



July 2004

Grand Mecure Hotel
Gold Coast, Queensland Australia

**6th Young Geotechnical
Professionals Conference**

6th young geotechnical professionals conference

Proceedings of

**The 6th Australia New Zealand Young
Geotechnical
Professionals Conference**



Australian Geomechanics Society



New Zealand Geotechnical Society

Welcome

Welcome to the *Sixth Australia – New Zealand Young Geotechnical Professionals Conference*, Gold Coast, Australia, 2004. Following on from previous successful conferences held in Sydney (1994), Auckland (1996), Melbourne (1998), Perth (2000) and Rotorua (2002), this biannual event is a joint initiative of the Australian Geomechanics Society and the New Zealand Geotechnical Society.

The aim of the conferences is to:

- promote the professional development of delegates through sharing experience and ideas, and by presenting a paper to senior professionals and peers.
- expand and strengthen the lines of communication between young professionals within the field of geomechanics.
- promote an enhanced perspective of the varied roles, responsibilities and opportunities encompassed by the geotechnical profession;
- showcase the range of geotechnical projects currently underway in South East Queensland, highlighting the challenges facing geotechnical professionals.

The specific goal of the organizing committee has been to prepare an interesting, meaningful, informative, and enjoyable conference from which delegates can return with an enhanced understanding of their profession.

As with all previous YGP conferences, support of the geotechnical community is very important to ensure a successful end result. We would like to thank all companies and individuals that have provided assistance with the preparation of the conference. In particular, committee members: Symon Jackson, Patrick Kidd and Troy Weingarth, who have contributed considerable time and effort over the past 2 years. Thanks also to all those involved in paper review on behalf of delegates. Special thanks also to Vale Lee and Andrew Leventhal of the AGS and Max Ervin for their assistance and guidance in organizing this event.

Keynote presenters and Senior Mentors: Mr Patrick Wong, Mr Chris Haberfield and Mr John Simmons; without your valuable input the conference would lack much of what makes it such a success. Mr John Simmons will be acting as 'tour guide' for our field trip which is much appreciated and we look forward to an informative and pleasant afternoon.

Delegates should interact with their peers openly and maximize their exposure to the geotechnical community during the conference. A short question time has been allocated to each talk and I also encourage delegates to ask questions during this period. Workshop will be held on the last day of the conference and I look forward to interesting debates during this session.

Most of all, have an rewarding time at the conference. The organizing committee members look forward to meeting you all, and joining you in an enjoyable and informative three days.

Karen Allan
Chair – Organising Committee

Organising Committee

Ms Karen Allan
Mr Symon Jackson
Mr Patrick Kidd
Mr Troy Weingarth

Mentors

Mr Chris Haberfield
Mr Patrick Wong
Mr John Simmons

Special Thanks

Ms Val Lee
Mr Max Ervin
Mr Andrew Leventhal

Each delegate attending this conference was nominated by a senior mentor. As part of the paper submission process, each paper written by the delegates was reviewed by their mentors for technical accuracy, relevance and presentation.

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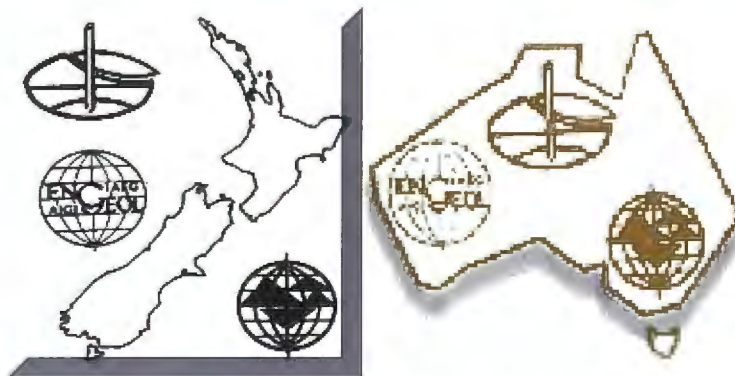
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As part of its role, EQC is delighted to support the 6th Australia-New Zealand Young Geotechnical Professionals Conference on the Gold Coast.

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Optimisation of Surface Wave Measurements in a Solid Waste Landfill Using Geostatistical Methods

Sina Avsar, Department of Civil Engineering, Monash University, Melbourne, Australia

Abstract: In solid waste landfill site characterization studies in-situ techniques are more useful and reliable because of the difficulties related to taking undisturbed samples from the site. Among these techniques spectral analysis of surface waves (SASW) takes an important place. Spectral analysis of surface waves (SASW) is a non-intrusive method for subsurface investigation that uses surface wave dispersion to indirectly determine compression and shear wave velocities. The non-intrusive nature of the method eliminates many of the health and safety concerns typically associated with conventional borings for geoenvironmental investigations. However, like any other site investigation technique, SASW profiles are generally applicable strictly only at the locations at which they are developed.

In this paper geostatistical methodology has been used to interpolate the shear wave velocity measurements taken from discrete locations in a solid waste landfill. Kriging technique was used for interpolation. This application provides a better understanding of the stiffness distribution along the site.

INTRODUCTION

Spectral analysis of surface waves (SASW) is a non-intrusive method for subsurface investigation that uses surface wave dispersion to indirectly determine compression and shear wave velocities. Advantages of SASW measurements include that they are non-intrusive, are made at known levels of stress and strain ($< 10^{-4}$ %), and involve essentially no material disturbance. SASW makes use of Rayleigh (surface) waves which propagate within a zone approximately one wavelength in depth (Stokoe et al., 1994). In ground where the stiffness changes with depth, these surface waves are dispersive in nature, which means that they travel at a velocity that is dependent upon frequency or wavelength. In SASW, analysis of the dispersion of surface waves of different frequency yields a profile of shear wave velocity versus depth. Knowledge of the shear wave velocity profile is valuable in a wide range of geotechnical problems, both directly (e.g., in a seismic response analysis) and indirectly (e.g. in evaluation of site stratigraphy). Variations of material stiffness in shear (i.e., in shear wave velocity) are often related to variations in other important properties, including density and material type (Luke and Stokoe, 1998). SASW is a particularly attractive method of investigation for landfill engineering because the non-intrusive nature of the method eliminates many of the health and safety concerns typically associated with conventional borings for geoenvironmental investigations. However, like any other site investigation technique, SASW profiles are generally applicable strictly only at the locations at which they are developed.

This paper presents a geostatistical approach to mapping (interpolating) the spatial distribution of shear wave

velocity at a given depth over the entire site from SASW measurements made at a series of discrete locations.

FIELD MEASUREMENTS

The study area (the Operating Industries, Inc., or OII, Landfill site) is located approximately 16 km east of downtown Los Angeles. The Pomona Freeway divides the 77-hectare site into two parcels: the North Parcel and the South Parcel. Landfilling operations began at the site in 1948. Over the life of the landfill, many types of waste were disposed of, including residential and commercial refuse, liquid wastes, and various hazardous wastes. In January 1984, the State of California placed the OII South Parcel on the California Hazardous Waste Priority List. The landfill stopped accepting wastes and was closed in late 1984. The U.S. Environmental Protection Agency (EPA) placed the OII South Parcel on the "Superfund" National Priority List in the same year and began conducting studies and taking actions to protect the environment, those who live near the site (approximately 23000 people live within 5 km of the site and 2100 people live within 300 m of the landfill (USEPA, 2002)), and the tens of thousands of people who commute past the site on the Pomona Freeway every day from hazards posed by the landfill. "Pre-design" geotechnical activities commencing in 1994 culminated in construction of an engineered "final cover" over the South Parcel in 1997 and 1998. These pre-design activities included a series of SASW measurements across the site. The SASW test locations in the OII site are presented in Figure 1.

In SASW testing, the ground is excited by impulse loading and ground response is measured at two or more receivers located along a common line from the energy source. Both the energy source and surface wave receivers employed in SASW testing are located on the ground surface. The depth of penetration of the surface

wave is function of the frequency and velocity, or wavelength, of the surface wave. An inversion procedure is used to determine the V_s profile of the ground from the frequency-Rayleigh wave velocity relationship deduced from the recorded ground response spectra. The frequency – Rayleigh wave velocity relationship is converted to a shear wave velocity profile based upon an assumed value of Poisson's Ratio (Kavazanjian et al, 1996, Bouazza and Kavazanjian, 2000).

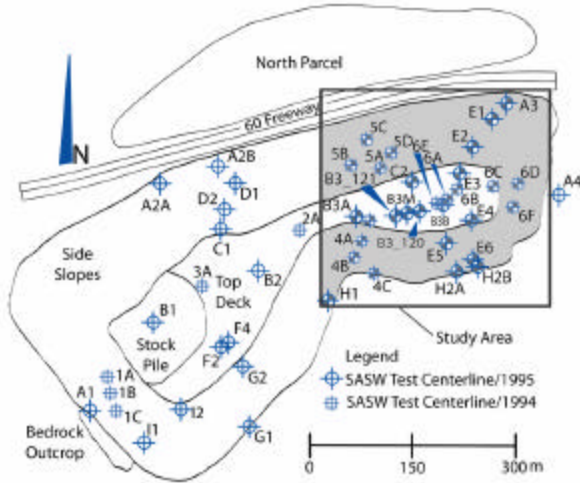


Figure 1. SASW testing locations at the OII site (GeoSyntec Consultants, 1996)

SPATIAL DATA DESCRIPTION

SASW measurements were made at a total of 41 different locations on the landfill. Twenty five (25) of these locations were underlain by at least 30 m of waste. Shear wave velocities (V_s) obtained for a depth of 9 m from these 25 points were employed to illustrate the application of geostatistics for interpolating the spatial distribution of the shear wave velocity at a depth of 9 m over the east part of the landfill as seen in Figure 3. In the work of Avsar et al, (2003), the same landfill area has been modelled using only the data from 1995 tests. However, in this paper another dataset has been combined to the 1995 data in order to improve the quality of the model. Also in the current study only the east part of the landfill has been modelled. The comparison of two different models of the same area is discussed in the final section of this paper.

In this paper the west part of the study area has been split out. The data number has been increased combining a new dataset to the 1995 data. Thus the east part of the study area could be modelled with sufficient number of data. The comparison of old and new datasets has shown that the stiffness of the material in the eastern part has a relatively smaller variability.

Table 1: Description of V_s for the study area.

Parameters	V_s (m/s) 1995 (Avsar et al, 2003)	V_s (m/s) 1994 and 1995
Mean	179.66	166.8
Std deviation	39.37	23.03
Sample variance	1550.10	530.79
Minimum value	134.42	134.42
Maximum value	268.53	218.20
Skewness	0.90	0.42
Kurtosis	-0.08	-0.74

As seen in Table 1, after combining the 1994 and 1995 values the sample variance and standard deviation has reduced the data to smaller values. This improved the reliability of the dataset, and it provided a good base for further modelling steps.

GEOSTATISTICAL METHODOLOGY

The theory of regionalized variables (ReV) can be used to represent the distribution of natural phenomena in any given space. This theory provides an estimate of the value of the ReV at every point in the study area, both sampled and unsampled. The geostatistical methodology for assessing a ReV employs two steps to evaluate the spatial distribution and reduce the uncertainty of the ReV. The first step of the geostatistical approach is to set up a mathematical function that describes the spatial dependence of the ReV called the semivariogram function (SV) (Matheron 1963). The SV can be evaluated from the field data using equation 1 (Journel and Huijbregts 1978, Isaaks and Srivastava 1989, Sen 1998, Avsar et al. 2002):

$$\gamma(h) = \frac{1}{2N} \sum_{i=1}^N [Z(x+h) - Z(x)]^2 \quad (1)$$

where;

$\gamma(h)$ = Semivariogram,

N = the number of pairs,

$Z(x)$ = magnitude of the ReV, and

$Z(x+h)$ = magnitude of the ReV at a point separated from the point representing $Z(x)$ by distance h .

In the second step of the geostatistical approach, once the SV function is obtained from the ReV values in the study area, kriging is applied to complete the estimation of the spatial distribution of the ReV throughout the study area. To obtain the ReV values of the unsampled points in the study area, the following equation can be used (Isaaks and Srivastava 1989).

$$Z(x_0) = \sum_{i=1}^m w_i Z(x_i) \quad (2)$$

where;

$Z(x_0)$ = the magnitude of the unsampled point,
 $Z(x_i)$ = the magnitude of sampled point i , and
 w_i = the weight coefficient for sampled point i .

Equation 2 linearly relates the magnitude of any unsampled point to the magnitude of all of the sampled points as a function of distance from the sampled points.

The values of γ_{ii} and γ_{vi} can be determined using the theoretical SV function obtained in the first step. The weight coefficients can be obtained from the following matrix by a matrix inversion procedure and after that any desired unsampled point's value can be obtained from equation 2. By applying this procedure it is possible to estimate the value of the natural phenomena at any point in the study area.

The relationship between the sampled and unsampled points is described by equation 3 (Isaaks and Srivastava 1989):

$$\begin{vmatrix} \gamma_{11} & \gamma_{12} & \cdot & \gamma_{1n} & 1 \\ \gamma_{21} & \gamma_{22} & \cdot & \gamma_{2n} & 1 \\ \cdot & \cdot & \cdot & \cdot & \cdot \\ \gamma_{n1} & \gamma_{n2} & \cdot & \gamma_{nn} & 1 \\ 1 & 1 & \cdot & 1 & 0 \end{vmatrix} \begin{vmatrix} w_1 \\ w_2 \\ \cdot \\ w_n \\ \mu \end{vmatrix} = \begin{vmatrix} \gamma_{v1} \\ \gamma_{v2} \\ \cdot \\ \gamma_{vn} \\ 1 \end{vmatrix} \quad (3)$$

where:

γ_{ii} = the value of SV between every sampled points,
 w_i = the value of weight coefficient for unsampled points,
 γ_{vi} = the value of SV between unknown point and the sampled points, and
 μ = is the Lagrange parameter.

APPLICATIONS

Experimental and theoretical SVs were developed for the shear wave velocity at a depth of 9 m in the study area (OII landfill) in order to describe the spatial dependence of V_s . As mentioned in the previous section combined dataset are used for geostatistical analysis. In order to develop a geostatistical model, first, an experimental SV is derived from the given data set and a theoretical SV is fitted to this experimental SV. A spherical model for theoretical SV was selected because the variation of the data seemed to be more logarithmic than symmetric

(Figure 2). After that, kriging technique is run to map the spatial distribution of V_s along the study area.

