS

The Canterbury earthquake sequence has been well documented by GNS Science via a network of strong motion sensors across the region. Using results from the GNS strong motion database, we can compare the predicted and observed liquefaction induced settlements as part of a liquefaction hazard assessment. This “sanity check” is an important step in verifying the methodology used for liquefaction hazard assessment, and the results of subsequent design scenario analyses.

The 4 September 2010 and 22 February 2011 events “tested” the CBD to comparable if not slightly higher ground motions as the current SLS (September) and ULS (February) design scenarios for liquefaction hazard assessment.

The 22 February 2011 earthquake had a high rupture velocity aimed directly upwards at the city. It was a very energetic earthquake with very high vertical ground accelerations (Bradley 2011; Holden 2011). The high vertical ground accelerations are likely to have contributed to increased liquefaction across the city in the 22 February 2011 event.

The social impacts of over-conservative engineering are of particular relevance in post-earthquake Christchurch. An over engineered or unrealistic engineering solution may result in the unnecessary closure of homes and businesses.

The calculation of liquefaction induced settlement and lateral spreading is recognised as an inexact process. Sound geotechnical engineering judgment should be used when reporting the results of these analyses. There is a real risk that liquefaction analyses software can be used without understanding of the methods used and the input parameters. This type of analyses can result in over and under predicted liquefaction induced settlements, with social and economic consequences. As geotechnical engineers we have other valuable tools such as the LPI to help us interpret analysis results and use engineering judgment to make sensible recommendations.

PGAs can vary significantly over short distances as a result of variable geology and earthquake source effects.

PGAs taken from the GNS strong motion database should be used with care and sound geotechnical engineering judgment should be applied to all results. The geometric mean PGA should be used to better characterise the ground motion intensity.

To apply sound geotechnical engineering judgment a strong geotechnical background is required, with a full understanding of the geological model and the variability of input parameters such as PGA.

7. ACKNOWLEDGMENTS

Review comments from Nick Harwood and Safia Moniz are greatly appreciated.

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DESIGN AND CONSTRUCTION OF THE COOPER STREET OVERPASS

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ABSTRACT

The Cooper Street Overpass is the largest structure on the South Morang Rail Extension Project, a \$500 million project intended to improve rail services to the northern suburbs of Melbourne. This piled, integral abutment bridge allows the newly extended rail line to pass beneath the existing Cooper Street, and into the newly constructed Epping train station.

Design of any integral bridge is a technically complex and challenging process. The location of Cooper Street in a busy suburban area, with a high density of underground services and heavy traffic flows created a unique set of additional challenges which drove innovation in the design. This innovation relied on site controls which enabled the design team to deliver an efficient, value for money design, despite limited geotechnical information.

Construction of the overpass involved an extensive program of piling and earthworks. In order to minimise disturbance and deliver value for money, a combination of bottom-up and top-down construction methods were implemented. This enabled construction to progress while keeping Cooper Street open to traffic at all times.

This paper discusses the challenges faced and innovations implemented in the design and construction of this critical structure. Particular focus is given to the design and construction of the piles supporting the structure.

1. INTRODUCTION

The South Morang Rail Extension Project is located in the northern suburbs of Melbourne. The project includes a 5km duplication of the existing Epping rail line from Keon Park to Epping and a 3.5km extension from Epping to South Morang. The project was delivered under an alliance contract involving the Department of Transport, VicRoads, Metro Trains Melbourne, AECOM as the lead designer and John Holland as the lead constructor.

The extension to the rail line is constructed in the corridor of a historic steam era rail line. The original rail line was closed in the 1950s and dismantled in the 1970s, but the rail corridor was retained and it is now bounded on both sides by residential properties for the majority of its length. To minimise the noise and visual intrusion to the neighboring properties, the new rail line was constructed in a cutting averaging approximately 6m deep.

The project includes six key structures: one culvert, two creek crossings and three road-over-rail bridges. The largest of these structures is the Cooper Street Overpass; a 120m long, 21m span integral abutment bridge which allows the newly extended South Morang Rail line to pass beneath the existing Cooper Street, and into the newly constructed Epping train station.

Cooper Street is located in a congested suburban area with a number of local amenities close by. The constraints associated with the location of Cooper Street created challenges for the delivery of this structure. The challenges commenced from the ground investigation and continued through to the closing stages of construction. This paper discusses the innovations implemented to overcome the geotechnical challenges associated with delivery of this key structure.

2. GEOLOGICAL SETTING

The natural subsurface material in the area of this project belongs to the Quaternary (Pleistocene) aged Newer Volcanics Group, comprising olivine basalt (Geological Survey of Victoria, 1981). The Newer Volcanics Group was formed by lava flows from a series of volcanic eruptions.

Each lava flow varied in both chemistry and thickness. Weathering of the resulting rock mass is influenced by the chemistry, and structure of the rock. The occurrence of joints and fractures is controlled by the speed of the cooling process which is a function of the thickness of each flow. Therefore, due to the variable chemistry and thickness of the lava flows, the extent of weathering and hence the engineering properties of this geological unit are highly variable.

The subsurface profile was divided into four major units with similar engineering properties, as presented in Table 1. For identification purposes, the basalt was divided into three units, broadly characterised by strength and weathering.

Table 1: Summary of Site Geology and Adopted Engineering Parameters

Sub-Unit	Description	Thickness (m)	Engineering Properties of Soil/Rock Mass					Pile Design	
			γ (kN/m ³)	C_u (kPa)	c' (kPa)	ϕ (°)	E' (MPa)	$f_{s,ult}$ (kPa)	$f_{b,ult}$ (kPa)
Residual Clay	Dark brown, of moderate to high plasticity, moist, very stiff	0.7 to 3.0 at surface	19	100	5	26	50	-	-
B3 Basalt	Medium to high strength, moderately to extremely weathered, closely spaced joints (~100 to 150mm)	Variable	20	500	50	35	200	750	3750
B2 Basalt	High to very high strength, moderately to slightly weathered, widely spaced joints (~500mm)	Variable	20	-	100	40	1800	1500	7500
B1 Basalt	High to very high strength, slightly weathered to fresh, widely spaced joints (~1000mm)	Variable	23	-	100	40	1800	-	-

3. THE OVERPASS STRUCTURE

3.1 INTEGRAL ABUTMENT BRIDGES

Integral abutments comprise a structurally fixed joint between the piled foundations and the bridge deck. The joint includes no bearings and is intended to minimise maintenance costs over the design life of the bridge. Due to this structural arrangement, all forces and deflections in the deck are transferred directly into the piles and vice versa. The bridge deck therefore behaves as a prop at the head of the piles, assisting with resistance of lateral earth pressures and hence minimising the required embedment depth of the abutment piles.

At Cooper Street the bridge deck is at approximate pre-construction street level with minimal engineered fill providing small approach embankments. The piled abutments retain the ground through the rail line cutting and support the deck of the structure over the bridge.

A schematic cross section showing the structural form of the overpass is shown in Figure 1, and a photograph of the overpass during construction is shown in Figure 2.

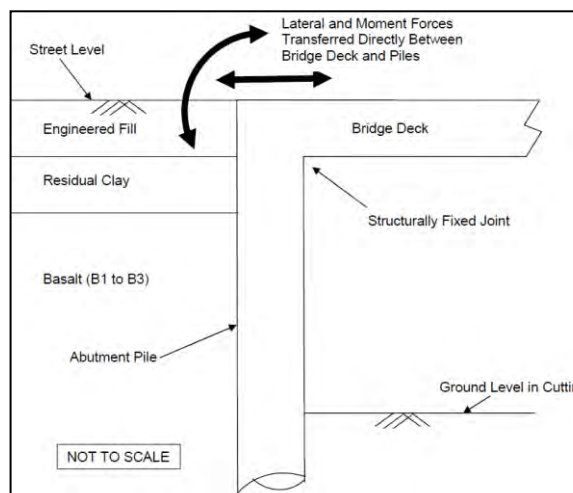


Figure 1: Integral Abutment (Conceptual)

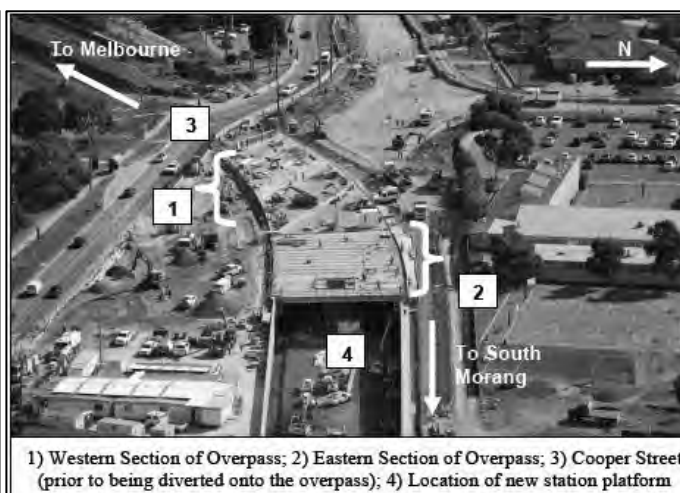


Figure 2: Copper Street Overpass During Construction

3.2 SITE SPECIFIC CONSTRAINTS AND OPPORTUNITIES

The Cooper Street Overpass includes two distinct sections: the western extent, supporting Cooper Street, and the eastern extent, supporting the new Epping Station building (see Figure 2).

Constraints and project demands were different for each section of the overpass. This provided an opportunity for the planning and construction sequencing for each section to be considered separately, and different construction methods were implemented. The key differences between the eastern and western sections are highlighted in Table 2 below.

Table 2: Western and Eastern Sections of Overpass – Key Differentiators

	Western Section	Eastern Section
Purpose	To carry Cooper Street, part of the VicRoads network, over the new rail line.	To support the new Epping Station building and associated infrastructure.
Applicable Design Standards	VicRoads Specifications and Design Guides. AustRoads: Road Design Guide Part 7 Geotechnical Investigation and Design. Australian Standards: AS5100.3-2004; AS2159-2009; AS4678 – 2002 .	Australian Standards. Primarily: AS5100.3-2004; AS2159-2009; AS4678 – 2002.
Details of Sub-Structure	82 No 750mm diameter bored and cast in place piles at 1.8m centre-to-centre spacing. Piles are 9.0 to 10.5m long, with a 2.5 to 3.0 m rock socket.	48 No 750mm diameter bored and cast in place piles at 1.8m centre-to-centre spacing. Piles are 9.0 to 10.5m long, with a 2.5 to 3.0 m rock socket.
Key Constraints and Opportunities	Construction of retaining walls to the west of the overpass was on the critical path and it was not possible to divert Cooper Street around the excavation works. To make space for work on the retaining walls to progress, and avoid closure of/disruption on Cooper Street it was necessary to re-align the road onto the new bridge as early as possible.	No existing infrastructure was located within the footprint of the eastern section. Therefore the logistical constraints were less onerous, providing an opportunity to utilise a conventional construction method.
Construction Method	Top Down: Piled abutments are installed, deck is constructed (propping the head of the piles), and soil and rock are then ‘mined’ out from beneath the bridge deck. See Figure 3.	Bottom Up: Piled abutments are installed; earth is excavated from between the abutments and the bridge deck is installed upon completion of the excavation. See Figure 3.
Advantages of Construction Method	As the excavation progresses the bridge deck behaves as a prop at the head of the piles. No additional excavation support is therefore required.	Excavation can be completed without any overhead constraints.
Limitations of Construction Method	Excavation progresses within a confined space, significantly slowing the pace of the excavation.	Without the deck of the bridge to support the excavation works temporary support was required. Passive ground anchors were used for this purpose.

The use of different construction techniques for each section of the structure yielded a number of benefits for the project. The balance between programme and cost was optimised and the needs of the local community were met by completing the structure without any road closures.

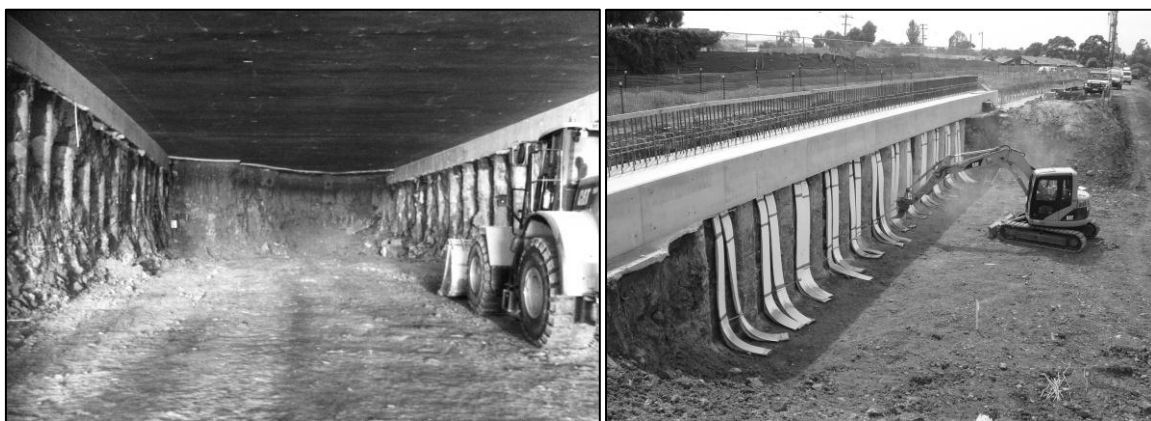


Figure 3: Cooper Street Overpass during excavation works: (left) Western Section; (right) Eastern Section

4. CHALLENGES OF THE PILE DESIGN PROCESS

4.1 MINIMUM GROUND INVESTIGATION REQUIREMENTS

Australian Standard AS5100.3 governs the minimum requirement for ground investigation for the whole structure. Clause 6.2 of AS5100.3 requires a minimum of ‘One [borehole] at each end [of the structure], and for intermediate locations, one at not more than 30m intervals’. Investigation points are required to extend through any strata that may influence the strength, stability or serviceability of the foundations.

The site investigation effort at the overpass was heavily constrained by access restrictions, existing underground services, overhead services and the construction programme. Within the wider Cooper Street area 11 boreholes, 5 auger holes and 2 large test pits had been undertaken. However, only three of these boreholes, each extending to 20m bgl could be completed within the footprint of the bridge. This did not satisfy the requirements of AS5100.3 and it was therefore necessary to find an alternative approach to satisfying the intent of the design standard.

4.2 GEOTECHNICAL STRENGTH REDUCTION FACTOR

In accordance with AS5100.3, the design ultimate geotechnical strength ($R_{du,g}$) is required to be reduced by a geotechnical strength reduction factor (ϕ_g) to define the design geotechnical strength ($R_{d,g}$).

The geotechnical strength reduction factor is defined (AS2159, Cl 4.3.2) as a function of the average risk rating category for the structure and the extent of pile load testing carried out. Average risk rating categories range from 'very low' for a risk rating of less than 1.5 to 'very high' for a risk rating of 4.5 or greater. The risk rating value is calculated taking into account the site geology, extent of ground investigation, experience of the designer with the site geology and type of structure, and the construction controls that will be implemented.

Based on the variability of the geology and the low density of investigation points within the footprint of the structure, an average risk rating of approximately 4.0, or high to very high, would have applied. For an average risk rating of 2.5 or greater, pile load testing (AS2159, Cl 8.2.4) and use of a low geotechnical strength reduction factor (approximately 0.4) is required.

The South Morang Rail Extension Project geotechnical design team have extensive experience with piled abutments embedded into the Newer Volcanics. Additionally, the Newer Volcanics has been extensively investigated across Melbourne. Its engineering properties and associated achievable pile load capacities are well understood. It was therefore considered appropriate to challenge the 'high to very high' risk rating and take measures to justify adopting a more favourable geotechnical strength reduction factor.

4.3 PILE LOAD TESTING REQUIREMENTS

The two sections of the overpass are each governed by different design codes and standards. The eastern section is subject to the requirements of Australian Standards only, whereas the western section is subject to VicRoads design guides and standards, and Australian Standards.

Both VicRoads and Australian Standards encourage static pile load testing (for axial and lateral loads) but do not require it. Australian Standards also encourage, but do not require dynamic pile load testing (for axial loads only). However, VicRoads Standard Specification (section 606.07) requires dynamic pile load testing to be carried out on 1 in every 30 piles constructed. Dynamic pile load testing was therefore required on the western section of the structure.

There were a number of difficulties associated with carrying out pile load testing at the Cooper Street site:

- To test the axial load capacity of the pile sockets, it would have been necessary to isolate the retained soil and rock from the pile, potentially affecting the lateral load capacity of production piles.
- Pile load testing is not possible with top-down construction methods.
- Due to the high strength of the rock, it is likely that the structural capacity of the pile would be exceeded before proving the geotechnical capacity, thus damaging the piles.
- Due to the anticipated very high axial load capacity of the piles, testing would not have been possible with conventional pile load testing equipment.

As an alternative to testing of production piles, pile load testing could have been carried out on sacrificial piles. However, due to the space and access constraints at the Cooper Street site, this was not considered a feasible solution.

The piles at the Cooper Street Overpass were designed to carry both axial and lateral loads and required socket length was governed by lateral load capacity. The provided socket length was therefore larger than required for achieving the required axial capacity. We therefore had confidence, without pile load testing, that the required axial load carrying capacity of the piles would be achieved. An alternative approach to quality control that would satisfy the intent of the VicRoads Specification was therefore sought.

4.4 SATISFYING THE INTENT OF THE DESIGN STANDARDS AND SPECIFICATIONS

Many of the design challenges could be resolved with additional information regarding the nature and variability of the sub-surface material. The proposed construction method of bored piles, followed by a large scale excavation, provided an opportunity to collect information relating to ground conditions during the construction phase. Full time supervision by a geotechnical engineer was therefore provided. All pile excavations were logged and inspected, a CCTV survey was carried out in selected pile bores, and the exposed faces of the cutting were inspected, mapped and photographed as the excavation progressed. The data collected formed the elements of a very detailed geological model.

The measures taken to satisfy the intent of the relevant design standards and specifications are summarised in Table 3:

Table 3: Summary of Design Challenges and Solutions

Challenge	Solution	Outcome
Minimum ground investigation requirements (AS5100.3, Clause 6.2) could not be satisfied.	Collection of geotechnical data during construction, including full-time supervision by a geotechnical engineer. Upon encountering adverse ground conditions, supervising geotechnical engineer adjusted the pile length accordingly.	Collecting geotechnical data during the construction phase, and where necessary, adjustments to the length of the piles, addressed the intent of the design code. The result was a detailed geological model.
Limited ground investigation resulted in an average risk rating in the 'high to very high' category (approx. 4.0). This would have required a very low geotechnical strength reduction factor, approximately 0.4, to be adopted.	Full time site supervision by a geotechnical engineer to log the cuttings from the pile bores provided a high degree of confidence regarding the founding conditions in each pile. Supervising engineer, able to adjust the design pile lengths in response to encountering adverse ground conditions..	The average risk rating was reduced to 2.4 (low) on the basis of the construction controls and quantity of geotechnical data. A geotechnical strength reduction factor of 0.56 was adopted. The lower geotechnical strength reduction factor resulted in shorter pile lengths.
Pile load testing, required by VicRoads standard specification, Section 606.7 and encouraged in AS2159, Clause 8.2.4, could not be carried out.	Collection of geotechnical data during construction, including full-time supervision by a geotechnical engineer. It was demonstrated that the critical case for design of pile socket length is lateral loading. Therefore, in terms of axial loading the provided socket length is conservative.	The intent of the VicRoads specification was addressed by demonstrating that: -The geology the piles were embedded into was compatible with the design assumptions. -The socket length provided was conservative in terms of the axial load resistance required.

5. ADDITIONAL IMPLICATIONS OF THE APPROACH TAKEN

5.1 RISK AND BENEFITS TO COST AND PROGRAMME

The requirements in AS5100.3 and AS2159 for minimum ground investigation and for defining a geotechnical strength reduction factor address two key areas of risk. They aim to provide confidence that sufficient geotechnical capacity is provided by the resulting structure and they minimise the risk of encountering unforeseen ground conditions during construction.

The site inspections and controls implemented provided the information necessary to improve confidence in the adequacy of the geotechnical design. However these controls could not address the risk associated with unforeseen ground conditions.

Encountering unforeseen and adverse ground conditions posed a significant risk to the cost and programme for construction of the Cooper Street Overpass, and for the wider project. The management team for the project decided that the immediate benefits to cost and programme from progressing without additional ground investigation data outweighed the risks incurred.

In some isolated locations, ground conditions encountered within the pile socket were significantly worse than those accounted for by the design. In such instances, the site-based geotechnical engineer was able to modify the pile design on site. This resulted in longer piles and in some cases the steel reinforcement cages were required to be extended. However, given that adverse ground conditions were encountered only in isolated locations, affecting a small percentage of piles constructed, the overall impact to cost and programme was minimal.

5.2 MODIFIED CONSTRUCTION SEQUENCE

The construction sequence for the overpass specified that, where bottom up construction was implemented, after each lift of earthworks, a shotcrete facing must be applied before progressing with the excavation. This measure was designed to eliminate the risk of loose and falling blocks from the exposed face of the excavation, between adjacent piles, while personnel were working in the area. However, several days is required to hang mesh, apply shotcrete and allow shotcrete to cure. This measure therefore slowed the pace of the excavation, and had implications for the overall programme and budget for the project. The construction team therefore requested that this requirement be relaxed.

As part of the geotechnical data collection, after each earthworks lift, the excavation was required to be inspected by site based geotechnical engineer. To address the request from the construction team it was agreed that this inspection would also be used as a safety inspection and would replace the requirement to install shotcrete after each lift. If any

loose blocks, potential wedges or blocks at risk of toppling risks were identified, the engineer would instruct the construction team to remove all hazards prior to continuing the excavation.

This approach required a break in excavation works of approximately just 30 minutes between lifts of earthworks, compared with several days for the original construction sequence. The construction programme was therefore accelerated considerably.

5.3 OPERATION OF CRANES

To account for variability in surface levels, material strengths and limited construction traffic at the top of the cutting, the geotechnical design team included a 20kPa surcharge in the analysis of all retaining structures, including the abutments for the Cooper Street Overpass.

Throughout construction requests were raised to operate cranes behind the abutments of the new overpass structure to facilitate the delivery of construction materials to Cooper Street. The loading from these cranes was substantially larger than the 20kPa allowed for.

The detail of information regarding the ground conditions behind the abutments, collected during the site supervision and excavation inspections, enabled us to re-analyse the overpass based on real in-situ ground conditions, rather than those assumed for the design. The geotechnical design team worked closely with the structural engineers to re-analyse the structure and were able to approve operation of cranes behind the overpass abutments, subject to a number of site controls and restrictions.

This accelerated delivery of construction materials to the site of the new station and also avoided introducing additional delivery vehicles to the already congested site, thus improving safety.

6. CONCLUSION

A number of challenges were presented to both the design and construction of the Cooper Street Overpass. The challenges of construction were largely confronted in the planning stage. An innovative approach, using both top-down and bottom-up construction methods for the one structure presented a 'best for project' solution, which enabled the overpass to be constructed within the project time frames. It also avoided the requirement for road closures though this congested area, thus minimising disruption to the local community.

Access, space and construction program constraints limited the scope of the ground investigation and prevented pile load testing from being undertaken at the Cooper Street site. This presented major challenges for meeting the requirements of the relevant design standards. The geotechnical design team implemented a flexible approach, relying on site controls, to ensure that the piles were embedded into the geology assumed by the design. This served to address the intent of the relevant design standards and facilitated an efficient design, delivering value for the client. Use of this approach did, however, introduce additional risk to the cost and programme for construction of this project.

The site supervision required to satisfy the intent of design codes resulted in several additional benefits for the project. The geotechnical design team were able to identify and address adverse ground conditions, modify the construction sequence and approve the operation of cranes in the area. This facilitated acceleration of the construction programme and contributed to improving safety of the construction site, and of the resulting structure.

Construction of the Cooper Street Overpass was completed in October 2011, and the new Epping Station was opened in November 2011. The extension of the rail line from Epping to South Morang was opened to the public in April 2012.

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2D HYDROGEOLOGICAL MODELING OF THE WESTERN RING ROUTE – WATERVIEW CONNECTION TUNNELS, AUCKLAND, NEW ZEALAND

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ABSTRACT

The Waterview Connection will complete a strategic motorway route bypassing the central suburbs of Auckland City, New Zealand. It will comprise 2.1 km of sealed driven tunnels, 500 m of drained cut-and-cover tunnel, and portals.

The alignment will run beneath an urban area and through a complex geological and hydrogeological environment. The effects on the environment needed to be assessed and potentially mitigated in order to gain a Resource Consent for construction and operation.

A combination of 2D and 3D numerical groundwater modelling was undertaken to assess the potential effects of tunnel construction and operation on the groundwater system. Modelling was also used to guide tunnel design and inform groundwater seepage volumes and water pressures.

The 2D modelling was used to assess the magnitude and extent of changes in groundwater level and the resulting potential for ground settlement, buoyancy effects and contaminant migration.

This paper describes the 2D groundwater modelling undertaken as part of the resource consenting stage of this project. The advantages and limitations of this method are discussed along with the extent of effects predicted, and proposals for mitigation of effects should these exceed those identified.

1. INTRODUCTION

The Western Ring Route (WRR) comprises a series of motorway corridors that once completed will link Manukau, Auckland, Waitakere and the North Shore while bypassing Auckland's CBD and the Auckland Harbour Bridge. It will provide an alternative to the SH1 route north for traffic passing through the region.

The Waterview Connection is the key project to complete the WRR and will comprise a section of drained cut and cover tunnels at the northern portal, two sealed driven tunnels, and a drained cut at the southern portal. The project alignment is shown in Figure 1. This will be the largest road tunnel project ever undertaken in New Zealand.

The process for gaining resource consent for new developments in New Zealand requires an assessment of potential effects from construction and operation of the proposed development, and where necessary, procedures for mitigation of these effects. Groundwater modelling was carried out to understand the existing environment and the effects of the project. Procedures for mitigation of potential effects and recommendations for tunnel design were based on modelling results.

2. GEOLOGICAL AND HYDROGEOLOGICAL SETTING

The project area is located in the Waterview and Mt Albert suburbs of Auckland. The tunnel alignment runs roughly parallel with the valley occupied by Oakley Creek between Mt Albert Volcano to the east and a bedrock ridge to the west (Figure 1).

A significant number of geotechnical and hydrogeological investigations have been undertaken to characterise the site. These include 305 boreholes, 4 geophysical surveys, installation of 231 piezometers, 7 pumping tests and 171 in-situ hydraulic conductivity tests.

The bedrock in this area comprises the East Coast Bays Formation (ECBF), Tertiary aged interbedded sandstones and siltstones deposited by turbidity currents. Lenses of coarse grained volcanoclastic Parnell Grit (PG) occur within the ECBF. Residual ECBF and PG soils of variable thickness overlie the ECBF rocks.

Mt Albert Volcano is the remains of a scoria cone which erupted over 30,000 years ago. Extensive lava flows erupted from the base of the scoria cone towards the north, west and south influencing the flow of Oakley Creek which now runs roughly along the edge of the flows. Two areas of landfill occupy historic basalt quarries adjacent to the creek.

Tauranga Group (TGA) Holocene aged alluvium is found beneath the lava flows, and within the Oakley Creek valley.

Seven hydrogeological units were identified based on geological description and hydrogeological parameters derived from field testing. The units and their measured range in hydraulic conductivity (K) are outlined in Table 1 below.

Hydrogeological Unit	Hydraulic Conductivity – K - (m/s)
Fill: variable in nature and thickness	Not tested
Auckland Volcanic Field (AVF): variably jointed and scoriaceous lava flows	10^{-4} to 10^{-6} mean: 1×10^{-5}
Tauranga Group Alluvium (TGA): soft to stiff, <i>compressible</i> clays, silts and sands	10^{-9} to 10^{-6} mean: 1×10^{-7}
Weathered East Coast Bays Formation (WECBF): bedded silts, sands and clays	10^{-8} to 10^{-7} mean: 2×10^{-7}
Weathered Parnell Grit (WPG): soft to firm <i>sensitive, compressible</i> clays	10^{-8} to 10^{-7} mean: 1×10^{-7}
East Coast Bays Formation (ECBF): interbedded sandstone and siltstone, variably fractured	10^{-9} to 10^{-5} mean: 2×10^{-7}
Parnell Grit (PG): coarse volcanoclastic sandstone	10^{-7} to 10^{-5} mean: 2×10^{-6}

Table 1: Hydrogeological Units (modified from France et al, 2011)

Regular water level monitoring undertaken from 2001 indicates a north to north-westerly gradient on the regional water table towards the coast. The gradient within the ECBF and PG rock is around 3 % with seasonal variation of up to ± 1 m. Water levels in the basalt and TGA are on average 2 m higher than in the underlying rock and have an average seasonal variation of ± 1.5 m. Pumping test results suggest a limited connection between the aquifers within the rock and the overlying basalt and TGA. Artesian water levels are recorded in some piezometers originating from within the Parnell Grit.

Limited flow gauging data from Oakley Creek suggests the flow is highly variable and that the component of flow contributed from groundwater within the project area is small.

3. PROJECT CONSTRUCTION

The Waterview Connection comprises three distinct construction zones. The construction methodology adopted for recourse consenting and the groundwater model was as follows:

The northern cut and cover tunnel and portals are 500 m long, to be constructed using diaphragm walls and excavated from the top down in stages with internal propping installed. The excavation is up to 24 m deep and 40 m wide. Both a permanently drained and a partially sealed tunnel option were modelled.

The driven tunnels comprise 2.1 km of two 12 m high and 15 m wide tunnels. The deepest point will be 65 m below ground level. Cross passages will link the tunnels at 150 m intervals. Construction using a road-header and sequential excavation is proposed, commencing from the southern portal. Shotcrete and temporary support is to be applied in sections immediately following the excavation face, and permanent water tight concrete lining installed as the tunnels progress. The driven tunnels will be mainly within the ECBF rock, with a 250 m section through Parnell Grit.

The southern portal is a 580 m long section constructed through basalt, WECBF and ECBF rock. The basalt will be blasted and rock bolted; whilst soils and ECBF rock will be retained by secant piles. The excavation is up to 22 m deep and the floor slab will be permanently drained, although checks were made in the 2D model on likely effects from a sealed cut.

4. POTENTIAL EFFECTS AND DESIGN CONSIDERATIONS

Groundwater modeling of the construction and operation of the tunnels indicated potential for the following effects:

- Settlement of compressible soils and subsequent damage to structures resulting from groundwater drawdown;
- Changes to flow within Oakley Creek from groundwater drawdown;

- Leakage of contaminants into the tunnel or rock aquifers from groundwater drawdown; and
- Damming effects adjacent to the sealed tunnels.

The following design considerations were also investigated:

- Potential groundwater inflows during construction and long term in unsealed sections of tunnel; and
- Potential groundwater pressures on sealed sections of tunnel.

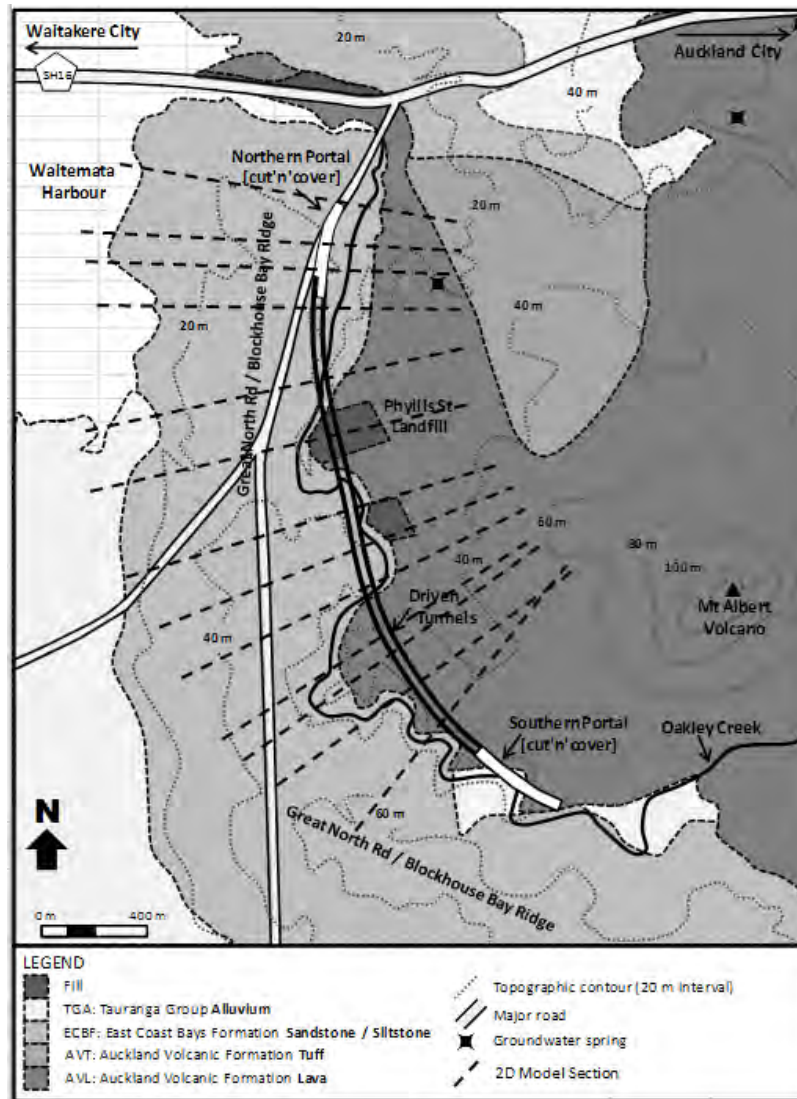


Figure 1: Map of the project area showing key hydrogeological features and 2D modelling sections (modified from France et al, 2011)

5. NUMERICAL MODELLING

Both 2D and 3D numerical groundwater modelling were undertaken in this assessment.

The 3D model developed using Visual MODFLOW Pro (Schlumberger) considered overall groundwater flow trends, broad scale effects and 3D flow (France, et al, 2011).

2D groundwater seepage modelling, in GeoStudio SEEP/W was undertaken to examine in more detail the effects on groundwater flow around the tunnel structure and walls. Because of the finer scale of the 2D models, and the software's ability to model unsaturated flow (such as that which occurs when an excavation is dewatered) the 2D modelling was used to consider the likely effects in immediate proximity to the tunnel (100 m either side).

Thirteen representative 2D cross sections through the tunnels were considered, located as shown on Figure 1. An example, through chainage 3400, is shown in Figure 2.

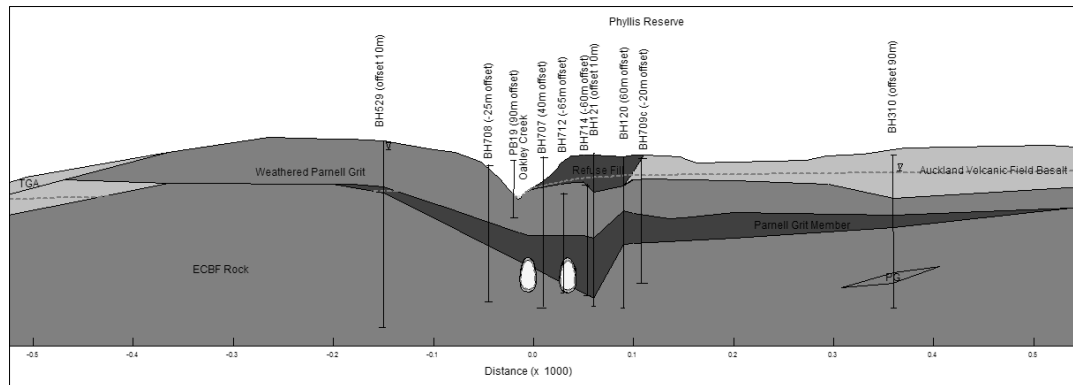


Figure 2: 2D Groundwater Section through CH3400 (2x vertical exaggeration, refer to Figure 1 for geological legend).

The sections were calibrated to replicate measured groundwater level data. The regional water table within the ECBF was used to calibrate the sections within the area of the driven tunnels. Pumping tests show limited connection between the deeper aquifer and the highest perched water tables, and as the tunnel is driven through the ECBF, the drawdown effects are unlikely to fully dewater the perched water tables at the surface.

Higher perched water tables were used to calibrate the cross sections through the northern cut and cover section and southern portal in order to predict the maximum water pressures on the tunnel walls and floor slabs and to determine likelihood of damming adjacent to the sealed structures.

A thirteen step tunnel construction sequence was modelled for the driven tunnels incorporating the initial excavation, shotcreting and permanent lining installation stages as well as the long term steady state condition. It has been assumed that for each section of tunnel the total construction sequence will take around 9 months to complete. The shotcrete and tunnel lining were modelled as regions two orders of magnitude ($K = 3 \times 10^{-9}$ m/s), and four orders of magnitude ($K = 3 \times 10^{-11}$ m/s) lower hydraulic conductivity than the ECBF rock respectively.

The construction sequence modelled for the cut-and cover tunnel and portals varied between sections.

The largest drawdown measured at the base of compressible soils (TGA and WPG) was utilised in consolidation settlement calculations (Figure 3). Settlement was calculated using the coefficient of volume compressibility (m_v) method (with m_v values for each soil type derived from laboratory consolidation testing).

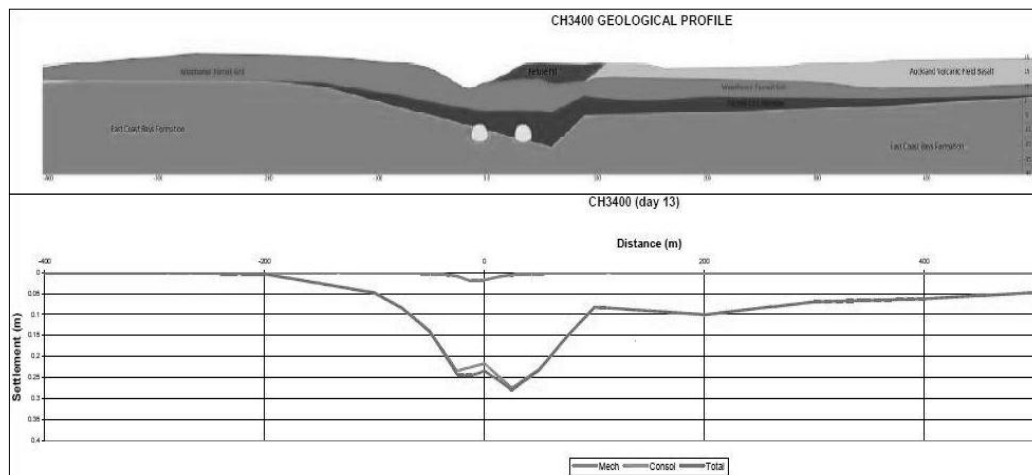


Figure 3: Estimated consolidation settlement due to groundwater drawdown and mechanical settlement from tunnel construction at CH3400

6. PREDICTED EFFECTS

As the northern and southern portals will be partially or fully drained, the greatest changes in groundwater level in these areas will occur in the long term when steady state conditions have been achieved.

For the northern portals and approaches, modelling suggested that around 60% of long term drawdown and settlement would occur during the construction period, and that the uplift pressures on a then sealed cut and cover tunnel would

require significant anchoring to counteract. Therefore as the corresponding settlement effects for a drained floor in this area were considered to be minor, a drained floor was developed as the preferred option.

At the southern portals and approaches permanent drainage has been proposed to relieve porewater pressures on the retaining walls. Drawdown of 2 m to 13 m is expected in the compressible soils immediately adjacent to retaining walls. The 2D model predicts some drawdown up to 500 m distance from the tunnels, but 3D modelling predicts measureable drawdown being typically limited in extent to less than 50 m from the walls. The 2D models are influenced by lack of up gradient recharge from the dominant groundwater flow direction and hence the 3D predictions of drawdown extent were considered more accurate. The predicted drawdown is likely to cause minor settlement effects.

High inflows (10 to 75 m³/day/metre length of alignment) were predicted during excavation through the basalt, although the peak rapidly reduces to an average of 2.5 m³/d/m. A proposed grout curtain through the basalt was modelled as a region of reduced hydraulic conductivity. Modelling suggests that the curtain may reduce initial inflows if large fractures are successfully cut off.

As the driven tunnel will be fully sealed in the long term, the largest changes in groundwater level result from the short-term dewatering during construction. Short term drawdown within the compressible soils of around 8 m immediately adjacent to the tunnels is predicted when tunnelling in ECBF, with up to 30 m in areas of PG. The drawdown rapidly reduces away from the tunnels with measureable drawdown typically extending less than 100 m. Long term drawdown of 0.3 m is predicted.

Within the PG a zone of higher hydraulic conductivity was detected during investigations, as the extent of this zone was not well defined the 2D modelling assumed a relatively high hydraulic conductivity throughout. The initial inflows during tunnel excavation through the high permeability PG are much higher than within the ECBF (up to 120 m³/day/metre length of tunnel, compared with 5 m³/d/m within ECBF). Modelling suggests that pre-grouting through this zone may significantly reduce inflow rates (2 m³/d/m of tunnel).

Monitoring of water levels in boreholes screened within the refuse fill suggests that there is almost no water residing within the fill in this area, the fill is also underlain by several metres of low permeability soils. The potential to spread contaminants through the underlying low permeability soils in response to drawdown was therefore predicted to be almost nil.

7. MITIGATION AND DESIGN

The drawdowns predicted in the 2D modelling translated to consolidation settlements that were significant in localised areas (Figure 4), but as the gradient of drawdown was low outside of the designation for the tunnels, the differential settlements were also low and effects on structures were predicted to be minor (Burr and Alexander, 2011). Prior to commencement of construction, procedures must be in place to monitor drawdowns and settlements to ensure effects don't exceed those predicted in the modelling. Groundwater and Settlement Management Plans have been developed to provide monitoring, alert levels and mitigation recommendations.

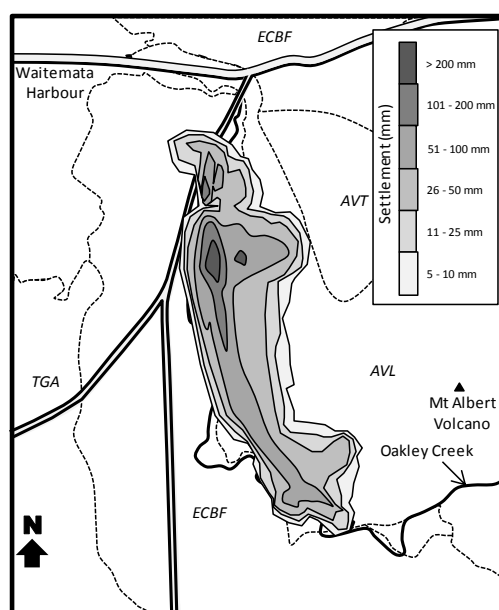


Figure 4: Predicted maximum settlement (total) contours (after Burr and Alexander, 2011).

8. LIMITATIONS

As the dominant groundwater flow in this area is parallel to the proposed tunnel, it runs perpendicular to the 2D cross sections. Attempts to account for this flow through addition of flux at discrete and diffuse points were unsatisfactory. As a result, the majority of recharge provided into the natural system was not adequately accounted for in the 2D model. The estimates of drawdown and damming were therefore considered to be conservative.

In order to reduce the chance of edge effects influencing drawdown measurements the 2D models were extended some 500 m from the tunnels. The model calibration was accurate within 200 m of the tunnel, but at greater distances the variance increased. In addition the lack of recharge to the model lead, in some instances, to the extent of drawdown exceeding that predicted in the 3D modelling. Due to this artefact of the 2D modelling the drawdown predictions at more than 100 m from the tunnels were based on a combination of the 3D and 2D models.

Despite relatively conservative drawdown effects being input into the settlement model, the resulting settlement effects were generally considered to be minor in this instance and hence the conservative estimates were effective for the purposes of resource consenting, and for allowing some flexibility in the detailed design and construction. It is evident however that consideration of both 2D and 3D models is essential when assessing potential environmental effects in complex scenarios.

9. CONCLUSIONS

A combination of 2D and 3D numerical groundwater modeling was used to predict the effects of construction and ongoing operation of both sealed and drained tunnels excavated through a combination of basalt, low permeability soils and sandstone/siltstone.

The 2D models were used to measure drawdown in close proximity to the tunnels in order to assess damming effects and porewater pressures exerted on walls at sealed sections of tunnel, as well as estimated drawdowns leading to potential settlement.

The models predict minor groundwater related environmental effects induced by the construction and operation of the project, which can be monitored and, if necessary, mitigated.

The drawdown predictions and effects on Oakley Creek are considered conservative due to the orientation of 2D models perpendicular to regional groundwater flow. As a result 2D modeling alone was not sufficient to determine the regional effects of the project. Despite this the finer scale achievable in the 2D compared to the 3D models, and the software's ability to model unsaturated flow make it a useful tool to simulate conditions in close proximity to the excavation face.

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EFFECT OF WATER CONTENT ON THE LATERAL CAPACITY OF PILES SOCKETED INTO MELBOURNE MUDSTONE

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ABSTRACT

Many factors such as mineral composition, depositional history, and weathering condition govern the strength and deformation behaviour of rocks. For laterally-loaded rock-socketed piles, these factors have pronounced effects on the lateral capacity of piles. Melbourne mudstone, which forms the bedrock in the Melbourne, Australia, is known for its intact strength and deformation behaviour to be related to its water content. To the authors' knowledge, there have not been sufficient studies carried out to investigate the effect water content of mudstone on the lateral capacity of piles. In this paper, extensive numerical modelling was undertaken for laterally loaded piles socketed into mudstone rock using a three-dimensional discrete element package, 3DEC. The numerical model was calibrated based on a full-scale lateral pile load test conducted in mudstone. Different strength and deformation behaviour of mudstone simulated by changing its water contents were carried out and the corresponding pile head load-deflection behaviours were obtained and analysed. Subsequently, a recently-developed analytical method for laterally-loaded rock-socketed piles was employed to examine its reliability in predicting the load-deflection behaviour of piles socketed into Melbourne mudstone. It was found that water content of mudstone, signifying different strength and deformation behaviour, has significant impact on the lateral capacity of piles and considerable inaccuracy was obtained based on using the recently-developed analytical method.

1. INTRODUCTION

Civil engineering infrastructures such as high-rise buildings, bridges, retention walls and transmission towers are designed to sustain high lateral forces generated by wind, waves and earthquakes. For laterally-loaded piles founded in soils, a number of design methods have been established and compared with the results of full-scale field tests. However, the same cannot be said for laterally-loaded rock-socketed piles. Only a few notable analytical methods (Reese, 1997 and Yang, 2006) have been developed for a limited range of rock types and their application to other rock types may not be appropriate (Chong, 2011). The cause of the significant discrepancies can be due to substantial difference in strength and deformation behaviour of rocks associated with variations in mineral composition, depositional history, weathering grade and presence of secondary structures such as discontinuities and fractures in different rocks. For the local Melbourne mudstone, limited studies (Francis 2003) have been carried out to date to ascertain the lateral capacity of piles socketed into the rock. Francis (2003) investigated the effect of joints on the lateral capacity of piles socketed into synthetic mudstone in laboratory setting. While the work by Francis (2003) gave an important insight into the lateral performance of piles socketed into mudstone, several shortcomings of the method of investigation were identified such as replication of insitu behaviour (ie stresses, joint roughness etc), scale effect (rock mass) and intact rock strength.

For the local Melbourne mudstone, two aspects which govern strength and deformation behaviour of the rock are the intact rock strength and the secondary structures. This paper investigates the first aspect only, which is the effect of intact rock strength and deformation behaviour on lateral capacity of piles socketed into mudstone, on the basis that Williams (1980) found through his extensive insitu pressuremeter test that a highly fractured mudstone is able to retain approximately 80% of its intact rock strength. Generally, the intact strength and deformation properties of the mudstone can be related to its saturated water content and a reliable set of correlations have been developed to estimate the strength and deformation properties (Chiu 1980). To the authors' knowledge, there have not been sufficient studies conducted to address this aspect, and therefore it is important to carry out a thorough parametric investigation to examine the effect of intact rock strength, simulated by using different water contents, on the lateral behaviour of piles socketed into Melbourne mudstone. In addition, it is imperative to investigate the reliability of existing analytical method (Yang, 2006), based on subgrade reaction method (p - y), for rock in predicting the behaviour of laterally loaded piles socketed into Melbourne mudstone.

In this study, three-dimensional distinct element code, 3DEC, was employed for the numerical modelling of laterally loaded piles socketed into Melbourne mudstone. A field-calibrated model established by Chong et al. (2011) and Chong (2011) was employed to investigate the effect of water content, simulating different strengths of mudstone, on the p - y

curves and its corresponding lateral capacity of piles socketed into Melbourne mudstone. Subsequently, a recently developed analytical method (Yang, 2006) for laterally-loaded rock-socketed piles was utilised to examine its reliability in calculating the load-deflection behaviour of piles socketed into intact Melbourne mudstone.

2. NUMERICAL MODELLING

Williams (1980) and Chiu (1981) performed extensive laboratory tests on Melbourne mudstone rocks and developed a set of correlations between various mudstone properties with water content. The water content, which indicates the intact strength of the mudstone rock, is considered the primary factor that controls the strength and deformation behaviour of the mudstone. Hence, for the case of laterally-loaded piles socketed into mudstone, it is vital to firstly study the effect of this factor on the p - y curve and its corresponding pile-head load-deflection behaviour.

In this study, the numerical modelling was carried out using a three-dimensional discrete element program (3DEC) (Itasca, 2008). The model was based on field test conducted for laterally-loaded piles socketed into Melbourne mudstone. The benchmark model (Chong et al., 2011 and Chong, 2011) has a pile diameter of 0.3m with a socket length of 0.6m. A typical layout of the numerical model is shown in Figure 1. For more detailed explanation about the calibration of the numerical model, refer to given references.

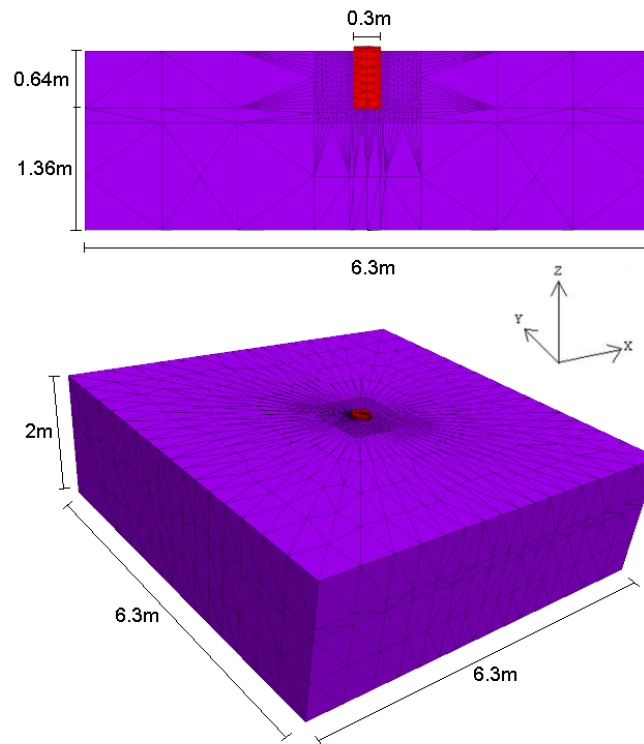


Figure 1: 3DEC modelling of full-scale pile load test (Chong, 2011)

3. EFFECT OF DIFFERENT WATER CONTENTS OF MELBOURNE MUDSTONE

In this study, the field-calibrated benchmark model established by Chong et al (2011) and Chong (2011) was employed for the investigation of the effect of intact rock strength, simulated by changing the water content of the mudstone, on pile-head load-deflection behaviour of laterally-loaded piles socketed into mudstone rock. Four different water contents (5%, 10%, 15% and 20%) of mudstone were simulated where 5% and 20% indicate fresh (high strength) and extremely weathered (low strength) mudstone conditions, respectively. The input properties for the numerical modelling were estimated based on the empirical relationships established by Chiu (1981). Table 1 presents the mudstone properties for the four water contents investigated.

A comparison of average p - y curves for different water contents as shown in Figure 2a depicts a significant variation in p - y responses of the four water contents simulated in the numerical modelling. A mudstone with lower water content provides higher rock resistance, which results in stiffer p - y behaviour of the rock mass compared to a mudstone with high water content. Therefore, it is not surprising to observe a substantial increase of stiffness in the p - y response for

5% water content compared to 20% for both the elastic and non-linear part of the curves. For further detailed comparisons, the initial sub-grade reaction modulus, K_i , and the ultimate rock resistances, p_u , for the four water contents were calculated based on the numerical results as shown in Figures 2c and d.

Figure 2c indicates that the K_i values vary considerably for different water contents of mudstone. A reduction in the initial sub-grade modulus of more than 50% can be observed when the water content of mudstone increases from 5% to 10%. For a low strength mudstone simulated by water content of 20%, the K_i decreases by more than 90% compared to a high strength mudstone simulated by water content of 5%. A similar marked variation can be observed for the ultimate rock resistance as shown in Figure 2d. By comparing the p_u values for water contents of 5% and 10%, a reduction of more than 50% in ultimate rock resistance can be observed. A highly weathered mudstone retains only 15% of the strength of fresh mudstone rock. This is consistent with rock strengths corresponding to different weathering grades where the UCS of fresh mudstone (5% water content) is 5.7MPa while for highly weathered mudstone (20% water content) it is 0.65MPa, which is a reduction of more than 90% (Chiu, 1981).

Table 1 Mudstone properties for different water contents

Water Content (%)	Rock Modulus (MPa)	Cohesion, c' (kPa)	Friction Angle, ϕ'	Tensile Strength (kPa)
5	1320	1430	41	890
10	540	710	39	350
15	220	360	34	140
20	90	180	30	55

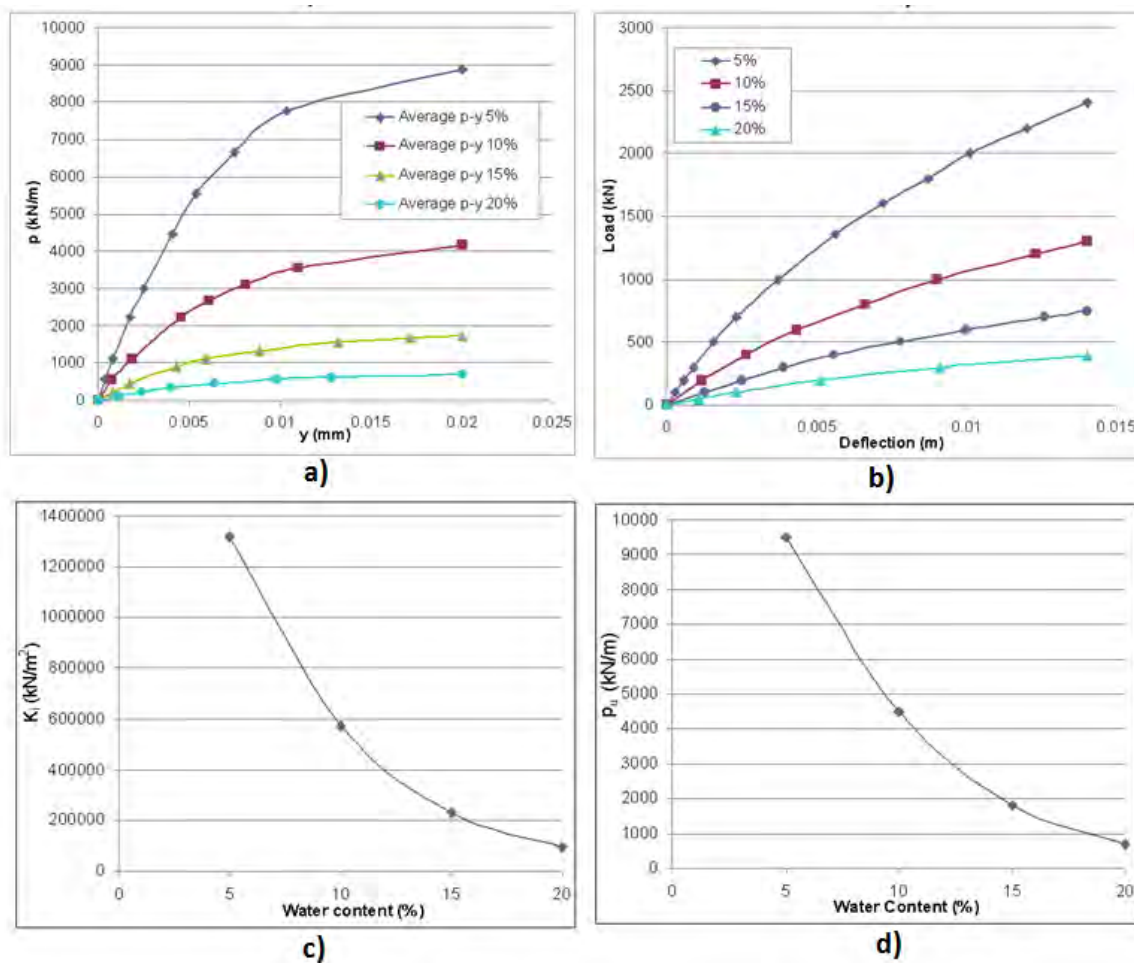


Figure 2 a) p - y curves, b) Pile-head load-deflection behaviours, c) Initial subgrade modulus, K_i , and d) Ultimate rock resistance, p_u , for different water contents.

The effect of water content on the corresponding pile-head load-deflection responses was subsequently examined. The variation in pile-head load-deflection responses for different water contents is shown in Figure 2b. A stronger mudstone rock simulated by a lower water content of 5% demonstrates higher load-carrying capacity compared to a higher water content of 20%, simulating weaker rock condition. The elastic behaviour of the pile, which is crucial for designing piles under serviceability conditions in practical engineering design, reduces significantly when the water content changes from 5% to 20%, as shown in Figure 2b. The maximum load to be sustained within the elastic region for 5% water content is 600kN, compared to 100kN for water content of 20%.

The results obtained clearly demonstrated that water content has a significant effect on the p - y behaviour of laterally-loaded piles socketed into mudstone. Currently, no analytical method is available for determining p - y curves for the local mudstone rock. Therefore, a recently-developed p - y method by Yang (2006) was employed to firstly examine its reliability to analyse the lateral behaviour of piles socketed into mudstone. Generally, the p - y method by Yang (2006) is based on hyperbolic equation (Equation 1), where K_i is the initial subgrade reaction modulus and p_u is the ultimate rock resistance. The p - y curve computed will be used in conjunction with computer programs such as PYGMY (Stewart, 2000) to generate its corresponding pile head load-deflection behaviour.

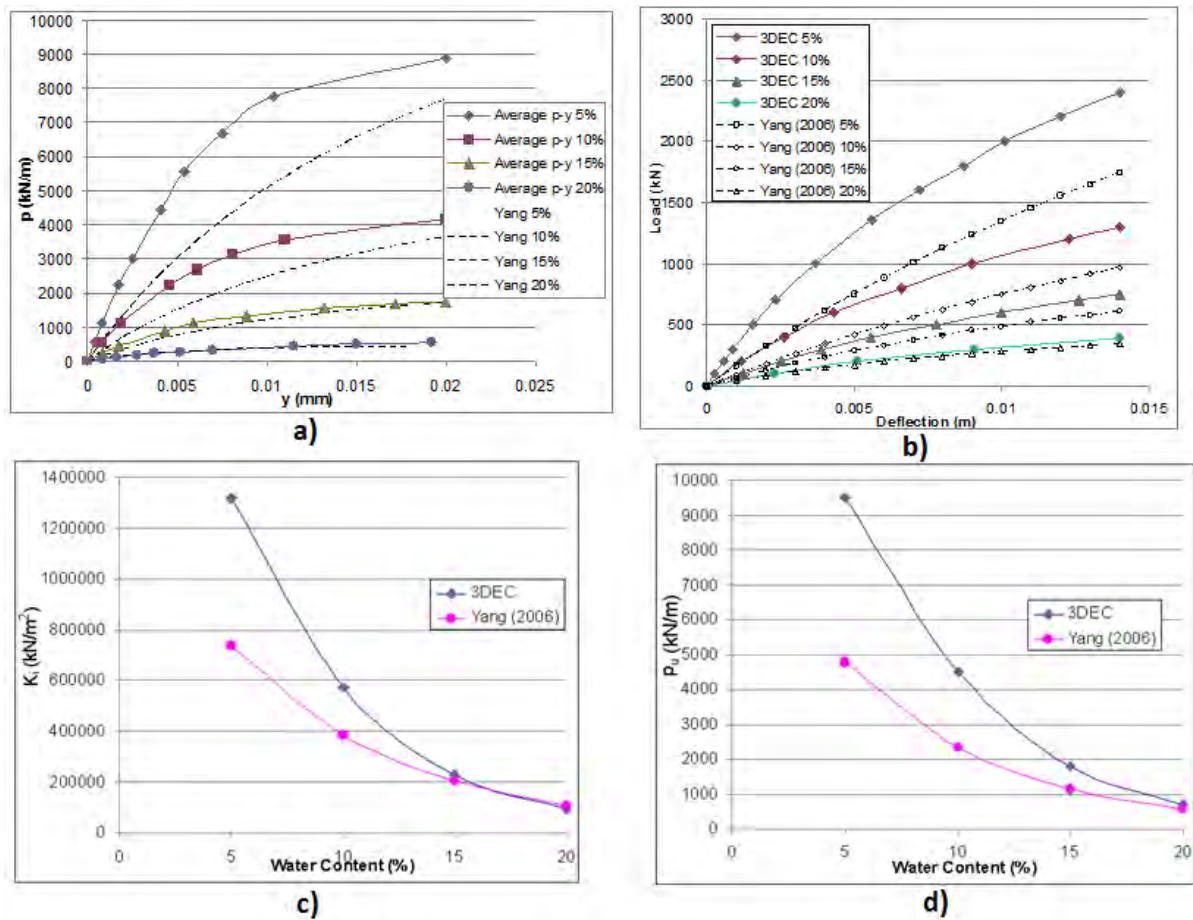


Figure 3 a) p - y curves predicted using Yang (2006), b) Pile head load-deflection behaviour predicted using Yang (2006), c) Comparison of K_i values and d) Comparison of p_u value.

$$p = \frac{1}{\frac{1}{K_i} + \frac{y}{p_u}}$$

Equation 1

The mudstone properties shown in Table 1 were used in the computation of p - y curves using Yang (2006) and the corresponding p - y curves are shown in Figure 3 by the dotted lines together with the numerically obtained average p - y

curves shown as solid lines. It can be seen in Figure 3a that Yang's (2006) p - y criterion is not able to predict the p - y behaviour of the mudstone, especially for low water content. Yang's (2006) p - y method fails to capture both the linear and non-linear parts of the p - y curves obtained from the numerical simulations. For further analysis, a comparison of the initial sub-grade reaction modulus, K_i , calculated using Yang (2006) and obtained from 3DEC simulation is shown in Figure 3c. Yang (2006) consistently underestimates the K_i values where the worst-case scenario results in an underestimation of more than 50%. Similar under-prediction can also be observed for the ultimate rock resistance, p_u , as shown in Figure 3d.

The discrepancies in these results may be due to the fact that the empirical relationships that were developed for relatively harder rocks such as shale and sandstone as reported in the field tests by Yang (2006) may not be suitable to simulate soft rocks such as the Melbourne mudstone. In addition, this method requires the use of GSI ratings based on the Hoek-Brown criterion (Hoek et al., 1988) to estimate the ultimate rock resistance is heavily reliant on the user's assessment of the rock, which may have a significant effect on the accuracy of the method. This highlights that the use of a p - y criterion based on a specific geological condition may not be appropriate when applied to other rock types.

The corresponding pile-head load-deflection behaviour using the p - y curves estimated by Yang (2006) are shown in Figure 3b, together with the 3DEC predictions. As expected, the load-deflection behaviour for different water contents is substantially underestimated by Yang (2006), especially for low mudstone water content of 5%. Although a low water content reflects a high strength mudstone which is a relatively hard rock with similar strength to the shale and sandstone rocks used to derive the p - y criterion of Yang (2006), the inability of this method to capture the behaviour of the mudstone highlights the difference in behaviour of different rock types and their associated behaviour. Hence, it is important to develop an appropriate p - y method that specifically captures the behaviour of local mudstone rock.

4. CONCLUSION

Overall, this study has shown that water content, simulating different intact strength and deformation behaviour of the local mudstone rock, has a significant effect on the p - y and corresponding pile-head load-deflection behaviour of laterally-loaded piles. A very low strength mudstone simulated by a water content of 20% may exhibit more than 50% reduction in the initial stiffness of the p - y curve as well as ultimate rock resistance compared to a fresh mudstone simulated by 5% water content. This implies that the corresponding pile-head load-deflection is also similarly affected by the change in the mudstone's water content, which reflects the strength and deformation properties. The p - y criterion of Yang (2006) has shown to be unable to predict the p - y curves of the mudstone, especially for the low water content condition, with some under-prediction of the initial sub-grade modulus, K_i , and the ultimate rock resistance, p_u , by more than 50% being observed. This may be due to the method by Yang (2006) was developed for a specific rock which may not be applicable to other rock types based on many reasons as stated before. In addition, the p - y criterion of Yang (2006) requires the use of correlations with GSI to estimate the rock mass modulus and the ultimate rock resistance, which further undermines the practicality of this method.

The lateral performance of piles socketed into mudstone is governed by two main aspects which are the intact rock behaviour and the influence of secondary structures such as discontinuities and joints. This paper has covered the first aspect only which is the effect of intact strength and deformation behaviour simulated by a different water content of mudstone. However, it is also imperative to incorporate the crucial effect of secondary structures as the local mudstone is generally jointed and dominant bedding planes are of common occurrences. To develop a more practical and reliable p - y method for laterally loaded piles socketed into mudstone, it is suggested to investigate the physical effect of joints rather than using GSI system (Yang, 2006) to model the homogenous behaviour of the rock.

5. ACKNOWLEDGEMENT

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BACK-ANALYSIS OF SOFT SOIL BEHAVIOUR USING FIELD INSTRUMENTATION DATA

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ABSTRACT

The behaviour of soft foundation soils under embankment loading is dependent on a number of geotechnical properties including the materials grading, plasticity, hydraulic conductivity, drainage path length and stress history to name but a few. Various forms of in-situ test are available to help estimate the geotechnical properties of soft foundation soils and predict their behaviour under load. This paper reviews the results of an in-situ testing program undertaken to assess the geotechnical parameters of a soft foundation soil beneath a section of dredge spoil reclamation bund earmarked as a Trial Embankment. The behaviour predicted using the in-situ test results is then compared with monitoring data collected from a series of piezometers and settlement plates installed at the Trial Embankment location. Finally, the instrumentation data is used to back calculate a number of key geotechnical parameters allowing for a review of the design methodology during construction.

1. INTRODUCTION

Over recent years, the rapid expansion in the size and number of coal mines in Australia, and particularly Queensland, has created a significant increase in the volume of material passing through the State's existing ports. Increased demand from both internal and overseas markets has resulted in the development of a number of large port infrastructure projects along the Queensland coast. In many cases, the proximity of port infrastructure to the marine environment means that there is a need to build over recently deposited sediments which are often still very soft. Construction in and around these marine environments can pose significant problems for engineers.

The behaviour of soft foundation soils under loading is an important consideration for many reasons. Issues associated with soft soils often arise during both the construction and service phases of the project and can have cost, schedule and safety implications if not addressed properly. Essential to the management of these issues is an adequate understanding of the behaviour of soft soil foundation under loading.

A number of quantifiable soil properties can be used to characterise the behaviour of soft soil when subjected to loading. These properties, which include compressibility and consolidation characteristics, grading, plasticity and stress history, form the basis with which to predict important indicators such as magnitude of settlement, rate of settlement and foundation stability. The prediction of these indicators allows engineers to modify the design and control, for example, the magnitude of post-construction settlement, the time to achieve target consolidation and factor of safety for foundation stability.

Limited in-situ and laboratory testing as well as the non-homogeneous, anisotropic nature of soil generally mean that the prediction of soft soil parameters is not straight forward. Data often has a high degree of variability and as a result, interpretation of soil parameters tends to err on the side of conservatism, rather than best-estimate.

In this paper, data collected from field instrumentation installed at a Trial Embankment is used to back-analyse key properties of soft foundation soil at the site of a large port infrastructure development in Gladstone, Central Queensland. This review of field monitoring data enables realistic soil parameters to be calculated during construction, based on the actual behaviour of soil under load. These back-analysed properties are then compared with the predicted properties used to design key elements of infrastructure on the project.

2. SITE GEOLOGY AND GEOTECHNICAL MODEL

2.1 SITE GEOLOGY

Published geological maps show the Project Site covering a number of geological units ranging from older mudstones and cherts, formed during the Early Carboniferous and Early Devonian age, to younger tidal mudflats whose sediments were deposited during the Quaternary period. These recent Quaternary age deposits are of particular interest due to both their geographical dominance of the Project Site and their engineering properties.

Predominately consisting of clays, silts and fine sands, the soft soils associated with the tidal mudflats dominate the Project Site and form the foundation for two key pieces of project infrastructure (see Figure 1), namely:

- Reclamation Area 1 dredge spoil containment bunds (location of site Trial Embankment)
- Reclamation Area 2 dredge spoil containment bunds

The clays and silts of the tidal mudflats tend to be normally to lightly overconsolidated and are typically between 1-2m thick. In some localised areas, usually associated with ancient paeleo-channels, soft muds extend to 5-6m in depth.

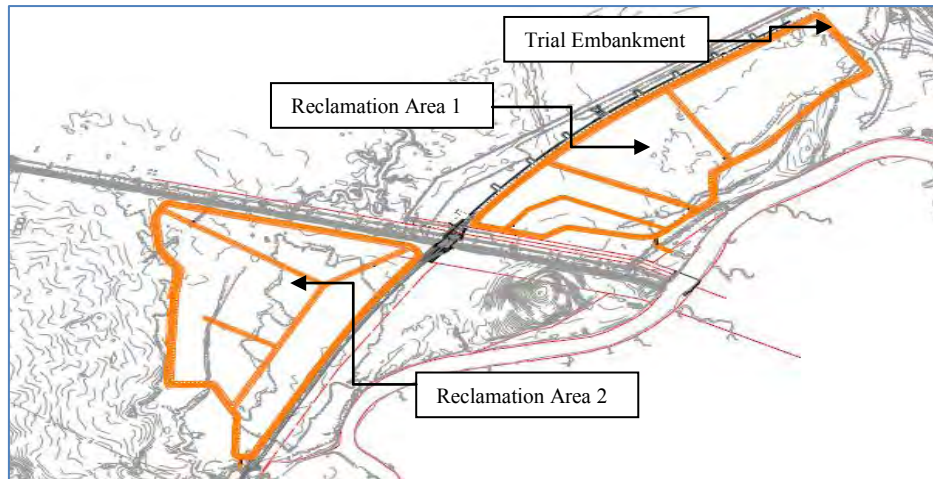


Figure 1: Location Plan showing Soft Soils

2.2 DESIGN GEOTECHNICAL MODEL

A number of extensive site investigations were undertaken across the tidal mudflats between 2006 and 2011. The primary purpose of the investigations was to gather information which would then be used to develop a series of geotechnical models to help characterise these soft soils. A number of different in-situ sampling and testing methods were used to assess the engineering properties of ground.

A series of test pits, boreholes, Cone Penetration Test's (CPT's), dissipation tests and seismic survey lines (Multi-channel Analysis of Surface Waves – MASW) were undertaken to define the thickness, strength, stress history, compressibility and permeability of the soft soils. Sampling and testing methods were utilised to provide specific information required to develop the geotechnical model.

Using data gathered during the site investigations and laboratory test results, a geotechnical model was developed for the foundation beneath the Trial Embankment. The thickness, undrained shear strength and compressibility and consolidation characteristics of the soft soil layer were defined and are presented in Table 1.

Thickness of Soft Soil, H (m)	Undrained Shear Strength, c_u (kPa)	Coefficient of Compression, c_c	Initial Voids Ratio, e_0	Coefficient of Consolidation, c_v (m ² /year)	Max. Drainage Path Length, H_d (m)
0.8 – 1.4	4 - 9	0.78 – 0.90	1.8 - 2.2	5.5	0.75

Table 1: Trial Embankment Design Geotechnical Models

3. INSTRUMENTATION AND MONITORING

Instrumentation was installed beneath the Trial Embankment to monitor the behaviour of the foundation during loading. Specifically, data collected from the instrumentation was used to assess the following:

- Stability of the embankment during construction
- Magnitude of settlement both during and post construction
- Back-analysis of soil parameters for comparison with design values
- Refine the geotechnical model to help predict future behaviour of the soft soil

The back-analysis of soil parameters is of particular interest as the variability in the properties of the soft soil identified during the geotechnical investigation resulted in the adoption of a moderately conservative geotechnical model. As

such, the field monitoring data is critical in terms of reviewing the initial design values and refining the geotechnical models to more accurately predict future behaviour.

Prior to the start of construction, a 100m section of embankment at Reclamation Area 1 was identified as a suitable location for a Trial Embankment. An early construction start and increased instrumentation at the Trial Embankment locations allowed for an early review of field monitoring data which was considered beneficial to the project. Two (2) Vibrating Wire (VW) piezometers and eight (8) settlement plates were installed beneath the Trial Embankment.

VW piezometers were installed to measure the change in pore water pressure within the soft soil layer during embankment construction. Piezometers were typically installed at a depth of 0.8m below natural ground surface below the zone of wetting-drying. Piezometers were installed within a cased vertical borehole drilled to approximately 1.0m depth and backfilled with a sand-grout mixture. A baseline reading was established to determine the in-situ pore water pressure at the tip of the piezometer prior to the beginning of construction. Typically, readings were taken for days on which fill was placed and every 3 days at all other times. A survey of the embankment height was taken for days on which fill was placed.

Settlement plates were installed to measure the settlement of the soft soil foundation during embankment construction. A steel plate of 0.6m by 0.6m was installed above a thin granular working platform constructed on natural ground. Cased steel riser rods were connected to the plate with the top of the riser rod used as a reference point for readings taken during construction. As with the piezometers, readings were taken for days on which fill was placed and every 3 days at all other times. A survey of the embankment height was taken for days on which fill was placed.

4. BACK-ANALYSIS OF MONITORING DATA

Back-analysis of the monitoring data collected at the Trial Embankment allowed a review of the design soft soil parameters and opportunity to refine the geotechnical model, where appropriate, during the construction phase. A focus was placed on the interpretation of settlement plate and piezometer data for the purpose of reviewing the compressibility and consolidation parameters estimated during design. The compressibility and consolidation parameters are critical in terms of predicting the rate and magnitude of settlement as well and the rate and magnitude of strength gain in the soft soil foundation due to loading. Each measurable is of particular interest due to their importance during construction and their difficulty to predict during design.

By reviewing piezometer data in conjunction with settlement plate data at the Trial Embankment, a relationship can be developed between the degree of consolidation and magnitude of settlement at a fixed point in time. Figures 2, 3 and 4 plot the degree of consolidation, magnitude of settlement and fill height at the Trial Embankment with respect to time.

Assuming a baseline static pore water pressure prior to loading, the degree of consolidation U at time t can be estimated from piezometer data using Equation 1.

$$U(t) = \frac{P_{peak} - P_t}{P_{peak} - P_{static}} \times 100\% \quad (1)$$

Where p_{peak} is the peak pore pressure, p_t is the pore pressure at time t and p_{static} is the baseline pore pressure prior to loading. Using pore water pressures taken from piezometers P01 and P02 at the Trial Embankment, Equation 1 can be used to plot a graph of the degree of consolidation versus time (see Figure 2). The plot shows that after 220 days, the soft foundation has, on average, undergone approximately 80% of primary consolidation. As additional data is collected over time, this value will approach 100% at which time the excess pore water pressure will have completely dissipated.

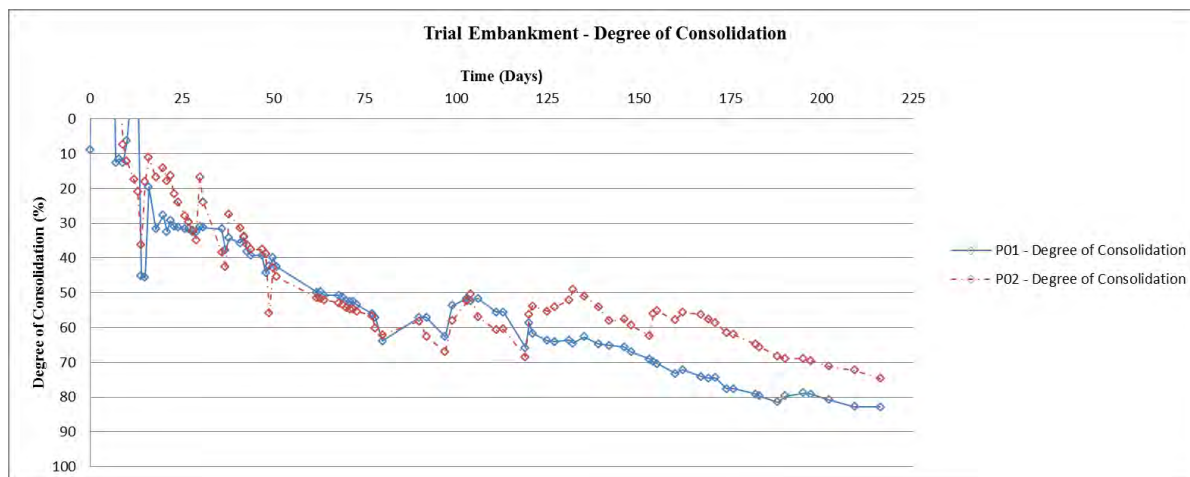


Figure 2: Degree of Consolidation – Trial Embankment Piezometers



Figure 3: Magnitude of Settlement – Trial Embankment Settlement Plates

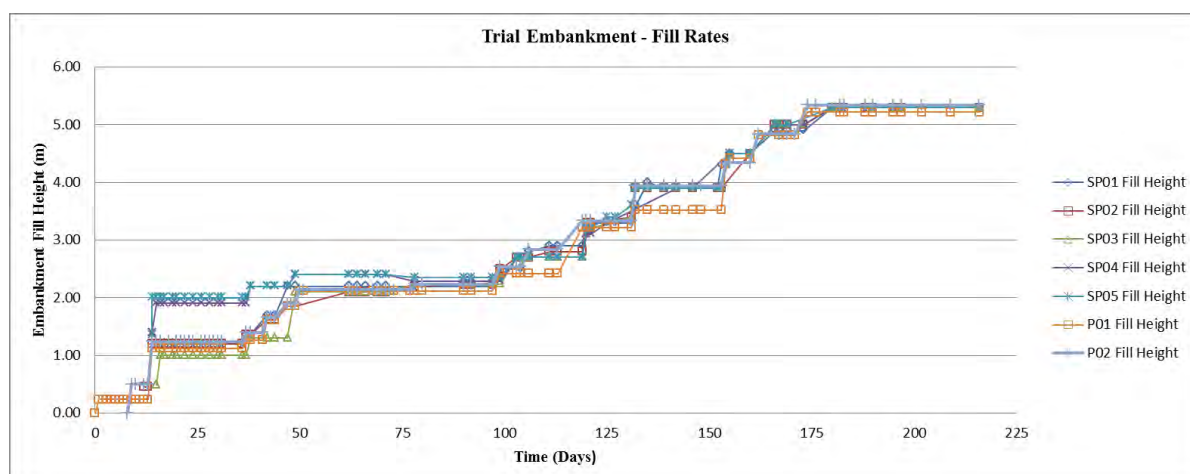


Figure 2: Embankment Fill Height – Trial Embankment

The magnitude of settlement at each of the settlement plates is plotted with respect to time in Figure 3. Typically, after 220 days, the magnitude of settlement for SP04 and SP05 plates is between 80mm and 100mm. The magnitude of settlement for SP01, SP02 and SP03 plates is between approximately 160mm and 180mm. The reason for the difference

in magnitude is primarily due to the fact that at SP04 and SP05, the Trial Embankment was constructed over an existing access track approximately 1m high.

For the case of a normally consolidated soft soil, the magnitude of settlement at the end of consolidation is defined in Equation 2.

$$s_c = \frac{c_c \log(\sigma'_1 / \sigma'_0)}{1 + e_0} H \quad (2)$$

Where (σ'_1 / σ'_0) is the change in vertical effective stress (determined using Figure 4) and H is the thickness of the soft soil layer. The magnitude of settlement at any time t is directly dependent on the degree of consolidation within the foundation. This relationship is defined in Equation 3.

$$s_c(t) = U(t)s_c \quad (3)$$

Utilising piezometer and settlement plate data from the Trial Embankment and the relationship from Equation 3, the magnitude of settlement at the end of consolidation, s_c , can be predicted. This estimate of the settlement likely to have occurred at the end of consolidation can be cross checked using a method developed by Asaoka (1978).

The Asaoka method uses settlement plate data to predict the final magnitude of settlement at the end of consolidation. Asaoka's method starts with a plot of settlement with time for a particular change in vertical effective stress. A series of settlement values are selected i.e. $p_1, p_2, p_3, \dots, p_n$ where p_n is the settlement at time n . The Asaoka method requires the time step between settlement readings to be equal i.e. $\Delta t = (t_n - t_{n-1})$. Using the selected settlement values (p_{n-1}, p_n) is plotted. Generally, the plot can be represented by Equation 4

$$p_n = p_0 + \beta_1 p_{n-1} \quad (4)$$

Where p_0 is an initial settlement value (dependant on Δt) and β_1 is the slope of the line. **Error! Reference source not found.** shows an example of a settlement with time plot and the coinciding Asaoka plot. A line of best fit is plotted through the settlement data and extrapolated to intersect with a theoretical slope where β_1 is 1. This line in effect represents the condition where no additional settlement occurs with time i.e. $p_n = p_{n-1}$. The point of intersection between the two lines coincides with the maximum predicted settlement for that particular change in vertical effective stress.

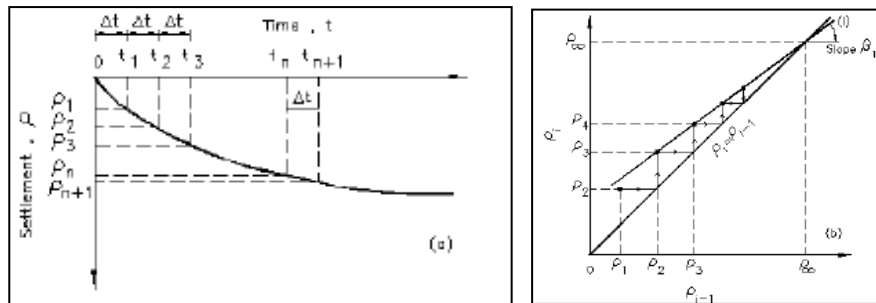


Figure 5: Asaoka Plot

4.1 COEFFICIENT OF CONSOLIDATION

The coefficient of consolidation, c_v , is of principle interest when assessing the rate of consolidation. As shown in Equations 5 and 6, c_v is the variable that defines the rate of consolidation and therefore the rate and magnitude of settlement at any stage during construction.

$$U(t) = \sqrt{\frac{4c_v t}{\pi H_d^2}}, U < 60\% \quad (5) \quad U(t) = 1 - 10^{\left[\frac{\left(\frac{c_v}{H_d^2} \right) t + 0.085}{-0.933} \right]}, U > 60\% \quad (6)$$

The coefficient of consolidation can be back-analysed using the Asaoka Plot. Equation 7 presents a relationship between the slope of the line of best fit (β_1) and the drainage path length (H_d) which can be determined from CPT testing.

$$c_v = -\frac{5}{12} H_d^2 \frac{\ln \beta_1}{\Delta t} \quad (7)$$

4.2 COMPRESSION PARAMETERS

The compression parameters of interest are the initial voids ratio (e_0) and coefficient of compression (c_c). Together e_0 and c_c are the variables that define the magnitude of consolidation and therefore both the magnitude of settlement at any time during construction and total settlement at the end of consolidation. Both compression parameters can be back-analysed from settlement plate data using the relationships developed in Equations 2 and 3. The key compression parameters can be normalised to create a single parameter, $c_c/(1+e_0)$, which can be effectively used to model the magnitude of settlement for the back-analysis.

5. COMPARISON BETWEEN PREDICTED AND BACK-ANALYSED GEOTECHNICAL PARAMETERS

As discussed in Section 4, data collected from the piezometers and settlement plates installed at the Trial Embankment location can be used to back-analyse consolidation and compression parameters for the soft soils. This back-analysis is particularly useful as it allows a review of the suitability of values used during design and assists in the prediction of soft soil behaviour into the future.

Data from the 2 piezometers and 8 settlement plates was reviewed using the methods discussed in Section 4. Back-analysed consolidation and compression parameters were derived and then compared with values used during design. Table 2 presents a comparison between design parameters and those back-analysed using field data.

	Coefficient of Consolidation, c_v (m ² /year)	Compression Parameter, $c_c/(1+e_0)$
Design Value	5.5	0.24-0.32
Back-Analysed Value	6-10	0.10-0.22

Table 2: Comparison between Predicted and Back-Analysed Geotechnical Parameters

The comparison shows that the soft soil parameters back-analysed from field data are generally better than those estimated for design. The field data shows that the value for the rate of consolidation, represented by c_v , is typically higher than that used in design, meaning that consolidation (and therefore strength gain) of the soft soils will be faster than predicted. The field data also shows that the value for the magnitude of settlement due to consolidation, represented by $c_c/(1+e_0)$, is typically lower than that used in design, meaning that observed settlements are likely to be smaller than those predicted during the design phase.

Using these revised values, a reassessment of the construction methodology was undertaken. The increased rate of consolidation, and subsequent increase in the rate of strength gain within the soft foundation resulted in an approval to increase the rate of embankment construction. In addition to this, a reduction in the estimated total settlement resulted in a review, and subsequent reduction, of the required embankment surcharge duration. Both outcomes combined resulted in a reduction in the length of the construction schedule and cost saving to the project.

6. CONCLUSIONS

The non-homogeneous, anisotropic nature of many soft soils means that, even with a reasonable amount of in-situ and laboratory testing, their behaviour under embankment loading can be difficult to predict. For this reason, field observations made using instrumentation data often provide the best insight into soft soil behaviour. Well established analytical and empirical relationships exist that allow back-analysis of key geotechnical parameters using monitoring data.

From the assessment of piezometer and settlement plate data at the Trial Embankment, key geotechnical parameters were back analysed to revise and update the conservative predictions made during the design phase. By utilising data collected from the field instrumentation, modifications were made to the construction methodology that resulted in both cost and schedule savings for the project.

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CASE STUDY: CONSTRUCTION OF AN EXTENSION TO A CONTINUOUSLY SETTLING PILE RAFT IN THE YARRA DELTA GEOLOGY OF MELBOURNE

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ABSTRACT

This paper case studies issues associated with differential settlement between an existing structure on a continually settling raft and a proposed modern extension. The subject site, comprising a 2 storey office building was constructed in 1987 and was one of the first piled raft projects in Australia. Accordingly a paper was written on the project in 1988 detailing the design. The selected foundation system was a suspended raft supported by driven, cast in-situ and enlarged based piles founded in Coode Island Silt. The progression of piling technology has seen cheaper more robust options become readily available. As the settlement of the existing 1987 raft is not linear with respect to time, the issue of differential settlement with respect to the proposed 2011 extension, to be founded on rigid precast piles to rock, became a serious issue prior to construction. This paper summarises the two dimensional finite element analyses of varying the pile lengths for the new extension and their influence on calculated differential settlement and induced ground beam forces. The site restrictions virtually eliminated the possibility of articulation and accordingly an iterative approach between the geotechnical engineer and structural engineer was adopted to arrive at a foundation solution which would not lead to excessive ground beam forces.

1. INTRODUCTION

The Melbourne Docklands and South Melbourne areas experience significant ongoing creep as a result of the normally consolidated silty clay known as Coode Island Silt (CIS) that is present in the Yarra Delta geology.

Buildings and embankments founded above this CIS within in the Yarra Delta group have experienced settlements of up to 790mm over 66 years. Ongoing creep settlements within the CIS layer are known to vary between 3mm to 10mm per year depending upon the thickness of the CIS.

The CIS layer varies in thickness throughout the Yarra Delta group and presents a significant issue for highly loaded structures. To limit differential settlements, buildings are often supported on deep piles founded within the Melbourne siltstone beneath the Coode Island Silt.

In 1986 a foundation solution was adopted for this site, which bucked the trend for multistorey developments of variable heights within Coode Island Silt affected geology zones. In lieu of deep end bearing piles, this multistorey building, which varied between 1 to 3 storey's in height, was founded on a pile stabilised raft, the first of its kind in Australia.

The foundations for this building derived their support within the CIS layer and were designed only to limit the differential settlement of the structure, ensuring its serviceability. It was expected that the building would experience large initial settlements, and ongoing creep due to the CIS layer.

The initial settlement combined with ongoing creep results in nonlinear settlement with respect to time. This would prove to be a difficult issue to overcome for extensions to the building many years later.

2. SITE CONDITIONS

As stated above the site falls within the Yarra Delta geology overlying Silurian age Melbourne siltstone. The Yarra Delta geology group contains some or all of the following members; Port Melbourne Sand, Coode Island Silt, Jolimont Clay, Newer Volcanics, Burnley Basalt Flow, Fishermens Bend Silt and Moray Street Gravels. The subsurface profile for the site is summarised in table 1 below.

The CIS layer is of a fairly uniform thickness and firm consistency across the site. Cone penetrometer tests within the CIS demonstrated an increase in cone tip resistance with depth within the range of 30z to 50z kPa (where z is depth).

Nearby observations from an adjoining street from 1981-1985 indicate an ongoing settlement rate of 10mm per year, a figure which is consistent with other measurements over similar thicknesses of CIS.

Though the CIS is noted as being normally consolidated, tests carried out in 1986 prior to construction, demonstrated a small apparent preconsolidation pressure, indicating that initial soil response would follow the reloading curve and then the virgin loading curve.

Depth	Subsurface Layer
0-2.5m	Variable Fill Layer
2.5m-19m	Coode Island Silt (CIS) – Silty CLAY, CH, Firm, highly compressible with shells and organic matter.
19m-24m	Fishermens Bend Silt (FBS) – Sandy SILT, MH, Firm to Stiff
24m-28m	Moray Street Gravels (MGS) – Clayey SAND with gravels, medium dense to very dense.
28m+	Melbourne Mudstone/Siltstone. Highly Weathered

Table 1: Design Subsurface Profile

3. ORIGINAL BUILDING AND FOUNDATION SYSTEM

The original building was founded on varying lengths of driven cast in-situ enlarged base piles with a base diameter of 750mm and a pile shaft diameter of 450mm.

Piles were designed to stiffen the settlement response of the building raft based on the method of analysis published by Randolph and Carter et al (1985). It should be noted that the piled raft design was based on elastic theory and nonlinear soil behaviour was not considered as pressures and forces were designed to fall within the elastic portion of the soil settlement curves. Creep settlement was analysed via an equivalent Young's modulus for total creep over the 50 year design life. Figure 1 below shows the footprint of the original 6 buildings.