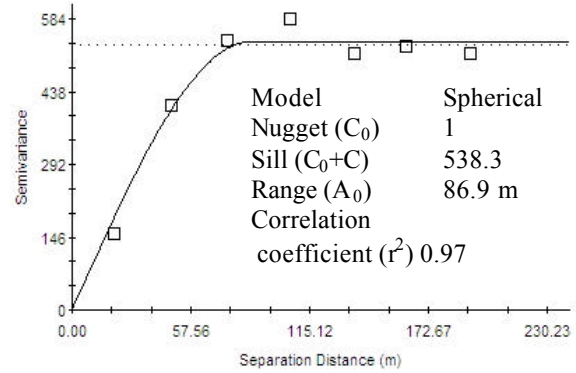


Figure 2 Semivariogram of V_s values at 9 m depth

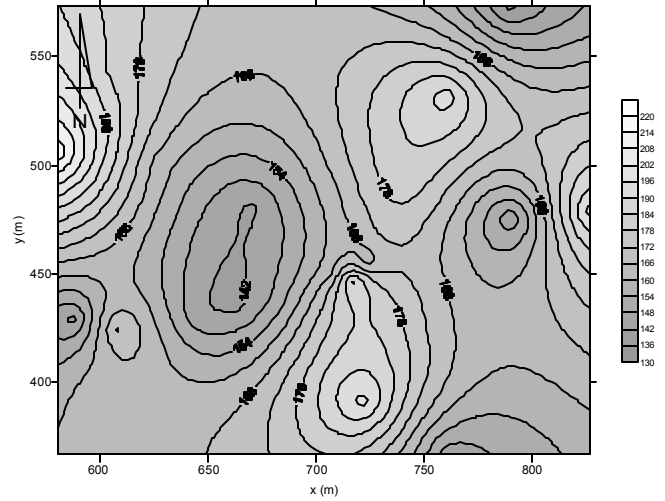


Figure 3 Kriging model of V_s values at 9 m depth

The SV models given in Figure 2 show that V_s is a highly correlated parameter because of the highly correlated theoretical SV. The variogram parameters can be seen in Figure 2. The correlation between theoretical and experimental SV is very good and this is important to make a more reliable kriging model.

The current model for 9 m depth given in this study was compared to the model published by Avsar et al (2003). As a result of this comparison it can be concluded that a better model has been achieved using the combined dataset and including eastern part of the study area. The possible reason for this improvement in the model is due to the use of a smaller study area but with denser data point distribution and decreased variation of the new dataset.

The mapping of V_s at a depth of 9 m shown in Figure 3 provides an estimation of the zones where the waste is compact and where it is less compact. The distribution of the V_s was found to vary between 130 and 220 m/s.

CONCLUSIONS

In this paper, a geostatistical methodology was used to estimate the spatial distribution of shear wave velocity (V_s) in a solid waste landfill for both sampled and unsampled points. A previous model in the same area has been improved with additional data. Geostatistical methodology has provided a better understanding the distribution of the V_s along the study area. The same geostatistical methodology can be used for the interpolation of other in situ measurements (i.e. pressuremeter, inclinometer, etc.) or to find correlations with other parameters such as unit weight, shear strength, etc.

ACKNOWLEDGEMENTS

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REFERENCES

1. Avsar, M.S., Öztürk C.A., Bouazza A. (2002) Comparison of kriging and point cumulative semivariogram techniques to make spatial description for a given deposit. Proceedings 9th Congress of the International Association of Engineering Geology and the Environment, Durban.
2. Avsar M.S., Bouazza A., Kavazanjian E. Jr. and Öztürk C.A. (2003) Interpolation of solid waste shear wave velocity using geostatistics. Proceedings XIIIth European Conference On Soil Mechanics And Geotechnical Engineering, Prague, 25-28th August, Vol.1, pp.21-26.
3. Bouazza, A. and Kavazanjian, E. (2000). Characterization of municipal solid waste sites using the continuous surface wave method. Proceedings. GeoEng2000, Melbourne, CD Rom
4. GeoSyntec Consultants (1996). Pre-design analyses of the OII landfill. Technical Report Prepared for the New Cure, Inc., Monterey Park, California for submission to the USEPA Region IX. GeoSyntec Consultants, Huntington Beach, California. 9 Volumes.
5. Isaaks, E. H. and Srivastava, R. M. (1989). An introduction to applied geostatistics. New York: Oxford University Press.
6. Journel, A. G. and Huijbregts, Ch. J. (1978). Mining geostatistic, London: Academic Press.
7. Kavazanjian, E., Matasovic, N., Stokoe, K.H. and Bray, J.D. (1996). In-situ shear wave velocity of solid waste from surface wave measurements. Proceedings 2nd International Congress Environmental Geotechnics, Osaka, Japan, Vol. 1, 97-104.
8. Luke, B.A. and Stokoe, K.H. (1998). Application of SASW method underwater. J. Geotech. Engrg., ASCE, 124(6), pp.523-531
9. Matheron, G. (1963). Principles of geostatistics. Economic Geology, 58,1246–1266.
10. Sen, Z. (1998). Point cumulative semivariogram for identification of heterogeneities in regional seismicity of Turkey. Mathematical Geology, 30, 767 – 787.
11. Stokoe, K.H., Wright, S.G., Bay, J.A., and Roesset, J.M. (1994). Characterization of geotechnical sites by SASW method. Geophysical Characterization of Sites, International Science Publishers, New York, 15-26.
12. USEPA (2002) Open Report of Operating Industries, Inc. Landfill, California, EPA ID# CAT 080012024.

Field Assessment of the Strength and Deformation Properties of Limonite

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SUMMARY Detailed field investigations have recently been undertaken in the limonitic tropical soils of the South Pacific Region for a large industrial complex in New Caledonia. In situ investigations included piezo-Cone Penetrometer Tests (CPTU), Standard Penetration Tests (SPT), field and hand vane, pocket penetrometer tests, Menard pressuremeter testing and cross-hole seismic testing. Deformations of subsurface material under trial surcharge loading were measured using a magnetic extensometer. Several of the field techniques outlined were performed in close proximity allowing comparison of the interpreted soil properties through standard correlations. This paper compares and discusses the variations in soil properties assessed from different field techniques and compares them to properties assumed from observations of the soil under surcharge loading from a trial embankment.

1 INTRODUCTION

The site of a proposed large industrial development is located over 8000 hectares in the south eastern region of New Caledonia. New Caledonia is an island off the east coast of Australia, approximately 400 km long by 50 km wide, lying between 20 and 22 degrees latitude and 164 and 167 degrees longitude. The island experiences tropical weather patterns with an average annual rainfall of more than 3,100 mm (as recorded in the south east of the island). Extensive clearing of the land occurred during early European settlement and many slopes are deeply scarred by landslips and erosion. A relatively unique feature of the area are the dolines - dissolution cavities similar in nature to karsts in limestone terrain - that vary in size from less than a metre to tens of metres in diameter.

2 GEOLOGICAL SETTING

The site of the proposed industrial complex is underlain by the Late Eocene age Massif du Sud, Guillon (1). The Peridotite Massif du Sud is part of a larger structural unit, the Ultramafic Nappe that is present along both the eastern and western side of the island. Extensive faulting has occurred. Most of the large folds and faults follow the elongation of the island, striking at 110-130°, Brothers (2). Peridotite is a fine grained igneous rock, formed on the sea floor. It is dark grey green in colour and rich in iron and magnesium.

A soil profile that can extend to more than 50 m thickness has formed through the weathering and leaching of the Peridotite bedrock (Fookes (3)). Nickel and other minerals are leached from the upper layers of the Peridotite and concentrated lower in the weathering profile (Golder Associates (4)). Directly overlying the Peridotite is Sapolite, consisting of Peridotite gravel and corestones in a matrix of clayey silt soil. Limonite is present higher in the profile, overlying the Sapolite, where the Peridotite has been weathered completely into clayey silt. The Limonite has been leached of nearly all the silicate in the parent rock and is composed primarily of iron minerals. This extensive leaching has resulted in soils with high void ratio and moisture content that are stable in situ (Stocker (5)).

The Limonite exhibits considerable structure, seen in the alignment of minerals, along which the Limonite will preferentially shear. A cemented ironstone cap (Ferricrete) has formed at the surface in most of the plateau areas. The Ferricrete is of variable thickness and has been reworked in some areas whilst in others it can be seen to be forming over a period of some months.

On slopes and in erosion gullies the soil profile can be thin to non-existent with exposed Peridotite rock. Fans or thin sheets of alluvium have formed in some areas where soil has been washed off ridges and where landslips have occurred.

The movement of the closely related ground and surface water has led to the formation of voids in the Peridotite rock and the weathered profile. Dolines are formed when the voids reach the ground surface leading to surface collapse.

3 FIELD TECHNIQUES

Obtaining suitable field investigation equipment was difficult in New Caledonia where their use is limited. This led to the trial of various techniques but the majority of the investigation was performed using standard drilling and sampling techniques. Standard Penetration Tests (SPT) were performed in situ and pocket penetrometer and hand vane tests were carried out on recovered undisturbed tube samples in the cohesive soils. Further investigations were performed adjacent to many of the boreholes including piezo-Cone Penetration Tests (CPTU) and vane shear testing. Testing was often performed in close proximity due to the high density of the field investigations. Limited pressuremeter and cross-hole seismic testing was also performed at selected locations. Extensive laboratory testing was also performed on samples collected from the boreholes, but the results of these tests are outside the scope of this paper.

3.1 Standard Penetration Tests (SPT)

Standard penetration testing, while common in Australia, is not typically performed in New Caledonia. The local drillers were on a steep learning curve with equipment that was foreign to them and many errors were made in the early stages, including dropping

sampling rods. Conveying the importance of accurate sample depths and cleanliness of the borehole was made more difficult by language differences. These difficulties reduced the confidence that could be placed in the results obtained from these tests. In soft cohesive soils the penetrometer often sank under the self-weight of the rods or the rods plus the hammer. SPTs were used in the clayey silt Limonite to provide an indication of shear strength and Young's modulus based on the measured N value. Table 1 gives inferred soil consistencies based on the observed N values.

SPT N Blows per 300 mm	Shear Strength (kPa)	Consistency
0 to 2	< 12.5	Very Soft
2 to 4	12.5 to 25	Soft
4 to 8	25 to 50	Firm
8 to 15	50 to 100	Stiff
15 to 30	100 to 200	Very Stiff
> 30	> 200	Hard

Table 1. Inferred Shear Strength and Consistency based on SPT N values

Within the range of recorded SPT N values, Table 1 can be simplified to the following correlation between shear strength and N value.

$$c_u = 6.25N, \text{ kPa}$$

Several empirical correlations have been suggested (Blight (6)) to estimate the Young's Modulus, E , of a soil from the SPT N value. The correlation used for this paper was:

$$E = 1.6N, \text{ MPa}$$

3.2 Hand Vane

Undisturbed tube samples of approximately 50 mm internal diameter were taken at regular intervals in the Limonite. Selected samples were tested using a Pilcon hand held shear vane with either a 33 mm or 19 mm vane. The vane is imbedded in the soil and turned slowly, the dial reading is used to assess the shear strength of the soil. The shear strength is typically given by the following equation (Richards (7)):

$$S_v = T/K$$

where $K = \pi d^2 H/2 + \pi D^3/6$
 $T = \text{torque at failure}$
 $D = \text{Diameter of the shear vane}$
 $d = \text{diameter of rods}$
 $H = \text{height of shear vane}$

For the vanes used the shear strength for a given dial reading could be read off a graph devised based on the constant K , and corrected for the vane calibration. The tube samples were transported from site to the laboratory at the end of each day where the hand vane tests were performed. Some disturbance of the Limonite may have occurred during sampling and transportation of the tubes on rough roads.

3.3 Pocket Penetrometer

The undrained shear strength of all tube samples collected was also assessed using a pocket penetrometer. Pocket penetrometer tests were performed in the field directly following the collection of the sample, reducing disturbance due to transportation. Several readings were taken and

averaged to reduce the effect of the small scale of the test. Soil shear strength was assumed to be half of the measured pocket penetrometer reading.

Collection of the Limonite in an undisturbed state from the boreholes was often very difficult. Best recovery was achieved when tubes were inserted then left for several minutes to allow the sample to stabilise. The sample was rotated using a wrench on the rods at ground level before being lifted smoothly to the surface.

3.4 Piezo-Cone Penetrometer (CPTU)

A CPTU was performed adjacent to nearly all of the boreholes drilled during this field investigation. The CPTU test comprises pushing a probe into the soil at a steady rate and measuring the tip resistance, shaft friction and pore water pressure. The undrained shear strength can be estimated from the tip resistance, corrected for overburden, via the following equation from Robertson & Campanella (8):

$$c_u = \frac{q_T - \sigma_{v0}}{N_K}$$

where: $q_T = \text{corrected tip resistance}$

$\sigma_{v0} = \text{overburden pressure}$

$N_K = \text{cone factor, typically 15}$

The Young's Modulus can again be estimated using empirical correlations, with the following relationship (Stocker (5)), adopted in this study.

$$E = 20 q_T$$

Malfunction of the electrical equipment due to the high temperatures and humidity experienced on site resulted in lost and erroneous data. The rate of penetration was controlled by the drillers and often varied markedly from the standard 20 mm/second. The results of many studies on the effects of penetration rate are summarised by Lunne et al (9) and indicate that increased penetration rates can lead to high cone tip readings.

3.5 Field Vane

Field vane testing procedures are similar to the hand vane described earlier except that the field vane test is performed in situ, typically at the bottom of a borehole. A torque wrench was used to rotate the rods as a Geonor was not available. The peak resistance to rotation was measured and, following failure, the residual shear strength of the soil was also measured. The calculation of the shear strength of the soil uses the same equation as the hand vane. Bjerrum (10) has recommended the following correction to account for the time dependency of shear vane testing variation with plasticity index, PI , of the clay.

$$S_u = \lambda S_v$$

$$\text{where } \lambda = 1.7 - 0.54 \log(PI)$$

Standard testing procedures (AS1289 (11)) recommend that the vane be rotated 6 degrees per minute for peak shear measurement. In the absence of a Geonor the vane rotation speed was not as well controlled and was, in most cases, faster than standard procedures require. Effective refusal of the vane was achieved in very stiff and hard soils.

3.6 Menard Pressuremeter

Menard pressuremeter testing was performed in prebored boreholes to assess strength and modulus. This test comprises inserting a cylindrical probe into the ground and inflating the membrane of the probe in the radial direction. The measured change in volume of the probe with pressure allows soil strength and stiffness parameters to be assessed. Plotting the results as pressure versus volumetric strain, the gradient of the straight section of the plot is the shear modulus, G , as follows (Clarke (12)).

$$G = dp/d\epsilon_v$$

Young's modulus can be calculated by the following equation using an estimated Poisson's ratio, ν .

$$E = 2G(1+\nu)$$

Pressuremeter testing can be quite sensitive to disturbance of soil due to drilling and operator expertise. Local contractors supplied and operated the Menard pressuremeter for this investigation.

3.7 Cross-Hole Seismic

Cross-hole seismic testing can be performed on soil or rock to assess the in situ dynamic material properties including Poisson's ratio and Young's modulus. The test is performed in a pair of cased boreholes approximately 3 m apart. A source capable of generating shear and compression waves is lowered into one borehole and a geophone receiver is lowered into the other. The travel time is used to assess shear and compression wave velocities versus depth down the borehole. The following equation (McCann (13)) is used to calculate Young's modulus from velocity.

$$E = 2\rho V_s^2(1+\nu)$$

where ρ = mass density
 V_s = shear wave velocity

ν = Poisson's ratio

$$= \frac{0.5(V_p/V_s)^2 - 1}{(V_p/V_s)^2 - 1}$$

V_p = compression wave velocity

To increase the accuracy of the data interpretation the inclination of the boreholes from vertical was measured.

4 TRIAL EMBANKMENT LOAD INSTRUMENTATION

4.1 General

The trial embankment provided full scale in situ measurements of compressibility. Instrumentation was installed prior to the construction of the embankment, which reached a maximum height of thirteen metres. Instrumentation included a magnetic extensometer and several settlement plates to measure incremental settlements at depth and the total surface settlement.

4.2 Magnetic Extensometers

A magnetic extensometer was installed near the centre of the trial embankment site. Magnetic extensometers measure the settlement or heave of soil using targets placed at regular depths down a borehole and linked via a collapsible outer tube. A probe is lowered

through a central access tube to measure the location of the targets and the differential movement is used to calculate incremental settlements at depth. As the trial embankment was placed, the extensometer was extended through the fill to allow uninterrupted monitoring. Extensometers can be quite difficult to construct and install in the correct manner. A planned second extensometer literally fell apart during installation and was abandoned. The operating extensometer required repair work from time to time to repair damage caused by earthmoving plant during placement of fill. This resulted in a reduction in the amount of useful data that could be collected.

4.3 Settlement Plates

Metal settlement plates with vertical risers were placed at the ground surface prior to the placement of the trial embankment. As the load was built up the vertical risers were extended. The relative level of the risers was measured at regular intervals to allow calculation of total surface settlement under the surcharge loading. Although care was taken during fill placement, risers were occasionally damaged by heavy plant, considerably reducing their accuracy.

5 FIELD MEASUREMENTS

5.1 Shear Strength

Shear strength values were calculated from field investigation results using the methods presented in Section 3. Figures 1 to 4 show the assessed variations in shear strength with depth at four field locations.

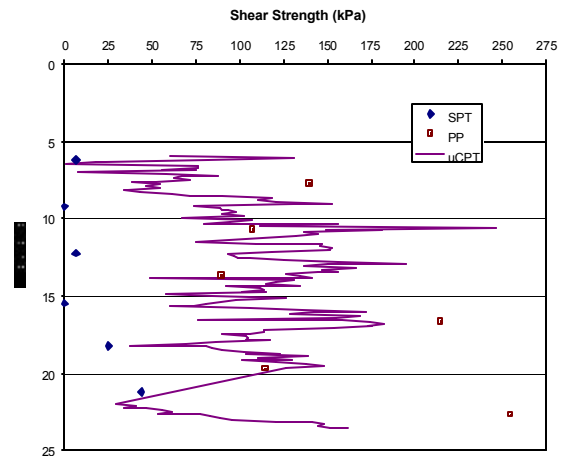


Figure 1. Field Location PL05

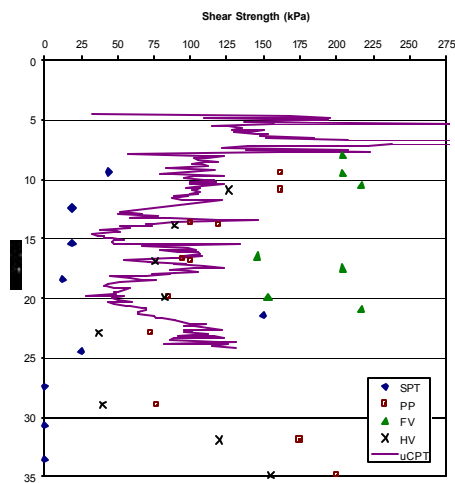


Figure 2. Field Location PL18

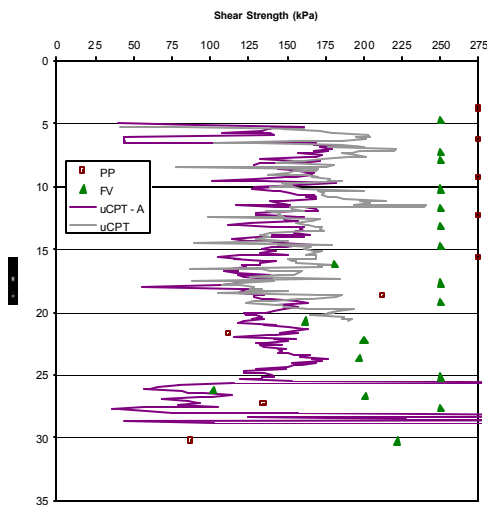


Figure 3. Field Location PL41

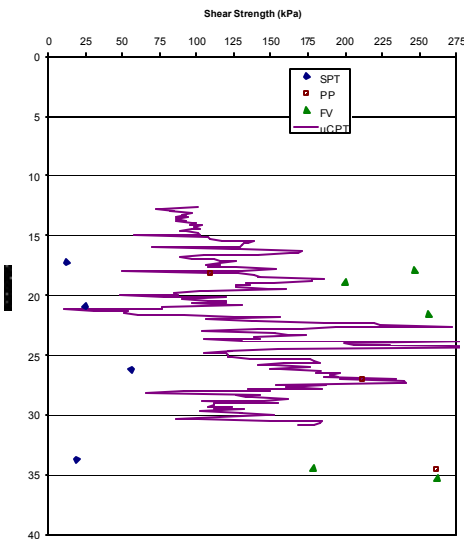


Figure 4. Field Location PL44

Figures 1 to 4 show that the assessed shear strength varies with the field technique used. For each of the field techniques the range of shear strength values obtained and their averages are given in Table 2.

Field Technique	Shear Strength (kPa)		
	Min	Max	Average
Standard Penetration Test	0	150	24
Pocket Penetrometer	72	>300	171
Field Vane Test	109	>250	215
Hand Vane Test	37	155	91
Piezo-Cone Penetration Test	1	1416	129

Table 2. Range and average of assessed Limonite shear strength.

5.2 Young's Modulus

The Young's modulus of the Limonite was estimated from the Standard Penetration Tests and piezo-Cone Penetration Tests using the empirical correlations outlined in Section 3. Young's modulus was also obtained directly from the strain measurements obtained through the pressuremeter and cross-hole seismic testing. The high values of Young's modulus measured using cross-hole seismic techniques are expected due to the small strain properties of the test. Stress-strain behaviour is typically non-linear with very high stiffness's measured at small strains. At intermediate and large strains, plastic behaviour dominates and the stiffness becomes more sensitive to strain and approaches a minimum value as the soil reaches failure (Matthews (14)). The assessed Young's moduli at field location PL05 are given in Figure 5.

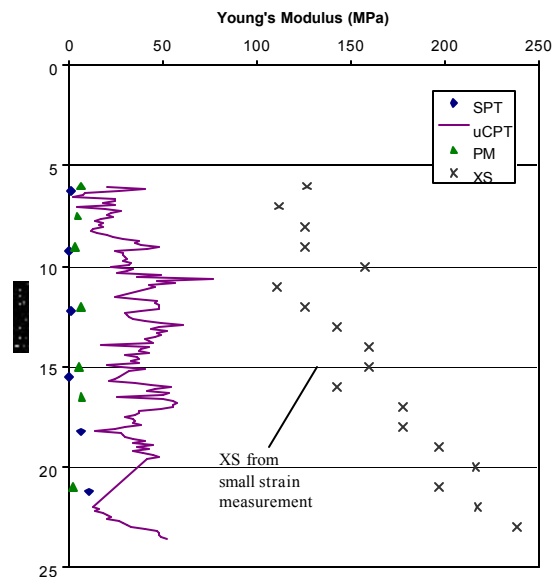


Figure 5. Field Location PL05

6 SURCHARGE LOAD MEASUREMENTS

Settlements under the trial embankment were measured from the settlement plates and movement of the extensometer at depth. At the maximum applied surcharge (350 kPa) Stocker (5) estimated the secant Young's modulus of the Limonite from the ratio of pressure to local strain. The computer program UNISETTLE was used to estimate the pressure profile for each depth increment and the local strain was based on measurements from the extensometer. It was observed that consolidation settlement occurred rapidly and could not be differentiated from elastic effects.

Two CPTUs were also performed at the site of the trial embankment, prior to fill placement, and the Young's moduli calculated from these are shown with the test load results in Figure 6.

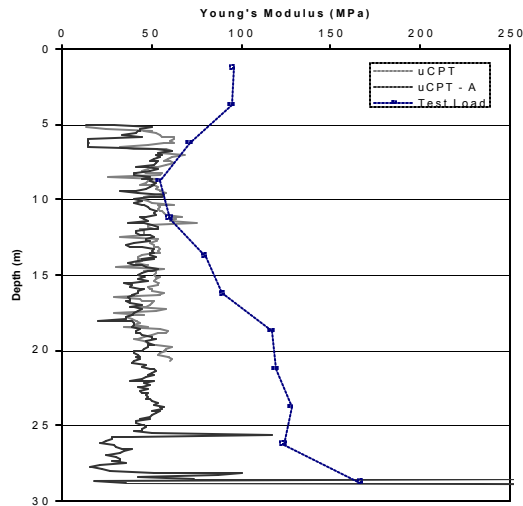


Figure 6. Young's Modulus of Limonite under the Trial Load

7 DISCUSSION

7.1 Shear Strength

Results of the SPTs indicate soil consistencies ranging from very soft to soft, lower than estimated from all the other field techniques. The average of these results is an order of magnitude lower than results of field vane testing. Such dramatically low values may be caused by breakdown of structure of the Limonite under the dynamic load applied during SPTs. The release of "trapped" water seen in the split spoon samples of SPTs supports this contention.

The measured shear strength from the hand vane testing also gave lower than expected results, with a greater variation. Measured consistencies ranged from firm to very stiff. The tube samples on which the tests were performed, although theoretically undisturbed, usually appeared to have undergone considerable change from their in situ state.

The field vane testing returned markedly higher shear strengths, averaging greater than 200 kPa. It would appear that the method of retrieving and transporting samples for hand vane testing caused enough disturbance to reduce the shear strength to approximately half of that for an in situ vane test.

Shear strengths calculated from pocket penetrometer readings averaged about 170 kPa, very stiff, and were on average only marginally lower than the strengths indicated by the field vane. Pocket penetrometer testing was performed in the same samples as the hand vane testing and consistently gave higher results by approximately 30 kPa. Pocket penetrometer testing was generally performed immediately following removal of the sample from the ground, with no transportation or time lapse.

The results of the CPTU were, on average, 90 kPa lower than the field vane testing. These differences are likely due to the disturbance associated with the UCPT and the effect of the assumed cone constant, N_k .

Two anomalies witnessed during these tests may go some way towards explaining the low values.

Firstly, pore pressures were seen to increase to well over 1 MPa during testing only to drop instantly when tip advancement was halted to add rods. Stocker (5) surmises this is related to the structural collapse of the Limonite and that the tip measurement is actually the resistance due to some combination of peak shear strength near the cone tip and the residual strength of extremely distorted soil near the rear of the cone. With the residual strength being only a small fraction of the peak strength.

Secondly, there were several recordings of negative pore pressures, both spikes and sustained readings. It appeared that lenses of soil were dilating under the applied pressure.

7.2 Young's Modulus

Figure 7 shows the variation in Young's modulus with depth as assessed from all of the field tests and from back analysis of the trial embankment measurements.

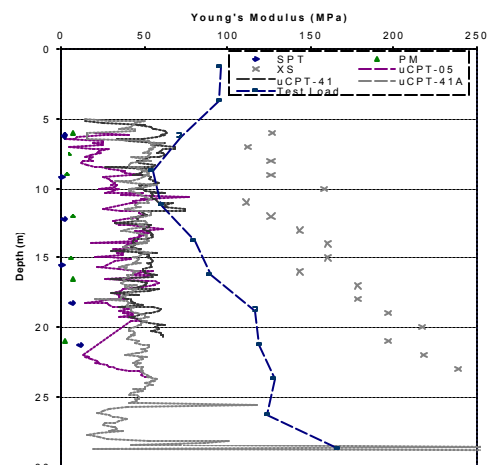


Figure 7. Young's Modulus from all Measurement Sources

The results from the cross-hole seismic testing and the test load indicate similar material behaviour with depth, although the numbers differ in magnitude. That is decreasing values to approximately 10 m then increasing with depth. The cross-hole seismic values are small strain moduli and are expected to be higher than the results from the other field measurement techniques. Small strain moduli measured through cross-hole seismic appear to be 2-3 times larger than those assessed from the trial embankment deformations. The cross-hole seismic moduli are from non-destructive techniques, unlike the other field methods that involve failure of the soil. It is also unlikely that the trial embankment caused failure within the Limonite. The absence of structural breakdown of Limonite led to higher values of modulus assessed from embankment measurements.

Young's moduli estimated from SPT N values are again very low and, as with the shear strength results, this is suspected to be due to the structural disintegration of Limonite. The preboring before pressuremeter testing may also have caused the breakdown of the Limonite

structure close to the borehole walls and resulted in the low moduli values that can be seen in Figures 5 and 7. The CPTUs performed in the strata at the site of the trial embankment give higher Young's moduli than the CPTU at field location PL05, showing that there is some natural variation of stiffness across the investigation site. The CPTU assessed moduli appear to remain within a uniform range with depth. As outlined in Section 3.4 the Young's modulus is based on the measured cone tip resistance via an empirical correlation. The reduced strength of collapsed Limonite will therefore lead to a reduced estimate of Young's modulus. The assumed ratio of 20 may also be adjusted to vary Young's values.

8 CONCLUSIONS

Due to the high void ratio and open structure of the Limonite, formed through the extensive weathering of Peridotite in a tropical environment, standard field investigation techniques and empirical correlations used in transported soils and less leached soils were found to be of limited value.

SPT N values should only be used with extreme care as a source of information on the properties of Limonite. Even following testing procedures perfectly, their use may need to be limited to the collection of samples for logging and laboratory testing.

Use of hand vane and pocket penetrometer tests in undisturbed tube samples should be applied with consideration of the sensitivity of Limonite to disturbances during sampling and transportation. All testing should be performed immediately following sampling to reduce these effects.

Field vane testing appears to provide a realistic indication of shear strength but the results of laboratory testing should be used to correct for plasticity. The field vane is limited in its use in very stiff and hard Limonite where refusal occurs.

The assessment of Limonite properties from CPTUs may need to be limited to cone tip results as sleeve friction and pore pressures are measured in collapsed Limonite behind the tip. The use of any CPTU results should be based on site specific correlations derived on the basis of other field techniques and assessment of behaviour under surcharging.

Menard pressuremeter testing gave low indications of stiffness compared to other methods due to the initial disturbance caused by borehole drilling. Self-boring pressuremeters may give better results but it is likely that some disturbance of the Limonite will still occur and effect the results.

Cross-hole seismic testing appears to give a reliable estimate of the dynamic small-strain Young's modulus of Limonite. Its expense and the applicability of small strain measurement to design may limit the general use of this technique.

Instrumentation of a trial embankment provides results of a large scale field test not involving disturbance to the Limonite, allowing the measurement of modulus values without soil collapse. Time and cost constraints may restrict the use of this method but the measurements obtained can be used to develop site

specific correlations and to gain a better understanding of the soil response to loading. Attention needs to be paid to the quality of the settlement data being received as small measurement errors can lead to big differences in calculated properties.

9 ACKNOWLEDGEMENTS

I would like to thank our client for graciously allowing this paper to be published. I want to acknowledge the extensive collation and analysis of Limonite data that was performed by Peter Stocker (GAP, Brisbane) and is presented in Reference 5. I have inspected only a very small subset of the data. Thanks to Pierre Legeron (GAP, Brisbane) who passed on as much data as he could find. Finally, thanks to the field engineers who worked in difficult conditions to obtain field data.