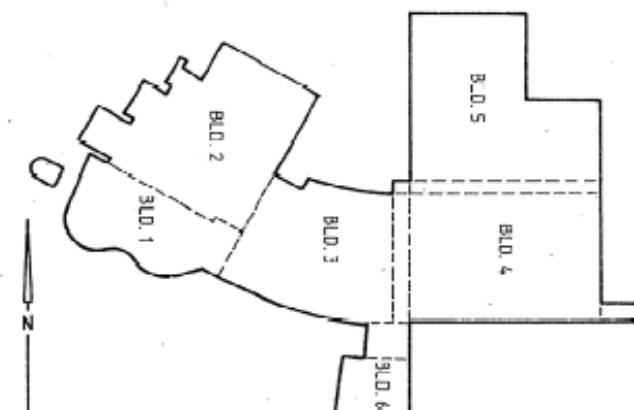


Figure 1: Layout of the existing buildings constructed in 1986

Piles were installed to varying depths within the CIS layer. The depths of piles were varied relative to the load they were carrying, such that piles carrying the highest load had the thinnest amount of CIS below their bases. This proportioning of thickness of CIS below pile base was to limit the differential settlement across the building.

Building	Long Term Bearing Pressure on Building Footprint (kPa)	Pile Depth (m)
1	38	18
2	15	7
3	29	18
4	30	18
5	15	7
6	21	12

Table 2: Original Building pressures and Pile Depths

Final design of the stiffened piled raft predicted total settlements of 110mm over the 50year design life. Of note is that over the design life the six buildings were designed with articulation between each other to accommodate 50mm differential settlement.

4. DIFFERENTIAL SETTLEMENT ISSUES FOR THE PROPOSED EXTENSION TO EXISTING BUILDINGS

The ongoing creep of the CIS layer beneath the existing building indicates that the original buildings 2, 5 and 6, constructed in 1987, will continue to settle with time. It should be noted however that buildings 1, 3 and 4 are not expected to experience significant creep due to being constructed on piles founded near the base of the CIS layer.

The design life for these buildings continues through to 2037, 50 years post construction, at which point differential settlement has been designed to be less than 50mm between each of the buildings 1 to 6.

The available space for new extensions on the site is restricted by the site boundaries; this virtually eliminates the possibility of articulation between the existing buildings and the new extensions. Consequently differential settlements between the existing buildings and the planned extension were required to be limited such that the building ground beams and connections can withstand induced moments and shears.

Considering the above details, two foundations options became apparent each with their own pros and cons:

Founding the new extension on precast piles driven to effective refusal on either Silurian Melbourne Siltstone or Moray Street Gravels:

- Initial settlement would be minimal;
- Creep settlements would be eliminated from new structure;
- Cost effective solution due to less piles with more load per pile; and
- Differential settlements could increase by 10mm per year after initial settlement between the existing and the new buildings.

Founding the new extension on a stiffened raft slab with piles founded within the CIS layer:

- Initial settlements would be large leading to differential settlements between the extension and existing buildings;
- Significant differential settlements would occur early in the structures life;
- The foundation would likely approach similar creep rate as existing buildings on the site; and
- More expensive and an extended piling program due to more piles with less load.

A report by Golder Associates assessed the likely settlement performance of each of the above proposed solutions and prepared a preliminary graph comparing predicted settlements to the upper and lower bound settlement performance of the existing buildings.

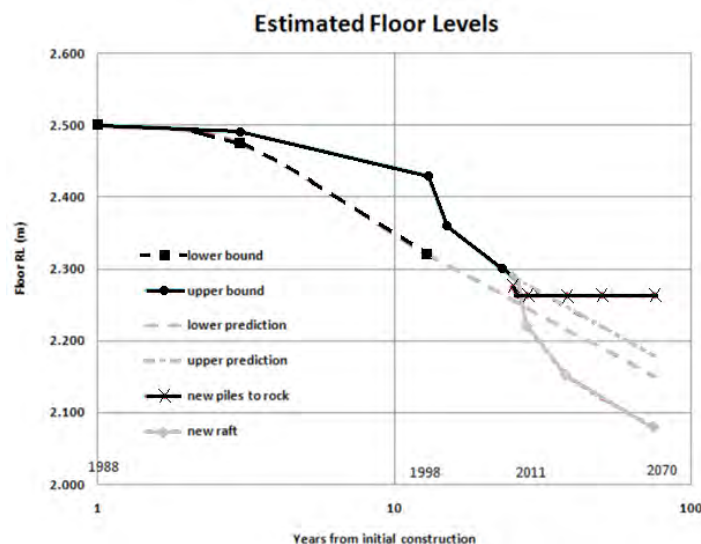


Figure 2: Estimated Floor Level Over Time (Golder Associates Report 11762009-001L dated 1/2/2011)

Figure 2 demonstrates that, considering the long term performance of the structure, adopting a piled raft solution would be the most appropriate method of supporting the new extension due to the creep rate of the extension having a better match in the long term to the creep rate of the original building. As the site restrictions precluded the use of building

articulation to accommodate differential settlement, an estimate of induced ground beam forces was made to ensure that the development remains serviceable throughout its design life.

5. A PREDICTION OF GROUND BEAM FORCES IMPOSED BY DIFFERENTIAL SETTLEMENTS

To accurately assess the differential settlement between the existing buildings on the site and the proposed extensions, the full construction process must be analysed, including the in-situ ground conditions prior to construction of the original development in 1987 and the creep experienced to date.

To carry out this analysis Golder Associates utilized the finite element package PLAXIS-2D. The soft soil creep model was used to model the CIS with the following material parameters which have been based on previous laboratory results and published technical data:

- A modified compression index $\lambda^*=0.1$
- A modified swelling index $k^*=0.02$
- A modified creep index $\mu^* = 0.004$

Table 3 below, gives the parameters adopted by Golder Associates for their analysis based on the available geotechnical investigations, test results and correlating the model against available settlement data for the existing buildings.

Depth	Layer	γ	c'	Φ'	E	ν
		(kN/m ³)	(kPa)	(deg)	(MPa)	
0-2.5m	Fill	19	1	32	25	0.20
2.5m-19m	CIS	16	1	25	2+0.3/m	0.15
19m-24m	FBS	18	1	27	20	0.15
24m-28m	MSG	20	0	36	50	0.20
28m+	Siltstone	22	100	37	300	0.25

Table 3: Adopted Soil Parameters and Ground Profile

γ = unit weight, c' = drained cohesion, Φ' = drained friction angle, E = Young's Modulus and ν = Poisson's ratio

The loading and construction sequence modelled in the PLAXIS analysis was as follows:

- Step 1 – Initial ground conditions;
- Step 2 – Set displacements to zero;
- Step 3 – Install existing piled raft foundation;
- Step 4 – Set displacements to zero;
- Step 5 – Apply load to the existing building piled raft;
- Step 6 – Creep settlement for 23 years (from 1988 to 2011);
- Step 7 – Install new precast pile foundations;
- Step 8 – Apply new building loads; and
- Step 9 – 50 years creep settlement (from year 2011 to 2061).

Given the different pile depths and loads for each of the 6 existing building rafts and the desire to analyse various lengths of piles for the building extensions in order to limit differential settlements, several sections across the existing building and proposed extensions were analysed in PLAXIS-2D for varying new pile lengths.

As the PLAXIS-2D model required input of stiffness for the ground slab and ground beams to accurately model the load spreading and settlement interaction, there were inevitably iterations between Golder Associates and the structural engineer to bring both the structural model (RC Building) and PLAXIS-2D models into agreement. It should be noted that significant iterations and difficulties were encountered in fine tuning ground/structure interaction stiffness's such that both programs demonstrated agreement.

As part of the design process of the piles, the effect of creep on differential settlement was investigated for varying pile lengths across what was considered a critical section of the existing building and proposed new building. Five pile lengths were investigated each with varying thicknesses of CIS. The results are shown in Figure 3 below.

The graph demonstrates the piles founded 4m above the CIS achieve the least amount of differential settlement over the 50 year design life of the proposed extensions. This information was then used to progress the design for the other critical building sections.

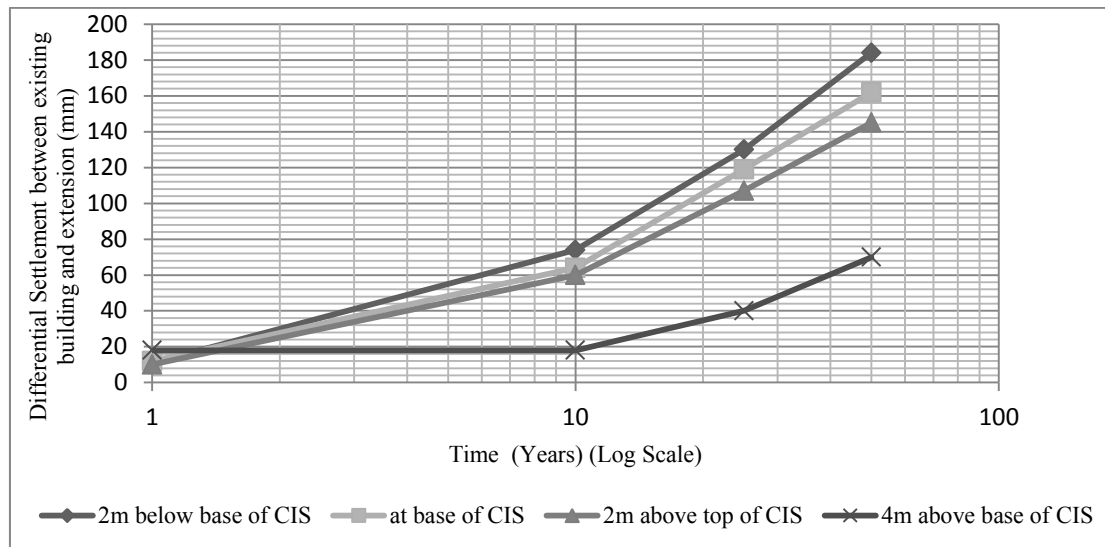


Figure 3: Differential Settlement between existing buildings and proposed extensions

As part of the design it was necessary for the structural engineers to have confidence in the calculated ground slab peak moments and shears induced by differential settlement. These forces were obtained from the PLAXIS analyses carried out by Golder Associates and RC Building analyses carried out by the structural engineer, after fine tuning ground stiffness values the results proved to agree within 10%.

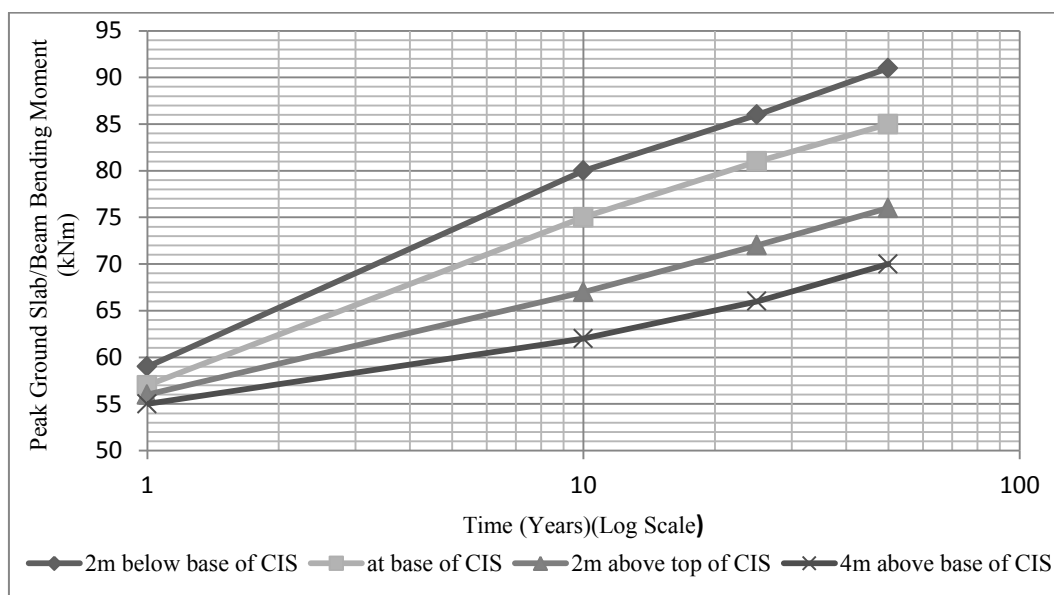


Figure 4: Induced peak ground slab/beam bending moments

Figure 4 above, demonstrates that as expected a lower differential settlement results in lower peak bending moments within the ground slab/beam system. Shear forces exhibited a similar correlation.

6. CONCLUSIONS

The original piled raft solution was a state of the art design solution at the time, which led to significant cost and program reductions for the client. The presence of the normally consolidated layer of Coode Island Silt (CIS) however meant that any significant extensions to the building were bound to be problematic.

The progression of piling technology has seen an influx of cheap, fast, rigid piling solutions. Accordingly one would assume that a rigid piling solution providing the least amount of settlement would be the appropriate solution. However in this case the ongoing high creep rate of the CIS and long design life of the building, after inspection, almost instantly voided a rigid piled solution and a less stiff settlement reducing mechanism was required.

Following extensive geotechnical modeling it became apparent that some form of articulation was necessary to limit ground beam and slab forces. Accordingly slab connections between the existing buildings and proposed extension were detailed and designed as pinned connections such that they only transferred shear forces.

A deflection profile for each critical section was developed by Golder Associates using PLAXIS and this was then mimicked by the structural engineer's analyses in RC Building which in turn provided shear forces and bending moments which the structure could be designed for.

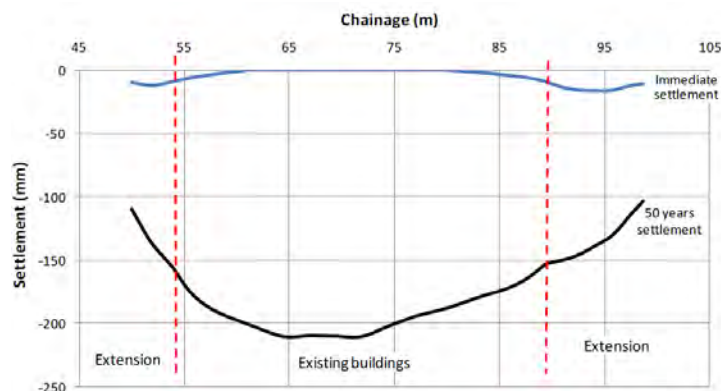


Figure 5: Example settlement profile from Golder Associates Report 117612009-012-L-Rev0 dated 13/7/2011

The final solution for the foundations was a series of piled rafts for the proposed extension buildings. The rafts involved 15m, 18m and 21m long precast piles founded within (or immediately below the base of) the Coode Island Silt layer. The piles were designed as settlement reducers and not to act in the typical sense of a pile. The focus of the design was not to limit total settlement but differential settlement such that the serviceability of the building would be maintained for its design life of 50 years.

The interaction of the initial settlement, creep and structural/soil load redistribution is complex and intertwined. Accurately analysing the problem and designing a solution required significant amounts of computation time and iterations between the geotechnical engineer and the structural engineer and would have been nigh impossible 30 years ago without the assistance of modern computing techniques and programs.

The stark partition that often exists between the geotechnical and structural engineering disciplines and the methods of analyses developed for each, is an area of soil structure interaction that requires attention. This project could be taken as an example of what can be accomplished when geotechnical and structural disciplines work closely together, however it should be noted that the physical and computational segregation of geotechnical and structural analyses hindered the process.

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SEISMIC TECHNOLOGY FOR SLOPE STABILITY INVESTIGATION IN YEPPON NORTH QUEENSLAND

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ABSTRACT

Slope instability has become one of the most significant geotechnical risks in recent times. Unfavourable geologic and climatic conditions, combined with anthropogenic activities, result in thousands of slope failures each year. Standard methods of geotechnical investigation such as boreholes and SPT-tests are not consistent in providing the geotechnical data for the site as a whole. Leaving engineers to assume the soil properties for most of the site, a disadvantage that may significantly increase the cost of remediation measures. Geophysics, in particular seismic surveying can provide alternative methods to solve this geotechnical problem. Although seismic surveys are widely used throughout the mining industry, it has not been applied to slope stability problems in Australia. This is mostly due to a lack of awareness, availability and acceptance/application of modern science. This paper seeks to establish the advantages of seismic methods in resolving slope instability issues.

This paper illustrates the advantages of using seismic technology by presenting a case study, seismic technology for slope stability investigation in Yeppoon North Queensland. After significant rain in February 2008 a medium sized landslide occurred at the top of an embankment in the road reserve at Statue Bay, Yeppoon. The landslide regressed into the rear of the private properties above and therefore remediation measures were required. This is a reasonably standard geotechnical task, but when combined with steep vegetated slopes and very limited access, standard methods were not adequate. Seismic refraction and Multichannel Analysis of Surface Waves (MASW) surveying methods were used to generate 2D & 3D sub-surface images. These images distinctively illustrated the sub-surface material profiles, including the rock line and areas of slip prone material.

1 INTRODUCTION

Slope instability has become one of the most significant geotechnical risks in recent times. In Australia the Thredbo Landslide in 1997 (Tayler, 2010) in which 18 people lost their life, is probably the most well known. This landslide in particular, highlighted to councils and engineers throughout Australia the importance of undertaking an appropriate slope instability investigation.

Landslides occur when the downward forces, including the self weight exceeds the shear strength holding the soil on the slope. Shear strength loss can be attributed to excess pore water pressure, weathering of the soil profile and anthropogenic activities. The most important information required when conducting a slope stability investigation is soil depth, soil profile, soil strength and groundwater conditions (Varnes, 1978).

Most geotechnical engineers at some point have been left inadequately equipped to undertake slope stability investigation when supplied with a common AS2870 & AS1726 compliant geotechnical reports. These are based on standard test methods. These methods give results at the test point. However, even with numerous test points, the engineer is left to assume the soil profiles in-between these points. These assumptions regularly leave the engineer at risk of missing vital sub-surface data, such as the presence of colluvium, soft soils, a spike and or a depression in the rock line.

Seismic technology has been applied to many geotechnical problems, such as bearing capacity evaluation, earthquake site classification, compaction certification, subsidence investigation and now slope stability investigation. It gives the geotechnical engineer another tool to compile the geotechnical model for the site. Combined with standard methods the seismic results can join the test points, reducing assumptions and the giving the engineer confidence in the sub-surface profile. This paper will show how seismic technology can be used to aid in slope stability investigations.

2. CASE STUDY



Figure 1: A view of the landslide from the top of the bluff.

This site is located on the top of a bluff in Statue Bay, Yeppoon North Queensland. It consists of two residential sized blocks with a 27m high steep vegetated embankment at the rear (Figure 1). The embankment was cut by the military in the 1930s to construct a road around the base of the bluff. The construction drawings show that the natural slope was about 30-40° before the cut and that the cut batters vary from about 60-65° in the sedimentary rock, to vertical in the tuff. The road below is considered a main road that connects outlying communities to the city centre of Yeppoon.

The residents advised that the landslide was activated during heavy rain in February 2008. This was confirmed by the rain records where 385mm of rain fell in 3 days, over 30% of the yearly total (Bureau of Meteorology). The landslide occurred in the road reserve below and extended about 1.5m into the rear of the private properties above (Figure 1&3). The toe of the landslide also blocked the road below and it remained closed for a number of days during the cleanup operation. The complex debris slide (Varnes, 1978) was about 16 x 9 x 3m deep and had a volume of approximately 285m³ (Figure 2&3). To ensure the safety of the road users and the residents remediation measures were required.

This is a reasonably standard geotechnical task, but when combined with steep vegetated slopes and very limited access, standard methods were not considered adequate. Therefore seismic MASW and refraction surveying methods were utilised to give additional information on the sub-surface material.

2.1 FIELD INVESTIGATION

The site was tested by augering four boreholes to provide information on the soil profile in accordance with AS1726-1996, Geotechnical Site Investigations. Due to the site access limitations this was undertaken using a small power auger to about 1.5m then hand augers to greater depths. These results were augmented by the use of four seismic surveying lines, five dynamic cone penetrometer tests and shear vane testing. Disturbed and undisturbed samples were also collected for laboratory testing. Tests selected were Atterberg limits and Shear box testing. A brief site level survey was also carried out to enable computer aided slope analysis.

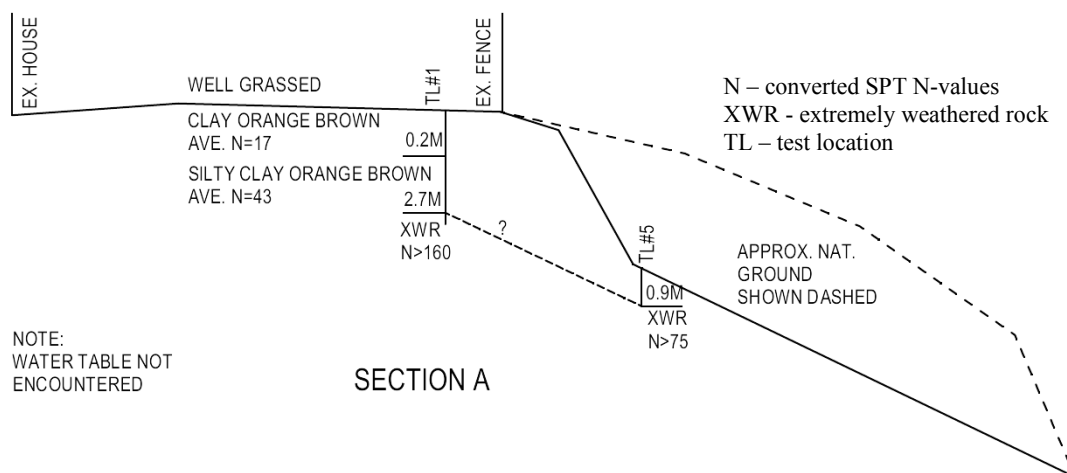


Figure 2: A view of section A.

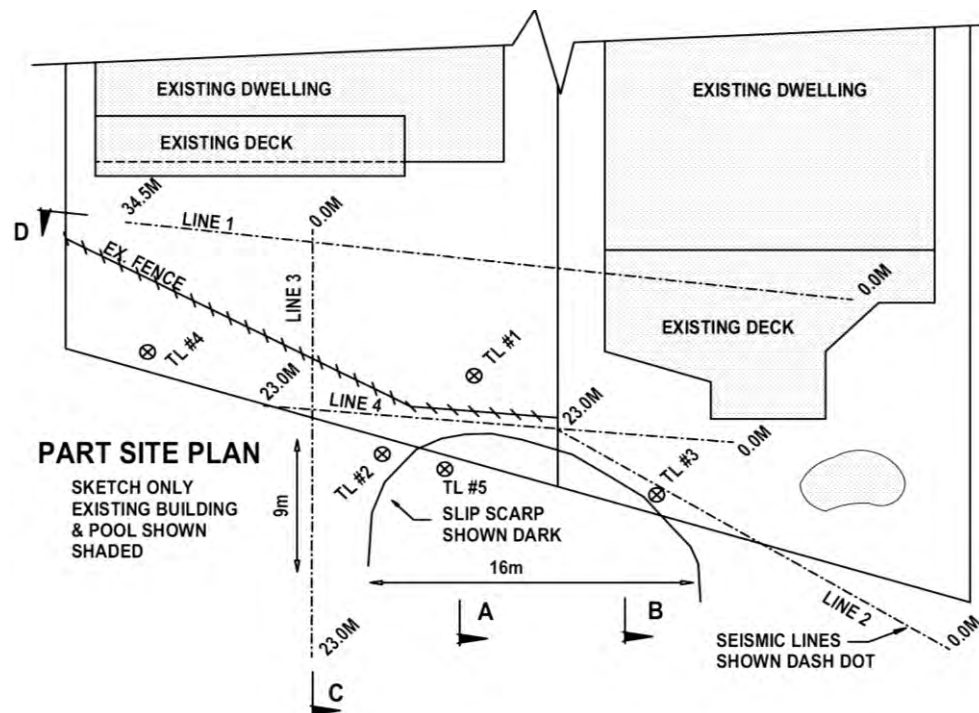


Figure 3: A view of the site plan (Note: TL indicates test location).

2.2 INVESTIGATION RESULTS

2.2.1 BORELOGS

The boreholes indicated a variable soil profile. Generally the profile consisted of a clay layer overlying extremely weathered rock. The clay layer varied in thickness from 1m at test location (TL) number 4 to 3m at TL 2. Two examples of the borelogs are included in Tables 1 & 2.

TL#1

Geologic Profile	Depth (m)	Soil or Rock Structure	Consistency	Shear vane (kPa)
Fill	0 to 0.2	SILTY CLAY, dark brown topsoil, cobbles, moist layer, CH	firm	20, 40,50
Residual?	0.2 to 1.2	SILTY CLAY, orange brown, moist layer, CH	stiff	N/A
	1.2 to 2.7	SILTY CLAY, with gravel and sands, orange brown, dry layer, CL	very stiff	N/A
	2.7	UTP, XWR?		

Table 1: Is the borelog from test location number 1 (Note: TL - test location, XWR - extremely weathered rock, UTP – unable to penetrate).

TL#3

Geologic Profile	Depth (m)	Soil or Rock Structure	Consistency	Shear vane (kPa)
Residual?	0 to 1.6	SILTY CLAY, light brown with gravels, dry layer, CH	stiff	N/A
	1.6	UTP, XWR?		

Table 2: Is the borelog from test location number 3 (Note: TL - test location, XWR - extremely weathered rock, UTP – unable to penetrate).

2.2.2 DCP RESULTS

The standard DCP results indicate variable strength material throughout the soil profile. As previously stated, these results are only relevant at the test point. In contrast, the seismic velocities are averaged over about four meters. If these averaged seismic velocities are converted to DCP results, using the empirical formula by Karai (1966), the profile is more consistent and generally increasing in strength with depth. Figures 4&5 show how the standard DCP results compare to the converted DCP results, for test locations 1&3. These figures show a remarkable correlation between the standard DCP results and the converted DCP results.

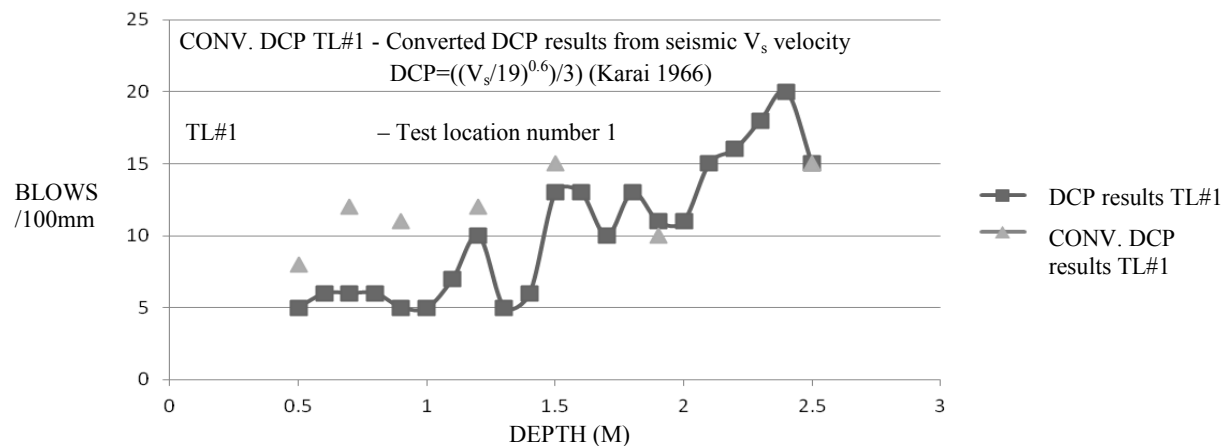


Figure 4: Is an example of the correlation between standard DCP results and Converted DCP results from seismic velocity for test location number 1.

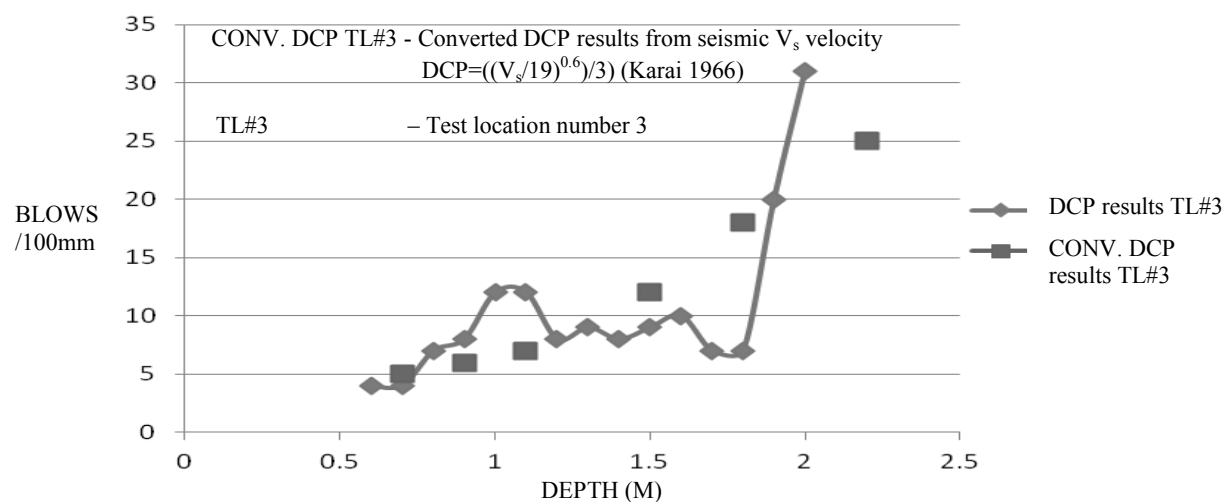


Figure 5: Is an example of the correlation between standard DCP results and Converted DCP results from seismic velocity for test location number 3.

2.2.3 LABORATORY TESTS SUMMARY

Material/location/ depth	Initial moisture content %	Liquid limit %	Plastic limit %	Plasticity index %	CLASSIFICA- TION
SILTY CLAY / TL#1-4 / 1.0-2.0m	23- 28	60-75	27-34	36-40	CH

Table 3: Is a summary of the Atterberg limits results for numerous samples throughout the profile (Note: TL - test location, CH – high plasticity clay).

Material/location/depth	Range of Internal Friction Angles (phi)	Cohesion (C) Range
SILTY CLAY / TL#1-3 / 1.0-2.0m	34-39°	30-40 kPa

Table 4: Is a summary of the shear box results for three undisturbed samples (Note: TL - test location).

The laboratory results indicate that the clay across the site has similar properties and parameters. The values of cohesion appear higher than expected. This was considered to be due to the strain rate being too fast to allow the pore water to dissipate resulting in higher cohesion values.

It is worth noting that, that the internal friction angle is generally the maximum slope that the soil can remain stable at in the long term. In this case, the natural slope was between 30-40°, which is similar to the measured internal friction angle. After the road was cut, the embankment was steepened to about 60-65°, well above the internal friction angle. This significantly increased the risk of instability of the embankment. This combined with unfavorable climatic conditions and high plasticity clays were some of the main causes of the landslide.

2.2.4 SEISMIC RESULTS

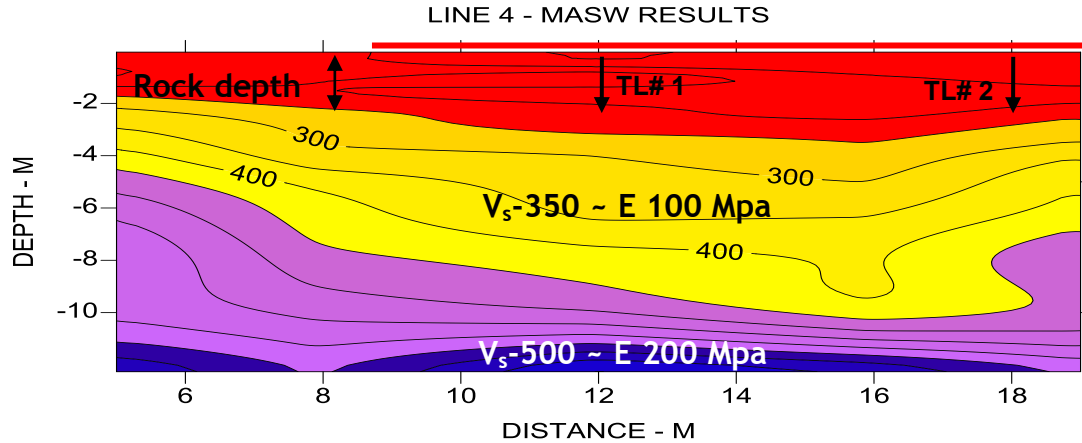


Figure 6: Results of an example of the 2D V_s profile (Note: red - soft clays, yellow extremely weathered rock (XWR), purple & blue fresh rock, E- Young's modulus, red line - location of landslide).

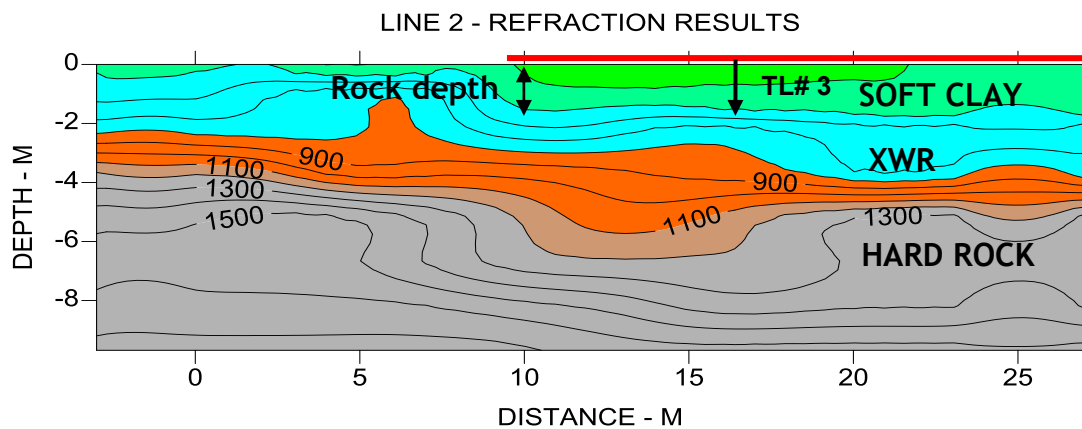


Figure 7: Results of an example of the 2D V_p profile (Note: green -soft clays , aqua & orange extremely weathered rock (XWR) and grey harder rock, red line indicates location of landslide).

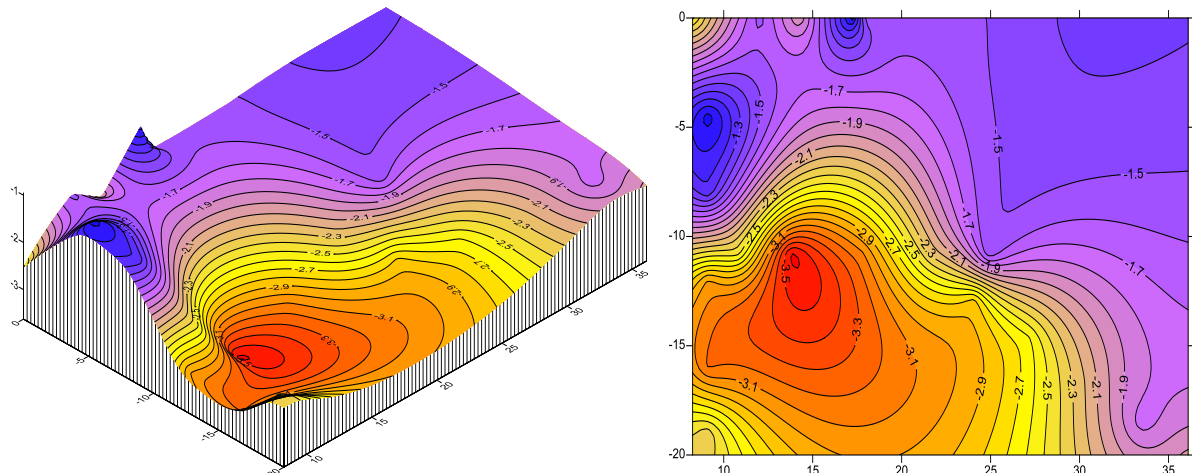


Figure 8: Results of the MASW 3D and contour plan of $V_s \sim 250$ (rock line) profile (Note: slip occurred in the depressed zone).

There are many correlations from seismic velocities to geotechnical properties. The two most notable are the empirical relationship to SPT-N value and the mathematical relationship to Young's Modulus. Surface wave velocity (V_s) has been correlated by numerous authors to the SPT N values. The correlation used in this paper is $V_s = 19(N_{60})^{0.6}$ by Karai (1966). The mathematical relationship to Young's Modulus is relevant at small strain. If the strain is higher a correlation factor is required to a factor down the calculated Young's Modulus. In this case it was used to assess the strength of the underlying rock. This was considered to have low strain and the correlation factor was not required. The formula used in this case study is as follows:

$$V_s = [E / \{2(1+\nu)\rho\}]^{1/2} \quad (1)$$

V_s = Seismic surface wave speed, as obtained on site by testing (m/sec)

E = Young's modulus at small strain (kPa)

ν = Poisson's ratio usually as tabled in numerous text books (eg Look 2007)

ρ = in-situ weight of material, usually as tabled in numerous text books (g/cm^3)

Note: To assess E , ν and ρ text book values are usually adequate.

3. DISCUSSION

The main cause that contributed to the landslide was that the embankment was significantly steeper than the internal friction angle of the soil. The trigger of this landslide, as with many, was the significant rain event that occurred in February 2008. This rain saturated the high plasticity clays causing an increase in pore water pressure and a decrease in effective stress, resulting in instability.

The seismic output gave an overall view of the sub-surface profiles and strengths, this made it easy for the engineer to construct the geotechnical model for the site and assess the geotechnical issues. In this case, the seismic survey gave a clear indication of how the rock and soil profiles varied over the site. It clearly shows that the rock profile is deeper within the slip zone (Figures 6-8), overlaid by weaker clays. The standard soil testing methods confirmed this result.

Even though the seismic results are averaged both vertically and horizontally the converted DCP results correlated well to the standard DCP results (Figures 4&5). The results did differ, however the differences can be associated to the averaging effect of the seismic survey.

The seismic survey indicated sub-surface strengths to about 12m. Due to the poor site access, this wasn't possible using standard methods. The depth of penetration of the seismic survey enabled the geotechnical engineer to access the rock strengths to a greater depth as required.

The seismic survey provided the engineer confidence to design an economical solution. If this survey had not been carried out the design engineer would not have the required information. He would have been forced to make assumptions which can lead to overly conservative or even worse an overly optimistic design.

4. CONCLUSION

In conclusion, the seismic survey was an efficient and economical way to test the site as a whole. When combined with standard test methods, it enabled the geotechnical engineer to gain a deeper understanding of the sub-surface profile and strength. This ultimately led to an appropriate design solution.

Soil investigations within Australia would greatly benefit by the application of seismic technology. Seismic surveying techniques offer the geotechnical engineer another tool to build the geotechnical model for the site. The depth of penetration and the relationships to sub surface material properties gives the engineer an overall view of the site. The seismic results reduce assumptions and give the engineer confidence to construct a geotechnical model for any site.

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CASE STUDY: EVALUATING THE NEED FOR A CAPILLARY BREAK IN A TAILINGS STORAGE FACILITY IN SOUTH AUSTRALIA

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ABSTRACT

The closure of a Tailings Storage Facility (TSF) generally involves the design of a cover system. Some of the various types of covers are nominated by their functionality such as isolation, barrier, capillary break, store-and-release covers, and are selected upon the particular region climate and project closure objectives. As part of the cover design for a TSF in South Australia, a capillary break for the site specific climatic and geographical conditions was evaluated. This paper presents how tailings and cover candidate materials were characterized using results of a specific site campaign and water retention tests. The analyses included a seepage model using a true surface flux boundary to weigh the need of the capillary break cover and predict the short and long term seepage behaviour of hyper saline process water that was disposed of with the tailings. Pre and post rehabilitation scenarios were modelled to understand the best timing strategy for placing the cover. As part of the conclusions of the work, it was determined that a capillary break was not needed, since the wetting front due to local storm events would not significantly seep through the isolation cover into the tailings body.

1. INTRODUCTION

When a Tailings Storage Facility (TSF) is no longer in use, the impoundment is generally closed for reasons such as environmental commitments, liabilities, safety, etc. Capping systems used at closure are directly related to the closure objectives and site specific conditions. Some types of covers are:

- Isolation Cover – prevents direct contact of animals and vegetation with the underlying material;
- Barrier Cover – acts as a physical barrier that incorporates at least one low permeability layer to physically limit the infiltration of water and migration of gas or air;
- Capillary Break – uses contrasts in hydraulic properties to prevent the rise of moisture from the capped material through capillarity;
- Store-and-Release Cover – active layers that store the water from storms to release it later to the environment as evapo-transpiration, avoiding the infiltration into the covered material.

This paper presents a case study, where the need for a capillary break was evaluated as part of the closure plan for a TSF in South Australia.

2. CLOSURE OBJECTIVE AND CONDITIONS

The owner of the TSF intended to progressively rehabilitate their decommissioned facilities with vegetation, which would require the placement of a cover material able to support the growth. One of the risks involved in this plan was the migration of salts from the tailings process water by capillary rise into the growth media that could damage the vegetation. Therefore, the need of a capillary break that could prevent this from happening was to be investigated.

The climate in the area is characterized by a marginal rainfall, high evaporative demand and hot summers and mild winters with mean temperatures of 24°C and 11°C, respectively. The mean annual precipitation is 202 mm, while the potential evaporation is 2,542 mm i.e. large evaporative surplus.

The TSF being studied was an empty pit receiving tailings, still in operation. The facility was used to impound flocculated fine fraction tailings co-disposed with non-mineralised sands. Free water was pumped back to the operation. The local stratigraphy was composed of 5 to 20 m deep sands overlaying granite and gneiss bedrock. Groundwater was generally not encountered in the sands and is associated with the bedrock at depths of approximately 50 m.

Site investigation indicated that the tailings are loose with densities not varying in depth. No phreatic surface was encountered within the tailings mass and the gravimetric moisture content varied only slightly down the profile from around 10 to 20%.

The availability of the materials that could be used for the closure was a constraint. The owner had 3 types of materials at its disposal:

- Red and Brown Loams – comprised of different arrays of silty sands;
- Sand – fine grained and well sorted, with a lower amount of fines;
- Operational oversize – well graded gravel, but not enough in quantity;

The loams and sands were sampled and characterised during the investigation, while the oversize material was ruled out due to its scarcity.

3. MATERIAL CHARACTERIZATION

3.1 INDEX PROPERTIES

Tailings, Loams and Sands samples were taken to the laboratory to obtain their index and hydraulic properties. Figure 1 shows the Particle Size Distribution (PSD) results, which was used to guide the soil water retention testing.

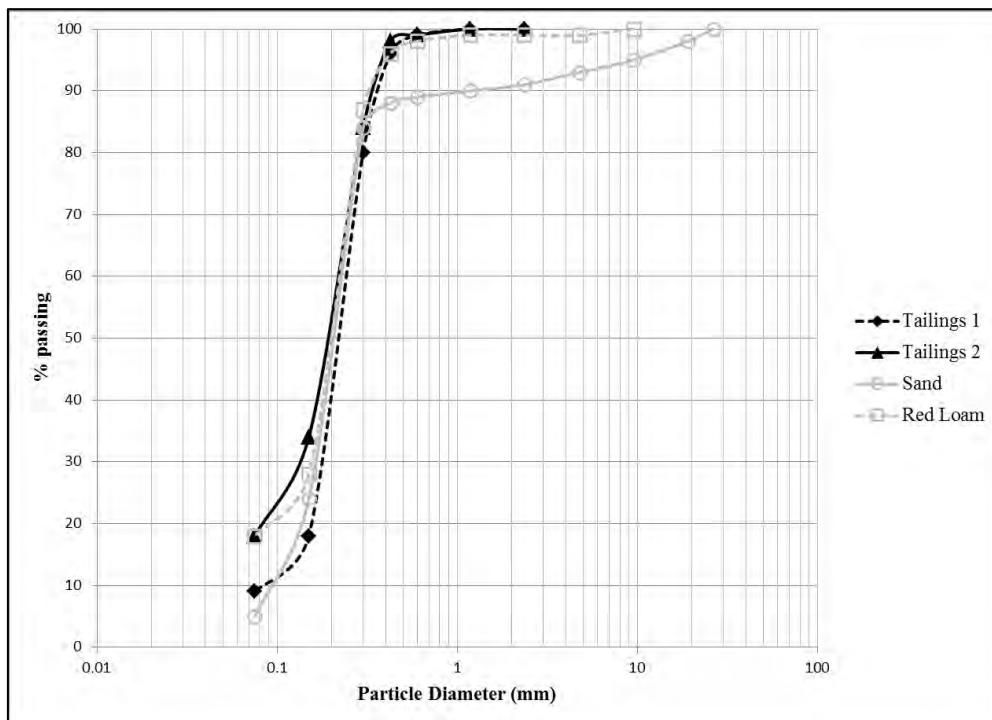


Figure 1: Materials PSD

In general, tailings samples taken from across the TSF profile and at different depths indicated that there is little variation in PSD across the tailings material. Results for cover candidates also presented a similar PSD. Materials were all poorly graded sands with different amounts of fine content.

The different materials were tested for saturated permeability and showed a value ranging from 2 to 5×10^{-5} m/s which is considered practically equivalent.

3.2 SWCC

The PSD results of the various materials were used as parameters to obtain first order approximations for the Soil Water Characteristic Curves (SWCC) by searching in a commercial soil database, which includes more than 6,200 related laboratory tests results entries. This database search yielded 61 samples with SWCC test data and similar characteristics. These results were used to direct the laboratory to concentrate the tests around the more likely Air Entry Value (AEV), which had a geometric average of 1 kPa in the search. The requested testing suctions were 0.5, 1, 10, 30, 200 and 1,500 kPa, but the laboratory didn't have the facilities to test suctions below 10 kPa.

Figure 2 presents the lab results and fitted SWCC curves after Fredlund & Xing (1994). Results show that SWCC are similar across the materials tested. This was expected as their PSDs were fairly similar and permeability results were all within the same order of magnitude.

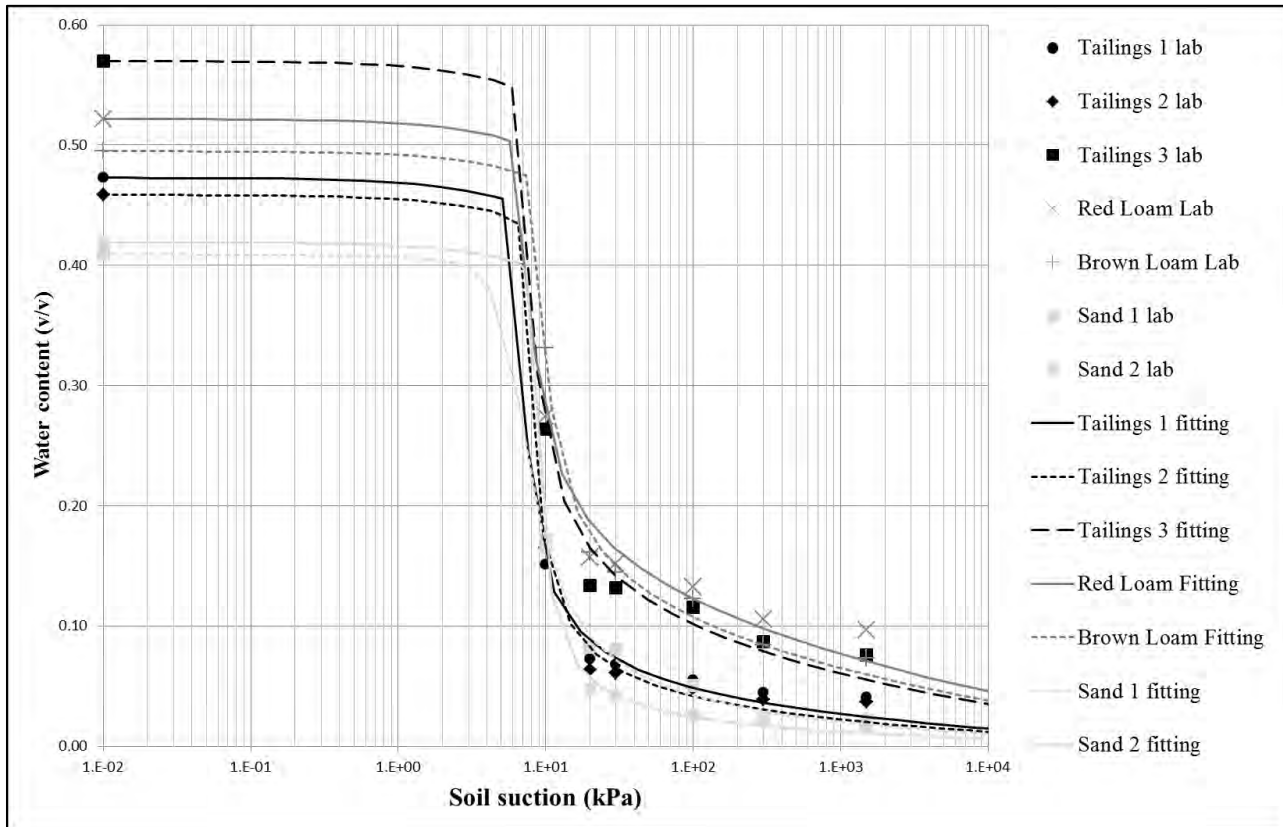


Figure 2: SWCC lab results and curve fitting

4. INFILTRATION MODEL

4.1 MODEL CHARACTERISTICS

A numerical, true surface, finite element, saturated and unsaturated infiltration model was prepared in order to describe moisture migration within the tailings. The commercial software SVFlux was utilised for this exercise. Two steps were identified as necessary to synthesize the closure process, pre-closure (Step 1) and post-closure (Step 2). Pre-closure predicts the conditions that the tailings mass will develop after deposition, but before rehabilitation, while post-closure predicts the conditions after a cover is installed with no capillary break. The hypothesis was that if there is no upward migration from saline tailings water into the vegetation growth layers, a capillary break was not needed.

Climatic boundary conditions comprised of daily precipitation and actual evaporation (AE), which was calculated by the model from fundamental thermodynamic principles using the modification of the Penman (1948) equation proposed by Wilson et al. (1994). Input data required for the calculation of AE included minimum and maximum air temperature, relative humidity, wind speed and solar radiation.

A 17 m thick 1D vertical model was set up for Step 1, mimicking the deepest tailings column. This case would be better isolated from the environment and would then represent the worst case scenario in terms of evaporative force per moisture storage capacity. In Step 2, a 1.5 m thick cover was placed on top of the tailings column composed of a mix of the materials defined as “Loams” following the owner's directions.

The initial conditions in Step 1 were a set of moisture contents in depth obtained from the site investigation, while in Step 2, the moisture content for the tailings were taken across from the final moisture content profile from the pre-rehabilitation scenario (Step 1) after reaching stable conditions (residual moisture content). Initial conditions for the placed cover materials have been considered as a gravimetric moisture content of approximately 15%, a value slightly higher than the typical moisture content used for construction of granular covers.

As indicated before, a true surface boundary condition was applied at the top of the column. The model was set up with a conditional boundary allowing for free gravitational flow at the bottom for the period immediately after cessation of tailings deposition. After equilibrium of the excess pore water pressure was achieved, the boundary condition at the base was set to zero flux.

4.2 MODEL RESULTS

During Step 1, the column shows a progressive drying up to reaching residual moisture content through the whole profile at around 2 years, as shown in Figure 3. After this, the model shows sensitivity to storms down the profile, but only with minor increases in moisture content that are evaporated within days.

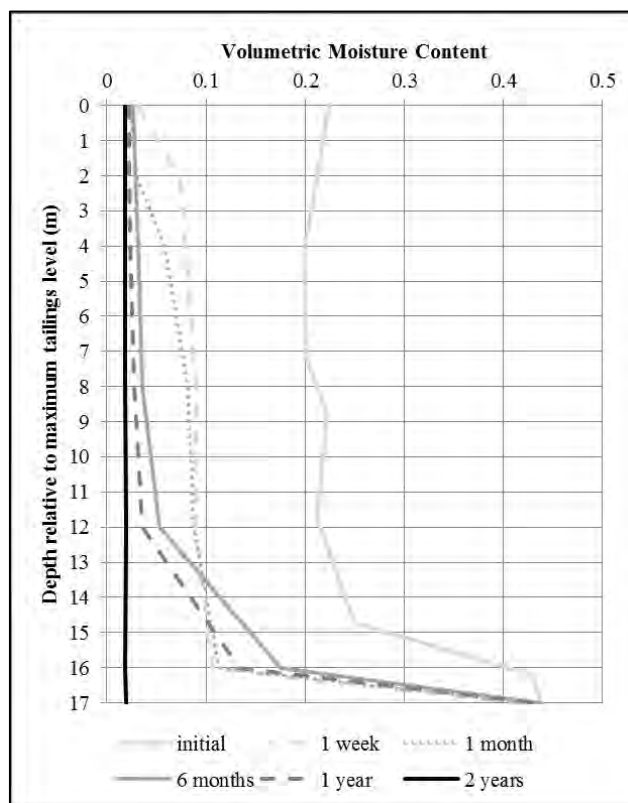


Figure 3: Pre-Closure moisture content variation

For Step 2, a 1.5 m thick cover was installed. Results of running this model for 10 years is presented in Figure 4, which shows variations in moisture content in time and depth. After the cover is installed, almost all climatic moisture exchange is produced within the cover.

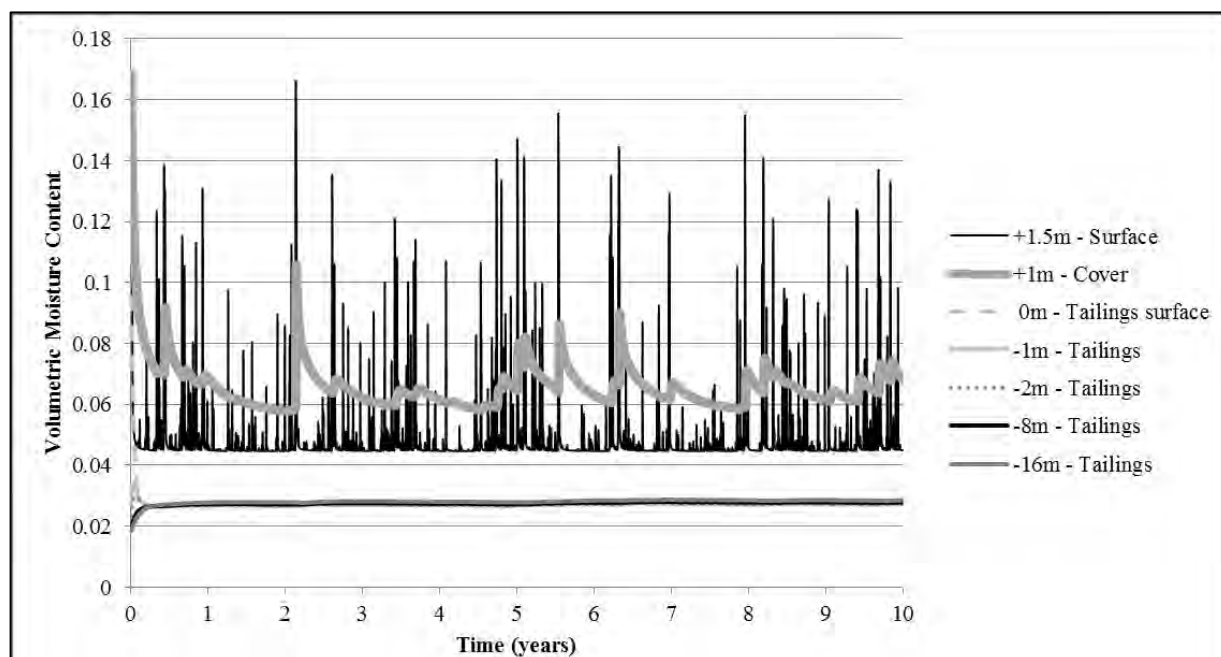


Figure 4: Post-Closure moisture content variation

During Step 2, the system's net flux fluctuates around zero, with all precipitation evaporating within days. The direction and size of any moisture migration is of particular interest to the problem being studied. In this regard, particles at different depths within the tailings mass and cover were tracked in time to observe their location. Figure 5 shows the position of the tracked particles after the first year of transient behaviour post cover installation. It can be seen that the greater variability is produced within the cover, while the movement of particles in the tailings would be less than 0.5 mm.

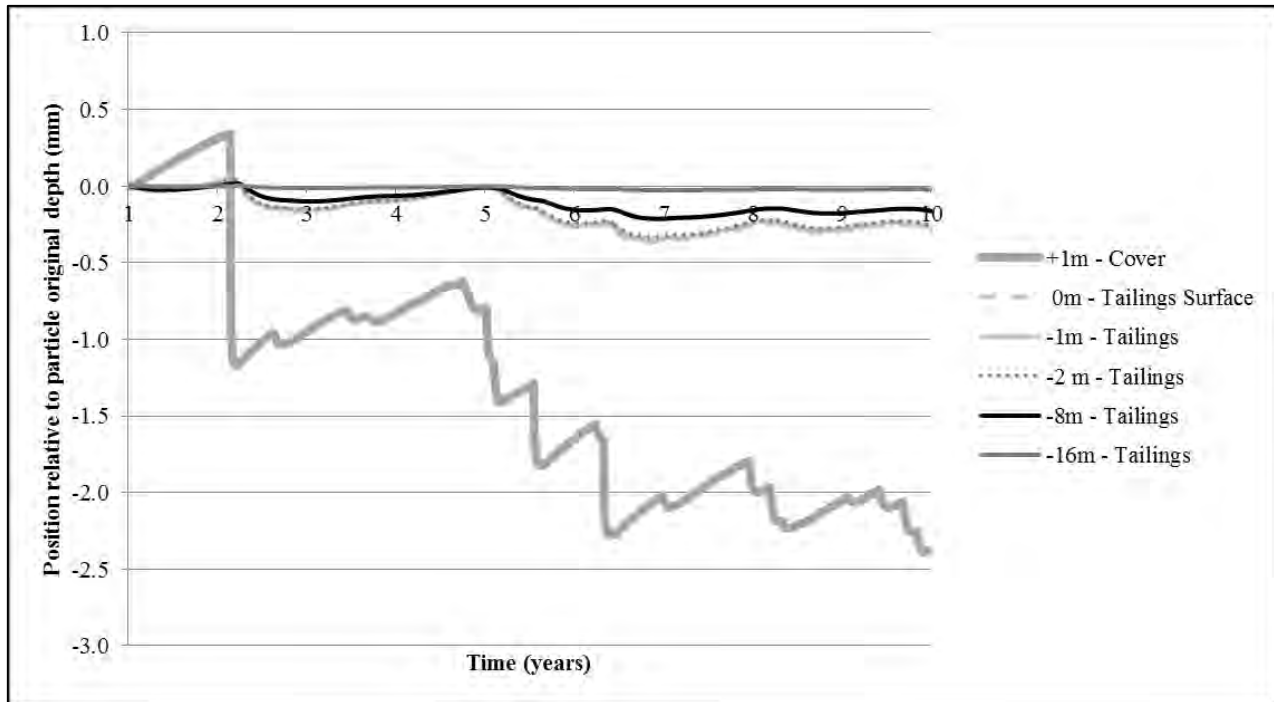


Figure 5: Post-Closure tracking of particles

5. CONCLUSIONS

An exercise at assessing the need for a capillary break when placing a closure cover was performed. The necessity of the capillary break was ruled out based on the modeling results that indicate no upwards flux from the tailings into the cover after closure. The following are a list of the main conclusions from this study:

- Physical and hydraulic testing has confirmed that the tailings, cover and capillary break materials are similar in nature.
- The candidate capillary break material is not sufficiently different to expect it to act as a capillary break between the tailings and available cover material; therefore, if a capillary break is required an alternate source should be identified.
- Numerical modelling suggest that once tailings deposition ceases the tailings mass will continue to drain for up to two years after which it will be in a state of equilibrium with the moisture content profile close to residual moisture.
- Placing a cover does not change this behaviour. The model indicated that the moisture that enters through the top climatic boundary will remain within the cover and there is no net upward flux from the tailings into the cover.
- A capillary break would not be required since the model indicates that the pore water from the tailings is quickly drained, with a trend of reaching residual moisture in the order of two years, while water entering the system through rainfall fluctuates within the cover depth.
- Sensitivity analyses should be carried out to assess the model results in case the cover materials vary largely in their hydraulic properties.

6. ACKNOWLEDGMENT

The author would like to thank Mr. Juan José Moreno (SRK Consulting) for his valuable comments.

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POST-EARTHQUAKE ROCK FALL ASSESSMENT IN THE PORT HILLS, CHRISTCHURCH, NEW ZEALAND

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ABSTRACT

On the 22 February 2011 a magnitude 6.3 earthquake struck the city of Christchurch, New Zealand causing extensive rock fall damage in the Port Hills area south of the city.

Following the February earthquake emergency protection measures were put in place with extensive scaling of rock bluffs, installation of rock bolt and mesh arrangements, and placement of debris barriers to stabilise rock fall areas and provide protection to lifeline infrastructure.

After the emergency stabilisation works were completed, detailed mapping, rock fall assessments and a review of aerial photography led to the development of permanent remedial solutions. On June 13 2011 a second magnitude 6.3 earthquake struck Christchurch, this provided a unique opportunity to compare damage caused by rock fall prior to and after this second M6.3 event.

This paper presents the results of extensive field mapping, aerial photograph review, risk assessments, and rock fall remediation. The paper continues by discussing the performance of temporary and permanent remedial works and un-remediated slopes prior to and after the June earthquake and discussing the implications of rock fall hazards to lifeline infrastructure and property. It concludes by exploring options for the management of rock fall hazards and the implementation of emergency rock fall protection measures.

1 INTRODUCTION

On the 22 February 2011 a magnitude 6.3 earthquake struck the city of Christchurch, New Zealand causing extensive rock fall damage in the Port Hills area south of the city. The epicentre was located approximately 10km from the centre of Christchurch City and at a depth of 5km. Peak ground acceleration of 2.16g was recorded at Heathcote Valley Primary School, some 2 km from the epicentre (GNS Science and Earthquake Commission, 2011).

The earthquake and resulting aftershocks caused widespread and extensive damage to the central Canterbury region; including the Port Hills to the south of the city, where significant rock falls occurred on the western slopes of the Sumner valley, above Wakefield Avenue.

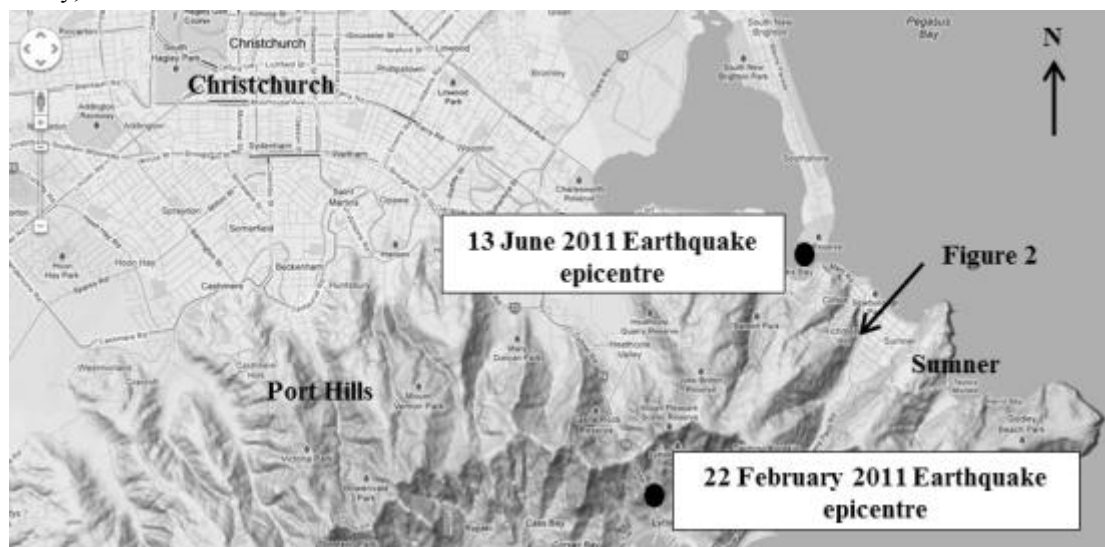


Figure 1, Christchurch map and approximate location of earthquake epicentres (Google Maps).

Wakefield Avenue is a lifeline route to Sumner, Christchurch therefore and provides an access to Sumner for emergency services and critical supplies in the event of a potential future earthquake. The slope above Wakefield Avenue was divided into two work areas, Wakefield Stage 1 and Wakefield Stage 2. These slopes were heavily affected by rock fall and posed a risk to Wakefield Avenue and residential properties further to the south.

1.1 GEOLOGY

The rocks forming the 400-500 m high ridge, slopes, and sea cliffs on the northern side of Lyttelton Harbour (in the Port Hills, Summit Road, Sumner and Redcliffs areas) belong to the *Lyttelton Volcanics Group* rocks of late Tertiary (Miocene) age, and are about 10–12 million years old (Sewell et al., 1992). These volcanic rocks comprise layers of hard, jointed basaltic and trachytic lavas with numerous dykes, interbedded with breccia, agglomerate (coarse angular gravel) and compact sandy tuff beds from the remnants of two large and overlapping Miocene volcanoes – Lyttelton in the northwest and Akaroa in the southeast – as well as several minor volcanic centres. The outer flanks of the volcanoes are dissected by deep radial river valleys which have existed for millions of years (Forsyth et al., 2008).

The volcanic rocks in the Lyttelton area are relatively strong, and the interbedded scoria and tuff layers are softer but compact. Most lava flows in the Lyttelton area are closely and irregularly jointed, resulting in a blocky rock mass that periodically releases individual or multiple blocks of rock that roll down the face and accumulate as a talus deposit at the base of slopes.

The steep coastal cliffs around Lyttelton, and extend further inland in the suburbs Sumner and Redcliffs areas are remnants of an old (~12,000 years B.P.) sea cliff. These near vertical (~75–85°) cliffs are, and are typically 15 to 30 m high and up to ~70 m high in some places. The old sea cliff has been modified by quarrying in several areas, including the major rock quarries located at Redcliffs and Sumner (Brown and Weeber, 1992).



Figure 2, Western slope of Sumner valley. The slope is characterised by the prominent basaltic and trachytic lava flows. Captain Thomas walking track can be seen in the lower left foreground (Camilla Gibbons).

2 RECORDED ROCKFALL

An initial aerial survey of the Port Hills was conducted via helicopter immediately following the 22 February 2012 Christchurch Earthquake. From this survey it was recognized that significant rock falls had occurred and that there remained significant rock fall hazards which posed a high risk to properties and critical infrastructure below.

Following the aerial survey a ground investigation was carried out to individually identify rock fall hazards and provide a risk assessment of potential future rock fall. The initial ground investigation began in the work area known as Wakefield Stage 1 with Wakefield Stage 2 commencing soon after.



Figure 3, Typical slope profile of Sumner Valley. The source bluff is located near the crest of the slope to the right with Wakefield Avenue and Sumner at the toe of the slope. (Leon Gerrard)

2.1 WAKEFIELD STAGE 1

The Wakefield Stage 1 work area is located on the western slope of the Sumner valley immediately above Wakefield Avenue and approximately between the intersections with Arnold Street and Paisley Street. The typical slope profile for this area is characterized by a continuous 400m long and 5-6m high vertical bluff near the crest of a thinly vegetated approximately 250mslope.

The ground investigation within Wakefield Stage 1 was carried out between late March 2011 and mid May 2011. The investigation identified 50 sites where rock fall had occurred and stopped before reaching the toe of the slope but still posed a risk to Wakefield Avenue below. 49 sites were identified as being a potential rock fall source, both on the bluff and the slope, which showed movement as a result of the earthquake but remained in their pre-February 2011 earthquake footprint and still posed a risk to Wakefield Avenue. 87 sites were identified where rock fall that reached the toe of the slope and posed no immediate risk to Wakefield Avenue.

2.2 WAKEFIELD STAGE 2

The Wakefield Stage 2 work area is located south of and adjacent to Wakefield Stage 1, approximately between the intersection of Wakefield Avenue and Paisley Street, and Sumnervale Drive and Cascade Place. The typical slope profile for this area is characterized by two prominent bluffs; the upper bluff is continuous 100m long and 5-6m high vertical bluff near the crest of the slope, the lower bluff is a discontinuous 550m long and 10-15m vertical bluff approximately half way down the slope. The distance down- slope from the upper bluff to the toe is 350-400m, the lower bluff is approximately 100m below the upper bluff. The slope between the upper and lower bluffs is thinly vegetated, and below the lower bluff the slope is moderately to heavily vegetated.

The initial ground investigation within Wakefield Stage 2 began in late March 2011 and ran through to early June 2011. The investigation identified 38 sites where rock fall had occurred and stopped before reaching the toe of the slope but still posed a risk to properties below. 32 sites were identified as being a potential rock fall source, both on the bluff and the slope, which showed movement as a result of the earthquake but remained in their pre-February 2011 earthquake footprint and still posed a potential risk to Wakefield Avenue. 19 sites were identified where rock fall posed no immediate risk to properties below.

3 EMERGENCY PROTECTION MEASURES

Several rock fall protection measures were employed in the work areas of Wakefield Stage 1 and 2 as an emergency procedure to reduce the risk of potential future rock fall. As Wakefield Stage 1 affected the lifeline route below, this area was given priority to implement protection measures and reduce the risk. These included; rock scaling, rock blasting, rock anchors and cable tie backs, temporary catch fences and earth bunds. Figure 4 shows the typical emergency protection measures implemented in Wakefield Stage 1 and 2.

The most common technique implemented to reduce the risk of rock fall was rock scaling. However, this technique required some form of protection barrier placed at the toe of the slope to catch potential rock fall run out towards residential properties adjacent the eastern side of Wakefield Avenue. Shipping containers were placed along Wakefield Avenue as a means of protection and catching the earthquake induced and scaled rock fall. The containers remain in place as a temporary barrier to potential future rock fall.

Wakefield Stage 2 employed several protection measures to reduce the immediate risk to residential dwellings along Wakefield Avenue, Finnsarby Place and Sumnervale Drive. The most prominent was a 70m long temporary earth bund located in a property along Wakefield Avenue. However, implementing these protection measures was punctuated by the earthquakes which struck on 13 June 2011.



Figure 4, typical emergency protection measures carried out. (center) temporary catch fence, (clockwise from top left) rock anchor and cable tie back, temporary earth bund, rock scaling, and shipping containers as a temporary protection barrier (Leon Gerrard)

4 13 JUNE 2011 CHRISTCHURCH EARTHQUAKE

On 13 June 2011 at 14:20 NZST a magnitude 6.3 earthquake struck Christchurch, New Zealand. The epicenter was located approximately 10km to the east of the city center at a depth of 6km and approximately 1.5km from work areas Wakefield Stage 1 and 2. The peak ground acceleration, greater than 2g, was recorded at Scarborough Hill on the eastern slopes of the Sumner valley (GNS Science, 2011).



Figure 6, major rock fall 300m from the epicentre of the 13 June 2011 earthquake (Leon Gerrard)

The close proximity of Wakefield Avenue to the epicenter of the 13 June 2011 earthquake offered the unique opportunity to evaluate the performance of the emergency protection measures put in place on the slopes and compare them to nearby slopes where no protection measures were put in place. A site walk-over was carried out on 14 June 2011 to re-assess the slopes within Wakefield Stage 1 and 2 to identify any potential rock fall hazards that may pose a risk to the residential properties and Wakefield Avenue below.

4.1 WAKEFIELD STAGE 1 PERFORMANCE

Some rock fall may have occurred following the June earthquake however, no additional sites were identified that posed an immediate risk to Wakefield Avenue below which required emergency protection measures put in place.

4.2 WAKEFIELD STAGE 2 PERFORMANCE

Before the 13 June 2011 earthquake struck approximately 50% of the area within Wakefield Stage 2 had been covered in the ground investigation. In conjunction with the ground investigation a temporary earth bund was constructed in a property along Wakefield Avenue, near the toe of the slope, as a means of protection and catching potential earthquake induced and scaled rock fall. Neither of the two major bluffs had any rock scaling work carried out, only one small isolated site was completed before the 13 June 2011 earthquake.

A site walk-over was conducted along the length of Wakefield Stage 2 on the lower slopes, and along the upper continuous 100m long bluff. The lower discontinuous bluff was not inspected during the site walk-over due to the unknown nature of the bluff.

The site walk-over identified 17 sites where rock fall may have occurred as a result of the June earthquake and a potential risk remained to the properties below. Three small boulders were identified up against the temporary earth bund. The more detailed ground investigation was continued following the site walk-over. This identified 17 sites as being a potential rock fall source both on the bluff and the slope but posed a potential risk to the properties below.

5 DISCUSSION

The scale of rock fall following the Christchurch Earthquakes was unlike anything experienced in Christchurch since European settlement (Hancox, et al., 2002). The idea of carrying out emergency protection measures on the Port Hills was to reduce the level of risk along lifeline routes so that populated areas may not be cut off and isolated from emergency services and critical supplies. Wakefield Avenue is a lifeline route to the suburb of Sumner and needed to remain accessible in the event of potential future earthquakes.

The ground investigations carried out in Wakefield Stage 1 and 2 identified a large number of sites where rock fall had occurred. The rock fall contained individual boulders varying in size from 0.1m³ (cubic meters) up to 10m³ and varying in shape from angular to rounded. A conservative approach was taken when remediating the rock fall sites. This meant the majority of sites within Wakefield Stage 2 were not treated before the June earthquake struck. However, this event did present the opportunity to compare the slope performance of Wakefield Stage 1 following the emergency works to the slope performance of Stage 2 where minimal protection work was carried out and improve the efficiency of carrying out the work. In retrospect, a greater area would have been covered if the focus was more on identifying the higher risk

and larger rock fall sites which would allow more time to construct temporary protection barriers. This may then provide adequate protection for properties and critical infrastructure until a permanent solution is constructed.



Figure 7, a large unstable boulder approximately 1.5m in diameter held in place by steel mesh and cables (Leon Gerrard)

6 CONCLUSIONS

The magnitude 6.3 earthquake which struck south east of Christchurch City on 22 February 2011 caused significant rock falls across the Port Hills to the south of the city center. The aim of implementing emergency protection measures on the slopes above Wakefield Avenue was to reduce the risk so as to remain accessible in the event of a potential future earthquake. The most significant outcome from comparing the performance of the slopes above Wakefield Avenue was that the protection measures were well implemented and significantly reduced the risk of potential future rock fall.

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A CASE STUDY OF GEOTECHNICAL DAMAGE TO TWO ROAD EMBANKMENTS FOLLOWING THE JANUARY 2011 QUEENSLAND FLOODS

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ABSTRACT

The heavy rainfall and widespread flooding which occurred in South East Queensland during December 2010 and early January 2011 affected a significant proportion of the road transport network and associated repair work is underway. Typical styles of geotechnical damage include shallow and deep seated failures of road embankments, failure of soil and weathered rock cut slopes, erosion of culverts and drains, erosion and drainage related damage to bridge spill – through embankments and development / reactivation of landslides of varying scale.

This paper presents two case studies of embankment failures which took place during the floods and discusses the damage to the road, rainfall prior to and during failure, failure mechanisms and contributing factors.

Keywords: Queensland 2011 floods, TNRP, slope stability, road embankment, cut slope, failure mechanism

1. INTRODUCTION

Intense and prolonged heavy rainfall during late December 2010 and early January 2011 across Eastern Queensland resulted in flooding of an area larger than France and Germany combined, affecting approximately 70 towns and 200,000 people. In South East Queensland, December 2010 was particularly wet with rainfall data showing some Queensland towns had more than six times the rain usually expected in December. The above average rainfalls experienced in the area meant that many catchments became saturated. When further record rainfalls were experienced in January 2011, already soaked catchments could not absorb the excess rain and in turn, flooding was experienced across the state. Some of this flooding was caused when river systems overflowed; some was the result of short-period falls where considerable amounts of rain fell on small areas within short time frames causing flash flooding such as in Toowoomba and the surrounding region on the afternoon of 10th January 2011 (Queensland Floods Commission of Inquiry, 2011).

The road transport network across the state was hit particularly hard; out of the overall transport network length of about 33,000 km across eastern Queensland, approximately 6,700 km was affected with 85% being from pavement damage, 10% due to slope instability and the final 5% from damage to bridges, culverts and floodways (TNRP internal newsletter).

In early 2011, the Queensland Department of Transport and Main Roads (TMR) commissioned AECOM to undertake the Engineering, Procurement, Construction and Management (EPCM) of the flood reconstruction works in Brisbane Metropolitan and Darling Downs regions. Funding of the project was provided by the Natural Disaster Relief and Recovery Arrangements (NDRRA) under the Transport Network Reconstruction Program (TNRP).

More than 150 sites were identified and inspected in the Darling Downs and Metropolitan regions. Typical styles of geotechnical damage included shallow and deep seated failures of road embankments, failure of soil and weathered rock cut slopes, erosion of culverts and drains, erosion and drainage related damage to bridge spill – through embankments and development / reactivation of landslides of varying scale. The majority of the failures occurred above the road due to erosion of marginally stable road cuts that consist of various types of soil such as colluvial and residual soils and reactivation of existing failures.

The following sections present case studies of two road embankments that were adversely affected by the extreme rainfall. We describe and discuss the failure that occurred, the rainfall before and on the day of failure, the damage mechanism and other contributing factors. Rainfall measurements obtained by automatic rain gauges managed by the Bureau of Meteorology (BoM) were used. The precipitation data was available in terms of rainfall cumulative values per day and accessed through their website (www.bom.gov.au).

2. SITE A: GATTON AREA (BRISBANE METRO REGION)

The road section at this location is approximately 330 m long and has been formed as a sideling cut and fill, with the material cut from the hill slope used to build up the required formation width of the road. The terrain above the road

comprises undeveloped sloping farmland. Two natural drainage channels (erosional gullies) direct hillside stormwater onto this site and two batter drains have been constructed in the road cut slope, directing stormwater under the road through two culverts. The culvert outlets discharge onto the slope below the road.

The fill below the road at various locations is partly supported by either a stacked masonry stone block wall or by a 2 m high gabion wall constructed adjacent to and below the shoulder. A creek flows parallel with the toe of the natural slope approximately 20 m below the road level. The pre-failure fill batter slope is inferred to have been steeper than 40 degrees (1V:1.2H).

A key geotechnical feature at this site is the presence of a 2 m to 4 m thick layer of colluvium which exists as a mantle across the broader hillside slope (Figure 1). The colluvium contains gravel to boulder size rock in a sandy silty matrix and is responsible for a number of active landslides in the surrounding terrain. The colluvium on slopes is subject to shallow slips triggered by saturation and erosion during heavy rainfall events. Underlying the colluvium is a deeply weathered sedimentary rock with a variable weathering profile. The weathered rock comprises extremely to highly weathered sandstone, occasionally interbedded with bands of moderately weathered mudstone. The weathered rock was found to be highly erodible and potentially dispersive.

It is understood that the original road was constructed at a lower level and the road was subsequently raised and widened in January 1988. Tension cracks and broader failures of the shoulder started to develop in April 1988 shortly after completion of the road widening works. Subsequent geotechnical investigation concluded that the newly widened section of the road had been founded on colluvium rather than bedrock. The shoulder of the road was stabilised by constructing gabion walls across the damaged sections.

On Monday 10th January 2011, a part rotational / translational slip occurred below the fill side lane. The slip extended into the traffic lane, reducing traffic to one lane under permanent traffic conditions. The slip has affected the road shoulder for a distance of about 30 m (Figures 2, 3 and 4). The inlets of the culverts were blocked by eroded weathered rock and colluvium from the cut slope.

2.1 RAINFALL DATA

Rainfall data obtained from four stations located within 4 km of the site are available. During December 2010, the area experienced prolonged and intense rainfall with cumulative values ranging between 350 mm and 450 mm, exceeding 3.3 to 4 times the December average. From 1st January to 12th January 2011, the cumulative rainfall exceeded 335 mm over the 12 day period, with about 200 mm from 10th January to 12th January only. Rainfall on the day of failure was measured to be between 42 and 50 mm.

2.2 FAILURE MECHANISM

During the rainfall events in December 2010, the culverts were obstructed, or possibly blocked, by eroded material. The reduced capacity of the culverts diverted the storm water collected from the hillside onto the road and table drains. The superelevation of the road is measured to be about 2 degrees toward the fill side of the road. Therefore, the majority of the water on the road was directed across the road. In the absence of table drains along the road and batter drains on the embankment slope, the embankment shoulder was subjected to erosion.

A damaged kerb was observed in the photos taken prior to the floods. There is a possibility that surface runoff was discharging directly onto the embankment developing localised eroded areas on the embankment.

The uncontrolled discharge from the two stormwater culverts, directly onto the slope, may have also contributed to the erosion and undercutting of the slope. Moreover, one of the culverts was dislocated at a section underneath the road, potentially allowing water to feed into the embankment over time. The presence of a colluvium layer and various fill layers underneath the widened section of the road has formed a potential slip surface along the colluvium / fill interface providing an easy path for the infiltrated rainwater to flow and compromise the stability of the shoulder. Figure 5 shows a sketch of the road cross-section and possible failure mechanisms.

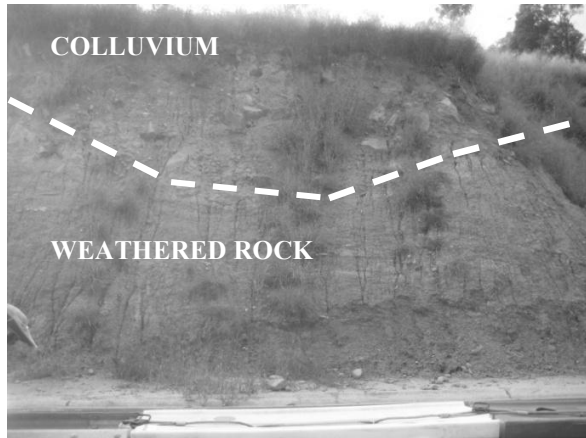


Figure 1. Irregular boundary between colluvium and weathered rock exposed in road cut



Figure 2. View of failure from crest of roadcut



Figure 3. Failure extended into the traffic lane. Note the culvert outlet discharging water onto the slope



Figure 4. View of failure from toe of slope adjacent to creek

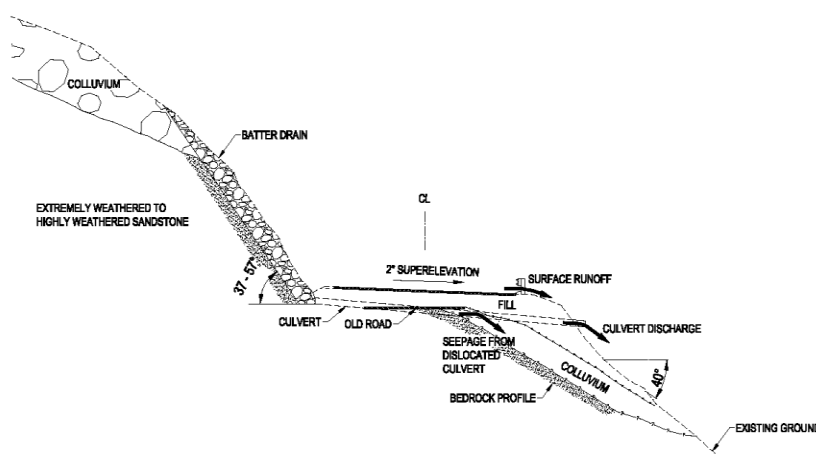


Figure 5. Cross-section of the road and possible failure mechanisms

2.3 LESSONS LEARNED

This case study highlights the importance of an engineered stormwater control drainage system as well as the need for regular maintenance and inspection of the drainage system, particularly culverts, after heavy rainfall events. Moreover, it shows the requirements for adequate subgrade preparation and fill placement when widening the cut and fill roads. Limited budget for construction and narrow width of road reserve on the cut side of cut-fill roads often favours road widening with larger fill embankments. However, due to construction difficulties unsuitable materials (such as colluvium in this case) are often left in place in older rural roads resulting in developing a potential slip surface between the original slope and new fill. Road widening towards the embankment side of road forms over-steep fill batter slopes and prevents adequate levels of compaction for underlying layers due to construction difficulties. Not removing unsuitable material also means that the original slope is not benched.

3. SITE B: ESK AREA (DARLING DOWNS REGION)

This road has been constructed on an embankment located in a natural broad gully. The embankment forms a sag curve over a curved alignment. The road has a superelevation of 2 degrees towards the downstream. The original embankment height ranged up to 10 m with slope batter angles as steep as 1V:1.5H. A cast in-situ arch culvert 2 m high with concrete headwalls and wingwalls was constructed underneath the embankment. The road alignment traverses through cut profiles at the start and end of the embankment. The exposed geology in the cuttings comprises basalt colluvium overlying extremely weathered to highly weathered basalt. The excavated materials from cuts were used in construction of the road embankment. The material from the cut comprises silty clay of high plasticity and was found to be highly erodible, highly friable and non-dispersive. It is understood that the culvert and embankment were constructed in the early 1940's. Two small dams are located upstream of the road embankment, being used for farming purposes.

On Monday 10th January 2011 a failure occurred in the downstream embankment shoulder (Figure 5). It is postulated that the failure was initially localised around the outlet of the culvert but developed retrogressively into the embankment. Following loss of the embankment toe, broader failures (slumps, dropouts, scours, sloughs etc) developed in the embankment crest, adversely affecting the full width of the road.

3.1 RAINFALL DATA

Rainfall data obtained from three stations located within 10 to 16 km of the site was available. However, December 2010 rainfall data was available for only one of the rain gauges. During December 2010, the area experienced prolonged rainfall with cumulative values of 325 mm and exceeding 2.5 times the December average. From 1st January to 11th January 2011, the cumulative rainfall ranged between 335 mm and 400 mm over the 11 day period. Rainfall of about 120 mm to 136 mm and 150 mm to 167 mm was measured on 10th and 11th January 2011, respectively. The historical average January rainfall of the three stations is approximately 120 mm.

3.2 FAILURE MECHANISM

Figures 5 to 10 present different stages of the embankment failure over a period of about 24 hours from 4 pm 10th January 2011. Photos were provided to the TNRP team by a local land owner living near the site. The times on the photos are indicative only, showing when the pictures were taken and not the time of failures. Site observations and review of post failure photographs and reports noted that the failure of the road embankment and culvert structure appeared to have occurred mainly due to a combination of the following two factors:

- Blockage of table drains: heavy rainfalls caused collapse of the road cut slopes located at the start and end of the embankment. Table drains were filled up with debris and the stormwater was pushed onto the road and over the embankment batter, eroding the embankment downstream slope. Figure 6 provides indications of the heavy stormwater flows and washed materials at the site.
- Upstream impoundment: due to obstruction of the culvert or inadequate capacity, water impounded behind the embankment resulting in high velocity stormwater flows at the culvert efflux and subsequent scour of the embankment toe. Failure of the two farm dams upstream of the road embankment may have contributed to increased ponding levels.

Upstream impoundment generally takes place by a combination of two different conditions (Polimo et al. 2011):

- Flash floods: the first condition can be defined as “flood event” and concerns the occurrence of either peak flood discharge in the drainage network or so-called secondary floods (stagnancy of rain on low permeability surfaces) as an effect of heavy rains. Such events can generally be identified as flash floods (local floods of high intensity and short duration)
- Inadequate culvert capacity: the second condition is represented by the “hydraulic weakness” of the transportation embankment, which is determined by drains, culverts, bridges and in general, hydraulic sections. These elements can be obstructed during a flood or may be under-designed from a hydraulic point of view for the specific flood event.

It is presumed that both of the above mentioned conditions have contributed to upstream impoundment taking place at this site.



Figure 6. Failure of embankment slope around culvert



Figure 7. Loss of embankment slope



Figure 8. Damage extended to the centre line



Figure 9. Heavy downstream scour



Figure 9. Failure extended into the adjacent lane

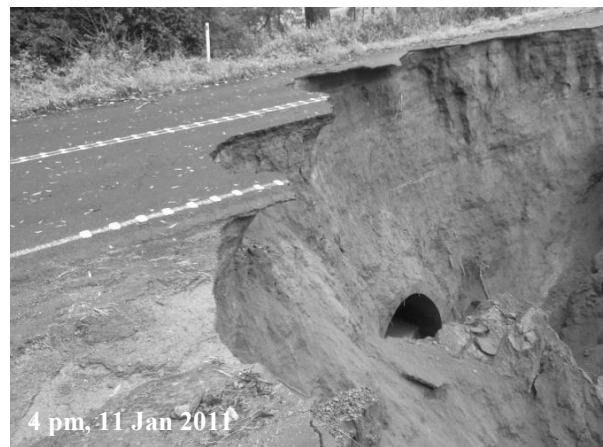


Figure 10. Complete loss of embankment

3.3 LESSONS LEARNED

This case study shows the importance of an adequately sized hydraulic system underneath the road. Developments in rural areas and change from forests to agricultural and farming lands can alter the hydrological model of the area. It is therefore necessary to re-assess the capacity of various components of drainage network systems of existing roads against recent and more accurate hydrological models.

The study also highlights the role of efficient table drains on the road and the importance of maintenance after heavy rainfall events, particularly in areas where adjacent cut slopes are prone to erosion.

4. CONCLUSIONS

The paper presents two typical road embankment failures that took place during the January 2011 heavy rainfall and flood events in the Brisbane Metropolitan and Darling Downs regions. The failure mechanisms, triggering rainfall events and contributing factors to the embankment failure have been discussed. The failures highlight the importance of the following aspects in road embankment construction:

- Adequate subgrade preparation and fill placement
- Avoiding over-steep batter slopes in design
- Properly engineered drainage system on the road, as well as adequately sized culverts underneath the road
- Regular maintenance of different elements of drainage systems, particularly table drains, in areas prone to erosion
- Protection of embankment slopes against erosion by maintaining adequate vegetation or other suitable methods

When inspecting heavy rainfall induced failures, it is also important to:

- Understand the flow path of water and how it contributed to the failure
- Inspect all the drainage network elements for obstruction / dislocation
- Approach locals to get first hand versions of events and photos
- Carry out sufficient geotechnical investigation to understand the ground conditions and minimise assumptions
- Look at the *bigger picture* to understand the terrain conditions and indications of what geological mechanisms are occurring
- Utilise published information such as pre-failure photos or aerial photos for indications of previous instability

5. ACKNOWLEDGEMENTS

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GEOTECHNICAL DESIGN FOR TEMPORARY WORKS

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ABSTRACT

Effective temporary works design is essential for successful project delivery. However, temporary works often present significant design challenges due to site constraints and interaction with the permanent works. These challenges offer the designer an opportunity to partake in a collaborative, risk assessment based design and construction approach with the civil contractor.

Many temporary structures can be designed with greater tolerance of risk, however, it is critical to convey this to all stakeholders throughout the design and construction process. Often for geotechnical structures, significant rewards are available through considering the extra dimension of time and the contractor may be able to accept these constraints to achieve their goals. Temporary works design must be reactive to site and project requirements. The designer has the opportunity to review the applicability of prescribed standards and adopt simplified design approaches to provide solutions in short timeframes.

This paper presents discussion on the design and communication processes. Also presented is a case history where the process was successfully implemented on a recent major civil infrastructure project in Brisbane, where the identification of common goals to arrive at practical, timely and cost effective solutions with a clear understanding of risk contributed to the successful outcome.