10 REFERENCES

1. Guillon, J.H., "New Caledonia" Mesozoic - Cenozoic Orogenic Belts, Scottish Academic Press, 1974, pp 445-452
 2. Brothers, R.N., "Regional Geology of New Caledonia and Northern North Island, New Zealand", AusIMM, Pacific Rim Congress 87, 1987, pp 61 -63.
 3. Fookes, P.G., "Tropical Residual Soils", The Geological Society London, 1997.
 4. Golder Associates Pty Ltd, "Site Investigation Guidelines, Document #I-062", May 2002.
 5. Stocker, P. "Characteristics of Goro Limonites", Golder Associates Pty Ltd, Doc. #B-616, 2003.
 6. Blight, G.E., "Mechanics of Residual Soils", A.A. Balkema, 1997.
 7. Richards, A.F., "Vane Shear Strength Testing in Soils", ASTM, 1987.
 8. Robertson, P.K. & Campanella, R.G., "Interpretation of cone penetration tests. Part II: Clay", Canadian Geotechnical Journal, Vol. 20, 1983, pp 734-745.
 9. Lunne, T., Robertson, P.K. & Powell, J.J.M., "Cone Penetration Testing in Geotechnical Practice", Blackie Academic & Professional, 1997.
 10. Bjerrum, L., "Problems of Soil Mechanics and Construction on Soft Clays", Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, 1973, pp 111-159.
 11. Australian Standards "AS 1289.6.2.1-1997 Methods of testing soils for engineering purposes- Soil strength and consolidation tests-Determination of the shear strength of a soil-Field test using a hand vane", Australian Standards, 1997.
 12. Clarke, B.G., "Pressuremeters in Geotechnical Design", Blackie Academic & Professional, 1995.
 13. McCann, D.M., Culshaw, M.G. & Fenning, P.J., "Setting the standard for geophysical surveys in site investigation", Modern Geophysics in Engineering Geology The Geological Society London 1997 pp 3-34
 14. Matthews, M.C., Hope, V.S. & Clayton C.D.I., "The geotechnical value of ground stiffness determined using seismic methods", Modern Geophysics in Engineering Geology, The Geological Society, London, 1997, pp. 113-123.
- A number of the referenced documents are project reports and are not available for general viewing.

The Diversity of the Geology in the Auckland Region and Some of the Associated Challenges for Land Development

James Beaumont : Foundation Engineering Limited

The diversity of the geology within the Auckland region presents a range of challenges for the geotechnical professional. This paper provides an overview of the Auckland geology and presents case studies of three recent land development projects within different geological settings, all of which presented a range of geotechnical problems to be addressed. The three lithologies include very soft organic peats within the alluvial lowlands of South Auckland, the Waitemata Group sedimentary deposits on the North Shore, and the Northland Allochthon/Onerahi Chaos deposits of Silverdale. This paper also discusses examples of appropriate engineering solutions to some of the problems presented by these lithologies.

1.0 INTRODUCTION

The Auckland region has a complex geological history. This history has implications for land development. It is important for geotechnical professionals to have an understanding of the formation and engineering properties of the geology at the location of their project. This paper will discuss some of the challenges that the various geological units in the Auckland area pose for the geotechnical professional. To do this it will discuss the challenges on three recent projects undertaken within different geological settings and their associated engineering solutions.

2.0 GEOLOGY OF THE AUCKLAND REGION

Beneath the varying topography of the Auckland region is a complex geology. This geology is made up of materials deposited during the late Triassic to Holocene times (200 million years ago to the present) and comprises sediment of differing origins.

From Mesozoic to Cretaceous times the Waipapa Group and Murikiku Terrane 'greywacke' was uplifted and deformed. Following this period of uplift and deformation the land was eroded and peat began to form. This peat was later transformed into the Waikato Coal Measures which was further overlain by the Te Kuiti Group (Kermode, 1).

Later the Waitemata Basin began to form. It extended 80km to the north and south of present day Auckland. The basin began to fill up during the early Miocene (24 to 15 million years ago) firstly with materials of the Kawau Subgroup followed by 2km of sediments consisting mostly of mud with occasional large quantities of coarser silt and sand (Kermode, 1). This sequence of sediments became known as the East Coast Bays Formation and forms the vast majority of sea cliffs and rolling hill country in the Auckland area.

While the Waitemata Basin was filling up, several sheets (Northland Allochthon/Onerahi Chaos) of material were

emplaced over the newly forming Waitemata Group. This material originated from the north-east of Northland and is intensely deformed (Edbrooke, 2).

Beginning in the Pleistocene (2 million years ago) the Taupo volcanic zone produced sediments that have made a significant contribution to the topography around the Manukau Harbour (Kermode, 1). In between these volcanic sediments peat and organic clays have formed at several levels (Tauranga Group).

Finally in more recent times 48 volcanoes have formed within the Auckland volcanic field producing lava fields, scoria cones, ash mantles and tuff rungs over much of the Auckland area (Kermode, 1).

Figure 1 on the following page shows the simplified stratigraphic relationships between the various geological units in the Auckland area.

3.0 POINT RIDGE RESIDENTIAL SUBDIVISION, ALBANY

The Point Ridge residential subdivision is situated in Albany, a suburb on the North Shore in Auckland. The land gradients on this site generally slope from north to south and the block is underlain by Miocene Age Waitemata Group sedimentary deposits.

These sediments typically weather to residual orange, grey, pink and cream clays and silts. The dark grey transition zone material which comprises very stiff/dense clays, silts, and sands is overlain by the residual overburden. Beneath this transition zone lies bedrock which generally consists of very weak, highly weathered, slightly cemented dark grey sandstones and siltstones.

3.1 SITE INVESTIGATION

During preliminary desk top studies a number of instability features were identified. Upon investigation we concluded that the movements were both circular and planar in nature. The circular failures were found to generally lie within the residual overburden materials while the planar failures were found to occur between the residual overburden and transition zone materials at approximately 8 to 10 metres depth.

As part of the investigation a comprehensive series of laboratory tests were undertaken. Tests included Atterburg limits, consolidation, compaction and triaxial testing. From this testing realistic shear strength and settlement characteristics were adopted for stability analyses and settlement calculations.

Some of the challenges encountered at this site were:

- Variable bearing capacity conditions
- Shrinking/ swelling properties
- Shallow and deep seated slope instability
- Settlement

3.2 SHALLOW AND DEEP SEATED SLOPE INSTABILITY

As the site was showing signs of land movement, back analysis was undertaken for an assumed pre-failure profile. This yielded effective stress parameters of $c' = 0 \text{ kPa}$ and $\phi' = 18 \text{ degrees}$ (Bevin, 3) for soft to marginally firm strata directly overlying stiff/ dense transition zone materials.

Table of Soil Properties		
Description	c' (kPa)	ϕ' (degrees)
Stiff certified clay filling	10	30
Soft to marginally firm strata	0	18
Firm to stiff silty clays, clayey silts and silts	7	28
Very stiff clayey silts, silts and sandy silts	5	28
Bedrock	50	40

Figure 2: Table of Soil Properties (Bevin, 3)

Using the parameters in Figure 2, stability analyses were then performed using the Simplified Bishop and Janbu Methods for circular and planar analyses. As a result of these analyses it was proposed that two shear keys (buttress fills) should be installed on the south facing slope.

The analyses were carried out for normal groundwater conditions, elevated groundwater levels, and extreme groundwater levels. For normal groundwater conditions

the assessed factors of safety were in excess of 1.5 while for elevated groundwater conditions factors of safety lay between 1.2 and 1.5, and for extreme groundwater conditions the factors of safety were just greater than unity.

This showed that even with the construction of two shear keys, appropriate groundwater control was also needed in order to maintain factors of safety in excess of 1.5.

The shear keys needed to adequately stabilise the slope were approximately 15 metres wide at the base and needed to be founded on very stiff/dense transition zone and/or bedrock material, below the shear planes. They were oriented in an east-west direction and keyed into competent identified intact ridges at both ends. The lower shear key was situated at the toe of the slope while the upper shear key was situated mid-slope as depicted in figure 3 shown below.

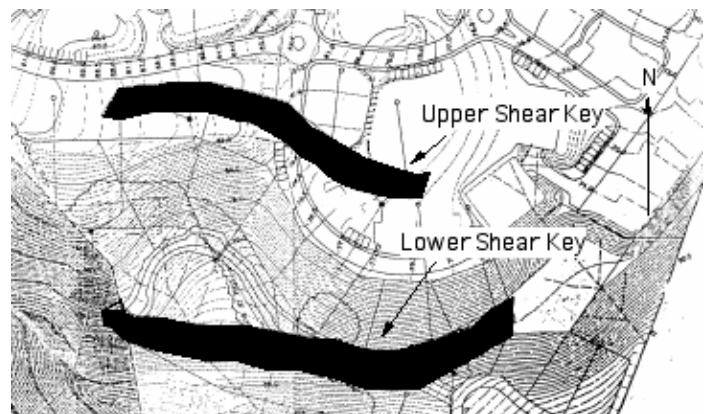


Figure 3: Approximate Shear Key Locations

3.3 SETTLEMENT

Results from consolidation testing showed that likely total settlements beneath the structural fills were of the order of 300mm (Bevin, 4). Although this appeared excessive the results also showed that the settlement would occur relatively rapidly (ie. months) and that t_{90} would be reached before commencement of building construction.

However, of more significance was the potential for differential settlement, particularly in cases where settlement sensitive buildings would be sited partly on fill and partly on cut ground, or where they would be sited entirely on filling whose depth changes significantly across the building platform.

In areas of concern, post construction settlement monitoring was recommended to assist in confirming that t_{90} had been achieved and that any long term settlement would be within code limits. As a further precautionary measure recommendations were also provided in regard to careful detailing of control joints in brittle exterior

cladding such as brick veneer, block work, solid and stucco plaster etc.

3.4 SHRINKING/ SWELLING PROPERTIES

A common problem encountered with Waitemata Group materials is their seasonal shrinking and swelling characteristics. To assess this criteria from AS 2870:1996 were used. It provides foundation solutions for five defined site classes. These classes are good ground (A), slight (S), moderate (M), high (H), and extreme (E). AS2870:1996 provides a range of foundation solutions which typically involve combinations of footing depths and widths.

The site was assigned an expansive class of S (Slight) Bevin, 3). An appropriate solution was to found all conventional strip and pad foundations 450mm below finished ground level.

3.5 VARIABLE BEARING CAPACITY

Generally residual Waitemata Group sediments are of sufficient strength to provide a geotechnical ultimate bearing capacity of 300 kPa, as required for NZS 3604:1999, "Timber Framed Buildings". However areas exhibiting a reduced bearing capacity are often encountered. Several areas of soft natural ground were found at this site and accordingly a reduced geotechnical ultimate bearing capacity of 210 kPa was recommended.

4.0 KINGSWAY SCHOOL, SILVERDALE

As part of site development Kingsway School have added playing fields and several blocks of classrooms. The land gradients within the site are typically gentle with some steep areas along the southern boundary of the site. The site is underlain by Onerahi Chaos/ Northland Allochthon materials.

This lithological unit is highly deformed and is generally comprised of crushed limestone, mudstone, conglomerate, and siltstone within a matrix of mudstone, siltstone, and sandstone (Bevin, 5).

Generally overlying this shattered rockmass is a cap of residual clay soils which acts as a semi-impermeable layer limiting the amount of rainfall infiltration. This layer also helps to confine the underlying rockmass.

Onerahi Chaos materials can be unstable at slope angles as low as 5 to 10° (Rafferty, 6). It weathers rapidly and swells due to stress relaxation when exposed.

4.1 SITE INVESTIGATION

The site investigation for this development found that the semi-impermeable layer of clay was between 1.5 and 2.0

metres thick. This layer was underlain by the shattered Onerahi Chaos rockmass.

Some of the challenges identified on this site were:

1. Rockmass Degradation
2. Soil Creep and Slope Instability at Relatively Shallow Angles
3. Significant Shrinking/ Swelling Properties

4.2 ROCKMASS DEGRADATION

As part of earthworks for the proposed development the semi-impermeable clay cap was removed. This exposed the underlying Onerahi Chaos rockmass at several places. In order to maintain a semi-impermeable layer above the rockmass it was decided that the best course of action was to undercut the rockmass at these locations by 250mm and to replace it with compacted clay (Lander, 7). This served to limit rockmass degradation and reduce swelling due to stress relief. Figure 4 below shows the cut that would expose the Onerahi Chaos.

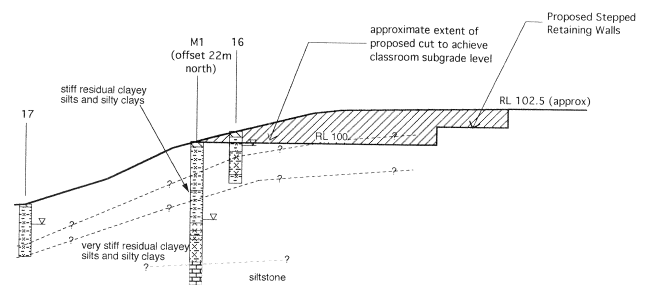


Figure 4: Cross-Section Showing the Cut Exposing the Onerahi Chaos (Lander, 7)

4.3 SOIL CREEP AND SLOPE INSTABILITY

Onerahi Chaos materials can be unstable at slopes 10° (Rafferty, 6). The majority of land gradients on this site were gentle, however the localised gradients adjacent to the southern boundary were as steep as 36°. Accordingly the new classroom block was given a generous set-back from areas having such gradients.

In addition to this, the leading edge foundations were piled. Piles were bored and concreted to depths of 4.5 metres and they were designed for a 2 metre deep creep zone (Lander, 7). This allowed for any long-term creep between the semi-impermeable clay layer and the underlying Onerahi Chaos rockmass.

4.4 SHRINKING/SWELLING

The residual Onerahi Chaos soils tend to be expansive. At this site the expansive class assigned was M (Moderate). Accordingly all conventional strip footings

were constructed 600mm below finished level (Lander, 7).

5.0 ADDISON BLOCK RESIDENTIAL DEVELOPMENT, TAKANINI

This development comprises a medium density residential subdivision. The site is generally flat and is contained within the South Auckland Lowlands. This area is underlain by organic/peaty clays of the Tauranga Group.

The Tauranga Group alluvial sediments are typically soft and are among the most recent to be deposited in the Auckland Region. These materials typically include peats, organic clays, and lenses of pumiceous material. Groundwater levels are generally close to the surface.

5.1 SITE INVESTIGATION

The site investigation found that organic/peaty clays were present beneath a 1.0 to 1.5 metre deep crust of firm organic material to depths beyond 10 metres. The groundwater level typically lay between 1.2 to 1.5 metres depth (Melville-Smith, 8).

The materials underling the site pose several challenges:

7.0 Low Bearing Capacity and California Bearing Ratio (CBR)

8.0 Potential for Large Total and Differential Settlements

5.2 LOW BEARING CAPACITY AND CBR

Peats and organic clays generally have a low bearing capacity and CBR. The field investigation showed that an ultimate geotechnical bearing capacity of 180 kPa and an equivalent CBR of 4 was available (Melville-Smith, 8).