1. INTRODUCTION

Successful Temporary Works (TW) designs can often make the difference between a smooth running, successful project and a difficult, unsatisfactory outcome, as these aspects directly affect the programme and sequencing of the works. Despite being often considered as early stage project issues, geotechnical temporary works design is required throughout the entire life cycle of a construction project. The scope of work can vary from creating site access roads or foundations for site offices, right through to working platform design to allow removal of the site offices at completion.

TW are often seen as an afterthought; and in these circumstances there is limited opportunity to add value. However, in some cases, critical temporary structures are considered in detail during the planning and/or tender stage of the project, which can lead to rewarding and successful outcomes for the contractor through lower tender prices, potential for reduced contingency, programme savings and fewer surprises during construction.

Risk, and communication of risk in particular, are key components in any geotechnical TW design. Risk decisions should be a collaborative process between designer and contractor and supported by a well structured observational approach.

This paper explores a case history where a positive relationship between the TW designer and contractor was developed and successful outcomes were achieved.

2. TEMPORARY WORKS DESIGN PROCESSES

Construction processes are considered during tender stage by the bidding team regardless of the form of contract used for delivery. Typically, TW are rarely considered in detail by the permanent works designer as the construction process is not fully developed and the contractors are seen as being best placed to identify the need and scope based on their expertise and equipment. Figure 1 shows a typical process diagram for TW delivery. It details the steps and interactions between the parties which should be adopted to provide good opportunity for a cost and time effective solution to be developed.

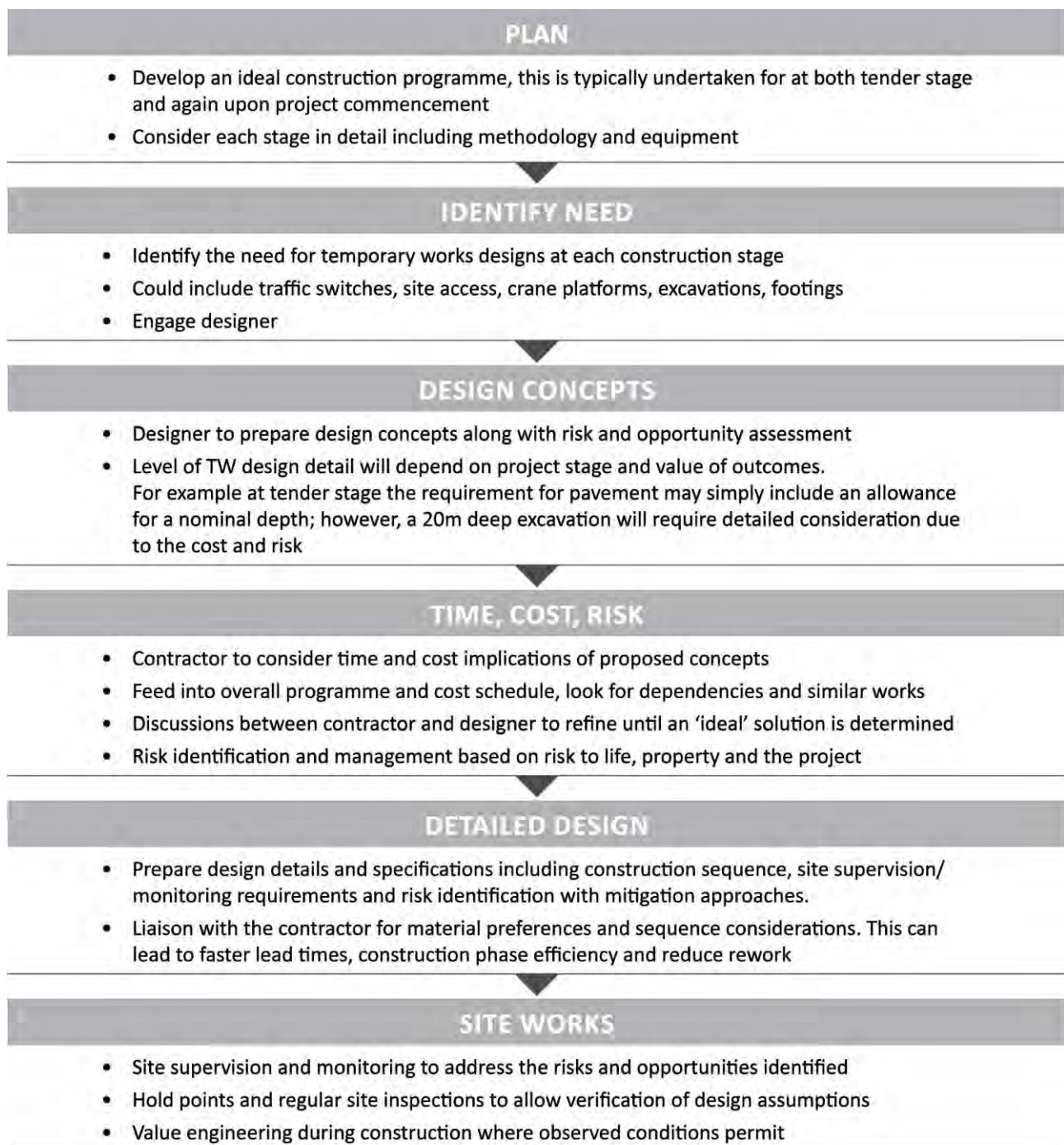


Figure 1: Process diagram for the development of temporary works solutions through the construction lifecycle

Frequently, this process is not followed due to the low priority typically given to temporary works designs throughout the construction lifecycle. This can lead to project time and costs overruns as the lack of planning leads to rushed or poorly planned solutions, or missed opportunities to achieve better than “routine” project outcomes. The key message is the collaboration between parties which allows risks to be managed and better design solutions to be prepared.

3. RISK AND OPPORTUNITY

TW designs offer the opportunity to depart from prescribed standards and design methods. This allows the designer to prepare more efficient designs in the short term, but increasing risk with time. This risk must be communicated to all parties and any limitations on the structure must be clearly defined. In particular, the consequences of the risks that are transferred to the construction team must be advised, along with appropriate mitigation and management measures.

The following are a number of areas where opportunities are available to the TW design team. Many of these are intrinsically linked to timescales from construction to decommissioning or time durations where personnel or equipment may be at risk.

REDUCED FACTORS OF SAFETY

A common approach is a reduction in the required factors of safety (FOS), either for global factors in working stress approaches or partial factors for limit state design methods. In geotechnical works, this is illustrated by the typical approach of adopting a working stress FOS of 1.3 for temporary highway embankment geometries against a FOS of 1.5 for the permanent case.

The selection and use of these FOS should be considered in detail. The purpose behind the FOS is to account for uncertainties in loading, input parameters and model geometry. However, if these uncertainties can be managed during construction, it may be appropriate to further reduce the target value. The designer must be aware of where conservatism has been used in the design and how readily construction can be validated. Knowledge of these accumulated “benefits” may give the designer confidence to accept slight reductions in the target FOS; for example a narrow cut required for a period of two days with a favourable weather forecast.

In some cases, an assessed FOS of 1.0 or less may be acceptable providing the risks are correctly managed. This could be considered on two levels.

- If all aspects of the design and construction are known, or can be controlled adequately, then theoretically a FOS of 1.0 may be appropriate; or
- Assessment of the consequences of the assessed hazard has been undertaken, and these consequences can be managed through alternative approaches to designing for higher FOS so that the residual hazard is acceptable. These could include exclusion zones, catch fences or monitoring systems.

REALISTIC SURCHARGES

Many prescribed standards and project specifications define minimum surcharge loads for permanent works design. These are typically applied in a plane strain model as a blanket pressure, typically in the range of 5kPa to 20kPa depending on the standard used, land use and slope of the ground. These surcharges are defined with consideration for future use of the site within the structures lifespan, which could be 100 years for permanent structures. They therefore allow for loads which are the maximum foreseen knowing that it cannot be readily controlled.

TW are different, as they are typically constructed and decommissioned within the lifecycle of the overall construction project. Therefore, the contractor will have direct control of the activities on site and of access into areas which may increase the load behind a slope or wall. In addition, the project lifecycle is limited, so the likelihood of incidental loads is less due to the shorter timeframe. As a result, the contractor may be willing to accept increased risk to achieve construction cost and programme savings. These risks are often mitigated through isolating the site (e.g. placing barriers, placing signage and defining exclusion zones).

Consideration should be given to those loads that will actually be applied on or at the crest of a slope or wall during the construction works. Sources of load would include live loads (e.g. crane loads) and dead loads (e.g. material storage). This decision is critical and must be made collaboratively during the design phase to avoid placing difficult restrictions on the construction team for minor design benefits.

DURABILITY

Structural inclusions in geotechnical systems include ground anchors, soil nails, rock bolts, sheet piles and reinforced concrete for various purposes. Each of these has material durability issues for permanent works which are related to the aggressivity of the ground; particularly sulphates, chlorides and salinity. Of particular concern are highly stressed, tensile components such as ground anchors as they can fail in an undesirable brittle fashion. For this reason, permanent ground anchors are typically required to have extensive corrosion protection systems. Durability concerns in reinforced concrete are typically addressed through concrete cover to reinforcement, galvanised steel, sulphate resistant cement and careful selection of aggregates.

The definition of what timeframe is considered acceptable for the materials to reduce corrosion protection requirements will vary on a case-by-case basis. Depending on the environment, the life of a structure where no specific corrosion protection could be allowed for may typically be in the range of 1 to 5 years. This is a field where significant research has been undertaken and each situation should be assessed on its own merit.

By considering each case specifically, it may be evident that designing for predicted corrosion rates by adopting increased bar diameter or plate thicknesses is a more cost effective option for TW than other systems such as protective coatings. By selecting a short design life at the outset, it is critical that sufficient contingency is built in so that if there are overruns in the construction program, it will not go beyond the selected life of the structure.

UNDRAINED STRENGTHS

For geotechnical TW, the use of undrained strengths instead of drained strength parameters in analyses can provide benefit in cases where it can be demonstrated that the construction timeframe is shorter than the time to dissipate the suction pressures.

As a general rule of thumb, undrained strengths are applicable to short duration temporary works. Their use will typically provide a beneficial design solution for cuttings due to these suction pressures. Over time, the pore pressures will dissipate and the soil will behave in a drained manner. For this reason, the observational approach is highly relevant to these situations as piezometers can be installed to monitor pore pressure changes with time and inclinometers, survey markers or settlement plates can be utilised to observe the settlement and lateral movement.

Adopting undrained strengths in the design of retaining walls and cut slopes can offer significant benefits to the contractor through cheaper designs which are quicker to build or even allowing for a 'do nothing' approach instead of building retaining walls. They do, however, carry increased risk. The FOS of a cut slope typically decreases as the pore pressures dissipate, so a wall designed using undrained strengths and a FOS of 1.3 is likely to have a FOS below this value over time. This risk must be communicated to, and accepted by, the contractor and appropriate mitigation measures adopted. The following points should be followed in these instances:

- Clear communication of timeframe restrictions (1 day, one week, one month etc; however, because often no information on rate of pore pressure dissipation is available, these are difficult to define and must be coupled with regular site observations)
- The slopes/walls should be regularly inspected for signs of distress including sudden movement, cracking behind and seepage.
- Monitoring systems should be considered including survey prisms, inclinometers, piezometers and settlement plates depending on the risk in the design.
- Consideration could be given to increasing confidence that undrained conditions will prevail by reducing the rate of moisture content change. This could be achieved by placing a shotcrete membrane.
- Consideration should be given to adopting higher FOS for total stress designs

The use of undrained strengths in design for temporary works is a reasonable approach, but must be carefully managed throughout the design process. It is critical that the decision is made holistically with consideration of all aspects of the basis of design.

ALLOWABLE DEFORMATIONS

Allowable deformations, or serviceability considerations, are presented in a number of design standards and guidance. Deformations are typically defined based on two key criteria:

- Deformation which limits damage to the structure itself
- Deformation which limits damage to adjacent infrastructure

As the contractor is on site during the works, it may be acceptable to cause some damage to adjacent works if it can be readily and economically repaired. Further, it may not be necessary to be concerned with impacts if they are contained within the construction site. Where there are risks of structural damage, these must be identified, communicated and appropriate measures put in place to monitor and mitigate the risk.

4. CASE STUDY – DEEP SOIL NAILED EXCAVATION

The site is in a highly constrained urban environment, surrounded on all sides by buildings and infrastructure with little available space between. Tunnels were proposed to pass under and through the walls of the excavation. The contractor required a near vertical excavation of 18m depth to enable construction of an arched slab for a tunnel roof, supported on a contiguous piled wall. The excavation was required to remain open for two years. The arched slab and piled walls formed part of the permanent works design.

Figure 2 shows the site boundary, excavation extents and key features. The contractor had an initial design which formed the permanent works, comprising top down construction using concrete slabs as props to contiguous pile walls. As an alternative, the contractor was keen to explore the suitability of soil nailing due to perceived programme savings.

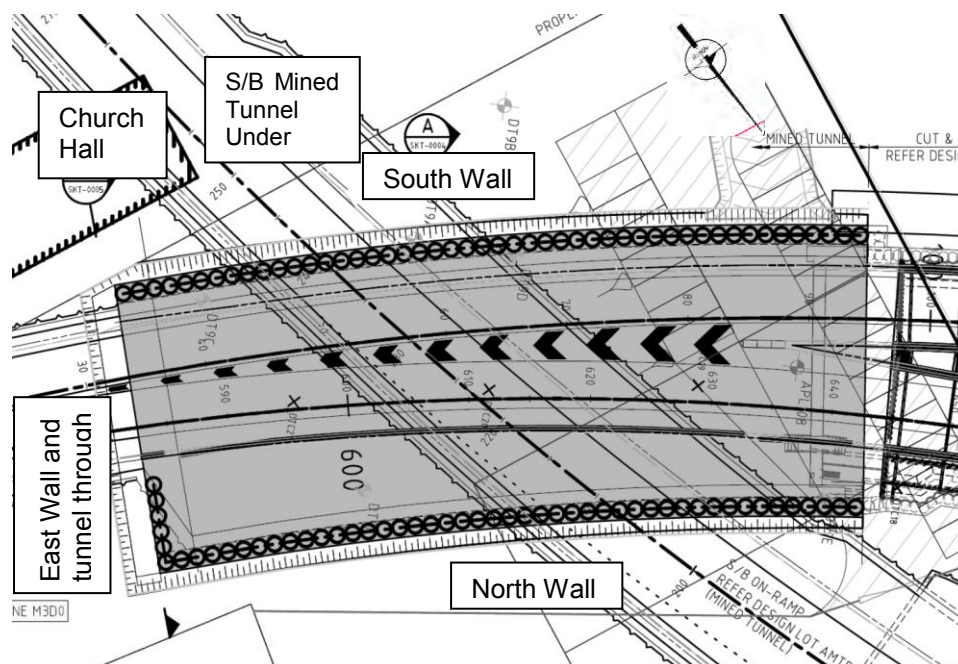


Figure 2: Project layout showing crest of excavation, alignment of surrounding tunnels, site boundary and contiguous wall layout

The following were key site constraints:

- Tunnel through the eastern wall and a tunnel (north-south) underneath the excavation. Further excavation below the toe of the wall
- Church hall 5m behind the crest of the wall, hotel behind the northern wall and a protected tree.
- Construction loading to deliver materials required to build the excavation and capping slab. This included concrete pumps, mobile cranes and a tower crane within 1m of the wall crest

A concept solution was quickly identified using simple limit equilibrium analysis to enable cost and programme to be assessed. This included a detailed register of risks and uncertainties with the solution which would need to be closed out during detailed design. These were primarily related to uncertainties due to the tunnels and the impact of likely significant deformations on adjacent structures. Through a process of clear communication and risk review, the contractor understood the risks and elected to develop the design further.

The detailed design phase required close coordination with the permanent works designer as amendments to the soil nail wall could impact on the forces in the permanent works below. This utilised more complex finite element analysis methods to predict deformations of the structure and adjacent properties.

Further consideration was also given to the design life and it was agreed that a two year period would be nominated which would allow soil nail bars without galvanised coatings or plastic sheaths as corrosion protection. This provided cost and time savings.

During the detailed design process, collaborative construction phase planning was undertaken to ensure the design catered for the range and combination of temporary construction cases. These cases included the order in which the tunnels were excavated; a temporary access ramp into the structure to improve access; and the applied loading from construction equipment. Design wall surcharges were restricted by project deed requirements for a blanket 10kPa however care was taken to prevent “doubling up” of surcharges for example where mobile cranes or concrete pumps were modelled, consideration was given as to whether any additional surcharge was possible between outriggers. The design FOS was taken as 1.3 although consideration was given to adopting a higher FOS due to the assessed project risk. It was agreed between parties that site supervision and monitoring systems would be used to increase confidence in the outcome rather than designing to a higher FOS.

This detailed planning reduced time delays during construction as the design had planned for each case in advance. A factor which assisted this process was the full time site presence by an engineer from the design team who could provide ongoing support to these decisions and observation of the structure. Some construction stages were modelled as undrained following judicious selection of constitutive models so a key responsibility of the site engineer was to control the time periods between excavation and completion of the wall lift. Monitoring was undertaken during the works to validate the design basis and the site engineer used these results to confirm that the structure was behaving in line with predictions. This included obtaining information from prisms, inclinometers and piezometers where were installed prior to excavation proceeding. The completed design incorporated typically 15m long soil nails with a reinforced shotcrete facing. Tunnelling through the wall was facilitated by using Glass Fibre Reinforced Plastic bars.

Figure 3 shows a photograph of the structure with surrounding site constraints after construction of the capping slab.



Figure 3: Site constraints shown in an aerial photograph taken during construction of the capping slab. Note the mobile crane, tower crane and church hall.

The works were completed successfully and within the programme requirements, the excavation has now been backfilled over the tunnel. This case history illustrates the stages identified in the process chart against a real situation and utilises some of the techniques identified to produce more effective temporary works designs.

5. CONCLUSIONS

This paper has described processes which could be followed to achieve good outcomes with respect to temporary works designs in construction projects. The process outlines the importance in early consideration of requirements and the benefits which are available at each stage. Opportunities for leaner design have been identified which typically mean a greater acceptance of risk by the contractor. Steps have been presented to assist with managing each of these risks, for each, it is the communication which is considered to be a critical component. It is critical that a thorough understanding of where decisions have been made which are considered to be conservative and that a holistic view of the overall risk is maintained to prevent potentially unnecessary “layering” of conservative assumptions.

A recent case history has been presented where the permanent works designs required complex and potentially high risk temporary works to allow construction to proceed. It has shown how the residual risk can be mitigated using observational approaches and specifically developed design methods to provide solutions within tight time and resource constraints. Early planning and identification of the requirements for temporary works enabled, in some cases permanent design modification to assist the overall project and provided construction program savings.

ANALYSIS OF BASEMENT EXCAVATION THROUGH ORGANIC SILT

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ABSTRACT

A 25 storey tower block in Quy Nhon City, Vietnam included a two level basement which required excavation up to 10.5m deep. A diaphragm wall with temporary props was proposed to support the basement excavation. This project was made particularly complex due to the low strength of the ground, particularly the presence of a layer of soft, compressible, organic silt located between 11m and 16m below ground level.

These ground conditions meant that the initial analysis (an 800mm thick wall – a solution used successfully elsewhere for similar sized excavations) resulted in the diaphragm wall's predicted deflections being well beyond the acceptable limits. To reduce the deflections the wall's stiffness was increased. This was achieved by increasing the thickness of the wall. However, the resulting maximum bending moment in the wall was so great that the cost of strengthening the wall sufficiently would make the solution impracticable.

Several alternative options were considered and the final preferred solution involved placing the temporary props at very specific depths and pre-stressing the bottom layer of props. The result was higher prop forces, reduced maximum bending moment in the wall, and the deflections reduced to acceptable levels.

1. INTRODUCTION

There are many different solutions available to manage the problem of supporting a basement excavation depending on various factors such as depth of excavation, site constraints, the properties of the subsurface soils and the available materials and technology. This paper presents the challenges faced in designing a retaining wall solution for a two level basement in Quy Nhon City, Vietnam.

2. SITE CONDITIONS

The site is approximately 3500m² with a proposed building footprint of 43m by 59m. The building is to be 25 storeys tall with a two level basement.

The ground conditions have been investigated by drilling four boreholes to 70m depth with Standard Penetration Tests (SPTs) at 1m intervals and an extensive suite of laboratory tests including Particle Size Distributions (PSDs), Atterberg tests, Oedometer tests, Consolidation tests, Shear Box tests, and various types of Triaxial tests. In total over 450 laboratory tests were carried out.

The site is typically underlain by 68m of cohesive and cohesionless soils overlying basalt rock. The ground water table is approximately 1m below ground level. For the purposes of basement retention design the properties of the upper 30m of soil were analysed and their properties are summarised in Table 1.

Of particular note is the presence of an organic silt layer between 11.0m and 15.5m below ground level. Test results indicate that the layer is very soft having an undrained shear strength (S_u) of 15kPa and modulus of elasticity (E_s) of 3,000kPa.

Layer	Material	Depth to Top of Layer	ϕ	c	S_u	E_s
1	Sand with some clay	0.0m	29.0°	4.0kPa	-	14,000kPa
2	Organic Silt	11.0m	-	-	15kPa	3,000kPa
3	Sand with some clay	15.5m	30.5°	3.5kPa	-	14,900kPa
4	Clayey Sand	20.0m	-	-	100kPa	21,600kPa
5	Clay/Silt with some sand	27.0m	-	-	150kPa	17,500kPa

Table 1: Soil Properties for Basement Retention Design (obtained from laboratory test data)

3. METHODOLOGY

To assess the forces and deflections of the basement retention system the computer programme WALLAP by Geosolve was used. WALLAP is designed specifically for the analysis of retaining walls, piles and retained excavations (e.g. basements). The programme allows analysis of forces and deflections at various stages during construction. All results presented in this paper refer to the stage where the deflections and internal forces in the wall are at their greatest - when the excavation is at its greatest depth (10.5m) and before the reinforced concrete floors have been constructed.

The soil properties used for the analysis (see Table 1) were determined by reviewing the results of the in-situ SPT tests and laboratory test results and a representative value for each layer has been selected. Layers with a fines content of >35% were assumed to behave in a cohesive fashion. The borehole logs show small variations in the depth and thickness of each layer.

4. PRELIMINARY ANALYSIS

The preliminary analysis was to determine the most appropriate form of basement retention. The options considered were:

- Sheet piles
- Diaphragm wall
- Secant pile wall
- Contiguous pile wall

By assuming a maximum acceptable deflection of 100mm the required stiffness (the product of modulus of elasticity (E) and second moment of inertia (I)) of the wall was determined as 1320MNm² per metre length of wall. This assumed two layers of temporary props installed to support the excavation. The required wall parameters to achieve this stiffness were assessed for each wall option.

Sheet piles were quickly dismissed as very large pile sections with welded UC columns would be required. In addition, the sheet piles would only be temporary (basement walls would be constructed within the excavation and the sheet piles subsequently removed) so this option was unlikely to prove economic.

A contiguous pile wall would require very large diameter piles. This would be technically achievable, however with a high water table constructability issues due to groundwater seepage, and possible 'squeezing' or 'running' of the ground, means that it is a high risk option.

A secant pile wall would eliminate the groundwater issues and should provide a water tight wall. However, the time to construct the required number of piles was unacceptable to the client and this option was dismissed.

The remaining option was a diaphragm wall. This has been used many times before in Vietnam for basement construction. By founding the wall 28m below ground level in a clay layer an effectively watertight excavation could be achieved and the wall could be designed to operate as the final basement walls once excavation was complete.

Note: The excavation depth at this stage of analysis was 8.95m. As architectural design proceeded, this was increased to 10.5m.

5. DESIGN CHALLENGES

The major design challenge in assessing a suitable basement retention system is keeping the deflections within acceptable limits (in this case 100mm maximum allowable deflection to limit damage to nearby buildings) while reducing the forces in the wall to manageable levels. The main reason why this problem was particularly challenging was due to the presence of the organic silt. This layer applied a large active force to the back of the wall and provided very little passive resistance at the front of the wall. The organic silt was located just 0.5m below the maximum excavation depth and extended for 4.5m. The effect of very weak ground immediately below the maximum excavated depth was similar to having the maximum excavation depth extended a further 5m, but without the option of providing temporary props.

This combination of high active force and low passive resistance meant that, in order to reduce deflections to an acceptable level, the wall itself is subjected to very large forces both in shear and bending.

6. FORCE VS. DISPLACEMENT

In order to reduce the forces within the wall, the effect of reducing the stiffness in the wall was assessed. Reducing the stiffness of the wall reduces the amount of load within the wall by shedding more of the load into the struts, and into soil below the wall which provides passive resistance. However, the result is that the wall deflects further. Excessive deflections can result in settlement at the surface behind the wall which may damage adjacent buildings and infrastructure, or severe disturbance to the soil on the passive side (aka base heave) which may eventually lead to loss of strength and passive failure of the retention system.

Shown in Figure 1 is a plot demonstrating how varying the stiffness of the diaphragm wall influences deflections and the level of bending moment in the wall. This plot was produced by conducting three WALLAP analyses varying only the thickness of the wall (i.e. varying the stiffness of the wall).

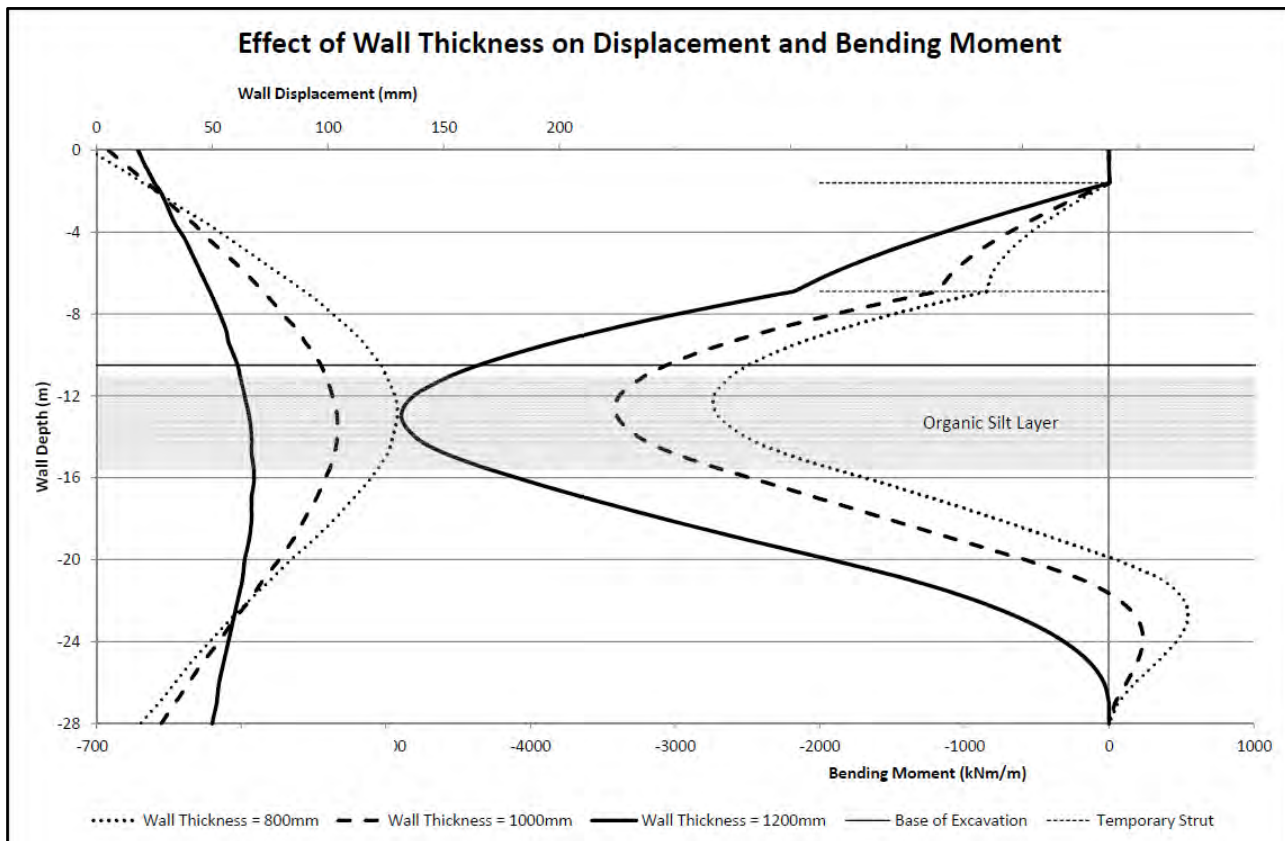


Figure 1: Effect of Diaphragm Wall Thickness on Displacement and Bending Moment

As shown in Figure 1, there is a balance that needs to be found between the level of deflection in the wall and the amount of bending moment induced in the wall. The problem faced was that the level of bending moment in the wall was unacceptably high for the 1000mm and 1200mm thick walls. However, the level of deflection of the 800mm thick wall was also unacceptable. The challenge was to find a way of reducing the bending moment to acceptable levels without increasing the level of deflection.

7. SOLUTION

In order to ensure that the basement excavation remained a practical option several possible solutions were assessed. These were:

- Additional props
- Prestressed props
- Base grouting

By adding additional props a larger proportion of the forces would be resisted by the props and a lesser amount by the wall. Hence the internal forces would be reduced and, by rearranging the props for maximum efficiency, the maximum deflections in the wall could be reduced. Additional props were the cheapest option as the equipment would already be on site and cost of installing additional props would be relatively small. Prestressing or 'jacking' the props would reduce wall deflections and bending moment but with higher prop forces.

However, the greatest forces and deflections occur within the problematic organic silt layer. The most beneficial location to add an additional layer of props would be near the centre of this layer. The idea behind basement grouting was to effectively create a strut within the organic silt by injecting grout from the surface in a horizontal sequence of 'bulbs'. From a technical perspective this was the most effective solution to the problem. However, the technology had not been used before in Vietnam and getting specialist contractors in would likely prove very expensive.

Shown below in Figure 2 is a plot demonstrating the effect of these proposed solutions on the deflections and the magnitude of bending moment in the wall.

The solution adopted was to add a third layer of props including prestressing the bottom layer. This decision was based on cost and time efficiency as well as risk as base grouting was a method unproven in Vietnam. Other solutions such as ground freezing were also considered and quickly dismissed due to the cost implications and the desire to use technology already proven in Vietnam.

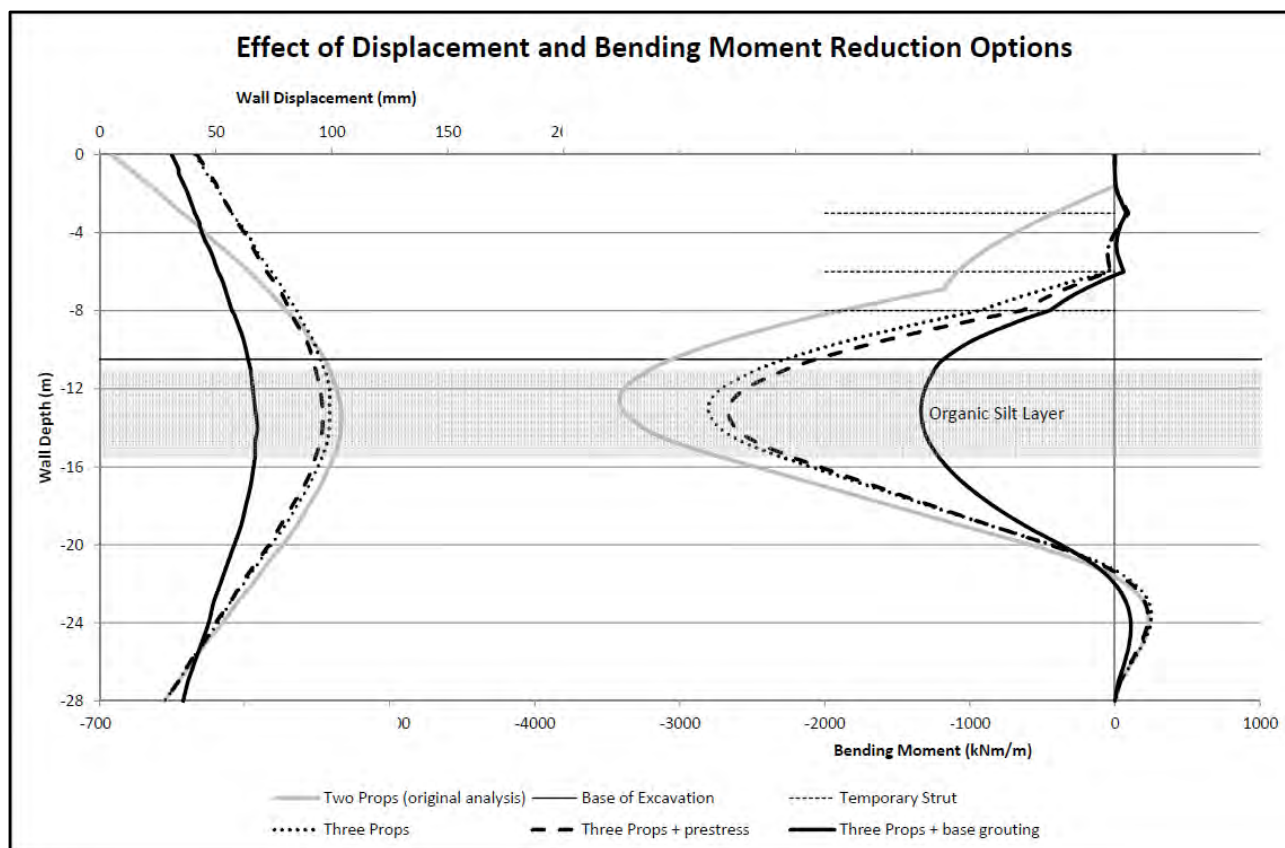


Figure 2: Effect of Displacement and Bending Moment Reduction Options

8. CONCLUSIONS

When designing a deep excavation, finding a balance between an acceptable level of deflection in the basement retention system and the level of load that the retention system must resist is a complex balancing act. Many factors must be considered to ensure that the level of deflection in the wall does not exceed safe limits or result in unacceptable levels of settlement behind the wall which may damage adjacent buildings and/or infrastructure. While increasing the stiffness of the wall is the simplest way to reduce deflections, this results in the level of load in the wall significantly increasing.

The placement of temporary props to help the wall resist the imposed loads is often utilized. This is a relatively cheap and effective solution which is commonly used during basement excavation. However, in weak soils temporary props alone are not always sufficient. Other methods such as grouting below the walls and/or strengthening the retained soils (e.g. by freezing) can be very effective but also very costly – sometimes to such an extent that it is more economic to reduce the depth of excavation. However, additional time spent during the design stage can often save a lot of unnecessary additional cost and build time. By analysing the problem to more efficiently distribute the load into the temporary props, and prestressing the temporary props to further reduce deflections and the level of load in the wall, a practical solution can often be obtained without significantly increasing the cost of the project.

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GEOTECHNICAL ASSESSMENT AND DESIGN OF A CANTILEVERED RETAINING WALL IN A SENSITIVE VOLCANIC SOIL

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ABSTRACT

This paper discusses geotechnical aspects of the design and construction of a large bored pile cantilever retaining wall in Hamilton, New Zealand.

Intrusive geotechnical investigations at the site identified sensitive volcanic soils at founding level and raised concerns of strain softening in the soil during an earthquake. This was considered likely to lead to unacceptable deformation of the wall and damage to surrounding properties in a seismic event.

As well as traditional techniques, the investigation included cyclic triaxial testing and seismic dilatometer testing (sDMT) of the sensitive soils. Information from other sites in similar materials was also gathered and used in the assessment.

Assessment of the testing results indicated that excessive deflection of the walls was a risk during a seismic event and that ground improvement was required to mitigate the risk to acceptable limits.

Deep soil/cement grout mixing of material in the passive zone was determined to be the most cost effective solution.

Observations from construction are discussed, sharing challenges faced due to the ground conditions.

1. INTRODUCTION

Wairere Drive (Hamilton, New Zealand) is being extended from Crosby Road in the north, eventually connecting the north east with Cambridge Road and SH1 (Cobham Drive) to the south. The project involved the construction of approximately 3.9km of new four lane carriageway and a new section of approximately 1.8km of two lane road adjacent to Dey Street through Hamilton East.

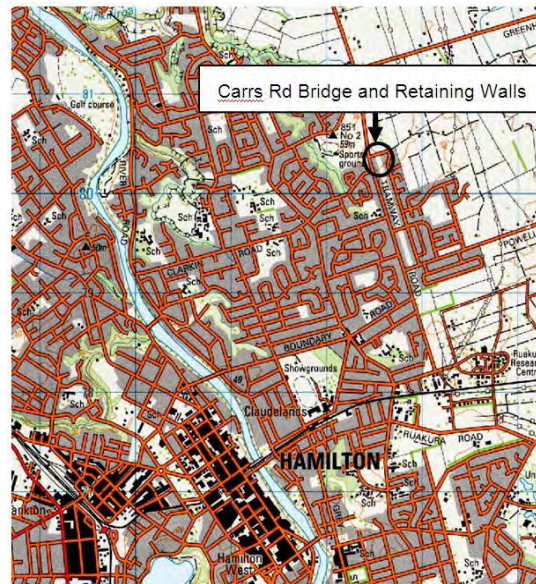


Figure 1. Location of Retaining Walls

The majority of the proposed alignment traverses flat to gently sloping terrain, with the exception of the northern portion of the alignment that bisects several low hills.

At the location of the retaining walls that are the subject of this paper, the geometric design of the road alignment called for an 8m deep cutting to allow for the new road to pass under the existing Carrs Road. The narrow designation and deep cut created either oversteep slopes or significant retaining walls. The location of the walls is shown on Figure 1.

Geotechnical investigation was scoped to develop a ground profile for the design of the bridge and the cut. The drillholes and CPT tests showed a significant layer of volcanic material (weathered unwelded ignimbrite of the Puketoka Formation, a soft silt and clay that can be sensitive) with very low in situ strength test results although it initially appeared to be competent in investigation pits. On working by hand, the material lost its initial structure and 'popped' releasing moisture and becoming paste-like. These test results and observations indicated that the material was potentially prone to softening when strained, highlighting a potential deformation issue during an earthquake.

A precedent study of cut slopes in the area showed that historically, cuttings had been made at very steep angles which degraded over time by frittering. Back analysis indicated that these slopes had been constructed without being designed for earthquakes. A steep cutting in the same geology created around the same time suffered a small failure and needed soil nail stabilisation part way through construction, indicating low static factors of safety.

The material behavior encountered in the completely weathered ignimbrite at Carrs Road appeared to be non standard and a search of published information failed to find relevant research that could assist the design. Classical liquefaction techniques deal primarily with granular and non-volcanic materials and there is limited research regarding strain softening. What research is available appears to be material specific.

The approach was to undertake extensive testing, both insitu and laboratory and to use these results to develop appropriate design parameters. Cantilever piled retaining walls were required to limit the potential impacts on the surrounding properties. The design determined unacceptable deflections of these walls in the earthquake case and so alternative solutions were explored. Propping and anchoring were ruled out due to the proximity of the neighbouring properties and cutting geometry. Ground improvement in the passive zone in front of the piles at ground level was selected and a ground improvement subcontractor was engaged to undertake the design.

2. GEOLOGY

The rolling hills around Hamilton city are the remnants of an ancient plain that was extensively eroded to form the characteristic isolated hills and long, sinuous ridges common around the city. The ridges and hills were subsequently covered in volcanic ash beds, commonly known as Hamilton Ash, up to 5m thick. Later sedimentation by the Waikato River substantially inundated the rolling hills to produce the modern broad plains of the Waikato valley. The plain is known as the Hinuera Surface and the alluvial sediments are termed Hinuera Formation.

The Hamilton Ash formation caps the low hills of the city and consists of rhyolitic tephra deposits weathered to a firm to stiff dark brown and yellow-brown clay. The thickness of Hamilton Ash is typically between one and three metres and it overlies older alluvial sediments of the Karapiro and Puketoka formations. The Karapiro Formation comprises current bedded, weathered sand and gravel and is relatively uncommon in the city area as it has generally been heavily eroded. The Puketoka Formation comprises beds of compact sand, interbedded with stiff clay, silt and minor peat. In places, the Puketoka Formation also includes unwelded pumiceous ignimbrite, which locally is deeply weathered to a soft, sensitive silt and clay. These unwelded ignimbrites are the product of pyroclastic flows (density currents) from distant rhyolite volcanoes that have poured into the Hamilton basin. The resulting Puketoka Formation consists mainly of the distal parts of three thick ignimbrites together with alluvium eroded from them.

At Carrs Road, the hills rise about 8.5m above the level of the surrounding plain, diminishing in height to the north and south. The ash at this location is approximately 3m thick and the unwelded, weathered ignimbrite varies in thickness from 4m to 10m thick. Underlying the weathered soil is dense, relatively unweathered alluvial pumiceous sand and gravel.

3. TESTING RESULTS

The Ash and residually weathered (RW) ignimbrite are relatively straight forward engineering materials and will not be discussed in this paper. Beneath the Ash and the RW ignimbrite is completely weathered (CW) Puketoka Formation ignimbrite. This material is an unwelded pumiceous ignimbrite that is deeply weathered and presents as a sensitive clayey silt soil. There is a relict structure present in the material that, when destroyed by working, releases trapped moisture and turns the sample to a very wet, soft material. This behaviour means that traditional in situ testing such as CPTs and SPTs which involve significant deformation, return extremely low results ($N = 0$ to 1 , $q_c = 1.2\text{MPa}$) in contrast to the apparently good undisturbed strength of the recovered core.

The CPT similarly destroys this structure and so returns relatively poor results (Average q_c approximately 1.2MPa) although these are better than the SPT.

Hand held shear vane tests undertaken in recovered drillcore had peak shear strength results of between 5 and 25kPa. Due to the nature of this material, it is suspected that the core was disturbed during drilling and these test results are not considered representative of the insitu shear strength.

Following the correlation recommendations after Stroud and Butler (1975), the SPT results indicate the CW ignimbrite has undrained shear strength between 0 and 12kPa. Due to the very dynamic nature of the SPT test, it is considered that the disturbance to the structure is such that these results are unrealistically low and as such were discounted.

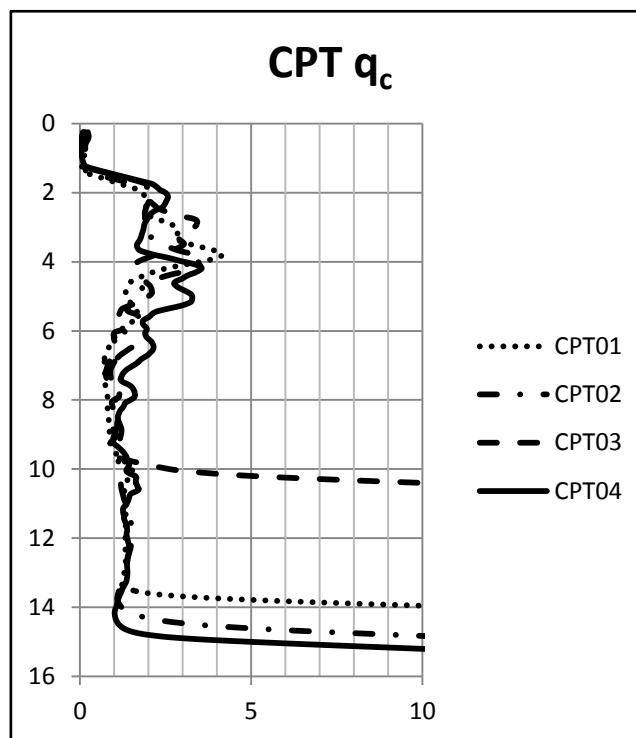


Figure 2. CPT q_c results in the area of the retaining walls

The correlation with the CPT q_c tests gives a higher average strength of between 60 and 100kPa (given an average q_c value of 1.2MPa).

Unconsolidated undrained triaxial tests were also undertaken on two samples of the CW ignimbrite. The samples were taken using thin walled steel tubes to minimise disturbance. Observation of the extruded samples confirmed that there was minimal disturbance. The test results returned undrained shear strength values of 51.5 and 55kPa. The design undrained strength for the CW ignimbrite was taken as 50kPa.

Consolidated undrained triaxial testing (with pore pressure measurement) was undertaken on four samples from the CW ignimbrite. The tests were fairly consistent. A summary of the results is included in Table 1 below.

Sample	Confining pressures (kPa)	Unit weight (γ) (kN/m ³)	c' (kPa)	ϕ' (°)
BH 03 – 8.0m	100, 200, 400	14.7	12	30
BH 03 – 11.0m	100, 200, 400	15.1	32	26
BH 04 – 7.5m	100, 200, 400	16.4	19	31
BH 08 – 8.35m	100, 200, 400	15.6	20	28

Table 1. Triaxial test results

Experience in similar materials in the area and back calculations of slopes and cuttings, indicates that the effective cohesion can be highly variable and should be used in design with caution. In addition, staged testing on samples has been shown to overestimate effective cohesion (Fell, 2005). For this reason, a moderately conservative c' was chosen. The drained design parameters for the CW ignimbrite were $c' = 5\text{kPa}$ and $\phi' = 28^\circ$.

Following the traditional assessment and initial design checks, there was still considerable uncertainty over the behavior of the CW Ignimbrite, especially during a seismic event. To obtain better data for the assessment and design, additional testing was undertaken. The Seismic Dilatometer (sDMT, Marchetti 1975) was mobilized to site for insitu testing and The University of Auckland (UoA) was engaged to undertake cyclic triaxial testing on thin walled tube samples recovered from boreholes.

The flat plate shape of the dilatometer probe has been shown to cause less disturbance than the CPT cone (Baligh and Scott 1975) and so is expected to give more accurate, undisturbed strength and stiffness correlations, especially in sensitive soils. The sDMT is fitted with geophones and so shear wave velocity measurements can be taken at regular intervals.

UoA tested three samples of the CW ignimbrite. All three tests were on visually similar material however, the results varied significantly. The material looked to be relatively undisturbed once extruded from the tubes. One sample behaved very well during testing, with minimal straining even after numerous cycles (over 1000 cycles to $r_u = 0.9$). Another sample displayed near liquefaction state after approximately 100 cycles while the third sample became unstable and effectively liquefied after only 3 cycles. This was consistent with observations in the field of variable strength and some sections of the core liquefying in the core box while being transported and other sections unaffected.

All test and sampling methods are likely to cause disturbance to this material and so it is likely that the real strength may be underestimated. The testing undertaken on the relatively undisturbed samples taken using thin walled steel tubes was considered the most likely to be representative of the real strength.

4. LIQUEFACTION ANALYSIS

Initially, a screening assessment was undertaken using Liquefy Pro (Civil Tech). This follows the guidelines from the NCEER workshops (Youd and Idriss 2001). This assessment identified layers in the overlying ash material as potentially liquefiable but didn't indicate the ignimbrite to be cause for concern. The ash is a stiff, insensitive clay and so is not likely to be prone to either liquefaction or strain softening.

Following the site identification of the ignimbrite material as sensitive, further in situ testing including the sDMT was undertaken. The seismic dilatometer (sDMT) gives the additional benefit of measuring shear wave velocity. This is another parameter that can be used to calculate the cyclic resistance ratio (CRR), a measure of the liquefaction potential of a soil. The CRR is compared with the cyclic shear ratio (CSR) which is the expected loading from the design earthquake to get the likelihood of the material liquefying.

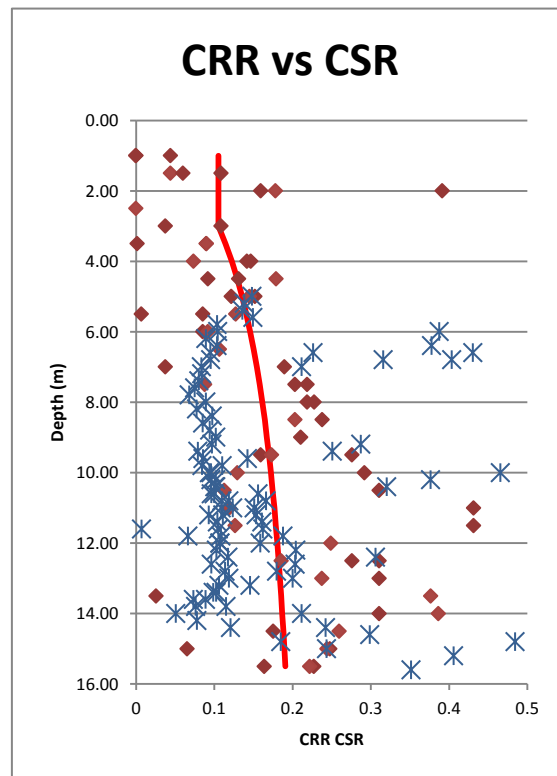


Figure 3 sDMT CRR for Vs and K_D

The CRR was calculated using q_c (from CPT) and v_s and K_D (from sDMT). Of these three methods, both K_D and v_s showed that significant layers of the CW ignimbrite layer were potentially susceptible to liquefaction. Figure 3 shows the CSR from the design earthquake as a red line. The square points are the cyclic resistance ratio (CRR) based on shear wave velocity (v_s) and the stars are CRR based on K_D , a stress state parameter obtained directly from the sDMT.

Boulanger and Idriss (2007) published research examining the susceptibility of fine grained soils to strain softening. The method proposed in their paper was used to assess the factor of safety and it showed the CW ignimbrite had a factor of safety against strain softening of below 1.

The combination of these test results and assessments confirmed that seismic deformations could potentially be excessive and mitigation measures were warranted.

5. RETAINING WALL DESIGN

Given that there was an issue identified with the behaviour of the sensitive ignimbrite layer under earthquake loading, a cutting was ruled out and a retaining wall chosen as the design solution. Stabilising the slope was a potential option considered however, the behaviour of soil nails was uncertain given the sensitivity of the material.

It was clear from the outset that the sensitive material in the passive zone was unlikely to perform adequately as support for the wall throughout its design life. In order to add support to the wall, a number of options were considered. Anchoring the wall was ruled out due to the required length of the anchors meaning that they would have extended under the neighbouring properties, requiring legal easements or alterations to the designation. The geometry of the cutting meant that propping was not an option and so a cantilever wall with improved ground in the passive support zone was chosen.

The wall design procedure was relatively conventional. Material parameters were developed from field and laboratory testing, although more weight was put on the laboratory testing as it was assumed that the dynamic nature of the field testing had damaged the material structure and would therefore give unrealistically low results. The retaining walls were modelled using Wallap (Geosolve) and the engineering parameters developed from the testing results. At this stage the potential for the soil to lose stiffness or strength was ignored and the deflections under the various load cases were assessed.

Under the seismic case, the deformations were maintained at a level at which the neighbouring properties would not suffer damage related to the new retaining wall. The damage prediction analysis was based on a volume balance method similar to methods used to determine ground surface deformations from bored tunnels.

Predicted deflections were up to 100mm at the top of the wall in the long term (drained) case. During the design ULS seismic event, up to 150mm of lateral movement is expected at the top of the wall. This level of movement was assessed as being low enough to not cause damage to neighbouring properties.

The design was split into three sizes. Bored reinforced concrete piles 450mm, 600mm and 900mm in diameter and of varying retained heights up to 5.5m were specified. The piles had to extend through the CW ignimbrite into the dense sand and gravel below. Without keying into the underlying dense layer, deflections and overturning factors of safety were unacceptable.

6. DEEP SOIL MIXING

The ground improvement contractor was engaged to undertake an assessment and design for ground improvement to limit deformations to those deemed acceptable in the previous phase of design. The retaining wall design had assumed unsoftened behaviour and so the brief for the ground improvement was to provide enough confinement or improvement to prevent the onset of strain softening or liquefaction. The contractor undertook a detailed design and optimised the ground improvement design to match the required deflections determined in the design.

The ground improvement design was a triangular grid of deep soil mixed (DSM) columns. The columns are formed by twisting an auger head into the ground and pumping cement grout under high pressure to the tip. This cuts the insitu soil and mixes it with the grout, resulting in a reasonably uniform soil/cement mix. QA testing during ground improvement construction showed that the columns had UCS of 6 to 8MPa, higher than the design requirement.

DMT testing before and after the installation of the DSM columns showed improvement in the test results

7. CONSTRUCTION OBSERVATIONS

During construction, the piling contractor encountered difficulties constructing the piles. Due to the difficult ground conditions, casing was required for all piles to the dense sand founding layer. Despite this, many piles suffered overbreak or partial hole collapse on extraction of the casing. Testing was undertaken on all piles and a number were

identified with potential deficiencies in structural section. Steel reinforcing cages were often not centred in the pile and significant remedial work was required after the walls had been excavated to design level to ensure minimum cover was achieved and to improve pile cross sections that had been reduced due to hole collapse.

The deflections of the walls were monitored using inclinometers. At the time of writing, monitoring was ongoing but all deflections were well within the design predictions.

8. CONCLUSIONS

It is clear from the assessment undertaken in the course of the design of these retaining walls that the volcanic materials in the Waikato Basin are not well understood, particularly pumiceous soils. Traditional investigation techniques such as SPT and CPT potentially underreport the strength of the material leading to overconservative design. Sampling for laboratory testing can also create disturbance and so could also lead to lower test results that aren't representative. This behaviour however indicates that these soils can be sensitive and may be prone to softening during an earthquake. Additional research in these soils will help to determine the engineering behavior and assist in optimising the design of structures.

The sDMT, while relatively new to New Zealand, is well proven overseas and has good potential in sensitive soils due to the reduced disturbance of inserting the blade when compared to a CPT or SPT. Research is being undertaken to confirm the validity of overseas DMT research in New Zealand soils.

A robust design was developed by careful consideration of the observed material behaviour and targeted testing to confirm the design issues and parameters.

Given the low deflections measured at time of writing, back analysis of the stiffness of the improved soil block compared to insitu testing could provide more information on the engineering behaviour of the materials. sDMT testing has been undertaken by the ground improvement contractor before and after installation of the DSM columns, this data could be used in a back analysis of the wall performance and compared against the measured wall deflection.

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SEISMIC RETROFITTING OF AN EXISTING CONCRETE DAM: GROUND ANCHOR INVESTIGATION TRIALS

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ABSTRACT

The Arnold Dam located on the West Coast of the South Island of New Zealand was constructed in the 1930s. Initial assessment of the 13m high, mass concrete spillway section indicated that seismic retrofitting was required to meet current seismic safety criteria. Inclined, multi strand post tensioned ground anchors have been proposed to improve the sliding and overturning stability of the gravity spillway section.

This paper includes a review of the investigations that were developed to target key design and construction risks associated with the anchoring works. The investigations included five multi strand trial anchors and supporting boreholes located within the foundation material. The investigation, design and construction risks associated with the installation of the permanent ground anchors are discussed with a particular focus on the significance of progressive debonding of the ground anchors.

1. INTRODUCTION

The Arnold Dam is located on the West Coast of the South Island of New Zealand. It consists of an existing concrete core earth embankment dam and a mass concrete gravity spillway. The dam is up to 13m high and 300m in length and has been classified as a medium potential impact category structure in accordance with the New Zealand Dam Society on Large Dams guidelines (2000). This is a reflection of a medium consequence in the event of a dam breach. The dam was constructed in the 1930s with a run of the river hydroelectric power scheme commissioned in 1932 which is currently still operational.

Preliminary assessment of the stability of the concrete gravity spillway was undertaken in 2009 and noted several deficiencies under extreme loading conditions (seismic and flood). Inclined, multi strand, post tensioned anchors were proposed to improve the sliding and overturning stability of the spillway block.

In 2010 Tonkin & Taylor Ltd (T&T) planned and implemented site investigations that included a two phase approach to the investigations. Phase one was focused on characterizing the site and involved developing a detailed geological model and characterization of the foundation materials. It included a series of boreholes and laboratory testing that targeted the foundation materials directly below the spillway and the likely bond zone for the ground anchors. Subsequently a series of anchor trials were conducted as part of a second phase of investigations. These were based on the preliminary geological model and material characterizations developed in phase one. A preliminary analysis of the structure was undertaken as part of the phase one investigations to provide a good indication of the likely orientation and loading of the permanent anchors.

The design of the trial anchors aimed to target the key geological conditions, design considerations and project risks to provide a robust basis for detailed design of the permanent anchors. Several of the key construction and design risks identified are set out in the subsequent sections with a focus on the potential for progressive debonding to occur in the fixed anchor length.

2. GEOLOGICAL SETTING (PHASE ONE INVESTIGATIONS)

2.1. GENERAL

The importance of understanding the geology (regional and local) and hydrogeology in any anchoring project cannot be understated. A detailed and accurate geological model can provide a good understanding of the materials, target anchor depths, groundwater conditions and potential geotechnical risks. Site specific anchor testing is critical in understanding the performance of high capacity anchors and cannot be replaced by subsurface investigations and laboratory testing alone. A two phased approach to the investigations allowed major issues that may affect the capacity of the permanent anchors to be identified early and targeted in the subsequent trial anchors. The potential risks identified during an earlier desktop study included smearing, slaking and “squeezing” of the borehole as well as the strain sensitivity of the material.

2.2. PRELIMINARY GEOLOGICAL MODEL (DESKTOP STUDY)

The Arnold Dam is located in the middle reaches of the Arnold River where terraces of glacial derived gravels grade laterally with more recent alluvial terrace surfaces. The local basement rocks are weak sedimentary rocks and can be seen outcropping downstream of the dam and in the ridges on either side of the valley. At the dam site these consist of unweathered blue-grey Siltstones and Sandstones (Blue Bottom Group). The spillway is founded within the Blue Bottom Group and was the focus of the investigations and anchor trials.

The phase one investigation plan was developed on the basis of an initial desktop study of the available information, including previous investigations in the vicinity of the dam. A preliminary geological model was developed based on the information gathered in the desktop study and surface mapping of the site.

2.3. SUBSURFACE INVESTIGATIONS (PHASE 1)

The phase one investigations consisted of a series of boreholes and test pits that were used to confirm the preliminary geological model.

The Siltstone was observed in test pits and boreholes as generally having extremely weak strength. While bedding defects were not observed in outcrops in the vicinity of the Arnold Dam, bedding plane defects/shears were observed in several of the investigation boreholes. The bedding shears are likely to have originated from flexural shearing as a result of regional folding. The bedding planes observed in core were predominantly planar, smooth, very tight and clean. Several bands of clayey Silt were observed up to 10 mm thick parallel to bedding.

No ground water flows were encountered during the phase one investigations due to the low permeability of the Blue Bottom Group, however hole “squeezing” was noted in several boreholes. High pore pressures (artesian) were recorded using vibrating wire piezometers installed in the boreholes. The potential for softening and smearing of the drill hole wall, thereby reducing the grout to rock bond was also identified as a risk. Methods to minimize these risks were subsequently included in the installation methodology for the trial anchors.

The development of a detailed geological model following the phase one investigations allowed the designer to undertake an accurate preliminary design of the proposed anchoring system. This was subsequently incorporated into the phase two investigation plan.

3. CONCEPTUAL DESIGN CONSIDERATIONS

3.1. GENERAL

A well developed concept design provides an opportunity to clearly define some of the constraints with respect to materials, design loads, access and installation methodology. This allows a trial anchor investigation to be planned that is likely to be more representative of the final anchor installation and design methodology. Specific design parameters should be targeted in the phase two investigations (anchor trials) based on the results of the concept design.

3.2. PERMANENT ANCHOR GEOMETRY

Inclined ground anchors (45 degrees) were assessed to be the most economical method of stabilizing the Arnold dam spillway. A cross section of the geometry of the spillway block is shown in the figure 1 below.

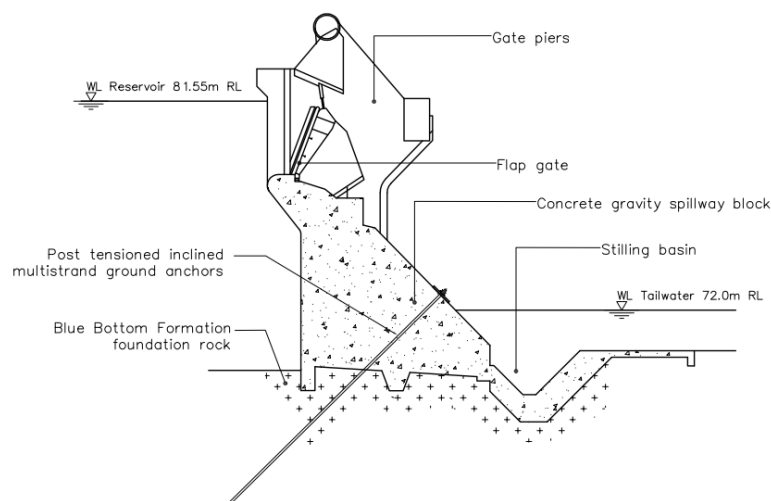


Figure 1: Arnold Dam – Spillway Cross Section

The inclination of the anchor will increase the normal force acting on the foundation interface while also providing a component that acts directly upstream to stabilise the dam. The 45 degree inclination of the anchor does however, have a significant impact on the installation methodology and anchor geometry. The free length of the anchors will need to be installed through the existing concrete spillway block which further constrains the diameter and installation methodology.

4. KEY DESIGN ASSUMPTIONS

4.1. LOADING SCENARIO AND DESIGN LOADS

The Arnold Dam is located in a high seismicity region with the main Alpine fault and the Brunner fault just 20km and 8km from the site respectively. The site specific seismic accelerations assessed were significant with a maximum design earthquake peak horizontal ground acceleration of 0.66g (1:2500yr) and a peak vertical ground acceleration of 0.5g (1:2500yr).

Preliminary stability assessment of the structure indicated that it will not meet current safety criteria for several load cases with the critical being seismic. The assessment indicated an ultimate anchor load of 3000kN based on an anchor spacing of 2.5m. The Ultimate load capacity was shown to be heavily reliant on the stiffness and interaction of the reservoir, foundation and structural anchor system. The load displacement curve of the anchor head has a significant influence on the load attracted by the structure and the anchoring system under seismic loading and represents a key design risk.

Generally anchors are modeled as a linear-elastic spring with a stiffness equal to the rigidity of the anchor bar or tendon. In reality the response of the anchor head is likely to be non-linear. This is due to the progressive yielding in the bond that is required to mobilize the resisting shear strength and is covered in more detail in section 3.2.3. The displacement required to mobilize the design shear strength within the bond length can be a significant portion of the total displacement of the anchor head and result in a nonlinear load displacement curve for the anchor head. Careful monitoring of the trial anchors can provide sufficient data to characterize the bond stress distribution within the bond length of the trial anchors. Displacement curves can then be extrapolated to provide estimates of displacement for greater bond lengths where required for design.

In order to provide a robust analysis of the structure, the anchor trials were carefully monitored. This will enable the expected load transfer mechanism to be established and extrapolated to longer bond lengths. The analysis of the anchor trials will provide a reasonable basis for estimating of the load displacement curve for the permanent anchor lengths.

4.2. CREEP/ACCEPTANCE CRITERIA

It is important that a trial anchor is loaded for a sufficient amount of time to establish that the bond stresses are stabilizing rather than continuing to creep. Eurocode EN1537 provides guidance for defining the ultimate capacity of an anchor based on the rate of creep.

Under initial loading of the anchor, the stresses within the bond zone tend to redistribute over time until they reach equilibrium. This is particularly evident in foundation materials that exhibit strain softening. The creep displacement of the bonded length of the anchor will result in loss of post tensioning load over time. Every material has different creep characteristics and it is necessary to obtain a good understanding of the performance of the anchors under sustained loading during the trials. The rate of creep in the test anchors aid the designer in establishing acceptable criteria for proof testing of the permanent anchors as well as the ongoing monitoring and restressing of the anchors.

4.3. LOAD TRANSFER MECHANISIM

The majority of design codes assume that the grout to rock shear strength is developed evenly over the full length of the bond. Based on this assumption the anchor load will reduce linearly from the proximal (top) end of the anchor bond length towards the distal end (bottom). Figure 2 below provides a graphical representation of this simplified assumption.

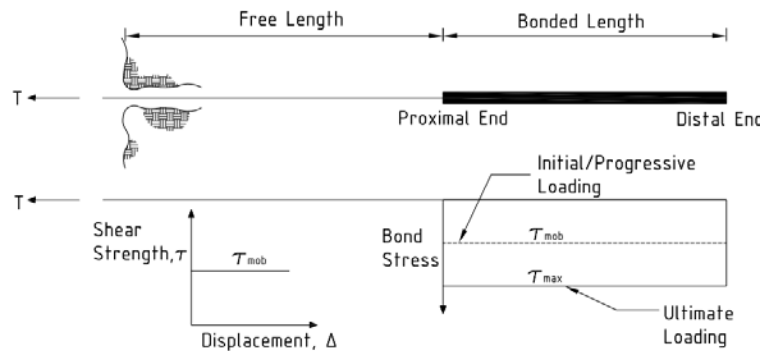


Figure 2: Simplified bond stress distribution along normal bond length (strain independent)

In reality, straining of the rock is required in order to mobilise shear strength in the bond length (grout to rock). Under initial loading, straining tends to be confined to the proximal end of the bond length while the distal end remains unloaded. As the load in the anchor is incrementally increases, the straining in the anchor bar penetrates further into the bond length. This has the effect of mobilising more of the fixed length of the anchor. The expected stress distribution and anchor loads are illustrated in Figure 3 below.

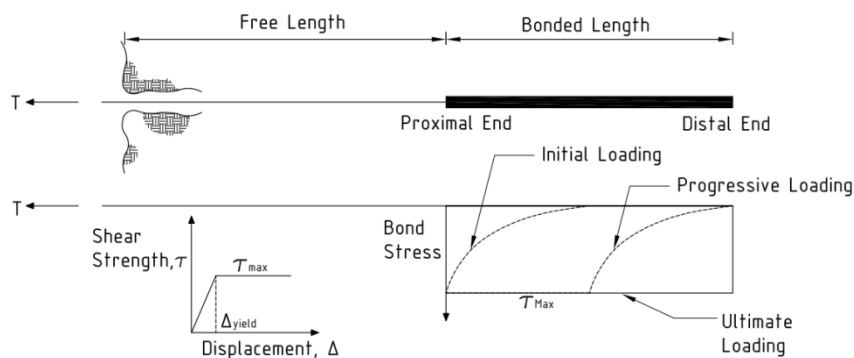


Figure 3: Bond stress distribution along normal bond length (strain dependent)

This effect is largely controlled by the stiffness compatibility between the anchor tendon and the ground. It is assumed that the grout column does not affect the rigidity of the anchor significantly as any tensile stresses developed in the grout will be accommodated by radial cracking in the grout column. Generally stiff rock is more prone to stress concentrations at the proximal end of the anchor. The high stiffness relative to the anchor tendon will often result in the distal end of the anchor remaining unloaded.

Particular care needs to be taken when dealing with materials that are strain sensitive where a residual strength is mobilized following the peak strength. The result is the development of a phenomenon known as progressive debonding. This effect was first reported in work by Ostermayer and Schelle (1977) where instrumented anchor tests in sands developed concentrated bond stresses at the proximal end of the anchor. As the anchors were loaded further the distal end began to pick up load at which point the proximal end of the bond length had displaced enough for the shear strength to have reduced to residual strengths. This phenomenon is generally a risk in lower strength materials such as soils and soft rock due to the longer bond lengths required to develop the design capacity. Figure 4 below illustrates the concept of the incremental loading of an anchor in strain sensitive materials.

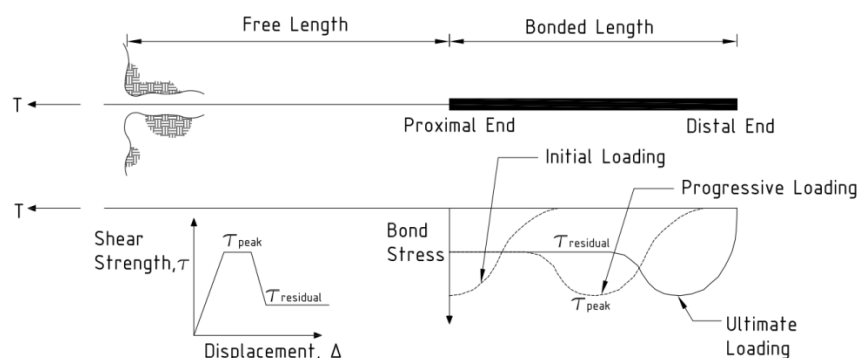


Figure 4: Progressive debonding along a normal bond length (strain softening)

The Blue Bottom Group materials (siltstones and sandstone) located at the Arnold dam site are classified as soft rock. Previous instrumented anchor trials in similar materials reported by Weetasinghe and Littlejohn (1997) showed significant peak strengths mobilized over the bond length with significantly lower residual strengths. Shear box testing conducted as part of the phase one investigations indicated that there was a significant difference between the peak and residual strengths for the Blue Bottom Group. As such, progressive debonding was identified as a significant design risk that would need to be addressed in the phase two trial anchor investigations.

The material was expected to have a relatively high bond strength which would make it difficult to test a full production length anchor to failure. This meant that results from shorter test bond lengths would need to be extrapolated to full design bond lengths. In the case of the Arnold Dam, extrapolation would need to be made from a 3m test length to close to a 10m bond length, based on estimated values for grout to rock shear strength. Typical methods for dealing with progressive debonding have been limited to the application of empirical efficiency factors. Currently these have been developed for granular and cohesive soils, however very little guidance is provided on efficiency factors for high capacity anchors in soft (strain softening) rock.

This presents a real risk that the design bond length may be unconservative if the correct efficiency factor is not applied. Careful monitoring of the displacement of the proximal and distal end of the anchors can be used to assess the load distribution in the anchor and provide a good understanding of the stress distribution in the bond length.

4.4. CONSTRUCTION METHODOLOGY

The complexity of the load transfer mechanism makes the results of a set of anchor tests difficult to interpret if there are too many variables. The trial anchors should provide a good representation of the permanent anchors to reduce the number of variables that need to be considered when completing the detailed design of the permanent anchors. This combined with the fact that the capacity of an anchor is generally very sensitive to the installation methodology means that it is important that the installation methodology for the test anchors is carefully controlled and matches the method proposed for the permanent anchors.

Due to the importance of the structure, access constraints and risks associated with flood flows, early contractor involvement was used to develop a safe, practical solution for the permanent anchors that could be properly assessed during anchor trials.

5. TEST ANCHOR DESIGN METHODOLOGY (PHASE TWO)

Based on the phase one testing and conceptual design we were able to stipulate with a degree of confidence, the key design parameters that required investigation. Four shallow, vertical trial anchors were proposed with a fifth inclined anchor installed at depth within the proposed bond zone below the spillway block. The four vertical anchors were installed at a shallow depth to reduce the number of variables involved in the anchor trials. The vertical installation and shallow depth allowed the bond length to be carefully controlled and verified and provided an economic means of testing several anchors. The inclined anchor provided an opportunity to test a preferred and more developed methodology for the permanent anchors. It also investigated the effects that the inclination had on the load capacity. The homogeneous nature of the rock and the lack of any significant defects meant that the risk of obtaining vastly different results from the inclined anchor compared to the vertical anchors was relatively low.

Softening/smearing of the hole is considered a significant risk when drilling in weathered siltstone materials. As such the drilling methodology adopted for the anchoring works avoided the use of drilling mud as a method for flushing the hole. The drill hole was required to be flushed with water and the anchor installed and grouted within 24 hours to avoid “softening” and degradation of the hole.

The preliminary design provided a reasonably good estimate of the geometry and design loads for use in establishing the trial anchor arrangement. The sensitivity of the detailed design to specific design assumptions was assessed and particular care was taken in developing an investigation plan that targeted the critical design assumptions. This included careful monitoring of the load displacement curve of the trial anchors for use in the modeling of the structure – anchor interaction. Collection of sufficient data to allow extrapolation of the bond length capacities from the relatively short trial anchors to the full anchor bond lengths.

Finally the ability of the anchors to carry the design loads is generally assessed on the rate of creep. Stabilisation of the stresses within the bond length may take a considerable amount of time to reach equilibrium due to the nature of the materials. Every material behaves slightly differently and high rates of creep may not necessarily be indicative of the failure of the bond length. The European code EN1537 provides guidance on acceptable creep rates however it does allow higher creep rates that have been proven during investigation tests. The testing of the anchors was completed in six cycles up to the test load to allow the creep performance at the peak load for each cycle to be monitored. The load was held for a fixed time or until the creep was shown to be reducing over several time intervals. A good understanding

of the creep performance of anchors at the site can provide confidence when assessing the load displacement curves from acceptance testing of the permanent anchors.

6. CONCLUSIONS

The importance of understanding the geology and hydrogeology of a site cannot be understated when planning a series of anchor trials. A two phase investigation approach was used to develop a good understanding of the geological model, groundwater conditions and material properties that were likely to affect the performance of the main anchoring works for the Arnold Dam. It also provided an opportunity for a robust preliminary design of the main anchoring works to be completed on the basis of the information gathered in the phase 1 investigation.

Following the phase 1 investigation the following risks were identified as being critical to the success of the main anchoring works:

- Softening/ smearing of the drill hole

This can often occur in weak fine siltstones and can significantly reduce the design capacity of an anchor. The anchoring methodology was adjusted to control the quality of the finished hole.

- Load Displacement Curve

The load displacement curve can have a significant impact on the load attracted by an anchor, particularly under seismic loading. Loading of the test anchors in cycles up to a peak design load were used to provide a detailed load displacement curve for each anchor for use in the detailed modeling of the structural interaction.

- Creep/Acceptance Criteria

The performance of an anchor under sustained loading is generally used as a means of defining failure in an anchor. It is also regularly used as criteria for acceptance testing of the main anchoring works. Every material and anchor geometry is different and it is important to gain a good understanding of the creep performance during the trial anchors to provide confidence when assessing the results from acceptance tests.

- Load Transfer Mechanism (Progressive Debonding)

Test anchors often have a smaller fixed length than the main anchoring works due to the difficulty in testing a production anchor to failure. As a result bond capacities are often extrapolated to longer fixed lengths using generic efficiency factors. If the bond stress distribution is well understood during the anchor trials, a higher efficiency factor may be able to be justified.

It must be acknowledged that the development of the capacity of a ground anchor is dependent on many factors and is in reality is a very complex mechanism. As such, site specific trial anchors are important in assessing the expected performance and should resemble the permanent anchors as closely as possible. A two phase investigation process allows a more accurate estimate of the final anchor design to be formulated and key risks to be identified and addressed in the anchor trials

7. ACKNOWLEDGEMENTS

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DESIGN OF DYNAMIC FOOTINGS IN THE PILBARA REGION OF WESTERN AUSTRALIA

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ABSTRACT

With Western Australia's ongoing resources boom, the construction of specialised mining infrastructure in remote locations provides interesting and unique challenges to geotechnical engineers. Large ore-processing structures, such as screening plants and crushing facilities, are designed to vibrate as part of their routine operations. This forces the footing to withstand dynamic loading that can potentially result in large differential settlements or bearing capacity failures if not founded on suitable material or designed appropriately.

To assist with the structural and mechanical design of these dynamically loaded structures, a dynamic shear modulus value is required for the soil-structure interaction. The unique geological and geotechnical conditions of the Pilbara region present specific challenges, and yet there is limited published information available regarding methods of assessing the dynamic shear modulus for these conditions. This paper presents a review of the available information, investigation methods and a weighted average method of assessment to estimate the dynamic shear modulus for typical Pilbara ground conditions.

1. INTRODUCTION

The design of dynamic footings requires an assessment of parameters for the design subsurface soil profile. The methods for assessing the dynamic shear modulus (G_d) for soils in the Pilbara region will depend on the subsurface material present at the investigation site. This paper presents a method for calculating a dynamic shear modulus value for use in the design of dynamic footings, applicable to the subsurface conditions typically encountered in the Pilbara region.

2. PILBARA SUBSURFACE CONDITIONS

The subsurface conditions in the Pilbara region are greatly diverse due to the size of the region and its complex geology and geomorphology (Hickman, 1983). Based on recent geotechnical investigations by the author, the surficial soil and rock that may be encountered in the zone of influence for a foundation can be generalised as:

- *Alluvium* – typically silty/clayey sands (fine grained, red-brown, approximately 20% to 30% low to medium plasticity fines, loose to dense) or sandy clays (medium to high plasticity, red brown stiff to hard);
- *Colluvium* – typically silty/clayey sandy gravels (fine to coarse grained, sub angular, approximately 20% to 30% low to medium plasticity fines, often with cobbles, medium dense to very dense);
- *Cemented materials* – conglomerate or ferricrete (weakly to well iron-cemented Alluvium and Colluvium deposits);
- *Tertiary aged sediments* – silt/clay, siltstone/claystone (fine grained, extremely low to medium strength, residual soil to distinctly weathered rock);
- *Igneous rocks* – typically dolerite or granite (fine to medium /coarse grained, very low strength to high strength, extremely weathered to moderately weathered, generally increasing in strength and becoming less weathered with depth);
- *Proterozoic metasediments* – chert, shale, banded iron formation (fine grained, layered, high to extremely high strength, slightly weathered to fresh).

Groundwater is generally not encountered at depths influenced by loading.

3. IN SITU TESTING

The two general *in situ* methods for assessing the dynamic properties of a soil involve geophysical and geotechnical tests. The use of geophysical or seismic tests to assess dynamic soil properties is generally accepted as a reliable method of assessing the dynamic soil properties the soil profile. Dynamic parameters of the soil can be derived from measuring the velocity of seismic waves, including compression or primary (P) waves (V_p) and shear or secondary (S) waves (V_s).

Investigation methods can be divided into two distinct areas: - non-intrusive surface wave tests; and intrusive down-hole soil profiling using cased boreholes. Non-intrusive test methods include Multi-channel Analysis of Surface Waves (MASW) and Refraction Micro-tremor (ReMi), both of which involve measuring the velocity of surface (shear) waves using uniform spaced geophones. As these tests record V_s from the ground surface, the confidence and accuracy of the test results generally decrease with increasing depth. Tests can also be affected or not identify bands of low velocity material in between high velocity material and vice-versa.

Intrusive methods typically require the drilling of boreholes and installation of PVC casing in uncemented gravels for down-hole or cross-hole seismic tests to ensure good contact is made between the probe and the soil mass. The cross-hole seismic test can be performed with two or more boreholes to improve the confidence level of the recorded values. These tests are generally only limited by the depth of the borehole and can pick up bands of varying velocity material.

As the Pilbara soils are often coarse alluvial gravels or parent rock, most geotechnical investigations include the drilling of boreholes to perform *in situ* tests (i.e. standard penetration test and pressumeter), visually logging the material, and collecting core samples for laboratory testing (i.e. point load index and uniaxial compressive strength tests). These tests are used to assess the elastic modulus at a large strain level (E_s), which can then be converted into a dynamic shear modulus value relevant to that large strain level (G_s). Alternatively, Kramer (1996) provides a summary of direct correlations for the dynamic shear modulus at very small strain from conventional *in situ* geotechnical tests.

Very small strain ultrasonic pulse velocity tests based on bender elements (Vigianni and Atkinson 1995), can also be performed during unconfined compressive strength tests of core samples to provide values for V_p and V_s , although these are not suitable for soil strength material, i.e. gravels. As an alternative investigation technique to drilling boreholes, plate load testing of foundations can be performed to estimate a value of E_s .

The test method most applicable to individual site will vary depending on factors such as foundation material, size and configuration of the structure, site access and available budget. In addition seismic testing, including a direct measurement of the shear velocity profile, should ideally be performed in conjunction with conventional geotechnical testing at the proposed structure location.

4. DYNAMIC SHEAR MODULUS

Values for G_d are generally required for the design and analysis of foundations under dynamic loading. These values can be calculated using both dynamic and static soil parameters, assuming the soil behaves elastically (Bowles, 1997).

It is customary to use V_s when calculating the G_d using the following equation:

$$G_d = \rho V_s^2 \quad \text{Where } \rho \text{ is the density of the soil in kg/m}^3 \quad (1)$$

A value for V_s can also be derived from the P wave velocity (V_p), using Poisson's ratio (ν):

$$V_s = \sqrt{V_p \frac{1-2\nu}{2(1-\nu)}} \quad 0 \leq V_s \leq 0.707V_p \quad (2)$$

Values for V_p and V_s are generally derived from *in situ* geophysical tests as mentioned in the previous Section.

An example of a typical V_s profile from an investigation site in the Pilbara is presented as Figure 1.

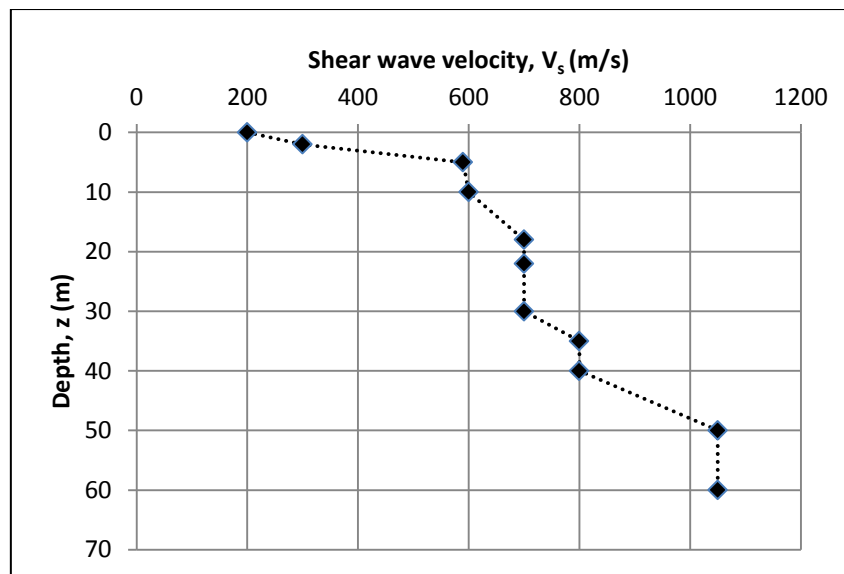


Figure 1: Example plot of V_s versus depth for a site in the Pilbara region.

The relationship between the shear modulus of a soil and its stress-strain elastic Modulus (E) is the same for dynamic conditions as it is in static conditions, which is as follows:

$$G_s = \frac{E_s}{2(1 + \nu)} \quad (3)$$

Values for E_s and therefore G_s can be derived using conventional *in situ* geotechnical tests.

When calculating G_d using 'static' geotechnical tests, the G_s value will need to be adjusted for the relevant level of strain, due to the non-linear stiffness-strain relationship of a soil. Although this relationship is well known to decrease with increasing strain, defining this relationship is still the focus of many studies in this field (Atkinson, 2000).

Most studies consider a normalised dynamic shear modulus (stiffness) value derived from dividing G_d by the assumed maximum dynamic shear modulus value, G_{max} . This value of G_{max} is typically considered to be G_d at a very small level of strain (typically $G_d \leq 10^{-4}$). For static loads, the level of strain is generally considered to be large i.e. greater than 10^{-2} . For vibrating and mechanical structures the level of strain developed is generally considered to be small and between the range of about 10^{-3} and 10^{-2} (Arya et al., 1979).

The majority of studies assessing stiffness-strain relationship have generally focused on fine grain soils (clays and silts) and sands, which are generally not applicable for sites in the Pilbara. Although there are areas of recent and Tertiary aged alluvial deposits in the Pilbara the majority of sites investigated for crushing and screening plants are typically located in areas of shallow colluvium overlying weathered siltstone/granite or fresh chert/banded iron formation.

Seed et al. (1986) highlighted the limited amount of published information on the dynamic modulus of gravel materials, and provided an assessment of the stiffness-strain relationship for a range of gravel materials. The assessment included a dynamic modulus estimate using cyclic undrained triaxial laboratory tests on crushed rock and natural gravels from the Californian area. Results published by Seed et al. (1986) identified the stiffness-strain relationship for the full range of gravel materials to be relatively consistent, if not slightly flatter, than the comparisons developed for sands by Seed and Idriss (1970). Of note, the authors also commented that the variation of material grading and relative density had little effect on the dynamic shear modulus compared with more significant variations from changes in strain.

The range of normalised dynamic shear modulus values versus shear strain for gravel soils presented by Seed et al. (1986) has been reproduced as Figure 2.

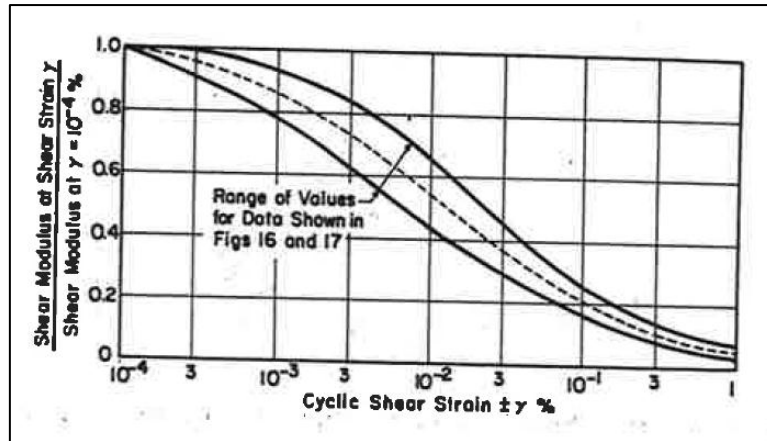


Figure 2: Variation of dynamic shear modulus with shear strain for gravelly soils (Seed et al., 1986)

5. WEIGHTED AVERAGE DYNAMIC SHEAR MODULUS

The response of a dynamic foundation under loading is often difficult to model due to the complexities of modelling soil and structural or dynamic loads simultaneously. Often the soil-structure interaction is modelled using a lumped mass or lumped parameter method (i.e. springs and dampeners) by mechanical/structural engineering software. Using this method, values for dynamic shear modulus and Poisson's ratio are required to calculate spring and dampener constants to simulate the soil behaviour. As the soil profile is typically non-homogeneous, calculating a weighted average dynamic shear modulus is one method for representing the behaviour of the soil profile in the zone of influence beneath a footing.

Assuming the soil behaves elastically, the weighted average dynamic modulus can be calculated by using the well known Boussinesq (1885) soil stress distribution equation or Influence factor (I_R) charts as developed by Fadum (1948) as presented in Figures 3 and 4 respectively.

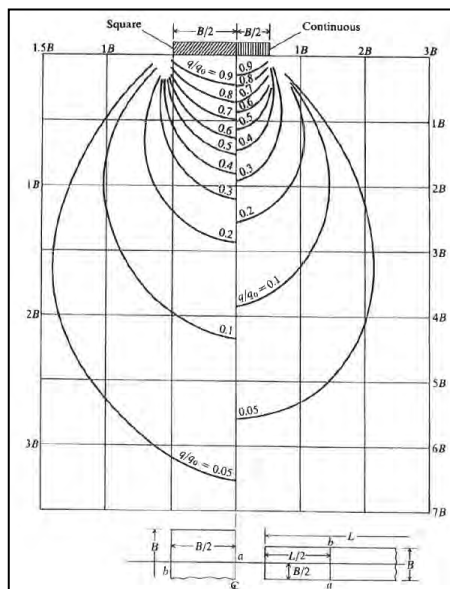


Figure 3: Boussinesq pressure distribution

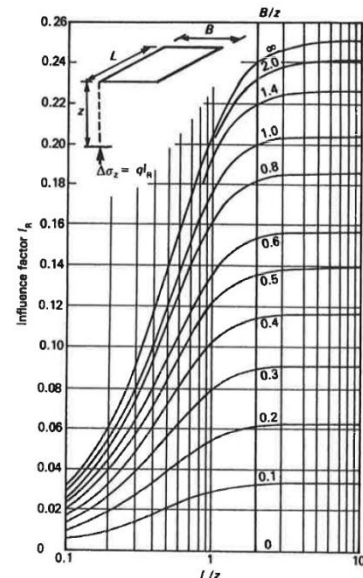


Figure 4: Fadum's chart

More advanced stress distribution formulas and soil analysis software that consider a soil mass with layered strata have also been developed, such as Westergaard (1938) and Small and Booker (1986).

Once the soil profile and dynamic soil parameters have been developed for the site, a weighted average dynamic shear modulus can be calculated. As previously mentioned, the dynamic stress-strain distribution behaves consistently with the static stress-strain distribution beneath a footing, allowing us to reduce $G_{\max}$ in proportion to the stress reduction factors presented by Boussinesq (1885), Fadum (1948) or others to calculate G_d for each

stratum. The weighted average value for $G'_{\max}$ can be calculated using trapezoidal integration with respect to depth or by using soil analysis software.

An example method for assessing the weighted average G_d using the V_s data from Figure 1, Fadum's chart (Figure 4) and numerical integration has been presented as Table 1. The example assumes a rectangular footing of 30 m width (B), founded on a homogeneous soil with density (ρ) of 2.5 t/m^3 . A stiffness-strain reduction factor of 0.7 has been assumed from Figure 2.

Depth, z (m)	V_s (m/s)	$G_{\max}$ (GPa)	B/z	Influence factor, I_R	G_d $\{=0.7 \times I_R \times G_{\max}\}$ (GPa)	Numerical integration of I_R {A}	Numerical integration of G_d {B}	Weighted average G_d $\{=B/A\}$ (GPa)
0	200	0.10	0.00	1.00	0.07	0.00	0.00	
2	300	0.23	15.0	1.00	0.16	2.00	0.23	0.114
5	590	0.87	6.00	1.00	0.61	5.00	1.34	0.276
10	600	0.90	3.00	0.97	0.61	9.92	4.43	0.446
18	700	1.23	1.67	0.91	0.78	17.4	9.98	0.573
22	700	1.23	1.36	0.84	0.72	20.9	13.0	0.620
30	700	1.23	1.00	0.70	0.61	27.1	18.3	0.674
35	800	1.60	0.86	0.61	0.69	30.4	21.5	0.707
40	800	1.60	0.75	0.54	0.61	33.3	24.7	0.743
50	1050	2.76	0.60	0.43	0.83	38.1	31.9	0.837
60	1050	2.76	0.50	0.34	0.63	41.9	39.3	0.936

Table 1: Example method for calculating weighted average of G_d

By dividing the maximum numerical integration of G_d by the maximum numerical integration of I_R , a weighted average G_d of 0.938 GPa was calculated for the full soil profile. Figure 5 presents a plot of the measured $G_{\max}$ values and weighted average G_d values from Table 1 versus depth.