The low bearing capacity and relatively consistent round conditions made the site suitable for raft/ pod type foundations. The raft/ pod type foundations were suitable because the loads are spread over an increased area therefore reducing the pressure applied to the subgrade.

Along with the implications for the construction of housing, the low bearing capacity and CBR presented additional problems for road pavement construction and for the installation of service lines.

The solution that was adopted for the placement of the service lines was to undercut 300mm below the pipe bedding and replace with hardfill/sand wrapped in filter cloth as shown below in figure 5. The pipe bedding was one sixth of the pipe diameter.

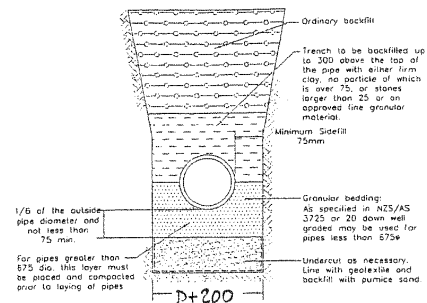


Figure 5: Typical Service Line Construction Detail

For pavement construction the initial solutions involved subgrade stabilisation using lime. Lime testing showed an increase in CBR from 4 to 5 with 6 kilograms of lime used per square metre per 250mm depth (Melville-Smith, 8).

This solution was then discarded in favour of an increased pavement depth. The pavement depth was increased to 400mm with a further undercut of 300mm to be replaced with sand (Melville-Smith, 9). At the time of writing this paper a proposal to use geogrid reinforcing beneath the road pavement was being considered. The aim of this is to reduce the depth of undercut required.

5.3 POTENTIAL FOR LARGE TOTAL AND DIFFERENTIAL SETTLEMENTS

The groundwater level on this block was relatively high. Due to the presence of compressible peats it was imperative that the groundwater level be maintained at its pre-development levels to reduce settlement that may occur.

To help achieve this the subsoil drains along the accessways were designed to serve the dual purpose of protecting the road subbase and allowing the groundwater to recharge the peaty soils.

Because water flows from high to low head, it was important that the groundwater flow was unobstructed. If the backfilling of service lines was compacted to a higher standard than the insitu material the groundwater flow could be obstructed. This higher density material could act as a dam causing the water table on the low head side of the service line to drop causing a change in effective stress resulting in settlement.

6.0 CONCLUSIONS

The three case studies that have been presented in this paper illustrate the variation in geological conditions in the Auckland Region. They also show that the

geological conditions and associated challenges present in the Auckland Region require differing engineering solutions for the problems that are encountered at a specific site. In effect, an engineering solution that may be appropriate for a particular site may not be appropriate for another.

Therefore it is important for the geotechnical professional to have a sound knowledge of the geological conditions present at their project location. This knowledge combined with the data obtained during the site investigation should enable the geotechnical professional to provide an appropriate engineering solution to a site specific problem.

7.0 REFERENCES

1. KERMODE, L.O., "Geology of the Auckland Urban Area", Institute of Geological and Nuclear Sciences, 1992, p8
2. EDBROOKE, S.W. (Compiler), "Geology of the Auckland Area", Institute of Geological and Nuclear Sciences, 2001, p19, 20, 23
3. BEVIN, J., "Supplementary Geotechnical Report for Gills Road Residential Subdivision", September 2000, p6, 7, 11
4. BEVIN, J., "Geotechnical Investigation Report for Gills Road Residential Subdivision", February 2000, p26
5. BEVIN, J., "Geotechnical Investigation Report for Proposed Extensions at Kingsway School", October 1998, p4
6. RAFFERTY, W.J., "Pre-Quaternary Geology of Southern Northland", August 2001, p29
7. LANDER, S.G., "Foundation Investigation Report for Proposed Building Developments at Kingsway School", January 2003, p10, 12, 31
8. MELVILLE-SMITH, R.W., "Geotechnical Investigation Report for Addison Residential Development Stage 1A", December 2002, p8, 10
9. MELVILLE-SMITH, R.W., "Geotechnical Completion Report for Addison Residential development Stage 1A", November 2003, p7

Field Moisture Content Effects **on Shrink-Swell of Clays**

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ABSTRACT

Geotechnical engineers have substantial involvement in classification of sites for residential developments in accordance with AS2870-1996 “Residential Slabs and Footings” to aid in footing design.

The standard procedure for site classification is based on soil strata and properties. In Australia, the most commonly adopted model incorporates the AS1289 7.1.1-1998 Shrink-Swell Index test method, which is taken to represent the surface movement per unit depth per unit suction change.

The result of the Shrink-Swell test is assumed to be the same if the soil is sampled at different moisture contents. However, the validity of the results when sampled at substantially dry or wet insitu moisture content is uncertain, particularly for highly reactive clays.

This paper discusses an experimental study to measure Shrink-Swell Index of two heavy clays of varying moisture contents. An experimental model was developed to achieve uniform moisture contents within the samples. The results were compared with those from undisturbed samples of the same clay. Statistically significant results were obtained.

INTRODUCTION

Geotechnical engineers are required to investigate suitability of sites for residential development. The assessment of the site allows the engineer to determine the type of foundation appropriate for residential developments. The demand for foundation assessment, particularly subdivision development has increased with the population, mainly on the east coast of Australia.

Site assessment for residential developments is carried out in accordance with the Australian Standard AS2870-1996 “Residential Slabs and Footings”. This Standard sets out the procedure for site classification of sites and the appropriate foundation type for each class. These categories are based on ground movement limits, which are defined in terms of design surface movement (γ_s), Cameron et al., (1997).

Design surface movement (γ_s) is a hypothetical parameter representing the “difference between the surface movement that would occur if the design range of moisture conditions (dry to wet) was experienced.” Cameron et al., (1997). The two moisture extremes of wet and dry would occur at different periods of site development and therefore not simply the range encountered in the field before development.

The main consideration, is the soil behaviour under load from the structure and with variations of moisture content. On the east coast of Australia, foundation materials are typically clay and are prone to variations in the moisture content of drying (receding) and wetting (advancing) the soil causing shrinking and swelling, respectively. The time elapsed in-situ depends upon the soil properties and conditions. For example, Marshall (1967) indicates that some clays in England, continue to swell for about three months after the wet season.

This potential to swell and shrink (expansion or volume change characteristic) affects the structures by causing damage such as cracking and settlement, amounting to millions of dollars throughout the world. Thereby, the engineer is faced with heightened legal accountability.

A lot of emphasis is placed on testing methodologies for determining the characteristics of the soil. In Australia, engineering practices most commonly adopted the model which incorporates the AS1289 7.1.1-

1998 Shrink-Swell Index test method, which is taken to represent the surface movement per unit depth per unit suction change. This method of determining the reactivity or shrink-swell potential theoretically should not vary with variations in moisture content. However, the validity of the results when sampled at substantially dry or wet insitu moisture content is uncertain, particularly for high plasticity clays.

An experimental study was undertaken to develop a model to simulate the in-properties of clay by comparing varying moisture contents with shrink-swell characteristics in a short time frame.

EXPERIMENT

Site classifications is based on the measurement of potential expansion of soil. The common method in Australia for determining the expansive potential of clays, shrink-swell index test, was developed by Cameron, and further confirmed by Fityus in 1996, Cameron et al., (1997). The shrink-swell index (I_{ss}) can be determined, by the following equation, after the individual simplified shrinkage test and swell test have been completed.

$$I_{ss} = ((e_{sh}/2) + e_{sw}) / 1.8$$

I_{ss} = shrink-swell index that is the percentage of strain per pF change in suction. e_{sh} = the total shrinkage strain (%) and e_{sw} = total swelling strain (%). The factor of 1.8 is an effective suction change for the test, derived by Cameron in 1989 and confirmed by extensive testing carried out by Fityus in 1996.

The validity of the shrink-swell index value gained when the sample is obtained at extremely dry moisture contents and alternatively wet moistures is called into question. The concept of the shrink-swell index (I_{ss}) is that the value should be the same regardless of the moisture content. In the field, the period between extreme moistures varies from months to years, depending on the landscape, soil conditions, land use, climate, precipitation etc.

The purpose of this experimental test design was to simulate the extreme drying and alternatively wetting conditions on highly reactive homogenous clay. The model was to achieve these variations in moisture content and then carry out shrink-swell testing. The main constraints of the project were material homogeneity, moisture control and a short time frame.

Two high plasticity clays were sourced from a site near Nowra, NSW, sampled within the individual strata layers and denoted throughout the experiment as Clay 1 and Clay 2. Using the Unified Soil Classification System (USCS) analysis; Clay 1 is (CH) CLAY, high plasticity, red-red/brown, trace fine to medium gravel. Clay 1 was underlain by Clay 2: (CH) Silty CLAY/CLAY, high plasticity, yellow/pale brown. Both clays had field moistures of greater than the plastic limit ($M > W_p$).

Previous shrink-swell index results collected over the last three years from the general site area; for Clay 1 ranged from 2.8% to 5.2% with associated moisture contents ranging from 21% to 29.6%. Clay 2 shrink-swell index results ranged from 2.7% to 5.1% with associated moisture contents ranging from 18.6% to 29.7%, Network Geotechnics Pty Ltd.

Experimental model:

The objective of the experimental model was to simulate the excessively wetting and excessively drying process on homogenous, high plasticity clay, within a very limited time frame and with minimal structural damage to the clay properties.

The model was developed as follows:

1. Compact the homogenous material into moulds at 100% Standard compactive effort at Optimum Moisture Content (OMC) and allowed to soak (or alternatively air dry) for nominal time periods. All samples were first cured to a homogenous condition at OMC in order to reduce the disturbance of clay. Two sets of three moulds were used for obtaining samples with the desired moisture content. One set was for air drying and the other was soaked for varying lengths of time. In each set, the first mould was used for measuring the moisture profile, the second for obtaining a sample for a swell test and the third to obtain a sample for the shrinkage test.

2. U50 tubes were then to be driven into the samples and moisture contents taken in increments on the extracted sample.
3. The resulting profile was the basis of further testing on selected increments, with the moistures being chosen at dry and alternatively wet values.
4. Carry out the shrink-swell index test in accordance with AS1289.7.1.1(1998) at selected moisture contents in order to meet the objective of the model.

Moisture Variation and Control

As moisture control was the major quality constraint, the model had to try to achieve uniformity and consistent moisture content. In the case of the increasing the moisture content above optimum, water was required to soak uniformly into the specimen for the whole depth. Similar uniformity/consistency conditions were required with the air-dried.

The process was advanced in groups of three moulds, as depicted in Figure 1 below, as a time saving mechanism.

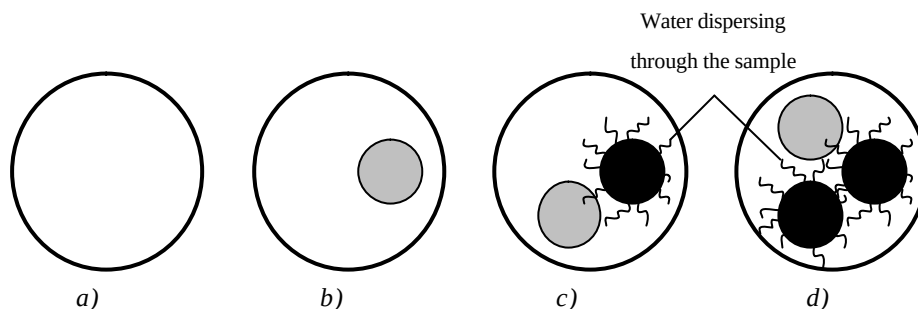


Figure 1 Stages for obtaining samples with desired moisture content within the given time frame.

For each mould the process was undertaken four stages, whereby *a*) is the initial compacted specimen, *b*) is the mould after one tube sample is taken (say 7 days), *c*) is the mould after say 10 days after the 2nd tube sample is taken (note that the first tube hole was filled with sand) and finally *d*) is the mould having the last tube taken.

RESULTS & DISCUSSION

The experimental model adopted was able to provide moisture contents typically ranging from 18% to 33% for Clay 1 and 22% to 27% for Clay 2. Clay 2 was not tested as extensively as Clay 1.

The methodology of reliability assessment incorporated the works of Jaksa et al., (1997) of the first-order Second Moment (FOSM) analysis which found that the model was approximately 70% accurate. The co-efficient of variance of the model was 15%, which was very close to the co-efficient of variances of 20% and 17% for the actual swell and shrinkage moistures respectively.

One of the drawbacks of the experimental model for the drying component was that the specimen had the tendency to break at the compacted layer boundaries. The result is that the test becomes invalid. This scenario occurs in the field, in situations such as excessive cracking of soils.

Experimental Results

The compaction requirements of Clay 1 and Clay 2, were to achieve a Maximum Dry Density (MDD) and (OMC) of 1.63t/m³ at 23% and 1.55t/m³ at 25% respectively.

Shrink-swell index tests results were compared with the moisture contents. The shrink-swell index results for Clay 1 ranged between 4% to 6%, typically 4.8%. The associated moistures prior to swell testing ranged between 18% and 33%. Clay 2 was not tested as extensively as Clay 1, however, the shrink-swell index ranged between 4 % and 6%, typically at 5% for the moisture variation of 22.5 to 27%.

In order to assess the experimental results, comparison was made with previous site history results, as shown in Figure 2 for Clay 1 and Figure 3 for Clay 2. The experimental results of Clay 1 showed similar trend to actual in-situ test (previous) results, although significantly higher by about 25% to 35% for Clay 1.

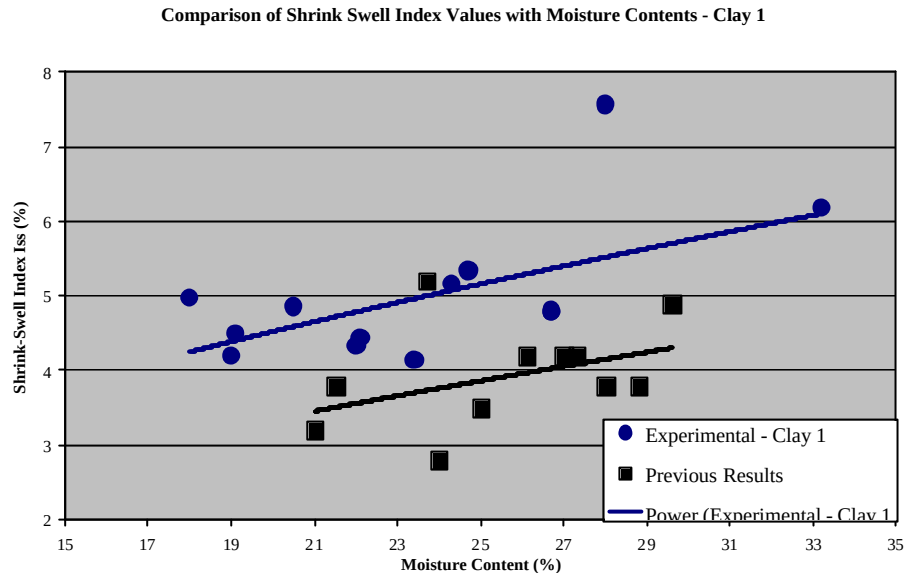


Figure 2 Comparison of Experimental Shrink-Swell Index (Iss) with Previous Clay 1 Results at different Moisture Contents.