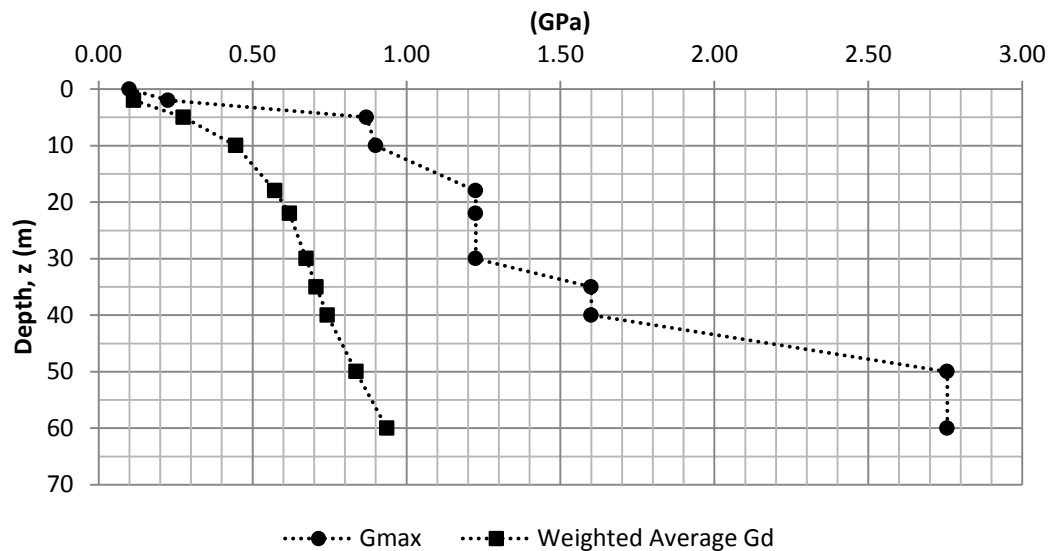


Figure 5: Dynamic shear stress values

6. CONCLUSIONS

Based on the limited studies that have been performed on the stiffness-strain relationship for gravel materials, the relationship can generally be assumed to fit the limits of the sand stiffness-strain curve.

Variations to Poisson's ratio and the soil density typically have little effect on the dynamic soil parameters, however, variations in strain level can significantly affect the foundation design.

A combination of both geophysical and geotechnical test methods is recommended when assessing soil parameters for a soil profile.

Due to the comparable stress-strain behaviours between the dynamic and static shear modulus, a weighted average dynamic shear modulus value can be assessed using a stiffness-strain distribution below a footing. Consideration should be given to limiting the depth of the soil profile used when assessing a weighted average for G_d as material greater than 1.5B depth can significantly influence the value as shown by the increasing weighted average G_d values in Table 1 and the data in Figure 5.

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QUANTITATIVE ROCK FALL HAZARD RISK ASSESSMENT AT CATHEDRAL COVE, HAHEI, NEW ZEALAND

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ABSTRACT

Several rock falls have occurred within the last two years within the sea arch at Cathedral Cove, North Island, New Zealand. Cathedral Cove is one of New Zealand's top five tourist attractions, with over 150,000 visitors passing through the sea arch in 2009. Regular geotechnical inspections and assessments of the rock fall hazard risk within the arch were undertaken at the request of the Department of Conservation.

Rock fall hazard risk was calculated using conventional probability theory, and the calculated risk was used to inform decisions on public access through the arch. Engineering options to reduce the risk were presented to the Department of Conservation, and the decision was made to undertake low impact scaling of identified unstable zones in the arch using specialist rope access contractors.

The recognised hazard areas of the arch were scaled in November 2011 to an extent that has reduced the perceived risk to an acceptable level. This work has also minimised any negative visual impact on the arch. A monitoring program has been set up to manage the future rock fall hazard risk, as the arch is subject to ongoing weathering and erosion.

1. INTRODUCTION

Cathedral Cove is a natural sea arch located near Hahei on the North Island of New Zealand. The arch is situated in the intertidal zone and at low tide provides pedestrian access between two beaches (Figure 1). The headland and cliffs around Cathedral Cove comprise partially welded lithic- and pumice-rich ignimbrite of Tertiary age.

Three separate rock falls occurred from the arch between October 2009 and May 2011. Tonkin & Taylor (T&T) was engaged by the Department of Conservation (DOC) to undertake a geotechnical assessment of the arch. Detailed inspections of the arch interior and also of the roof of the arch were undertaken, which identified distinct rock fall hazard areas within the arch. DOC took the initial precaution of erecting signs at the entrances to the arch warning of the rock fall hazard, as well as closing the arch to visitors.

T&T was also requested to assess the most appropriate way of determining the risk to human life based on the historic number of visitors arriving at the arch supplied by DOC. To undertake this task, the historical visitor data was assessed with an appropriate allowance for growth rate. Subsequently, the brief was widened to incorporate recent tourism and recreational data obtained from various sources.

Previous published rock fall hazard assessments have focussed primarily on the probability of fatality resulting from the interaction of vehicular traffic at the base of cut or natural slopes associated with civil infrastructure (Pierson et al, 1990; Morgan, 1991). This study provided an opportunity to assess a rock fall hazard risk in a natural sea arch where rock falls are typically from overhanging faces and where the area of highest pedestrian traffic coincided with the area of highest perceived rock fall hazard risk.

Visitor data and knowledge of rock fall frequency enabled a quantitative assessment of the annual risk of a fatality in the arch based on a series of concurrent probabilities. This paper presents the results of the risk assessment and how it was used to inform decisions regarding ongoing management of the rock fall hazard risk in the arch.

2. RISK ASSESSMENT

The risk of a fatality within the arch was analysed using conventional probability theory. The probability of a fatality caused by falling rock within the Cathedral Cove arch is dependent upon the factors discussed in the following sections, which must act concurrently for a fatality to occur.

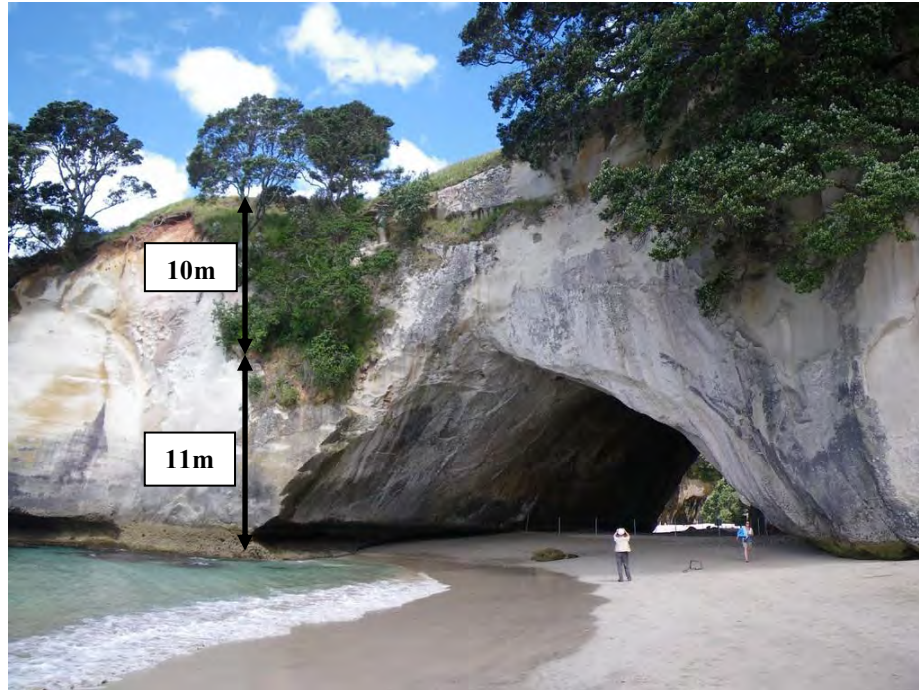


Figure 1: Cathedral Cove Arch, Hahei

2.1 NUMBER OF PASSES BY AN INDIVIDUAL THROUGH THE ARCH

The Department of Conservation has foot traffic counters within the Cathedral Cove Reserve which are totalled every month. As visitors trigger the counter both entering and leaving the cove, the raw DOC figures would normally be halved in order to determine visitor numbers to Cathedral Cove. However, it has been assumed that all visitors will pass through and return through the arch once in order to reach the beach on the other side; i.e. the figures presented in Figure 2 represent the number of passes through the arch, irrespective of direction. A fault in one of the counters has resulted in incomplete data for 2010 and 2011, and 2009 is therefore the most recent year for which complete visitor data is available.

The DOC counters only record the number of individuals both arriving and departing from Cathedral Cove by foot. However, visitors to the cove also arrive by sea through a combination of recreational motor boats and those using commercial sea kayak tour operations. We have estimated an additional 10% of visitors that arrive by sea. Projected passes through the arch are also presented in Figure 2, assuming compound growth rates of between 2.5% and 10%. Figure 2 shows that there is an obvious increase in the number of visitors to the arch every year. Between 2005 and 2009, the compound growth rate was 26% per annum.

2.2 ROCK FALL FREQUENCY

From discussions with DOC staff with considerable knowledge of Cathedral Cove, large rock falls in the arch occur on average approximately every 3 to 5 years. The recent series of rock falls are considered to represent a period of particularly active erosion of the arch, and extended periods of time of up to several years without any rock falls have previously been noted by DOC. The probability analyses were therefore carried out assuming that on average a potentially fatal rock fall occurs every 4 years.

The analyses undertaken have assumed the probability of a rock fall is the same for all seasons of the year. It is likely that rock falls would be more frequent in wetter months, when fewer people are visiting Cathedral Cove, but the lower probability of rock falls in dry weather is offset by higher visitor numbers.

2.3 LENGTH OF TIME AN INDIVIDUAL IS IN THE ARCH

The plan dimensions of the arch of Cathedral Cove are approximately 50m long and between 10m and 20m wide. Based upon a length of the arch of 50m and an average walking speed of 2.57 km/hour (42.9 metres/minute), the average time each individual is within the arch is 70 seconds.

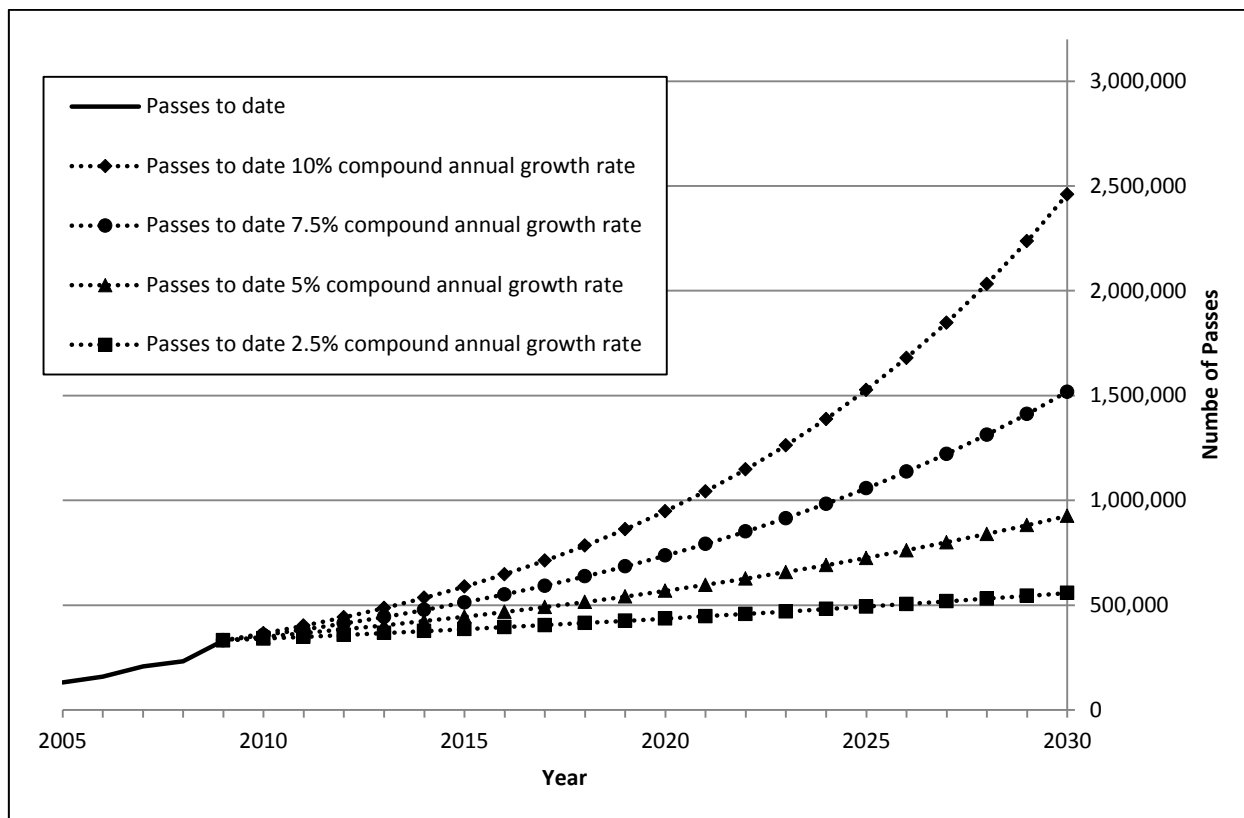


Figure 2: Annual and projected passes through Cathedral Cove Arch

2.4 ROCK FALL SIZE

The area that a rock fall impacts will govern the number of people potentially impacted. A larger rock fall is likely to strike more individuals than a smaller rock fall. DOC staff indicated that the October 2009 rock fall was the largest rock fall in the 20 years for which DOC has records. The size of that rock fall was 5m long $\times$ 1m wide $\times$ 0.5m deep. Based upon a more likely average rock fall size (4m²) and the footprint of the arch (500m²), an average rock fall has been assumed to cover 0.8% of the area of the arch.

2.5 WARNING OF ROCK FALL

Immediately prior to a large rock mass movement (e.g. collapse of rock within a quarry), noise and rock dust are generally generated, even in brittle material like the ignimbrite that forms the Cathedral Cove arch. This dust would be expected to fall immediately prior to a rock fall within the arch. The effectiveness of this warning needs to also consider the ability of the individual to identify the rock dust as a warning and to leave the arch. The probability of this warning being ignored was considered to be 50%, assuming that signage at the arch entrances will have previously alerted people to the possibility of rock fall.

2.6 IMPACT LEADING TO A FATALITY

It was considered that not all rock falls that strike an individual will be fatal. It was estimated that 50% of the all rock fall impacts of the assumed average 4m² size will cause a fatality.

3. DISCUSSION

3.1 PROBABILITY OF A FATALITY

3.1.1 Individual Risk

The annual probability of a fatality to an individual was calculated by multiplying all probabilities of the pre-conditions described in Section 2. This logic is illustrated in the event tree in Figure 3, which was created using the @Risk software programme. This annual probability of a fatality to an individual was then multiplied by the number of passes expected to be made by the most exposed individual who used the arch in 2009 to calculate the annual probability of a

fatality to that individual. A kayak tour operator is likely to be the most exposed individual and would be expected to pass through the arch four times a day on average, which equates to an annual probability of 1 in 622,665 for a fatality as a result of a rock fall in Cathedral Cove. This calculated individual risk is well below the generally accepted upper limit for involuntary (i.e. general public access) risk of a fatality of 10^{-5} (1 in 100,000) (Nielsen et al, 1994).

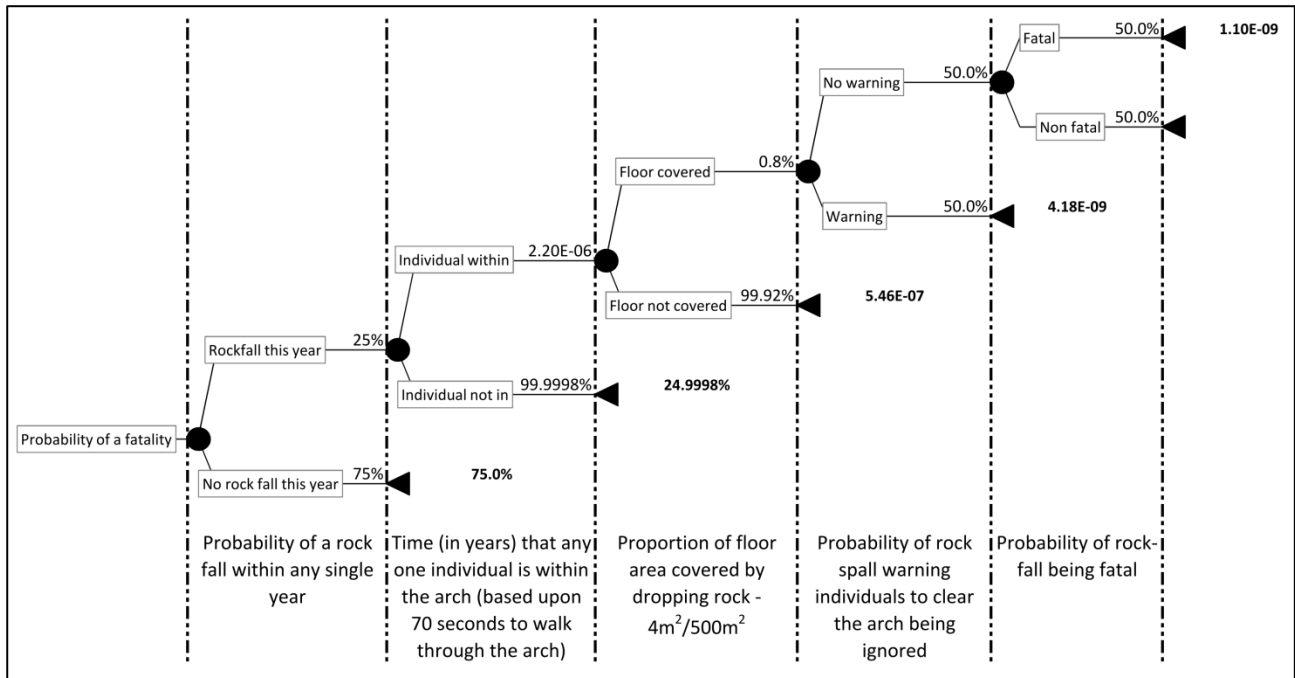


Figure 3: Annual probability of a fatality to an individual making a one way trip through the arch

3.2 GROWTH RATES

Based upon data from the visitor counters installed in 2005, the measured growth rate in visitor numbers is approximately 26%. However, at some point the maximum number of visitors to Cathedral Cove would be expected to be self-regulating. Large crowds on the beach and difficult parking would also be expected to detract from the visitor experience and minimise future growth rates. This is outlined in Table 1, which indicates that in 2009 an average 57 people use the beach every hour in summer. With growth rates of 2.5% and 25%, by the year 2030 there would be respectively approximately 2 times and 100 times the number of visitors that presently use Cathedral Cove. Incomplete counter data from 2010 and 2011 also indicate that visitor numbers have actually decreased overall from a peak in 2009. The current measured growth rate is therefore not considered to be sustainable.

Predicted annual growth figures for both tourists (NZ Ministry of Tourism, 2010) and permanent residents in the area (Ministry of Economic Development, 2010) over the next decade range from 0.5% to 6%. This range is considered to represent a more reasonable growth rate for visitors to Cathedral Cove.

3.3 TOTAL RISK

The total (or societal risk) is the relationship between the frequency and the number of people suffering from a specified level of harm in a given population from realisation of specific hazards. Although a distinct fixed population group is not at risk from rockfall in this case, the societal risk criteria adopted by the UK HSE (Trbojevic, 2005) provide a useful framework by which to gauge the relative effect of the anticipated growth in total visitor numbers on the annual probability of a fatality in the arch. The generally accepted upper tolerable limit for an incident in the UK where one fatality is the result is 1 in 1,000. Using an annual growth rate in visitor numbers of 2.5%, the probability of a fatality among the total visitor group is expected to remain well below this upper tolerable limit for societal risk in the near future, as shown in Figure 4.

Growth Rate		2009	2030
2.5%	Visitors / year ⁽¹⁾	182,807	307,039
	Visitors / summer daylight hour ⁽²⁾	57	96
25%	Visitors / year ⁽¹⁾	182,807	19,819,975
	Visitors / summer daylight hour ⁽²⁾	57	6,206
Note (1) Based upon 365,614 passes of people through the arch in 2009 i.e. 166,188 visitors+10% marine users			
(2) Average based upon present proportion of summer use of 42% and 91.25 days in summer with 14 daylight hours per day			

Table 1: Projected growth in visitor numbers to Cathedral Cove

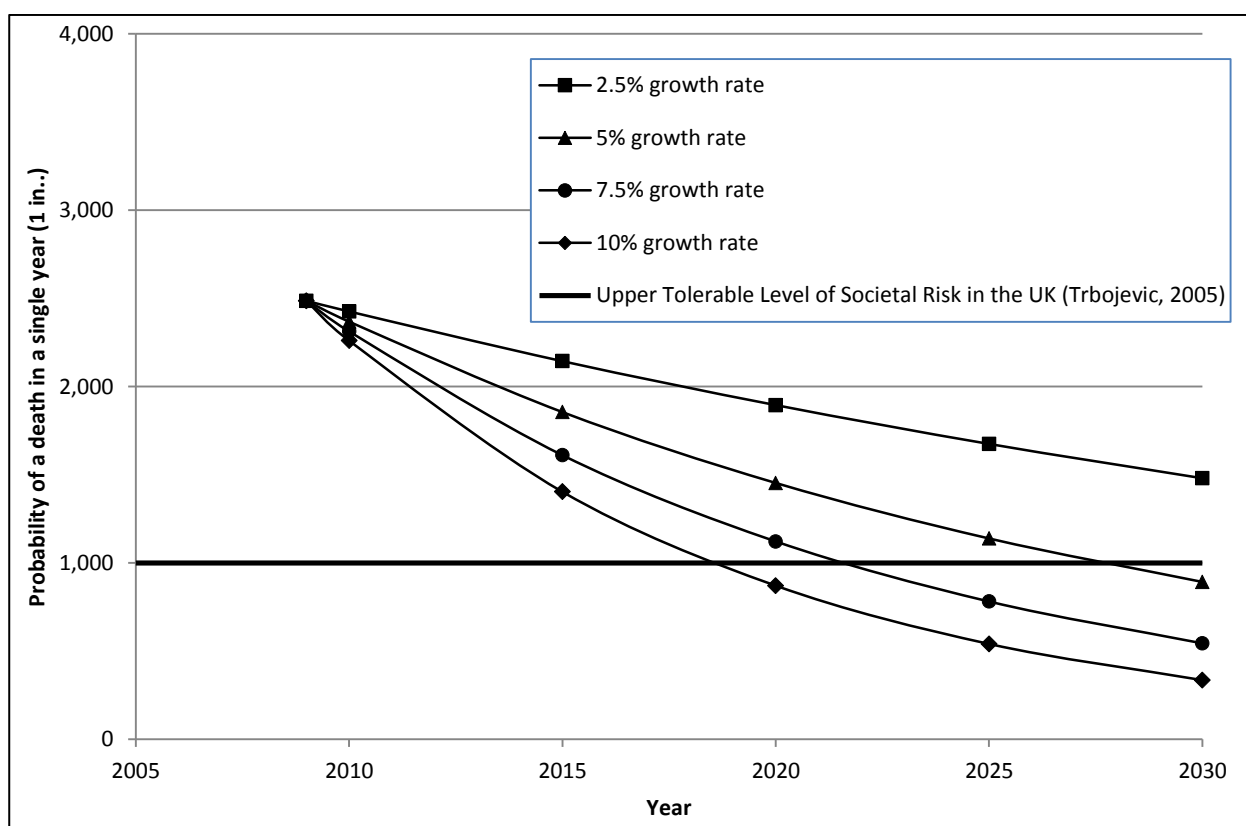


Figure 4: Calculated annual probability of a fatality within the arch for the total visitor group

3.3 REMEDIAL WORKS

The risk assessment enabled the relative risk of a rock fall fatality in Cathedral Cove to be compared with international standards, and also to assess the reduction in perceived risk by implementing remedial options. The calculated individual risk is currently within published international acceptable limits for a rock fall fatality. Projected growth figures also indicate that the annual probability of a fatality in the arch is not expected to exceed the generally accepted upper limit for societal risk of a fatality (1 in 1,000) in the near future. However, the arch is an actively eroding feature and the factors which affect the determination of the risk to the public may therefore change with time.

DOC therefore made the decision to proactively monitor and manage the risk in the arch, and requested T&T to investigate options for reducing the perceived rock fall risk. Options assessed included hard engineering fixes such as rock bolts and/or shotcrete which would minimise the rock fall hazard potential within the arch, as well as various options for discrete remote monitoring of perceived rock fall hazard areas. However, it was ultimately decided that

man-made engineering works would result in an excessive visual impact on the arch, and the brittle and rapid nature of rock failure in the arch precluded implementation of an effective remote monitoring system.

Low impact scaling of blocks within the perceived hazard areas in the arch was undertaken in November 2011 using specialist rope access contractors. This option was considered to have the least visual impact on the arch while also reducing the perceived risk. It is considered that the scaling works reduced the rock fall hazard risk within the arch to a level lower than that which existed prior to the recent series of rock falls which started in October 2009. This level of risk was considered acceptable enough for the arch to be subsequently re-opened to the public.

The arch will be subject to ongoing natural weathering and erosion and future rock fall events may occur unpredictably. Accordingly, the factors which may affect the likelihood and consequences of the rock fall risk in the arch may change with time. T&T have therefore implemented an ongoing monitoring program in conjunction with DOC which involves regular visual inspections of the arch for management purposes. The warning signs at the entrances to the arch were amended to reflect the remaining unpredictable rock fall risk. Visitor data also continues to be collected and reviewed annually to more accurately assess the annual growth rate in visitor numbers to the arch and to calculate any changes to the probability of a fatality as presented here.

4. CONCLUSIONS

Information gained from detailed geotechnical inspections was used in conjunction with visitor data provided by DOC to undertake a quantitative rock fall risk assessment for the arch at Cathedral Cove. This study provided a rare opportunity to assess a rock fall hazard risk in a natural “tunnel” where rock falls are typically from overhanging faces and where the general public regularly pass through the areas of highest perceived rock fall hazard.

The risk assessment indicates that, at present, the annual risk of a fatality due to rock fall within the Cathedral Cove is below the generally accepted upper limit for individual risk of a fatality of 1 in 100,000. However, due to the unpredictable and actively eroding nature of the arch structure, it was decided to proactively manage the risk by undertaking low impact scaling of areas of perceived rock fall hazard areas within the arch. The scaling works were considered to have reduced the rock fall hazard risk within the arch to a level lower than that which existed prior to the recent series of rock falls.

Despite the preventative measures undertaken, a rock fall event of relatively low probability could still occur at any time. As such, appropriately worded signs warning of the remaining rock fall hazard have been placed at both entrances to the arch, and a collaborative monitoring program has been initiated to manage the rock fall hazard risk.

The methodology used in this quantitative rock fall risk assessment could readily be applied to other natural environments where data on public use is available. Although a degree of uncertainty is inherent in any such risk assessment, the information obtained may be still used to inform decisions on the management of rock fall hazards, particularly where growth in general public use is anticipated in proximity to such hazards.

5. ACKNOWLEDGEMENT

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ANALYSIS OF FILLED STOPES IN OPEN PIT MINING

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ABSTRACT

The Black Star Open Cut (BSOC) Mine of Xstrata Zinc in Mount Isa, Queensland, started production in late 2004 for the extraction of zinc, lead and silver left behind from the old underground Isa Lead Mine.

The relatively new concept of mining through historic underground workings was a vast safety and geotechnical challenge faced by the BSOC mining engineers. Therefore, new methods for safe extraction of ore surrounding the workings and methodical approaches to mining have been developed which allowed for safer and more efficient mine production.

Due to the extensive mining history, stopes from the Isa Lead Mine were found to vary substantially with sizes of over 100m deep and 200m in length. Although most stopes were backfilled, the status of the fill could not be fully determined and inconsistencies were often found. As such, even once the stope crown had been successfully mined out and the fill exposed, the BSOC mining team could not be confident that travelling over stope backfill would be safe.

Until recently, filled stopes were strictly inaccessible to personnel and light vehicles. Safety concerns over the fill status meant that production in these areas slowed, and the need for a safe approach to crossing filled stopes was raised. It was found that by assessing and analysing the geometry, fill status and history of each stope individually, particular processes could be established to ensure a safe crossing over a filled stope. The paper will briefly present the history of BSOC mining, addressing the safety concerns in relation to crossing filled stopes as well as presenting approaches and analysis techniques used by BSOC engineers with the aid of case studies.

INTRODUCTION

In late 2004, the Black Star Open Cut (BSOC) Mine was developed for the extraction of zinc and lead ore left behind from the early underground Isa Lead Mine. In the establishment of the BSOC Mine, the process of mining into old underground workings was the highest safety and production risk associated.

During the early stages of the BSOC mine development, geologists and engineers continued to develop different methods for the analysis of the structural stability of underground workings to mitigate production and safety concerns. Due to the relatively new concept, methods were developed from mines with similar experience and adapted to local conditions.

As mining continued deeper, the complexity and size of underground workings became greater, with stopes over 50m wide and 200m deep being common. In the early stages of mining, stopes were generally small enough that production could occur in and around until the stope was completely mined out. In the later stages of development and present mining, stopes are so large that they will never be fully extracted through open cut mining. Due to this reason filled stopes present large production problems with inaccessibility.

This paper presents an overview of the mining at Mount Isa, safety involving filled stopes, steps taken by engineers at the BSOC Mine for the assessment of filled stopes and case studies for a 'stope crossing'.

MINING AT MOUNT ISA

1. LOCAL GEOLOGY

Geologically, Mount Isa is a very complex environment with many different stages of metamorphism making up the surrounding rock. The lead/zinc ore is found within a rock zone known as the Urquhart Shales. This rock zone is comprised mainly of dolomitic siltstones and shales with various intrusions of quartz. The ore is mainly found sedimentarily recrystallised in the shale, but is also commonly found within quartz intrusions. Although complexly metamorphosed, the bedding through the BSOC mine generally dips at 70 degrees to the west and as such the ore bodies as well (Conaghan 2003).

2. HISTORY

Mining at Mount Isa has been almost uninterrupted since the discovery of lead ore by John Campbell Miles in 1924. Early on, lead in carbonate form was mined on the surface through trenches developed in the ore bodies, but due to the steeply dipping ore bodies, was quickly moved to underground production, Figure 1 (Conaghan 2003)

The underground Isa Lead Mine is part of a network of mine workings that have been mined since the early 1920s. Due to the age of the mine, upper levels sometimes varied and were developed using a range of methods, including glory hole mining.

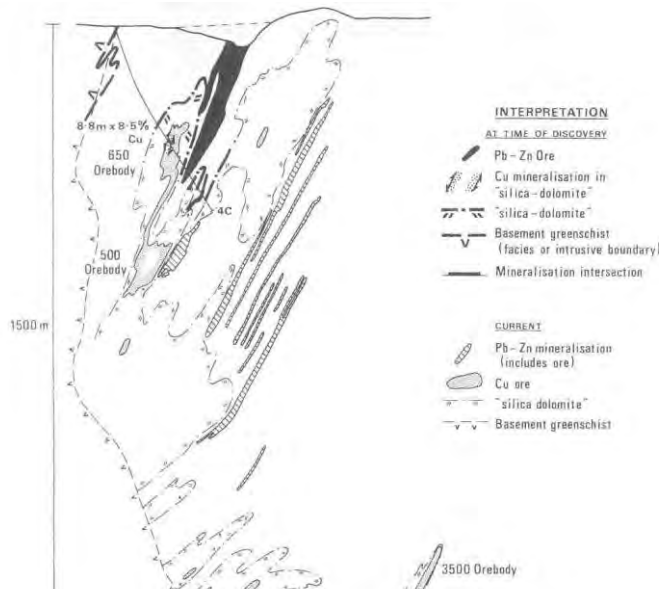


Figure 1, Common cross section at Mount Isa showing steeply dipping Pb, Zn ore bodies

As the mine progressed, sub-level open stope mining was the most commonly used method for extraction.

Once ore was extracted it was sent to the surface for processing. When a stope was completely mined out it was generally backfilled. Backfilling commonly occurred to maintain ground stability subject to backfilling material type (Grice 2008), but also in many cases so that stope pillars could also be extracted.

Fill types used in completed stopes were chosen depending on many different factors including access, fill availability and structural requirements. At Mount Isa, four types of fills were used which included; hydraulic, aggregate, cement aggregate and mullock. Due to the processing of ore within Mount Isa, hydraulic fills were commonly used due to the ability to fill quickly and as a way of recycling waste tailings.

COMPLICATIONS OF MINING THROUGH UNDERGROUND WORKINGS

Complications that result from mining through underground workings are different between each site or mine. This depends on specific ground conditions as well as underground and open cut mining methods, extents and scale of operations.

At the BSOC mine, the scale of underground mining is extensive, with many different methods of extraction completed through the underground mine life including sub-level open stoping (SLOS), sub-level caving and cut and fill mining. As excavation in the open cut mine has become deeper, the complexity of underground workings has become greater. Mining through underground workings poses both safety and production risks, although the greater the risk to safety, the greater the effect on production and mine sustainability.

1. ASSESSMENT OF RISKS

At BSOC, underground workings that intersect the mine are assessed individually to determine the safest technique of mining both around and through. Thorough assessments of underground workings that intersect the mine surface are divided into two distinct groups depending on their risk. The assessment of workings specifically reviews the history, previous investigations carried out, structural stability and consequences of collapse. Depending on the investigations and recommendations from the Voids Department, each working is delineated accordingly to indicate the risk.

The history of underground workings can be learned through different resources. Archives of information regarding ring firing plans, fill notes, geology notes and planning documents present a good indication of the ground conditions and any problems that may have occurred through excavation. Information through assessing horizontal development plans can indicate bulkhead locations as well as the process by which mining took place. Geotechnical information such as rock strengths, faults, and shear zones previously recorded through diamond drilling and past wall mapping presents information that can be used to assess possible failure zones or areas of instability. Historical information creates an archive of information that can be proven against current data to enforce known aspects of the working, Figure 2.

Investigations carried out in the open pit environment are one of the key principles to void investigation and assessment of risk. Many investigations are carried out depending on the specific situation to prove the historical information obtained. Probe drilling is one of the most common forms of investigation used in void analysis; this involves drilling into specific parts of a working and noting the ground conditions and depth at which a breakthrough is encountered. Such information can then be used to either prove or invalidate the historical data.

The assessment of the structural stability of mining into underground workings has been developed during the various stages of mining at the BSOC mine with specific geological and geotechnical conditions in mind. The first analytical method of assessment looks at two different situations. The first analysis is the structural stability of the overlying rock and the cover of solid rock needed above a void to be considered safe. This method involves the use of geotechnical information obtained for the host rock and the assumed unsupported strength above a void. The second analytical method also involves the use of geotechnical information to look at the consequence if the solid cover was to collapse into the void space and how far this collapse would progress to the surface.

Through this analysis, the Voids Department can achieve a better judgment on the safety of specific voids and the interaction with open pit mining.

2. PRODUCTION RISKS

Through analyses completed of particular underground workings, the review of risk to personnel, light vehicles as well as heavy vehicles needs to be assessed individually. Due to the risks for each void, certain limitations are imposed on personnel, light vehicle or heavy vehicle access, depending on the situation. Due to these limitations, impedance on production is created and the need for analysis techniques and processes to reduce these limitations needed to be formed. With the open cut mine becoming deeper and the stopes becoming larger, the need for these analysis techniques is increasingly important, Figure 3.

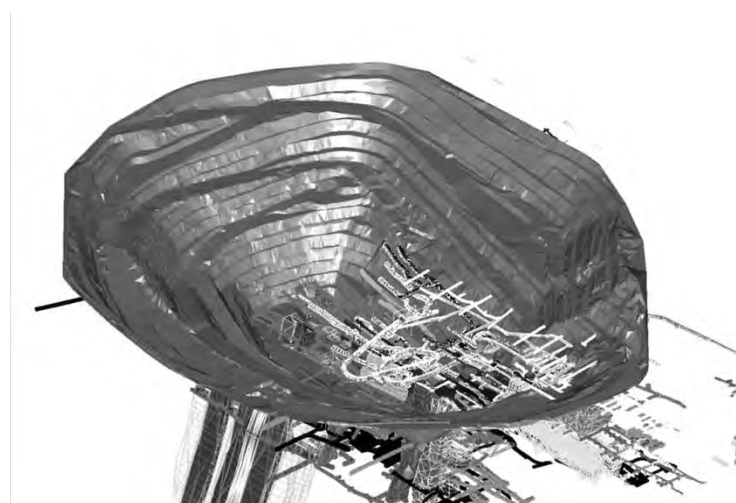


Figure 3, Underground workings intersecting the BSOC mine

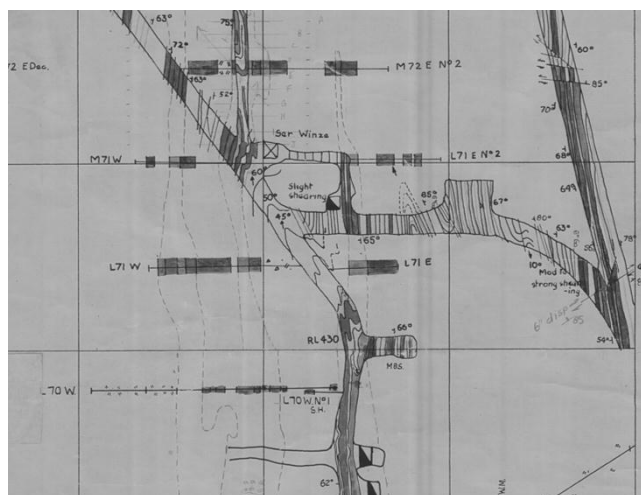


Figure 2, Common historical level plan

ASSESSMENT FOR CROSSING FILLED STOPES

Although mining can continue through filled stopes, risks are still present. As such, all filled stopes open to the surface limit access and therefore impede on production. This causes many issues including access for servicing of machines, operator pickup, surveying controls, geology grade mark outs and many other technical support and operational controls. Due to this, the Voids Department, in coordination with Geotechnical Department at BSOC, developed a way of assessing filled stopes in an attempt to create a “stope crossing” so that access could be allowed over a stope in a safe manner.

1. GENERAL CONSIDERATIONS

The process of investigation for a “stope crossing” involves proving the known aspects of a stope and assessing the geometry and fill types. Stopes of the Isa Lead Mine were generally well recorded, with plans of the geometry, fill type

and access. Although well recorded, experience has shown that many of these factors can be either incorrect or change over time; therefore each aspect needs to be proven before proceeding.

Common ways used by engineers for proving the known aspects of a stope include the use of probe drilling to confirm the geometry and fill type. Drill operators are specifically trained to determine when a breakthrough is encountered and what sort of fill material is found. This proves that the stope is on design and that tight fill is observed. Probe holes are generally designed to intersect specific parts of a stope that may intersect voids or areas that may be off design.

The assessment of the stope geometry plays a major part of stope assessment. Due to the way that stopes are filled, cavities can be common when complex geometries are encountered. Kick-ins or bell-outs on the stope walls are perfect conditions for cavities to occur due to fill not taking up these areas completely. When stope geometries involve too many complexities, it is generally not allowed to be crossed due to the risk.

2. GEOTECHNICAL CONSIDERATIONS

Investigations into the structural stability of fill in a stope are the most complex part of assessment. Although fill material may visually seem stable on the surface, its conditions below the surface present risks that can never be fully proven. In the assessment of backfill material, the aim is not to eliminate hazards, but to reduce them as low as reasonably possible. Different fill types present several different concerns and have to be assessed carefully.

Hydraulic Fill

Hydraulic backfills are composed mainly of silty sands and sandy silts free of clay fraction (Sivakugan, 2004). In the placement of hydraulic fills, porous bulkheads are installed on horizontal access drives. When slurry enters the stope, it quickly settles and decant water progresses to the top. Important design aspects of the drainage of the decant water are taken into account.



Figure 4, Dump truck tracks bogging in stope fill

The strength and stiffness of hydraulic fills is directly related to the density of the fill, and therefore has greater stability at higher densities (Sivakugan, 2004). Grice (1998) states that after full draining, a hydraulic fill has a porosity of around 50% and can be walked upon within 24 hours.

Although strength of freshly dried hydraulic fill is able to take load quickly after placement, the time lapse between the stope being filled and the time to be mined out in the open cut is an important element. Due to up to 50 years of backfilling at Mount Isa, the strength and competency of hydraulic fills cannot be assumed the same as that of fresh fill (i.e. most likely it is higher).

Information on fill density and condition can be obtained through investigation methods such as geophysical scans, yet the cost and time associated with these methods is not always viable in a mine situation. Therefore, at the BSOC mine a standard crossing over hydraulic fills has been developed to incorporate the removal of hydraulic fill to expose both hanging and footwall. Through experience, an excavation depth of 4.0m has been proven to be a sufficient depth to allow satisfactory compaction of the rock fill in 1.0m lifts. Blasted waste rock with varying sizes between 100-500mm and unconfined compressive strengths (UCS) between 100-150MPa is used as a backfill material. During the backfilling process 190 tonne trucks dump waste material into the excavation which aids in compaction.

Since this technique has been utilised, no issues with bogging of machinery or surface subsidence has been observed.

Cement Aggregate Fill

Cement aggregate fills (CAF) are composed differently between sites. At Mount Isa, a mix of heavy media reject (HMR) from processing, crushed silica, Portland cement and fly ash from smelting, are generally used. Stopes are filled in lifts to prevent excessive pressure on lower bulkheads. Typical 28 days unconfined compressive strength of CAF is approximately 1.5MPa, and can be exposed in large areas on several sides (Grice, 1998). At Mount Isa, CAF stopes are only used when successive stopes are planned on each side and pillar mining is to occur.

Due to the strength of such fills, investigations into the use of CAF stopes as traffic crossings are fairly simple. Firstly successive stopes on each side of the CAF stope are investigated to ensure the CAF did not fail at any time during the extraction of pillars. This indicates that the stope was tightly filled and that the strength of the fill was adequate. The second investigation ensures that tight fill below the crossing is proven by drilling probe holes directly into the fill. Thirdly, when designing a crossing, offset from adjoined stopes need to be considered to prevent potential shear failure of the CAF due to excessive loads of mining machinery.

Mullock Fill

Mullock fill is generally an unspecified fill, usually a mix of waste rock with varying particle size. Due to its unknown characteristics and varying particle size, possible washout of fines could create void spaces and surface subsidence. Experience with mullock filled stopes at the BSOC mine have indicated that void spaces are quite common and fill can vary substantially throughout individual stopes. Due to the uncertainty of mullock fills, crossings are not allowed.

Once a stope has been investigated and determined safe to cross, continual monitoring of the crossing for subsidence or surface ponding of water after rain events may change the structural stability and need to be continually assessed.

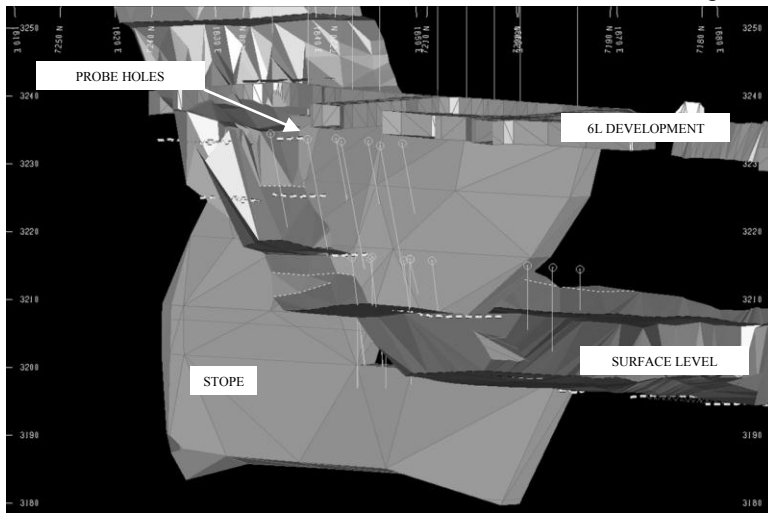


Figures 5, Stope fill subsidence and washout after heavy rainfall

CASE STUDY

1. 5F F 7L STOPE

The 5F F 7L stope is 85m long, 5m wide, 50m deep and dips to the west at 75 degrees. The crown is connected to the 6L development and its floor connected to the 7L development. Stope closure records note that 52,848 tonnes of material was extracted with a total volume of 11,900 m³. The stope has also been designed with a pillar located around



Figures 6, 5F F 7L stope and associated probe drilling

half way down the stope and is filled with hydraulic fill up to the floor of the 6L development. Mine plans show permeable brick bulkheads on the 7L development to confine the hydraulic backfill.

As a proof of known aspects, probe holes were drilled to confirm the hanging wall location and fill status of the 5F-F-7L stope from the surface. From probing data acquired, it was seen that the actual stope was less than 1m off design and competently filled.

In preparation for a stope crossing a further 8 probe holes were drilled to confirm the fill status and hanging and foot walls at specific locations. Probing data obtained showed no discrepancies between the actual and design of the stope geometry. Probe holes designed along the footwall of the stope did not

breakthrough at any point, showing that the footwall has also little variance from design.

Due to the pillar that runs approximately through the centre of the stope, concerns of a stope crossing over this area had arisen. When overhangs or pillars are present in a stope, filling cannot be fully confirmed due to the nature of the specific fill. If the stope was filled with an aggregate or mullock fill, certain areas would not be sufficiently filled due to the rill angle of the material and its inability to flow. Due to the confirmed hydraulic fill in the 5F F 7L stope, this was not an issue due to the high flowability of hydraulic fill and its ability to fill spaces unlike suggested previously.

To ensure structural stability on the surface, the hydraulic fill in the stope was mined out to 4m below the blasted level. The hanging and footwalls were completely exposed through excavation. A survey of these walls indicated that they were approximately 0.25m off design.

After excavation was completed, the stope was backfilled using competent waste rock in 1.0m lifts to ensure sufficient compaction from heavy machinery. Due to the relatively narrow width of the stope, the backfill was highly confined between the hanging and footwall. As a result of such confinement, risk of backfill movement, fill washout and potential surface subsidence were determined to be negligible.

As a result of the investigation into the 5F F 7L stope, it was determined that a crossing could be safely initiated.

CONCLUSION

The geology at Mount Isa although complex has been mapped to provide good detail of rock conditions. Due to the extensive history of underground mining, awareness of the mining methods and potential deterioration of void conditions over time through various causes (i.e. fill erosion, water infiltration, blasting) should be addressed.

The risks involved with mining through underground workings can be severe if not managed properly. It is crucial that Voids Engineers understand the possible failure mechanisms and structural stability of voids. Although underground workings may be well recorded, time and data inconsistencies can dramatically change and investigations need to be consistently proving known data.

Pre-crossing investigations need to specifically take into account the history of stope excavation and method of extraction. Development of new technologies for the investigation into fill composition will allow for greater proof of fill consistency. At this stage, the method currently adopted at BSOC has proven to be a safe and feasible process.

While production impedance needs to be reduced as little as possible, it should not be at the expense of risk to personnel or mine sustainability. Each specific site or mine needs to specifically develop analytical and practical approaches to meet their specific site conditions.

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SLOPE STABILITY ASSESMENT AT AN OPEN PIT MINE IN NORTH-WEST QUEENSLAND

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ABSTRACT

Century Mine located in North-West Queensland is the largest open pit zinc mine in Australia. The mine area has complex geology with several major faults intersecting the pit, where difficulties have been encountered in the stability of the pit walls. Of particular importance to the mine's continued progress is the Century Main Pit South Wall. The South Wall has experienced persistent stability problems and considering the structural geology of the area these problems might continue further down the wall. Mining One Pty Ltd is engaged in the geotechnical investigation of the site to better understand the structural geology and provide workable solutions to allow mining to continue to full depth. This paper describes the structural evolution of the wall and the outcomes of geotechnical investigation.

1. INTRODUCTION

Century Mine is located near Lawn Hill in North Queensland, approximately 250 kms north-west of Mount Isa, close to the Northern Territory border. It is the largest zinc open pit mine in Australia and the second largest in the world with the capacity to produce approximately 500,000 tonnes of zinc concentrate annually. Geotechnical investigation of the area started in early 1990s and the mine has been in operation since 1999. The current ore deposit is estimated to continue production until 2016. Century comprises two sites – the mine and processing operation at Lawn Hill, and associated concentrate dewatering and ship-loading facilities at the Karumba Port, on the Gulf of Carpentaria.

The deposit is located within a geologically complex area, bounded by many faults through soft sedimentary rock. The soft nature of sedimentary rocks lends themselves to intensive and complex deformation around major faulting features. As is common in mining projects, initial exploration efforts at Century focused primarily on the mineralisation of the deposit with few fully cored, oriented geotechnical holes targeting footwall lithology where the pit walls are planned. The lack of a good structural model in some areas has resulted in unplanned wall movements, costly remediation and redesign of the walls. Century Zinc Mine is an excellent case study for how gaps in geotechnical data can lead to ongoing, costly stability issues. This paper focuses on the stability problems encountered in the south-west wall of the Century Pit which is now known to comprise a major fault, several smaller scale faults and a structurally rotated block.

2. BACKGROUND

2.1. Site geology

Century deposit is a major sediment-hosted zinc, lead and silver deposit formed within the Proterozoic age (1640 Myr) of upper Lawn Hill Formation (Broadbent et al., 1998). It is partially overlain by Proterozoic Sandstone and Cambrian Limestone. The orebody is located in a structurally complex area, adjacent to the major north-west trending Termite Range Fault as shown in Figure 1. The Mine is divided to two main blocks by Pandora's Fault, which is an east trending and north dipping normal fault (Figure 1(a)). The northern block is bound to the north-west by the south dipping Nikki's Fault and is located in the Termite Range Fault Zone, overlain by the Cambrian Limestone with an unconformity. The southern block is bound on the south by the north dipping normal Magazine Hill Fault, on the east with an unconformity against Cambrian Limestone and to the west is a present day erosion surface (Waltho and Andrews, 1993; Waltho et al., 1993). The structural geology of the deposit is further complicated by small scale folding of the orebody into a north-south synclinal structure. Mineralisation is stratabound within the sedimentary sequence, and is mostly shallow dipping, lying from 0 to 350m below surface, and occurs as interbedded black shale, grey siltstone and brown sideritic siltstone.

Typical stratigraphy of the deposit consists of Cambrian Limestone, carbonate breccia, Pre-Cambrian weathering surface, Widdallion Sandstone, hanging wall shale-siltstone, mineralised sequence, footwall shale-siltstone, black carbonaceous shale, Bulmung Sandstone and black shale (Figure 1(b)) (Andrews, 1998; Waltho et al., 1993). Large blocks of Lawn Hill Formation material have been emplaced into the limestone by thrust faulting. Irregular veins of carbonate breccia are intersected throughout the Cambrian and Proterozoic units and can be found commonly in fault zones. The carbonate breccia contacts in the Proterozoic rocks are often sheared, and are thus thought to be intruded along pre-existing structures (Waltho and Andrews, 1993).

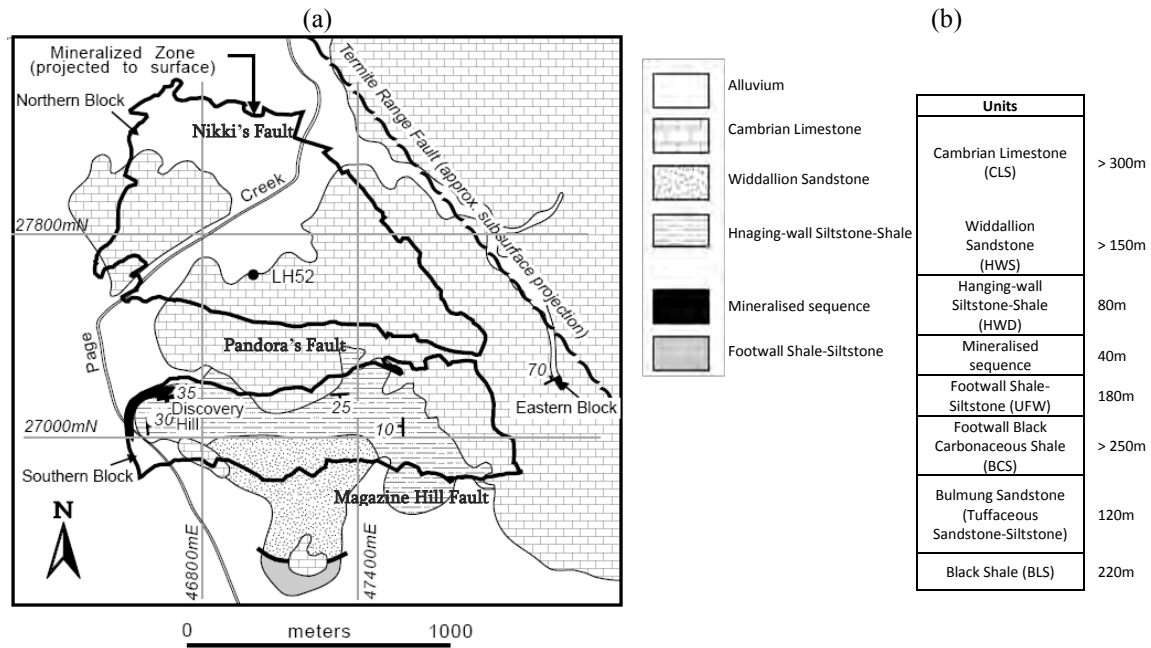


Figure 1: Century deposit (a) local geology (after Walther and Andrews, 1993) and (b) stratigraphy (after Andrews, 1998 and Walther et al., 1993)

2.2. Pit geometry and study area

During the operational history of the pit, there have been some ongoing geotechnical challenges regarding the stability of the north-west and south-west wall of the pit, identified as High Geotechnical Focus areas in Figure 2. Both these stability problems are associated with presence of major faults in the vicinity of the walls; Nikki's Fault near the North West Wall and Pandora's Fault that intersects the south-west wall. Mining in the North wall has now been completed and around the south-west wall is currently ongoing. Several small faults that splay off from Pandora's Fault structure within an associated zone of disturbance in the footwall have led to unexpected failures, causing delays in production. Mining One Pty Ltd is engaged in the geotechnical investigation of the site to better understand the structural issues and provide workable solutions to allow mining to continue to full depth.

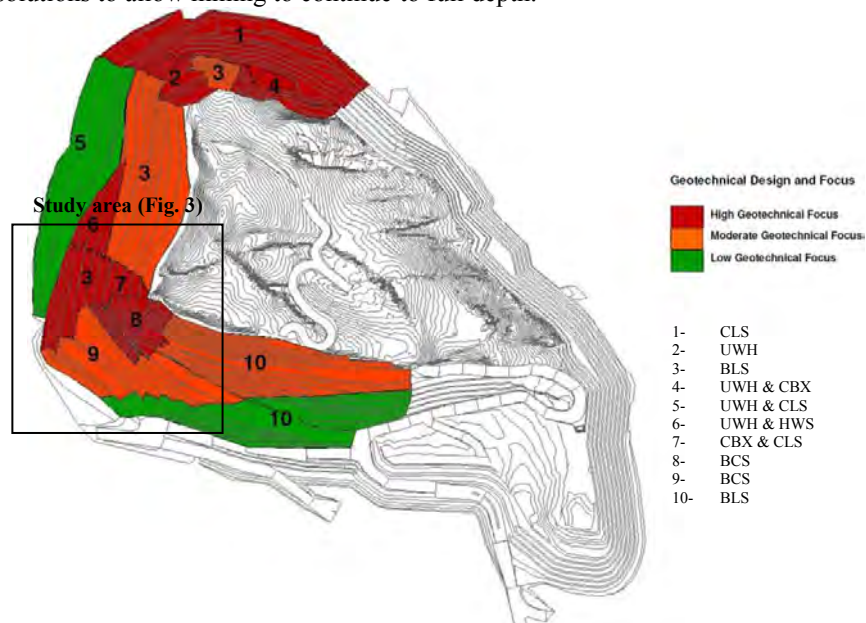


Figure 2. Pit geometry and geotechnical domain map

2.3. Geotechnical setting of the South-West Wall

The south-west wall is dominated by black laminated shales (BLS) and black carbonaceous shales (BCS). In the study area, the BCS is observed as low-strength carbonaceous shale with bedding partings, continuous joints and occasional bedding shears; whereas BLS is more competent rock with distinct laminated bedding.

The dominant structural feature in the south and south-west walls is Pandora's Fault. This fault strikes east-west through the south of the pit and curves to the north as it intersects the west wall. It dips north to north east at approximately 20° to 40°. The fault throw increases from east to west in the margins of the deposit and is characterised by the sporadic presence of intrusive carbonate breccias as well as clay coated shears and broken zones. The location and orientation of the western end of the fault has previously been modelled on fairly sparse exploration drillhole information, which recent drilling has shown to be inaccurate. The true location of the fault is estimated to be approximately 10m further south, nearer to the proposed final pit wall. In addition, recent geotechnical drilling data indicates that there is a zone of disturbance in the foot wall of Pandora's Fault including several splay faults. The extent of this zone of disturbance and the trend, dip and extent of splay faults have not yet been fully established as holes were terminated within this zone. This zone has typically experienced poor core recovery due to the fragile nature of the rock in these fault zones. As a result, very little data has been obtained to specify trends and dips of faults found in core below Pandora's Fault.

Century Mine South-West Wall has several dominant pervasive structures, the orientation of which changes spatially across the deposit. One such structure includes bedding which was initially logged to typically trend around 45°/090° (dip / dip direction). However, post excavation (after 2010 failure) revealed that this trend is manipulated around major structures such as Pandora's fault and can vary in dip and strike by as much as 20°. Three dominant joint sets have emerged in the south-west wall. Table 1 shows the results from an infill drilling program held in 2010 targeting these defects.

Structural Set	Dip	Dip Direction
Bedding	30	093
Joint Set 1	51	187
Joint Set 2	51	277
Joint Set 3	67	351

Table 1. Pervasive structures (revealed after 2010 drilling)

In the following sections, the data collated from mapping and drilling are presented and used in a kinematic assessment of the current wall design to understand the extent of failure that could be anticipated from planar failures alone. Defects were given a nominal friction angle of 30° based on site experience with failures on similar features. It should be noted that occasional bedding plane shears will have a much lower friction angle than has previously been estimated, with approximately 20° to 25°.

3. STRUCTURAL EVOLUTION AND STABILITY OF THE SOUTH-WEST WALL

3.1. Original design of the South West Wall

In 2007 the Century mine design incorporated a chamfered southern wall that merged on a north westerly trend into the west wall. At that time the wall and mining plan were designed to inhibit movement on Pandora's Fault, which was planned to lie behind the south wall, by ensuring the wall had sufficient weight at the toe of the fault to prevent unplanned movement. Bedding in the west wall to the North of Pandora's Fault was dipping to the east out of the face at approximately 45°. This wall was designed with batters parallel to the bedding dip (termed "Batters to Bedding") and the wall was depressurised with closely spaced drainage holes across the batter. This design had been trialled through previous mining stages to the north and had worked very well. Thus, the previous design adopted a similar approach for the South-West Wall and batters were designed with 12-24m batter heights, 75° batter slopes, and 9m benches. However, since mining commenced in the south-west wall area, there have been several stability issues caused by structural variations that were previously unknown that resulted in two major wall re-designs and remediation practices. As the mine progresses deeper, the structural geology is becoming clearer from exposures. The design has had to be altered several times during mining to account for unknown structures.

3.2. History of Failure Mechanisms and Re-design/Remediation

Late 2007 to December 2008

Once Fresh rock was uncovered during the initial excavation, a trend of small scale wedge failures emerged on the south-west wall. Wedges were formed between Joint Set 3 and the bedding planes. The failure mechanisms of these wedge failures are shown in Figure 4(a). This was managed at an operational level with site Geotechnical Engineers identifying wedges as they emerged and having them mined out. By July 2008 wedge failures were occurring over several benches and in October 2008 cracking aligned with bedding trends was found at the natural ground surface to the south of the pit wall. Cracks extended up to 40m from the pit crest indicating that a deep-seated wedge failure was developing. At that stage, it was apparent that mining in this area could not continue to the current design. However design alterations were restricted by current approval limits.

January 2009

In January 2009 part of the wall nearer the pit crest experienced a wedge failure (Figure 3). This was triggered by significant rainfall, totalling 279mm in five days. This failure led initially to a design of an in-pit fill buttress to stabilise the failure and prevent any further movement, until approvals could be obtained for a cutback of the wall. The mine would incur significant unrecoverable costs from leaving ore in the ground if a cutback was not approved. During 2009 a new design was developed and put through the approvals process to cut back behind Pandora's Fault, effectively removing the geotechnical risk associated with the fault and allowing the strike of the south wall to be altered so the mine operations could manage the geotechnical risk associated with wedge failures.

January to May 2010

In January 2010 excavation of the South Wall Cutback design commenced. By March a planar failure had occurred across two batters on bedding in the west wall that found a release on Joint Sets 1 and 3 (Figure 4(b)). An investigation into the failure indicated that bedding in this area south of the fault was unexpectedly flatter than the typical bedding observed on the wall north of Pandora's fault. Bedding was now shallower than design and daylighting in the face of the north striking wall. As excavations continued several minor faults were mapped in the west and south walls that led to small scale failures.

In May 2010 when excavation had progressed 80m below natural surface, a major failure occurred on a bedding shear that was approximately 15° dip shallower than was anticipated, and approximately 20° different in strike. The failure extended the full length of the west wall. The southern side of this failure was bound by Page Creek Fault which had been mapped in the wall. However, trends on the fault were difficult to obtain. At this time re-design options were limited as another cutback behind this failure would be costly and difficult to get approved due to the close proximity of the Page Creek. A risk assessment indicated that the failure was likely to propagate further and intersect the Page Creek Diversion if the wall continued as designed. The alternative that was taken was to change the wall orientation back to the south westerly strike and step out in front of the failure.

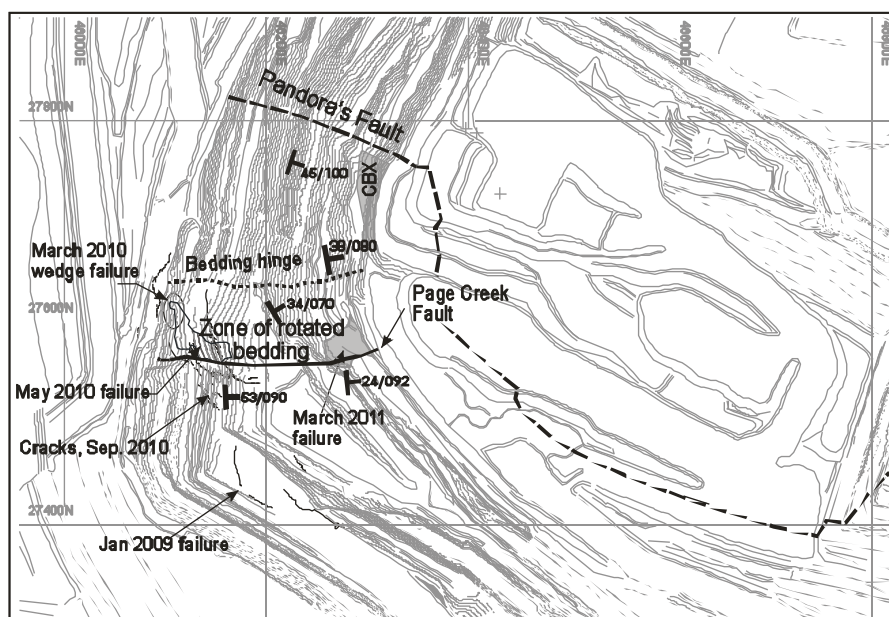


Figure 3. Ground movement history and current situation of the South-West Wall

After the bedding shear failure in May 2010, an investigation commenced to better understand structures at depth. It was revealed that there was insufficient geotechnical data collected in that section of the pit. To address this gap in the data a new drilling program was initiated to get a better understanding of the orientation of bedding at depth, and the locations of fault splay structures. This drill program was completed in May 2011. Many of the drillholes were not able to achieve full depth and had to be terminated in broken ground with poor recovery. The data collected clearly showed:

1. Bedding trends at depth south of Pandora's Fault remained shallow 25° - 35° ;
2. Pandora's fault is estimated to be 10m further South than originally anticipated and would now intercept the current design wall; and
3. Pandora's fault is significantly more structurally complex than originally thought. The data indicates that as Pandora's Fault curves to the North, splay faults have formed off the fault to accommodate the change in trend. These faults, in particular Page Creek Fault resulted in a rotated and deformed block that is highly sheared and broken that would intercept the designed south wall. These deeply seated structures within the wall had not been analysed and accounted for in the current design. The trend and continuity of these structures was difficult to determine prior to current exposures and detailed drilling.

The kinematic analysis shows that planar failure is highly likely with 56% of defects dipping out of the face steeper than the friction angle in a slope oriented dipping at $75^{\circ}/060^{\circ}$, see Figure 4(b). For clarity, the figure does not show all the data. Instead, an average bedding orientation of $30^{\circ}/090^{\circ}$ is shown.

The information gathered from 2010 drilling program was not able to be used to predict and prevent the pit wall failures of 2011. However, with the latest drilling, described below, it will assist in developing an improved remediation solution for the south west corner.

From May 2011 to early 2012

Excavation continued along the new strike in the south-west wall, and small wedge failures re-emerged. These were continually managed by operations by pulling them down as they became apparent in the face. Another concerning trend that emerged was the rotation of bedding planes by 20° to the north around Page Creek Fault. Their orientation of approximately $34^{\circ}/070^{\circ}$ allowed them to daylight in the batter as the wall orientation was $75^{\circ}/060^{\circ}$. In March 2011 a further 40m down (120m from natural surface) another failure occurred on the south-west wall. The location of this failure can be seen in Figure 3.

A second drilling program was then launched in 2011 to further assess the extent of the zone of disturbance in the footwall of Pandora's fault and specifically targeted the rotated block discovered in 2010 drilling. The results of this program are currently being compiled and at time of print no final conclusions are available. However, the following challenging geotechnical issues will once again need to be addressed in design to prevent any further losses in ore recovery:

1. The close proximity of Pandora's Fault to the wall will require a steeper overall pit wall angle or another cutback to remove the hazard;
2. The wall has clearly undergone significant deformation, as shown by the zone of deformation below Pandora's Fault, which was previously unknown. The overall stability of the rock mass will need to be re-assessed against the true strength; and
3. The rotated block has resulted in shallower bedding planes which are likely to continue to daylight in the batter face to full depth unless the orientation of that section of the wall is changed.

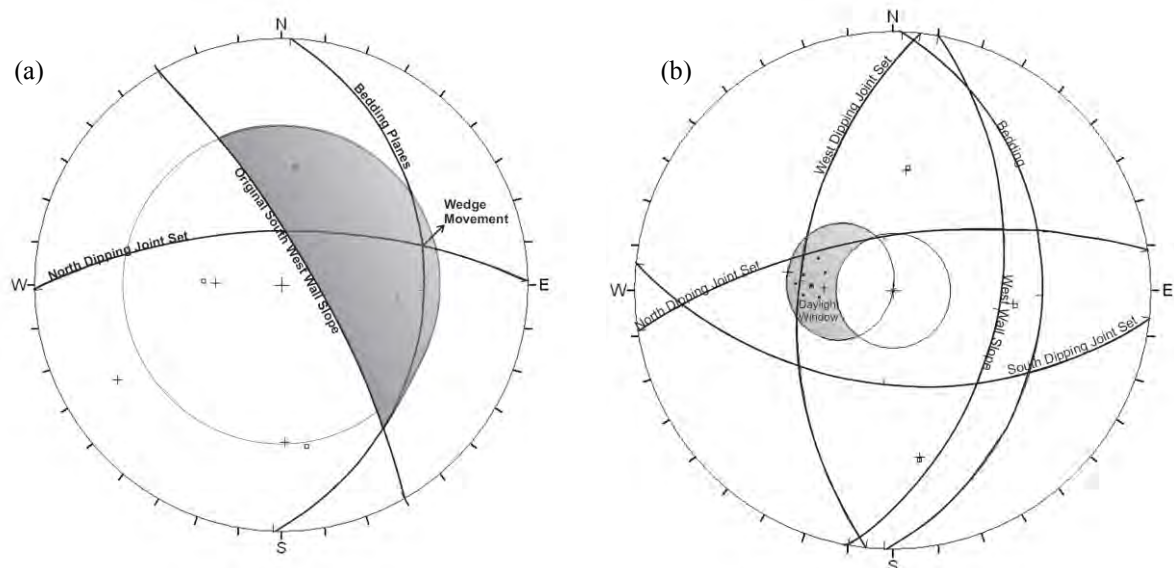


Figure 4. Failure mechanisms of (a) small wedge failures of late 2007 to December 2008 and the deeper January 2009; and, (b) 2010 planar failure mechanisms

4. CONCLUSIONS

The continuation of failures in the south-west wall occurred as a result of previously unknown complex structural geology that were not identified during initial design due to a lack of geotechnical data in that area of the pit wall. The failures have caused ongoing delays to ore production and resulted in unplanned wall movement and costs associated with remediation and redesigns. The wall is currently being closely monitored for any movements and it may be possible to continue to excavate the south wall by following the current design. However, the failures may continue to propagate in the current design and the full recovery of ore may not be possible. Alternative designs may need to be considered as a result of the latest investigations. This study has highlighted the need to gather good geotechnical data at early stages of planning and design, particularly in areas of complex structural geology, so that the issues like those outlined here can be anticipated and accounted for in design.

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ASSESSMENT OF THE ZONE OF INFLUENCE AND RELATIONSHIP BETWEEN PLT AND LFWD TESTS

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ABSTRACT

The Plate Load Test (PLT) is a relatively simple, insitu test which can provide a direct measurement of the modulus of a near surface material profile. The Light Falling Weight Deflectometer (LFWD) has recently been increasing in usage and also directly estimates the modulus of the near surface profile. The advantages of the LFWD over the PLT include test duration and an inbuilt weight, which thus negates the requirement of provision of an external load source.

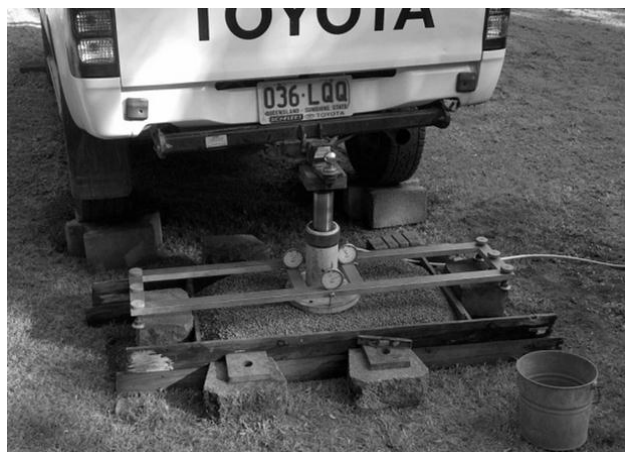
Both the LFWD and the PLT's zone of influence that affects the measured modulus value is considered to be approximately twice the diameter of the loaded plate and, thus, the plate diameter utilised defines the scale limitation of these tests. This paper assesses the actual zone of influence via the completion of a series of immediately adjacent PLT and LFWD test pairs upon a fill of a known, but varying, depth. The modulus values estimated by each test method are compared, and the effect of the different tested material types upon the estimated modulus values is also evaluated.

1. INTRODUCTION

The direct determination of a material's stiffness / modulus properties are inherently more useful in pavement or foundation design than commonly used indirect strength tests (eg CBR, which is itself an index test). This is because a modulus value is a deformation parameter that defines a material's response to an applied load, as opposed to inferring such behavior based on relationships with strength test results. However, the onsite labour, heavy plant and test duration required for static load test completion currently prevents wide-spread adoption of testing that characterises insitu stiffness properties.

This paper details the use of a Light Falling Weight Deflectometer (LFWD) to estimate the modulus (E) of two (2) uncompacted gravel materials placed over a weaker residual clay fill. By comparing modulus values derived from the LFWD investigation to those produced by a traditional static load test, the Plate Load Test (PLT), the accuracy of the LFWD has been assessed and a factor to convert LFWD to reloading modulus values (E_{r2}) has been determined. The repeatability of the results produced by the LFWD has also be assessed, as has the effect the size of the tested gravel has on derived modulus values.

Having defined the modulus of the various materials present within the investigated profile and by varying the plate size and energy imparted during LFWD testing, the zone of influence (depth below tested surface which affects the measured modulus value) of the LFWD test has also been estimated and compared to values commonly accepted for rigid plate testing.



(a)



(b)

Figure 1: Test equipment as used in this study. (a) Plate Load Test and (b) Light Weight Falling Deflectometer

2. TEST EQUIPMENT

Two (2) types of insitu load test were completed to undertake this comparative study. These included both the static Plate Load Test (PLT) and the quasi-static Light Falling Weight Deflectometer (LFWD).

2.1. PLATE LOAD TEST

The PLT involves the compression loading of a material via the use of a steel plate, hydraulic jack and reaction load (Figure 1a). The resulting deflection of the loaded surface is monitored by a number of dial gauges placed around the plate, producing an average plate settlement for each load increment and an overall load-deformation curve. Repeated loading / unloading cycles are generally completed to determine the extent of elastic deformations.

Composite elastic modulus (E) values over PLT's zone of influence, accepted at twice the rigid plate diameter ($2D$), can thus be derived by use of Equation 1:

$$E = K_s \times D \times (1 - \nu^2) \quad (1)$$

Where: K_s = Subgrade Modulus, derived from gradient of stress / settlement curve from loading cycles
 D = Diameter of Rigid Plate ν = Poisson's Ratio

2.2. LIGHT FALLING WEIGHT DEFLECTOMETER (LFWD)

The Light Falling Weight Deflectometer (LFWD or LWD) is a comparatively recent site characterisation tool and has been implemented by a number of parties, primarily government departments, as an insitu pavement and subgrade evaluation technique. The LFWD utilised for testing in this study is the PRIMA 100, manufactured by the Danish company, Grontmij A/S.

The LFWD is a quasi-static plate load test, whereby a sliding 10kg weight is manually raised upon a guide rod and dropped onto a rigid base plate instrumented with a load cell and velocity transducer (Figure 1b). The load pulse generated when the weight is dropped upon the rubber damper passes through the load plate and into the ground, and the deflection of the ground due to the imparted force is determined.

As both force and deflection values are measured over the duration of the load pulse, the composite elastic modulus (E_{LFWD}) over the zone of test influence can thus be derived by the classic static elastic theory (Boussinesq elastic half-space) equation, as shown in Equation 2. Previously identified limitations relating to use of static elastic theory for interpretation of LFWD results (which is a semi-dynamic test) are detailed in Fleming (1999, 2007), and include a phase lag between the timing of the observed peak force and maximum deflection values.

$$E_{LFWD} = [A \times P \times R \times (1 - \nu^2)] / d_0 \quad (2)$$

Where: A = Plate rigidity factor ($\pi/2$ for rigid plate) P = Maximum Contact Pressure
 R = Radius of plate ν = Poisson's Ratio
 d_0 = peak deflection

3. METHODOLOGY

PLT and LFWD tests were completed upon a specifically constructed test pad (Figure 1), over which the thickness of fill of a uniform river gravel material was varied. The base of the pad, located 200mm below the existing ground surface, was comprised of a recently placed residual gravelly sandy clay (SC) fill material. This underlying 'weak' material was chosen due to both limitations associated with the available reaction load (1 to 5T), and to be able to differentiate the influence of a weak material beneath a comparatively strong material. Prior to commencement of testing of placed gravel materials, both a PLT and LFWD test was completed at the base of the test pad. All testing was completed in general accordance with standard methodologies (PLT – ASTM D-1196 and LFWD – ASTM E-2835-11).

In the two (2) full iterations of testing completed, a uniform gravel material was placed upon the prepared pad and the modulus (E) value determined via both PLT and LFWD testing. The uniformly graded river gravels tested were nominally of 5mm and 20mm diameter, and the thickness of the placed gravel was incrementally increased in 100mm intervals up to a total thickness of 400mm. Although levelling of the pad was undertaken prior to each test, to ensure appropriate seating of the test equipment was achieved, no other compaction of the placed gravel materials was undertaken prior to testing. This was due to the limitation of the available reaction source – a loaded light vehicle, providing a maximum reaction force of approximately 15kN, equivalent to ~850kPa pressure on a 150mm diameter plate – the capacity of which would have likely been exceeded if compaction of material prior to testing had occurred.

After each completed test, the gravel within the anticipated zone of influence ($2D$) was removed and relaid in order to remove any compactive effort applied to the gravel during the preceding test. The depth of permanent deformation due to the load test (footprint depression) was also measured at the completion of each test.

Each LFWD test involved the completion of weight drops from three (3) drop heights – 210mm, 420mm and 630mm – in order to investigate differences within calculated moduli values with imparted energy variation. Each test was also repeated utilising rigid plates of varying diameter ($D = 100\text{mm}$ and $D = 300\text{mm}$) to investigate any effect that plate size had on calculated moduli values, as the zone of influence in rigid plate theory is commonly accepted to be $2D$.

PLTs involved the completion of three (3) loading / unloading cycles of a 150mm diameter steel plate, monitored by three (3) spring-loaded gauges to measure settlement throughout the test.

Interpretation of the PLT and LFWD test results produced equivalent modulus values via the previously presented rigid plate / Boussinesq elastic theory equations, with an assumed Poisson's Ratio (ν) value of 0.35. As this ν value was consistently adopted for all PLT and LFWD tests completed further investigation to determine the Poisson's ratio of each material was not required. Similarly, as the deformations measured at the completion of comparable LFWD and PLT tests were similar in magnitude, the modulus values determined from each of the test types completed on similar loose gravel/thickness arrangements were considered to be directly comparable.

4. RESULTS

4.1. PLATE LOAD TEST

The modulus values determined from the results of the static plate load tests were accepted as the baseline values to which the results of the LFWD testing was compared. For each completed PLT, both an initial (E_{V1}) and reloading modulus (E_{V2-3}) value has been determined, the former derived from the initial stress / settlement curve produced from the first (virgin) loading cycle, and the latter calculated from the stress / settlement curves produced by subsequent loading/unloading cycles.

Calculated modulus values for each completed PLT test are presented in Table 1. These results show the reloading modulus of both tested gravels appears to be in the order of 40MPa. Lower reloading modulus values due to the influence of the weaker, underlying fill ($E_{V2-3} = 11\text{MPa}$) were, as expected, seen in Tests 2 to 4 (100 to 300mm thick, 5mm diameter gravel). However, the same response was not identifiable in the 20mm diameter gravel tests, where the E_{V2-3} values remained within 7% of the average modulus value ($E_{V2-3} = 38.9\text{MPa}$) over the full range of gravel thicknesses tested.

Table 1: Initial and reloading modulus, PLT tests (assuming $\nu = 0.35$) on an uncompacted gravel material

Test ID	Gravel Diameter (mm)	Gravel Thickness (mm)	Initial Modulus, E_{V1} (MPa)	Reloading Modulus, E_{V2-3} (MPa)	Permanent Deformation (mm)
1	Natural Fill	0	0.9	11.2	20
2	5	100	8.6	14.5	15
3	5	200	5.0	34.9	5
4	5	300	2.4	31.6	30
5	5	400	0.9	38.8	Not Recorded
6	20	100	5.9	38.8	30
7	20	200	11.2	40.8	25
8	20	300	10.5	39.6	30
9	20	400	4.0	36.2	40

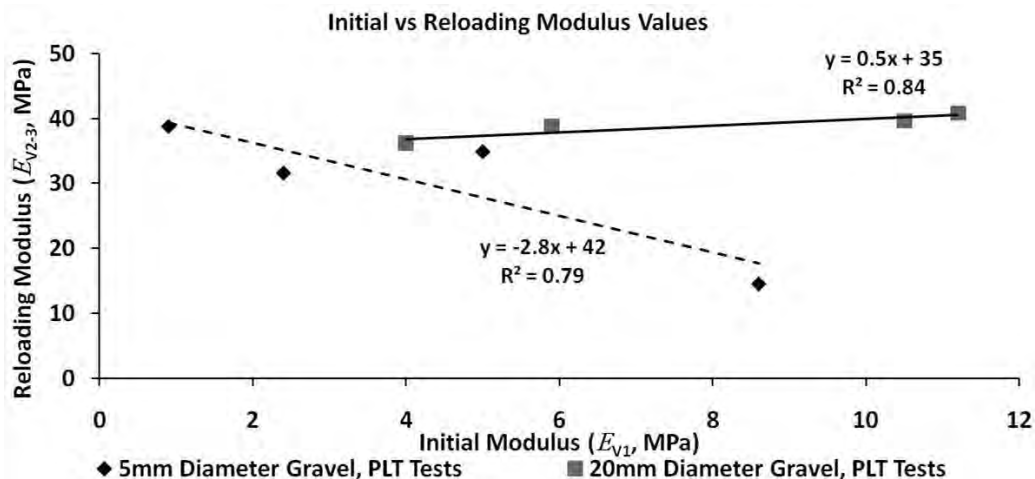


Figure 2: Plot showing interpreted linear relationships between initial (E_{V1}) and reloading (E_{V2-3}) modulus values

Interestingly, there appears to be a different response between the calculated E_{V1} and E_{V2-3} values for the PLT tests completed on each gravel material. As shown in Figure 2, an approximately linear relationship exists between the initial and remoulding modulus of each material, but with gradients of different signs – the 5mm gavel has a negative gradient, whilst the 20mm gravel's E_{V2-3} / E_{V1} relationship has a positive gradient. Calculated E_{V2-3} / E_{V1} ratios for the 20mm gravel PLTs are, after accounting for the lower relative compaction and density values associated with the loosely placed gravel considered in this study, generally consistent with the trends identified in data published by Schimdt and Rumplet (2003) where the calculated modulus ratio was found to increase as the degree of compaction decreased.

4.2. LFWD TESTS

Modulus values derived for LFWD tests completed on the pad base and gravel layer configurations are presented in Table 2. Repeated (3) LFWD tests (100mm and 300mm plate) were completed across the pad base prior to filling commencement, and a single LFWD test (100mm and 300mm plate) was completed on each 100mm increase in fill thickness, for both diameter materials. As each LFWD test results involved a number of test drops (typically $n \geq 10$), the average filtered value (after identification and removal of seating blows and outliers) is used for comparative purposes. These seating blows represent the initial compaction of material (analogous to the virgin loading cycle of the PLT), and identification / removal of these values improves comparability between the E_{LFWD} and E_{V2-3} values.

The Coefficient of Variation (CoV) for each test has been calculated from the filtered dataset (as opposed to use of raw data, inclusive of seating blows and outliers). Figures shown in italics in Table 2 indicate that the calculated moduli and CoV values include data that lies outside of the LFWD's calibrated range (and not used for subsequent data analysis).

Table 2: LFWD Results, derived modulus and COV values for each drop height / test location

Gravel Diameter (mm)	Gravel Thickness (mm)	Plate Dia. (mm)	Average Modulus, (E_{LFWD} , MPa)			Coefficient of Variation, (COV, %)		
			210mm	420mm	630mm	210mm	420mm	630mm
N/A - Natural material at base of pit	0	100	5.7	6.3	8.0	1.7	7.1	10.0
			5.2	5.6	12.0 / 31.9	12.2	13.4	29.5 / 43.3
			10.4 / 10.2	7.9 / 17.9	8.5	50.1 / 1.8	15.1	7.3
			6.1 / 9.3	Not Tested		2.6 / 11.8	Not Tested	
		300	32.8	25.9	18.7	0.9	5.1	9.1
			15.2	15.8	17.9	0.1	1.6	2.1
			5.3	3.5	3.7	5.4	10.2	8.9
5	100	100	5.5	5.3 / 5.9	8.5	6.6	21.2	24.6
		300	9.4	5.4 / 9.9	3.8 / 13.0	1.8	43.6 / 11.5	1.7 / 5.7
	200	100	20.1	8.8	8.7	13.1	5.4	7.0
		300	11.7	10.8	10.7 / 10.9	1.3	2.1	1.9 / 1.9
	300	100	42.7	41.9	37.2	2.2	5.1	2.5
		300	23.6	21.8	18.7	0.6	0.6	2.2
	400	100	33.0	53.2	37.1	5.5	4.5	4.6
		300	31.1	30.1	26.6	1.3	2.7	1.3
20	100	100	19.6	25.3	8.0	9.6	0.2	5.3
		300	13.7	11.8	11.4	1.8	4.6	5.4
	200	100	27.4	30.4	12.3 / 28.8	5.2	0.8	12.9 / 28.9
		300	16.6	14.8	15.4	10.1	4.2	4.0
	300	100	30.3	45.6	46.9	10.4	4.8	1.4
		300	20.5	23.8	20.0	4.0	5.4	2.0
	400	100	50.0	42.2	45.4	9.8	6.6	8.3
		300	21.9	21.7	22.1	5.9	3.5	4.7

5. DISCUSSION

Based on the assumption that the zone of influence of a rigid plate test is approximately twice the diameter of the rigid plate (2D), it was expected that the applicable zone of influence for the 100mm and 300mm rigid plate LFWD test variations would be approximately 200mm and 600mm respectively. As the completed PLTs utilised a 150mm diameter plate, the zone of influence contributing to the PLT derived modulus values was expected to be approximately 300mm. Accordingly, due to the presence of the weaker material underlying the placed gravel material, the calculated material resilient moduli (E_{V2-3} or E_{LFWD}) for any specified gravel layer thickness was anticipated to be highest for the 100mm plate LFWD test and smallest for the LFWD test that utilised the 300mm plate. The PLT modulus was anticipated to fall between the two (2) LFWD derived modulus values. The maximum tested modulus value should equal the gravel

material's modulus (ie with no influence of underlying weaker material), and approximated by both the 100mm diameter plate LFWDs and PLTs completed upon 300mm or 400mm thick layers of gravel.

Table 3: Comparison of derived reloading modulus (E_{V2-3}) and LFWD modulus (E_{LFWD}) for tested materials

Material	E_{V2-3} Modulus (MPa)		E_{LFWD} (100mm Plate) (MPa)		$E_{V2-3}:E_{LFWD}$ (100mm)
	Range	Average	Range	Average	
SC Fill	N/A	11.2	5.3 – 32.8	18.9	1.69
5mm Gravel	31.6 – 38.8	35.2	33.0 – 53.2	40.9	1.16
20mm Gravel	36.2 – 39.6	37.9	30.3 – 50.0	43.4	1.14

Via this rationale, the material modulus of each identified material has been determined, as presented in Table 3. A consistent multiplier of approximately 1.15 was derived to convert the PLT modulus values to the average E value calculated via the LFWD deflectometer (100mm diameter plate, all weight drop heights) for both of the gravel materials tested. A higher multiplier was calculated for the underlying residual clay fill, however the average E_{LFWD} value used for this derivation is considered questionable, as only a small percentage of the total number of tests completed upon this material conformed to the calibration parameters of the LFWD (predominantly 300mm diameter plate LFWD tests conformed whilst 100mm plate tests failed due to high measured deflections). The results presented in Table 3 also indicate the nominal effect (6 to 7%) that gravel diameter appears to have on the calculated modulus values.

The consistent $E_{V2-3}:E_{LFWD}$ (100mm) ratios replicate the comparative study completed by Nazzal et. al. (2007), who found an excellent correlation between modulus values derived from the LFWD and reloading cycles of the PLT ($R^2 = 0.97$). The published direct relationship, where E_{LFWD} value was found to be 1.14 times the reloading modulus, has been nearly exactly reproduced by the gravel materials studied by this project. Similarly, the results relating to CoV values produced by LFWD testing published by Fleming et. al. (2004) are supported by the results recorded for this study. Fleming et. al. noted a general decrease of CoV was associated with increased material grain size, and reported CoVs generally less than 15% for well graded crushed rock. The results calculated from the data collected support these findings, with the overall averaged CoV of each of the gravel materials being 5.1% (20mm) and 6.0% (5mm). CoVs were not observed to differ due to varying drop heights (all 5.6%), but was found to be greater for data collected using the 100mm plate (7.4%) than the data collected using the 300mm plate (3.7%)

Comparisons of the average E_{LFWD} (for each tested plate size) and PLT E_{V2-3} are presented in Figure 2, for both the 5mm and 20mm diameter gravel materials. The expected relationship ($E_{LFWD(100mm)} > E_{PLT} > E_{LFWD(300mm)}$) between calculated moduli values is well preserved for both the 300mm and 400mm thick gravel layers, but does not hold true for either of the thinner gravel thicknesses. In the case of both gravel sizes, the modulus value derived from the PLTs completed upon 100mm and 200mm thick layers of gravel exceed the modulus values determined by either of the LFWD tests. Additionally, in the case of the 100mm thick layer of 5mm diameter gravel, the modulus calculated via use of the LFWD with the 300mm plate attached is lower than that derived from the 100mm diameter plate LFWD results.

For each LFWD test completed, three (3) drop heights were considered. The variation within the calculated moduli for each of the three (3) drop heights, for any particular gravel size and thickness, ranged between 1.8% and 44% of the averaged E_{LFWD} value. Greater variation was exhibited within the tests completed in the 5mm gravel (average variation of $\pm 13.3\%$) than the 20mm gravel (average variation of $\pm 9.0\%$). Similarly, greater variation was observed within the data produced via use of the 100mm diameter plate ($\pm 14.4\%$) than the 300mm plate ($\pm 8.1\%$).

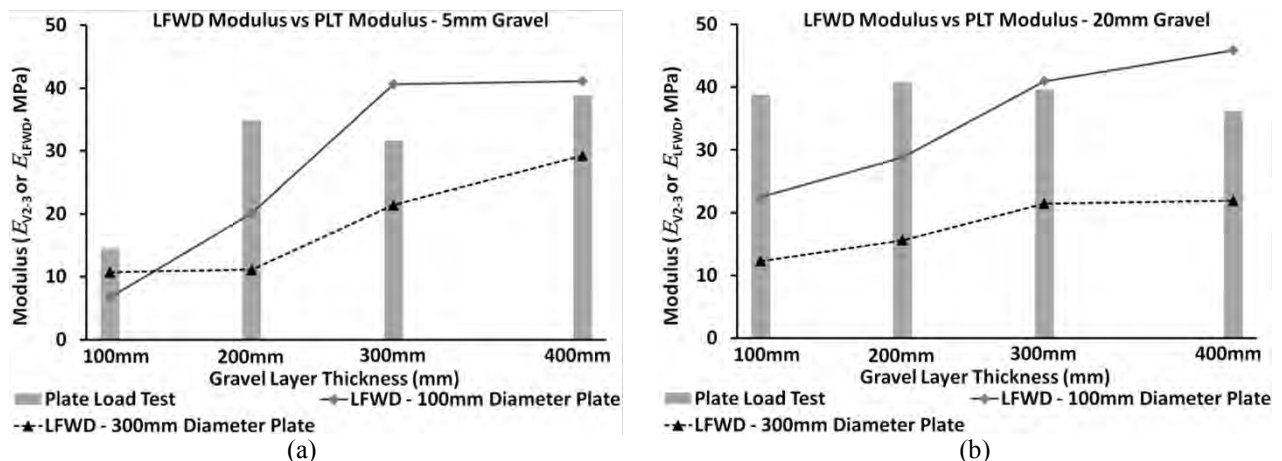


Figure 2: Comparison of E_{V2-3} and E_{LFWD} values for (a) tests completed upon 5mm diameter gravel; and (b) tests completed upon 20mm diameter gravel

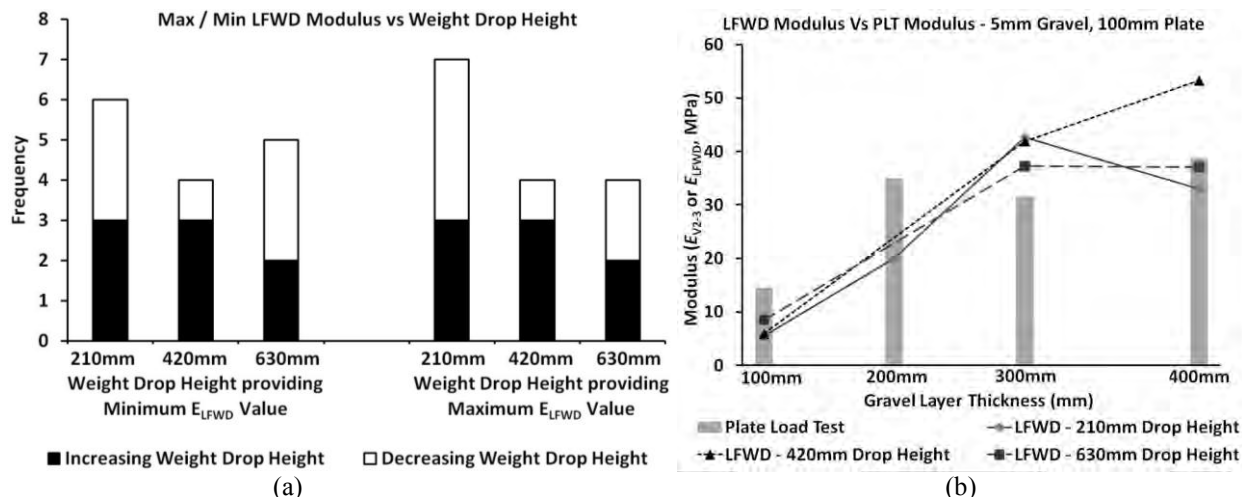


Figure 3: (a) Frequency that each weight drop height provides the minimum / maximum modulus value; and (b) typical example showing variation of weight drop height in provision of maximum / minimum modulus values

Of the 15 LFWD test variations considered, as depicted in Figure 3a, there is no indication that any particular drop height (and thus energy level) will result in the overall highest or lowest E_{LFWD} values; although there does appear to be a slightly higher chance that the 210mm height drop will provide either the maximum or minimum modulus value.