The experimental shrink-swell results for Clay 1 were higher by about 30% within the range of $\pm 2\%$ dry/wet of OMC. The equilibrium moisture content of clay underlying other strata layers is typically $\pm 2\%$ dry/wet of OMC, consistent with most earthwork compaction specifications. At the extremities, the range increased to about 35%, having receded about 4% dry of OMC. Alternatively, the range decreased slightly to about 25% greater than the actual for a corresponding moistures 6% wet of OMC.

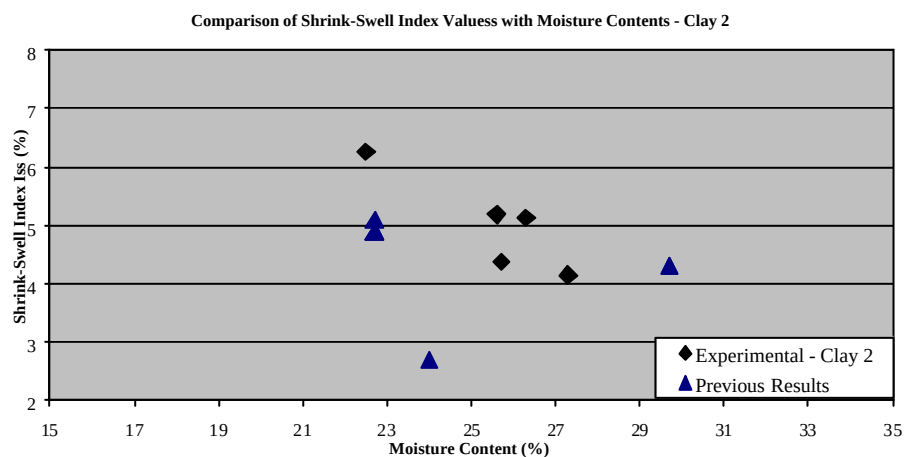


Figure 3 Comparison of Experimental Shrink-Swell Index (Iss) with Previous Clay 2 Results at different Moisture Contents.

Although Clay 2 was not tested as extensively as Clay 1, the results did not show the same trend as shown in Figure 2. However, as an indication, when plotted with the data of Clay 1, it showed that the results thus far follow a similar trend in relation to the moistures. For example, the moisture range of 21% to 29% is

tending towards Iss values of 5% to 5.6%, which is higher than the previous results typically ranging between 2.5% and 5%.

The homogeneity of clay used for the experiment generally showed less scatter than the previous site results, with the exception of one outlier in Clay 1. In regard to the use of high plasticity clays for applications of earthworks such as compacted fill, the shrink-swell index value should be increased, based on the trend encountered in this experiment. This is also the case for situations where sourcing parent material for application to fill would need to be increased by a *confidence factor (%)*. The numerical value of the confidence factor would need to be determined after extensive testing of various sites and plasticities of clay, before being adopted by engineering practices. As an indication from this experiment, of limited tests, the confidence factor would be in the order of about 10% to 15%.

The trend of the shrink-swell index values were lower at lower moisture contents, and conversely higher shrink-swell values at higher moistures. Particularly at the extremities, which proves that the confidence should be reduced in the use of shrink-swell index test AS1289.7.1.1-1998 when sampled at excessively dry or alternatively wet moisture contents.

CONCLUSION

In conclusion, the shrink-swell index values of the selected high plasticity clays vary with the moisture variations. Typically the lower the moisture, the lower the shrink-swell index and the reverse was true for higher than optimum moisture contents.

The greatest variation is in the difference between in-situ sampled clays compared to compacted clays. Although the homogenous nature of the material used in the experiment reduced the scatter of results, the shrink-swell values were significantly higher.

The experimental results set out in this report are site specific. However, the measured variations of 25-35% in shrink-swell index values is statistically significant enough to warrant further study. Further investigation may be directed to establish a modifying factor for shrink-swell index when the moisture content of the sample is substantially different to the expected equilibrium moisture content.

The above moisture content effects are disconcerting in terms of the amount of confidence placed on the shrink-swell index value in relation to geotechnical engineering practices.

ACKNOWLEDGEMENTS

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REFERENCES

CAMERON, D.A., 1997, "*Tests for Reactivity and Prediction of Ground Movement*", **Article** *Australian Geomechanics*, , I.E.Aust, V.CE 31, No. 3, Dec, pp121-132.

CAMERON, D.A., WALSH, P.F., 1997, "*The Prediction of Moisture Induced Foundation Movements Using the Instability Index*", **Article** *Australian Geomechanics*

JAKSA, M.B., CAVAGNARO, R.L., CAMERON, D.A., 1997, "*Uncertainties Associated with the Visual-Tactile Method for Quantifying the Reactivity of Expansive Soils*", **Article** *Australian Geomechanics*, I.E.Aust, No. 31, June, pp84 – 91.

MARSHALL, T. J., "*Relations between Water and Soil*", Commonwealth Agricultural Bureaux, Bucks, England.

Standards Australia

Management of Groundwater System for Citylink Tunnels, Melbourne

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Abstract: The management of the groundwater system in the vicinity of the Citylink Tunnels is concerned with protecting private and public assets above the tunnels.

The regional geology consists of the Yarra Delta Sediments and basalt flows overlying the Melbourne Mudstone bedrock. The Yarra Delta sediments comprise a number of alluvial and marine sedimentary deposits. The alluvial deposits are typically granular in nature and produce important aquifers. The marine deposits are more recent in age and are susceptible to consolidation settlement.

The groundwater management includes monitoring of over 100 standpipes and vibrating wire piezometers, monitoring of over 500 survey pins, and monitoring and control of the groundwater recharge system. The groundwater recharge system consists of 5 permanent recharge wells, 10 passive recharge wells and a Water Treatment Plant. The monitoring data is stored in Microsoft® Access 2000 and GIS software package. ArcView is used to assist in assessing the behavior of the aquifers and settlement rates. The use of a database and GIS software tools allow rapid and easy review of a vast volume of data to show groundwater and ground surface trends.

INTRODUCTION

The Citylink Tunnels were constructed in the late 1990's and consists of two tunnels, known as the Burnley Tunnel and Domain Tunnel, and form an integral part of the Melbourne Arterial network.

The geology and groundwater encountered during excavation of the Citylink Tunnels instigated a number of design and construction modifications. The modifications included the development of an expanded permanent groundwater management system that comprises of the control of permanent groundwater recharge, and monitoring of groundwater levels and ground surface settlement.

The permanent groundwater management system includes monitoring of over 100 standpipe piezometers and vibrating wire piezometers, monitoring of over 500 survey pins, and monitoring and control of the groundwater recharge system. The groundwater recharge system consists of 5 permanent recharge wells, 10 passive recharge wells and a Water Treatment Plant.

In addition to the monitoring data collected on the surface, groundwater inflow and external hydrostatic pressure data is collected from within the tunnels.

The permanent groundwater management system includes the management of a vast volume of data that is used to plot settlement and groundwater level trends and is used to generate groundwater drawdown contours and

settlement contours. Database and GIS software is used to assist in managing and interpreting the groundwater management data.

A secondary system of shallow passive recharge to maintain the groundwater regime is utilized around South Bank Interchange. This secondary system is not discussed in detail in this paper.

REGIONAL GEOLOGY AND HYDROGEOLOGY

The main geological units from youngest to oldest are:

- Quaternary units of the Yarra Delta Group represented by:
 - Coode Island Silt (Q_c)
 - Holocene Alluvium (Q_{ha})
 - Newer Volcanics (Q_{nv})
 - Fishermen Bend Silt formation (Q_f)
 - Moray Street Gravels formation (Q_m)
- Tertiary Port Phillip Basin units represented by:
 - Werribee Formation (T_w)
 - Older Volcanics (T_{ov})
 - Brighton Group (T_b)
- Silurian Siltstone and sandstone of the Melbourne formation (S)

Detailed descriptions of the main geological units are discussed elsewhere (Golder 1997, Short *et al* 1999, Hutchison *et al* 1999). The geological units that are pertinent to the groundwater management system are the Coode Island Silt, Moray Street Gravels, Holocene Alluviums and Silurian bedrock.

A brief description of the geology is as follows:

- The Silurian bedrock, consisting of moderately to highly jointed, interbedded siltstone and sandstone, was folded, faulted and intruded by igneous rocks during the Mid-Devonian.
- Alluvial sand, gravel and lenses of silt and clay were deposited in deep erosion valleys in the Silurian bedrock. These alluvial sediments were deposited during a period when the sea levels were much lower than today, and are known as the Moray Street Gravels (MSG).
- Another valley was cut down into the Silurian bedrock during the Holocene and alluvial sands and gravels were deposited into this valley to become known as Holocene Alluvium (HA).
- Typically overlying the MSG and HA is the Coode Island Silt formation (CIS). The CIS was deposited in a moderately shallow marine or estuarine environment.

The CIS is a highly compressible soil, and susceptible to secondary consolidation (sometimes referred to as creep in the literature). The effects of secondary consolidation of CIS are well documented in the geotechnical literature.

The CIS is also susceptible to primary consolidation settlement through groundwater lowering. Groundwater lowering results in a decrease in pore pressures and an increase in the effective weight of the soil in the zone of groundwater drawdown and leads to primary consolidation of the soil. The consolidation of the CIS is governed by the increase in effective stress (ie the drawdown), thickness of the CIS, its' compressibility and the length of time over which the increased loading is

applied.

A typical section of the Yarra Delta sediments in South Melbourne are shown on Figure 1.

The impact of the regional geology and hydrogeology was summarized by Golder 1997 as *"The hydrogeological interaction between the underlying bedrock, the alluvial aquifers within the various old courses of the Yarra river, the effect of drainage to the tunnels and the presence of highly compressible clay (Coode Island Silt Formation), combine to provide a complex geotechnical environment."*

TUNNEL CONSTRUCTION

The Citylink project was delivered under a build-own-operate-transfer (BOOT) arrangement. An overview of the design process and development is provided by others (Short et al, 1999, Talyor et al 1999) and an 'observational approach to design and construction' was adopted.

The Citylink Tunnels consist of two tunnels, the longer (and deeper) tunnel called the Burnley Tunnel and the other, the Domain Tunnel. As well as passing under some of the city's prime business areas and heritage parklands and bridges, a section of tunnel travels beneath the Yarra River. The locations of the Tunnels are shown on Figure 2.

The tunnels consisted of cut and cover tunnel sections and driven tunnel sections. The tunnels are both

Figure 1 Typical section of the Yarra Delta sediments

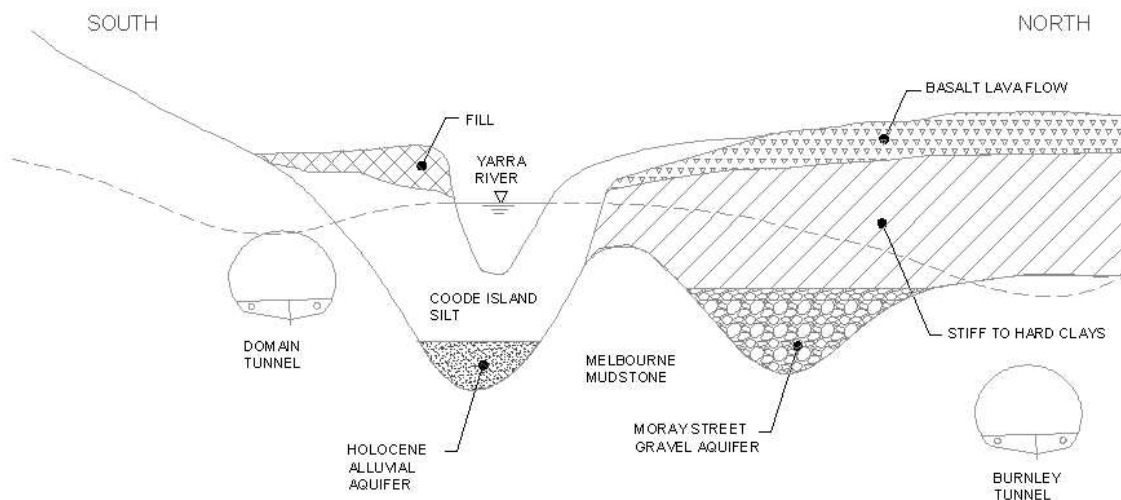
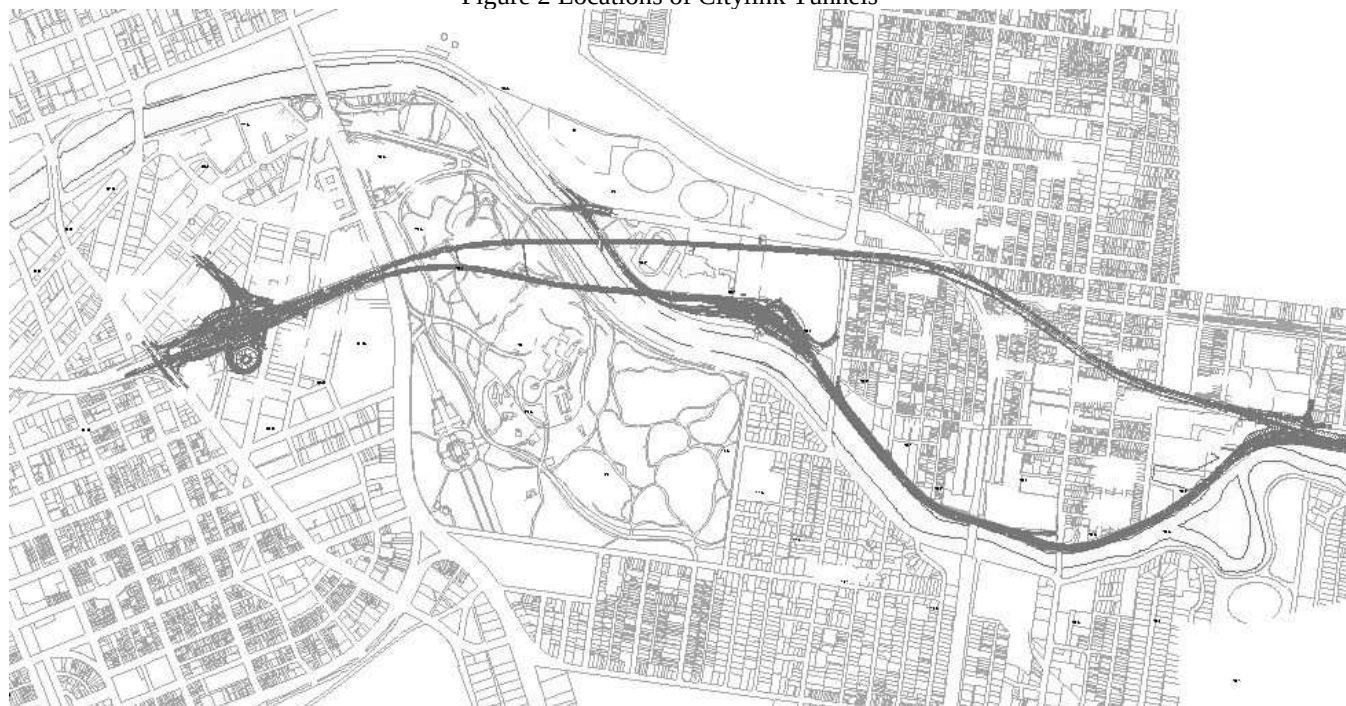


Figure 2 Locations of Citylink Tunnels



approximately 16m wide and 9m high. A modified ellipsoid shape was adopted for the tunnels because it efficiently accommodates the traffic envelope and various appurtenances and is efficient for roadheader excavation and tunnel support.