Similarly, there is no indication that the order of testing influenced the test results for the gravel materials tested. Some tests (20mm gravel) were completed in an order whereby the weight drop height was continually increased, whilst others (5mm gravel) were completed in decreasing weight drop order. However from inspection of Figure 3a there does not appear to be any observable relationship between the maximum / minimum derived modulus values and the order of tested weight drop height. This, and the absence of material degradation due to testing, implies that the various loads applied by LFWD testing were largely within the elastic deformation range of the gravel. It is not expected the same would be true for all materials (eg weak cohesive soils), where significant dynamic compaction of the material undergoing testing may occur due to the repeated loading associated with the LFWD test. It is likely, however, that such weak materials would not be suitable for LFWD testing, as they would likely result in deformations above the maximum calibrated value (2200 μ m).

6. CONCLUSIONS

From the work presented in this paper, site and material specific PLT and LFWD correlations have been determined. A number of comparable PLT and LFWD tests have been completed to characterise the resilient modulus of two (2) loosely placed gravel materials (nominally 5mm and 20mm diameter). For each LFWD test, both a 100mm and 300mm diameter rigid plate was used to determine the material's modulus value.

Based on the comparison between average results obtained from the 150mm rigid PLT and 100mm rigid plate LFWD tests (ie tests which had the zone of influence fully contained within the river gravel fill material) a consistent $E_{V2.3}:E_{LFWD}$ (100mm) ratio of 1.15 was determined to be applicable for both of the gravel diameters tested.

The results of the LFWD tests suggest the zone of influence of the 100mm rigid plate test is between 200mm and 300mm from the tested surface. For all tests completed utilising the 300mm plate, the zone of influence of the LFWD test was greater than the maximum depth of gravel tested (400mm). Analysis of the calculated CoV for the LFWD results suggests that, for the material tested, the use of a larger rigid plate (300mm) will generally provide a tighter spread of results within the repeated load drops, compared to a small plate (100mm).

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CUNNINGHAM'S GAP CONSTRUCTION AND SITE ENGINEERING

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ABSTRACT

Since its construction in the 1920s to 1940s Cunningham's Gap, along the Cunningham Highway within Main Range National Park, Queensland has been subject to significant slope instability. Portions of the cuts above the highway and lower embankments immediately below have been affected by heavy rainfall over the years, although during heavy rain in late 2010 / early 2011 emergency works were necessary as instability accelerated.

Several of the lower embankments were recently remediated to improve highway conditions and stabilise recent landslides. From design to the construction stage modifications were often necessary on site to continue the project in a timely and cost efficient manner. Engineering solutions were provided quickly and QA/QC supervision of contractors was necessary during remedial work onsite, many of which were completed under 24 hour operations as full highway access was required by the end of 2011. The remedial options employed included excavations to 6 metres, installation of soil nails, drainage, shotcrete with steel reinforcement, and rockfill placement. Five (5) sites were remediated with design and construction supervision.

1. INTRODUCTION

1.1 SITE OVERVIEW AND LOCALISED GEOLOGY

Cunningham's Gap straddles the Cunningham Highway where it intersects the Great Dividing Range, within Main Range National Park in steep, terrain in Queensland, Australia. The location is approximately 85km southwest of the city of Brisbane. The Cunningham Highway is part of the federally funded National Network which provides a key connection between Brisbane and Warwick, representing not only a busy transit route, but a key freight route for the area. The Main Range National Park belongs to the World Heritage Site Gondwana Rainforests of Australia.

The Geology of Cunningham's Gap can be attributed to flood basalts originating from a shield volcano during the Late Tertiary period (between 25 and 22 million years ago). The main lithologies comprise trachytes overlaying basalts of the Main Range Volcanics, forming several ridgelines of, and surrounding the Range. For the large majority of rock exposure, the cliffs and slopes above the road comprise basalt of variable weathering and strength (Olgers, 1972).

A layer of variable thickness (1m to 8m) agglomerate is evident in cliffs in parts of the southwestern side of the range. The Main Range Volcanics unconformably overlie the Walloon Coal Measures of the Middle to Upper Jurassic period. These comprise horizontally bedded sedimentary rocks including sandstone, siltstone and mudstone (Olgers, 1972). Sandstone and siltstone are evident in cut slopes and found within boreholes completed during site investigations.

Increased gas charged and explosive eruptions deposited zones of tuff (Willmott, 1995); which are evident as intermediate layers between basalt flows. More recent materials include colluvium and fill (consisting of reworked colluvium). The colluvium is composed of a mixture of cobbles, boulders, and soils; typically soils are a gravelly clay. In less steep locations the hillside is mantled with residual soils derived from the basalt. Many boulders are evident on the slopes, both loose and bound within a matrix of colluvial soil.

The highway was constructed and officially opened in 1927. Construction, completed initially by volunteers, included removing the mountainside with cuts into the colluvium and the basalt mountain side. Long side fills were placed on the embankments for expected depths up to and exceeding 20m. From the very early stages of construction the highway had many structural problems and was closed within a month (The Brisbane Courier, 1927), re-opening again after over 20 years, in 1949. Increases of heavy vehicle volumes, axle sizes and weight were unlikely to have been factored into the original highway construction design.

1.2 TRIGGERING EVENT OF FAILURES

Between December 2010 and January 2011 extreme rainfall triggered rock falls and landslides along several embankment sections of the highway. A portion of the Cunningham Highway, referred to as Clayton's Gully, subsided over 1 m through slope failure to create an approximately 60 m long scarp, effectively making two lanes of the downhill carriageway untrafficable. At this time several other areas along this section of the highway experienced instability to a lesser vertical extent, but over similar or longer lengths.

1.3 RECENT CONSTRUCTION OVERVIEW

Services for remediation were first commissioned in December 2010 to begin 'Emergency Works' and provide advice on the threat posed by falling rocks above the highway sections. Works continued in the area as movement of the fill embankment slopes became more evident necessitating further investigations and remedial designs for slope stabilisation. Boreholes were drilled and monitoring was installed by the Department of Transport and Main Roads (TMR).

Previously collected core from boreholes collected by TMR were logged and geotechnical investigations for other sites were undertaken, in order to create geologic models and provide detailed designs for these sections of the highway. Rock and soil materials were assigned geotechnical design parameters and geotechnical software programs such as Slope/W were used to aid in design.

Time for completion of construction remediation was a restricting factor and designs were often completed shortly after field work. The locations of works designed and supervised were identified as Figtree West (140 m), Cut 14 (90 m), Cut 17 (40 m), Cut 18 (90 m), and Cut 18 West (40 m). Embankments that were designed or overseen onsite by other companies included Figtree Gully (100 m), National Trail (240m), Claytons Gully (110 m) and Six Penny (120m). Actual constructed extents often exceeded design length limits provided herein. Design guidelines of Main Roads Technical Standard's (MRTS's) 03 and 04 were followed (Queensland Government, 2009). Remedial designs included cuts to 6m, then drilling, installing, and grouting temporary and permanent soil nails with minimum bond lengths into weathered basalt, installation of fabric reinforcement and shotcrete on facings, and placing and compacting approved rockfill with geogrid.

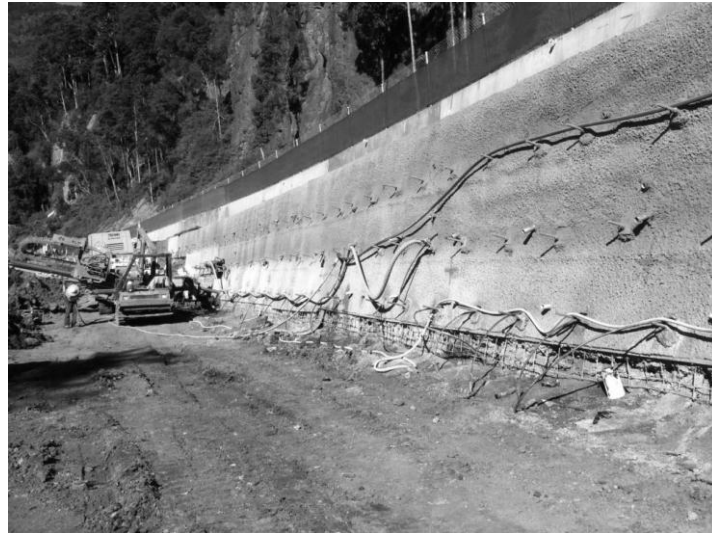


Figure 1: Cut 18 Temporary Soil Nail Wall

2. CONSTRUCTION AND ENGINEERING CHALLENGES

Given the complexity of ground conditions, limited survey and investigation, a number of onsite decisions had to be made daily to address design and construction issues as they were recognised. The following challenges were faced onsite:

- Soil nail installation and grouting;
- Effective monitoring and removals;
- Access and restricted construction areas; and,
- Installation of TERRA[®] mesh and rockfill placement.

2.1 SOIL NAILS AND GROUTING

One of the most common difficulties was loss of grout within drill holes for each soil nail installed, due to the ground conditions. 32 MPa grout was required for soil nails with grout to be continuously installed to the face of the hole (or edge of embankment). The typical drilling length of the soil nails was 12m, as a minimum grouted bond length of 5 m to 6 m was necessary (dependent on whether the nails were for temporary or permanent works). Bond length refers to the resistance or frictional force acted upon by the ground mass on the soil nail bar. Generally the nails intersected 3 m

to 6 m of fill and colluvium before reaching moderately to highly weathered basalt. A bond length within weathered basalt was the basis for design.

A temporary soil nail wall was constructed to access and install permanent nails on the lower embankment at Cut 18; significant grout loss was experienced within the fill, colluvium and basalt (Figure 2). Loss was attributed to voids within the fill and colluvium, and open defects within the basalt which are visible throughout the cliff line. Defects often included open, nearly vertical joints. Grout most likely followed these defects and filled up voids between cobbles and boulders that made up the colluvium, rather than forming around the nail and filling the hole initially. Strategies to reduce grout loss began with altering the consistency of the grout mix by reducing quantities of water and the water-reducing admixture rheobuild, which was used to prevent grout hoses from clogging the pump. Additionally the length of casing used to stabilize the hole for drilling was slowly pulled during grouting operations as sections along the nails were grouted. A “blow” hose, attached to the soil nail was used to determine the depth of grout during this process. Nails were able to be grouted in this area by these modifications, although large grout losses were unavoidable. Use of a stiffer grout mix and sand aggregates is also recommended in these conditions (U.S. DOT, 1994).



Figure 2: Grout Losses within Basalt and Colluvium in Cut 18

Permanent soil nails were installed 6 m below the highway surface and drilled through the underlying fill that was placed up to 80 years ago; materials placed are undocumented and unlikely to be in accordance with current standards.

Supervision of the 6m cuts and test pits at the base of the excavations in Cut 14 and Figtree West revealed areas of loose cobbles and boulders in a clayey soil matrix (Figure 3). These conditions provided a pathway for the grout to flow, sometimes exiting well below the area of works in the valley further down the embankment (sometimes at depths exceeding 20 m below the works area).



Figure 3: Loose Fill Materials at Base of Excavation in Figtree West

Soil nailing within these areas encountered more difficult soil conditions; and grouting was found to be incurring substantial costs based on quantities used and lost within the ground. An effective solution was necessary for these increasing losses of grout and sequential setback in completion date of construction. Utilizing a geotextile, as a filter sock, around the length of the soil nail was proposed. As the filter sock had expandable properties it could stretch as grout was inserted, yet small perforations (approximately less than 300 microns) allowed excess water to dissipate but

would prevent the grout from exiting the geotextile. If the geotextile was used along the length of the soil nail a pullout test was required to verify that the bond stress was not reduced and maintained design strength within the weathered basalt. In the case of the permanent soil nails, which were double encapsulated by a water proof corrugated sheathing, the problem area was typically the loose fills within the upper 6 m, therefore using the filter sock only along the upper portion of the nail (where the grout was lost and not within the bond zone) was required (Figure 4). Grouting with a “sock” has been used in the past and is documented as working successfully (U.S. DOT, 1994).



Figure 4: Filter Sock Wrapped On Soil Nail Sheathing for Figtree West

Additional recommendations on soil nail installation included:

- Modifying locations and drill angles based on excavation extents, access, and soil and rock conditions;
- Locating pullout test locations to represent conditions present; and,
- Advice on drilling thru cobbles and boulders and direction on embedment depths into weathered basalt.

2.2 MONITORING AND REMOVALS

A controlled monitoring system was an important part of the construction process as remediation involved heavy machinery and work over recent landslides. The plan included use of survey monitoring targets and prisms placed parallel to the highway above and below the landslides in a regular spacing interval. A base survey was completed and regular surveys were taken 1-2 times daily. Maximum target levels for movement and their respective plan of action required if exceeded were developed. Often there was a time delay to receiving outputs from survey and start of construction. To further ensure movements were within acceptable levels, visual markers, consisting of star pickets were placed and inspected throughout the day. Vibrating Wire Piezometers (VWPs) were installed to monitor ground water levels daily to establish safe working conditions within excavated areas. Maximum water levels and their respective actions for response were established. As part of continued movement in Figtree West, safety precautions included closing off the area, setting up an extensive monitoring program, determining extents of movement and modifying design which included additional soil nails and instructions on safely accessing these areas with drilling rigs.

A main portion of the construction works included the 6m excavation from the top of the road surface for each site. Identifying failure mechanisms before design was essential and included determining lateral slope displacements, tension cracks and exposed scarps. On site, the limits of works were further assessed based on features recognized at the start of construction. These included aspects along the highway such as depressed or deteriorated pavements, any further tension cracking along pavements, condition of kerbs, surface soils, drainage and guard rail alignment. To define failure extents along the highway section the lantana and vegetation was removed and limits were further evaluated and increased as necessary.

Additional recommendations provided during removals included:

- Providing assessments of rock to determine if materials can remain in-place, based on potential for undermining the roadway and making the distinction between colluvium boulders and basalt;
- How to secure cut slopes if stability was compromised due to over excavating large boulders or exposing surface drainage; typically involved using geotextiles, a shotcrete cover and/or steel reinforcement; and,
- Provide additional drainage in the form of horizontal drains, weep holes and megaflo® as seen appropriate based on surface conditions as exposed.

2.3 ACCESS AND CONFINED CONSTRUCTION AREAS

The geometry of the highway was restricted to a narrow width of typically one (1) 3.5 m lane in each direction with the mountain (or cut side) with a 1-2 m wide spoon drain and a 1 m shoulder; and the embankment side with a 1 m shoulder

with a kerb and guardrail on the edge of the typically steep escarpment. Each landslide site had unique features on the embankment side which significantly affected the remedial approach. Design and construction was limited by allowing public traffic to continuously flow (with maximum stoppages of 10 minutes); therefore closing more than one lane was not permissible. To install soil nails a limited access 20-tonne track rig or a 40-tonne long reach excavator with a drilling mast was used.

Cut 18 provided particular challenges in this regard. To allow for drilling activities a 6 m wide (minimum) platform was necessary for the drilling rig, steel casings, drilling rods, crane, excavators, and satisfy a minimum of a 1 m setback to maintain a satisfactory margin of safety with respect to slope stability. To aid in bearing capacity and trafficability a half-meter thick layer of rockfill was placed along the base of the cut. Cut 18's embankment dropped steeply (in some areas greater than 45°) from the roadway so a cut approach was not satisfactory based on the ground conditions and angle of excavation necessary. To provide access to the area below the highway a 6 m high temporary soil nail wall was constructed along the face at an 80° cut angle (Figure 5). Temporary wall support consisted of 12 m long temporary nails at 1 m spacing's horizontally and vertically. To aid in support of drilling and installation of soil nails steel casings were temporary driven in the loose fill and colluvium.



Figure 5 (left): Temporary Soil Nails at Cut 18; Figure 6 (right): Installation of Permanent Soil Nails at Cut 18 West
Drilling rigs mounted to track excavators were specially designed to access normally unreachable soil nail locations, especially at Cut 18 West which required five (5) rows of soil nails below the highway (Figure 6).

Once to the base of the excavation another challenge came during removal of the trees along the proposed permanent soil nails. Soil materials were removed during tree excavation further reducing the work area; however supervision was provided to ensure reduced impact on the lower excavation. Tree trimmers were used to mulch the trees to the root system below ground to limit disturbance.

To ensure that the required highway width was provided at the final stages of construction the design was modified to incorporate steel wire mesh baskets near the crest to allow backfill to stand at a steeper angle than initially designed. A TERRA[®] mesh system was chosen for Cuts 14, 18, and Figtree West, as these were the narrowest of sites. Further discussions on this system are provided in the following section, Section 2.4.

2.4 INSTALLATION OF TERRA[®] MESH AND ROCKFILL PLACEMENT

Use of an appropriate rockfill, with a mechanically interlocking structure when compacted was necessary. Local quarries were inspected and a test trial was completed to assess the maximum angle the rockfill would retain after normal placement and compaction, as the design was based on a 45° slope angle. The trial proved a relatively steep angle for a larger rockfill used, between 40° and 48°; however, with varying rockfill and to ensure final road widths were satisfied a 70° facing TERRA[®] mesh system, as 0.6 m tall “baskets”, designed by Maccaferri were chosen (Figure 7).

The system is commonly used for granular soils; therefore special procedures on placement and compaction of fill were required. Advice on placing the mesh system included:

- Positioning locations at the toe of each lift, especially on areas where basalt rock outcrops were exposed;
- Reviewing slope stabilization calculations for steepening sections;
- Compaction offset distances from edge of placement (greater than 1m was found to minimally affect the prefabricated angle), rockfill to use within “baskets”, and how to place rockfill within (hand placing along outer face);

- Tying together “baskets”, including locations and how to attach, and offset distances of attachments; a combination of steel rings and wire lacing were utilized;
- Review of batter angles and determining projected heights of each consecutive layer; and,
- Choosing select materials in upper 1m for guard rail driving.



Figure 7 (left): Rockfill and TERRA® Mesh at Cut 18;

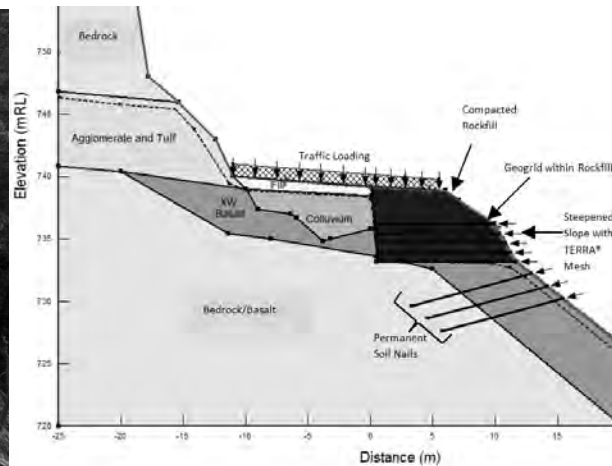


Figure 8 (right): Ground Model and Design at Cut 18

3 CONCLUSIONS

Construction projects are constantly dealing with issues that need to be resolved quickly and efficiently based on timeframes and budgetary constraints. It is vital for engineers and contractors to work together onsite to establish effective solutions to complications or difficulties that may arise as work proceeds. The general practices outlined in this paper should be considered in future works for sites that may require similar solutions. A flexible approach to design and construction is essential to ensure that modifications can be made as necessary to reach the targeted end results.

At Cunningham's Gap, from design stage to construction proven solutions included establishing temporary works for confined areas (soil nail wall and additional support/monitoring as needed), solutions to effectively grout in poor soil and rock conditions, modifying the geometry of the embankment and rockfill (use of TERRA® mesh), and recommendations on accessing and installing soil nails.

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MINEFILL DRILLING WORKS AT HUNTER EXPRESSWAY

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ABSTRACT

The Hunter Expressway, scheduled for completion by the end of 2013, will provide a critical link between the F3 Freeway and the Upper Hunter region. One of the challenges to be overcome during the design and construction of Hunter Expressway is the presence of mine voids beneath the alignment. This paper explores the necessity for mine void filling and the resultant design philosophy. Minefill methodology, drilling equipment, borehole analysis procedures, sub-surface conditions and validation strategies are also presented.

1. INTRODUCTION

The Hunter Expressway is a project built for the NSW Roads and Maritime Services (RMS) which, when completed, will provide a critical link between the F3 Freeway and the Upper Hunter Region. The project is currently under construction in two sections, a 27km western section delivered by a Design and Construct package; and a 13km eastern section delivered by an Alliance framework comprising of the RMS, Thiess, Parsons Brinckerhoff and Hyder Consulting. Abandoned underground coal workings exist beneath approximately half of the eastern section of the Hunter Expressway. Analysis undertaken during concept design of the project identified potential subsidence related settlements of 500-800mm. This is far greater than the 25mm limit specified in the bridge designs of the F3 freeway interchange and the viaducts across the Sugarloaf range. To mitigate the risk posed by surface subsidence it was decided to stabilise the mine workings beneath potentially affected bridges by means of grout infill, i.e. minefill. As the bridges were deemed more susceptible to subsidence related damage than the carriageway, filling was limited to the mine workings beneath bridge foundations.

To complete the minefill works at the Hunter Expressway, the Hunter Expressway Alliance engaged Keller Minefill - a joint venture between Piling Contractors and Keller Ground Engineering. The scope of work for the Hunter Minefill Sub-Alliance included over 145,000 metres of borehole drilling, production and placement of 200,000 cubic metres of grout and corresponding validation works.

2. MINE GEOLOGY

The mine workings to be treated lay within the Borehole Seam and Young Wallsend Seam. These seams form part of the Permian era Newcastle Coal Measures (Ives, 1995). The Borehole Seam is the deeper seam of the two, with its depth below the bridge structures varying from 70 to 130 metres below surface level. The workings of the Borehole Seam occurred between the late 1890's and early 1920's, with intermittent secondary extraction of workings during the same period. From mine record tracings it was determined that the Borehole Seam was hand mined in bord and pillar pattern with extraction ratios varying from 40 to 70%. This variable extraction ratio has resulted in mine conditions varying from intact open roadways, to local roof collapse, panel collapse, upward void migration and suspected surface settlement. Figure 1a details the typical pattern of workings within the Borehole Seam. Shaded areas represent coal that was extracted after the initial workings (secondary extraction).

The Young Wallsend Seam overlays the Borehole Seam and was found at depths between 65 and 85 metres below surface level. Mine record tracings from the Young Wallsend Seam reveal that it was mined by mechanical means during the mid 1980's. An extraction ratio of approximately 70% within the treatment area has caused near complete collapse of the mine roof, some panel collapse and possible upward void migration resulting in surface settlement. Figure 1b shows typical working patterns from mechanical mining in the Young Wallsend Seam. The shaded sections represent secondary extraction areas.

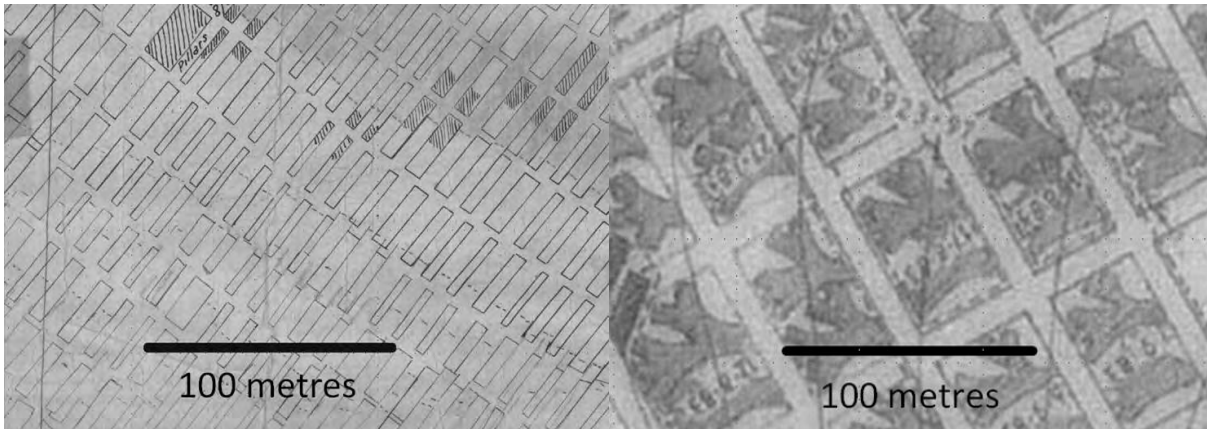


Figure 1a: Borehole Seam Workings

Figure 1b: Young Wallsend Seam Workings

3. DESIGN PHILOSOPHY

The assumed failure mechanism for the mine workings beneath Hunter Expressway is a cascading pillar failure. When coal is mined from bord and pillar workings, solid pillars of coal are intentionally left standing to support the roof. Increased compressive stress acting on a standing coal pillar due to extraction of the surrounding coal causes the faces of the coal pillar to spall- reducing the remaining load bearing area of the pillar. The reduced compressive load capacity of the pillar accelerates further spalling until the pillar fails, redistributing the overburden loads to adjacent pillars. If sufficient pillars fail, a wide span of roof above the mined area can collapse, resulting in upward void migration and possibly surface subsidence.

To economically mitigate the mine subsidence risk, it was decided to inject a low strength grout into the abandoned mine workings. The grout completely fills all voids then hardens around the existing coal pillars to provide lateral confinement of the pillars, preventing section loss through spalling and maintaining the existing load bearing mechanism of the mine.

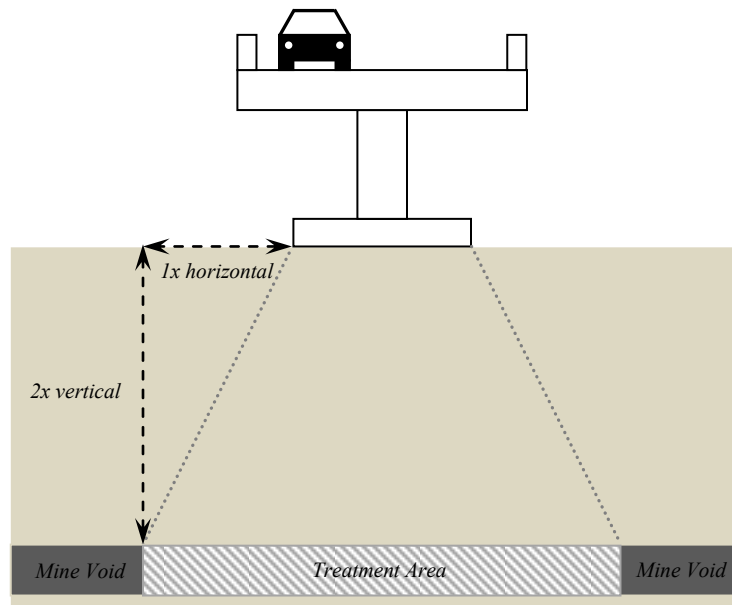


Figure 2: Angle of Draw Schematic

The filling boundaries were based on a zone of influence which used the 'angle of draw' principle. The angle of draw may range from vertical (90 degrees), where only the mine void immediately below the plan area of structure is filled, to angles

as shallow 30 degrees where a large area beyond the plan area of the structure is filled. The angle of draw varies for reasons including the geology- particularly the depth of soil between surface and bedrock, the mine void depth, and the level of acceptable risk. Minefill works on this project adopted an angle of draw of 1 x horizontal to 2 x vertical, ie the radius of minefill below each bridge pier and abutment is equal to half the depth from surface to the mine void- as shown in Figure 2. As the mine workings were relatively deep compared to the spacing between adjacent bridge foundations, treatment areas typically overlapped and were combined.

4. MINEFILL METHODOLOGY

To fill the mine workings, a cement-flyash grout was pumped from modified concrete batching plants, through steel pipelines into the mines via inclined boreholes that were drilled to achieve nominal 12 metre spacing along roadways within the mine void. Grouting operations typically began at the down-dip extent of a treatment area and proceeded up-dip from there. This is typically the best practice approach as it tests for down dip leakage early in the filling process. Grouting up-dip from down-dip extremities also minimises the potential for the grout to mix with ground water within the mine void and allows the filling to be undertaken and monitored in a progressive manner.

4.1 WORKING AREAS

Minefill works were completed from specially constructed drilling platforms which were positioned to provide optimal coverage of the mine treatment area. When planning drill platform locations, the coverage area of the mine from each platform was calculated using a maximum borehole inclination of 15 degrees from vertical (15 degrees was agreed from previous work to be the inclination limit for reasonable accuracy). Due to steep terrain, vegetation clearing restrictions, environmental and cultural exclusion zones and the location of existing roads and bridges, a 15 degree maximum inclination was not always possible. To ensure complete coverage of the minefill treatment area, inclined boreholes were drilled up to 40 degrees from vertical.

4.2 DRILLING EQUIPMENT

Over 1600 separate boreholes were drilled over the duration of operations. Drilling was undertaken by two types of drill rigs, Hutte HBR205's and Atlas Copco L8's. The Hutte HBR205 is a dual rotary, simultaneous casing rig developed for geothermal drilling. The 205's used Polycrystalline Diamond cutting heads and casing shoes with cuttings returned to the surface by an air/water flush. An external compressor was required to provide the flush as the 205 does not carry an onboard compressor. Although production rates were lower, the dual rotary system was chosen as it could maintain drill flush and borehole stability when drilling through the fractured ground frequently encountered around heavily extracted mine workings.

The other drill rig used for mine void drilling was an Atlas Copco ROC L8. The L8 is a dedicated open hole rig originally developed for blasthole drilling. The L8 drilled using a down the hole hammer powered by an onboard compressor. This compressed air also acted as the drill flush to return cuttings to the surface. As the L8 drilled open holes it would easily lose flush into any significant fractures in the ground, increasing the risk of blocking the hole or jamming the rods. The L8 could not drill areas where the Young Wallsend workings overlapped the Borehole Seam workings, nor could it drill areas of panel collapse. The advantage of the L8 was faster and cheaper drilling when ground conditions suited.

4.3 TRAJECTORY SURVEYS

After completion of drilling the trajectory of each borehole was surveyed using a wireline inclinometer. At 5m intervals the inclinometer would measure the local bearing using an electronic compass and local inclination using an accelerometer. Survey data was then uploaded to a computer and plotted on CAD software to produce a visual representation of drill trajectory. Trajectory information was used to determine where the borehole had penetrated the mine workings with respect to the mine record tracings. Trajectory information was also used to estimate likely future drilling work so the trajectories of future boreholes could be adjusted prior to drilling to compensate. Analysis of over 1600 boreholes revealed trajectory deviations to be effected by the following general factors

- **Drill type:** due to the counter-rotating heads of the Hutte 205, azimuth deviations with this machine were typically smaller in magnitude than deviations measured from L8 holes. Although the deviations measured from Hutte holes deviated less in general, the deviations were less predictable - adjacent boreholes with similar drill lengths, drill inclinations and drill bearing frequently deviated in opposite directions. By comparison, the L8's tended to produce more consistent and predictable deviations, albeit of greater magnitude.

- **Borehole inclination:** deviations in borehole inclination were observed to increase as borehole inclination increased from vertical. This increase is presumably due to gravitational effects.
- **Borehole length:** longer drill lengths tended to produce greater azimuth and inclination deviations. This is easily explained by the fact that any slight angular variation in any straight line will produce a larger deviation at the line's endpoint. Additionally, any curvature developed in the borehole trajectory will be similarly magnified over a longer borehole.
- **Feed force:** anecdotally it was noted that reducing the feed force improved accuracy, at the expense of drilling speed.
- **Other factors:** other factors that were not measured but were suspected to affect drill string deviation include cutting head wear, local irregularities in the rock strata and operational settings used by individual drillers.

Although the above factors were observed to influence drill string deviation, no all-encompassing trend could reliably predict drill string deviation. It was found that trajectory deviations were best predicted using deviation data sourced from nearby holes with similar azimuths, inclinations and drilled by similar drill rigs.

4.4 CAMERA SURVEYS

In addition to inclinometer surveys, each borehole was also filmed using a wireline video camera. The camera was lowered down the full depth of the borehole to record continuous footage of the borehole walls and of the mine void. The video footage provided valuable information about the borehole such as depths of any subsidence related fractures and or blockages. At the bottom of the borehole the camera was lowered into the mine void and panned around to film the local mine workings. To assist with drill trajectory planning, the video footage complemented the trajectory survey by providing a visual approximation of the borehole location within the mine workings. The video footage also provided information regarding the size of voids and extent of roof collapse and was used extensively to validate the minefill process.

5. CONDITIONS ENCOUNTERED

Drilling operations encountered bedded conglomerate, sandstones, siltstone, tuff and coal. Groundwater was encountered above mine level however the mines were not flooded. Conditions encountered in the mine workings varied immensely across the project, from open intact roadways, to completely subsided areas. A summary of mine conditions is presented below:

- **Intact Workings:** a small amount of the treatment area consisted of intact workings. Camera footage of these workings revealed a void height of 1.2-1.5 metres, with the roof supported by rows of timber props. Figure 3a shows camera footage of intact workings
- **Collapsed Workings:** the majority of the workings consisted of solid coal pillars with varying degrees of roof collapse observed within the workings. The roof collapse varied from local roof fall where the approximate roadway alignment within the workings was still visible; to goafed areas that consisted of broken rock at mine level and extending several metres above. Typical roof collapse can be seen in Figure 3b



Figure 3a: Intact Workings



Figure 3b: Roof Collapse

- Panel Collapse:** panel collapse is a failure mechanism where the pillars supporting multiple adjacent roadways have crushed or been completely extracted, causing thick layers of the overlying rock to fracture and drop into the mine. Panel collapse results in upward void migration, which if continued to surface will result in surface subsidence (see below). Areas of panel collapse were identified by 'no void & no coal'. Typically, panel collapsed appeared as a small amount of crushed coal and/or rock at mine level with open, sub horizontal fracturing extending tens of metres above the mine workings. Fracturing associated with panel collapse can be seen in Figure 4, note that the particular fracture shown was encountered over 30 metres above mine roof level.

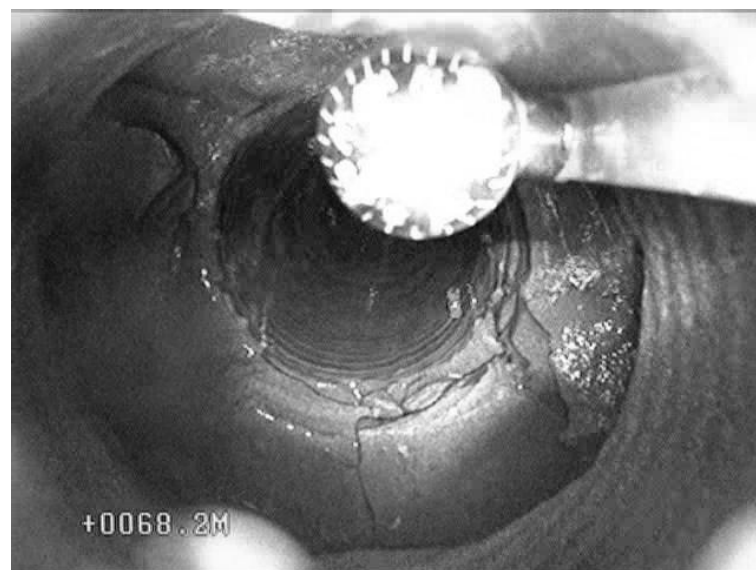


Figure 4: Horizontal Fracture above Mine Workings

- Full Subsidence:** In completely extracted or crushed areas full subsidence may occur. Full subsidence is characterised by minimal broken rock found at mine level and, because the surface has subsided, very little high level fracturing as found in areas of panel collapse. Due to the undulating terrain along the alignment of the project, subsidence could rarely be identified using surface survey.

6. VALIDATION TECHNIQUES

Upon completion of grouting, validation works were undertaken to confirm that the mine voids had been completely filled. A summary of validation techniques is listed below:

- **Core drilling:** coring was used to confirm grout presence in mine voids and in fractures above mine level. In areas where the mine roof was intact, cores taken at mine level demonstrated contact between the grout and the roof of the mine- hence proving complete filling of the void.
- **Real time camera validation:** grout flow through the mine was monitored using the wireline video cameras described in section 5.4. Whilst grouting, the cameras were lowered down boreholes adjacent to the injection point to confirm grout flow between the boreholes. Upon filling of each borehole a camera was lowered into the hole to record the standing grout level in the borehole and confirm that it was filled.
- **Grout quantities:** by monitoring the grout volumes pumped into individual boreholes, the grout takes of particular sections of the mine could be compared with theoretical grout yields (calculated from extraction ratios). Although yield data alone could not validate an area, the information did provide a degree of substantiation for validation.
- **Additional construction boreholes:** additional boreholes were drilled into grouted areas and the drill spoil was collected to examine for the presence of grout. In addition to sampling drill return, video cameras were lowered into the completed borehole to record the borehole and examine the grout at mine level. Upon completion of geophysical logging and video recording the hole was grouted, and the grout volume pumped into the hole again compared to theoretical volumes.
- **Geophysical logging:** geophysical logging in the form of wireline mounted density and gamma radiation tools was utilised when results obtained from coring and camera footage were inconclusive. Geophysical logging enabled a profile of the rock strata to be obtained hence identifying the presence of coal/grout. Data obtained from geophysical logging proved particularly useful to confirm areas of panel collapse or suspected subsidence when local grout takes were low

Industry practice typically involves coring in conjunction with monitoring of grout volumes pumped into the mine workings. The suite of validation techniques employed on the Hunter Expressway is considered to have set a new standard of industry best practice.

7. CONCLUSION

The risk posed by the presence of mine voids beneath the Hunter Expressway has necessitated one of the most sophisticated minefill projects to date. Analysis of data collected during minefill and subsequent validation operations has enabled successful completion of mine stabilisation works and provided significant improvements over standard industry practice. The minefill process allowed a particularly thorough level of validation to be conducted, while the planned and systematic approach resulted in the work being carried out within the normal range of unit costs, given the logistic constraints of the site.

8. REFERENCES

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CASE STUDY, ROCKSLIDE GEOTECHNICAL WORKS, OBI OBI ROAD, KIDAMAN CREEK, QUEENSLAND

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ABSTRACT

On 29 April 2010, a large rockslide occurred closing the Obi Obi Road, the main link road between Mapleton and Kenilworth on the Sunshine Coast Hinterland. The slide had an aerial extent of about 60m (high) by 100m (wide) with boulders up to 3 m in diameter and an estimated volume of about 20,000m³. This paper describes the geotechnical investigation methods, analysis, remedial design and construction process.

1. INTRODUCTION

This paper presents a summary of the geotechnical works completed for the Obi Obi Road rockslide remediation project, in Queensland, Australia. The rockslide occurred within a 55m high road cutting with a lateral extent of approximately 100 m. Debris including boulders up to about 2 m to 3 m in diameter ran out on to the road forcing traffic closure.

At the time of the incident, a DTMR officer was on site conducting an inspection of the site after a Council road officer had raised concern regarding a number of boulder falls in the preceding days. With the assistance of a passerby the DTMR officer was able to close the road. There were no injuries, casualties or accidents as a result of the event.

Following the rockslide, the Department of Transport and Main Roads (DTMR) commissioned Golder Associates Pty Ltd (Golder) to undertake:

1. Initial geotechnical investigation and assessment.
2. Remedial options assessment.
3. Detailed design.
4. Designer's site inspections to confirm design intent during construction.
5. Additional geotechnical investigation during the construction phase
(as access to the site was progressively obtained during bulk earthworks).
6. As constructed survey utilising photogrammetric methods.

The remedial design involved bulk earthworks to remove the debris material and re-profile the cutting a height of more than 75 m, at an overall angle of approximately 34°. Structural support comprised installation of ground anchors (passive dowels and soil nails) in pattern arrangement together with inclined drains, reinforced shotcrete, erosion control matting and re-vegetation. The drainage system was also redesigned and constructed as part of the remedial works.

2. SITE DESCRIPTION AND BACKGROUND INFORMATION

The Obi Obi Road site is located in the hinterland of the Sunshine Coast, approximately 40 km west of Nambour and 5 km east of the town of Kenilworth. At the site of the rockslide the road was formed by a large cutting up to about 55 m high on the east, with a steep natural rock slope on the west down about 20 m to Obi Obi Creek (a protected wetland).

Reference to Australian Geological Survey information indicates that the site geology is within the Devonian-Carboniferous Amamoor Beds, and is close to contacts with the Late Triassic North Arm Volcanics to the north, and an unnamed Triassic granodiorite intrusion to the east.

Informal quarry operations had been historically carried out at the site by local Council. Anecdotal information suggests that there had been previous occurrences of instability within the borrow pit and Council ceased operations. Progressive deterioration of the slope continued to occur, which eventually culminated in the sudden large-scale failure on 29 April 2010. The site location and an image of the site shortly after the rockslide event are given below in Figure 1.



Figure 1b –View of Rockslide looking north

3. GEOTECHNICAL INVESTIGATION AND ASSESSMENT WORKS

3.1 INITIAL PHASE OF GEOTECHNICAL INVESTIGATION

The initial investigation consisted of review of available data and site geomorphological mapping. The desktop study comprised review of geological information, initial design documents for the road cutting, aerial imagery, available topographic survey information prior to the rockslide, as well as discussions with Council officer's to obtain anecdotal information regarding borrow pits operations.

After the rockslide occurred safe access to the site was limited to the fringes of the debris and to the area above the backscarp. The debris slope was assessed to be too unstable to conduct any works from upon or below (i.e. any disturbance or trafficking on the debris would have been likely to cause further movement of the scree and was deemed to be too dangerous). An elevated working platform (70 m *Skylift*) was utilised to allow geotechnical engineering and geological staff to make visual appraisal of the site conditions.

Surface features were assessed, including mapping of intact rock outcrops together with appraisal of the geometry and surface features of the failure. A reflective laser survey scan of the site was undertaken to obtain the surface profile of the rockslide. Several test pits were also excavated above the crest of the backscarp following clearing of the area during the initial phase of the remedial works.

The rockslide had a near vertical backscarp in the order of 6 m to 10 m high. Initial observations from road level indicated a deep profile of residual soil and weathered rock in the northern part of the backscarp, with less weathered rock apparent in the south. However, test pit investigations above the backscarp in that area revealed a maximum thickness of about 2.5 m of residual soil and extremely weathered rock. It was therefore inferred that the rockmass within the headscarp could be smeared with soil. Scree consisted mostly cobbles to boulders less than 1 m across (with occasional boulders up to 3 m diameter) and was present to a depth of at least 5 m in places although the depth of scree could not be observed over much of the failed area.

3.2 REMEDIAL DESIGN OPTIONS ASSESSMENT

There were several main constraints that limited the feasibility of remedial design options that could be considered as feasible:

- The available geotechnical data, at the time of the remedial design options assessment was limited to information obtained from desktop study and surface mapping of the site, with no subsurface information regarding the natural ground conditions nor the depth and extent of the debris on the slope.

- Assessment of the depth of scree was based on comparison of the post and pre rockslide survey information. The prior survey information was regional information based on an aerial laser scan and the accuracy was uncertain as was the extent of borrow pits activities following that survey.
- Any earthworks or construction activities needed to be carried out from the “top-down” as disturbance to the toe of the debris slope would likely cause further instability or run out of scree. Such instability was deemed too high a risk to the safety of construction personnel/machinery working below or on the slope.
- The land ownership was divided between DTMR road reserve (along road alignment) and Council land (across the cutting area and extent of the failure area) with privately owned land above the backscarp. Access to the backscarp for top down construction was only available through two routes across parcels of privately owned land.
- Realignment of the road to the east away from the cutting was not feasible due to the steep natural rock slope and layout of the environmentally sensitive Obi Obi Creek (a protected wetland).

The following options were considered for the remedial works:

- 1) Undertake minimum works such as clearing of debris from roadway. (high risk from road safety and future maintenance costs perspective)
- 2) Re-align the road away from the failure slope utilising structural works such as building a palisade wall with cantilever bridge or a full bridge (high cost option and potential for future instability on the slope above the structure).
- 3) Undertake minimal earthworks and stabilisation of potential unstable blocks/wedges on the batter (minimal earthworks but potential for risk from road safety and future maintenance costs perspective); and
- 4) Undertake full remedial earthworks with stabilisation by structural support of potentially unstable batters and blocks. Two options with 40° and 45° degree 10 m high batters, intermediate 4 m wide benches and a 10 m horizontal buffer at the toe to the road verge were considered. The 40° option would have required acquisition of privately owned land.

The full remedial earthworks solution with 45° batters was adopted as the preferred remedial design option. A sketched section of the initial remedial design is given in Section 3.4 refer to Figure2. The design detail was given in a set of earthworks design drawings together with typical details for support arrangements such as rock bolts, passive dowels, rockfall protection mesh and anchor points.

3.3 GEOTECHNICAL WORKS AND INVESTIGATION DURING CONSTRUCTION PHASE

Construction Monitoring and Assessment

Routine inspections were carried out by geotechnical engineering staff to provide assessment of temporary stability issues (and advise construction staff of signs to monitor for) as well as for the purpose of providing foundation assessment for drainage infrastructure and the fill area.

Hold Point Inspections

Hold point inspections were required following each ‘lift’ of the excavation so that geological mapping, assessment of stability and recommendations for support measures could be carried out. The lift depth was limited to a third of an excavation on each batter (i.e. at about 3.3 m intervals) to ensure that installation of support measures and ground anchors could be achieved with the machinery available on site.

Witness Points

Witness point inspections were conducted prior to casting of shotcrete or grouting of ground anchors. The witness points were typically carried out by a DTMR inspector, with Golder, as the designer attending approximately 10 % of witness points to confirm our design assumptions.

Probe Hole Drilling

Probe holes were drilled from intermediate benches to assess the conditions in the underlying ground for the subsequent batter excavation. The probe holes were drilled utilising the rotary percussion drill rig that was on site for installation of ground anchors.

Borehole Drilling

Once the excavation had reached a depth of 20 m, two boreholes were drilled from the 2nd bench level to assess the subsurface strata with the purpose of indentifying any possible deep seated failure planes. The boreholes were drilled to a depth of 40 m with NMLC coring used to recover core samples. Monitoring instruments were installed in the boreholes consisting of inclinometers and piezometers. Ongoing monitoring during construction indicated that no movement was occurring.

Geophysical Testing

Seismic methods were utilised to assess the subsurface conditions once the excavation of the batters reached a depth of 40 m. At this depth the excavation was beginning to “daylight” into the scree and soil material at the fringes of the excavation (refer Section 3.4). The seismic technique was used to assess the profile of scree overlying the rockmass.

Laboratory Testing

Samples were obtained during the initial phase of the construction works for Emerson Class Number (ECN) testing to assess dispersivity. Several select rock samples were collected for preparation and petrographic analysis to determine the rock type so that the geological conditions could be confirmed.

Stability Analysis

Stability analyses were undertaken to assess ground support requirements, based on the potential failure mechanisms identified from the geotechnical mapping and probe/borehole drilling results. Wedge stability analyses were undertaken using the Swedge software (from RocScience), incorporating the identified wedge geometry, to calculate the Factor of Safety (FOS) of potential wedges. Stability analyses was also undertaken using the Slope/W software to assess potential circular (or truncated circular) failure of the scree and extremely weathered rock encountered in the in the southern and northern extremities of Batters 2 and 3. The purpose of these analyses was to design the soil nail support system that was ultimately installed.

3.4 SUMMARY OF INVESTIGATION FINDINGS

As construction proceeded and excavation was carried out the geological conditions became apparent. The ancient meta-siltstone bedrock was found to be cross-cut by many narrow faults and at least two different aged igneous dykes. The geological structure of the rock mass was observed to be complex, with the original sedimentary character of the rock mass having been overprinted by later metamorphic foliation, igneous intrusions and faulting. These processes have been so extensive that the original sedimentary bedding is no longer distinguishable. Typically, the meta-siltstone was found to be blocky, moderately to highly fractured, and moderately to highly weathered. The rock mass varied significantly with dilated conditions encountered where the excavation encroached on the failure surface and a large fault zone across the southern end of the upper batters.

The key pervasive structural features observed included metamorphic foliation, five major joint sets as well as several major discontinuities (faults and shears). In terms of slope stability, all of those features had the potential to act as persistent planes of weakness that could control large scale wedge failure mechanisms. The mapped foliation and one of the joint sets combined to form a kinematically feasible wedge. However, the line of intersection between these two sets was typically quite shallow ($<20^\circ$) and therefore a relatively high factor of safety.

A major fault zone in Batters 1 and 2 became less distinct in Batter 3, apparently diffusing into several smaller discrete structures towards the bottom of the batter. The zone of scree and extremely closely fractured rock encountered south of the major fault zone in Batter 2, terminated abruptly in Batter 3 along an inclined contact with the underlying bedrock. A wide fault zone (up to 20 m wide) is inferred, extending from the far southern end of Batter 3, diagonally down across Batters 4 and 5. It appeared to dip moderately towards the south or south-south-west. Off-sets in the andesite dykes suggest that this is a normal fault.

Two andesite dykes were observed in the central part of Batter 2 and from the top of Batter 3 down to Batter 5 merging at Batter 4. These andesite dykes were offset by faulting in several locations, ranged in thickness from about 0.5 m to 3 m, and the lower dyke often had thick clay seams at its contacts. Three felsic dykes were also observed in the northern end of Batter 2, two of which terminated at or above Bench 2 with only one found to continue down Batter 3 and partway down Batter 4 where it terminated.

The borehole drilling from the second bench indicated that the deeper condition of the rock generally did not deteriorate with depth with general improvement of rock quality and no indication of adverse conditions (i.e. geological contacts or fault/crushed zones) which could initiate a deep seated failure.

Oversteepening of the geologically complex slope for the initial road cutting and during subsequent informal quarry operations combined with poor drainage and ongoing deterioration of the slope was deemed to have lead to the failure.

4. REMEDIAL DESIGN

4.1 GEOMETRY AND SUPPORT OF EXCAVATION

The initial remedial earthworks design comprised seven 10 m high 45° batters, with 4 m wide intermediate benches and a 10 m horizontal buffer from the toe of the slope to the adjacent to the roadway. Batters 1 to 5B and Benches 1 to 4 were excavated and supported with the installation of ground anchors and reinforced shotcrete facing. The anchors consisted declined passive dowels and soil nails installed in a pattern arrangement at typically 2 m centres in lengths of 4 m to 9 m

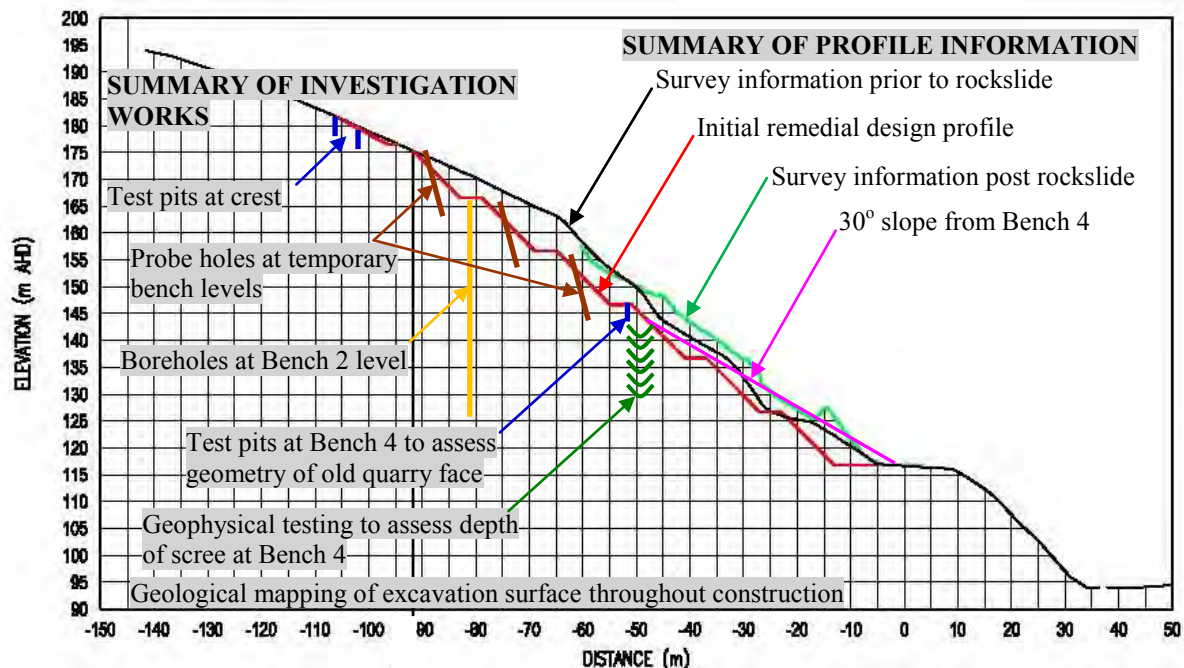


Figure 2 – Indicative Section illustrating surface profiles, investigation works and typical support arrangement

Following excavation down to Bench 4, unsuitable scree and soil materials were exposed at the southern and northern most extremities of the batter (i.e. essentially the excavation “daylighted”). Based on the ground conditions encountered, the design was revised to consist of a 30° batter from below Batter 5B to the toe of the slope. This geometry was achieved by removing the 10 m horizontal buffer at the toe as well as the two lower intermediate benches. Due to the geometry of the site, further down the slope the 30° batter daylighted above the toe of the slope adjacent to the roadway from chainages CH-25m to CH43m (ref Figure 1). The rock face exposed in this area (designated Batter 7C) was supported by installation of ground anchors and reinforced shotcrete facing.

A portion of the materials excavated from site were placed as fill in the southern gully. This was done to minimise costs associated with export of the materials from site. The balance of material was exported from site. The near vertical face of the pre-existing quarry to the south of the excavation and above the southern gully was also supported by installation of ground anchors and shotcrete facing. Photographs 1 and 2 illustrating the general construction details are shown below.

4.2 DRAINAGE

Surface drainage across the site was controlled by the construction of:

- A diversion drain above the crest of the batter;
- Longitudinal drains formed along the benches at the toe of each batter; which outlet to
- Bench pools on the southern end of each of the benches; outletting to
- Chute drains formed down the southern end of each batter (including the 30° slope) with flows controlled with the intermediate bench pools on Benches 1, 2 and 3 an inlet structure formed on Bench 4 to collect water from the chute drain above, and then direct it into the chute drain down the 30° slope; to
- An inlet chamber at the roadside; with
- A table drain formed along the toe of the slope to collect sheet flow off the 30° slope; all to
- A culvert under the road and into the Obi Obi Creek.

Within the southern gully fill a diversion drain was formed above the fill embankment to cut off surface water flows onto the fill area. Subsurface drains (‘agi-lines’) were also installed in wet areas during subgrade preparation.

4.3 AS CONSTRUCTED SURVEY

As constructed survey of the site remedial works was undertaken utilising photogrammetric methods. Digital images of the works were taken from a camera mounted on a remote controlled helicopter. The images were taken in overlapping stereo pairs with ground control survey points included in the images and then digitised into a Digital Terrain Model (DTM) utilising the software ADAM. Further discussion on the methodology and techniques used are given in the paper prepared by Booth, Starr and Woodsford for the 2012 ANZ Geotechnical Conference.



Photograph 1: General Site Appearance Looking South



Photograph 2: General Site Appearance Looking North-West

5. CONCLUSION

A large scale rockslide occurred in a road cutting at Obi Obi Road, Kidaman Creek, Queensland on 29 April 2010 with initial remedial works comprising bulk excavation based on review of available data and surfacial assessment of the site as access was not possible to the slope. Subsequent supervision, investigation and assessment of the works were undertaken as the construction proceeded to enable the remedial works design and construction to proceed. The final slope design consisted a hybrid type configuration with the upper parts battered with intermediate benches and the lower slope formed at 30° due to the excavation “daylighting” approximately 40 m below the crest of the excavation.

The road was reopened in July 2011, 14 months after the failure, following excavation of approximately 73,000 m³ of debris, scree, soil and rock, installation of ground anchors totally a length more than 7 km, and placement of 1,400 m³ of shotcrete together with drainage works.

6. ACKNOWLEDGEMENTS

I would like to thank DTMR for granting permission to publish this paper.

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RETAINING WALL AT HAYWARDS SUBSTATION FOR A NEW CONVERTER BUILDING

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ABSTRACT

Transpower NZ Ltd. has undertaken extensive site improvement works to accommodate replacement of the Pole 1 converter building at Haywards Substation (Wellington) and to seismically strengthen existing infrastructure to current design standards. As part of the improvement works, a tied-back bored pile retaining wall was built to facilitate siting of the new Pole 3 building close to, and at a lower level to, the existing Pole 1 building. The bored piles were tied back using screw piles as passive anchors. Difficulties arose when the screw piles had to penetrate variable ground conditions and fill materials. The fill material included concrete debris from past construction and dense gravel layers. As the site is situated close to the active Wellington Fault, the key geotechnical issue was that design needed to accommodate a 1g horizontal ground acceleration while limiting the amount of movement during a 1 in 2500-year earthquake.

1. INTRODUCTION

The high voltage direct current (HVDC) link connects electricity transmission networks between the South Island and North Island of New Zealand. In the South Island, the link starts at Benmore Hydroelectric Power Station, converting power from alternating current (AC) to direct current (DC). DC is converted back to AC in the North Island at Haywards Substation, Wellington. The course can also be reversed from North Island to South Island if required. Haywards Substation has undergone many developments over the years and the recent one comprises the addition of a new Pole 3 converter building. The upgrade was to increase the capacity of the HVDC link from 1000MW to 1200MW and to ensure current seismic criteria were met.

There are four switchyard platforms shown in Figure 1; three existing (A, B and D) and one proposed (C). The existing substation has been constructed on three levels as the topography is steep with rugged hillsides. Switchyard A is on the upper platform, the existing Pole 1 building and DC switchyard are located on mid-level Platform D, and Platform B is located on the lower level.

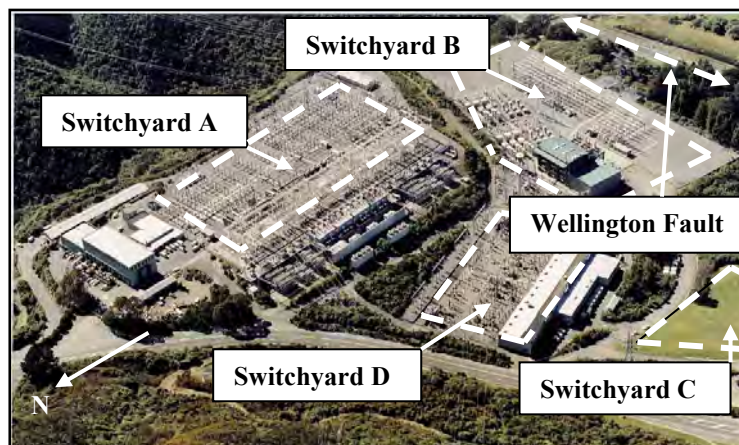


Figure 1: Hayward Substation Oblique Photo

The key geotechnical issue at Haywards is the high seismic design loading. Haywards Substation is located 200m away from the active Wellington Fault. Current ultimate limit state (ULS) peak ground acceleration (PGA) for design is 1.01g for a 1 in 2,500-year event. Ground damage as a result of earthquake shaking may not be easily repaired and may further result in permanent damage that is irreparable. The adopted criteria is two-fold; ground displacement should be less than 20mm at design level (1 in 1,000-year event, PGA 0.7g) to limit damage to structures and equipment, and under ULS should be less than 200mm to prevent collapse of structures.

Therefore, a number of slopes affecting existing and proposed structures needed to be strengthened to mitigate the potentially excessive permanent ground displacement under the design earthquake. Specifically, this case history

discussion will focus on the DC switchyard extension retaining wall which was constructed to retain the cut area between mid-level platform (Switchyard D) and the bottom level platform (Switchyard B). The wall was required so that the new Pole 3 building could be constructed close to the existing Pole 1 building but at a lower elevation. The retaining wall is made up of 30 bored piles and 94 screw piles as tie-back anchorages.

2. GEOLOGICAL SETTING AND TOPOGRAPHY

The principal mapped geological unit from published sources is Rakaia terrane greywacke of late Triassic to early Jurassic age. The greywacke comprises indurated grey, quartzo-feldspathic sandstone-mudstone sequences. Deformation of these rocks is common, caused by multiple folding events and widespread faulting, leading to highly fractured rock masses. There are a number of south-west to north-east trending faults in the vicinity of the site. Active strike-slip faulting occurs 200m to the east associated with the Wellington Fault and 5km to the west on the Moonshine Fault. These faults are typical of the strong NE/SW structural alignment in the Wellington region.

The former steep hilly topography has been significantly altered by past cut-and-fill earthworks to develop the substation. The Y-shape deep gully through the site was filled in the 1960's with limited compaction control with 'non-engineered' fill, creating parts of Switchyard D and Switchyard B and access to Switchyard B. Ground conditions at the retaining wall comprise this fill and alluvium overlying highly fractured greywacke rock (at 26m depth). Figure 2 indicates the location of the Y-shape deep gully fill (to 30m thick) and the retaining wall.

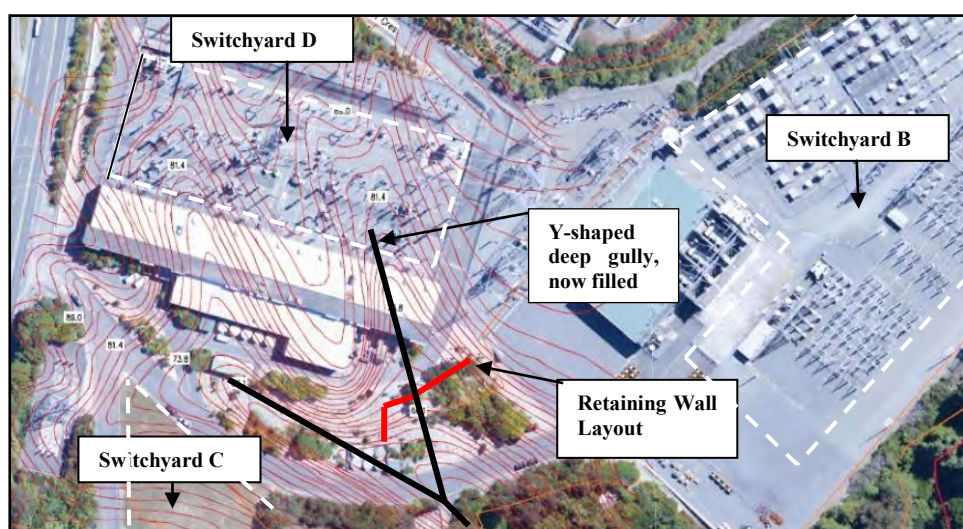


Figure 2: Contour plan of gully before it was filled to form Switchyard platform D

3. GEOTECHNICAL AND SEISMIC DESIGN PARAMETERS

Design parameters are shown in Table 1 and Table 2. The retaining wall is embedded in and retains non-engineered fill. The generalised Hoek-Brown criterion is used in stability analyses for rock. Equivalent Mohr-Coulomb parameters shown in Table 2 apply at the tangent to the Hoek-Brown failure envelope at a confining stress of 100 kPa.

Property	Closely jointed greywacke ¹		
Weathering Grade	Completely	Highly	Moderately
Unconfined Compressive Strength (MPa)	2	5	15
GSI (geological strength index)	20	25	30
m_i (greywacke)	15	15	16
Disturbance factor, D	0 (nil disturbance)		

1. Closely jointed greywacke includes the GSI rock structure categories "Blocky/ Disturbed/ Seamy", "Disintegrated" and "Laminated/ Sheared"

Table 1: Greywacke Rock Mass Hoek-Brown Failure Criterion Parameters

Geological Unit	Unit Weight (kN/m ³)	Effective Cohesion, c' (kPa)	Effective Friction Angle, Φ' (degrees)	Undrained Shear Strength (kPa)
Platform D and gully fill (non-engineered)	18	0	35	120
HW Closely jointed greywacke ¹	20	30	45	N/A

Table 2: Geotechnical Design Parameters

A site specific assessment carried out by GNS Science provided the latest seismic design accelerations. The peak ground acceleration (PGA) for Class C shallow site (applicable to most areas in the substation) at ULS for civil structures (100 years design life and Importance Level 4) is 1.01g, compared to 0.96g in 2007 and 0.7g when the substation underwent redevelopment in the early 1990's. The increase in the seismic load results in many existing structures being under designed with respect to current seismic criteria.

4. GROUND DISPLACEMENT AND SLOPE STABILITY ANALYSIS

Analysis of seismic ground displacements for the infilled Y-shaped gully, using FLAC, indicated potential maximum displacements between 0.6m and 0.75m at ULS loading. This exceeds the allowable permanent ground displacement limit of 200mm (ULS).

Based on the analysis, the palisade wall of bored shear piles was capable of limiting wall displacements locally to within the proposed limits at the new Pole 3 converter building area. However, the design has also allowed for the movement of the gully fill carrying with it the retaining wall. The client has agreed that the retaining wall only restricts the movement of the ground retained but not the gully fill movement globally. For static stability, only the top anchors are required. The four rows were required to resist the high seismic loading and limit wall displacement (Figure 4A). Slope stability analysis using Slide (by Rocscience) showed that if the cut area was supported with a retaining wall (without anchors) an overall factor of safety of 1.3 could be achieved under 1.01 PGA.

5. RETAINING WALL DESIGN

The retaining wall was modelled using WALLAP (by Geosolve) using anchored and cantilevered retaining wall analysis. In the analysis it was assumed that additional surcharge load of 5kPa (within 5m of the top of the wall) and 10kPa beyond 5m, for both static and seismic cases, to model loads from the traffic on the access road. The proposed wall comprises 31 bored piles (750mm diameter) with 94 screw piles at various heights as the tie backs. The wall is approximately 60m long with a highest retained height of 7.5m above switchyard B. Table 3 summarises the WALLAP analysis results.

Wall height (m)	Load case	Maximum anchor force (kN) ¹	Max. wall deflection (mm)	No. of anchor rows
7.5	Seismic	410	54	4
	Static	104	13	
6.2	Seismic	435	60	3
	Static	71	10	
5.2	Seismic	454	53	2
	Static	99	4	
4.2	Seismic	273	41	2
	Static	32	7	
3.2	Seismic	562	60	1
	Static	0	11	

1. Wall is tied back using multi row anchor (0 to 4 rows). Anchor forces are unfactored and the forces shown are only for anchors at the top of wall.

Table 3: Summary of WALLAP Analysis Result

6. RETAINING WALL CONSTRUCTION

In total, 30 bored piles (750mm diameter) were installed. These piles acted as a shear key to stabilise the cut area. Figure 4 shows the elevation and plan of the piled wall and tie back anchors.

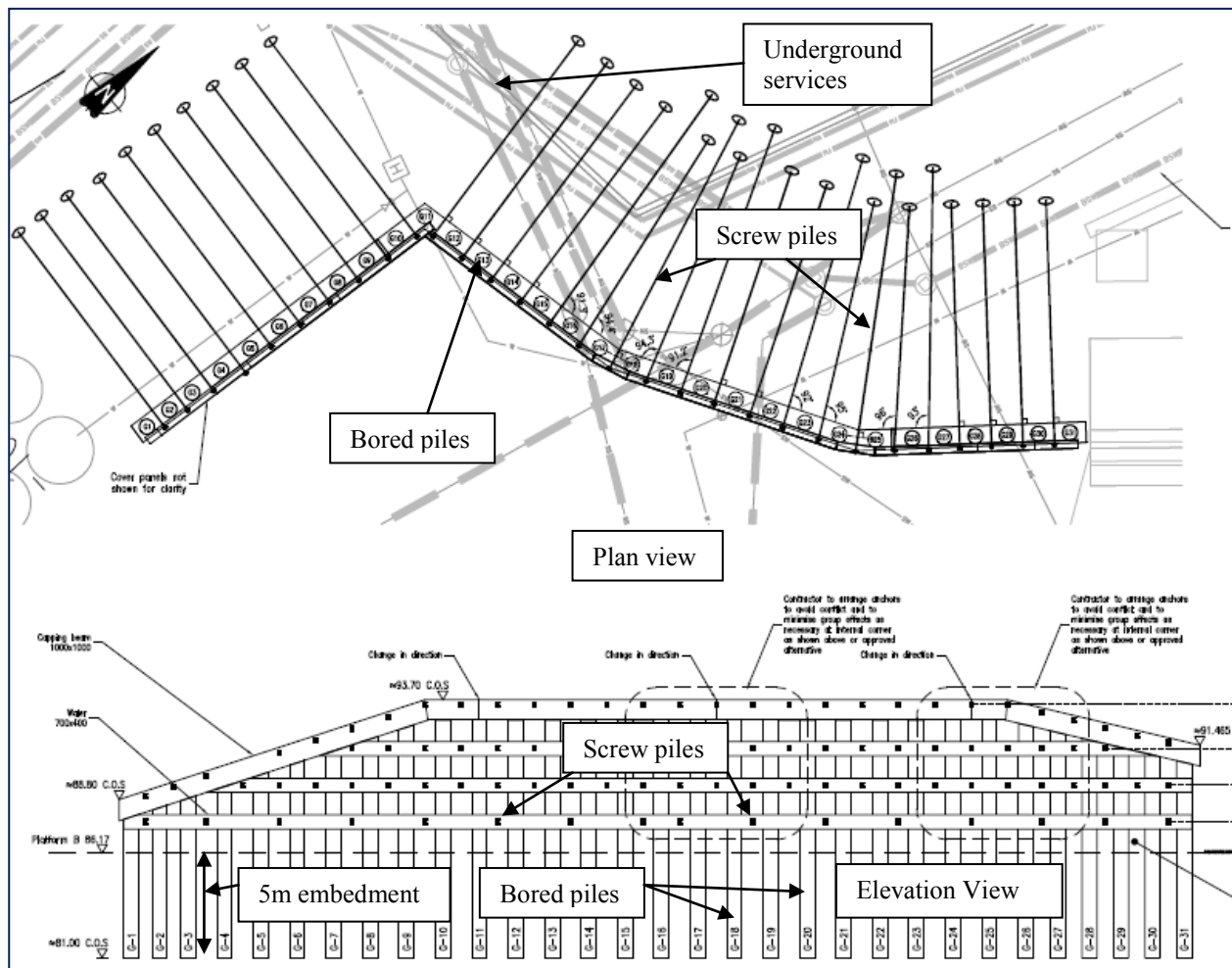


Figure 3: Elevation and Plan of Retaining Wall

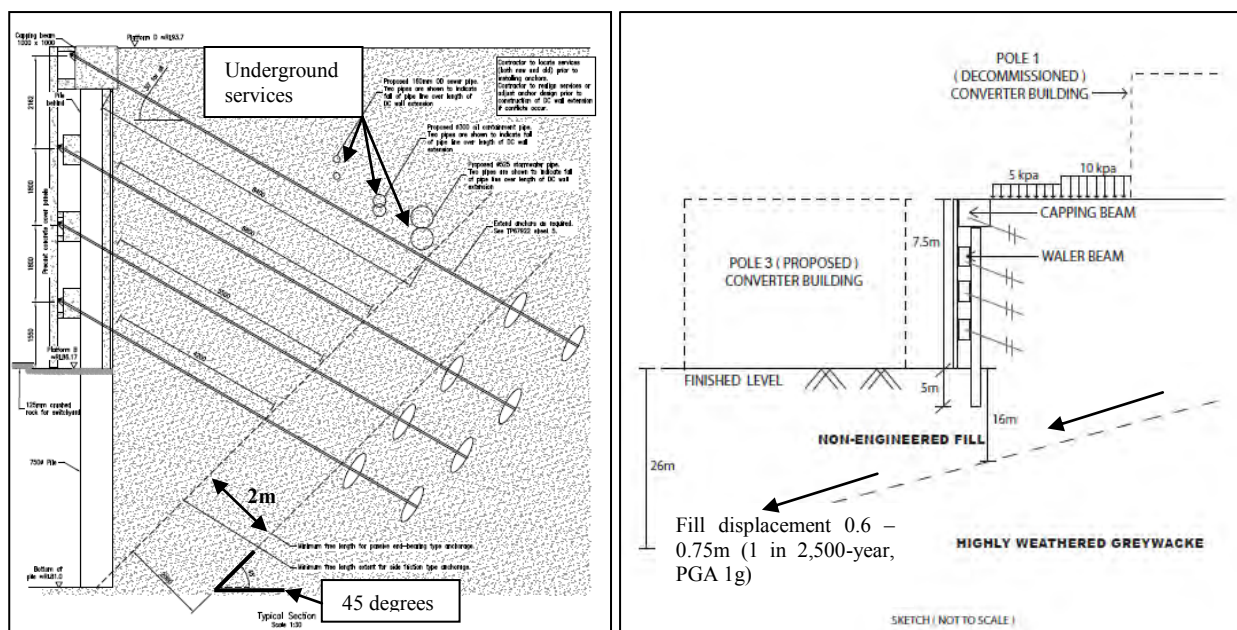


Figure 4: Typical Section through Retaining Wall

In total, 94 screw piles were installed and the lengths varied depending on the wall height above finished level. At higher elevations longer screw piles were required to reach the passive zone. The design assumed an active wedge of 45 degrees, inclined to the horizontal, with minimum clearance of 2m between the edge of the active zone to the first helices (refer to Figure 4). After review of the drillhole logs (especially the SPT N-values), the supplier of the screw piles initially selected the thicknesses of the shaft and helices to be used for the proof test. The initial proof test caused the screw pile with a single helix to fail as the ground encountered was reported to be denser than that purported by the investigations. The second proof test was undertaken with a double helix. The third proof test with a double helix confirmed the need to use larger machinery to screw the piles into the ground. The key point in designing the pitch and spacing between helices was to ensure that helices will groove harmoniously into the ground with the least soil disturbance. Shaft and helices thickness included a sacrificial thickness of 3mm for corrosion in accordance to AS2159 for a 100 year design life. Details of the proof load test screw pile are given in Table 4.

		Proof Test 1	Proof Test 2	Proof Test 3
Shaft	Outer diameter, mm	219	219	219
	Wall thickness, mm	10	10	10
Helix	Outer diameter, mm	650	650	650
	Wall thickness, mm	25	20	20
	Pitch: (inner), mm	120	116	116
	Pitch: (outer), mm	120	N/A	N/A
	Number of helices:	1	2	2
	Helix locations (distance from toe, mm):	0	0	0
			1680 (2.6D)	1680 (2.6D)
	Steel grade (MPa)	350		
	Surface coating	N/A	N/A	N/A
Bottom End	Angle cut	Yes	Yes	Yes
	End plate	Yes	Yes	Yes
	Concrete fill	No	No	No

Table 4: Details of Proof Load Test Screw Piles

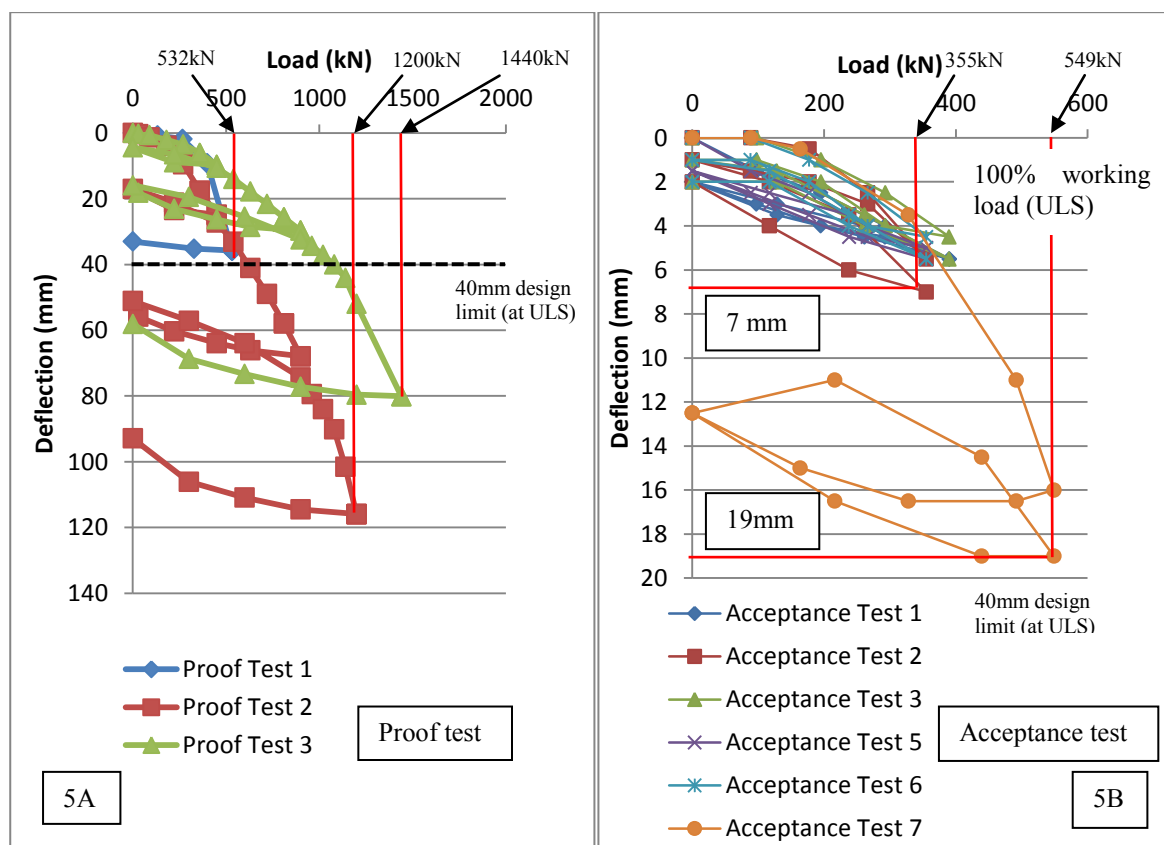


Figure 5: Proof Tests and Acceptance Results

Proof load test results, shown in Figure 5A, show that increasing the number of helices almost doubled the pull-out resistance (also refer Table 4). Screw pile capacity of a single helix (Proof test 1) with factor of safety (FOS) 2 was limited to 266kN. Screw pile capacity of a double helix (Proof test 2 and 3) with FOS 2 was 600 to 720kN but limiting deflection to 40mm, the maximum useable capacity was limited to 600kN. Proof load tests were undertaken vertically on level ground and the supplier used the results to derive the torque factor for working anchors installation. The torque factor correlation with screw pile capacity was then used to estimate the screw pile capacity at 30 degrees inclination.

Upon completion of the wall the supplier was required to carry out acceptance tests on working anchors. Both proof and acceptance test were in accordance to AS 2159 – 1995 which recommends acceptance tests to be carried out for 10% of the total number of screw pile installed and selected by the engineer. Acceptance load tests are to 100% of the scheduled design working load, with 40mm maximum deflection allowed. The acceptance test results in Figure 5B illustrate that deflection was typically less than 8mm for a test load of 355kN and less than 20mm for a test load of 549kN. Comparison of proof load and acceptance tests indicated that the production screw piles would perform satisfactorily.

7. CONSTRUCTION DIFFICULTIES AND RISKS

The difficulties and risks during construction are as follow

- Screw piles had to penetrate fill material. The fill is a mixed of clayey silt or silty clay material with some concrete debris from the past construction. A few screw piles had their helices separated from the shaft. Increasing thickness of the weld and screw piles plate mitigate this risk.
- One bored pile was eliminated due to the existence of old tower foundation (unforeseen).
- There were many underground services identified immediately behind the wall and resulting in screw piles installation to be in parallel with the excavation and replacement of ground services to ensure no conflict occurred with any services.
- Major construction risk on the sequence for acceptance testing. Acceptance tests took place after the waler beam had been completed. If the screw pile failed the acceptance test, major remediation work had to take place. Final results of the acceptance tests were a pass for all 9 screw piles tested.
- Contractors have to work simultaneously with many heavy machinery activities in a confined area.

8. CONCLUSIONS

Observations and lessons from this project were:

1. It is prudent to study the site as a whole initially and design the local solutions. In this case gully fill movement during an earthquake was allowed for in design but local displacements of the retaining wall was restricted to agreed limits (with the client).
2. A collaborative work programme, and team work between the contractor, client and consultant helped achieve completion of construction in a timely manner even with unforeseen problems such as unknown old foundations and many underground services.
3. Screw piles perform satisfactorily. This technique has good quality control and a quick installation time.
4. Installing screw piles as inclined ground anchors results in slower production rates than for vertical screw piles as the site had limited space.

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A CASE STUDY HIGHLIGHTING MULTIDISCIPLINARY DESIGN & QUALITY ASSURANCE PROJECT COMPLEXITIES

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ABSTRACT

Design projects often require multidisciplinary teams to provide suitable designs, drawing packages, technical specifications and quality assurance services during construction activities. Even on simple projects, the coordination, compiling of design packages, and provision of quality assurance services can be a difficult and demanding task. The complexities are magnified when the multidisciplinary team includes external consultants, the design is for a foreign client on a remote site, and the construction contractor is also the owner/client.

This paper presents a case study for the design/construction of a sediment dam and diversion structure on a remote island in South East Asia. The paper summarises the design process, client interactions, construction and quality assurance activities provided throughout the project. The focus of the paper is to highlight the many complexities which may arise during projects of a similar nature which must be dealt with by the engineer.

The Author is a geotechnical engineer who was involved in the design and management of a multidisciplinary team as well as acting in the role of site engineer in charge of quality assurance (on behalf of the client) during the construction phase.

1. INTRODUCTION

Multidisciplinary input is required for the vast majority of infrastructure/construction projects. These days a large proportion of consulting companies are endeavoring to provide a 'one stop shop' for clients. This 'one stop shop' can include multidisciplinary teams spanning all disciplines (i.e. geotechnical and civil through to electrical). Managing multidisciplinary teams is an arduous task when the entire team works under one company banner (i.e. large consulting firm). The management complexities multiply with smaller consultancies, as they often require input from external consultants to provide the 'one stop shop' for clients. The Author works in a small civil and geotechnical consulting firm (Galt Geotechnics Pty Ltd – Galt) which occasionally uses external consultants in order to provide a complete service for its clients. Sourcing external consultants in the current period of strong economic growth and high demand in Western Australia is yet another difficulty. All projects are different and can be managed in a multitude of ways depending on the parties involved and the intended outcomes of the project itself. A general project delivery structure for design/build type projects can be summarised as follows:

1. **Project Definition:** done by client or in association with the preferred design consultant.
2. **Design Award:** client awards design contract to preferred design consultant.
3. **Design Phase:** design consultant produces design to be approved by client.
4. **Construction Tender Phase:** design consultant provides drawing package and technical specifications for contractor tendering.
5. **Contractor Award:** preferred contractor is awarded project (generally based on understanding, proven track record and tender price).
6. **Construction Phase:** design consultant provides quality assurance services during construction phase to answer technical queries and ensure the contractor adheres to the drawing package and technical specification.

Even on simple projects the above process can be difficult and demanding. The complexities are magnified when the design is for a foreign client on a remote site and the construction contractor is also the owner/client (i.e. exclude items 4 and 5 from the above project delivery structure). This paper presents a case study relating to the Author's involvement in the above situation as an attempt to highlight the many difficulties which may arise during projects of a similar nature. Although consulting geotechnical engineers (particularly those involved primarily in site investigations) often have a narrow view when involved in specific projects, the geotechnical engineer is in fact part of a larger team (i.e. project managers, designers, engineers, etc) that works together to deliver a project. The project discussed in this paper highlighted that fact and provided valuable learnings for the Author.

The case study briefly summarises some of the challenges that confronted the Author and Galt during the design and construction phases of the project. Challenges relating to interaction with the client during the project are also discussed.

2. CASE STUDY OVERVIEW

Galt was contracted through a foreign consultancy (FC) to provide design and quality assurance services for the construction of a sediment dam and diversion structure at a remote mine site on an island in South East Asia. The FC undertook geotechnical investigations of the sites and provided full time, on site construction quality assurance personnel (managed by Galt) throughout the construction phase. Galt provided the following services:

- Direct liaison with the client throughout the project.
- Detailed design including drawing package and technical specification for a sediment dam.
- Detailed design including a drawing package and technical specification for a diversion structure.
- Management of quality assurance activities throughout the construction phase including monthly site visits by Galt personnel.

The construction of the sediment dam and diversion structure was undertaken by the client, who hired an expatriate superintendent and local supervisors to manage a local labour force. For the purposes of this paper, “the contractor” refers to the actions undertaken by the superintendent/supervisors and “the client” refers to the project owner who was ultimately responsible for the actions of the contractor. “The client” and “the contractor” are different arms of the same entity, which owns and operates the mine.

3. DESIGN PHASE

Galt undertook the majority of the required design ‘in house’ with the following multidisciplinary input/services obtained through a number of external consultants:

- **Hydraulic Analysis:** sediment dam spillway and diversion structure.
- **Structural Analysis:** reinforced concrete design.
- **Drafting/Civil Design:** drawing packages.

The Author found that acquiring local (Perth, Western Australia based) external consulting services for the above (relatively small) packages of work proved to be the first of many obstacles encountered during the project. Consulting services for the hydraulic analysis package proved to be the most painstaking as a number of existing industry contacts were unable to assist in the required timeframe. This forced the Author to resort to ‘cold calling’ local consultants which resulted in a number of meetings without successful outcomes. The end result was a delay in the hydraulic analysis which was ultimately undertaken by an existing industry contact.

The Author found that management and liaison with the external consultants proved to occupy a larger than expected proportion of time. Concepts are generally more clearly grasped through face-to-face interactions which proved to be difficult as each component of the multidisciplinary team were working in different offices across the city and in the case of the Drafting/Civil Design consultant, after regular business hours. Even simple changes to drawings, which may only require a five minute face-to-face conversation, required long-winded emails with numerous sketches and follow up telephone calls. With no regular face-to-face communication with the external consultants, misunderstandings and miscommunications did occur and occasionally required re-analyses and reissue of portions of the design package. These occurrences were both time-consuming and costly.

4. CLIENT INTERACTION

When working for a foreign client in a foreign country, it is expected that there will be cultural and professional differences between the working parties. Having an understanding of these differences is an important first step to maintaining an effective working relationship. When working in a foreign country, one must remember that he/she should adapt to the cultural differences rather than expect others to accommodate his/her differences. Typical Asian cultural/interaction traits which differ greatly from western traits can be summarised as follows (Katz, 2007):

- Strongly group oriented: individual preferences are less important than having a sense of group belonging and maintaining harmony among its members.

- Concept of 'saving face': reluctant to admit wrongs or misunderstandings. Do not want to disappoint. Reputation and social standing strongly depend on a person's ability to control emotions and remain friendly at all times.
- Confrontation is avoided: problem or areas of difference are alluded to in an indirect manner.
- Limitless time: do not like to be hurried or have Western punctuality forced upon them. Have little sense of urgency.

Our main client contact, a relatively young local engineer for whom English is a second language, displayed some of the above traits. He would often not admit to any wrong doing or mistakes in judgment. It seemed that he did not want to disappoint his superiors and to that end often ignored Galt's advice when it would impact on cost or schedule.

Interactions with the contractor were difficult as the supervisors spoke little to no English. They were also reluctant to admit that they did not understand directions given by Galt. 'Yes' was the answer to most questions as they did not want to disappoint or admit misunderstandings. This took some getting used to as 'Yes' commonly meant 'I don't understand'. This led to the Author learning some local words and phrases which greatly assisted in interactions and with the contractor relationship.

Having an understanding of the different cultural/interaction traits greatly assisted Galt in working with the client and contractor. The majority of differences were easily accepted once understood. However, the little sense of urgency was more difficult to accept as urgency was required during some portions of the works.

Project specific interactions with the client and contractor are discussed in the next section.

5. CONSTRUCTION PHASE QUALITY ASSURANCE

5.1 GENERAL

Daniel and Koerner (2007) describe construction quality assurance (CQA) as "a planned system of activities that provides the owner and permitting agency assurance that the facility was constructed as specified in the design. CQA includes inspections, verification, audits and the evaluation of materials and workmanship necessary to determine and document the quality of the constructed facility. CQA refers to measures taken by the CQA organisation to assess if the installer or contractor is in compliance with the plans and specification for a project."

The complexities during construction revolved around the fact that the client was directly responsible for the actions of the contractor and was usually unwilling to remediate works which would commonly be considered substandard if constructed by a general contractor (who would otherwise be awarded the contract through a 'conventional relationship' with the project owner). The specifications for the project were largely disregarded by the contractor whose actions were on most occasions supported by the client. This resulted in the client often ignoring our advice through the construction phase. This section summarises a number of areas where our advice was unheeded and highlights the consequences which were ultimately accepted by the client.

If the actions summarised below were undertaken by a general contractor, the client would likely not have accepted the works as a whole. However, as the client was essentially both the project owner and contractor, it often did not agree with issues raised by Galt and accepted at times, poor construction practice.

5.2 THE RIGHT TOOL FOR THE JOB

Acquiring unplanned tools at short notice was difficult given the remote nature of the site and the cost consciousness of the client. This was compounded by the client's poor planning in resourcing the projects for the construction phase.

During the first concrete pour, the Author observed that the contractor was using makeshift trowels and chisels made from formwork and rebar cutoffs. These tools provided a poor concrete finish and made remedial works lengthy and in some instances unachievable. The client indicated that proper tools had not been ordered for the project. However, after lengthy discussions, the client agreed to order the tools which arrived well into the construction schedule. The poor concrete finish would slow water (due to increased roughness) and therefore increase the flow depth within the structures (Massey, 1970). Checks by the hydraulic sub-consultant showed that the increase in flow depth could be accommodated within the design.

Prior to construction commencing, the client indicated that all concrete walls for the sediment dam spillway and diversion structure would be poured using the client-owned pump truck. The pump truck provided the best placement method for the walls due to access constraints and that it would allow the concrete to have an unrestricted vertical drop into the forms and minimise segregation (Basham, 1993). However, when wall construction began, concrete was being dumped into the wall formwork with an excavator bucket. The client indicated that the pump truck was broken and

would be fixed promptly. When the pump truck was in working order it was still not used, as there were no personnel on site who were qualified to operate it.