The driven tunnel section of the Burnley Tunnel was constructed by excavating a centrally located shaft, known as the Swan Street Shaft and two teams working in opposite directions excavated the tunnel using roadheader machines.

Significant groundwater seepage was encountered during the excavation and the tunnel liner detail was modified to a tanked liner and thickened concrete floor (Adams *et al* 1999).

The Burnley Tunnel is up to 60m below the ground surface level and is 3.6km in length. The Domain Tunnel is 1.6km long, with the invert reaching RL -20m

The final permanent design of the tunnels included drained and undrained (tanked) sections.

The Domain Tunnel and Burnley Tunnel were open to the public in April 2000 and December 2000 respectively.

INFLUENCE OF TUNNELS ON GROUNDWATER SYSTEM

As a consequence of groundwater inflows during tunnel construction, groundwater drawdown occurred in the local aquifers and Silurian bedrock. As previously

mentioned the CIS overlying the tunnels and aquifers is susceptible to settlement through groundwater lowering and a number of modifications were made during construction to control groundwater drawdown and these modifications include:

- Sections of the tunnel liner were modified from drained liner to a tanked liner, the tanked sections were lined with a full water proofing membrane;
- A number of recharge wells were installed to pump water back into the local aquifers.

Other modifications made during construction include:

- Rock anchors were installed to support the liner against increased hydrostatic pressures and increase the tunnel liner integrity.
- Installation of settlement monitoring points

The installation of rock anchors has penetrated through the water proofing membrane in the tunnel causing some seepage in the tanked tunnel sections.

Groundwater inflows within the tanked and drained sections of the tunnels are collected within tunnel drainage system and pumped to the ground surface.

Ongoing grouting through the tunnel liner has further reduced the groundwater inflow into the tanked sections of the tunnels.

Five permanent deep recharge wells are now in operation and recharge water into the HA and MSG aquifers. Initially, the recharge water used potable water. CityLink Melbourne Limited (CityLink) recognized that this was

an inappropriate use of potable water and undertook to find an alternative source of recharge water.

In 2002 Citylink Melbourne Ltd constructed a Water Treatment Plant (WTP) adjacent to the Swan Street Shaft at a cost of \$1.3million dollars (The Age 2002). The WTP receives water pumped from the drainage system in the tunnels, treats the water, and then pumps the recycled water to the permanent recharge wells and re-injects it back into the aquifers.

The modification works during tunnel construction have assist in reversing the groundwater drawdown observed in the HA, MSG and Silurian.

Since 2002 Maunsell Australia Pty Ltd has managed the management of the groundwater system in the vicinity of the Citylink Tunnels.

POST CONSTRUCTION - GROUNDWATER MANAGEMENT SYSTEM

The groundwater management system for the Citylink Tunnel consists of managing groundwater recharge flows, collecting groundwater monitoring data, populating a database, reviewing tunnel inflow, groundwater and settlement trends and assessing these trends against management targets. Monitoring data for key instruments and the groundwater assessments are presented in a report issued to the State Government (or there representative) at regular intervals. The reports are currently issued monthly.

MONITORING DATA

As part of the groundwater management system, the following is undertaken:

- At the surface:
 - Monitoring of about 100 standpipe piezometers and vibrating wire piezometers
 - Monitoring of about 500 survey settlement pins
 - Monitoring and control of 5 permanent deep recharge wells
 - Monitoring of 10 shallow passive recharge wells
 - Monitoring of total recharge flow from the WTP.
- In the tunnels:
 - Monitoring of external hydrostatic pressures on the tunnel liner
 - Monitoring of tunnel floor movement
 - Monitoring of groundwater inflows in drained and tanked sections of the tunnels
 - Monitoring of tunnel drainage pump data.

Currently, the groundwater monitoring data is collected manually and at frequency ranging from weekly to every 6 weeks.

Monitoring of some instruments date back to 1992.

DATABASE

During construction the monitoring data was stored in Microsoft ® Excel¹ due to its ability to produce charts and widespread use. However, it was considered that Excel was not a suitable database to store and assess the groundwater data as a long term system. The data set recently passed 100,000 numbers and is growing. It was deemed appropriate to search for a suitable database.

Microsoft ® Access 2000 was adopted as the database for the groundwater data as:

- It is able to process large volumes of data;
- Compatible with most software packages;
- Readily available (with Microsoft Professional Office 2000);
- Provides different levels of security, allowing the database manager to check and correct data, and allow users to enter data only.
- Provides a switchboard manager enabling easy navigation around the database.

To assist in managing the input of data two data input templates were developed within Access. The first template allowed the user to populate a large volume of data into the database quickly, such as survey data. The second template allows the user to enter single data values. This template displays previous data readings and flags if the most recent data is approaching or exceeding management target limits. Flags and management target limits will be discussed further later on.

The data input templates reduce the raw reading into more meaningful data. For example, groundwater levels in standpipe piezometers are recorded as depth from ground surface level to groundwater level and the data input templates reduce this data to a RL of the top of groundwater. In addition to reducing the raw data, the database calculates the rate of change of the reduced value since the previous reading.

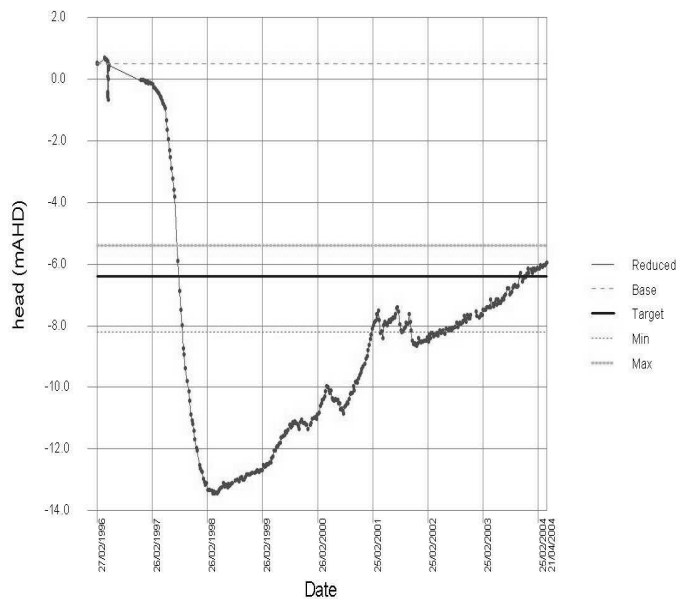
The management system consists of four management target levels:

- Maximum level;
- Target level;
- Minimum level; and
- Maximum rate of change.

The Target level is a nominated level at which the actual data should be heading towards. The Maximum level and Minimum level are upper and lower bound limits respectively. The Maximum rate of change is the upper limit of change between two consecutive readings. An

¹ Microsoft ® Access 2000 and Excel are property of the Microsoft Corporation

Figure 3 Groundwater Management Target Levels



example of the management target levels is shown on Figure 3.

Flags operate on two levels:

- Orange flag; and
- Red flag.

Both the reduced level and rate of change can raise a flag. If the reduced level is closer to the maximum level or minimum level than the target level, an orange flag is raised and provides a warning that the reduced level is moving away from the target level. If the reduced level

moves outside the maximum or minimum level then a red flag is raised and provides a warning that the reduced level is outside management target levels. The same applies to the maximum rate of change.

The philosophy behind the two color flag system is to allow early identification of data trending away from the target level and provide an opportunity to make changes to the system, ie increase or decrease recharge flow. Not all instruments have target levels due to the reading accuracy and error margin associated with particular instruments. For example, some inflow monitoring instruments have an accuracy range of $\pm 50\%$.

ArcView is able to quickly produce charts for groundwater data and settlement data. It is able to produce contours of groundwater drawdown and settlement, see Figure 4.

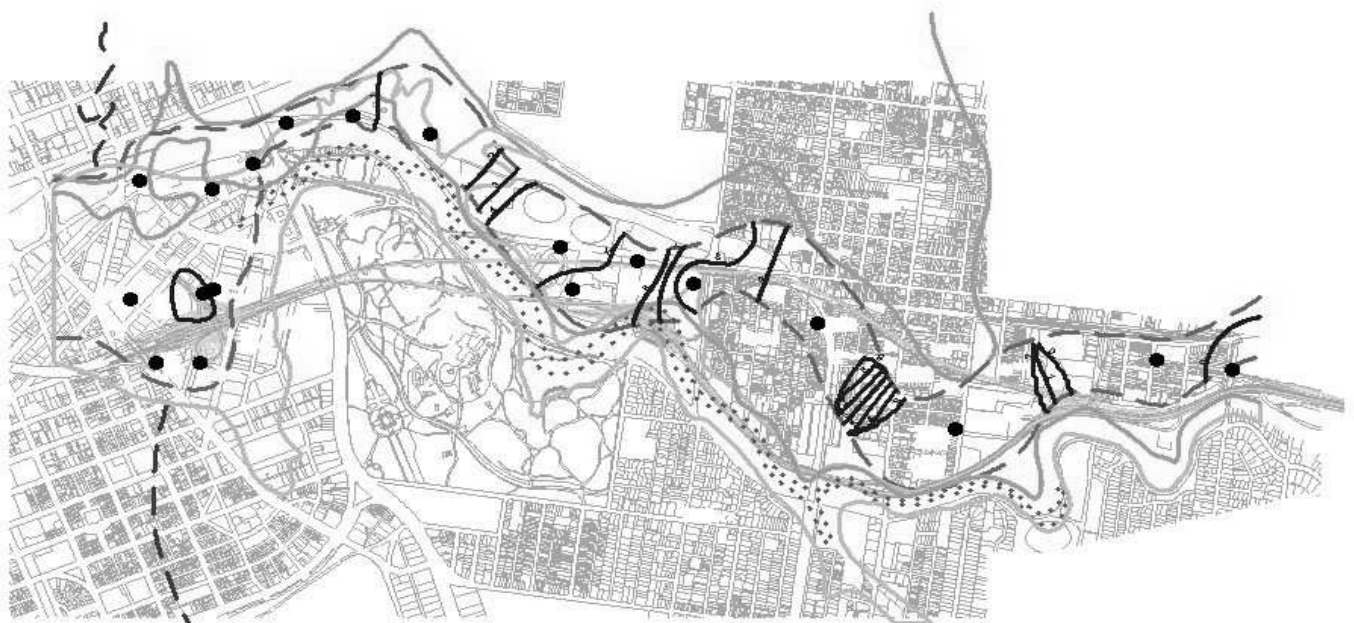
Groundwater drawdown contours are generated in ArcView where the drawdown is the difference between the current groundwater level and the pre-construction 'base level'. The base levels have been assigned based on results of historical data and groundwater modeling undertaken by Golder Associates during construction.

ONGOING MANAGEMENT ISSUES

The existing monitoring network will be progressively reduced as groundwater recovery occurs.

Ongoing improvements to the recharge system include automation of recharge system with telemetry support, the use of dataloggers in the tunnel, and investigating methods of increasing the operating life of recharge wells.

Figure 4 Groundwater drawdown contour



Upgrading of the database will require significant input due to the non-compatibility between ArcView 3.1 and the upgraded current version, Version 8.

CONCLUSIONS

The management of the groundwater system in the vicinity of the Citylink Tunnels is concerned with the protecting the Tunnels themselves as well as private and public assets above the tunnel. The groundwater management system will be an ongoing process for the life of the tunnels.

The groundwater management system is primarily concerned with monitoring the behaviour of the groundwater in the important aquifers and ongoing settlement of compressible soils.

The monitoring data is stored in Microsoft Access 2000 and the GIS software package ArcView (version 3.1) is used to assist in assessing the behavior of the aquifers and settlement rates. The use of a database and GIS software tools allow rapid and easy review of groundwater and ground surface trends. Developing and implementing management target levels assist in reviewing and assessing vast volumes of data.

I have been actively involved in developing and implementing a system for the management of the groundwater system. I was initially involved in developing and populating of the database, collecting data and over the last year have managed this project.

ACKNOWLEDGEMENTS

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REFERENCES

1. Golder (1997), "Report on Further Geological Investigation, East of Princes Bridge, Driven Tunnels, Melbourne City Link Project", by Golder Associates Pty Ltd, October 1997.
2. Short, M., Weeks, N., Woodall, R., Porter, S., (1999) "Design of the Melbourne City Link Tunnels", Tenth Australian Tunnelling Conference, Melbourne, March 1999.
3. Hutchison B. J., Lamb I.A., (1999), "Construction Management of Hydrogeological Aspects of Melbourne's City Link Project", Tenth Australian Tunnelling Conference, Melbourne, March 1999.

4. Taylor E., Munro G., (1999), "The Race to Finish – From Design to Construction – 5km of Three Lane Road Tunnel in Three Years", Tenth Australian Tunnelling Conference, Melbourne, March 1999.
5. Adams, D.N., McRae, M.T., Hutchison B.J., and Porter, S.J., (1999), "Design and construction of the City Link driven tunnels", 1999, p 553-569
6. The Age (2002), 'CityLink starts recycling project', by Liz Minchin, 23 July 2002.

Climatic Factors for AS2870 for the Sydney Region

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Summary AS2870-1996 “Residential Slabs and Footings – Construction” correlates depths of design suction change (H_s) to regional climatic zones as delineated by the Thornthwaite Moisture Index (TMI). TMI is a measure of aridity, quantifying the cyclical wetting and drying of soils. TMI is a function of rainfall, potential evapotranspiration and soil water-holding capacity, and provides an indication of moisture balance.

In this paper TMI is calculated for various Bureau of Meteorology sites across Sydney, based on meteorological records. The results are plotted and contoured to give a map of TMI isopleths for the metropolitan region. This paper complements previous work in other States, and will provide improved estimates of H_s needed for the calculation of characteristic surface movement (y_s) in metropolitan Sydney, for classification of residential sites under AS2870-1996.

NOTATIONS

a	Factor for calculating adjusted potential evapotranspiration
d_i	Deficit for month i
e_i	Evapotranspiration for month i
e_i'	Adjusted potential evapotranspiration for month i
h	Depth from ground surface
h_i	Heat index for month i
Δm_i	Moisture balance for month i
n	Number of years of record
p_i	Precipitation for month i
r_i	Runoff for month i
t_i	Temperature for month i
$t_{max\ i}$	Mean monthly maximum temperature for month i
$t_{min\ i}$	Mean monthly minimum temperature for month i
Δu	Suction change
y_s	Characteristic surface movement
D_i	Day length correction factor for month i
D_y	Deficit for year y
E_y	Evapotranspiration for year y
H_s	Depth of design suction change
H_y	Heat index for year y
I_{ps}	Shrinkage index
I_{pt}	Instability index
N_i	Number of days in month i
R_y	Runoff for year y
S_i	Storage for month i
S_{i-1}	Storage for month $(i-1)$
S_{max}	Maximum soil storage
TMI	Thornthwaite Moisture Index

1. INTRODUCTION

Residential foundation design is primarily governed by soil movement, with the majority of structural damage caused by excessive soil movement. This paper presents the use of TMI as a tool to better predict soil

movement behaviour for the design of residential slabs and footings.