Given the relatively narrow walls (300 mm) the concrete was visibly segregating when dropped into the formwork using the excavator bucket. The concrete segregation was compounded by the fact that the contractor was using the vibrator to move the concrete horizontally (Basham, 1993) and the vibrator hose was not long enough to reach the base of the formwork. All of this resulted in large scale 'honeycombing' of the walls as shown in Figure 1.



Figure 1: Concrete 'honeycomb' structure within diversion structure wall

Lengthy discussions with the client and contractor resulted in ordering of new vibrators and development of a revised pouring method which minimised 'honeycombing' of the concrete.

5.3 POOR PLANNING

The contractor's poor construction planning resulted in numerous issues as well as areas of substandard construction. This is illustrated with the following situations:

- A non continuous supply of concrete during pouring was a recurring planning problem which was an issue throughout the construction phase. Regularly, slab/wall pours would commence and then suffer delays in supply of concrete as mixer trucks would either be diverted to other works or break down on route to the site. These delays would range from hours to days. The contractor also regularly took lunch breaks during pours requiring fully loaded mixer trucks to standby for over an hour. These delays resulted in a number of cold joints within the slabs and walls of the structures.
- Prior to a concrete pour for the sediment dam spillway, the contractor indicated to the Author that access was not an issue for the pour. The contractor was confident that a long reach excavator could deliver the concrete to the spillway slab. However, the contractor was unable to reach the centre of the slab and without consulting with the Author, excavated into the final trimmed upstream slope of the clay core sediment dam to allow for access. This resulted in extensive repair works of the slope.

5.4 DIFFERENCE OF OPINION

Disagreements regarding construction practices were frequent. The requirements of the specification were almost completely disregarded by the client/contractor. This was particularly common during concrete works where Galt's advice regarding maximum pour volumes, formwork removal and concrete curing were ignored. This advice related to minimising the potential for shrinkage cracks within the concrete which, overtime, could propagate and lead to hydraulic jacking of the slabs (Frizell, 2007) and ultimate failure of the structures.

Through consultation with the consulting structural engineer, the specified maximum length of concrete pours for the slabs/walls of the diversion structure were 10 m. The value was based on a maximum concrete volume between construction joints and was a 'rule of thumb' followed by the structural engineer. The client disputed the requirement and ultimately conducted 20 m concrete pours to accelerate the construction schedule.

The specification required that the concrete reach 80% of its design strength prior to formwork being removed. However, the contractor began removing formwork the day after the first concrete pour (less than 24 hours after pouring). As the contractor was reusing the formwork and had limited supplies, the client considered it necessary to

strip the forms shortly after pouring to maintain the construction schedule. The client again ignored Galt's advice and allowed the contractor to remove the formwork after 24 hours regardless of the compressive strength testing results.

During regular inspections by the Author, it was noted that the contractor was not curing the concrete or protecting it from direct sunlight after pouring. The client indicated that curing was not regularly done on site and was not necessary for these works. The client also indicated that it did not have the materials or resources to cure the concrete as specified in the specification and therefore no curing was done.

Inspections (by the Author) of the walls and slabs approximately 2 months after pouring revealed countless discontinuous cracks (as shown in Figure 2), which are typical of shrinkage cracking (InspectAPedia, 2011).



Figure 2: Typical concrete shrinkage crack

5.5 CQA SUMMARY

Galt's CQA involvement identified a number of areas where the contractor was not in compliance with the drawings and specifications. These areas were promptly discussed with the client/contractor, however Galt's advice was not followed in all instances, particularly in areas which affect the long term serviceability of the structures. Deviations from our advice are unlikely to result in catastrophic failure of the structures provided inspections/maintenance is carried out on a regular basis. However, it is likely that the structures will require a higher degree of maintenance throughout their serviceable lives than envisaged in the design. This point was highlighted to the client.

6. LESSONS LEARNT

Although the project resulted in abundant headaches and stresses for the Author, it also proved to provide the following learnings:

- Extensive multidisciplinary contact networks are crucial for small consultancies.
- Work with external consultants that are both known and trusted.
- Mutual working agreements between small consultancies will assist in providing services in a timely manner.
- Regular face-to-face communication reduces misunderstandings between working parties and will generally save both time and money when compared to telephone/email communication alone.
- When working in a foreign land/for a foreign client, familiarise yourself with the appropriate standards and relevant cultural/interaction traits.
- Learning a foreign language (even if just a few phrases) goes a long way to building relationships with the locals.
- Acceptable construction practices vary widely between countries.
- Where the project owner is also the contractor:
 - the project may be cost/schedule driven rather than quality driven;
 - the consultant may be pressured to approve substandard works; and
 - the consultant should highlight the critical points to the client prior to the commencement of construction activities.
- A risk based assessment (focusing both on project complexities and client relationship) should be completed by the consultant prior to undertaking any high risk projects to assess if the risks outweigh the rewards

7. CONCLUSIONS

The time required to assemble and manage multidisciplinary project teams should not be underestimated, especially where these teams are made up of external consultants working in separate offices. Where the project is for a foreign client one must take the time to understand the cultural and professional difference between the working parties. Where the project is also in a foreign country, one must make the effort to accommodate these differences to ensure an effective working relationship.

On projects where the project owner is also the contractor, the consultant should be aware that advice may not be 'blindly' accepted and may require extensive justification. Advice having negative impacts on cost/schedule may not be directly followed, if at all. The client may also pressure the consultant to accept practices which would normally be considered marginal or substandard in an effort to cut costs and/or save on schedule. Prior to construction, it may be useful for the consultant to highlight to the client the critical portions of the works with a view of reiterating the necessity to adhere to the design and specifications.

This paper has documented some of the various complexities an engineer may face in a multidisciplinary design and quality assurance project undertaken in a foreign country. The difficulties presented were varied and are likely to be faced by engineers at all levels during projects of a similar nature. However, young engineers, such as the Author, are unlikely to find their knowledge and experiences have prepared them to singlehandedly manage all situations and will therefore need to rely on the experience of mentors and supervisors. In difficult situations one should not hesitate to ask questions or admit when one 'does not know the answer'. Pressure from clients and contractors will always be present and it's important to assess each situation separately and how variations from a design may affect the end product. By presenting the affects and documenting the variations, one is able to allow the client/contractor to make an informed decision and proceed at their own risk, where the consultant's advice is not followed. In these cases, although not ideal, the young engineer can be assured they have managed the situation both competently and ethically. However, it is noted that the consultant will invariably be considered responsible in the event that the design does not perform as envisaged, regardless of communications detailing variations from the consultant's advice.

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DESIGN AND FABRICATION OF A LOW COST TRUE TRIAXIAL CELL FOR TESTING MULTIPLE SIZE SPECIMENS

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ABSTRACT

Only a true triaxial cell has the ability to recreate 3D in situ stress conditions. Consequently this ability means research can be conducted into the true relationship between 3D stress and rock strength. Therefore a true triaxial cell was designed and fabricated. The cell was calibrated to ensure the lateral strains on the specimen were known during testing and a standard operating procedure was created. The first true triaxial tests in South Australia were conducted. This paper describes the design, fabrication, calibration and preliminary use of a low cost true triaxial cell capable of testing rock specimens of various sizes.

1 INTRODUCTION

Rock stresses can be simplified into three principal stresses: the major principal stress σ_1 , intermediate principal stress σ_2 , and the minor principal stress σ_3 (where $\sigma_1 \geq \sigma_2 \geq \sigma_3$). Only a true triaxial cell has the ability to independently apply three non-equal principal stresses to a rock specimen to estimate its strength. Currently rock strength is tested using conventional triaxial tests. These tests can apply no more than two non-equal principal stresses and hence can only provide rock strength information for one type of 3D stress configuration.

A true triaxial cell has been developed to provide true triaxial data for rocks.

2 PROBLEM DEFINITION

The true triaxial cell was designed to satisfy the following criteria:

- Independently apply three principal stresses to cubic rock specimens
- Have a design strength to withstand at least four times the applied force for all components
- Apply a uniformly distributed load on each side of the cubic rock specimen
- Test different sized cubic rock specimens of 50 mm, 60 mm, 70 mm and 80 mm
- Have the ability to apply maximum stresses of at least 100 MPa and up to 200 MPa on 50 to 80 mm cubic specimens
 - 50 mm specimens require 500 kN to achieve 200 MPa
 - 80 mm specimens require 640 kN to achieve 100 MPa.

3 PRELIMINARY DESIGN

The new true triaxial cell has been developed from a design published by King *et al.* (1997), shown in Figure 1.

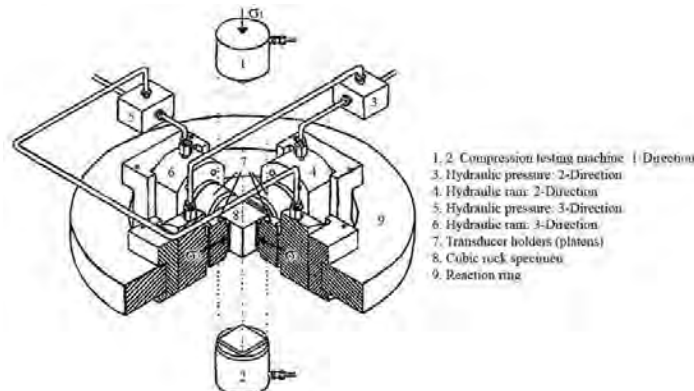


Figure 1: Isometric sketch of loading system (from King *et al.*, 1997)

The dimensions were scaled using a specimen size of 50 mm to provide a general idea of the size of each component.

4 SOURCING HYDRAULIC JACKS

The design of the true triaxial cell was based around the selected hydraulic jacks as they were required to provide sufficient lateral force outlined in the objectives.

The initial design parameters of the four hydraulic jacks were:

- Equal pressure rating for all hydraulic jacks, in order to achieve no confusion between hydraulic lines
- Equal force for all hydraulic jacks, in order to achieve a complete range of stresses

The ENERPAC RSM-750 hydraulic rams were selected for the design due to their capacity of 718 kN, their compact flat design and their mounting holes, which allowed for fixture to the cell.

The hydraulic ram capacity of 718 kN produces a maximum stress of 287, 200, 147 and 112 MPa on a 50, 60, 70 and 80 mm cubic specimen respectively, which is similar to the design parameters.

5 CHALLENGES OF THE TRUE TRIAXIAL CELL

5.1 DESIGN CHALLENGES

5.1.1 SELECTION OF MATERIALS

The strength and size of the components were reliant on the material chosen. The materials selected were steel alloy 4140 (Callister, 2007) for the reaction ring and steel alloy 4340 for the other components. All components were able to be oil-quenched and tempered (at 315°C) to produce 1570 MPa yield strength for steel alloy 4140 and 1620 MPa yield strength for steel alloy 4340. To achieve a factor of safety of 4 the highest allowable stress value on the reaction ring was 392.5 MPa and the highest allowable stress value on each of the other components was 405 MPa. Yield strength was used as the governing strength parameter, to prevent permanent deformation of the cell.

5.1.2 TRANSFERRING THE FORCE FROM A CIRCULAR PISTON TO A SQUARE SPECIMEN

The hydraulic jack needs to transfer the force from a circular piston (or saddle) to a square specimen. It was decided to split up the single platen shape into two components: the platen base unit and the platens. The platen base unit provided an attachment for the platens. This arrangement allows the platens to be changed with relative ease when the specimen size is altered.

5.1.3 ATTACHMENT OF THE PLATENS TO THE HYDRAULIC SADDLE

The platen base unit was required to be semi-permanently attached to the saddle. An attachment strategy based on magnetism was chosen. Magnetism provided a quick attachment method, and allowed for re-alignment to compensate for rotation of the hydraulic pistons.

5.1.4 PLATEN DESIGN (FOULING)

If the platens were the same size as the rock specimens (i.e. the platens cover the entire surface area of the rock), fouling of the platens would occur during rock deformation. Therefore the platens were bevelled to allow for deformation of the specimen. The maximum failure strain for rocks in uniaxial compression is approximately 0.45% strain. A factor of safety over 4 was applied to the bevel of the platens, which gave 2% strain to allow for unknown strains created in the 3D stress regime (Figure 2). Therefore the chamfers were 0.5, 0.6, 0.7 and 0.8 mm for the 50, 60, 70 and 80 mm platens, respectively.

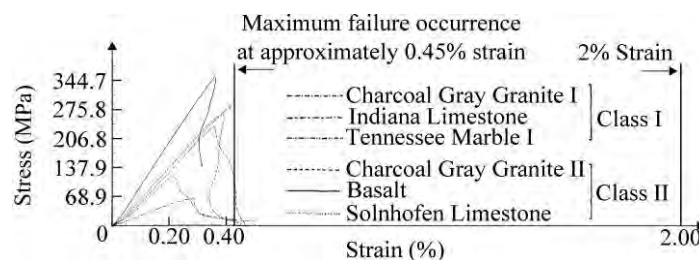


Figure 2: Stress-strain curves for six rock types in uniaxial compression with 2% strain identified (adapted from Wawersik and Fairhurst, 1969)

5.1.5 ATTACHMENT OF THE HYDRAULIC JACK TO THE REACTION RING

The hydraulic jack could not be directly attached to the reaction ring, because it is believed that if the structure of the reaction ring was compromised (i.e. bolt holes were created) the safety of the cell would be greatly diminished.

To avoid any problems associated with changing the structure of the reaction ring, an intermediate component was created, named the jack base unit (Figure 3). The jack base units provide two bolt holes corresponding to the mounting holes in the hydraulic jacks. The bolt holes were slightly oversized to allow for manual adjustment to centralise the jacks. The jack base units have two flanges (top and bottom) that wrap around the reaction ring. The top flanges have three threaded bolt holes to firmly attach the jack base units to the reaction ring.

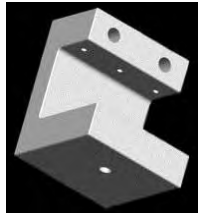


Figure 3: Jack base unit

5.1.6 DESIGNING A METHOD OF TESTING MULTIPLE SIZED ROCK SPECIMENS

The platen base unit was designed to allow the various platen sizes with 50, 60, 70 and 80 mm square faces to slot into the one fitting. The 65 mm square platen ends fit into the 65 mm square recesses in the platen base units. The 50 and 60 mm platens taper inwards and the 70 and 80 mm platens taper outwards (Figure 4).

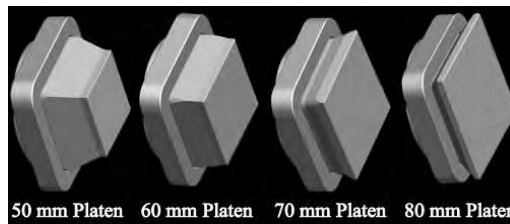


Figure 4: The four platens slotted into the platen base unit

The lengths of the platens were reduced incrementally by 5 mm from the 50 mm platen to the 80 mm platen, to ensure the gap between the specimen and the platen would remain the same. The changes in lengths of the platens were critical because it meant that the same machine could be used and the hydraulic pistons would be in the same range. The true triaxial cell was designed to have the pistons extend half of their range (8 mm) until the pistons touch the specimen, which allows for dilation of the specimen during a test.

5.1.7 FITTING ALL COMPONENTS INSIDE THE MATERIAL SIZE LIMITS WITH A FACTOR OF SAFETY OF OVER 4

The width of the reaction ring was limited by the components (Figure 5) that fit inside it. Each component inside the reaction ring was designed first to have a factor of safety of over 4 under single axis loading. The force was assumed to be uniformly distributed on each contact area. Finite element modelling was used to analyse the stress on each of the components. Trial and error was used to refine the design to ensure there was an optimal amount of steel being used. As the reaction ring is the most critical component to the safety of the cell, it was decided to increase the factor of safety above 5, i.e. 5.4.

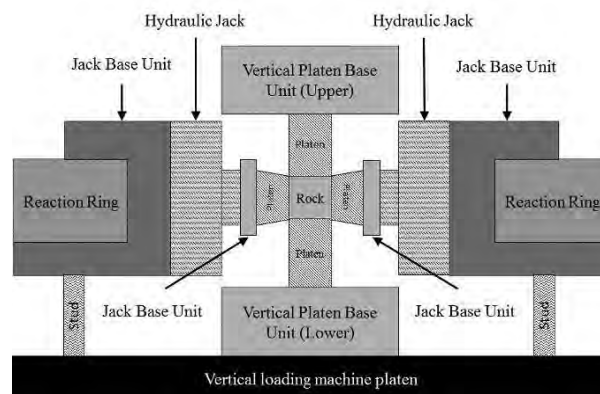


Figure 5: Diagram of cross-section view of design (not to scale)

5.1.8 CLEARANCES

The clearance symbol used for all components was H7/h6 (Madsen, 2004). This clearance fit provided a close fit for the location of stationary parts, and allowed components to be easily assembled and disassembled.

5.2 MANUFACTURING CHALLENGES

5.2.1 PLATEN BASE UNIT

The platen base unit was rounded with a radius of 4.9 mm and the platens were rounded with a radius of 5.15 mm, ensuring the edges had the same clearance fit (H7/h6).

Two versions of the platen base unit were designed (Figure 6). The final design was rounded squares on both sides, with a circular extrusion to attach the platen base unit to the piston. The design was approved by an independent consultant.

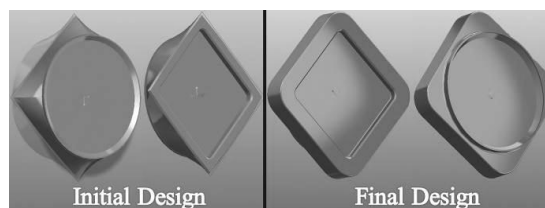


Figure 6: Two platen base units

5.3 IMPLEMENTATION CHALLENGES

5.3.1 ALIGNMENT OF COMPONENTS

The components require precise alignment in order for a rock specimen to be tested. To ensure centralisation of the specimen, measurements were marked on the platform of the vertical loading machine to position the lower components. The lateral alignment was achieved by the use of one of the four aluminium cubes manufactured. The 50, 60, 70 and 80 mm aluminium cubes were manufactured with the same bevel on the corresponding platens to allow the hydraulic jacks to be aligned. A digital level was purchased for aligning the platen base units. The digital level has a resolution of 0.1°, which means the 80 mm platen can be adjusted within +/- 0.07mm and the 50 mm platen can be adjusted within +/- 0.04 mm.

5.3.2 MAGNETISING THE PLATEN BASE UNIT

Magnetism was used to attach the platen base unit to the hydraulic piston (or saddle). This was done by attaching four rare earth magnets on the corners of the platen base unit, which induced magnetism in the component and firmly attached the unit to the saddle.

6 MANUFACTURE

The fabrication of the true triaxial cell was outsourced to LRD Engineering, South Australia. The reaction ring was the largest component that was manufactured (Figure 7).



Figure 7a: The removal of the centre of the original blank slab



Figure 7b: Grinding outside of reaction ring



Figure 7c: Finishing the inner surface of the reaction ring

Once all the components were manufactured they were zinc plated. The true triaxial cell was assembled once delivered (Figure 8).

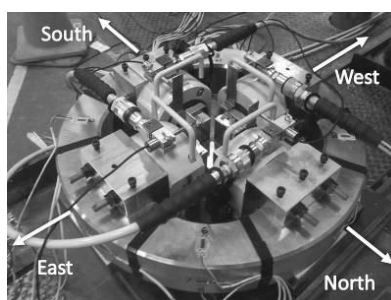


Figure 8: Assembled true triaxial cell

7 DESIGN AND ASSEMBLY OF HYDRAULICS

The lateral force is applied by two hydraulic circuits, each pressurised by a hand pump and each loading one set of opposing jacks (Figure 9).

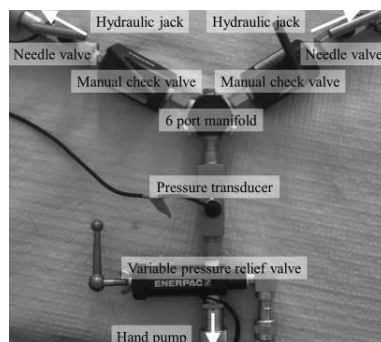


Figure 9: Hydraulic circuit for one axis

The total cost of the true triaxial cell and hydraulics was A\$30,000 excluding the linear variable differential transducers (LVDTs) and hours spent on design and fabrication. The design process took a total of approximately 1200 hours.

8 TRUE TRIAXIAL TESTING

8.1 SPECIMEN ROCK TYPE

Kanmantoo Bluestone exhibits all the desired characteristics needed for true triaxial testing; homogeneity, isotropy, fine granular structure and uniaxial compressive strengths (UCS) below 250 MPa. Preliminary tests were carried out to determine the rock strength parameters of the Kanmantoo Bluestone (Table 1).

Table 1: Rock strength parameters

Hoek-Brown rock strength parameters				Mohr-Coulomb rock strength parameters	
UCS (σ_c)	m_b	s	a	c	ϕ
184.2 MPa	15.7	1.0	0.5	39.4 MPa	46.3°

8.2 TEST RESULTS

Two true triaxial tests were completed on 60 mm cubic rock specimens (Figure 10-12). The rock specimens of the first and second tests failed at $\sigma_1 = 293.4$ MPa and $\sigma_1 = 418.9$ MPa respectively (Table 2a and 3b).

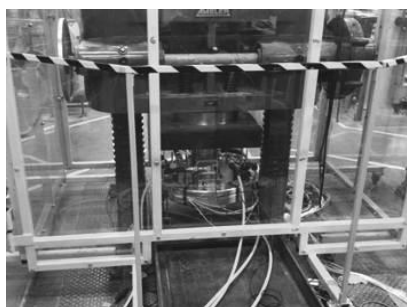


Figure 10: True triaxial test of 60 mm cubic rock specimen

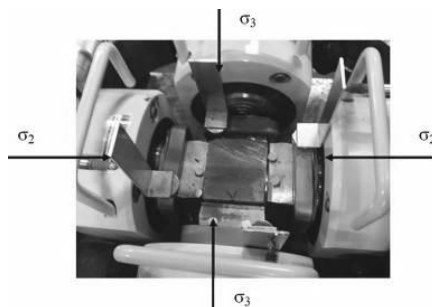


Figure 11: Failure of specimen 1, demonstrating the location of failure planes parallel to σ_2

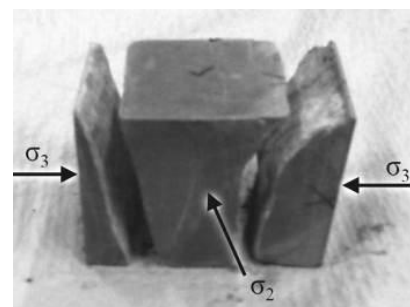


Figure 12: Failure of specimen 2, demonstrating the location of failure planes parallel to σ_2

Table 2a: Results for true triaxial test 1

Test 1	kN	MPa
σ_1 at failure	1096.9	293.4
σ_2	41.1	11.1
σ_3	0	0

Table 3b: Results for true triaxial test 2

Test 2	kN	MPa
σ_1 at failure	1530.8	418.9
σ_2	111.1	30.9
σ_3	39.0	10.8

Both rock specimens displayed failure planes parallel to σ_2 (Figure 11 and 12). This mode of failure was expected as theoretically, rock dilates more towards the lowest stress.

Vertical stress and strain of the specimen were also measured during testing. Both tests follow the shape of a typical stress vs. strain curve of rock, and indicate brittle failure of the specimen (Figure 13a and 13b). Cracking of the rock was heard prior to sudden failure.

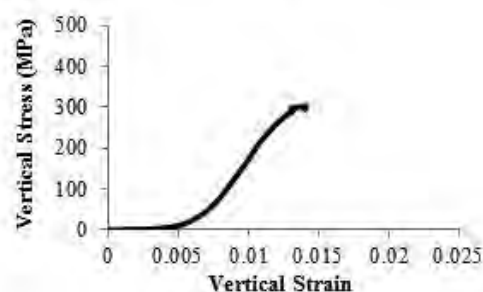


Figure 13a: Graph of stress vs. strain for Test 1, with $\sigma_2 = 10$ MPa and $\sigma_3 = 0$ MPa

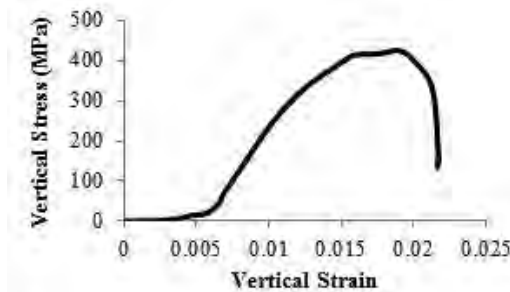


Figure 13b: Graph of stress vs. strain for Test 2, with $\sigma_2 = 30$ MPa and $\sigma_3 = 10$ MPa

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CONSTRUCTION OF RAIL EMBANKMENTS ON PEAT, WELLINGTON, NEW ZEALAND

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ABSTRACT

Tonkin & Taylor Ltd (T&T) has been engaged by KiwiRail Network (KiwiRail) to provide investigation, detailed design, consent applications and construction supervision for remedial works to stabilise an existing 8m high rail embankment. The rail embankment supports a busy urban commuter line located near Wellington, New Zealand. This embankment was identified to be at high risk in a slope hazard rating study undertaken by KiwiRail.

Subsurface site investigations by T&T revealed an 8m thick layer of peat deposits beneath the existing rail embankment and adjacent access track. Modelling has been completed to assess the potential failure mechanisms and consolidation settlement of a proposed new stabilising fill buttress. The rotation and settlement of the existing rail embankment and operating line has also been considered. Analysis indicated that the fill had to be constructed in a staged manner to maintain acceptable stability.

A settlement monitoring system has been established to record settlements during an initial stage of filling. Initial modelling and sensitivity analyses have been revisited to calibrate the model. This paper discusses the initial parameter selection and refinement with calibration against measured settlements. The refined model and parameters have been subsequently used to analyse the effect of additional stages of fill placement.

1. INTRODUCTION

In September 2010, following an extended period of wet weather, a landslide occurred on the North Island Main Trunk rail line approximately 26 Km north of Wellington, New Zealand. This landslide caused the derailment of one train and the subsequent collision with another oncoming train. Both trains in the collision were passenger services. Fortunately there were no injuries, however the incident caused significant disruption to KiwiRail services.

Earthworks to stabilise the landslide scarp provided a source of fill to construct a stability buttress at an adjacent high risk section of embankment. This high risk embankment section is the subject of this paper and is located 450m south of the landslide, refer Figure 1. Boreholes and test pits undertaken by T&T identified the presence of approximately 8m of peat underlying the existing rail embankment and proposed new fill buttress.



Figure 1: Existing high risk rail embankment

2. PROJECT CONSTRAINTS

The design and construction of fill embankments on soft soils is complex. Placing loads onto soft saturated soils has the potential to cause large settlements and the generation of excess pore water pressures. Added to the complexity of this project is that the embankment is within the rail environment and supports a busy urban commuter line.

Commercially it is critical for KiwiRail that rail services are maintained with minimal disruption at all times. For this double tracked section of rail typically the maximum time equipment or plant can foul (prevent movements on) the track ranges from 20 to 40 minutes. This severely limited the access to areas of the site, including for site investigations.

KiwiRail occasionally schedule “blocks of line” which are times when passenger services do not run and the overhead power lines are turned off along a section of the rail to enable maintenance work to be undertaken. A constraint on this project was that the construction (including movement of all fill from the opposite side of the line) had to be undertaken during a ten day block of line over the 2011/2012 New Year period. The site investigations, design, consenting and tendering all had to be undertaken within a time period of less than six months to enable the use of this rare block of line.

There are a number of health and safety issues that are associated with working within the rail environment. These put constraints on the site investigations as well as the construction methodology and programme.

3. GEOLOGICAL SETTING

T&T undertook geological mapping and subsurface ground investigations in the project area which included 2 boreholes, 11 test pits and 7 scala penetrometer tests. The project area is located on the side of a peat swamp formed by the natural infilling of a valley. The railway is built on a cut and fill platform that sidles the slope at the edge of the peat swamp. The natural slopes comprise greywacke, sandstone and mudstone. These rocks are evident in the cut slopes that have been excavated during the rail construction.

Where the embankment truncates side gullies, joining the main swamp, the embankment overlies soft saturated alluvial silts and peat. The alluvium and peat layers are interfingering. At the project site the soft swamp and peat deposits are up to 8m thick. The peat is predominately fibrous.

4. PLAXIS, BACKGROUND AND MATERIAL MODELS

PLAXIS is a finite element software package that has been developed specifically for the analysis of deformation and stability in geotechnical engineering projects. PLAXIS software allows the new fill buttress to be modelled with a staged construction sequence. The software also allows calculation of excess pore water pressures, consolidation times between construction phases, stability, and deformations within the same model.

An elastic – plastic Mohr-Coulomb model was used for all of the soil materials except for the peat where largest consolidation settlements were expected. The Mohr-Coulomb model contains six significant soil parameters:

- E = Young's modulus
- ν = Poisson's ratio
- c' = Cohesion
- ϕ = Friction angle
- γ = Density
- k = Permeability coefficient

The peat layer was modelled using the PLAXIS advanced soil model Soft Soil-Creep. Soft-Soil Creep is considered to be a more accurate model for normally-consolidated, highly compressible, soft soils. The model simulates the increase in stiffness of the peat with the increase in pressure as the fill buttress is constructed. This differs from the Mohr-Coulomb model that assumes a constant average stiffness. The Soft Soil Creep model requires the following significant input parameters:

- c' = Cohesion
- ϕ = Friction angle
- γ = Density
- k = Permeability coefficient

The following additional significant stiffness parameters are also required:

- κ^* = Modified Swelling Index
- λ^* = Modified Compression Index

λ^* and κ^* can be obtained by fitting a stress-strain relationship to a one dimensional consolidation laboratory test for a particular material.

5. SOIL PARAMETERS

5.1 SOFT SOIL CREEP

One push tube sample of the peat was obtained from borehole investigations. Attempts were made to obtain additional samples from greater depths, however the peat could not be recovered within the push tube. A one dimensional consolidation (oedometer) laboratory test was conducted on the peat sample recovered at 1.7m beneath the proposed embankment.

Given that only one sample of peat from the site had been tested, and that this sample was not considered to be representative of the peat present, it was decided that the peat input parameters should be conservative and based on more conservative testing data from other sites in the area.

Figure 2 shows the stress – strain curve for various sample oedometer tests, and the proposed soft soil creep model adopted in the initial PLAXIS model.

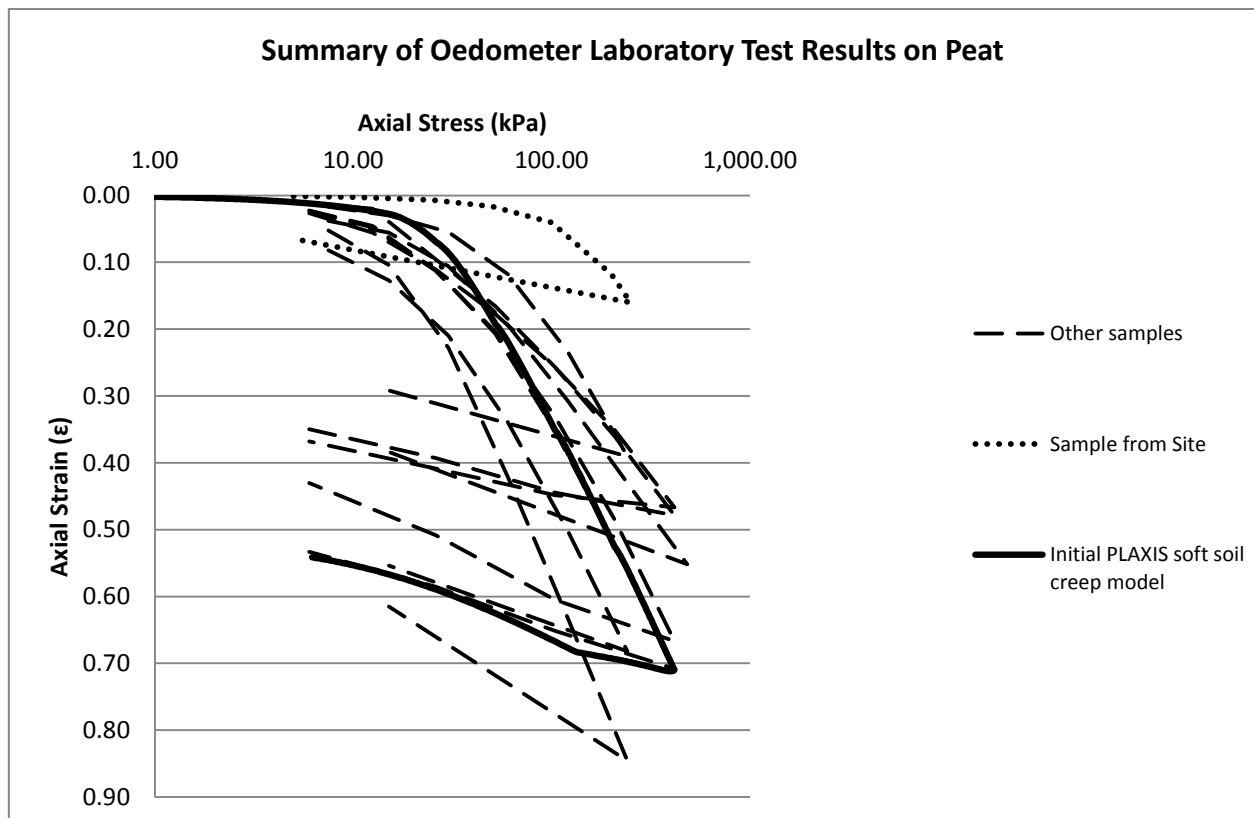


Figure 2: Summary of oedometer laboratory test results on peat

5.2 MOHR COLOUMB

Specific laboratory testing was not undertaken on any of the other soil materials identified on the embankment site. These materials (greywacke rock, colluvium, alluvium, and fill) are reasonably well known and consistent regionally. Back analysis was completed on the existing profile to provide minimum bounds for all of the soil input parameters. The soil strength parameters assigned to these materials are consistent with previous T&T experience in the area.

6. MODEL DEVELOPMENT

Geological cross sections through the existing rail embankment and adjacent access track were developed. A critical cross section was selected for further analysis and design. The critical cross section was through a gully where site investigations had shown the thickest layer of peat was present. A simplified ground model was assumed for input into PLAXIS as shown in Figure 3.

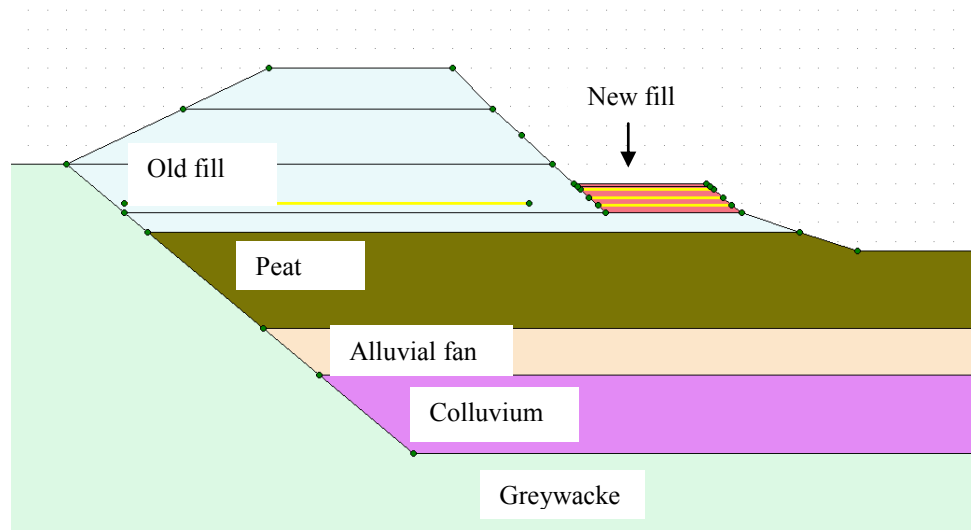


Figure 3: PLAXIS ground model

The model was constructed using nodes and lines and a mesh was created from 15-noded triangular elements. Boundary conditions were defined using standard fixities. This is full fixity along the base (no displacement in the horizontal and vertical directions) and horizontal fixity along the sides allowing vertical movement.

The initial pore pressures in the model were generated based on a phreatic surface. Ground water levels were based on groundwater encountered in test pits and boreholes and the level of the surface water evident in the swamp.

7. INITIAL MODELLING

The first calculation phase was a back analysis of the existing rail embankment to provide minimum bounds for the soil parameters as mentioned above.

The staged construction calculation steps were used in the model. A 10 day time interval was specified because the construction period was limited to the 10 day block of line. A 4m high fill buttress was required to provide a sufficient increase in the FOS of the rail embankment to a satisfactory level. Initial modelling indicated that the 4m high fill buttress could not be constructed in a single lift within the available 10 day construction period. The development and slow dissipation of excess pore water pressures was likely to induce slope instability within the peat foundation. While the long term FOS was acceptable, the FOS immediately after construction (prior to consolidation and the associated strength gain) was 1.0 which suggests the fill buttress and outside edge of the existing embankment is close to equilibrium.

Analysis of a fill height of 1.5m indicated that the FOS of the existing embankment was maintained during filling and improved (marginally) after fill placement. This staged approach to filling manages stability risk and also enables the settlement monitoring results to provide an initial feedback loop to the analysis and modelling for refining settlement and stability predictions for future filling stages. The next stage of filling is constrained by weather and budget but is programmed to take place towards the end of 2012.

8. SETTLEMENT MONITORING

A profilometer was installed along a 90m length of the existing access track prior to filling. This embankment section was identified as being at the highest risk of settlement due to the depth of peat at this location. Profilometers measure vertical displacements in relation to a base reading which is undertaken prior to construction. The profilometer used for this project had an accuracy of approximately +/- 10mm.

The critical cross section modelled in PLAXIS was within the 90m length monitored by the profilometer which enabled a direct comparison between the settlements predicted by PLAXIS and the actual settlements recorded. Had the settlements recorded during the first construction period been larger than acceptable or excess pore water pressure

dissipation slower than anticipated, filling would have been stopped immediately. Trigger levels were set out in an emergency action plan and if reached the Site Engineer would have stopped the filling operations and notified KiwiRail. Rail speeds through the area would also be immediately lowered.

Profilometer readings were taken at regular intervals during the 10 day construction period and the following months to monitor settlements as the excess pore pressures beneath the embankment dissipated. Settlement monitoring is currently still ongoing at the time of writing this paper.

9. COMPARISON OF PREDICTED AND ACTUAL SETTLEMENT FOR STAGE 1 FILLING

Figure 4 shows a plot of the initial settlement predictions using PLAXIS and the actual settlements recorded by the profilometer.

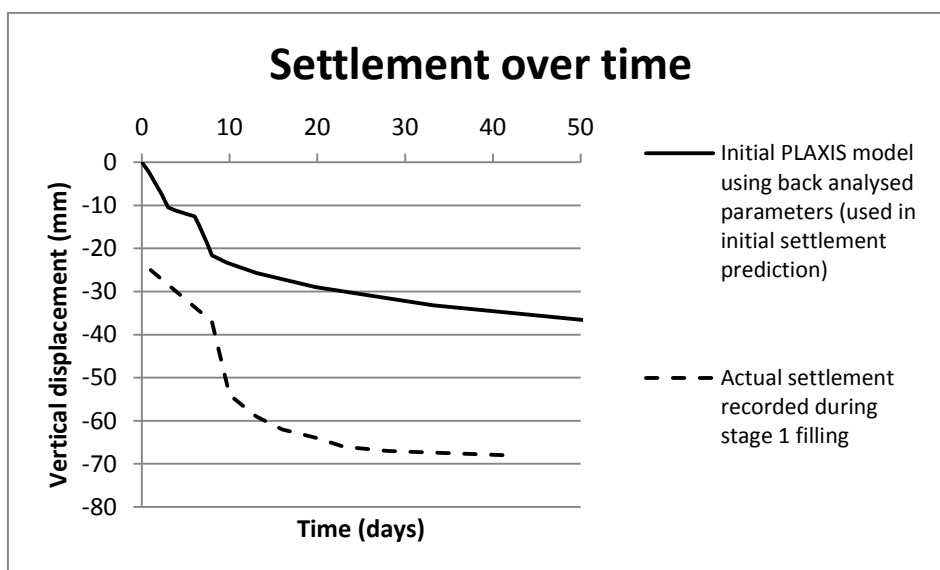


Figure 4: Plot of predicted and actual settlements over time

The settlements predicted in the initial PLAXIS model were less than the actual settlements recorded during and immediately after the first stage of filling. The rate of settlement was also significantly faster than predicted.

10. MONITORING AND CALIBRATION OF THE MODEL

There are a number of uncertainties within the model which may have caused PLAXIS to initially under predict the magnitude and rate of settlement during the stage 1 filling. Calibration of the model after the first construction phase was planned so that the model could be refined and more accurately predict settlements during future, more critical construction phases.

The first step in the calibration of the model was to simulate the construction sequence that had actually occurred. The original model assumed the 1.5m layer of fill was placed evenly over a ten day period. In reality the fill was placed over an eight day period with a three day period in the middle where no fill was placed due to poor weather.

It was noted that the long term (over one year) settlements predicted in PLAXIS were similar to the actual settlements recorded following approximately one month of consolidation. This indicates that the permeability of the peat is likely to be greater than initially assumed. Based on the geology it is likely that the peat is interfingered with alluvial layers of higher permeability. These layers reduce the drainage path distances and therefore allow excess pore pressures to dissipate more rapidly. Settlement therefore occurs at a faster rate.

A sensitivity check was undertaken on the settlement predicted in PLAXIS using soft soil creep input parameters based on the oedometer results from the peat sample obtained from the site. The settlement predicted is shown in Figure 5.

To fit the settlements predicted in PLAXIS with the actual settlements recorded the soft soil creep permeability parameters were increased and the stiffness parameters adjusted slightly. The settlement predicted by the calibrated model is also shown in Figure 5.

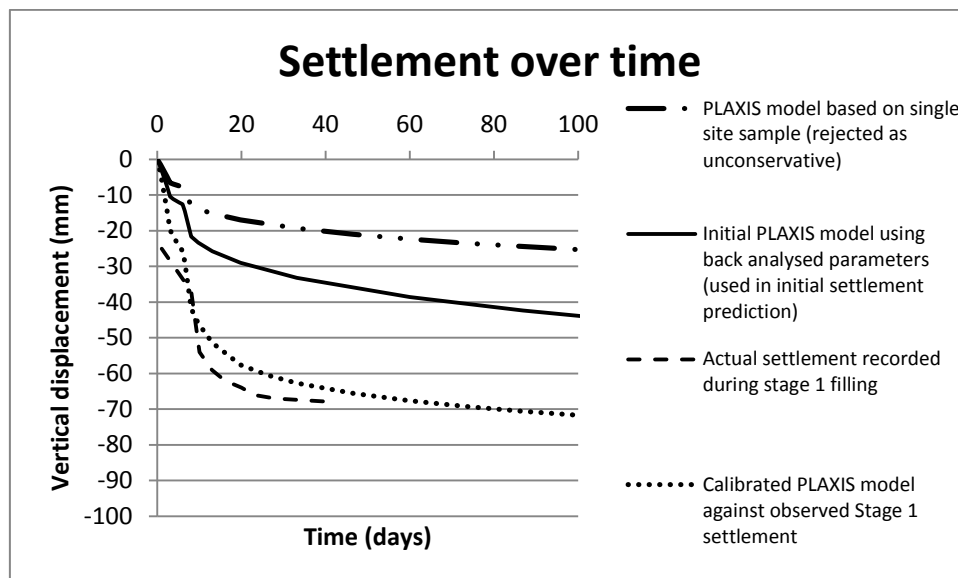


Figure 5: Plot of settlement over time of the PLAXIS model and observed settlement

The profilometer was proven to be a very effective settlement monitoring device and was able to be installed and monitored for a reasonable cost. Groundwater monitoring would also have been very beneficial as it would have helped to confirm the suitability of the permeability parameters of the peat and the degree of excess pore water pressure rise and amount of dissipation with time. These elements are difficult to correlate with settlement monitoring alone. During the site investigations, a piezometer was installed with the intent of monitoring the groundwater pressures during the construction period. Unfortunately the piezometer was destroyed during the enabling works by the Contractor. This highlights the need to install redundancy of critical instrumentation and ensure that Contractors are aware of and understand the importance of all monitoring devices on the site.

11. FUTURE SETTLEMENT PREDICTIONS

The calibrated PLAXIS model has been used to predict the settlement that will occur during the second stage of filling. Based on a 365 day consolidation period between the first and second fill stages the total maximum settlement after Stage 2 filling is expected to be approximately 200mm after the pore pressures have dissipated by 90%.

Settlements will be monitored during the second stage of filling and the model re-calibrated if necessary prior to the third and final stage.

12. CONCLUSIONS

PLAXIS is a useful tool however it is important that the ground model and input parameters are well understood and calibrated against early monitoring results. The soft soil creep model provides a good method of predicting settlement in soft soils. The use of the soft soil creep model in this project allowed a correlation to be made between the predicted and observed settlement to a greater level of accuracy than a Mohr Coulomb model would have allowed.

This case study also demonstrates the importance of understanding the priorities and overall aims of the client. T&T, working with KiwiRail, were able to come up with a solution that fitted within the operational requirements and optimized both time and cost. Stage 1 settlement was useful to refine the ground and PLAXIS model in advance of later, more critical filling stages. Overall time and cost savings were able to be made using this approach.

13. ACKNOWLEDGEMENTS

The author wishes to thank KiwiRail and the Geotechnical team at T&T Wellington for their assistance with this project and paper.

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FULL SCALE TESTING OF HELICAL PILES IN RESIDUAL MERCIA MUDSTONE

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ABSTRACT

As part of the M4 Motorway Variable Speed Limit Upgrade near Cardiff, UK, helical piles were utilised as the foundation option for the majority of the 32 variable message signs (VMS) and five new superspan gantries that were constructed. Using normalised static load test data obtained from helical piles, installed into residual Mercia Mudstone along the M4 motorway route, an empirical relationship establishes that tensile capacity is approximately 40% less than compressive capacity for helical piles installed under these specific geological conditions. The likely reasons for the variation in capacity, such as disturbance of the soil during installation and the effect of increasing clay stiffness with depth within the soil profile are explored. Furthermore, the often quoted relationship between installation torque and the predicted pile capacity (in both compression and tension) is analysed using a novel normalisation technique which uniquely qualifies this relationship within the residual Mercia Mudstone material. The static load test data obtained from the M4 motorway project is subsequently compared with static load test data from other projects across the UK where helical piles have been installed into fine grained strata which shows that a relationship between installation torque and the predicted pile capacity is likely to exist.

1. INTRODUCTION

In recent times, helical piles have been utilised throughout the UK as a quick and efficient foundation option for signs and gantries across the highway and railway networks. As part of the M4 Motorway Variable Speed Limit Upgrade near Cardiff, UK, helical piles founded within residual Mercia Mudstone were utilised to support numerous cantilevered signs and gantries. Although empirical relationships show that higher installation torques give higher ultimate pile capacities, piles which were not installed to their design depth (due to high installation torques) were tested under static loading conditions to ensure that the piles satisfied the displacement criteria. This paper provides a brief background to helical piles, their use on the M4 Motorway project and an analysis of the static load pile testing undertaken. Three static load pile test results will be utilised to analyse pile performance in terms of the original pile design, pile performance based on a bearing area normalisation and pile performance based on a normalisation for torque and pile displacement. This will quantify the expected bearing capacity variation under compressive loading compared to tensile loading. The load test results will also be compared to load test results undertaken at other sites with a similar soil profile from across the UK to explore the relationship between installation torque and the predicted pile capacity.

2. PROJECT BACKGROUND

2.1 GENERAL INFORMATION

As part of the M4 (Newport) VSL, Screwfast Foundations was engaged to undertake the detailed foundation design and foundation construction for 32 variable message signs (VMS) and five new superspan gantries. The Welsh Assembly Government commissioned the “scheme which will make driving conditions safer and lead to more reliable journey times for motorway users” (Welsh Assembly Government, 2011). As a key aim of the upgrade was to reduce motorway traffic congestion, the disruptions involved with the construction of the upgraded motorway section was to be kept to a minimum, resulting in the decision to use helical piles as the foundation options for the majority of new structures to be constructed.

2.2 GEOLOGY OF TEST SITES

The motorway traversed considerably varied geology within the 12 km project boundary, ranging from deep cuts into Mercia Mudstone to deep Quaternary alluvial profiles associated with the River Usk.

The Mercia Mudstone Group is characterised by a sequence of brown, red-brown, calcareous clays and mudstones with occasional beds of impersistent green siltstone and fine grained sandstone. In its fully weathered state, it is a reddish-brown, ‘very soft’ to ‘hard’ silty clay but frequently contains less weathered mudrock clasts. The weathering profile is

generally progressive with strength and stiffness tending to increase with depth. The depth of weathering is typically between 10 to 15 m but may be considerable greater (Hobbs et al, 2002).

3. HELICAL PILE BACKGROUND AND DESIGN THEORY

Helical piles were invented and patented by Alexander Mitchell, an Irish engineer, in 1833 (Perko, 2009). The popularity of this foundation system grew rapidly in the 19th Century with helical piles being used in the construction of many Victorian era piers (e.g. Brighton Pier, Hastings Pier), breakwaters and lighthouses (Screwfast Foundation Ltd, 2005).

Modern design of helical piles with multiple helices is based on the two possible modes of failure – failure of the soil immediately surrounding an individual helix ('bearing failure') or failure of the entire soil column ('column failure'). Research has shown that the failure method is highly dependent on the spacing of the helices and the soil type (Perko, 2009). The 'bearing failure' design methodology, used as the design methodology for all piles on the M4 project, utilises the Meyerhof bearing capacity of a circular plate and assumes the elastic Boussinesq vertical stress distribution. From Boussinesq theory, it can be calculated that the change in vertical stress due to an applied load is approximately 8% of the applied stress at $2 \times D$ below the helix plate and approximately 4% of the applied stress at $3 \times D$ below the helix plate, where D is the individual helix plate diameter (Arup Geotechnics, 2005). As such, 'bearing failure' is likely to dominate in both compression and tension when a helix spacing of at least $2 \times D$ is maintained as the reduction in bearing capacity for any helix plate will be a maximum of 8% of the applied load. This has been investigated using photographic analysis of scale model pull out tests undertaken by Narasimha Rao et al. (Narasimha Rao, 1991).

Helical pile installation torque has been empirically related to the ultimate capacity of the pile by numerous authors (Perko, 2009). This is dependent on a large number of variables, not limited to soil type, soil strength, shaft diameter, shaft shape, helix diameter, number of helices and helix pitch. Ultimate pile capacity and/or minimum installation torque can be predicted using Equation 1.

$$Q_{u,pile} = T_{Install} * TF * FoS \quad (1)$$

Where: $T_{Install}$ = torque measured over final 1 m of pile installation (kNm)

TF = torque ratio (m^{-1}), approximately 10 m^{-1} has been suggested for use (Perko, 2009).

FoS = Factor of Safety (lumped), usually ranging from 2.5-3 in compression and 3-4 in tension.

4. PROJECT SPECIFIC PILE DESIGN AND SPECIFICATION

4.1 GENERAL PILE INFORMATION

Helical piles for the M4 project, designed in accordance with BS8004, varied in size and length on a site by site basis. Helical piles consist of multiple 2 m long circular hollow sections (CHS) members connected using a socket attachment with a 3 bolt connection. CHS wall thicknesses are chosen to prevent torsional failure of the section during installation and any requirement for sacrificial steel allowances. Bearing sections are constructed by welding 12 mm thick circular steel plates, known as helices, to a CHS member at 1 m centres. A summary of the pile configuration for each test site on the M4 project (identified as MS416, SS1A and SS5B respectively) can be found in Table 1 and Table 2.

Test Site ID	MS4 16	SS1A	SS5B
Overall Pile Length (m)	6	6.5	9
Helix 1 - Diameter (m)	0.45	0.45	0.45
- Depth (m BGL)	4.5	2	5.5
Helix 2 - Diameter (m)	0.35	0.45	0.45
- Depth (m BGL)	5.5	3	6.5
Helix 3 - Diameter (m)	-	0.45	0.45
- Depth (m BGL)	-	4	7.5
Helix 4 - Diameter (m)	-	0.35	0.35
- Depth (m BGL)	-	5	8.5
Helix 5 - Diameter (m)	-	0.35	-
- Depth (m BGL)	-	6	-

Table 1: As built test pile configuration

Test Site ID	MS4 16	SS1A	SS5B
Design SWL (kN)	90	190	250
CHS Diameter (m)	0.17	0.19	0.22
CHS Wall Thickness (m)	0.008	0.01	0.008

Table 2: Test pile specification

One borehole was undertaken at each of the test sites. Although the sites were more than 1 km apart, the soil profile was generally consistent, with made ground encountered over the upper 2.5 m and residual Mercia Mudstone encountered to approximately 12 m below ground level. SPT 'N' values ranged from 10 to 18 in the made ground and ranged from 18 to 35 in the residual Mercia Mudstone. The SPT 'N' values were shown to increase with depth in all boreholes.

4.2 STRUCTURE TYPES AND LOADING

Helical pile groups consisted of at least four piles connected to a rectangular or cruciform shaped steel grillage. Under critical loading conditions (impact and wind loads), moments applied at the top of the grillage were resolved into a push-pull pivot system requiring opposing individual piles within the pile group to restrain compression and tension loads. Calculated safe working loads (SWL) for individual test piles can be found in Table 2.

5. PILE TESTING

5.1 PILE TESTING REASONING AND METHODOLOGY

Pile testing was undertaken to verify the pile performance at locations where piles could not be driven to the design depth due to installation torque exceeding the calculated structural torsional deformation limits. Pile deflections were not expected to exceed the SWL deflection limits as larger than expected installation torque are likely to result in an increase in the ultimate pile bearing capacity as per Equation 1.

The reasons for higher than expected torque readings during installation are mainly due to harder than expected ground conditions (due to poor site investigations or variations in geology over short distances), differing ground conditions to that shown in the site investigation (e.g. material type or depth to rock) and lack of groundwater (especially if groundwater was encountered during the site investigation).

Working piles were tested using a modified version of the static load testing guidance provided by the Institution of Civil Engineers Specification for Piling and Embedded Retaining Walls (SPERW) 2nd Edition (ICE, 2007). The modified SPERW static load test procedure shortened the duration of each load increment to reduce health and safety risks to the field staff located on the motorway shoulder. Load was applied to the test pile via a hydraulic jack and two adjacent piles as reactions. Pile deflection was measured by four gauges located at the top of the test pile and one gauge located on each reaction pile.

5.2 PILE TESTING RESULTS

A compressive static load pile test was undertaken at site MS4 16 and tensile static load pile tests were carried out at sites SS1A and SS5B. The pile test results can be seen raw in Figure 1 and as a function of the pile SWL (i.e. load as a percentage of the design SWL) in Figure 2.

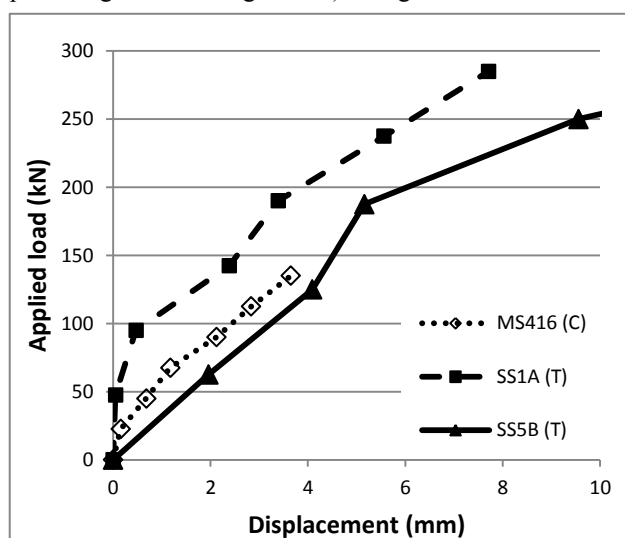


Figure 1: Raw pile test results

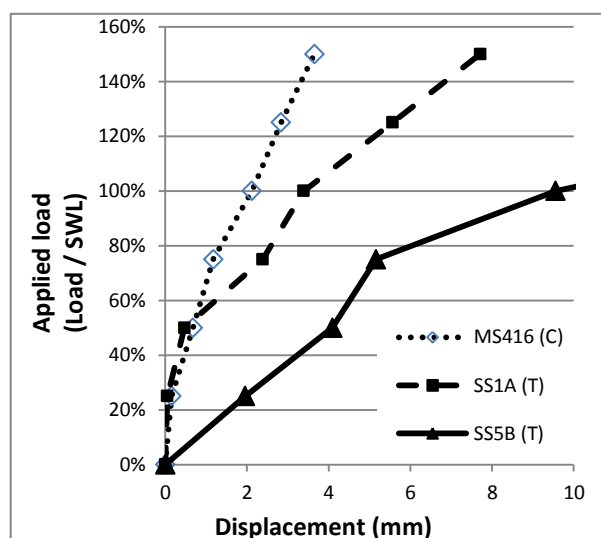


Figure 2: Pile test results in terms of design pile capacity

Figure 1 shows that all test piles had a relatively consistent response to loading. Figure 2 shows that the deflection of all test piles was less than 10 mm at 100% of the SWL and that the pile tested in compression performed better than the piles tested in tension in terms of the design SWL..

This is to be expected for the following main reasons:

1. Given that the typical residual Mercia Mudstone profile shows an increase in strength with depth, compressive load is likely to be transferred to soil with greater strength, especially below the base of the lowest helix. Under tensile loading, load will be concentrated between the lowest helix and the ground surface which will generally have reduced strength compared to the deeper soil layers.
2. Disturbance of the soil during installation of the piles alters the in situ strength characteristics of the soil towards the residual strength. During a 'perfect' pile installation, the path taken by the first helix is followed by subsequent helices causing little soil disturbance. During installations where piles are not able to be driven to depth, this may not be the case and some soil disturbance may be caused resulting in a lower ultimate bearing capacity. As the soil below the helices will not be disturbed, soil disturbance has a greater detrimental effect on the pile tensile capacity.
3. Where piles have not been installed to the design depth and the upper helix is less than about 4 helix diameters below ground level, the tensile pressure bulb of the upper helix (i.e. the helix closest to the ground surface) may extend to the ground surface. This will reduce the tensile capacity of the pile.

5.3 PILE PERFORMANCE NORMALISATION

In order to relate the observed pile performance and observed installation torque to the torque factor and the variation in performance due to axial loading direction, a number of unique normalisations have been performed. Although the small number of piles tested on the M4 project precludes reliable conclusions to be drawn from this data alone, Section 6 provides a brief comparison of the M4 static load pile test results with data from similar projects across the UK.

5.3.1 NORMALISATION FOR BEARING PRESSURE

The normalisation for bearing pressure accounts for the difference in total helix bearing area between test piles and allows for a more realistic comparison of pile performance under compression and tension loading. The bearing pressure is calculated by dividing the load by the total pile bearing area (the summation of the individual helix bearing areas) which assumes that the pressure applied at the face of each helix plate is equal. Whilst this is clearly a very simplistic assumption, the factors listed in Section 5.2 preclude an accurate estimate of the load distribution between helix plates.

Figure 3 shows that the pile tested in compression performed better than those tested in tension. On average, Figure 3 shows that the average helix pressure in tension is 40% less than the average helix pressure in compression per mm of displacement. The increased displacement under tensile loading can be attributed to the factors listed in Section 5.2.

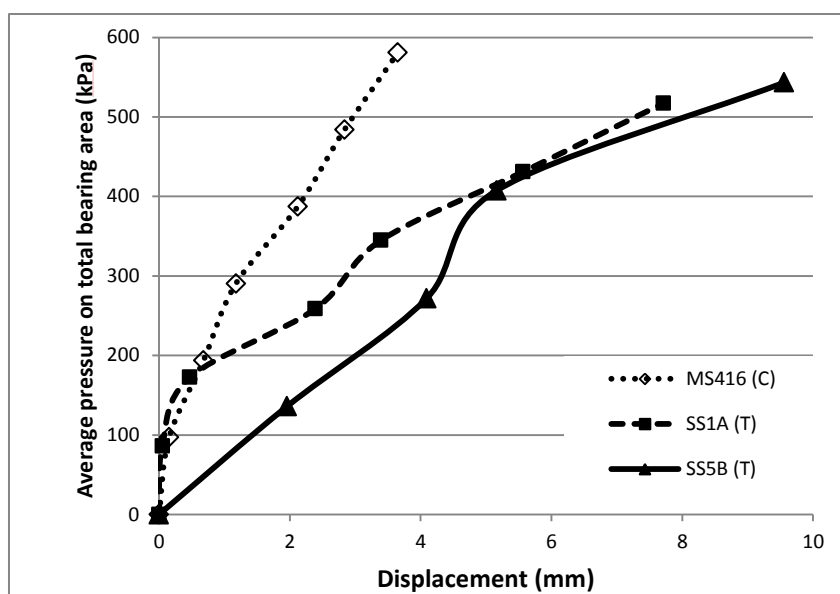


Figure 3: Bearing pressure vs displacement

5.3.2 NORMALISATION FOR TORQUE AND PILE PERFORMANCE

The normalisation for torque and pile performance allows for a comparison of the measured installation torque and the observed pile displacement whilst taking into account the bearing area of the pile. This normalisation does not assume an equal bearing pressure across the helices as only actual applied loads are taken into account.

Initially, pile performance is analysed as a function of applied load and displacement normalised by the bearing area (Figure 4). The results are noticeably different to analyses in previous sections (e.g. Figure 2) where SS1A (tensile test) is shown to perform better than the other test sites. As annotated on Figure 4, the installation torque required to install SS1A to the final depth was considerably greater than for other sites. By taking the gradient of the points plotted in Figure 4 (therefore giving a unit kN/mm/m^2) and plotting this against the measured installation torque, it can be seen that greater installation torques are generally shown to result in better pile performance. Figure 5 thus shows the relationship between installation torque and normalised pile performance.

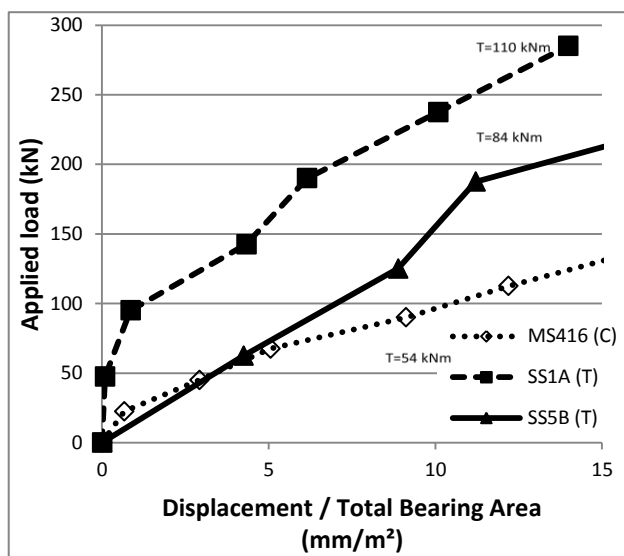


Figure 4: Load vs normalised displacement

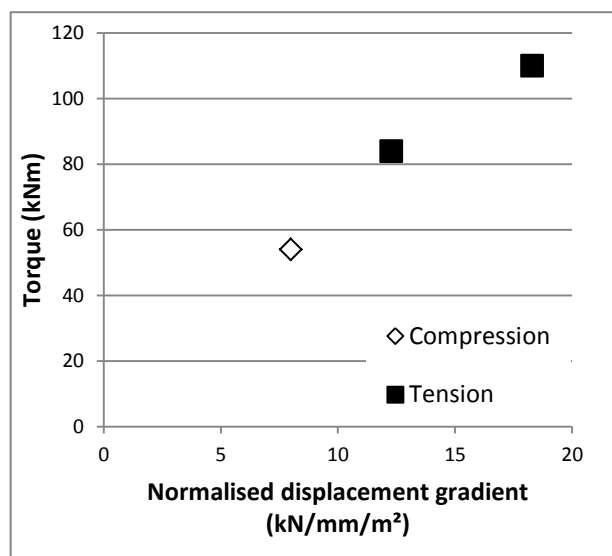


Figure 5: Torque vs normalised pile performance

6. COMPARISON WITH OTHER PILE TEST RESULTS

Verification of pile test results from the M4 Newport project can be undertaken through a comparison with results of pile tests in similar soil profiles from projects across the UK. Projects used for this comparison include the A14 VSL upgrade and the M1 VSL upgrade. In addition, results from the M25 widening project test site have been included for comparative purposes where the lower helices were installed into coarse grained material.

Project ID	Tensile reduction factor
M4	0.4
M1	0.45
A14	0.45
M25	0.7

Table 3: Average tensile reduction factor for test sites across the UK

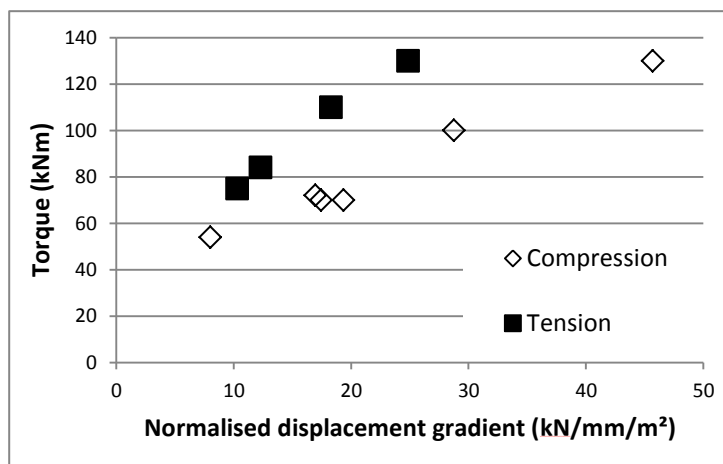


Figure 6: Installation torque vs normalised displacement gradient

The tensile reduction factor, defined as the average difference between the compressive pile capacity (measured in terms of average pressure vs displacement) and the tensile pile capacity, is shown in Table 3. These results show that at least a 40% reduction in helical pile capacity can be expected in tension compared to compression where piles are installed at sites with silty clay materials and increasing strength with depth profiles. This is not the case for locations where helices were installed into coarse grained material as the M25 results show that a 70% reduction in average helical pile tensile capacity was observed. Due to the small sample size, it is recommended that these tensile reduction factors be used as a guide only and not for design purposes.

Figure 6 shows that a linear relationship exists between installation torque and the normalised displacement gradients (NDM) in both compression and tension for the test sites analysed in Table 3. For all installation torques analysed, $NDM_{tension}$ is between 45% and 50% less than $NDM_{compression}$. Although no torques below 70 kNm were analysed for tensile static load pile tests, the gradients of the linear relationships indicate that the difference between $NDM_{compression}$ and $NDM_{tension}$ reduce as torque decreases implying that this relationship is not valid for installation torques below about 40 kNm. The results of this torque normalisation match well with those from the bearing area normalisation showing that capacity of a helical pile in silty clay material in tension is approximately 40% lower than in compression. This also confirms the assumptions stated for the FoS in Equation 1 where tensile FoS is usually taken as between 3 – 4 whereas compression FoS is usually taken as 2.5 – 3.

7. CONCLUSIONS

The M4 Newport Motorway Variable Speed Limit (VSL) Upgrade near Cardiff, UK successfully utilised helical piles to support numerous VSL structures in the residual Mercia Mudstone materials. Where piles could not be installed to their design depth, working pile tests undertaken at three sites on the M4 motorway showed that deflections at SWL loads were within tolerable limits. When normalised, piles tested in compression were shown to perform better than those in tension as expected due to the residual Mercia Mudstone soil profile and soil disturbance during pile installation. Pile performance based on a bearing area normalisation showed that tensile capacity was approximately 40% less than the compressive capacity. This was verified through a comparison of the M4 test with other pile tests in similar ground conditions across the UK. In addition, pile performance based on a normalisation for torque and pile displacement showed that greater installation torque led to greater pile capacity. Through a comparison with other pile installation from across the UK, tensile capacity was shown to be between 45% and 50% less than the compressive capacity when based on the installation torque alone. Although these relationships were shown to be relatively consistent for sites across the UK, they are valid for soil profiles similar to that of the residual Mercia Mudstone and should be used as a general guide during pile installation rather than a pile design tool.

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DESIGN AND ANALYSIS OF TEMPORARY STABILISATION FOR A RAIL UNDERBRIDGE ABUTMENT

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ABSTRACT

In order to improve traffic flow and vertical clearances below an existing rail underbridge in Bankstown NSW, upgrading of the bridge structure, roadway and associated stormwater culvert was required. The culvert upgrade included excavation in close proximity and below the level of the footing base for one of the existing rail bridge abutments. In order to support the bridge abutment, a temporary stabilisation system was required to retain the toe of the abutment and its foundation to allow the excavation and construction works to be completed during live track usage.

Due to geometrical constraints, the retention system had to be less than 0.5m wide (including temporary anchor heads), and be capable of being installed with less than 2.5m of available headroom.

Detailed field investigations of the existing abutment structure and subsurface conditions was completed followed by finite element analysis using Plaxis 2D to model each of the stages of the works, including bridge deck replacement, temporary anchoring of the abutment, contiguous pile wall installation, excavation, culvert construction, backfilling and anchor de-stressing. The analysis included sensitivity analysis for the soil material parameters and consideration of flooding whilst the works were underway.

A number of iterations of the construction staging were completed following review and feedback from the builders, structural designers and RailCorp.

Survey monitoring of the rail tracks was completed during retention system construction, excavation and backfilling and showed relatively good correlation with the modelling.

1. PROJECT BACKGROUND

The original bridge at this site comprised a steel transom deck structure supported on brick abutments and two intermediate piers. Due to the configuration of the existing bridge, only two traffic lanes of limited clearance were feasible below the original bridge.

To improve local traffic flows Bankstown Council required the bridge deck to be replaced with a single span structure, allowing for a total of four traffic lanes. In conjunction with the upgrading of the bridge structure and pavements, replacement of an existing open culvert with a new box culvert was also required to improve flood capacities.

The new bridge superstructure was to be of steel and concrete construction, supported on new concrete headstocks. Four new piers, located at the corners of the bridge were to support the new headstocks.

In the area adjacent to the city (eastern) side abutment, the new culvert was to comprise a twin box culvert structure, supported on a cast in-situ concrete base slab.

Initial geotechnical involvement on this project comprised the drilling of vertical boreholes for the design of piles to support the new bridge superstructure. Subsequent to this, additional investigation and design work was completed for the new pavements and the new culvert running below the bridge in close proximity to the city side bridge abutment. At this stage, the extent of excavation required for the construction of the new culvert was assessed and the issue of the potential impact on the existing city end bridge abutment raised with the Client. However, due to the age of the existing bridge, only very limited information was available on the details of the existing abutments, and in particular, no information on the founding depth, or foundation materials, was available.

2. SITE GEOLOGY AND EXISTING CONDITIONS

The 1:100,000 Geological Map of Sydney indicates that the site is underlain by Ashfield Shale of the Wianamatta Group with Hawkesbury Sandstone at greater depth. Typically, the Ashfield Shales in Sydney are overlain by a residual profile comprising very stiff and hard silty clays. However, the initial investigation works for the bridge piles and road pavements indicated a subsurface profile comprising granular pavement materials and clayey fill overlying natural clays the weathered shale and sandstone bedrock. The natural clays varied from stiff to hard strength. Given the location of the bridge within a gully feature, the clays were assessed as most likely being of alluvial origin. During some early stage works, inspection of a temporary excavation indicated that the alluvial clays exhibited a dramatic loss

in strength if allowed to become saturated. No investigation of the existing rail embankment fill was completed, however, from past experience across the Sydney rail network, it was assumed that the embankment fill would most likely comprise a poorly compacted clayey material with granular inclusions.

3. EXISTING AND PROPOSED STRUCTURE

In order to determine the founding details of the existing bridge abutment, two angled boreholes were drilled through the abutment and into the foundation profile. These boreholes showed the bridge abutment to be founded within the silty clay profile. The relationship between the existing abutment, and its footing, and the proposed new culvert is indicated in Figure 1. As can be seen from Figure 1, the excavation for the proposed box culverts was to extend in close proximity, and slightly below the existing city side abutment.

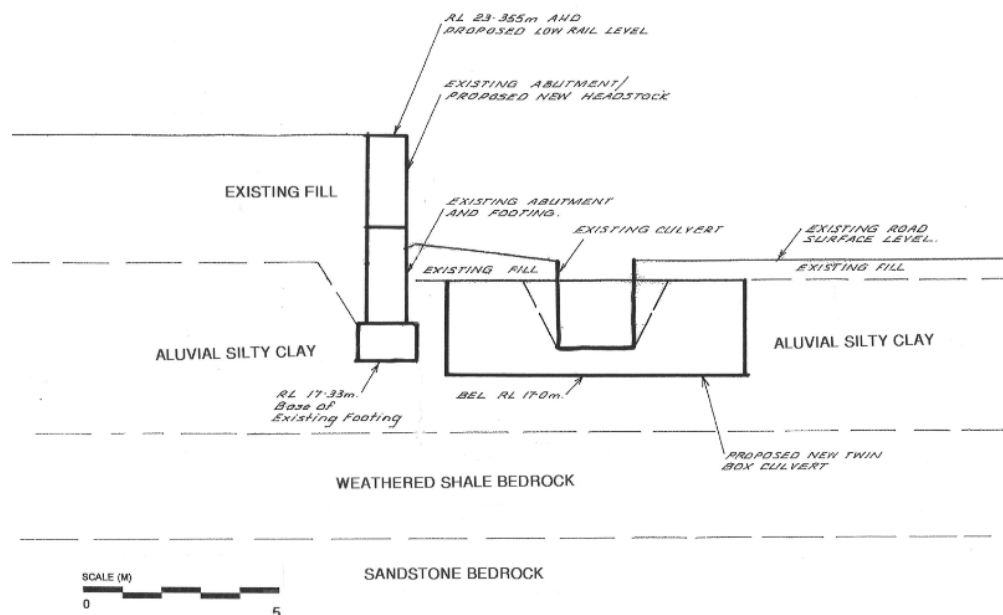


Figure 1: Cross Section showing relationship between existing bridge abutment and proposed new culvert

4. ABUTMENT STABILISATION DESIGN ANALYSIS

4.1 DESIGN CONSIDERATIONS

The principal aim of the excavation staging and stabilisation system for the bridge abutment was to reduce the risk of abutment instability and settlements of the track behind the abutment to levels acceptable by RailCorp (the asset owner). Whilst absolute (or total) settlement of the track was considered, the main concern was to limit any differential settlements, and hence any curvature of the track, as this has a much greater impact on the running of trains.

Given the restricted space between the existing bridge abutment and the proposed new box culvert, any stabilisation system works had to be no greater than about 0.5m wide. Further, the new bridge deck limited the headroom for construction equipment to less than 2.5m for the initial works.

The excavation staging and stabilisation system design also had to allow for the possibility of flooding through the excavation after demolition of the existing culvert but before the new culvert could be installed. Due to the relatively large upstream catchment, diversion of flood flows using pumps or similar was not feasible.

As all of the works were to be completed whilst the rail track was operational, a surcharge equivalent to a full passenger train (37kPa) was applied at track level.

All these restrictions combined with time limitations for completion of construction meant that a conventional pile, shotcrete and anchored solution was preferred by all parties.

3.2 ANALYSIS PROCEDURE

Analysis of the excavation staging was completed using PLAXIS 2D (Reference 1), a two dimensional finite element modeling program. This program allows for a number of stages of works to be considered. A number of iterations of the model were considered to assess the impact of alterations to the stabilisation system and excavation stages. Given the relatively small displacements anticipated, a hardening soil model with small strain stiffness was adopted for the soil

and bedrock materials. Such a material model more accurately models the displacements of soils at small strain increments as a higher material modulus is adopted over these smaller strains. For this model, a small strain limit of 1.5mm was adopted. For larger strains, conventional hardening soil model then applies.

As the analysis required detailed assessment of the soil/structure interaction during staging of the works, conventional analysis techniques were not deemed sufficient for this project. However, as a design check, the loads on the rear face of the abutment predicted by PLAXIS 2D were checked using conventional lateral earth pressure calculation methods and assessed to be reasonable.

A number of cases were also considered with a high water table within the excavation, to assess the possible impact of flooding prior to the new culverts being constructed. A rapid drawdown case, with no water within the excavation, and high groundwater within the soil profile was also considered.

3.3 MATERIAL PROPERTIES

Material properties for the soil and bedrock materials were based on detailed review of the available subsurface information, our experience with similar materials and published data (References 2&3). The following material parameters were adopted:

Geotechnical Parameters Adopted for Soil and Bedrock						
Parameter	Existing Fill - including embankment	Alluvial Clays	Engineered Fill (new)	Lean Mix Concrete	Weathered Shale Bedrock	Sandstone Bedrock
<i>Unsaturated Unit Weight</i> (kN/m^3)	18	18	19	21	20	23
<i>Saturated Unit Weight</i> (kN/m^3)	18	18	19	21	20	23
<i>Cohesion (c) (kPa)</i>	0	0	0	200	25	50
<i>Internal Angle of Friction (ϕ)</i>	30	27	33	40	30	40
<i>Modulus (E_{50}, E) (MPa)</i>	15	25	25	200	100	500

Table 1: Adopted Geotechnical Material Parameters

5. EXCAVATION STAGING AND STABILISATION SYSTEM

From our analysis, a stabilisation system comprising two rows of anchors and a row of contiguous piles was designed to support the existing abutment during construction of the new culvert. The purpose of the contiguous piles was to support the soil profile below the toe of the abutment, and protect it against possible erosion during flood flows. The purpose of the anchors was to limit any lateral displacement of the existing abutment. The stabilisation system designed is shown in Figure 2. Whilst analysis indicated that movement of the abutment would be sufficiently limited with only one row of anchors, given the lack of detailed knowledge of the abutment construction, backfilling etc. and risks associated with excessive movement of the abutment, we considered the installation of two rows of anchors to be prudent. The analysis also indicated that if the row of contiguous piles in front of the abutment footing were not incorporated into the design, there was a significant risk of bearing capacity failure of the foundation material, leading to excessive movement, or possible collapse of the abutment.

The following excavation and construction stages were adopted for our analysis:

- Excavate and construct existing culvert.
- Install new bridge headstock.
- Excavate to RL20m,
- Construct 1st stage of structural shotcrete, install first (upper) row of anchors and prestress.
- Excavate to top of existing bridge footing (RL18.33m).
- Install 300mm diameter contiguous piers and construct remaining structural shotcrete.
- Install second (lower) row of anchors and prestress.
- Demolish existing culvert and excavate to bulk excavation level for new culvert (RL17.0m).

- Construct new culvert.
- Backfill against new culvert. Backfill to comprise lean mix concrete between existing abutment and new culvert.
- Destress upper row of anchors.

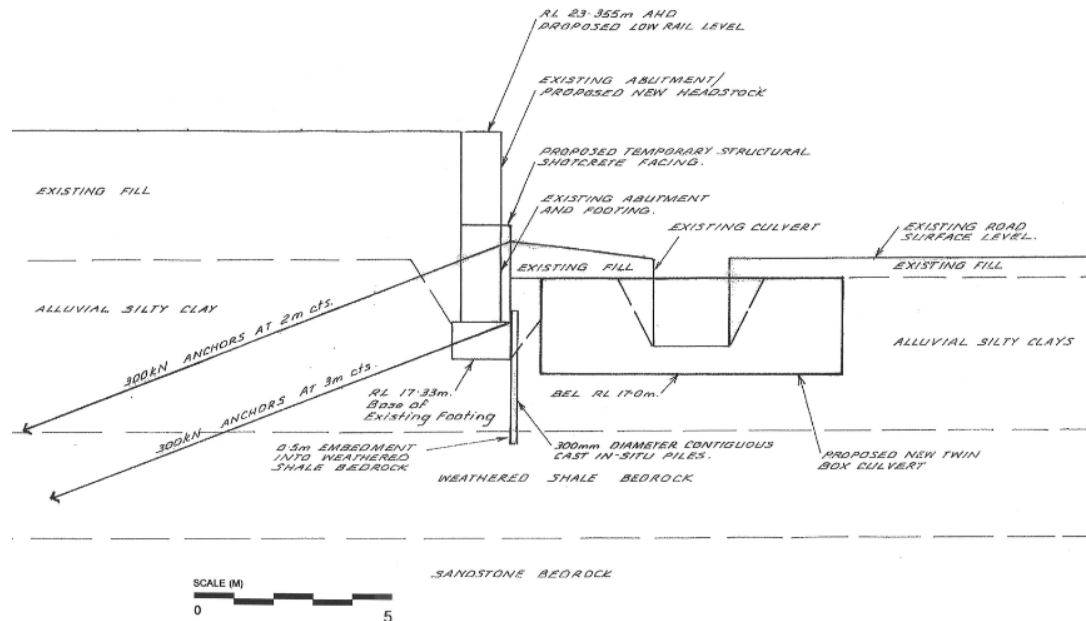


Figure 2: Abutment Stabilisation System

Cases considering potential flooding following the completion of bulk excavation, including rapid drawdown after such flooding were also considered. These flooding stages were found to be the critical stages in terms of both deflections and loads on the contiguous piles.

Following construction of the new culverts, the void between the abutment and culverts was backfilled with lean mix concrete due to the limited access available for the placement and compaction of conventional fill materials. As the long term lateral support of the abutment depended on this material transferring the loads to the new culverts, a poorly, or variably, compacted fill could result in ongoing creep of the abutment. The bottom row of anchors were left stressed as modeling indicated excessive movements could occur if they were destressed prior to backfilling. As the anchors were constructed of plain steel strand with no encapsulation system, it was considered likely they would corrode and destress with time. Given their location, it was assessed that there was a negligible risk they would be exposed or damaged while they were still stressed.

6. PREDICTED MOVEMENTS

The finite element analysis predicted maximum vertical settlements of up to 2mm at track level during the excavation and constructions works. Higher settlements of up to a maximum of 4mm at track level were predicted if an adverse flood and subsequent rapid drawdown event occurred whilst the excavation was open. These settlements were considered acceptable by RailCorp. The predicted settlements decreased linearly with distance back from the rear face of the abutment to near zero at about 10m from the abutment.

Removing the row of contiguous piles or adopting only a single row of anchors resulted in significantly higher settlements at track level, with settlements of 10mm or more predicted for the bulk excavation stage. Settlements of this magnitude would have been unacceptable to RailCorp, and hence, the second row of anchors and contiguous piles were incorporated into the design.

Consideration was given to adopting a higher drained cohesion for the alluvial clay profile, however, sensitivity analysis indicated that displacements were highly sensitive to this parameter. We therefore adopted a drained cohesion of 0kPa. Due to the limited timeframe available to complete the design and analysis, triaxial testing of the alluvial clays was not feasible.

An output plot from PLAXIS 2D showing predicted vector displacements just below track level is shown in Figure 3.

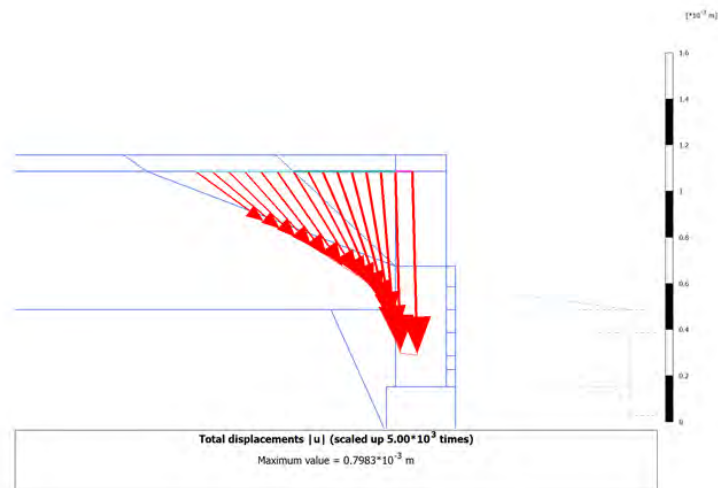


Figure 3: Predicted displacements at track level at bulk excavation stage.

7. ALTERATIONS TO CULVERT CONSTRUCTION STAGING

Due to the level of disruption to traffic flow caused by the works, Council requested that the new twin box culvert be partially constructed as a matter of priority to allow for the reopening of a portion of the roadway below the bridge.

Additional analysis was then completed to assess if construction of the western half of the culvert (i.e. the portion away from the bridge abutment) could be completed prior to anchor and pile installation. The analysis was completed using the same model conditions as for the initial case, and it was assessed that formation of a cut batter against the bridge abutment would provide sufficient support to allow for the construction of the western half of the culvert. The proposed temporary cut batter slope is shown in Figure 4.

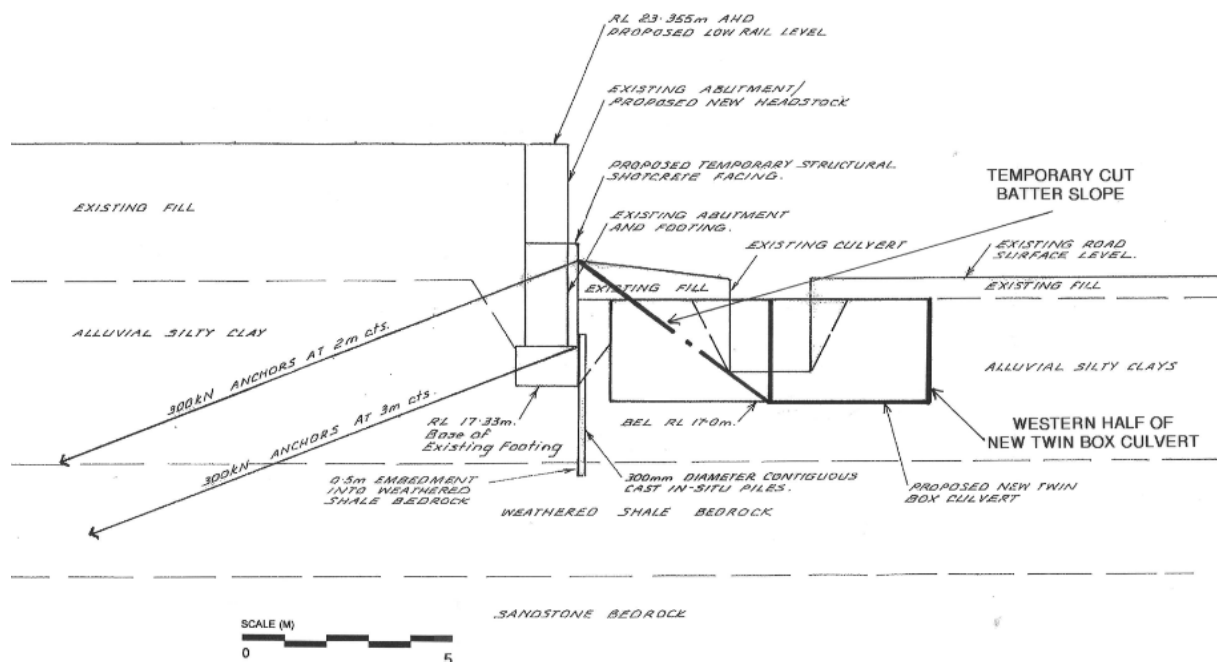


Figure 4: Revised staging to allow for construction of western half of culvert prior to anchor and pile installation

8. CONSTRUCTION MONITORING

A number of inspections were completed during construction to monitor the construction progress and confirm our assumptions regarding the subsurface profile across the site. Inspections were made of the anchor and pile installation and bulk excavation works.

Ongoing optical survey monitoring of the rails behind the abutment was completed during the course of the excavation and construction works. The monitoring was completed for both rails for both the up and down main tracks at 2m intervals for 18m from the rear wall of the bridge abutment. Measurement of horizontal position as well as elevation was completed.

Survey monitoring of the abutment structure was not completed during the excavation and construction works.

9. SURVEY MONITORING RESULTS

The survey monitoring of the rails behind the bridge abutment recorded maximum vertical settlements of 1mm over the course of the culvert construction works. In general, the settlements were recorded in the area between the rear of the existing abutment and 8m from the rear face of the abutment. We note however, that up to 8mm of settlement occurred in the period between the construction of the new bridge superstructure and commencement of works on the new culvert. It was considered that this early movement was primarily due to settlement of the track formation under initial loading. Further, no adverse track performance was reported during the course of the works.

Horizontal movement of the rail tracks of a maximum of 2mm was recorded, however, there was no consistent pattern to the horizontal movement, with some adjacent monitoring locations moving in opposite directions.

10. CONCLUSIONS

The design and analysis of the abutment stabilisation system produced a design which was both acceptable to the client in terms of constructability, and acceptable to the asset owner (RailCorp). Liaison with both of these parties during the course of the design process assisted in coming to a mutually acceptable system.

The displacements during construction were generally less than those predicted by our model, but followed a similar pattern of movement, indicating that the assumptions made regarding the material properties and geometries were likely to be somewhat conservative. Given the unknowns regarding the existing abutment construction and backfill, this is considered to be reasonable.

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ENGINEERING GEOLOGICAL DOMAINS AND CORRIDOR PROJECTS

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ABSTRACT

Engineering geological domains are areas with an assemblage of geological units and geomorphological landforms and processes, displaying similar engineering characteristics. In a corridor project, the combination of these domains makes up the preliminary engineering geological model. Compilation of the preliminary engineering geological model, and therefore the definition of domains, should be the first step in any corridor project and remains an integral database through subsequent phases. Defining domains for a project is relatively cheap, can be undertaken without acquiring land access permits, without conducting expensive ground breaking investigations, and requires fewer resources, compared to traditional investigation methods. The initial development of domains typically involves compilation of existing data including previous experience in the area, published geological and topographic maps, aerial photograph interpretation and site reconnaissance. Each subsequent project phase adds to the definition of the various units / features that comprise the domain. Over time, lessons learnt at specific locations can be extrapolated across whole domains. During recent inspection of operating corridor infrastructure, the individual aspects and advantages of domain based investigation approaches were reinforced. Earthworks features such as cuttings and embankments displayed remarkable similarities in terms of engineering characteristics throughout domains defined prior to construction. These observations highlight the value of domains in design, construction and operation / maintenance of infrastructure in corridors.

1. INTRODUCTION

In engineering a corridor project refers to the design, construction and operation of a piece of infrastructure that spans a great linear distance. Typical corridor projects include rail, road, pipeline and power line. The corridor for a given project refers to the space within which the limits of the project's activities take place. Due to their length, corridor project's activities will likely encounter variable surface and subsurface conditions. These variations tend to be controlled by the underlying geology and / or geomorphological processes that formed the landscape. Thus, commonly it is possible to define areas of relatively uniform conditions, namely engineering geological domains. Engineering geological domains can be defined as areas with an assemblage of geological units and geomorphological landforms and processes, displaying similar engineering characteristics. These will be termed 'domains' for the purpose of this paper.

Initial domain definition is often done early in a project and makes up the preliminary engineering geological model for a corridor. As such, domains become a key tool in providing design and construction considerations, estimating geo-uncertainty and identifying geo-hazards. Domains should form a dynamic database in terms of the life of the project, whereby their delineation should be emphasised and focused on during early investigation and then remodelled and updated during subsequent project phases as more detailed information comes to hand. Not only can latter investigation stages be used to update domains but domains themselves can be used to optimise the detailed investigation stages, focusing on problematic areas opposed to applying 'blanket' or 'grid' investigation methods. This approach aims to reduce geo-uncertainty while minimising invasive and expensive investigation methods. This paper discusses the advantages of applying engineering geological domains to each phase of corridor projects, citing specific examples from inspections carried out on operating heavy haul rail infrastructure.

2. APPLICATION

One of the key contributions of the engineering geologist in any engineering project, not just corridor projects, lies in the effective communication of the engineering conditions that can be expected given identified geological conditions. Domains provide a suitable and insightful platform across which engineering aspects can be communicated, particularly in corridor projects which can span tens to hundreds of kilometres and which have numerous design components. This platform can exist in the form of geo-referenced maps, cross-sections, three dimensional models, design-tables, or a combination of these. These data sets should be created according to the investigation phase for which they are to be applied.

Many names are given to the various phases and sub-phases of a corridor project, and are often tailored to the nature of the project, the type of infrastructure and the owners engineering models. However, for simplicity this paper will refer to four main phases, generalised as follows along with the relevant role of the engineering geologist in that phase:

- Preliminary Investigations – Desktop studies, site reconnaissance;
- Engineering Design – Detailed mapping, ground breaking investigations;
- Construction – Construction supervision; and
- Operation and Maintenance – Earthworks auditing, monitoring, maintenance and remediation as required.

Preliminary Investigation phases commonly comprise order of magnitude, scoping, and / or preliminary feasibility studies while Engineering Design phases typically comprise bankable feasibility and detailed design studies, or equivalent.

When applied effectively, domains have proven to provide valuable input into all phases of corridor projects. Figure 1 (Fookes, 1997) outlines the phase/s and expected geological methods that should be expended to acquire information required for input into the design phase.

2.1 PRELIMINARY INVESTIGATION PHASE

The preliminary investigation phase of a corridor project is an important phase for the engineering geologist. Baynes et al (2005) discusses in detail the importance of the preliminary investigation phase, whereby application of relatively cheap geological methods rapidly increase the engineering knowledge of the site. The collection of this low-cost information will support very broad decisions as to the direction of the project i.e. optimisation of alignments to reduce risk and cost, and what future land access is critical. Domains form the broad scale platform from which such decisions can be made as there is usually not enough information or time available for detailed, structure by structure, assessment. Because preliminary investigation methods are cheap, non-invasive and require few resources, they are easily implemented early in the project. However, it is the observation of the author that such early investigations are becoming increasingly restrictive in allocated timeframe. This can significantly compromise the quality of information yielded thus leading to less developed domain based assumptions.

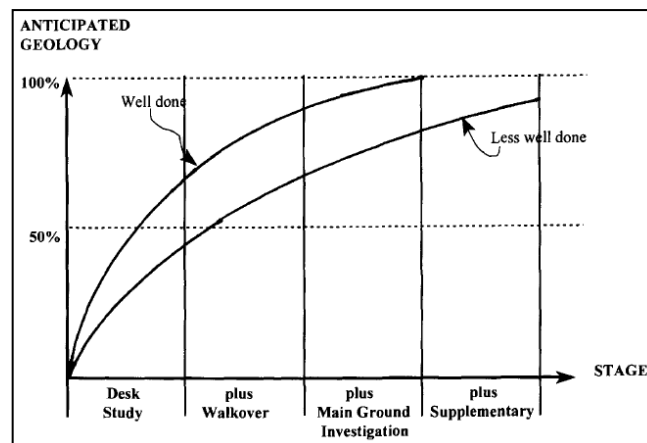


Figure 1: Expended geological effort through investigation phases. (from Fookes, 1997)

The preliminary investigation phase typically begins with a desktop study to compile existing and published information including any knowledge and experience project personnel may have in the area. Geological and geomorphological features are identified using aerial photograph interpretation combined with published geological maps. This is where domains are first delineated, commonly utilising some form of GIS (Geographic Information System) software. Large polygons are drawn around broad areas hosting similar geological units along with repetitive geomorphological landforms and processes. An example of preliminary domain classification for a corridor project is displayed in Figure 2 along with its corresponding map (note not all engineering geological factors usually depicted have been included in the example). Reconnaissance and mapping is carried out to confirm the domain classifications.

Due to the geological and geomorphological similarities, initial assessments on certain engineering aspects can be made and extrapolated across whole domains. At this early stage of investigation the results of these assessments are input into preliminary design such as earthworks geometries, excavatability and fill material identification and classification

DOMAIN	GEOLOGICAL AND GEOMORPHOLOGICAL INTERPRETATION	EMBANKMENTS					CUTTINGS				EARTHWORKS PLANNING ESTIMATION	
		FOUNDATIONS	RECOMMENDED EMBANKMENT SLOPE ANGLES (V:H)	BORROW AVAILABILITY			RECOMMENDED SLOPE ANGLES (V:H)	Rock %	Common %	MATERIALS PRODUCED FROM CUTTINGS (% OF CUTTING VOLUME)	RISKS	RECOMMENDED ALLOWANCES
				GENERAL BORROW	SELECT BORROW	ROCK PROTECTION						
X	Gentle to moderate colluvial foot slopes descending south from elevated Brockman surface striking NW-SE, predominantly comprising interbedded BIF and chert. Colluvial foot slopes comprise well graded uncemented gravels, sand and silt, commonly lacking plasticity. Southern part of domain occupied by minor to moderate SW-NE ephemeral valley drainage system encompassing moderately spaced wash channels dissecting the colluvial foot slopes.	Suitable on colluvial slopes, becoming variable within and adjacent to valley drainage system. Limited treatment may be required in poor areas, particularly for culverts in minor drainage lines.	Allow 1:1.5 Care must be taken with materials lacking plasticity as they will be susceptible to erosion.	Generally adequate from likely borrow and from cuts in mid to upper part of slope.	Adequate from likely borrow, again care should be taken with materials lacking plasticity.	Cuts unlikely to expose rock, source from adjacent domains or quarry.	1:1.5 Areas particularly low in plasticity may require slackening of slopes.	0%	100%	80% Type 2 (gravels), 20% Type 3F (Cobbly Gravels).	A. Greater zones of poor foundation. B. Batters not stable at recommended design slopes. C. Borrow quality not as good as anticipated.	A. Allow for increased treatment areas. B. Allow for cut back of some slopes to slacker angles. C. Allow for increased haulage of good quality fill.

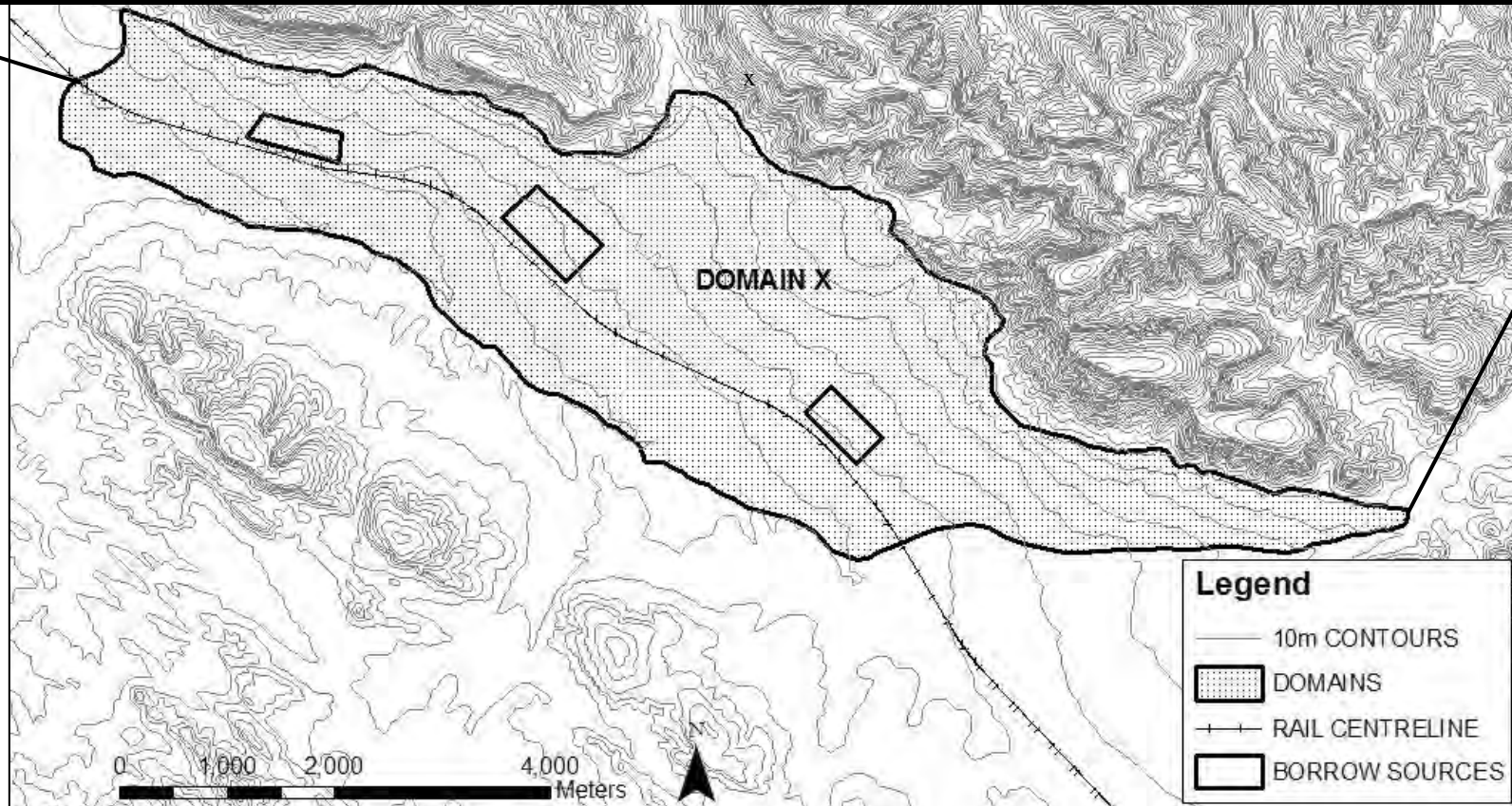


Figure 2: Typical domain classification complete with terrain map.

for input into mass haulage assessment. Initial assessments commonly include review of potential for geohazards such as rockfall / landslide, karst, fibrous materials, inundation, reactive clays, and collapsing soils etc. It is important that these initial design inputs are simple and easy to implement at this early stage, e.g. slope designs might be simply classified as slack (1V:1.5H), moderate (1V:1H) or steep (2V:1H). An example of preliminary domain classification with initial design assessments is illustrated in Figure 2. Early domain definition, which should cover the whole corridor being considered, allows repeated relative engineering assessments to be made due to realignments (both vertical and horizontal). This is an important advantage of the domain based approach as repetition of expensive (with respect to environmental impact, timeframe and financially) ground breaking investigation methods is avoided.

2.2 ENGINEERING DESIGN PHASES

During the Engineering Design phases the alignment is becoming more fixed, access to land is becoming more available and there is a requirement for more accurate cost estimates to construct the proposed project. During these phases the engineering geological model, and therefore domains, must be their most refined and comprehensive. For heavy haul rail projects this is done utilising detailed mapping and ground breaking investigations.

Detailed mapping typically includes observation of critical outcrops for initial assessment of excavatability and foundation conditions, assessment of potential fill materials, geo-hazard recognition, measurement of critical defects for input into design slopes, as well as assessment of accessibility for ground breaking activities. Ground breaking includes geotechnical drilling of foundations, deep cuttings and geohazards, test pitting of the centerline and potential borrow sources, as well as ripping trials to determine excavatability in areas of cut. Various geophysical methods may also be applied to provide valuable information. Due to their expensive nature and requirement for permits and approvals, it is advantageous to implement these investigations once the alignment of the infrastructure has been more or less finalised. In the traditional investigation approach, where ground breaking activities make up the great majority of the project's data, many investigation locations are required. An important advantage of the domain based approach is that the number of investigation locations can be optimised and data obtained at these locations can be extrapolated elsewhere in the domain. Baynes et al (2005) discusses these two approaches highlighting the function of mapping in planning ground breaking investigations to define the engineering geological domains in a more systematic fashion. The authors go on to compare the domain based approach, where investigations are targetted on expected geological conditions to resolve geo-uncertainty, to the traditional approach where ground breaking activities are planned without regard for the geology of the site, applying nominal intervals between investigation locations along the alignment based purely on cost and information distribution.

Given effective application of the domain based approach to the engineering design phase, ground breaking investigations should be used to truth the initial design assessments and geo-hazard allowances, reducing risk and providing more accurate project cost estimates. Because the alignment is more or less finalised within the project corridor, the truthed design assessments can be applied on a cut by cut and embankment by embankment basis thus increasing design confidence. In the event unexpected conditions are encountered during the ground breaking activities, new assessments and allowances can be made for these conditions and then extrapolated across the domain in which they were encountered.

Section 2 touched on the importance of effective communication between engineering geologists and project engineering personnel. This is discussed in detail by Baynes (1999). The first aspect of this communication is the ability to accurately interpret geological information in an engineering fashion. The second aspect of this communication is the ability of the interpreted information to be displayed in a comprehensive, useful and easy to digest manner. Domains provide an effective system allowing the great length of corridors to be separated into manageable work packages and presented accordingly. This is a crucial part of the engineering geologist's role and is commonly overlooked.

One example of domain based observations suitable for input into the engineering design phase was identified during inspection of an operating heavy haul rail. The domain in question (Domain X from Figure 2) consists of gently to moderately south sloping colluvial foot slopes beneath an east-west striking ridge of Banded Iron Formation (BIF) and shales. These colluvial foot slopes comprise well graded gravels, sand and silt. The colluvial slopes appear to overly a sequence of fine-grained valley infill deposits. Desktop and mapping investigations identified sections of the slope where the characteristics of the colluvial gravels appeared to conform to the sub-ballast capping specification (usually the most difficult materials to source naturally). Thus it was possible to target a series of potential borrow areas across the domain and apply for access to these areas to conduct ground breaking investigations. The use of mapping and domains limited land tenure activities and focussed investigations on the most probable borrow sources. Significant

time and budget were saved based on these informed decisions, highlighting the applicability of domains during investigation and design.

2.3 CONSTRUCTION PHASE

The traditional investigation approach mentioned in section 2.2 assumes all possible geotechnical information can be and is acquired and submitted during the design phase of a corridor project. This is generally not the case due to a variety of constraints including time, budget and land access. During construction of corridor infrastructure natural variability in the local ground conditions is commonly observed. Potential for such variability is commonly termed 'geo-uncertainty' and it is often how geo-uncertainty is dealt with during construction that determines whether completion of a project comes within budget or not. Geotechnical investigations for corridors generally cannot completely remove geo-uncertainty but should minimise it as much as reasonably possible. As such, allowances should be made to cater for geo-uncertainty and these can be quantified in the project cost estimate. The advantage of implementing the domain based approach in the construction phase is that allowances made for geo-uncertainty can be extrapolated across whole domains before being applied. This is most effectively implemented utilising the observational procedure, first applied to soil mechanics by Terzaghi and discussed in detail by Peck, 1969. An extract from Terzaghi et al (1996) best summarises the observational procedure:

"Design based on the most unfavourable assumptions is inevitably uneconomical, but no other procedure provides the designer in advance of construction with the assurance that the soil-supported structure will not develop unanticipated defects. However, if the project permits modifications of the design during construction, large savings can often be made by designing on the basis of the most probable rather than the most unfavourable possibilities. The gaps in the available information are filled by observations during construction, and the design is modified in accordance with the findings. This basis of design is called the observational procedure"

Peck (1969) states that often in his experience the observational procedure is only applied to an engineering project once unfavourable conditions have arisen during construction, and thus it is more a 'best way forward' procedure in comparison to the pre-planned procedure summarised above. He also states that to apply the observational method early on in the project, "...the designer must have in mind a plan of action for every unfavourable situation that might be disclosed by the observations". However, due to the great length of corridor projects, the variability in ground conditions is too great to quantify allowances for every unfavourable situation in every possible location. This is where domains come to the fore. Because domains are delineated based on uniformity of their anticipated ground conditions, it can be reasonably assumed that unfavourable conditions will also display uniformity throughout a given domain. Therefore, allowances for ranges of unfavourable or indeed more favourable conditions can be quantified and extrapolated across whole domains. Preconceived plans of action can then be modified and implemented during construction according to the actual conditions observed. Should conditions be more favourable it may be possible to implement design revisions to save project costs.

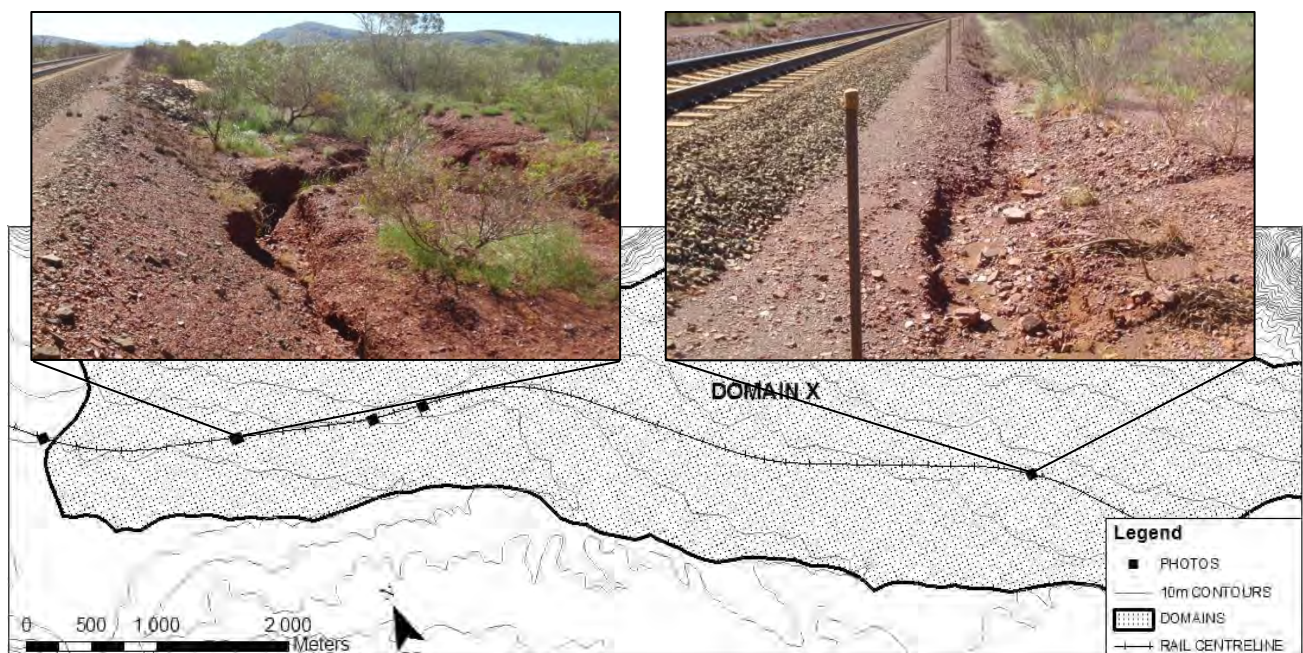


Figure 3: Erosion of transition drains repeated consistently within Domain X.

One such example of unanticipated and unfavourable conditions uncovered during construction was noted during inspection of an operating heavy haul rail. Construction issues were encountered across a domain encompassing moderately undulating colluvial mid – foot slopes. The colluvial slopes comprised moderately to strongly iron cemented pisolitic gravels and fines, often termed ferricrete. When excavated, these gravels dominated by rounded pisolitic clasts and lacking plastic fines proved difficult to compact during construction of embankments. These unfavourable conditions were encountered during the initial construction and it was considered that the same issues could be anticipated throughout the domain, and in other similar domains. Alternative designs appropriate to the conditions encountered were developed and implemented accordingly throughout the various relevant domains.

2.4 OPERATION AND MAINTENANCE PHASE

The life of a corridor project with respect to the engineering design team typically terminates upon construction of the infrastructure. Adequate performance of the structures over the duration of operation for which the infrastructure was designed, and often for significant additional time, is vital. In the heavy haul rail industry in Western Australia extension in project life is commonly accompanied by an increase in loading of the infrastructure beyond the design capacity. This can result in unforeseen serviceability issues and significant increases in maintenance. The cost to operations of reinstating heavily utilised infrastructure after failure / distress far exceeds the cost of investigation, design, and actual remediation of the problem areas. Additional monitoring and engineering input is therefore required during the operational life of the infrastructure. If this input is applied on a case by case basis, the resultant maintenance implementation is reactive and not proactive.

As with the project life, it is often thought that the function of domains expires at the completion of construction. However domains provide a logical solution for proactive monitoring and maintenance implementation. Observation of geotechnical issues within a given domain can act as an early identification system whereby preventative measures can be implemented throughout the domain, prior to problems affecting productivity arising, negating the requirement for shut downs and reactive maintenance. Domain X from Figures 2 and 3 is a prime example of this. During traverse of this domain it became apparent to the author that the transition drains (from cut to fill) on the upslope side of the rail formation consistently displayed signs of deep erosion (Figure 3). Mitigation of this erosion can be designed for, likely in the form of effective up-slope cut-off drains or detailing transition drains with rip rap, and can be implemented throughout the domain before embankment stability is compromised.

Another example was observed while traversing an undulating domain through broad residual hills and incised gullies, where the geological profile comprised weathered dolerite underlying up to three metres of iron cemented duricrust. The rail alignment through this domain exhibited deep cuttings displaying eroded faces, and moderate to high embankments. The deeper cuttings were capped by duricrust which resisted erosion much more readily than the underlying residually weathered dolerite. Slaking and erosion of the residually weathered dolerite often facilitated undercutting of the overlying and more resistant duricrust, slowly exposing the infrastructure to increased risk of rock fall. In addition, severe erosion of the weathered dolerite materials fills the toe-drains, impeding the flow of water during wet periods and allowing the formation to become saturated. In this case it was possible to rank structures prone to failure i.e. cuts <3m low risk as weathered Dolerite was not exposed, cuts 3m – 6m some drain clean outs / regular maintenance required due to only limited weathered dolerite exposed, and cuts >6m remediation required to prevent rockfall. Thus monitoring and maintenance in this domain can be focussed on higher risk structures.

3. CONCLUSIONS

When applied effectively, engineering geological domains provide valuable input to all phases of corridor projects from preliminary investigations through to operational monitoring and maintenance. The advantages and efficiencies of domains have been highlighted citing specific examples from heavy haul rail infrastructure.

Domains are first delineated during the preliminary investigation phase using existing information and site reconnaissance providing a platform for broad based decision making. During domain classification initial assessments can be made for input into feasibility and preliminary design with greater confidence and without the need for site access for expensive ground-breaking investigations. Domains are then utilised to select and optimise infrastructure alignments within the corridor. The fact that domains cover complete corridors means that multiple alignments can be assessed by the engineering teams on a relative basis without repeated investigation. Utilising more traditional approaches is often not possible due to land access issues and when possible they are far more expensive and almost inevitably result in duplication of works as alignments commonly move about during these early phases as designs develop. Domains provide an informative dataset for easy communication with project engineering personnel. The

length of the corridor can be separated into manageable work packages according to domains. Utilisation of the observational procedure combined with domains during the construction phase provides powerful tools to deal with geo-uncertainty. Allowances made for geo-uncertainty can be extrapolated across entire domains due to expectations of similar favourable or unfavourable conditions. Preservation of domain based datasets through to operation of corridor infrastructure can provide an early identification system whereby proactive and preventative maintenance can be implemented.

While the domain approach has been discussed with particular reference to heavy haul rail the core principles and advantages of domains outlined, when applied effectively, are just as applicable in most other forms of corridor project.

4. ACKNOWLEDGEMENTS

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NUMERICAL MODELLING OF SOIL-STRUCTURE EARTHQUAKE RESPONSE INCLUDING UPLIFT AND SOIL YIELDING

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ABSTRACT

Investigation of the rocking and uplift of a single degree of freedom (SDOF) soil-structure system on a bed of springs (Winkler model) has provided interesting insight into the earthquake response of shallow foundations. Rocking and uplift during earthquake excitation have been suggested to have potential benefits of reducing the forces transmitted to the structure. A widely-used structural design software package (SAP2000) was employed to model this integrated system.

The springs were allowed to detach from the structure to model separation of part of the foundation from the supporting soil, and this made the analysis highly nonlinear. Initially sinusoidal excitations were run through the model to evaluate the effect of detaching springs on the steady-state response of the system. Next the response of the system to actual time histories from the El Centro earthquake on 18 May 1940 and the Christchurch earthquake on 22 February 2011 was computed. Varying amounts of uplift were induced by scaling the earthquake records. The final stage of the modelling was to allow the springs to yield when the compressive loads would indicate bearing failure of the soil beneath.

This paper will demonstrate the effect of yielding of the foundation soil and foundation uplift on the earthquake response of the soil-structure system.

1. INTRODUCTION

In earthquake resistant design there is an increasing awareness of the need to model the foundation and the structure supported by the foundation in an integrated manner. The range of analysis tools available for civil engineering design enables this to be done but it requires those with expertise in geotechnical engineering and those with expertise in structural engineering to communicate effectively (Pender, 2007). Integrated numerical models provide a basis to undertake a more holistic design approach, and probably the simplest and easiest method to implement these without markedly compromising theoretical rigour is to model the interaction between the soil and the foundation as a bed of springs (Winkler model). This type of modelling sets out to balance ease of use in general engineering practice with more rigorous theoretical solutions for which there may be uncertainties in the input parameters (Harden and Hutchinson, 2009).

Traditional soil-structure interaction (SSI) involves linear elastic interaction between the soil and the foundation. This has had moderate success in promoting the significance of integrated modelling of the foundation and the structure. However, there has been wide consensus that for the benefits of SSI to be realized the nonlinear soil deformation effects and nonlinear geometrical effects need to be accounted for (Paolucci, 1997; Gazetas et al., 2007; Pender et al., 2009; Deng et al., 2010; Pecker and Chatzigogos, 2010). In the seismic design of structures, these nonlinear effects include rocking and uplift of the foundation, and the nonlinear response of the soil.

This paper presents numerical modelling of a single degree of freedom (SDOF) soil-structure system on a bed of springs that captures the significance of foundation uplift and soil yielding on the earthquake response of the system. Modelling is undertaken employing the widely-used structural design software package SAP2000, and the model is subject to static push-over analysis as well as horizontal nonlinear time history excitation in the form of sinusoidal excitations and scaled actual time histories from the El Centro earthquake on 18 May 1940 and the Christchurch earthquake on 22 February 2011. The springs were allowed to detach from the structure to model uplift from the supporting soil and were allowed to yield when the compressive loads indicated bearing failure of the soil. This provides an interesting insight into the earthquake response of shallow foundations.

2. SDOF MODEL FORMATION

Numerical modelling was undertaken using a SDOF “lollipop” type structure as shown in Figure 1. Early research into the rocking and uplift of structures subjected to dynamic excitation came about from observations of tall, slender structures having been relatively undamaged following earthquakes (Housner, 1963; Priestley et al., 1978). The model

presented in this paper was developed with this in mind, with a 50 tonne lumped mass supported by a 20 metre high reinforced concrete column, which sits on a rectangular concrete beam 8 metres wide. In SAP2000, the column and foundation beam were set as massless elements, and the lumped mass was applied at the top of the column. In this way a simple SDOF structure was developed.

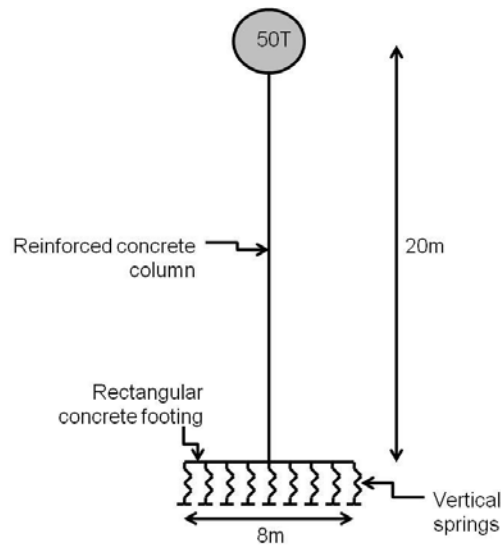


Figure 1: SDOF model implemented in SAP2000

The foundation of the SDOF structure was assumed to be resting on the ground surface. The soil on which the foundation is sitting was assumed to be clay with an undrained shear strength of 50kPa and a shear wave velocity of 180 m/s. The interaction between the structure and the foundation soil was modelled by 9 vertical springs, the stiffness of which provided the vertical and rotational stiffness of the foundation. The stiffness of these springs was ascertained from equations developed by Gazetas et al. (1985) using a small strain shear modulus as defined in Equation 1:

$$G_0 = \rho V_s^2 \quad (1)$$

where ρ is the density of the soil and V_s is the shear wave velocity. This value was then modified using a reduction factor of 0.5 as recommended in Eurocode 8-Part 5 (2003) assuming a peak ground acceleration of 0.2g. The static stiffness value was then altered using the method outlined by Gazetas (2002) to account for dynamic loading effects. The individual spring stiffness values were determined by dividing the total stiffness evenly between the 9 springs, resulting in equal individual spring stiffness values of 50,000kN/m.

To model uplift, the foundation needs to be able to detach from the springs when the force in a spring reduces to zero so that the foundation does not apply tensile forces to the underlying soil. For the analysis of uplift only (no yielding), this was achieved in SAP2000 by setting the springs as nonlinear gap elements. These elements can take compression loads only so that when the spring force reaches zero a gap opens up between the foundation beam and the spring.

For analysis of combined uplift and yielding, the springs were modelled using the multi-linear plastic with Takeda Hysteresis property available in SAP2000. To represent uplift, zero force was set for all displacements in the positive tensile range. When a spring was compressed, the load deformation behaviour followed a bilinear relationship in that once the load reached 0.8 times the ultimate vertical load, determined using the static bearing capacity for the tributary area of each spring, the spring yielded and the spring stiffness reduced. When unloading occurred after yielding, the initial stiffness value was followed until it reached a secant path with the force deformation relationship specified in the tensile range i.e. once there was zero force in the spring there was unrestrained movement of the foundation. In this way, a hysteresis loop is formed in the compressive zone only and occurs once the vertical force exceeds 0.8 times the ultimate vertical load.

For simplicity, the horizontal equilibrium of the system was maintained by restraining the central node at the base of the column against horizontal motion.

3. ANALYSIS AND RESULTS

Numerical analysis was undertaken using SAP2000. Firstly, static push-over analysis was carried out to investigate the effects of uplift. Then nonlinear time history analyses were undertaken using varying sinusoidal excitation to evaluate the steady state response during uplift and combined uplift and yielding. Finally, actual earthquake time histories were run through the model.

3.1 STATIC PUSH-OVER ANALYSIS

The first step in the numerical modelling was to investigate the static push-over characteristics of the SDOF soil-structure system. The vertical load was applied first so that the foundation springs were in a compressed state and this first step was maintained throughout all of the analyses. The horizontal load was then applied as an acceleration of the lumped mass in a step-wise manner until the critical moment was reached. The results of this analysis established the moment-rotation characteristics of the system.

The rotation of the foundation as a function of the applied moment is shown in Figure 2a. The secant rotational stiffness was determined from this data and then used to calculate the change in the natural period of a SDOF system as uplift increases, as plotted in Figure 2b. This was done using an equation developed by Wolf (1985) (equation 3.57) for the equivalent natural frequency of a SDOF system as shown in Equation 2:

$$\omega_n^2 = \frac{\omega_s^2}{1 + \frac{K_s}{K_h} + \frac{K_s h^2}{K_\phi}} \quad (2)$$

where ω_n is the natural frequency of the system, ω_s is the natural frequency of the fixed base structure, K_s is the stiffness of the structure, K_h is the horizontal stiffness of the foundation, K_ϕ is the rotational stiffness of the foundation, and h is the height of the structure. Figure 2c displays the change in axial force in each of the foundation springs as the moment increases. Once a spring uplifts there is zero force in that spring as explained above.

The plots show that once uplift is initiated the response of the system becomes increasingly nonlinear. It also demonstrates how the natural period of the system is dependent on the degree to which the foundation uplifts.

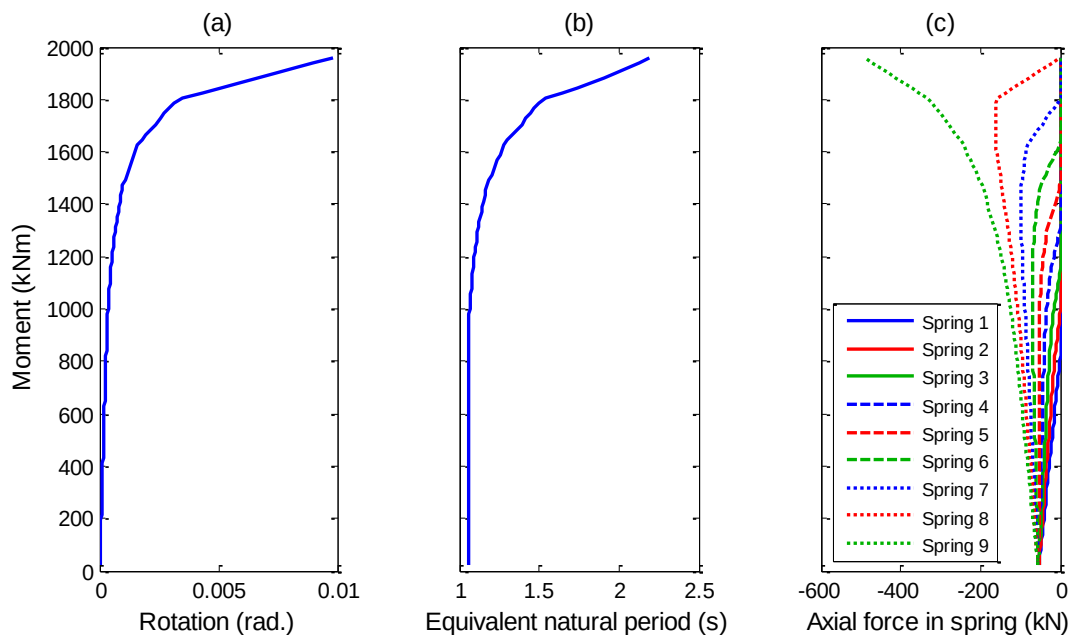


Figure 2: Static push-over plots (a) moment-rotation curve; (b) moment-equivalent SDOF natural period curve; (c) moment-foundation spring axial force curves.

3.2 NONLINEAR SINUSOIDAL TIME HISTORY ANALYSIS

Nonlinear time history analysis was undertaken by applying sinusoidal horizontal accelerations at various scale factors for a range of excitation periods. Sinusoidal functions were used as a preliminary means to represent earthquake excitation and allow the resulting data to be easily manipulated. The scale factors enabled the interpretation of the effects of varying degrees of uplift on the steady-state response of the system and comparisons with a fixed base system. The effect of soil yielding on the steady state response was then able to be compared for each of the scale factors.

Direct integration time history analyses were run in SAP2000 for a range of individual load cases where the period of the input sinusoidal excitation function was varied. The amplitude of the excitation was kept constant throughout. As in the static push-over analysis, the vertical load was applied first, then each excitation was applied for 30 seconds and the time history analysis run for 40 seconds in order to capture the steady-state response of the system and the subsequent

decay of motion. For each load case the damping was specified as directly proportional to the mass and the stiffness to achieve 5% system damping for a fixed base structure.

Once the analysis was complete the horizontal displacement of the lumped mass was obtained for all the excitation periods. The absolute maximum displacement was ascertained from the data for the time period where it was interpreted that a steady-state had been reached for each load case, usually between times of 15 seconds and 25 seconds. This could then be plotted against the associated excitation period to get the response of the soil-structure system at a particular scale factor. The procedure was then repeated for different scale factors.

The resulting response of the SDOF model at the different scale factors (in terms of gravitational acceleration g) for uplift only and combined uplift and yielding are shown in Figures 3a and 3b respectively. The fixed base response is also shown in both cases for comparison. The displacement was made non-dimensional by multiplying by the fixed base stiffness divided by the horizontal force, and the period was made non-dimensional by dividing by the fixed base natural period.

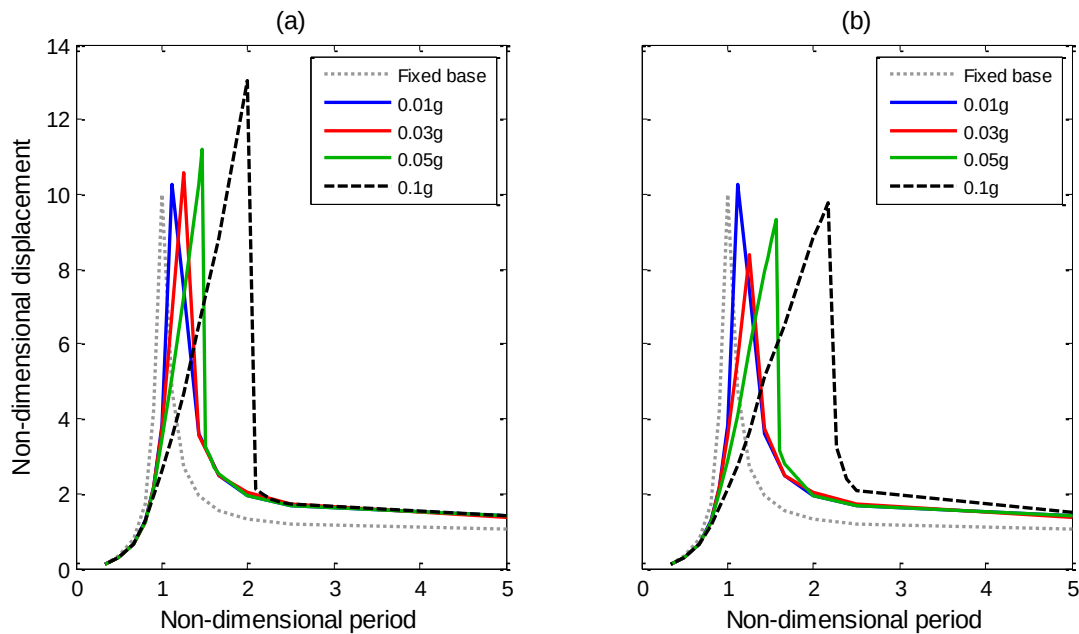


Figure 3: Response of the SDOF soil-structure system to nonlinear sinusoidal time history analysis with (a) uplift only and (b) combined uplift and yielding foundation springs.

It was found that as the scale factor increased the degree of uplift increased and once uplift was initiated, the degree of uplift appeared to increase almost exponentially as the intensity of excitation increased. At a scale factor of 0.01g there was no uplift even at the excitation period corresponding to the maximum steady-state response. At the excitation period corresponding to the maximum steady-state response at a scale factor of 0.03g approximately 70% of the foundation uplifted and at 0.1g the majority of the foundation uplifted so that the model rocks back-and-forth on the springs at the edge of the foundation.

A softening system was observed, whereby with increasing levels of excitation and corresponding uplift the maximum steady-state response occurred at a higher period of excitation. Also, as the degree of uplift increased there was an increase in the magnitude of displacement of the lumped mass.

Yielding of the foundation springs occurs only once uplift is initiated and results in further softening of the system. The magnitude of displacement of the lumped mass is reduced, with minor residual displacement of the foundation observed after excitation. However, uplift appears to have a more significant effect on the response of the soil-structure system than yielding.

3.3 EARTHQUAKE TIME HISTORY ANALYSIS

To investigate the response of the SDOF soil-structure system to actual earthquake excitation, time history acceleration data from the El Centro earthquake on 18 May 1940 (N-S component) and the Christchurch earthquake on 22 February 2011 (S45W component) was run through the model. Varying amounts of uplift were represented by scaling the earthquake records. The model was set up in the same way as described for the nonlinear sinusoidal time history analysis except the earthquake acceleration data was used as the time history function.

Once the analysis was complete the absolute horizontal acceleration of the lumped mass was determined for each scaled earthquake time history. The pseudo-acceleration response spectrum of this data was calculated in a piecewise manner to make comparisons between the responses of the system at different scale factors. As before, this was done for an uplift only case and a combined uplift and yielding case.

Figures 4a and 4b show the pseudo-acceleration response spectrum plots for the scaled Christchurch earthquake time histories with uplift only and combined uplift and yielding respectively. Similar results were obtained for the El Centro earthquake.

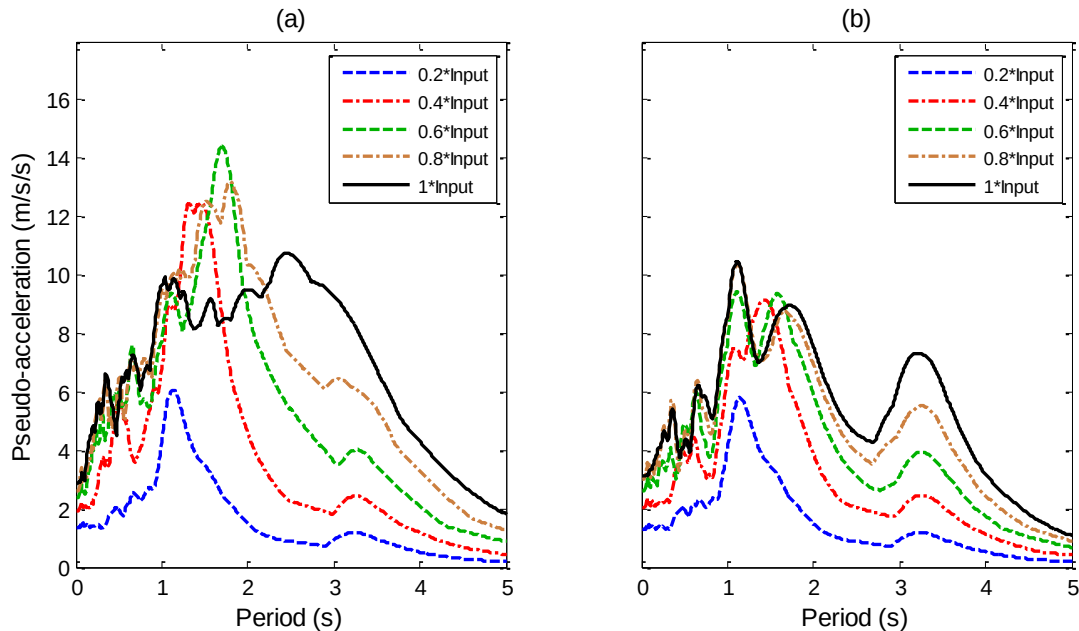


Figure 4: Pseudo-acceleration response spectrum plots of the absolute acceleration of the lumped mass due to the scaled Christchurch earthquake input excitation with (a) uplift only and (b) combined uplift and yielding foundation springs.

As in the sinusoidal time history analysis, a softening system was generally observed whereby the peak response occurs at higher period values as the level of excitation and corresponding uplift increases. It was found that the maximum amount of uplift for the 0.2 times Christchurch earthquake was approximately 30% of the foundation, whereas for 0.6 times and greater, the maximum amount of uplift involves the majority of the foundation. However, the occurrence of this maximum amount of uplift was very rare (one instant for the 0.6 and 0.8 times and three instances for the full scale excitation) rather than occurring continuously as in the steady-state response of the sinusoidal excitation.

It was noted that for the lower levels of excitation there appeared to be a well-defined peak in the response of the lumped mass whereas at higher levels of shaking there were multiple peaks and/or the peak became plateaued. Higher levels of excitation result in greater degrees of uplift, which was seen from the static push-over analysis to result in an increase in the natural period of the maximum response of the system. However, in general the acceleration response spectrum of the input earthquake has the greatest response at lower period values. The combination of the maximum response of the input earthquake being at low periods and the period shift to higher values as the level of uplift increases results in multiple and/or plateaued peaks in the acceleration response of the lumped mass for the higher levels of excitation.

Further softening of the system was again observed when the springs are able to yield. There is a definite reduction in the maximum response of the SDOF system and the maximum response generally occurs at a higher period values. The input earthquake acceleration again has an influence on the output response of the lumped mass and multiple/plateaued peaks are observed. The effect of yielding appears to be more significant when compared with the sinusoidal time history analysis.

4. CONCLUSIONS

This paper has shown, through integrated numerical modelling of a soil-structure system, how the nonlinear effects of foundation rocking and uplift as well as yielding of the foundation soil influence the earthquake response of shallow foundations. Firstly, the static push-over analysis demonstrated that once uplift is initiated the response of the system becomes highly nonlinear and that the natural period is dependent on the degree of uplift. Secondly, the time history analysis showed that as the excitation intensity was increased, the degree of rocking and foundation uplift increased and

resulted in a softening system, whereby the maximum response occurred at higher period values. When yielding of the foundation soil occurred, the system was further softened and there was a reduction in the magnitude of the response of the structure. In essence, rocking, uplift and soil yielding provided additional damping to the system and have the potential benefits of reducing the forces transmitted to the structure resulting in reduced damage during an earthquake.

Integrated modelling of the structure and foundation with allowance for the nonlinear effects of foundation uplift and soil yielding may provide a means to develop more rational and economically efficient earthquake resistant design. Recently there has been a paradigm shift to not only ensure life safety but to limit financial losses due to earthquake damage. This is likely to result in significant upfront construction costs if utilising conventional capacity design, where the ductile behaviour of the structure is assumed to dissipate seismic energy. It is therefore advantageous to allow for foundation nonlinearities so as to decrease the ductility demand of the structure, provided that the displacements and rotations of the foundation are considered (Pecker and Chatzigogos, 2010). Recent research suggests that these displacements may only be minor and what is required is a shift towards performance-based seismic design that integrates the nonlinear interactions of the soil, foundation and structure (Anastasopoulos, 2010; Gajan et al., 2010; Toh et al., 2011; Paolucci et al., 2011).

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POST EARTHQUAKE INVESTIGATION AND ASSESSMENT OF CFA PILE GROUND IMPROVEMENT

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ABSTRACT

The Press House building in Christchurch, New Zealand is constructed on ground improved by a grid of Continuous Flight Auger (CFA) piles¹ designed to mitigate liquefaction potential. The building performed well in the 22 February 2011 Christchurch Earthquake, undergoing minimal differential settlement.

Following the 22 February 2011 Christchurch Earthquake, a post earthquake investigation was undertaken to assess the condition of the CFA piles, and to enable assessment of expected future performance of the ground improvement. This investigation included locating a number of CFA piles, and using a drilling rig to core down the centre of the piles to assess condition and the degree of cracking in the concrete. A number of difficulties were encountered throughout the investigation, including equipment access, determining the exact location of the CFA piles, and determining the appropriate methods of testing to assess pile condition.

Four CFA piles were ultimately located and tested. Based on the assessed condition of the piles, a review of the original ground improvement design methodology indicates that the ground improvement is expected to perform well in a future severe earthquake, meeting the original design criteria.

1. INTRODUCTION

The Press House building in Christchurch, New Zealand, is a seven storey office building constructed on ground improved by a grid of CFA piles, designed to mitigate the potential for liquefaction.

A post earthquake assessment of the CFA piles was undertaken following the 22 February 2011 Christchurch earthquake, to assess the condition and likely future performance of the CFA piles. This paper covers the site investigation methodology including locating and testing of CFA piles, interpretation of the test results, and challenges encountered throughout the site investigation. Murahidy et al. (2012) covers the project as a whole, including initial design and post earthquake assessment.

2. PROJECT BACKGROUND

2.1 SITE CONDITIONS

The site investigation carried out prior to building construction indicated that the natural geological conditions underlying the site comprise alluvial gravel, sand and silt deposits. The water table typically fluctuates between 1.0m and 2.5m below ground level. Liquefaction analysis undertaken by the designer, based on the site investigation data, indicated lenses of material with a cumulative thickness of 3.0m to 5.0m, located between 2.0m and 9.0m depth, were likely to liquefy in an ultimate limit state (ULS) design earthquake.

2.2 CFA PILE GROUND IMPROVEMENT

Due to the presence of liquefiable material underlying the site, the designer proposed CFA pile ground improvement as an option to mitigate the potential for liquefaction to occur during a severe seismic event, and to improve the bearing capacity of the soil. This allowed the building to be constructed on shallow foundations comprising a stiff slab and ground beams.

CFA pile ground improvement, in this case, consists of 210 No. 600mm diameter lightly reinforced concrete piles constructed in a 2.8m triangular grid pattern over the entire building footprint, extending to 9.0m below ground level. This system acts to mitigate liquefaction by reinforcing the ground. As the piles are significantly stiffer than the

¹ CFA piles are constructed by using a machine driven auger with a continuous flight to bore a hole to the required depth, and pump concrete into the hole as the auger is withdrawn. A light reinforcing cage is then inserted into the wet concrete.

surrounding soil, earthquake induced cyclic ground stresses are concentrated in the piles. This results in a reduction in cyclic stresses within the soil, and a corresponding reduction in the potential for liquefaction (Murahidy et al, 2012). Figure 1 shows the CFA pile layout including an outline of the building and shallow foundations.

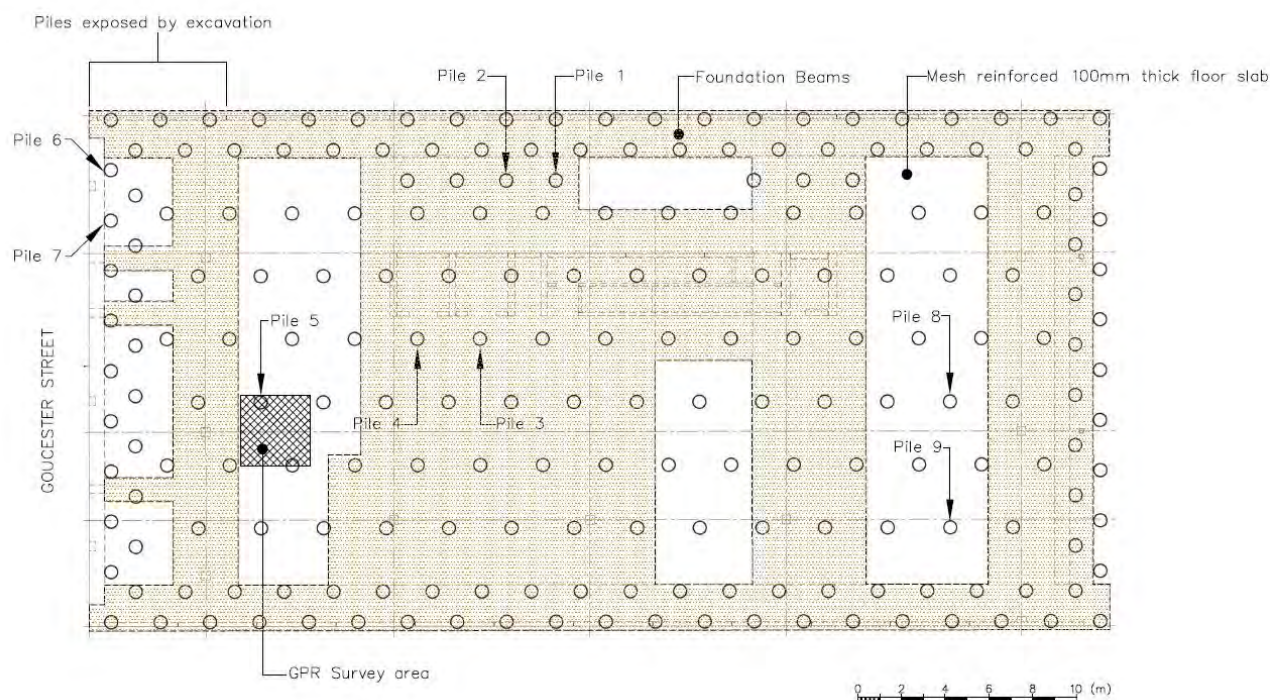


Figure 1 – CFA pile and foundation beam layout

2.3 PERFORMANCE IN CANTERBURY EARTHQUAKE SEQUENCE

The Canterbury earthquake sequence began with a magnitude M_w 7.1 event centered approximately 40km west of Christchurch, on 04 September 2010. This resulted in some land and structure damage throughout Christchurch but no damage was observed at the site itself.

The 22 February 2011 Christchurch earthquake was a magnitude M_w 6.2 event (a PGA of 0.48g was recorded near the site) which resulted in widespread liquefaction throughout much of Christchurch. The nearest surface evidence of liquefaction was observed approximately 150m east of the site. This event has been followed by a number of smaller aftershocks, as well as two significant aftershocks on 13 June 2011 (M_w 6.0 and M_w 5.7). These events resulted in some liquefaction throughout Christchurch but again no evidence of liquefaction was observed on the site itself.

The building performed well in the 22 February 2011 event. A post-earthquake level survey indicated only 30mm of differential settlement, which is within the construction tolerances of the building. It is important to note that no pre earthquake level survey of the building was undertaken, so this differential settlement may have occurred during construction of the building. No bearing capacity failures or significant cracking of the shallow foundation elements (reinforced concrete foundation beams and floor slab) was observed. No surface expression of liquefaction was observed on the site, however, liquefaction may have occurred at depth without presenting evidence at the surface.

Following 22 February 2011 earthquake event, the building owner engaged the author to undertake a post earthquake assessment of the CFA pile ground improvement and building foundations, and to assess the likely performance of the CFA piles in a future severe earthquake event. A number of CFA piles were located and tested to assess the degree of cracking (if any) sustained as a result of the earthquake events.

3. SITE INVESTIGATION METHODOLOGY

3.1 INTRODUCTION

The following methods were considered for investigating the degree of cracking of the CFA piles:

- Low strain integrity testing,
- Concrete coring,

- Water testing, and,
- Video camera inspection.

The methods which were carried out and the reasons behind this are discussed in the following sections 3.2 to 3.8.

3.2 LOW STRAIN INTEGRITY TESTING

Exposing the tops of a number of CFA piles and carrying out low strain integrity testing of the piles was initially considered, in order to assess the degree of cracking of the piles. A number of piles beneath the perimeter wall of the building were located by excavating adjacent to the perimeter wall and exposing the piles. The space between the piles and foundation beam above was approximately 200mm, which was insufficient to allow low strain integrity testing to be carried out.

Piles with better access were subsequently exposed (refer Section 3.5), however at this time the only set of equipment in Christchurch had been sent away for calibration, and was to be out of service for several weeks. Time pressure to complete the project meant that this approach had to be abandoned. It is questionable as to whether or not low strain integrity testing would have provided useful information on the degree of cracking sustained by the piles. Regardless, additional testing using other methods would still have been required in order to give confidence in any results.

3.3 LOCATING PILES FROM CONSTRUCTION PLANS

It was attempted to locate CFA piles by marking out pile locations using architectural plans of the building overlaid on a plan of the CFA pile layout (refer Figure 1). Four piles beneath foundation beams were selected on the basis that these piles would have been more heavily loaded during the earthquake. These four locations were also approved by the structural engineer to ensure that drilling through the foundation beam would not compromise structural integrity. An EP10 rotary drill rig was brought into the building to core through the concrete floor and foundation beams, and down the centre of the pile. Drilling was attempted in two locations (pile 3 and pile 4, Figure 1), as the drill rig was unable to access the remaining two piles (pile 1 and pile 2, Figure 1) due to insufficient headroom for drill rig access. Drilling in these locations failed to locate CFA piles.

Discussions with the contractor who constructed the CFA piles indicated that pile locations may have varied up to 300mm from the design pile layout. In addition, it is possible that errors were also introduced in overlying the architectural plans over the CFA pile layout. It is important to note that, because the piles are designed to improve the soil block as a whole, some variation in pile location is not an issue from a performance perspective. However, this did add a significant challenge to the site investigation.

3.4 GROUND PENETRATING RADAR

In a third attempt to locate CFA piles for testing, 400MHz GPR (ground penetrating radar) equipment was used to survey an area of floor slab (refer to Figure 1). This survey identified the edges of foundation beams and identified one possible pile location (pile 5, Figure 1), however steel mesh in the floor slab made identifying potential pile locations difficult. Piles beneath a concrete floor slab rather than beneath foundation beams were targeted for this survey as the available 400MHz GPR equipment did not have the capability to locate piles beneath the concrete foundation beams. Further drilling at the potential pile location identified by the GPR survey did not successfully locate a pile.

3.5 FLOOR SLAB REMOVAL

Four CFA piles were successfully located by removing sections of the concrete floor slab in areas based on the pile setout plan, and excavating the underlying hardfill to expose the pile tops (piles 6, 7, 8, and 9, Figure 1). The four piles located were cored and tested as discussed in the following sections 3.6 to 3.8. Figure 2 shows one of the exposed piles.

3.6 CONCRETE CORING

Machine drilling down the centre of selected CFA piles was used as the main approach to investigate the condition and degree of cracking of the piles. This approach allowed pile cracking to be examined by inspection of the drill core, video camera inspection and water testing.

Four CFA piles (piles 6, 7, 8, and 9) were cored to a depth of 6.0m below the pile top (piles are 9.0m long). The coring depth was limited in order to minimize the risk of penetrating the bottom or side of the pile, which would have rendered the pile unsuitable for carrying out water testing (discussed in section 3.7). A man-portable rotary drill rig equipped with NQ triple tube core barrel and a diamond drill bit were used for this work. Inspection of the recovered drill core showed two types of cracks. A number of cracks spaced at intervals approximately corresponding to the length of the drill run were worn partially smooth, where the drill rig operator had purposely snapped the core in order to retrieve it at the end of the drill run. Other cracks were not worn, and the two sections of core fitted together precisely. Figure 3

shows typical core cracks. It is assessed that these cracks may have been induced by impacts from the drilling process (i.e. when retrieving the core from the hole) or may have been already present in the pile, as a result of earthquake loading, shrinkage cracking during concrete curing, or a combination of both. The investigations discussed in the following sections were carried out in order to further establish the degree and cause of cracking within the CFA piles.



Figure 2. Exposed pile



Figure 3. Typical core cracks

3.7 WATER TESTING

Following coring of four CFA piles, water testing was undertaken to assess the degree of cracking in the piles by measuring the amount of water leaking in (and out) of the pile, using rising and falling head tests. This was done by removing drilling waste and water from the drill hole, and using an electronic dip meter to take water level measurements at recorded times as groundwater leaked into the pile. These measurements yielded a flow rate, which reduced exponentially with time until the water level in the pile was equal to the groundwater level (approximately 1.5m below ground level). This measurement was then confirmed by filling the drill hole to the top of the pile and then establishing the rate at which the water level returned to groundwater level. Maximum flow rates of 3.0mL/hour to 50.0mL/hour were recorded in three of the piles. A maximum flow rate of approximately 3000mL/hour was recorded in the fourth pile. The project team considered that further investigation of this pile was warranted to assess whether there was significant cracking or damage to the pile.

3.8 VIDEO CAMERA INSPECTION

A down-hole video camera (the type typically used for pipe inspections) was used to further investigate the source of the water leaking into the piles. No obvious cracks or leaks were revealed by the camera inspection in three of the piles. In the pile with the high leakage rate, the source of the leak was discovered to be a small jet of water at approximately 5.5m depth. Figure 4 shows a screenshot from the video camera footage of the leak. The discovery of this jet of water was unexpected as it had previously been thought that water was seeping into the pile through cracks rather than through a single point. It is thought that the presence of the water jet indicates a local intrusion (i.e. soil inclusion from CFA construction) or local voids which were present prior to the earthquake, rather than severe earthquake induced cracking. The removal of the concrete core allowed the groundwater pressure to force a localized jet of water through the defect and into the core hole. It is worth noting that air voids were present in the core recovered from this point in the pile.



Figure 4. Screenshot from video camera showing location of the jet of water

4. INVESTIGATION RESULTS

The general results of the testing discussed in the preceding sections are summarised as follows:

- Coring indicated that cracks may be present within the piles,
- Water testing confirmed that cracks must be present in the piles, evidenced by the leakage of groundwater into the drill hole, and,
- Cracks are likely to be minor i.e. hairline cracks, based on the low rate of leakage of water and the lack of obvious cracking found by camera inspection.

Discussion with the project structural engineer (Wilson, pers. comm. 2011) indicated/confirmed the following:

- Cracking of piles is likely to be a result of shrinkage cracking or minor flexural (i.e. earthquake induced) cracking,
- Cracking which would indicate severe damage or loss of pile strength/stiffness would most likely result in severe leakage of groundwater into the pile,
- Corrosion risk resulting from intrusion of groundwater into the piles through cracking is insignificant, based on a neutral pH environment and slow groundwater flowrate, and,
- The reinforcement layout of the piles has sufficient flexural and shear capacity to meet the original design demands, with zero axial load, even if the piles have experienced some minor flexural cracking as described above.
- The effect of low to moderate axial load (i.e. for piles located beneath foundation beams) is to increase the flexural and shear capacities of the piles.

Based on the above, the original design was reviewed with respect to the results of the site investigation. Discussion on this review and the outcome is covered in Murahidy et al. (2012). The outcome of the post earthquake assessment indicated that the improved ground could be expected to perform well in another severe earthquake.

5. RECOMMENDATIONS

Based on lessons learnt from the site investigation discussed in this paper, we make the following recommendations for any similar future projects:

- Accurate as built plans showing locations of CFA piles should be compiled for ground improvement projects in such a way that they may be accurately overlaid on architectural or other plans.
- Geophysics should not be discounted as a method for locating CFA piles or similar features. The operator advised that a GPR unit with a more suitable frequency would likely have been successful, if the equipment had been available.
- Post construction level surveys of buildings should be undertaken. This would enable assessment of the amount of differential settlement which is earthquake induced.
- Installation of inspection tubes down the centre of a number of piles would allow easy access to investigate pile condition following an earthquake.

6. CONCLUSIONS

A post earthquake assessment of ground improved by CFA pile reinforcement was undertaken in order to assess the likely future performance of the ground improvement. This involved site investigations aimed at assessing the degree of cracking of the CFA piles.

Initial difficulties in locating and accessing piles were overcome, and four CFA piles were cored to a depth of 6.0m. Inspection of the recovered drill core, water testing to measure groundwater leakage into the piles, and downhole video camera inspection indicated that the piles had undergone only minor cracking. Discussion with the structural engineer indicated that piles would still meet the design shear and flexure demands.

In Section 3.3 it is stated that piles located beneath foundation beams were initially targeted as these are the more heavily loaded areas of the foundation. However, as stated in Section 4, the effect of low to moderate axial load is to increase pile flexural and shear capacity. Hence it may be considered unconservative to test the piles beneath foundation beams due to the beneficial effects of axial load.

A review of the original design to assess the likely future performance of the improved ground, with the CFA piles having undergone some minor cracking indicated that a good level of future performance could be expected, similar to that observed in the 22 February 2011 Christchurch earthquake (Murahidy et al 2012).

The site investigation methodology which was ultimately used on this project proved sufficient to allow the assessment of pile condition.

7. ACKNOWLEDGEMENT

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GEOPHYSICAL SOLUTIONS TO ENGINEERING PROBLEMS

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ABSTRACT

Amongst the geotechnical community there is a mixed appreciation of the benefits of geophysics to geotechnical investigations. It seems many engineers have lost confidence in geophysics usually as a result of: insufficient communication of the capabilities, limitations and precision of the geophysical approach; poorly planned investigations; or lack of data integration. With good communications and planning, geophysics surveys can be used together with borehole and excavation data to provide greater certainty of site characteristics. The cost of a geophysical investigation is often paid for many times over by the reduction in the cost of engineering designs, which may otherwise be over specified to account for a wider range of possible site conditions. This paper presents three case studies of geophysical investigations that were carried to assist with geotechnical assessment and design. The case studies illustrate how geophysical data integrated with drillhole data can provide important geological interpretation that could not be achieved by interpretation from drilling results alone.

1. INTRODUCTION

With the difficulties associated with mapping the subsurface using borehole data alone, geophysical investigations have been known to assist with geological interpretation and geotechnical design (Fitzallen, 2010; Gul, 2011). The requirements of the engineer (i.e. mapping basement, calculating shear wave velocities to assist with dynamic modulus calculations, etc.) determine the type of geophysical technology that is employed to assist with the engineering investigation. Engineers, geologists and engineering geophysicists each interpret the use of geophysics in a different light (Whiteley, 1988; Stapledon, 1988; Fell, 1988), hence there is paramount importance in the clear communication of investigation objectives, site conditions, access issues and geophysical technology limitations.

The following case studies provide indications of the engineering problem and the geophysical solution that was employed in order to achieve the engineer's objective. Technologies included within the case studies are Multichannel Analysis of Surface Waves (MASW), seismic refraction, Electrical Resistivity Imaging (ERI) and Airborne Electromagnetics (AEM).

The case studies illustrate the positive results that can be achieved with good communication, planning and interpretation between engineers and geophysicists.

2. CASE STUDY 1 - MULTICHANNEL ANALYSIS OF SURFACE WAVES (MASW)

A field case study is presented from a Multichannel Analysis of Surface Waves (MASW) investigation carried out at a proposed port infrastructure site in Western Australia. The objective of the geophysical investigation was to determine the 3D shear-wave velocity structure of the subsurface which, when combined with material density, could assist engineers with dynamic shear modulus calculations.

The geophysical technology that was deemed the most suitable was MASW. MASW is a seismic method that uses the dispersion of seismic surface-waves to evaluate variation in the subsurface shear-wave velocity as function of depth (Park et al., 1999). MASW is a non-destructive technology, which involves using a seismic source (e.g. sledgehammer, drop-weight) to produce seismic surface waves and measuring the speed at which these waves travel across the surface using an array of geophones (or vibration sensors). The dispersion curves of the seismic shot records are then analysed and inverted to calculate the shear-wave velocity (V_s) as a function of depth. It is possible to then correlate calculated shear-wave velocities (V_s) with borehole data (Table 1).

The field set-up consisted of six parallel MASW lines in a north-south orientation, approximately 120 m long with 5 m station spacing and 3 m from the source to the nearest receiver. The MASW lines were spaced approximately 10 m apart. From the results it was possible to produce 2D shear-wave velocity profiles to a depth of 20 m and illustrate a simple three layer model. An example of two MASW sections with their nearest boreholes is shown in Figure 1.

Shear-Wave Velocity, V_s (m/s)	Lithology
<50	peat
50-150	soft clay/silt
150-250	stiff clay or unconsolidated silt/sand
250-500	stiff silt/sand or gravel
500-1000	cemented/compacted sand/gravel or fractured/weathered rock
>1000	crystalline rock

Table 1: General Table of Comparison for V_s and Lithology

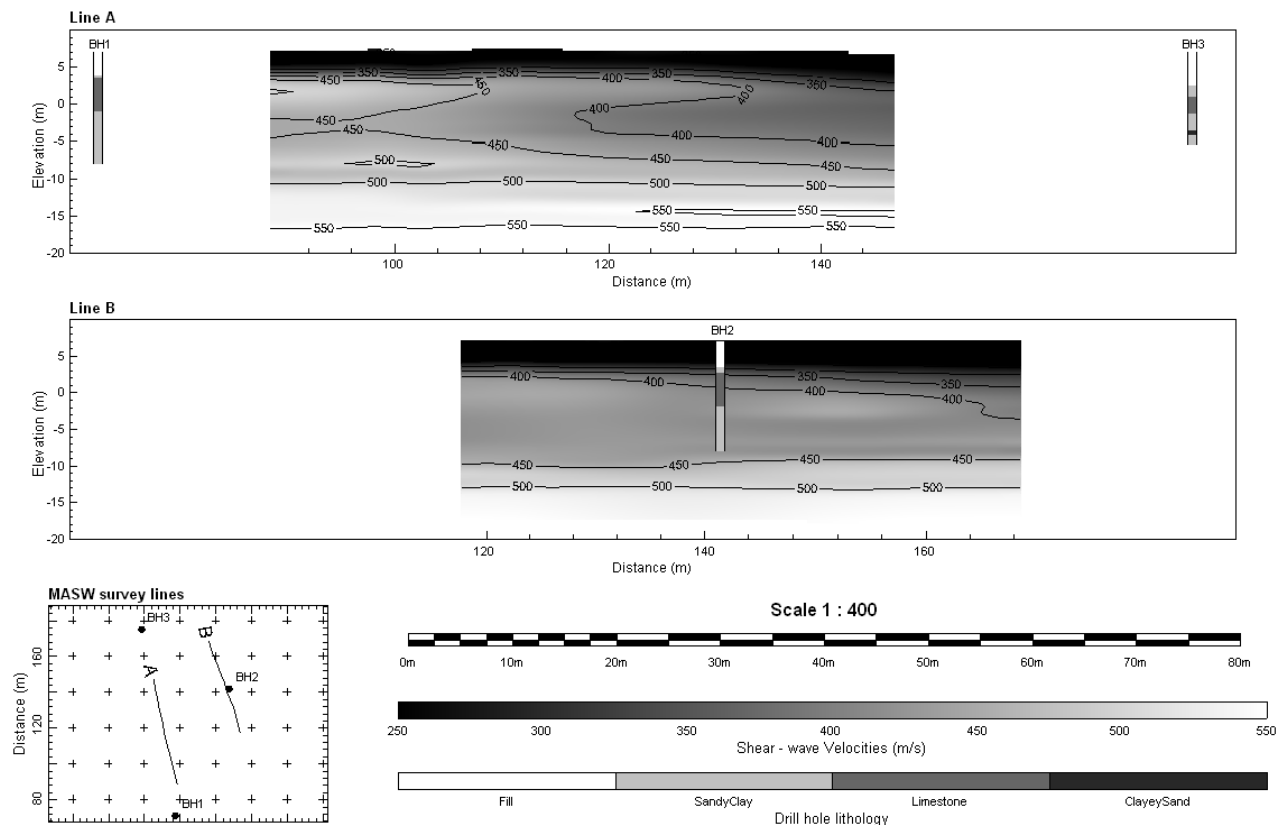


Figure 1: 2D MASW Sections with associated boreholes

Based on the MASW profiles and the borehole data a 3-layer model of the subsurface was interpreted (Table 2).

Layer	Shear-Wave Velocity, V_s (m/s)	Lithology
1	< 350	Dry, fine to coarse unconsolidated sand and gravels
2	350 – 500	Sandy clay and soft, weathered limestone
3	500+	Cemented/compacted sand/gravel or fractured/weathered rock

Table 2: Summary of MASW results with interpreted lithology

Layer 1 compares directly to the fill observed in the drillhole logs matching V_s of 250 – 275 m/s. The drillhole logs observe this lithology as being dry, fine to coarse sand and gravels. Layer 2 exhibited regions of velocity inversions (high velocity layer overlying a lower velocity layer); the higher velocity regions of Layer 2 were interpreted to represent regions of extremely weathered limestone. Regions in the northern end of the site illustrated thicker cover of softer materials (low V_s).

The geophysics was able to delineate a “step” feature in the geology between boreholes, which was observed across a few of the MASW lines (shown in Line A – Figure 1); this assisted in providing a 3D interpretation of the geology. The step feature in the basement would otherwise have been lost by geological extrapolation between boreholes alone.

3. CASE STUDY 2 – MAGNETICS, SEISMIC REFRACTION AND ELECTRICAL RESISTIVITY IMAGING (ERI)

A geophysical investigation was commissioned to assist with geotechnical assessments regarding an extension of an existing dam site in Western Australia. The objectives of the investigation were to map the depth to bedrock and variations in bedrock hardness and composition. Previous drilling had shown the presence of a dolerite dyke, so another objective was to map possible dolerite dykes and their orientation; the knowledge of their location is important as the presence of dolerite dykes could introduce leakage or dam failure (decomposed dolerite dykes would be less permeable than the surrounding granite). The geophysical technologies that were used for the investigation were ground magnetics, seismic refraction and Electrical Resistivity Imaging (ERI).

Magnetics (ground magnetics) measures the natural magnetic field of the earth, which can be locally influenced by magnetic materials such as hematite (Stanley et al. 1990). At this investigation site magnetics was used to define the location of the dolerite dyke due to the presence of magnetic minerals in the dolerite. Magnetics data were acquired in a grid of lines at 25 m spacing over the survey area.

To provide information regarding the stiffness of the rock and the depth of interest (25+ m), seismic refraction was deemed the most suitable of the seismic technologies. Seismic refraction (land) consists of a linear array of geophones, coupled to the ground, with a series of shots (source e.g. sledgehammer, drop weight, explosives etc.) initiated through the array and from offset (off-end) positions (Redpath 1973). The recorded first arrival of the seismic signal provides information of how long the seismic wave takes to reach the geophone after leaving the source and travelling through the geology; this makes it possible to calculate the approximate P-wave velocity (V_p) of the earth material. V_p correlates well with the material strength (see for example Caterpillar rippability charts (Caterpillar, 2008)). Two parallel seismic refraction lines were recorded (290 m and 332 m in length). Geophones were spaced at 5 m with approximately 9 shots per spread (including offset shots). The seismic source was explosives (75 g for in-spread shots, 185 g boosters for offset shots).

ERI is a non-destructive technology that involves injecting a current in the ground and measuring the resulting voltages through a series of electrodes coupled with the ground (Samouelian et al. 2005). ERI is strongly correlated with porosity and water content; this is particularly useful in mapping groundwater, cavities, clay layers or geological strata and structure. ERI was chosen as a geophysical technology for this investigation to assist with delineating weathered clay material (electrically conductive) and granitic bedrock (electrically resistive). ERI data was acquired in a grid of eight lines over the investigation area, with an electrode spacing of 5 m. A non-standard array of a combined dipole-dipole and Schlumberger array were used to optimise vertical and lateral resolution.

The results from the Total Magnetic Intensity (TMI) showed regions of high TMI amplitude (59650 – 59700 nT), which were interpreted to indicate the presence of near-surface dolerites; areas of moderate TMI amplitude (59300 to 59500 nT), which were interpreted to indicate the presence of deeper dolerite and areas of low TMI amplitude (59700 – 59200 nT) which were interpreted to be areas of granitic bedrock. Drill holes that intersected dolerite were all located in regions of moderate to high TMI (> 59300 nT).

The two seismic refraction lines illustrated a consistent velocity structure: an upper low velocity layer (~ 350 m/s) interpreted as soft, unconsolidated sediments; an intermediate layer (500 – 1500 m/s) interpreted to be consolidated sediments or potentially weathered bedrock (in areas of larger V_p) and a lower basement unit of high velocity (> 5000 m/s) interpreted as competent bedrock. The drilling showed good correlation with seismic data (Figure 2). No obvious correlation could be seen between lateral changes in V_p and large magnetic units. This indicated that the velocity of the dolerite unit did not differ significantly from the surrounding granitic host material.

The general electrical structure of the investigation area showed an intermittent resistive surface layer ($> 1000 \Omega m$) interpreted as resistive surficial sediments, a moderately conductive second layer ($< 500 \Omega m$) coming to surface where the surface layer disappears - this was interpreted as more conductive sediments and weathered bedrock - and a highly resistive basement layer ($> 2000 \Omega m$) interpreted to be bedrock. The results from the drillhole data indicate fresh bedrock was closely correlated with the ERI results (Figure 2). There was also a close correlation between the ERI and seismic results (Figure 2) – the 2500 Ohm.m contour and 2000 m/s contour boundaries, which could be interpreted as the top of the bedrock boundary, show a similar trend across the section.

Overall the ERI and refraction provided continuous profiles of the depth of bedrock and the magnetics showed the important locations of the dykes, neither of which could be achieved by drilling alone.

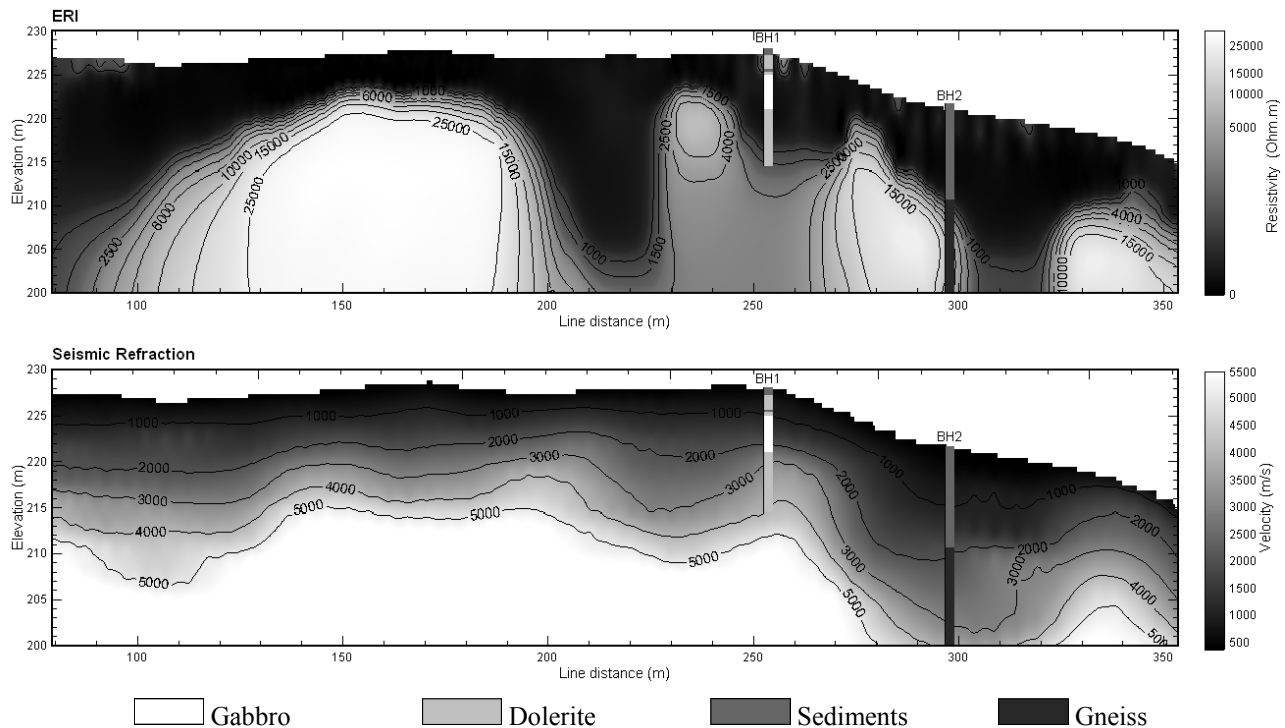


Figure 2: 2D ERI and Seismic Refraction Sections with associated boreholes

4. CASE STUDY 3 – AIRBORNE ELECTROMAGNETICS (AEM)

Increasingly airborne electromagnetics (AEM) is being used for a wide range of geotechnical purposes, including pollution mapping, geotechnical assessment on road and freeway alignments, assessment for major infrastructure such as pipelines, railways and tunnels, bathymetry and depth to bedrock (Christensen & Reid, 2012). AEM surveys can reduce planning risk by predicting likely near-surface ground conditions ahead of construction. In the case of oil and gas pipelines, these are often buried at a depth of a few metres and the cost of construction depends critically on whether the subsurface is composed of soft easily-excavated sediments and weathered bedrock (typically conductive) or hard rock formations (typically resistive) which require much heavier equipment and in some cases blasting.

A field case study is presented from a SkyTEM AEM survey conducted in 2011 along a proposed >500 km gas pipeline alignment in eastern Australia (Christensen & Reid, 2012). Here a gas pipeline was to be buried, requiring excavation to a depth of 3 m. The geology along the alignment was highly variable, ranging from thick sedimentary sequences to exposed crystalline basement. Areas of near-surface crystalline basement are the most problematic for excavation. The AEM survey objective was to identify areas of shallow basement along the profile – areas which were likely to require specialized excavation machinery and/or blasting (Christensen & Reid, 2012).

Figure 3 and Figure 4 display two example model sections from the survey. Figure 3 shows fairly low resistivities suggesting softer materials, whereas Figure 4 displays higher resistivities indicating the possible presence of harder rock. A statistical analysis of a set of equivalence models (i.e. a set of models that all match the data) was applied to assess the likelihood of encountering sediment or hard rock – this is shown as a black line in Figure 3 and Figure 4 (Christensen & Reid, 2012). In Figure 3 the estimated depth to hard rock is large, showing two minor zones where there is high resistivity near to the surface. The zone near 461700 m is surficial and may represent dry clay-free sediments rather than genuine shallow bedrock, however the zone near 465000 m extends to depth and may represent a genuine bedrock high (Christensen & Reid, 2012). The latter zone represents the greatest excavation risk on this section of the gas pipe alignment. Figure 4 shows a 9 km section over an area of thin unconsolidated sediments over crystalline bedrock, where drilling has shown the depth to bedrock is very variable, ranging from ~1 m to >10 m (Christensen & Reid, 2012). It would be difficult to predict bedrock depth along the parts of this section where there is no drillhole data. This is where AEM provided the information to fill the gaps. The bedrock interface determined from the statistical approach (as shown in black) is at >3 m depth along the entire section of line, and is consistent with all drillholes except the one drillhole to the far right of Figure 4. This mismatch may be due to weathered or fractured, and thus conductive, bedrock here (Christensen & Reid, 2012). The bedrock depth estimated from AEM shows very good agreement with the

drillhole at 402000 m which hit crystalline bedrock, and generally shows resistivities of $<60 \Omega\text{m}$ associated with unconsolidated sediments (Christensen & Reid, 2012).

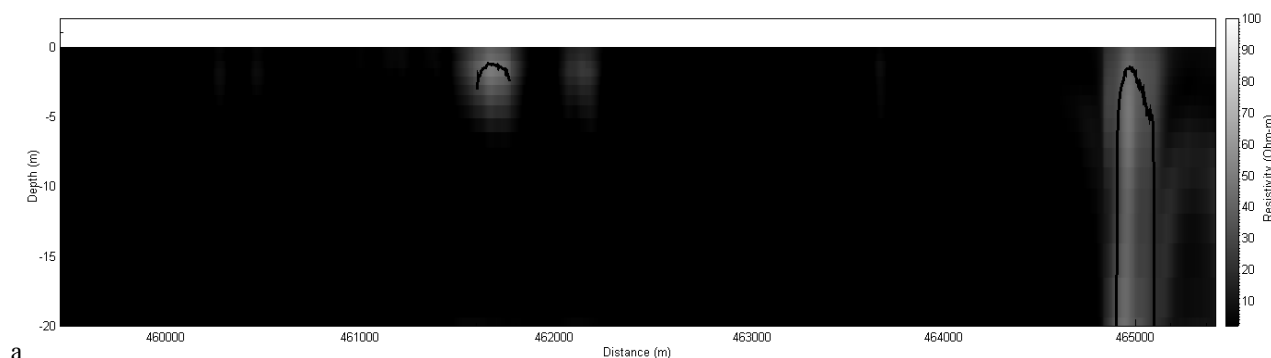


Figure 3: Resistivity cross-section along a section of the proposed pipeline alignment. Depth to bedrock estimated using the statistical approach is shown as a black line. This ~6 km section of line is generally very conductive and is likely to present few obstacles to excavation. (Christensen & Reid, 2012, Figure 2)

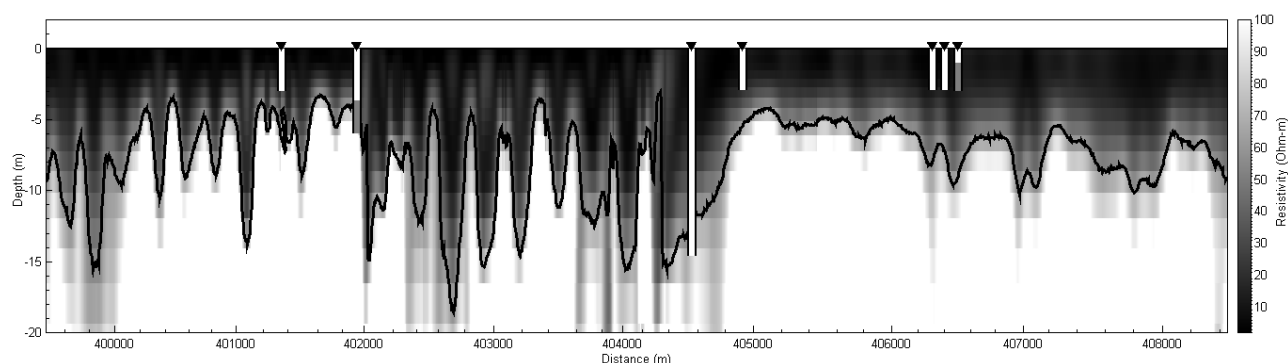


Figure 4: Resistivity cross-section along a section of the proposed pipeline alignment. Depth to bedrock estimated using the statistical approach is shown as a black line. Drillhole lithology logs show unconsolidated sediments as white and crystalline bedrock as grey. This ~9 km section shows the AEM-derived depth to bedrock is generally consistent with drilling results and suggests material above 3 m depth will generally be able to be excavated. (Christensen & Reid, 2012, Figure 3).

The AEM method successfully delineated the possible occurrence of hard formations in this case study and shows reasonable correlation with available drillholes (Christensen & Reid, 2012). AEM was rapid and cost effective and provided a very good early indication of areas which may be difficult to excavate over extensive areas where ground access was difficult.

5. CONCLUSIONS

Case studies have been presented in this paper that demonstrate that geophysics is a tool that can assist geotechnical engineering investigations. Case Study 1 assisted in providing a 3D interpretation of site geology which would otherwise have been lost by geological extrapolation between boreholes alone. Case Study 2 illustrated that the use of multiple geophysical technologies integrated with drilling results assisted with providing interpreted basement boundaries and interpreted locations of magnetic dykes, while Case Study 3 showed that airborne geophysics correlated with drilling results provided a rapid and cost effective interpretation of areas that may be difficult to excavate and/or access.

Geophysics is a tool that assists with engineering design and adds value to the final design product. Geophysics is not a tool that is independent of geotechnical and geological assessment; they are tools to be used together, to complement each other. Geophysics can reduce the costs of drilling programs by targeting drill locations and also assist with “filling in the blanks” between existing drillhole data. Where drilling access and ground disturbance is a problem, geophysics can offer a much more cost effective solution.

6. ACKNOWLEDGEMENTS

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NUMERICAL MODELLING OF LATERAL DISPLACEMENT OF THE GROUND DUE TO DEEP SOIL MIXING COLUMN INSTALLATION AND ITS EFFECTS IN A SLOPE STABILIZATION SCENARIO

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ABSTRACT

A 2D numerical model which incorporates lateral expansion induced by deep soil mixing column installation has been developed to examine its influence on the mechanisms for mobilizing resistance to lateral loading in plan view. The improvements to the soil surrounding the DSM columns and stress-transfer mechanisms are examined through ultimate pressure, displacement, and soil arching. Results show an increased capacity for pressure (total horizontal stress) near the pile face in the undrained condition and a reduction in horizontal displacement between the columns in the drained condition. In both drained and undrained conditions, the ultimate lateral pressure exerted on the model boundaries increased after implementation of the lateral expansion induced by DSM column installation.

1. INTRODUCTION

1.1. DEEP SOIL MIXING

Deep soil mixing (DSM) is a soil improvement technology that has several applications, one of which is to stabilize landslides. This is accomplished with a series of stabilized soil columns in which a binder, such as cement or cement/lime, is added and mechanically mixed into the existing soil. Binders can be introduced in slurry form (wet mixing) or in dry form (dry mixing). As the cement slurry or powder is injected into the ground under pressure, it causes local deformation of the surrounding soil. Quantifying the deformation is important to determine the extent of soil property changes around the columns, as well as possible effects on adjacent structures.

Chai et al (2005) utilized the theory of cavity expansion in an infinite soil mass (Vesic, 1972) to develop a series of equations that predict the amount of deformation in the soil surrounding DSM columns. The equations were validated against field and laboratory tests. This theory was then improved to consider the finite length of the installed column. In this study, the DSM columns are examined in plan view and the column length is not considered. Therefore, for simplification, the original method for quantifying lateral displacement has been utilized.

1.2. EFFECTS ON DSM COLUMN PERFORMANCE

(Guetif et al., 2007) reported from field observations that some of the mechanical characteristics (i.e. Young's modulus and undrained shear strength) of in situ soft soil surrounding stone columns was much higher than those measured before treatment. Soil confinement and changes in pressure between the columns due to soil deformation during installation may also influence the performance of DSM columns. By modelling the radial deformation due to column installation followed by lateral displacement in plan view using Plaxis 2D finite element software, this study examines the effects this displacement has on the ability of the DSM columns to mobilize resistance to lateral loading.

2. SOIL CHARACTERISTICS

Slope instability is a common issue in the Northland area of New Zealand and deep soil mixing columns are often used to stabilize slopes in this area. One of the most problematic soil types is Northland Allochthon soil, which is a soft to stiff, plastic, light coloured residual clay. The soil characteristics used in this study are shown in Table 1.

Table 1: Soil Properties

Soil Parameter	Units	Material		
		Soil (Undrained)	Soil (Drained)	DSM Columns (Drained)
E_{ref}	kN/m^2	22500	22500	1080000
v		0.25	0.25	0.2
γ (unsat)	kN/m^3	20	20	20
γ (sat)	kN/m^3	20	20	20
ϕ	°	0	24.6	32
ψ	°	0	2	2
c_{ref}	kN/m^2	60	0.1	667

3. COMPUTATION OF LATERAL DISPLACEMENT DUE TO COLUMN INSTALLATION

The amount of displacement occurring in the soil surrounding DSM columns can be computed through cavity expansion theory (Figure 1).

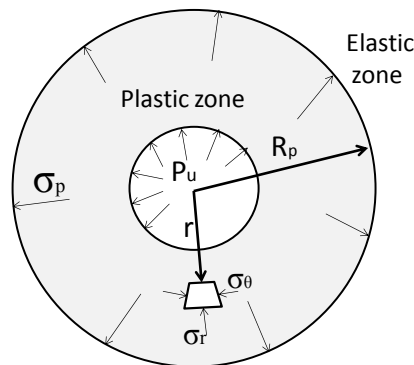


Figure 1: Expansion of a cylindrical cavity

In Figure 1, P_u is the cavity pressure, R_u is the radius of the cavity, R_p is the radius of the plastic zone, σ_p is the pressure at the interface of the plastic zone and elastic zone, and r is the distance from the column centre to the point where displacement is being measured.

In the case of DSM, the radius of the cavity (R_u) is not necessarily the same as the radius of the DSM column. Chai et al (2005) back estimated the cavity radius based on measured ground displacement for several case studies for a 1 m diameter column. The results of R_{u0} for several types of DSM with 1 m diameter are shown in Table 2.

Mixing Method	Mass Injected (m^3/m)	Injection Pressure (kPa)	R_{u0} (m) ($E_0 = 2,250$ kPa)	% of True Column Diameter
Slurry Double Mixing	0.146	100-200	0.21	42%
Dry Jet Mixing	0.036	500-700	0.46	92%
Wet Jet Mixing	0.146	20,000	0.58	116%

Table 2: Back estimated R_{u0} based on mass injected and injection pressure (Chai et al., 2005)

For TurboJET DSM in Northland Allochthon soil, the injection pressure is 15,000 kPa and mass injected is $0.1 m^3/m$. Therefore, the R_{u0} was estimated to be equal to 100% of the radius of the column (0.3 m). However, the results in Table 3 are for soil with a Young's Modulus (E_0) of 2,500 kPa, and thus must be recomputed for soil with a Young's Modulus of 22,500 kPa. Chai et al (2005) suggest the following equation to derive a new R_u for a different soil stiffness.

$$(1) R_u = R_{u0} \left(\frac{E_0}{E} \right)^{1/3}$$

Using this equation, TurboJET columns in Northland Allochthon soil with an E of 22,500 kPa and a radius of 0.3 m have an estimated R_u of 0.14 m (Table 3).

Mixing Method	Mass Injected (m ³ /m)	Injection Pressure (kPa)	R _{u0} (m) (E ₀ = 2,500 kPa)	% of True Column Diameter	R _u (m) (E = 22,500 kPa)
Wet Jet Mixing- TurboJET	0.1	15,000	0.3	100%	0.1442
Wet Jet Mixing- Chai et al (2005)	0.146	20,000	0.58	116%	

Table 3: Estimated R_{u0} for TurboJET in Allochthon soil.

Vesic, 1972 describes an analytical solution for the displacement (δ) at a distance r from the centre of the cavity. Ultimately, from this analysis the displacement in the elastic zone due to DSM column installation is:

$$(2) \delta = \frac{R_p}{r} \delta_p \quad (r > R_p)$$

Results for a case study for 1 m diameter DSM columns in Taipei clay with E of 14,500 kPa (Lin and Lin, 2006, Chai et al., 2007) were reproduced for validation purposes both in a spreadsheet (analytical solution) and in Plaxis 2D finite element software. The R_u value for the columns was 0.35, cohesion, c was 47 kPa with Poisson's ratio, ν of 0.5. The displacement at the edge of a column was computed in the spreadsheet and applied in the numerical model. Displacement was measured at several distances away from the columns and the results compared well to the analytical solution.

4. VALIDATION OF 2D MODELLING OF LATERALLY LOADED PILES IN PLAN VIEW

In a previous paper (Tatarniuk and Bowman, 2012b) a numerical model examining DSM columns under lateral loading in plan view was developed in Plaxis 2D and validated by comparing the results to an analytical solution of a pile pushed laterally into cohesive soil (Randolph and Houlsby, 1984), with good agreement. The pile was modelled using a beam element. Material properties for the model were derived from Chen and Martin (2002) with slight alterations to the properties of the steel pile. Both a rough interface and smooth interface were examined. The results of this model (normalized lateral pressure versus displacement) were within 1% of the theoretical values based on classic plasticity. This affirms the validity of Plaxis 2D for modelling a laterally loaded pile in plan view.

4.1. EFFECTS OF LATERAL DISPLACEMENT IN A SLOPE STABILIZATION SCENARIO

To assess the effects of the lateral displacement occurring in the soil surrounding DSM columns, a model was developed from that in Tatarniuk and Bowman (2012b) depicting two rows of columns (in a zig-zag pattern) in plan view. The additional row (one column) was added in order to get a better idea on the effects of increased confinement of the soil between the columns. The examination of DSM columns in plan view aids in the understanding of the mechanics of the mobilization of resistance from passive DSM column groups. Note that this is an ultimate state analysis where soil movement eventually results in failure due to soil plastic flow. Failure mechanisms of DSM columns in Northland Allochthon soil are discussed in Tatarniuk and Bowman (2012a). The DSM columns are loaded passively, and failure of the columns themselves is not examined. Figure 2 shows the model configuration and the location of the points examined in the analyses. The top and bottom boundaries of the model are fixed in the vertical direction, and free to move horizontally. Lateral stresses are applied to the left and right boundaries of the model in order to reproduce at first in-situ confining stresses, and then lateral loading.

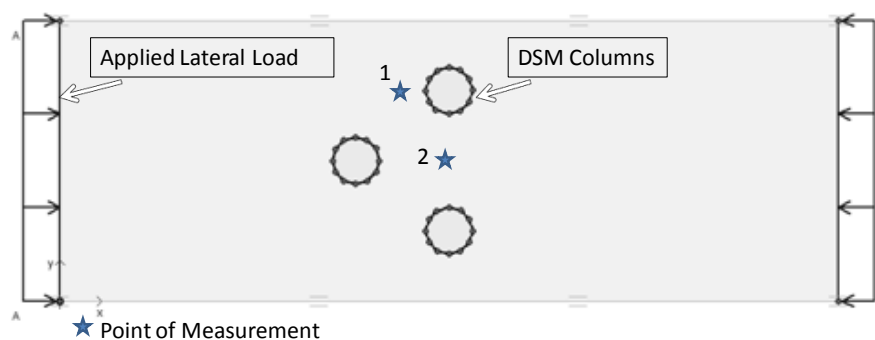


Figure 2: Model configuration for analysis of lateral displacement affects showing points of measurement.

In stage one, the model was brought to equilibrium with an in-situ confining stress at the model boundaries of 60 kPa (corresponding to 3 m below ground). In stage 2, the columns were installed, with zero displacement at the column boundaries. Then, a displacement of 0.032 m was applied to the column boundaries in the undrained case. In the drained

case, this displacement was applied in 3 increments (0.01 m, 0.02 m and 0.032 m). Finally, a positive horizontal load was applied in increments of 10 kPa to the boundaries of the model until failure. On the left side of the model, since the initial stress is in the negative direction, the lateral load was first decreased in increments of 10 kPa until zero, and then increased in increments of 10 kPa in the other direction (positive horizontal).

Figure 3 shows results in the undrained condition.

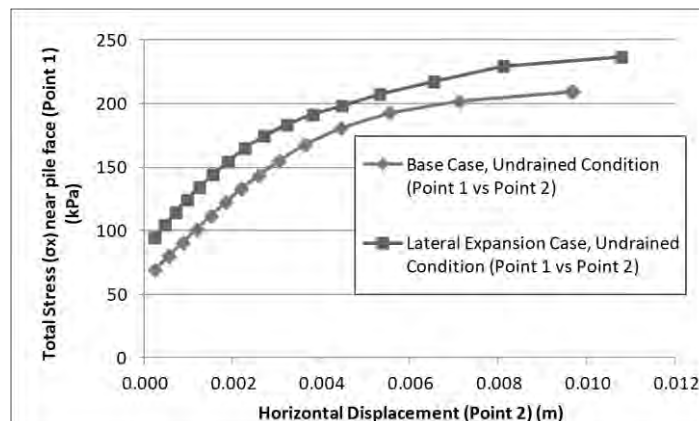


Figure 3: Total horizontal stress near pile face (at Point 1) versus horizontal displacement (at Point 2) under undrained conditions.

Figure 3 depicts the horizontal stress near the column face (for piles this is often taken at the pile-soil interface and termed simply as “pressure”) versus the displacement between the two rightmost piles. This was done in order to evaluate the influence of the lateral displacement in a fashion similar to the well-known pressure-displacement curves used for piles. Chen and Martin (2002) demonstrate that a decrease in pile spacing in the undrained condition results in a steeper pressure versus displacement curve and an increased ultimate pressure on the piles. Figure 3 shows that at the commencement of lateral loading, the total stress near the column face prior to failure has increased for the lateral displacement scenario. This indicates that undrained expansion of the columns has resulted in the ability of the columns to handle an increased lateral load.

Figure 4 and Figure 5 show results in the drained condition.

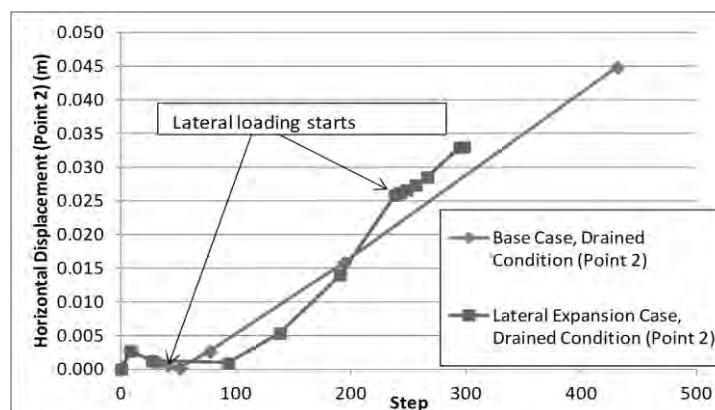


Figure 4: Horizontal displacement versus step at Point 2 under the drained condition.

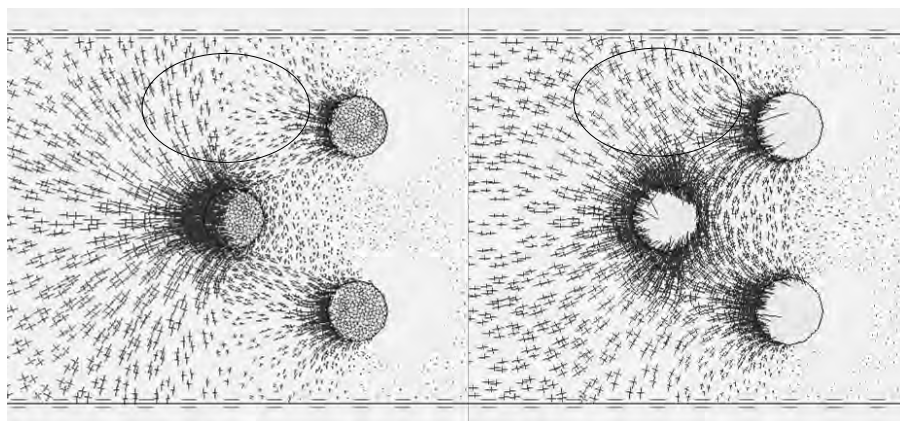


Figure 5: Principal stresses in the drained condition for the base case (left) and column expansion case (right).

Figure 4 shows that upon initiation of lateral loading, the horizontal displacement between the columns is significantly less in the lateral expansion case, likely as a result of an increase in stiffness. The transfer path of load from the soil to piles or DSM columns is physically manifested in the rotation of principle stresses (Chen and Martin, 2002). Figure 5 shows the change in the rotation of principal stresses from the base case to the radial displacement case. The stronger the rotation of the principle stresses, the better the transfer of the load and stabilization of the soil. The area circled in black in each diagram shows how the major principle stresses in the lateral displacement case change more rapidly with a more even distribution from towards the center column to towards the top right column indicating an improved stress transfer mechanism.

The applied lateral load at failure in the drained condition increased from 50 kPa in the base case to 60 kPa for the lateral expansion case. In the undrained condition, it increased from 110 kPa to 130 kPa. This also confirms that the columns can handle more lateral loading due to the stiffening affects cause by DSM column installation.

5. CONCLUSIONS

A 2D numerical model of lateral expansion induced by deep soil mixing has been developed to examine its influence on the mechanisms for mobilizing resistance to lateral loading in plan view. The incorporation of lateral expansion induced by DSM columns into a slope stabilization scenario (i.e. laterally loaded) has resulted in an increased capacity for pressure (total horizontal stress) near the pile face in the undrained condition. It has also resulted in a reduction in horizontal displacement between the columns in the drained conditions. In both drained and undrained conditions, the ultimate lateral pressure exerted on the model boundaries increased after implementation of the lateral expansion induced by DSM column installation.

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GEOLOGICAL DATA CAPTURE METHODS ADOPTED IN THE PORT HILLS, CHRISTCHURCH FOLLOWING 4 SEP 2010 DARFIELD EARTHQUAKE AND AFTERSHOCK SEQUENCE

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ABSTRACT

Following the 4 Sep 2010 Darfield earthquake and ongoing aftershock sequence, significant extents of rockfall and cliff collapse took place across the Port Hills.

To quantify the impact of these large scale events, effective data capture was required to assess the outcomes of future events. Identification of areas at risk of cliff / bluff collapse with subsequent release of material that would affect assets was required to appropriately consider mitigation measures.

Following the 22 Feb event, the Port Hills area was divided into nine Sectors with, Aurecon, Geotech Consulting Ltd, GHD, Opus and URS identified as the Port Hills Geotechnical Group. Each consultant managed 1 to 2 Sectors, on behalf of the Client, Christchurch City Council (CCC).

Following the initial event, fieldwork was on a 'reactionary' basis. With each successive event, methods of data capture have evolved to utilise information in a more effective manner. Data capture has ranged from spreadsheets to the current use of web based GIS database system (ArcGIS), able to be used in the field with live updates.

This paper examines each method employed to capture data, identifying limitations, effective solutions and lessons learned. These methods may be developed as a model for data capture in similar scale events.

1. INTRODUCTION

The centroid of the study area of the Port Hills is located approximately 10km south of Christchurch city centre and covers an area of approximately 65km² encompassing; forestry, parks and both residential and commercial land. Following the 4 Sep 2010 Darfield earthquake (EQ) and ongoing aftershock sequence, significant extents of rockfall and cliff collapse took place across the entire Port Hills.

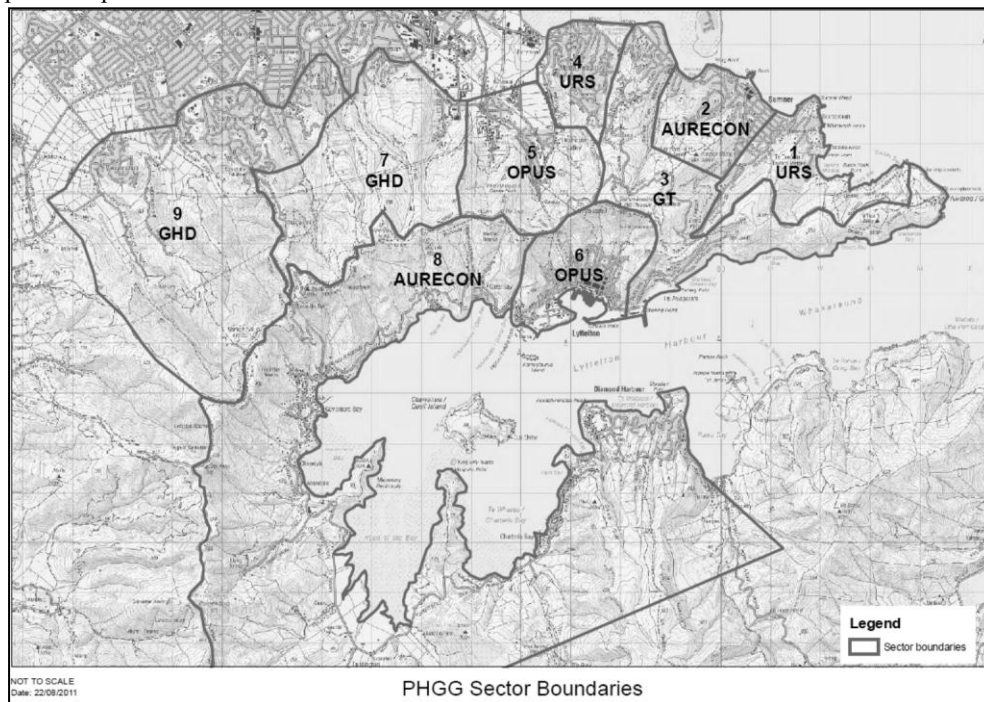


Figure 1. Port Hills Sectors

Following the 22 Feb 2011 aftershock several consultancies (Aurecon, Geotech Consulting Ltd, GHD, Opus and URS), formed the Port Hills Geotech Group (PHGG), splitting the Port Hills in to 9 Sectors and managing 1-2 Sectors each on behalf of the Client; Christchurch City Council (CCC).

Initially, information was gathered on a reactionary basis with an emphasis on identifying worst affected areas at risk of potential further rockfall / cliff collapse. This was focused around lifelines and inhabited areas. Following each event the challenges faced have changed, thus requiring an ability to adapt and alter methods appropriately. A range of data capture methods have been employed to obtain the most accurate and relevant information.

Data captured across the Port Hills has been utilised to put work packages together to mitigate rockfall / cliff collapse, assess the impact on lifelines and key assets, input information into a risk model for the Port Hills, input into detailed design and to calibrate rockfall modelling across the Port Hills.

In this paper, a summary of data capture methods along with limitations and lessons learned is explored. An examination of existing guidelines around data capture (for New Zealand and Australia) following a large scale event is examined.

2. DATA CAPTURE METHODS

The range of data capture methods employed following each event are summarised in Table 1.

2.1 4 SEP 2010

The challenge immediately following the 4 September 2010 Darfield Earthquake across the Port Hills was to ascertain levels of damage as the result of the earthquake. Early inspections were based around gathering information and data capture was generally in the form of photographs and written records in conjunction with GPS. The necessity to rapidly gain information precluded a more methodical approach.

Consultancies developed in-house methods to assess rock fall and cliff collapse with short term 'quick fix' mitigation methods employed in areas deemed to be 'high risk'. At this stage the actual incidences of rockfall was low due to the low peak ground accelerations (PGA's); 0.3g - 0.6g horizontal and 0.3g vertical, associated with the earthquake and consultancies were working in isolation to map rockfall / cliff collapse across the Port Hills.

2.2 22 FEB 2011

A more structured approach to management and data capture across the Port Hills was required following 22 Feb 2011 aftershock. The significantly higher PGA's; 0.5g - 2.1g horizontal and 0.5g - 2.2g vertical, associated with this aftershock greatly increased the incidence of rockfall / cliff collapse across the Port Hills. A set of criteria was developed by PHGG and CCC to better categorise extents of rockfall / cliff collapse, e.g. identifying overall risk of area as Low, Medium or High. All information was ultimately stored in a database developed by CCC and later maintained by Aurecon on behalf of CCC. Physical work packages were generated and mitigation works began to put Short Term (6-12 months) rockfall protection in place (Photograph 1).



Photograph 1. Meshing of failed / failing bluff (High Risk)

The main method of data capture and presentation at this stage was proformas containing the agreed assessment criteria, handwritten on site, then entered into Excel and submitted to the Port Hills Geotech GIS team for entry into the database. Other methods included using laptop on site for direct entry into Excel, and in some cases, relaying information verbally; from field engineer to GIS operative.

Many areas across the Port Hills are inaccessible on foot or by vehicle due to safety reasons. In these areas inspection methods employed included helicopter inspection, imagery from remote controlled craft and use of the Ike GPS. This method was employed by Opus to inspect bluffs directly underneath the pathway of the Christchurch Gondola. Co-ordinates of existing position are recorded as well as the co-ordinates of the object of interest, via laser beam.

2.3 13 JUNE 2011

In the wake of the 13 June 2011 aftershock, PGA's experienced ranged from; 0.4g – 2.2g horizontal and 0.1g – 1.1g vertical. The same set of criteria was used to gather information, however the use of iPad tablets with ArcGIS live database began to be used in the field. Aurecon had familiarity with the use of tablets across a range of field work situations and their staff had trialled the use of the iPad tablet in the Port Hills with great success. Based on this, CCC issued a directive for all PHGG staff to employ this data capture method for all future works across the Port Hills. An example of the output is presented in Figure 2.

This effectively allowed real time updating of the database in the field. Later, data from the Canterbury Earthquake Recovery Authority (CERA), Earthquake Commission (EQC) and GNS Science (GNS) was also added to the database, which enabled a range of information to be accessed whilst on site by a wide range of parties.

This method has significantly reduced data entry time and increased the accuracy and uniformity of site mapping. The database continues to be updated to capture more data allowing a comprehensive understanding of the overall problem.

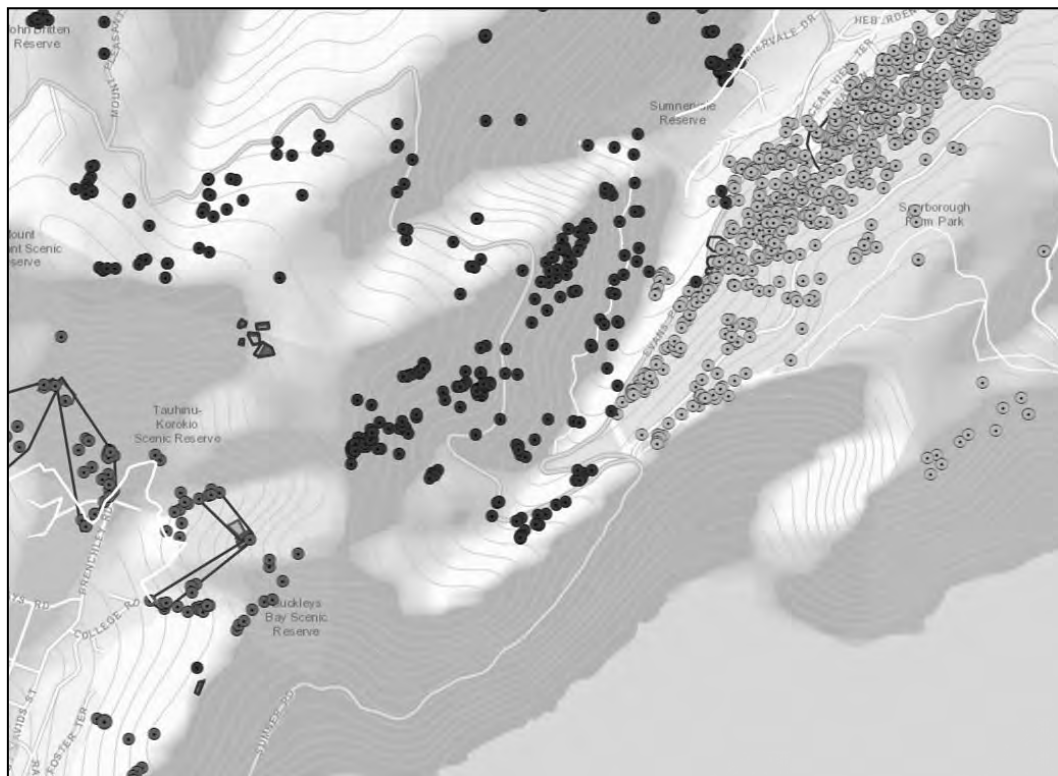


Figure 2. Screenshot of live database - Mapped Boulders (Fallen and In-situ)

2.4 PREFERRED DATA CAPTURE METHOD

It is clear that the most effective method of data capture used during site works throughout the Port Hills by PHGG has been the iPad tablet with live database. The array of information available whilst working in the field is of great benefit and the time spent on data entry has been significantly reduced.

Other agencies still use earlier methods that can be transferred to the GIS database, but not in 'real time'.

Data Capture Method Summary			
Method		Advantages	Limitations
Photography	• Standard camera • GPS camera • Aerial imagery; (helicopter & remote controlled craft)	• Accurate information immediately post event; • Conveys information to personnel not present on site; • Accurate spatial location (GPS camera); • Overview of area – direct resources to worst hit areas (aerial); • Visual record pre and post event; and, • Imagery of areas inaccessible by vehicle or on foot.	• Inaccurate labelling, both on site and in office; • Site photo log not kept; • Scale or reference point not captured; • Blurred image; • Expensive – helicopter aerial imagery; and, • Remote craft weather and battery dependent.
Notebook	• Handwritten on site	• First point for written field record; and, • Best results when used in conjunction with GPS and camera.	• Information can be lost if personnel leave or book is lost.
GPS	• Hand held • Icke GPS	• Record of track followed, good when mapping; • Cross referenced with photos and notes to give spatial reference to area of interest; • Main method used to map boulder trails by GNS; • Data can be uploaded and viewed on map; and, • Reference inaccessible areas (Icke GPS).	• Poor coverage can affect accuracy of location; and, • Icke GPS very sensitive to movement, hard to get accurate laser mark on to object of interest.
Spreadsheet / Laptop	• Handwritten in field • Typed up in the field	• Using agreed criteria when logging in field; and, • Ability to input data in field saves time (laptop).	• Hard to use in extreme weather conditions; • Potential to misplace sheets; and, • Time spent transferring hard copy in the office.
iPad Tablet	• ArcGIS live database	• Record information in live database on site; • Speeds up data entry process; • In-built GPS and camera; • Access to wide data set; • Information editable on site; and, • Rapid processing of data for statistical analysis.	• Poor coverage can affect signal; • Visibility can be difficult in harsh light; and, • Protective cover required to be useful in extreme weather conditions.

Table 1. Data Capture Methods Summary Table

3. A MODEL FOR DATA CAPTURE

In the likelihood of another large scale event, PHGG have experience and systems in place to manage this efficiently from the first instance. The lessons learned throughout this process could be used in other similar scale events (nationally and internationally) as a model for effective data capture methods.

It is also possible to draw on work undertaken in other countries to manage information pre and post disaster. An examination of existing guidelines and methods employed in New Zealand are explored and contrasted with methods used in Australia.

3.1 NEW ZEALAND

The New Zealand Ministry of Civil Defence and Emergency Management (CDEM) has guidelines on disaster management, risk assessment, preparedness etc. Reference is specifically made to information management in the CDEM framework (2009), which forms part of the CDEM Act (2002).

This document is specifically related to CDEM workers from a range of backgrounds and identifies competencies required. The emphasis is on disaster preparedness and response and managing those situations. There is reference to information management, but it is not specifically related to site work.

Consideration should be given to producing a similar document to be used beyond Civil Defence Emergency status, which could be used as a guideline on best practice pre and post disaster with reference to data capture methods.

Methods employed by agencies involved in disaster recovery in New Zealand vary between each institution. A national guideline which incorporates site work directives would be of great benefit. This would allow a more unified approach in data collection and management pre and post disaster. This would be a lateral initiative to encompass all aspects of site work, not specifically boulder roll / cliff collapse.

3.2 AUSTRALIA

The Australian Government Agency; Geoscience Australia (GA) is responsible for managing a range of geoscientific challenges in Australia to develop solutions through research and development. They work with State and Local Government agencies in relation to disasters, providing expert advice where necessary. There is a strong emphasis on disaster risk reduction and education for emergency managers and decision makers. They play a key role in national natural hazard initiatives in Australia and the Pacific region. This is governed under the National Risk Assessment Framework, (2011).

One of the fundamental aspects to work carried out by GA is the digital collection of data, both pre and post disaster to allow raw data to be entered directly into a GIS database system. GA asserts that confidence in results can only be achieved by accurate data collection, which is particularly important in modelling and risk analysis assessment.

“Geoscience Australia has developed the capability to collect post-disaster information and the technologies have been tested in numerous Australian and overseas locations following natural hazard events. As well as providing assistance to hazard risk management, data collection technologies can help response teams by transmitting near real-time spatial information between field personnel and coordinating centres.” (2011)

GA has been involved in data capture in a range of settings for example; surveying buildings affected by bushfires in Victoria in 2009; Survey of buildings affected by flooding in Lismore NSW in 2005 and performing an earthquake survey in Gujarat, India in 2001.

The preferred method of data collection used in the Port Hills is the main data collection method employed by GA. Whilst a database of field information was put in place immediately under the direction of CCC, knowledge of data collection methods such as those highlighted by GA would have been of tremendous benefit.

4. CONCLUSIONS

Data capture methods across the Port Hills have varied with each event, with effectiveness and limitations of each, outlined in this paper. The aim has been to highlight the data capture methods used immediately after a large scale event and the evolution of methods over time with each new challenge.

It is now clear, in relation to work in the Port Hills that using the live database has been the most effective method of data capture for the field mapping teams. The efficiency of using this data capture technique has far surpassed other methods used. In the likelihood of another large scale event, the Port Hills Geotech Group is well equipped to implement and manage information, along with the other necessary tasks involved.

It is noted at present in New Zealand there is a framework in place for CDEM which incorporates information management. The framework is intended for use by CDEM professionals. It is felt that this could be expanded beyond the state of Civil Defence Emergency to produce a set of guidelines to be employed for site work by other agencies / professionals involved in post disaster management. Work executed in the Port Hills would be of clear benefit as an example on how to gather data for future effective use.

These methods could be expanded to be used across a range of disciplines, for example; by structural teams performing building checks in the first instance i.e. Level 1 inspections; building type, damage etc.

In Australia, GA provides clear direction on data capture methods and management pre and post disaster. There are also several examples of how these methods have been employed both nationally and internationally. The same could be applied to New Zealand; in particular the work of the Port Hills Geotech Group could be used as an example of best practice.

The work completed to date serves as an effective model for use nationally and potentially internationally. It is vitally important that information is also gathered and used on effective models in other countries. Australia in particular is involved in research and follows best practice in data capture and management following a disaster.

The work performed across the Port Hills by the PHGG would serve as a good platform on which to base research into the area of data capture methods and management pre and post disaster and how this information can be accurately relayed, stored and used at a governmental level to develop effective policy on disaster management.

The use of the iPad tablets as a model for data collection to a live database as well as general site works continues to be explored. There is strong evidence for reliability and speed of information relayed back to necessary centres post disaster.

It is also clear that further co-operation between New Zealand and Australia in areas such as these would benefit each country greatly. Methods used in other disaster prone countries such as Japan, Hong Kong and USA, should also be explored.

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DAMAGE TO RESIDENTIAL HOUSE FOUNDATIONS RESULTING FROM THE 2010-11 SEQUENCE OF EARTHQUAKES IN CANTERBURY, NEW ZEALAND

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ABSTRACT

The 2010-11 sequence of Canterbury earthquakes resulted in widespread damage to residential house foundations within Christchurch. Ground accelerations from the sequence of earthquakes have directly resulted in shaking of the structures, liquefaction of saturated fine sand and silt soils and the compression of loose unsaturated soils.

This paper describes damage to residential house foundations in the least-damaged 'Green Zone' observed during geotechnical inspections for earthquake recovery work in Christchurch.

The typical damage to the foundations include: cracking and spalling of concrete ring beams; cracking of concrete slabs; sunken shallow piles and concrete slabs; rotation of shallow piles and concrete ring beams; displacement of foundations relative to timber framing and cladding.

Differential settlement is generally caused by liquefaction of underlying saturated soils, the ejection of soils to the surface, as well as compaction of loose unsaturated soils near the surface due to ground shaking. The cause of bearing capacity failure is generally a combination of earthquake loads overloading foundations and loss of soil strength due to liquefaction.

Other contributory causes in some cases include the interaction of different foundation types due to house extensions, and poor building design practice and/or workmanship.

1. INTRODUCTION

The sequence of earthquakes in and around Christchurch has resulted in damage to the foundations of numerous residential properties. This paper describes the typical damage in the least-damaged 'Green Zone' observed during geotechnical inspections for earthquake recovery work in Christchurch and briefly discusses their causes.

2. BACKGROUND

2.1 EARTHQUAKE SEQUENCE

On 4 September 2010, a magnitude M 7.1 earthquake struck Canterbury, New Zealand. The epicentre of the earthquake was about 40 km west of Christchurch (Cubrinovski et al. 2010), the largest city in Canterbury and the South Island of New Zealand.

There was worldwide coverage of this earthquake and the damage caused to property as a result of liquefaction, particularly in Kaiapoi (Orense et al. 2012). There were no fatalities as a result of this earthquake.

The 4 September earthquake was followed by hundreds of aftershocks including the 22 February 2011, magnitude M 6.3 Christchurch earthquake. The epicentre of the Christchurch earthquake was located near to Lyttelton about 10km southeast of Christchurch (Geonet 2012). The Christchurch earthquake resulted in much larger ground shaking and more widespread liquefaction in the city than the 4 September earthquake, as well as rockfall; particularly around the Port Hills. It resulted in 185 fatalities and several thousand injured (McSaveney 2012).

Other large aftershocks include a magnitude M 6.3 on 13 June 2011, centred 10km east of Christchurch and 10km northeast of Lyttelton and a magnitude M 6.0 on 23 December 2011 centred about 10km east of Christchurch and 10km north of Lyttelton. There was further liquefaction throughout areas of Christchurch already affected in the previous earthquakes, and no reported fatalities.

As of 23 March 2012 there were 459,046 claims lodged with the Earthquake Commission of New Zealand relating to approximately 120,000 residential properties in Canterbury (Dept. of Building and Housing 2012).

2.2 GROUND ACCELERATION

The ground accelerations in Christchurch were greatest during the 22 February 2011 earthquake due to the proximity of the epicentre to the city and its shallow depth. The maximum peak ground acceleration (PGA) recorded was 2.2g at Heathcote Valley Primary School. PGA's greater than 1.0g were recorded at other locations across the city.

2.3 GROUND CONDITIONS

The majority of residential Christchurch lies upon the Christchurch and Springston Formations (Brown et al.). The Christchurch Formation predominately in the east includes beach, dune, estuarine, lagoon and coastal swamp deposits comprising gravel, sand, silt, clay, shell and peat. The Springston Formation includes postglacial fluvial, overbank and swamp deposits comprising gravel, sand, silt, and peat. Although similar materials are present throughout Christchurch due to the deposition processes these can be highly variable over short distances.

2.4 LIQUEFACTION

Liquefaction is a process by which a soil behaves like a fluid, suddenly losing its strength, usually as a result of ground shaking during large earthquakes. Soils most susceptible to liquefaction are silt to fine sand sized particles that are loose and saturated. Dry soil will not liquefy during ground shaking. When rapid ground shaking occurs due to an earthquake, silt and sand sized particles tend to compress into the pore-space occupied by the groundwater, resulting in increased porewater pressure. As the pore water pressure approaches the overburden pressure, the effective confining pressure approaches nil and the soil liquefies; behaving like a fluid with a significant reduction its shear strength. (IPENZ, 2011)

The soils of Christchurch are susceptible to liquefaction when they lie below the groundwater level; which in Christchurch is usually varies from 10m below ground level to near the surface.

2.5 LAND CATEGORISATION

The residential land in Christchurch has been characterised into four different zones by the NZ Government and the Canterbury Earthquake Recovery Authority (CERA). The four zones and descriptions are shown on Table 1.

Land Zone	Brief Description
Green	Repair/rebuild process can begin
Red	Land repair would be prolonged and uneconomic
Orange – hold	Further assessment required; to be re-zoned Red or Green depending on further investigation
White – un-zoned	Parts of the Port Hills and CBD; still being mapped.

Table 1: CERA Land Zones (Dept. of Building and Housing 2011)

The land within the 'Green Zone' has been further subdivided into three technical categories. These were determined by the extent of damage to the land as a result of the earthquakes. The three technical categories are described on Table 2.

Technical Category	Brief Description
TC1 or Grey	Future land damage from liquefaction is unlikely, and ground settlements are expected to be within normally accepted tolerances.
TC2 or Yellow	Minor to moderate land damage from liquefaction is possible in future large earthquakes.
TC3 or Blue	Significant land damage from liquefaction is possible in future large earthquakes.

Table 2: 'Green Zone' technical categories (Dept. of Building and Housing 2011)

2.6 TYPICAL RESIDENTIAL HOUSE FOUNDATIONS IN CHRISTCHURCH

House foundations within residential Christchurch are typically one of three different types: 1) suspended floor supported by shallow piles, 2) suspended floors supported by a perimeter concrete ring beam and shallow piles within the ring beam, and 3) concrete slab-on-grade (Dept. of Building and Housing 2011).

Foundations types 2 and 3 can support both light weight cladding (e.g. weatherboards) and heavy cladding (e.g. brick or concrete masonry). They are usually wider and/or deeper in order to support heavy cladding.

3. OBSERVATIONS OF FOUNDATION DAMAGE

3.1 CRACKING AND SPALLING OF FOUNDATIONS

Cracking and spalling of concrete is common damage to concrete foundations. Observations of concrete ring beam foundations showed numerous cracks that extend all the way through the concrete ring beam. These cracks commonly occurred at the location of vents.

The cracks are typically around the vents and at the corners of the house (Figure 1).



Figure 1: Typical cracking of concrete ring beam foundations



Figure 2: Typical spalling at a join between two foundations

Observations of spalling showed that spalling typically occurs at locations where the concrete ring beam is compressing against another part of the ring beam. Figure 1 shows this occurring at an internal corner of the concrete ring beam and Figure 2 shows this at the join between two different foundations.

Minor spalling was also observed to occur as a result of brick masonry moving on top of concrete ring beam foundations.

3.2 SUNKEN FOUNDATIONS

Observations beneath houses show that the bearers and joists are wavy, and some have become disconnected from the top of the shallow piles, the worst observed being approximately 100mm apart. Many of the shallow piles are leaning and/or sunken, with no consistent gradient of settlement of the piles across the house footprint. The piles do not all lean in the same direction either. This is the cause of uneven floors within several houses that tilt in all directions.

At a particular house there were concrete and timber piles; the concrete piles appear to be the original piles as many of them have blocks atop them that suggest they have settled in the past and have been repaired by jacking and packing or replaced with the timber piles observed. These old repairs probably levelled the floors. However, now they are uneven again as many of the piles have sunken further (Figure 3).

The ground of the backyard around a split-level property in the suburb of Cashmere has settled between 100 and 180mm. As most of the house is founded lower than the backyard, the house has surprisingly little damage. However, the conservatory at the rear of the house is founded at the level of the backyard upon a concrete slab foundation. This concrete slab has sunk compared to the rest of the house resulting in a vertical step between it and the rest of the house as well as considerable deformation of the conservatory framing (Figure 4). A mass concrete doorstep on one of the side of the house has also sunken at one end (Figure 5) and services became disconnected during the earthquakes.

The soils of the backyard are most likely fill material placed to form the higher level backyard; therefore the drop in the level of the ground is most likely due to the compression of the loose backfill material.



Figure 3: Sunken piles and wavy bearers

At another house there is a sunken mass concrete doorstep and patio. These are most likely the result of compression of loose soils near the surface and liquefaction of the underlying saturated soils. These blocks of mass concrete lean away from the house, the reason for this is most likely to be because the soils close to the concrete ring beam have been more compaction due to construction activities during the building of the house than those further away from the house.



Figure 4: Vertical step where there wasn't meant to be one, between the conservatory and the house.



Figure 5: Differential settlement of a doorstep

3.3 ROTATION OF FOUNDATIONS

As described above there is a concentration of cracks around the corners of concrete ring beam foundations, sometimes these cracks were observed to be open more at the top than at the base of the concrete ring beam. This wedged shape crack (Figure 6) suggest that the concrete ring beam perpendicular, on the other side of the corner has rotated forward. Another example is shown in Figure 7; in this case a section of concrete ring beam has rotated outward from the house, as it has been released from the rest of the concrete ring beam by two large cracks at each end of a section. In both examples the concrete ring beam appears to have only shallow burial.

The rotation of the concrete ring beam is most likely the result of earthquake forces overloading the foundation in combination the presence of loose soils and/or the weakening of the soils due to liquefaction. In addition the shallow burial of the concrete ring beam has meant there is little soil to resist the earthquake forces being exerted on the concrete ring beam.



Figure 7: Rotating concrete ring beam foundation



Figure 6: Open cracks on either side of a rotating section of concrete ring beam foundations

3.4 DISPLACEMENT OF FOUNDATIONS RELATIVE TO TIMBER FRAMING AND CLADDING

At several houses it was apparent that the foundation and the rest of the house had moved independently of each other during the earthquakes resulting in structural elements of the house and elements of the foundations being displaced relative to one another.

Figure 8 shows a clear example of this. This is at the rear of a house where approximately 20mm of the concrete ring beam is exposed; where the timber framing and timber weatherboard cladding used to lie. At the front there is between 20 to 50mm exposed. In this case the extent of the foundations is now greater than the footprint of the house, therefore sections of the concrete ground beam must have moved outward from the house, this is possible in this case because of the multiple cracks within the concrete ring beam.

Where concrete ring beams have not been so cracked, the displacement has been observed to be equal and opposite on opposite sides of the house, i.e. 20mm of foundation exposed at the rear of the house, and overhanging by 20mm at the front of the house.

The cause of the observed displacements is caused by earthquake shaking directly moving the ring beam foundations and/or the rest of the house relative to one another. The independent movement indicates that there is likely no connection between the foundations and the framing, or that it is too weak, or in the case of older houses, that it is no longer functional.

3.5 HOUSE EXTENSIONS

The geotechnical inspections revealed a common theme of foundation damage where houses have been extended. Even if the foundations appeared to be the same type and roughly the same dimensions, there was typically damage to the foundation at the joint between foundations that have been built at different times.

At one house the floor within the extended section of the house, the floors sloped toward the rest of the house. While at another the slope was in the opposite direction from the rest of the house. This is an example of construction influencing the damage to foundations.

Like the corner effects of concrete ring beams described above the locations of greater load generally have more potential for greater settlement. Damage is probably exacerbated further if there is no connection between the two foundations, or if the two foundations have different dimensions.



Figure 8: 20mm displacement between the concrete ring beam and the timber framing and cladding



Figure 9: dip in weatherboards directly above the location of a join in the foundation

4. CONCLUSIONS

During geotechnical inspections of residential houses a variety of different types of damage was observed to the foundations, this included cracking and spalling of concrete ring beams; sunken shallow piles; rotation concrete ring beams; displacement of foundations relative to timber framing and cladding.

The cracked and rotated concrete ring beam foundations, which are also displaced relative to timber framing of the house were caused by earthquake loads directly moving the ring beam foundations and/or the rest of the house relative to one another; earthquakes loads also overloaded the foundations; and compaction of loose soils near the surface due to ground shaking and liquefaction of saturated silt and fine sand soils, causing the concrete ring beams to rotate in a bearing capacity type failure. It is likely that inadequate burial of the concrete ring beam also contributes to the foundation rotating. In addition there appears to be either a lack of or a weak connection between the foundation and the timber framing of the house, particularly in older houses.

There is a preference for cracking in the concrete ground beam around the vents, because they are the weakest points in the beam. Cracking near the corners of foundations is likely due to a greater concentration of applied load near the

corners. This concentration of load causes it to settle more than straight section of the foundations away from the corners.

Differential settlement of ancillary concrete footings appears to be due to a combination of uneven compaction of loose unsaturated fill soils near the surface due to ground shaking and liquefaction of saturated silt and fine sand soils. It is likely that fill materials close to the house might have subjected to compaction during construction compared to those away from the house.

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UNDERSTANDING FRACTURE NETWORKS FOR OPEN PIT SLOPE DESIGN

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ABSTRACT

The design of open pits, from a geotechnical perspective, comprises the derivation of an engineering geology model followed by stability analyses, which are assessed against a slope design criteria. Stability analyses are typically carried out using empirical, kinematic, or limit equilibrium methods. For large-scale open pit projects, numerical modelling has gained increased usage. A discrete fracture network (DFN) approach involves numerical modelling that explicitly represents how the engineering geology model structures may be spatially distributed through the rock mass. This approach has been widely used in oil and gas, and environmental projects, including radioactive waste management; it is now becoming more accepted in international mining projects. For slope stability or tunnelling projects DFN modelling provides a valuable geotechnical tool for visualisation of a rock mass and identification of likely failure mechanisms.

This paper presents a component of the DFN modelling undertaken for the design of a large open pit in Australia. The necessary input parameters and derivation of these, along with the importance of an accurate engineering geology model are discussed. The innovative nature of the work required iterative procedural developments, with the final methodology presented here. The development of this tool has been influential in improving our understanding of fracture networks for open pit slope design.

1. INTRODUCTION

Traditional kinematic and statistical analyses of geological structure for geotechnical engineering applications, while considered industry best practice, have some limitations. These analyses attempt to indicate the spatial distribution and orientation of defects with the results interpreted using experience and empirical rules. More recently, for advanced civil and mining rock engineering studies a discrete fracture network technique has been developed. Discrete fracture network (DFN) models are those that represent a body of rock as an assembly of rock blocks separated by discontinuities. The DFN technique utilizes the structural data in stochastic analyses to give a range of possible models, which indicate the real large-scale slope or tunnel performance and stability. Stochastic analyses are probabilistic simulations of a range of possible results based on a set of input parameters and modelled functions to describe the distributions of the data.

The DFN concept has been utilized in fluid flow modeling (e.g. Wilson et al., 2011), in modeling the performance of geothermal reservoirs and is widely used in the petroleum and nuclear industries. DFN modeling and its applications to geotechnical engineering has been limited to a few international projects, for underground excavations (Starzec and Andersson, 2002; Bakun-Mazor, 2009; Menéndez-Díaz, 2009; Merrien-Soukatchof et al., 2011) and rock slope engineering (Hadjigeorgiou and Grenon, 2005, Gasc-Barbier et al., 2008, Grenon and Hadjigeorgiou, 2008). Vyazmensky et al., (2010) used a combined DFN finite element modelling approach to investigate slope failure in a pit that resulted from interaction with existing underground mining.

Many fracture/joint models have been developed and reported in the literature. Dershowitz and Einstein (1988) summarized those models reported in the literature up until the late 1980s. While there has been significant progress since that time the main principles' regarding the input parameters and stochastic processes are similar to the early models of Venezianao, Baecher and Dershowitz (see Dershowitz and Einstein (1988) for a review). A number of computer codes have been developed in research and industry to perform DFN generation. These include FracSim3D (Xu and Dowd, 2010), OPS (Elmouttie, 2011) RESOBLOK (Merrien-Soukatchof et al., 2011) and FracMan (Dershowitz et al., 1998). These packages have a range of advantages and disadvantages depending on the input data and objectives of the investigation. In this paper the DFN code FracMan (Golder Associates) is employed.

A basic assumption of stochastic DFN models is that statistics associated with the fracture network can be measured from one and two-dimensional data and used to generate fracture systems with the same properties. Generally, the only data available to constrain the key DFN attributes (length, orientation, spatial distribution and intensity) are derived

from some combination of geophysical, borehole or outcrop data. The modeling assumes that the characteristics of the fracture population (e.g. orientation, spacing) can be assessed and structural regions defined. The fracture system modeling is typically built to represent the moderate to small-scale structures in structurally homogenous regions, not the large-scale structures that may be modeled deterministically (Starzec and Andersson, 2002).

The focus of this study is the construction of a major fault network for the crystalline basement rocks. Stochastic DFN's looking at small and moderate scale structures has been developed for each of these domains in the basement and cover sequence.

2. CASE STUDY SITE

The case study site reported here is, to the author's knowledge, the first mine site in Australia for which comprehensive DFN modeling has been undertaken for geotechnical purposes. The northern portion of the ore body is exploited by an existing underground mine. The DFN modeling reported here is part of a large body of work in planning for an open pit in the southern portion of the ore body. Figure 1a shows the location of the existing underground workings relative to the planned open pit. The existing mine has a comprehensive three-dimensional fault wireframe model, shown in Figure 1b.

The ore body at the site is contained in a breccia complex hosted by granite. These rocks are very high strength with a typical unconfined compressive strength (UCS) of 100 to 230 MPa. Consequently, open pit stability and interaction with the existing underground is likely to be largely controlled by structure, justifying a DFN modeling approach. Horizontally bedded sedimentary rocks locally referred to as the cover sequence, overlie the crystalline basement, so data in the southern mine area is constrained to borehole data. A comprehensive campaign of acoustic borehole imaging and quality control was undertaken at the site resulting in a large, high quality structural database from some 500 boreholes.

A prominent regional fault zone cuts across the site, as shown in Figure 1. This fault zone is not a discrete fault plane, simply a zone of increased fracturing. This fault zone is one of four broad structural domains defined for the site.

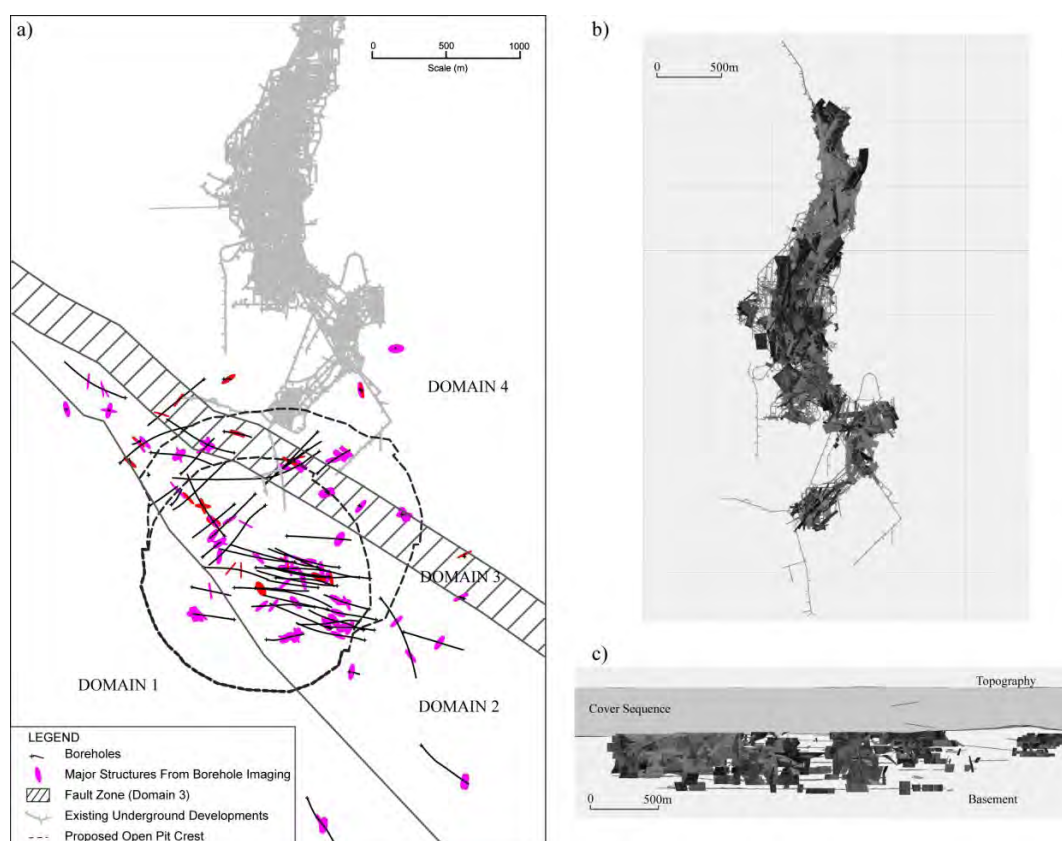


Figure 1: a) Location of boreholes, proposed open pit and existing underground developments; Existing underground three-dimensional fault wireframe model in b) plan view and c) cross-section.

3. DATA

3.1 INTRODUCTION

The aim of the fracture system modeling is to develop a network whose statistical and spatial measurements match those of the actual rock mass. Thus, a good engineering geology model and accurate definition of the necessary fracture properties is essential. The modeling assumes that the characteristics of the fracture population (e.g. orientation, spacing) can be assessed and structural regions defined. Typical DFN modeling also assumes that the region is structurally homogeneous, with models developed for individual domains.

The choice of input variables to generate a DFN depends on the complexity of the geological model of the site and the purpose of the modeling. The development of a fracture network for geotechnical applications requires four primary inputs, which are summarized in the following sections.

3.2 TYPE OF DISCONTINUITIES

The selection of the appropriate discontinuities to form the basis for the fracture network in this study was contingent on their likely scale. Major structures at the case study site were defined as faults and shears with an infill thickness of greater than 100 mm. This definition is based on experience from the underground mine at the site and from previous studies which have found that structures with large infill are typically more continuous (e.g Starzec and Andersson, 2002). Filtering of the dataset on this criterion identified 144 structures from 64 boreholes. The appropriate selection of the type of discontinuities for model generation was important not only from an engineering point of view but also because the generation of a DFN representing all fault sizes could not be used in the following 3D displacement modelling due to considerable demands on computer power and time.

3.3 FRACTURE ORIENTATION

A comprehensive campaign of acoustic borehole imaging and quality control was undertaken at the case study site so that there is a high degree of confidence in the fracture orientation data. The reliability of DFN models can be improved with the separation of fractures into distinct orientation sets (Dershowitz et al, 1998; Starzec and Anderson, 2002). Stereographic analysis showed some clustering of the data, as presented on a lower hemisphere projection in Figure 2. While 16 sets were identified the data was somewhat scattered.

When fracture orientation data is scarce or scattered, “bootstrap” sampling has been shown to provide a better alternative for orientation simulation (Efron, 1982; Starzec and Andersson, 2002). “Bootstrapping” implies that fractures generated in the DFN model have the same orientation as those measured in the field and no set separation is performed. The dispersion coefficient for this function may be estimated using the formulae for the Fisher dispersion coefficient (see Priest 1993). Considering the scattered nature of the sets at the case study site, the orientation input data was “bootstrapped” directly from the orientation of the 144 selected fractures, with a dispersion coefficient based on the Fisher dispersion.

Boreholes, like other linear sampling techniques, have a built in bias. Terzaghi (1965) developed a correction factor to address this sampling bias; however, in this case study application of this correction factor did not result in identification of any alternate fracture sets.

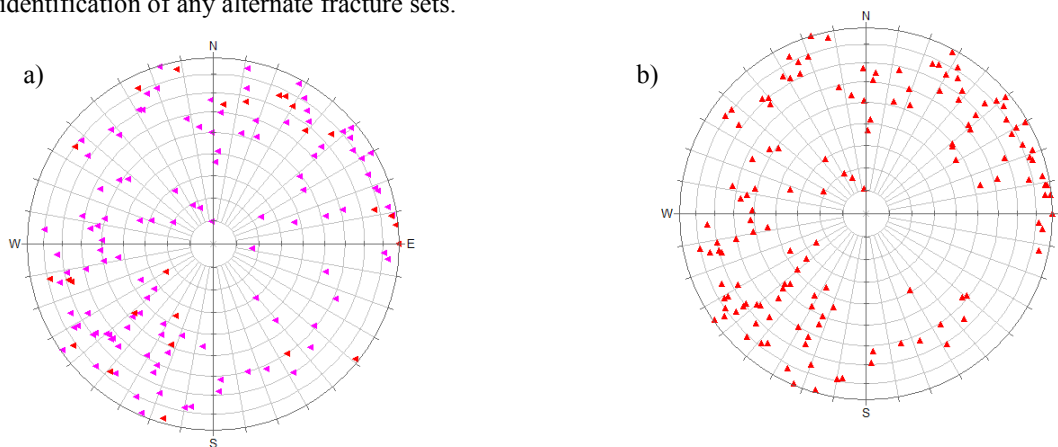


Figure 2: a) Poles to measured faults with infill greater than 100mm and b) Poles to synthetic faults. Lower hemisphere, equal area projection.

3.4 SPATIAL MODEL

The spatial distribution of fractures is estimated from spacing measurements along sampling lines such as boreholes or from two-dimensional rock face mapping. Previous studies have shown that a number of fracture location models such as, Levy-Lee or Nearest-Neighbour may best represent observed data (e.g. Dershowitz et al, 1998). However, Priest (1993) suggests that in the absence of a strong clustering pattern, a random fracture location model based on a Poisson process be adopted.

Analysis of fracture spatial patterns at the case study site was performed using the borehole structural data. The analysis was undertaken on the selected fractures in each borehole with no separation into sets. A power density plot was computed for each borehole and the slope of the plot used to assess different possible spatial models. For 80% of boreholes the slope of the line was approximately zero, implying the fractures in the boreholes are located according to a Poisson process. Consequently a Poisson spatial distribution was utilized for this DFN model.

3.5 FRACTURE INTENSITY

Volumetric fracture intensity is one of the most important fracture network characteristics. DFN theory uses a unique terminology of defect intensity, which include:

- P_{10} – a linear measure, i.e. the number of fractures per metre (1/m).
- P_{21} – an areal measure, i.e. the trace length per unit of mapped area (m/m^2).
- P_{32} – a volumetric measure, i.e. fracture surface area per unit of rock volume (m^2/m^3).

While P_{32} is an important parameter in DFN theory it is theoretically derived and cannot be measured in situ. P_{10} and P_{21} are obtained from field measurements and used to constrain fracture intensity. In the current study, P_{10} was used to build the DFN model for the case study site. The observed P_{10} for the site was between 0.001 and 0.05 fractures m^{-1} .

Fracture intensity was also correlated to the four broad structural domains at the case study site. This was to control intensity away from the boreholes, which were centrally located. The intensity was calculated for each domain as the number of major structures divided by the number of boreholes with acoustic imaging data. This calculation was then normalised against the fault zone intensity.

Domain	No. of Boreholes ¹	No. of Faults and Shears ²	Faults and Shears Divided by Number of Boreholes	Normalised to Fault Zone
1	70	22	0.31	0.13
2	216	195	0.90	0.38
3 (Fault Zone)	44	104	2.36	1
4	44	26	0.59	0.25

Table 1: Relative Fault Intensity

Note: ¹ Number of boreholes with acoustic imaging in that domain
² Number of fractures with infill greater than 50mm

3.6 FRACTURE SIZE

Fracture size is typically the most difficult parameter to quantify, particularly from borehole data sets. Fracture size is still difficult to obtain from field mapping data with trace length data not equivalent to fracture length and complicated mathematics required for conversion (Zhang et al, 2002). Some studies have estimated size using empirical relationship between fracture length and aperture (e.g. Starzec and Andersson, 2002). For the case study site the three-dimensional fault wireframe model from the existing underground mine was used to educate structure persistence for the DFN model. Analysis of the wireframe model found a log normal distribution with a mean of 102 m and a standard deviation of 41 m to be the best fit. Although considering that this was a major structure DFN, the size distribution was then truncated so that the minimum fracture size had a radius of 51m.

3.7 OTHER INPUTS

A number of other inputs are available for the DFN model, such as aperture and connectivity of the fracture network (termination). While these inputs are a crucial aspect for flow simulation they were not utilized here, as they were not considered essential input parameters for the purpose of this model.

4. FRACTURE MODEL GENERATION

A workflow for the construction of a DFN is provided in Figure 3. While DFN modelling typically uses a stochastic approach, in this example only a single model was developed for input into a three-dimensional finite element model. Due to computational constraints only a single finite element model with the DFN incorporated was to be run hence a single DFN was constructed.

The generation region for the fracture network was a rectangular prism with the dimensions 3900x4700x1500 m. The generated fracture shapes were approximated with four-sided polygons. The orientations of fractures were “bootstrapped” directly from the 144 fractures selected from the 64 boreholes. The representative fracture radii were generated from a lognormal probability distribution with a mean of 102 m and a standard deviation of 47 m. The actual borehole intercepts were used to directly control fracture intensity, with the 64 boreholes and 144 fracture intercepts imported into the program. The generation region was then populated until the average P_{10} across the boreholes was achieved.

The resulting major fracture network model is shown in Figure 4. The plan view of the network clearly shows a concentration of fractures along the fault zone with a lower intensity in the southern and northern mine areas.

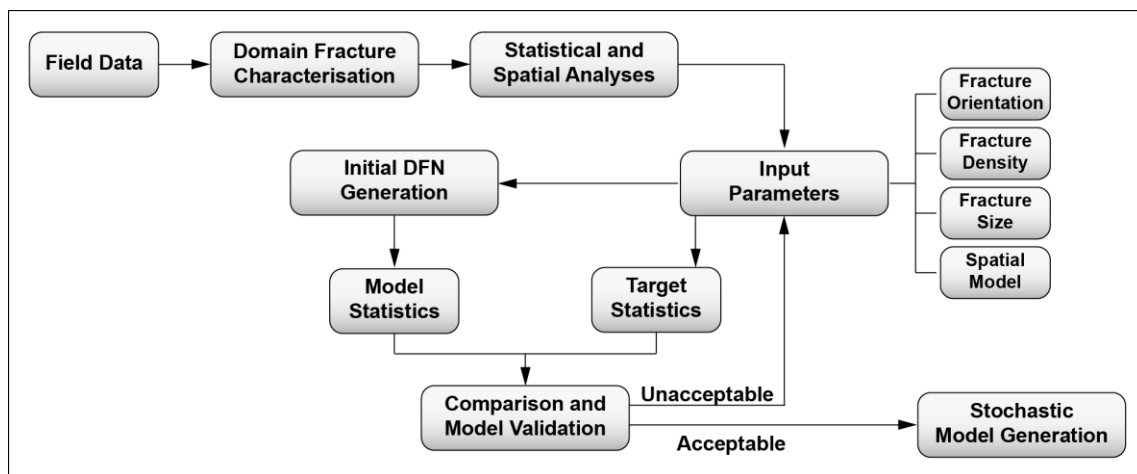


Figure 3: Workflow summary for DFN development

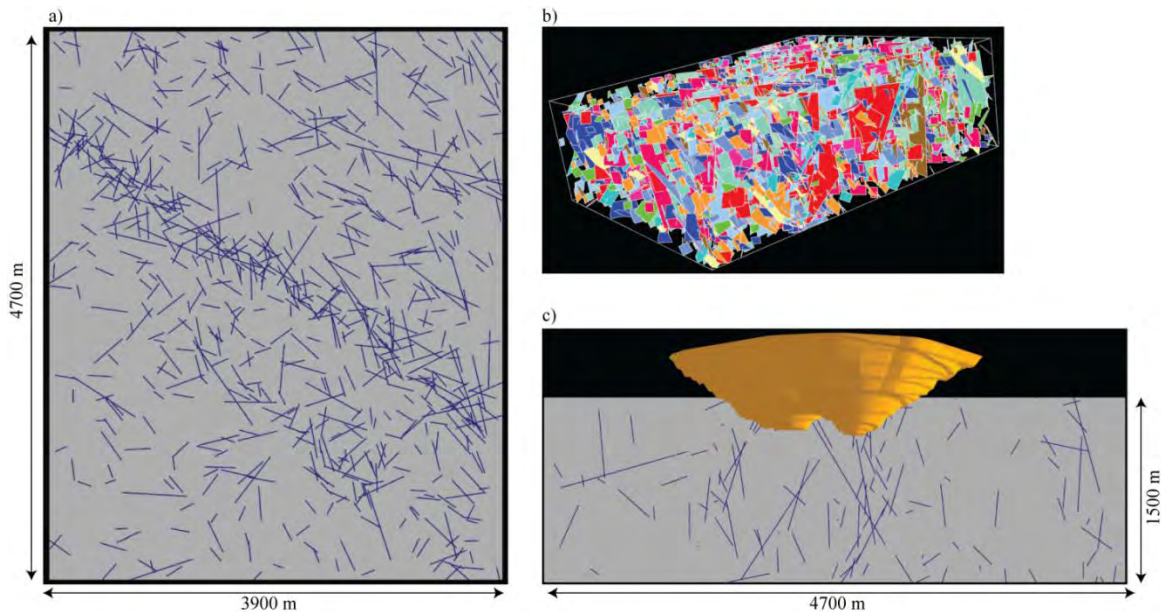


Figure 4: Resultant DFN a) Horizontal slice; b) Orthogonal View and c) Vertical north-south slice.

5. MODEL VALIDATION

Validation of the DFN model is a crucial aspect in evaluating the appropriateness of the synthetic network. While complicated mathematical models may be developed the final product must be critically verified.

As part of the validation process the generated network was intersected with the input boreholes. Stereographs of the original and simulated structures intersecting the boreholes are shown in Figure 2. The simulated orientations are a close match to the original data. However the number of structures is slightly increased as some fracture planes intersect a number of close boreholes.

The FracMan code provides the derived P_{32} as a model output, with a value of 0.0096 for the model derived in this case study. For calibration the three-dimensional structural wireframe model from the existing underground is 0.057. Considering the DFN model developed here is of major structures, this difference is considered appropriate.

Modelling is an iterative process, with refinement of input parameters until an acceptable agreement with the field measurements is obtained. For example, fracture orientation data was initially defined as 16 sets, with limited success due to data scatter, so orientations were then “bootstrapped”. It was concluded that the DFN model generated satisfactorily represented the major fracture network at the case study site and able to be utilised in further stability modelling.

6. CONCLUSIONS

This paper presents a component of a suite DFN modelling undertaken for the design of a large open pit in Australia. The DFN developed is unusual in that it was not stochastic analysis, as it was for input into a three dimensional finite element model. The necessary input parameters and derivation of these, along with the importance of an accurate engineering geology model were discussed. The innovative nature of the work required iterative procedural developments, with the final methodology presented here, including a workflow diagram.

Validation of the model is an important step in the process and showed in this instance to be a good fit to the measured field data. With recent improvements to computational abilities the DFN approach is likely to become much more highly utilised in civil and mining applications in Australia.

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HYDROGEOLOGICAL/HYDROTHERMAL MODELLING IN GROUNDWATER INVESTIGATION FOR MINING OPERATIONS USING FEFLOW AND PANDAS THERMOFLUID

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ABSTRACT

Hydrogeological investigation and groundwater modelling are essential for resource management decision makers and the community to quantify the environmental impacts, assess system performances and reduce the level of uncertainty. Hydrothermal analysis is necessary where a geothermal resource exists or a geothermal hazard in mining is encountered. In this study, PANDAS ThermoFluid, a finite element industry strength model developed at the University of Queensland, is introduced for simulating highly nonlinear coupled fluid and heat transfer with phase change. The mathematical models and numerical methods involved in groundwater/hydrothermal modelling are outlined. The model is used for simulating the impact of hydrothermal behaviour on mining operations at a geothermal active area, and for the preliminary design of a coffer dam in an operative mining site. To consider both unsaturated zone and multiphase phenomenon existing in the mining site, the model coupling both FEFLOW and PANDAS ThermoFluid is developed to predict the phreatic surface, steam occurrence as well as pit wall pressure/temperature conditions. This work could offer improvement in computational ability and efficiency for the multiphase modelling in mining and environmental hazard management.

1. INTRODUCTION

Hydrothermal flow in porous media is important in several practical areas, such as groundwater remediation, mining excavation, extraction of geothermal energy, and construction for waste isolation. For instance, high rock temperature and fluid pressure are common mining hazards in geothermally active areas. Numerical modeling is necessary to evaluate the impact of coupling phenomena on overall system performance. Great efforts have been devoted to modeling multi-phase coupled fluid and heat flow in porous media for both immiscible fluid and miscible fluid. The phase transfer of liquid/vapour two-phase in a non-isothermal system is widely observed, but considerably differs from the conventional cases. This makes modelling of such fluid flow behavior to be very difficult because the two phases in the system (i.e. liquid water and vapour) not only co-exist in the same volume (miscible), but also exchange mass during the phase change. Mass exchange between two phases with large density ratio (≈ 1000) often makes numerical modeling extremely difficult.

Modelling with phase transfer has been explored in the last decades and MODFLOW and TOUGH2, as two of the most popular finite-difference software, have been widely used in such a modelling. One of the drawbacks of finite-difference based models is their lack of applicability towards complicated domains compared to finite-element based models. TOUGH2 is a finite difference based numerical simulator which can model liquid/vapor two-phase problems with phase change. TOUGH2 treats phase change by a primary variables switching technique (Pruess et al, 1999); i.e. the primary variables are switched from pressure and temperature to pressure and saturation when a phase change occurs.

On the other hand, FEFLOW is also widely used for both two-dimensional and three-dimensional analysis of groundwater flow and solute transport in porous media. The program uses the finite element method to solve the groundwater flow equation of both saturated and unsaturated conditions as well as mass transport. Furthermore, FEFLOW supports coupled heat transfer analysis. Unfortunately, FEFLOW is not able to solve phase change problems in its current state.

In this paper, a finite element model, PANDAS ThermoFluid, is introduced for simulating the hydrothermal process in porous media with phase change. The pressure and enthalpy based governing equations and auxiliary assumptions/equations, in which fully couple the representation of liquid/vapour two phase fluid and heat flow in porous media, are formulated. A nonlinear finite element procedure is proposed to solve the highly nonlinear phase change problem. The latest steam table equations (IAPWS 2007) are used to describe material properties under various thermodynamic conditions. The model is applied together with FEFLOW to analyse the phreatic surface, steam occurrence as well as pressure/temperature conditions at pit wall under three excavation stages of a coastal open-cut mine at a geothermal active area.

2. GOVERNING EQUATIONS

Assuming thermal equilibrium exists among the liquid water, vapour and rock matrix, the general mathematical model describes the conservation of mass and energy of a two-phase reservoir (Faust and Mercer, 1977):

$$\frac{\partial(\phi S_p \rho_p)}{\partial t} - \nabla \cdot \left(\frac{K k_{rp} \rho_p}{\mu_p} \cdot (\nabla P_p - \rho_p g \nabla D) \right) - q_m = 0 \quad (1)$$

$$\frac{\partial[\phi S_p \rho_p H_p + (1-\phi) \rho_r H_r]}{\partial t} - \nabla \cdot \left(\frac{K k_{rp} \rho_p H_p}{\mu_p} \cdot (\nabla P_p - \rho_p g \nabla D) \right) - \nabla \cdot \left(\lambda_m \frac{\partial T}{\partial P} \nabla P_p + \lambda_m \frac{\partial T}{\partial H} \nabla H \right) - q_e = 0 \quad (2)$$

where ϕ is the porosity; S is the phase saturation; ρ is the density [kg/m³]; P is fluid pressure [Pa]; K is the intrinsic permeability tensor of the porous medium [m²]; k_r is the relative permeability of the phase; μ is the dynamic viscosity [kg/m·s]; g is gravity [m/s²]; D is the depth [m]; H is the specific enthalpy [kJ/kg]; λ_m is the thermal conductivity tensor of the porous medium [W/m·°C]; T is temperature [°C]; q_m and q_e are source/sink of the total mass and energy respectively [kg/m³·s, kJ/m³·s]. The subscript p denotes the phases; i.e. $p=w$ for liquid, $p=s$ for vapour. Subscript p also implies summation convention over the two phases. The subscript r denotes the rock matrix.

Other auxiliary assumptions and equations are listed as follows.

1. Fluid saturation. The volume saturation of each phases sums to 1: $S_w + S_s = 1$. In the pure liquid region, the liquid saturation is assumed to be 1, and in the saturated vapour region, it is assumed to be 0. In the two-phase region, phase saturations are derived and calculated by the density and enthalpy equations, so they are also functions of pressure and enthalpy.
2. Pressure. The pressure of each phase is equal; i.e. the capillary pressure is assumed to be negligible: $P_w = P_s = P$.
3. Enthalpy. The enthalpy of the liquid/vapour mixture is defined as: $H = (\rho_w S_w H_w + \rho_s S_s H_s) / \rho$.
4. Temperature. The temperature and its derivatives with respect to pressure and enthalpy are functions of pressure and enthalpy, which are determined by the fluid thermodynamic property functions.
5. Density and viscosity of the fluid. The density of the liquid/vapour mixture is defined as: $\rho = \rho_w S_w + \rho_s S_s$. The density and viscosity are functions of pressure and enthalpy according to the fluid thermodynamic property functions.
6. Porous media properties. Porosity, intrinsic permeability, rock density and specific heat are functions of spacial coordinates. Rock specific enthalpy is a linear function of temperature, given by $H_r = c_r T$, where c_r is the specific heat of the rock. Relative permeabilities of the liquid and vapour are functions of phase saturation. They are defined as: $k_{rw} = S_w^2$, $k_{rs} = (1 - S_w)^2$.

Using pressure and enthalpy as primary variables in Equations (1-2), the governing equations are rewritten in reduced parameter form as:

$$\frac{\partial}{\partial x} \left(\tau \frac{\partial P}{\partial x} \right) = C^P \frac{\partial P}{\partial t} + C^H \frac{\partial H}{\partial t} \quad (3)$$

$$\frac{\partial}{\partial x} \left(\lambda \frac{\partial P}{\partial x} \right) + \frac{\partial}{\partial x} \left(\beta \frac{\partial H}{\partial x} \right) = D^P \frac{\partial P}{\partial t} + D^H \frac{\partial H}{\partial t} \quad (4)$$

in which τ , C^P , C^H , λ , β , D^P , D^H are seven highly nonlinear coefficients which can be derived from Equations (1-2) and the previously listed auxiliary assumptions and equations.

Water thermodynamic properties, such as temperature, density and viscosity, are determined from published thermodynamic property functions. The fundamental work in developing the regression equations for a wide range of temperatures and pressures is from the late 1960s. The latest version of regions and equations is called IAPWS-IF97, which is valid for the following ranges: 273.15K ≤ T ≤ 1073.15K, P ≤ 100MPa; and 1073.15K < T ≤ 2273.15K, P ≤ 50MPa (IAPWS 2007). In this paper a full implementation of IF97 formulation is used to calculate water thermodynamic properties as functions of pressure and enthalpy. The derivatives of the thermodynamic properties with respect to

pressure and enthalpy, i.e., $\frac{\partial T}{\partial P}$, $\frac{\partial T}{\partial H}$, $\frac{\partial \rho}{\partial P}$, $\frac{\partial \rho}{\partial H}$, are also developed according to the original IF97 equations.

3. SOLUTION PROCEDURES

The two variables, P and H , specified as the primary unknowns in Equations (3) and (4) uniquely define the thermodynamic state of the system. Once P and H are known, the nonlinear parameters can be calculated using the prescribed equations together with the fluid thermodynamic property functions. Typical plots of τ , C^P , C^H , λ , β , D^P , D^H are shown in Figure 1. It is apparent that (1) these functions are highly nonlinear across the thermodynamic region; and (2) sharp discontinuities exist at the boundary of the two-phase and single-phase zones. These features make the two-phase problem very difficult to solve by conventional numerical methods when liquid and vapor phases coexist, and significant mass exchange between the two phases is occurring.

High nonlinearities and slope discontinuities that exist in the seven nonlinear coefficients in Equation (3) and (4) often cause convergence difficulties and derive severe restrictions on time step size when solving these equations. Furthermore, the hyperbolic character of the two equations often leads to oscillatory behaviour in numerical modelling process. The oscillations could become greatly amplified through calculation of the nonlinear parameters, especially on the phase change boundaries. This is the main reason that simulations of liquid/vapour flows coupled with heat flow and phase change with realistic physical properties are uncommon. To overcome these difficulties, we propose a nonlinear finite element technique in which Equations (3) and (4) are firstly discretized by using a general weighted residual formulation in space, and a second-order Crank-Nicolson differencing method in the time domain; and then a Newton-Raphson iterative scheme solving nonlinear algebraic equations.

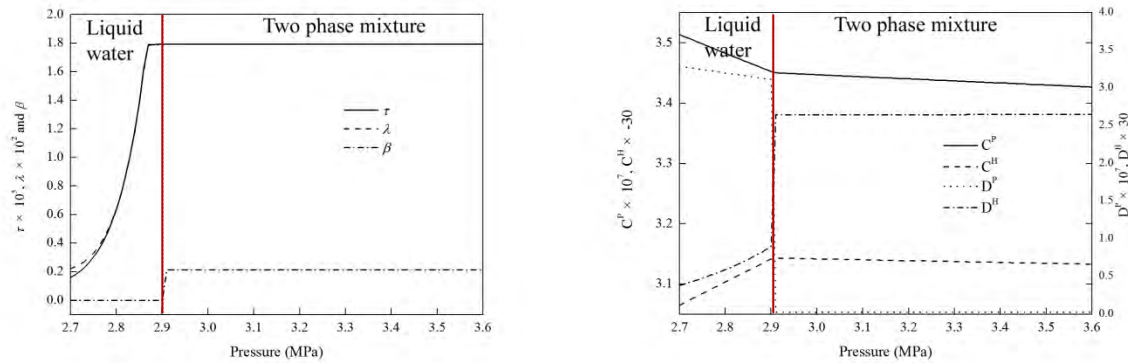


Figure 1: Variation of nonlinear parameters with pressure ($H=1000\text{kJ}$).

A finite element implementation process for solving equation (3) and (4) was proposed in Huyakorn and Pinder's publication(1978). PANDAS ThermoFluid uses a similar derivation, but with a different treatment for the right hand side of the Equations (3) and (4), which result in four nonlinear parameters on the right hand side instead of two. This is for linearization purposes for the Newton-Raphson procedure. On the other hand, a second-order Crank-Nicholson differencing method for the temporal discretization was proposed instead of the explicit method (Huyakorn and Pinder,1978) , which improved the accuracy of the solution. The thermodynamic properties of water are calculated by the IF97 formulation instead of the simplified regression formulae (Huyakorn and Pinder, 1978). The final equation of the Newton-Raphson iteration can be derived as:

$$[A] \begin{Bmatrix} \Delta P \\ \Delta H \end{Bmatrix} = \begin{Bmatrix} -R_m^k \\ -R_e^k \end{Bmatrix} \quad (5)$$

where the stiffness matrix

$$[A] = \begin{bmatrix} \frac{\partial R_m}{\partial P_J} & \frac{\partial R_m}{\partial H_J} \\ \frac{\partial R_e}{\partial P_J} & \frac{\partial R_e}{\partial H_J} \end{bmatrix} \quad (6)$$

and the residuals

$$R_m = \int_V \left[\frac{W_I}{\Delta t} C^P + \frac{\partial W_I}{\partial x_i} \theta \tau \frac{\partial N_J}{\partial x_i} \right] \Delta P_J dV + \int_V \frac{W_I}{\Delta t} C^H \Delta H_J dV + \int_V \frac{\partial W_I}{\partial x_i} \tau \frac{\partial N_J}{\partial x_i} P_J dV - \int_V W_I q_m dV \quad (7)$$

$$R_e = \int_V \left[\frac{W_l}{\Delta t} D^p + \frac{\partial W_l}{\partial x_i} \theta \lambda \frac{\partial N_j}{\partial x_i} \right] \Delta P_j dV + \int_V \left[\frac{W_l}{\Delta t} D^H + \frac{\partial W_l}{\partial x_i} \theta \beta \frac{\partial N_j}{\partial x_i} \right] \Delta H_j dV + \int_V \frac{\partial W_l}{\partial x_i} \lambda \frac{\partial N_j}{\partial x_i} P_j^t dV + \int_V \frac{\partial W_l}{\partial x_i} \beta \frac{\partial N_j}{\partial x_i} H_j^t dV - \int_V W_l q_e dV \quad (8)$$

and the derivatives of the residuals $\frac{\partial R_m}{\partial P_j}$, $\frac{\partial R_m}{\partial H_j}$, $\frac{\partial R_e}{\partial P_j}$, $\frac{\partial R_e}{\partial H_j}$ can be derived from Equations (7) and (8), where N_j and W_l are the basic shape functions and weighting functions. θ is the implicitness factor of the Crank-Nicholson scheme, $0 < \theta < 1$. Δt is the time step size and ΔP and ΔH are pressure and enthalpy increments. The superscript t denotes the value from last time step.

3.1 CONVERGENCE CONTROL AND PHASE CHANGE CONTROL

Equation (5) is solved by the mentioned iterative procedures to convergence. Three criteria are applied to determine convergence of the proposed Newton-Raphson algorithm. The primary convergence criterion is that the infinity norms of the fluid mass residual rates and the energy residual rates for all the nodes are less than a given threshold. Nodes with specified pressure and enthalpy boundary conditions are not included in the convergence test because their residuals do not decrease as the solution is approached. Their residuals determine the flow rates through the regional boundary faces.

If phase change occurs in the current time step and three iteration steps have been taken, the second convergence criterion is applied. This requires that the infinity norms of the relative changes in pressure and enthalpy over an iteration are less than the given threshold.

If phase change occurs in the current time step and four iteration steps have been taken, the third convergence criterion is applied, which is that the infinity norm of the absolute change in pressure is less than 1 kPa and the infinity norm of the absolute change in enthalpy is less than 0.01kJ/kg. This criterion is used in the circumstances that the Newton-Raphson algorithm has become stable despite the residual rates exceeding the given tolerance.

A phase change control module is used to assist with the model convergence. For any nodes which have changed phase state during the iteration, the pressure and enthalpy are adjusted mandatorily to near the phase boundary. Assuming the old pressure and enthalpy are (P_0, H_0) and the newly calculated pressure and enthalpy are (P_1, H_1) , the phase change control module employs the following algorithm (as shown in Figure 2): i. Calculate the pressure step increment δP and δH by dividing the pressure and enthalpy increment (i.e., $P_1 - P_0$, $H_1 - H_0$) by 100, i.e., $\delta P = (P_1 - P_0)/100$ and $\delta H = (H_1 - H_0)/100$; ii. Let $P_2 = P_0$ and $H_2 = H_0$; iii Repeat step iv-v until the loop terminates; iv. $P_2 = P_2 + \delta P$, $H_2 = H_2 + \delta H$; v. For the first iteration, if phase state at (P_2, H_2) is different with that of (P_0, H_0) , terminate the loop; for the second and the third iterations, if phase state at (P_2, H_2) is different with that of (P_0, H_0) , $P_2 = P_2 - \delta P$, $H_2 = H_2 - \delta H$, terminate the loop; vi. (P_2, H_2) are the modified pressure and enthalpy for the phase change node after applying the phase change control module.

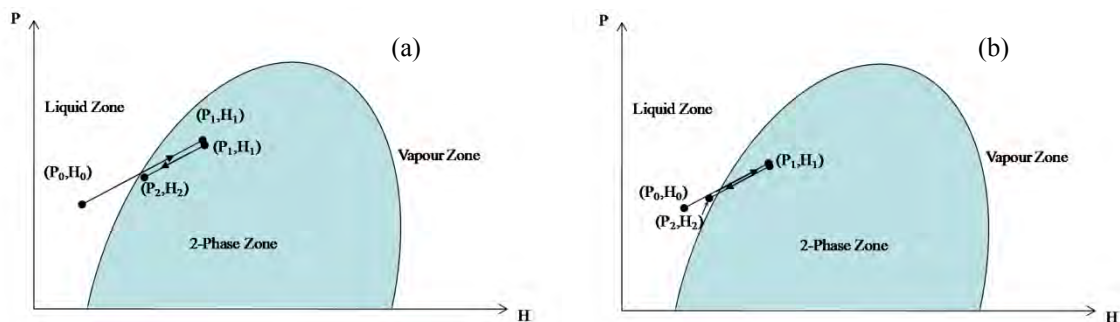


Figure 2: Schematic diagram of the phase change control module for (a) first iteration; (b) second and third iterations.

The proposed algorithm with mandatory adjustment of pressure and enthalpy is to slow the phase change process and avoid irrational jumps of pressure and enthalpy during the phase change. It is implemented in the current finite element model and is useful for avoiding the convergence issues for solving nonlinear coupled heat and fluid flow equations with phase change. Its impact on the entire solution set is minor as it is only applied to very limited number of nodes with phase change and it does not include any mandatory convergence control.

3.2 AUTOMATIC TIME STEP CONTROL

Automatic time step control is essential for strongly nonlinear coupled equations. The nonlinearity existing in equation (5) is severe and the parameters change rapidly at the phase change boundaries. Small time steps are required for a

stable computation and an accurate solution when the parameters change rapidly at phase change. To control the time step size at the moment of phase change, a simple but efficient algorithm is applied as followed when phase change occurs. A relatively large time step size is used at the beginning of the simulation. If phase change takes place in certain time step, the following criterion determines the number of phase change nodes (pc). If pc is greater than a threshold number of phase change nodes per time step (pc_{max}), the time step size has to be cut by a factor of α ($0 < \alpha < 1$). The multiplier α is the greater value between 0.5 and pc_{max}/pc , which means the updated time step size is always greater or equal to 50% of the current time step size. The current time step is calculated with the updated time step size in a loop until the number of phase change nodes is equal or below the given limit.

The proposed convergence criteria are applied together with the phase change module and time step size control module to find a stable solution with acceptable accuracy for a given time step under a wide variety of pressure, enthalpy, temperature and phase conditions.

4. AN APPLICATION

To demonstrate the feasibility and value of multi-phase fluid flow modeling as an advanced approach for risk analysis of geothermal hazards during mining in hot ground, a numerical model was developed for an open-cut mine in a geothermally active area using PANDAS ThermoFluid and FEFLOW. The mine excavation extends below sea floor and mining is scheduled to start after construction of a coffer dam to prevent ingress of seawater into the proposed pit. Figure 3 provides a schematic hydrogeological section through the mine area. A summary of the material parameters is shown in Table 1. For demonstration purpose three pit excavation stages were simulated. The pit is assumed to be excavated to -90m MSL bench in the first stage. In a second stage the pit is assumed to be excavated to -180m MSL bench reaching the top of the Boiling Zone. The pit is assumed to be excavated to -270m MSL depth in the final stage. FEFLOW was employed to predict the phreatic surface at the start of each pit excavation stage. The pressure results were transferred into PANDAS ThermoFluid as the initial pressure of each stage for simulation of temperature, pressure and steam occurrence in the pit seawall and foundation of the coffer dam along the cross section.

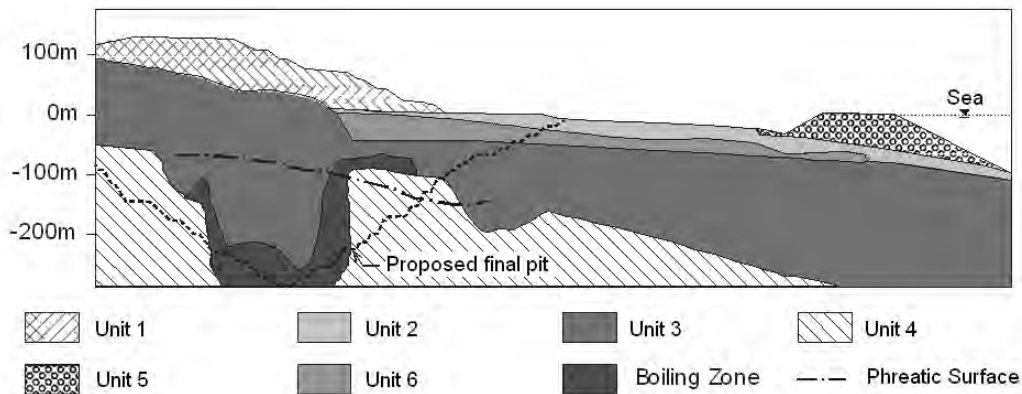


Figure 3: Hydrogeological cross section of an open-cut mine in a geothermally active area

Hydrogeological Unit	Permeability (m^2)		Heat Capacity λ_{dry} (J/kg·°C)	Thermal Conductivity λ_{sat} (W/m·°C)	Effective Porosity
	K_x	K_y			
Unit 1	4.8×10^{-10}	4.8×10^{-10}	850	1.0	0.30
Unit 2	1.73×10^{-12}	1.73×10^{-12}	850	1.0	0.20
Unit 3	8.16×10^{-15}	8.16×10^{-15}	850	1.5	0.10
Unit 4	3.27×10^{-15}	3.27×10^{-15}	850	2.9	0.08
Unit 5	8.16×10^{-15}	8.16×10^{-16}	850	1.2	0.30
Unit 6	2.45×10^{-13}	2.45×10^{-14}	850	1.1	0.12
Boiling zone	3.27×10^{-12}	3.27×10^{-12}	850	2.6	0.10
Water	-	-	2000-4200	0.65	-
Air	-	-	-	0.024	-

Table 1: Material parameters used in the open-cut pit cross section model.

The results of the modelling are shown in Figure 4. It was found that after deepening the pit by 90m steam did not develop throughout the model domain despite computed temperatures of over 100°C in some areas. This is due to high hydrostatic pressures preventing water from boiling. The simulation results for the second and third development stages

predicted boiling to occur below the pit floor and in the lower benches of the seawall. During the second development stage, modelling showed that the pit would cool relatively quickly and that the initial steam region would not be present after 2.7 months. However, after the third development stage, boiling continued to occur long-term in the base of the pit. After 1.4 month, a quasi steady state condition had developed in the pit.

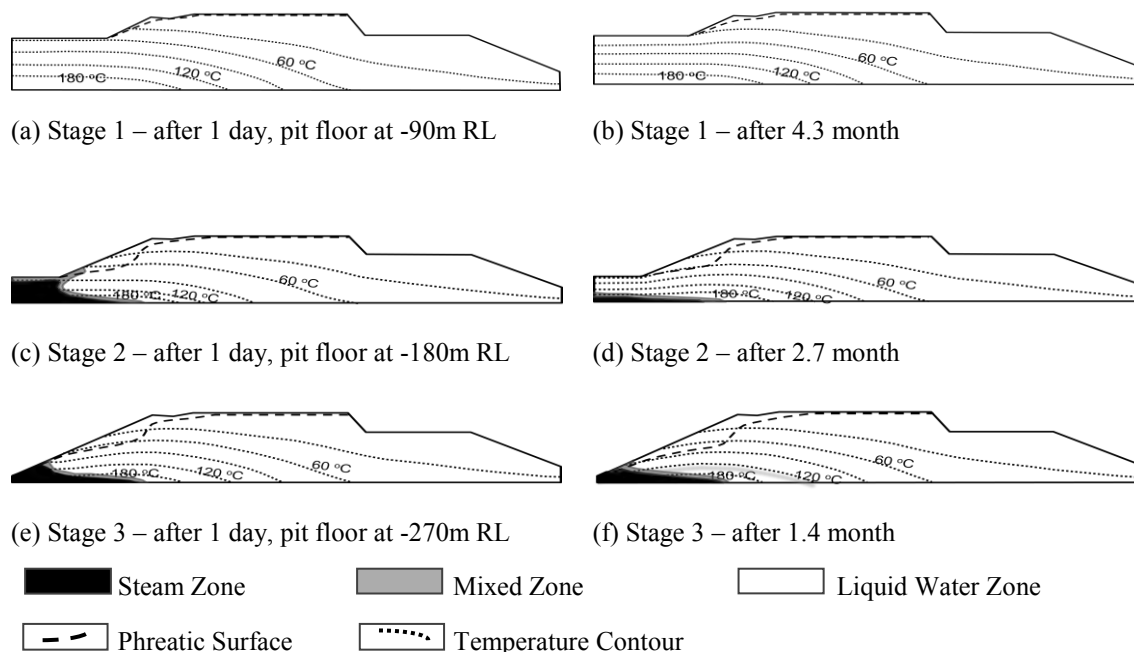


Figure 4: Numerical results of the pit cross section model for three development stages.

5. CONCLUSIONS

A finite element model PANDAS ThermoFluid for simulation of liquid/vapour two-phase coupled fluid and heat flow in porous media with phase change is introduced. High nonlinearities and severe slope discontinuities existing in the coefficients of the governing partial differential equations make them difficult to solve, especially with phase changes. A Newton-Raphson method is employed to solve the highly nonlinear coupled equations. The model is coupled with FEFLOW to simulate the phreatic surface, temperature, pressure and steam occurrence of a coastal mining field located at a geothermal active area. The modelling results demonstrated that the developed approach as a valuable tool can be used to support the development and implementation of geothermal risk management strategies to combat geothermal hazards.

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6. REFERENCES

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Please find my response to the submission topic for the 2012 Young Geo-professional competition.

I currently use in situ testing on a regular basis as an integral part of soft soil investigations for broad scale geotechnical studies for large housing/commercial subdivisions. The model that we employ is to test a large number of locations across the site with CPT tests, and then use boreholes in selected locations to collect samples for laboratory testing and to aid in site specific correlation of soils types, all in conjunction with established correlation charts.

The value of in situ testing, in relation to these types of studies, is its cost effectiveness and the high quality of the data in situ testing provides. Because of in situ testing I am able to test a higher density of locations which allows me to identify and quantify geotechnical risks associated with the project with greater confidence. I can use this methodology to assist with the construction programme of sites by identifying areas of shallow/deep runs of compressible soils and by developing soft clay contour maps. Also, I am better able to target areas that warrant further investigation.

Overall I am able to provide better quality analysis of potential geotechnical risks associated with each development in a very cost effective manner by employing in situ testing techniques, in comparison with a traditional borehole investigation.

Although the determination of soil parameters from CPT and dilatometer testing is very accurate, it does not fully replace the need for traditional laboratory testing as a part of a proper geotechnical investigation. Thus the increased capabilities of CPT rigs to collect high quality, large diameter undisturbed samples for laboratory strength and consolidation tests is an exciting development. This development reduces our reliance on boreholes for sampling purposes, thus further reducing costs associated with geotechnical investigations.

The potential cost savings, with being able to accurately determine critical soil parameters required for preloading, retaining wall and pile design via dilatometer testing is phenomenal. I foresee an increased demand for, and reliance on this type of testing into the future.

In situ testing already has an untouchable position in the market as being the most cost effective means in providing large volumes of high quality geotechnical data, thereby reducing uncertainty in design and construction and potentially significantly reducing overall project costs.

I believe the future for in situ testing is bright; as improvements in CPT rigs' sampling capabilities, coupled with increased use of dilatometer testing for determining critical soil properties, will further reduce our reliance on more traditional and expensive investigation techniques.

Yours Faithfully,



Geotechnical Engineer
For Cardno Bowler

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