AS2870-1996 “Residential Slabs and Footings – Construction”, defines the *characteristic surface movement*, y_s , as “the movement of the surface of a reactive site caused by moisture changes from characteristic dry to characteristic wet condition in the absence of a building and without consideration of load effects”. There are many variables used for the estimation of y_s , one of which is the *depth of design suction change*, H_s , the depth at which the soil moisture condition is constant with climate variations. H_s can be estimated by considering climatic factors, one of which is TMI . This paper presents the estimation of H_s , from TMI values for the Sydney region.

2. TRADITIONAL METHODS OF DETERMINING H_s

2.1 Development of AS2870

The Australian Standard AS2870 “Residential Slabs and Footings” came into existence to standardise various state regulations or practices on the design of residential footings. The Standard offers guidance, and sometimes definitive rules, for the design of house foundations in different soil materials, particularly in reactive clays. It is the authors’ opinion that this Standard represents close to world’s best practice in the design of footings for small structures on reactive soils.

In New South Wales, there had been many structural failures in buildings constructed on reactive soils, particularly in the late 1970s during a state-wide drought (NSW Builders Licensing Board, 10). As a result, the Builders Licensing Board commissioned the study of swelling soils in Sydney (Coffey and Partners Pty Ltd, 4), and subsequently published “Classification of NSW Soils for Housing” (BLB, 10) as a guide for engineers, designers and builders.

At a similar time, the Standards Association of Australia Committee published the draft Australian Standard for Residential Slab and Footings (DR85108, 15) for public comment. This draft Standard defined several key risk management objectives as follows:

- Design foundations allowing for a 5% chance of exceedence in the life of the building. It identifies the life of a building as 50 years.
- Calculate characteristic surface movements expected for 95% of all houses, thus allowing 1 in 20 houses to perform less satisfactorily.

AS2870 was first published in 1986, with revisions in 1988, 1990 and 1996.

2.2 Characteristic Surface Movement

The characteristic surface movement at any given site, in accordance with AS2870-1996, is given by:

$$y_s = \frac{1}{100} \int_0^{H_s} I_{pt} \Delta u \cdot dh \quad (1)$$

The following sections contain some reflection on the development of the variables governing y_s .

Suction Change Δu

AS2870 defines soil suction as “a measure of the internal stress caused by moisture in unsaturated clays. It is useful in the analysis of reactive clays because it is more strongly a function of the climate and vegetation than it is of soil type.”

Soil suction changes as a result of climatic variations. At a depth below the surface this seasonal suction change reduces to an equilibrium level (see Figure 1). The depth at which this equilibrium sensibly occurs is termed the *depth of design suction change*, H_s .

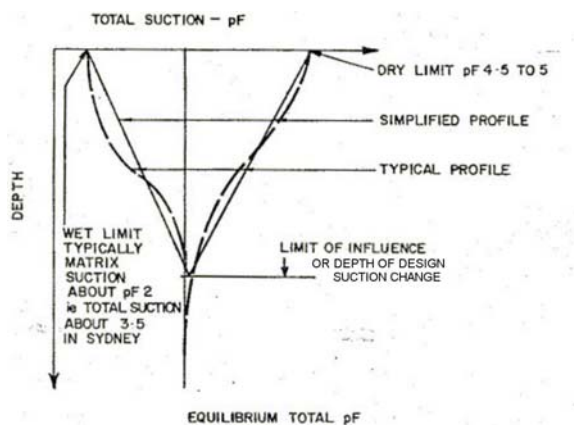


Figure 1. Soil Suction Profile with Depth (Coffey, 4)

Since its inception, the Standard has given a maximum surface suction change value for Sydney of 1.5 pF. There is little published support for this value but it is important to understand that the Standard is a “package”

where H_s , Δu and α combine together to provide an estimate of y_s .

Instability Index I_{pt}

The instability index, I_{pt} , is the change in height of clay per unit suction change, taking into account the applied stress, lateral restraint and suction range during the test. It is given by:

$$I_{pt} = \alpha \cdot I_{ps} \quad (2)$$

where

$$\alpha = \begin{cases} 1.0 & \text{cracked zone (unrestrained), or} \\ 2.0 - h/5 & \text{uncracked zone (restrained)} \end{cases}$$

Depth of Cracked Zone

The cracked zone relates to the zone in which predominantly vertical shrinkage cracks exist seasonally. Within this zone, swelling is unrestrained, that is, it occurs both horizontally and vertically. The depth of cracked zone for a particular location is dependent on its seasonal climatic conditions. Little research relating to this variable has taken place to date, and the Standard gives values ranging from $0.25H_s$ to $1.0H_s$.

Shrinkage Index I_{ps}

The shrinkage index I_{ps} is defined as “the percentage change in height of clay per unit suction change without any load or lateral restraint” (Walsh and Cameron, 23). The laboratory test methods used to determine the index include the shrink-swell test, loaded shrinkage test, and the core shrinkage test.

Where more than one test is performed on the same soil horizon, the Standard prescribes the use of average values of test data in the calculation of I_{pt} .

2.3 Depth of Design Suction Change H_s

H_s , as defined in the preceding section, is dependent on climatic variation. H_s is prescribed in the Standard for major cities. Different values for H_s have been suggested for Sydney in the Standard revisions, ranging from 1 m to 3 m. H_s is very difficult to quantify in the field, and empirical methods for its determination include the use of *TMI*.

AS2870-1996 uses *TMI* to calculate H_s in Victoria, based on the work by Aitchison and Richards (1). The Standard also states that *TMI* may be used to estimate H_s for other locations.

3. THE DEVELOPMENT OF THE THORNTHWAITE MOISTURE INDEX

TMI was originally developed as an agricultural index to aid farmers in their prediction of crop needs. It was adopted in the engineering field for the estimation of subgrade moisture, in terms of soil suction. Its relationship with soil suction distribution gives *TMI* relevance in the calculation for characteristic surface movement (y_s) in reactive clays.

3.1 TMI as an Agricultural Index

Dr. C.W. Thornthwaite, an American climatologist, developed a method of assigning soil moisture balance by way of assessing the climatic conditions. The relationship between water surplus (r) and water deficiency (d) and how it relates to water need (e) in soil constitutes an index of humidity or aridity. Dr Thornthwaite (22) proposed the following relationship:

$$TMI = \frac{100r - 60d}{e} \quad (3)$$

3.2 TMI as an Engineering Parameter

Work in the 1960s by engineers investigating pavement subgrade moisture conditions has led to the use of *TMI* in engineering, and in particular, in the soil physics field. Research by Russam and Coleman (13), and Aitchison and Richards (1) in Australia demonstrate a relationship between *TMI* and the soil suction in pavement subgrade. Richards, Peter and Emerson (12) drafted a map of *TMI* for Australia, as reproduced in Figure 2.

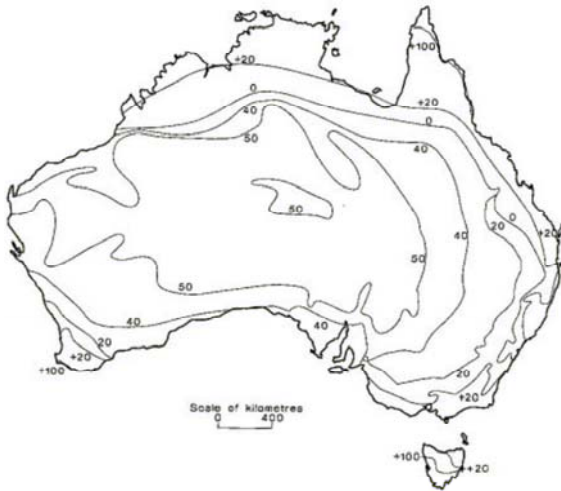


Figure 2 – TMI for Australia (Richards et.al., 12)

More recently, civil engineers have used the relationship between *TMI* and soil suction profile to assess the behaviour of footings founded on expansive soils (Mitchell, 9 and McKeen et.al., 8).

3.3 TMI for Calculating Soil Movement

Australian engineers such as Smith (14), Fityus et.al. (5), Barnett and Kingsland (2), and Fox (6,7) have broadened the work by Aitchison and Richards in the 1960s to use a climatic index, *TMI*, to assess H_s .

AS2870-1996 “Residential Slabs and Footings – Construction” correlates H_s to regional climatic zones as delineated by *TMI*, as reproduced in Figure 3 for Melbourne. However, to date, the relationship between H_s and *TMI* for the Sydney region has not been assessed in detail.

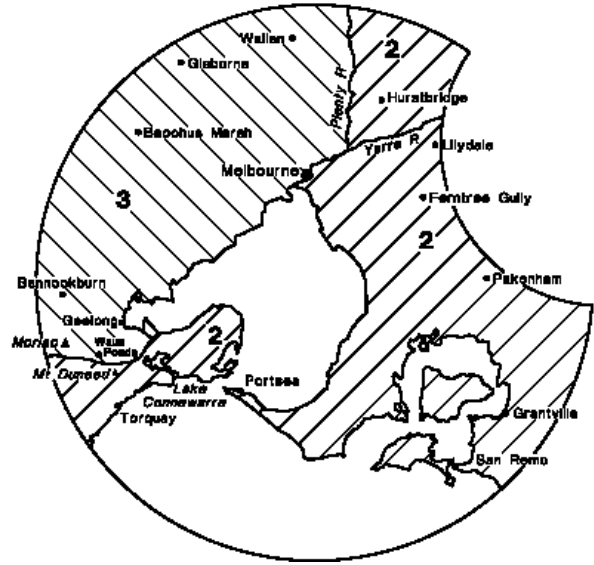


Figure 3 - Melbourne Climatic Zones as delineated by *TMI* (AS2870-1996)

4. METHODOLOGY FOR CALCULATING TMI

TMI is an annual water balance based on rainfall and estimated evapotranspiration, calculated on a monthly basis. High positive values indicate a wetter climate, and high negative values occur in arid regions.

The following, reproduced from Thornthwaite (22), are major components in the computation of *TMI*:

1. Determination of the adjusted potential evapotranspiration.
2. Computation of a moisture balance to determine allocations of water to storage, deficit, or runoff.
3. Computation of annual summations that are used to determine the index value.
4. Determination of statistical parameters, ie, mean, standard deviation, and range of annual *TMI*.

4.1 Determination of the Adjusted Potential Evapotranspiration e'_i

To determine the adjusted potential evapotranspiration e'_i , the following inputs are required:

- Mean monthly temperature (t_i), calculated as the average of $t_{max\ i}$ and $t_{min\ i}$ in monthly values for the period of record.
- Day length correction factor (D_i), given in Table 1, for twelve months.
- Number of days for each month (N_i).

1. Heat Index (h_i)

$$h_i = (0.2 \cdot t_i)^{1.514} \quad (4)$$

2. Annual Heat Index (H_y)

$$H_y = \sum_{i=1}^{12} h_i \quad (5)$$

3. Unadjusted Potential Evapotranspiration (cm/mth)

$$e_i = 1.6 \cdot \left(\frac{10 \cdot t_i}{H_y} \right)^a \quad (6)$$

$$\text{where } a = 6.75 \times 10^{-7} \cdot H_y^3 - 7.71 \times 10^{-5} \cdot H_y^2 + 0.017921 \cdot H_y + 0.49239 \quad (7)$$

4. Adjusted Potential Evapotranspiration (cm/mth)

$$e'_i = e_i \cdot \frac{D_i N_i}{30} \quad (8)$$

4.2 Determination of the Moisture Balance

The inputs for determining the moisture balance include:

- Monthly Precipitation (p_i), in cm, for the period of record.
- Adjusted Potential Evapotranspiration (e'_i), calculated as above, for the period of record.

The decision tree in Figure 4 illustrates the method to determine the storage (S_i), deficit (d_i) and runoff (r_i) for each month.

4.3 Calculation of TMI

The annual TMI_y for year y is calculated as follows:

$$TMI_y = \frac{100 \cdot R_y - 60 \cdot D_y}{E_y} \quad (9)$$

where for year y ,

$$R_y = \sum_{i=1}^{12} r_i \quad (10)$$

$$D_y = \sum_{i=1}^{12} d_i \quad (11)$$

$$E_y = \sum_{i=1}^{12} e'_i \quad (12)$$

TMI for a particular location is:

$$TMI = \frac{1}{n} \sum_{y=1}^{y=n} TMI_y \quad (13)$$

where n is the years of record. The equation calculates TMI in cm.

The statistical parameters, such as mean, standard deviation, and range of annual TMI , are important indicators of the climatic conditions of a particular location. Where less than 20 years of record is available for a particular location, these parameters will assist in determining the relevance of TMI data.

Latitude	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
20	1.14	1.00	1.05	0.97	0.96	0.91	0.95	0.99	1.00	1.08	1.09	1.15
25	1.17	1.01	1.05	0.96	0.94	0.88	0.93	0.98	1.00	1.10	1.11	1.18
30	1.20	1.03	1.06	0.95	0.92	0.85	0.90	0.96	1.00	1.12	1.14	1.21
35	1.23	1.04	1.06	0.94	0.89	0.82	0.87	0.94	1.00	1.15	1.17	1.25
40	1.27	1.06	1.07	0.93	0.86	0.78	0.84	0.92	1.00	1.15	1.20	1.29

Table 1. Day Length Correction Factor, Southern Hemisphere (McKeen et.al., 8)

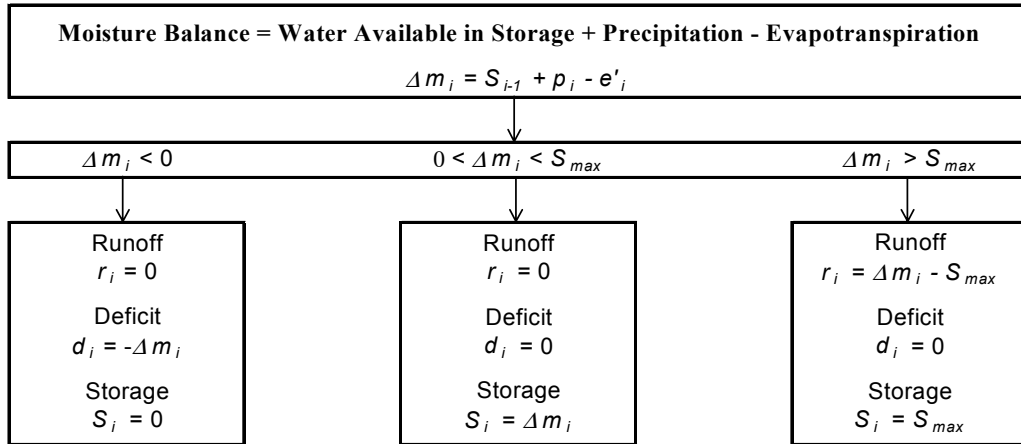


Figure 4. Decision Tree for Moisture Balance Computation

This methodology makes the following assumptions:

1. January is selected as the start month.
2. The soil storage capacity is the holding capacity of the soil, which is dependent on soil type. For the purpose of *TMI* calculation the value of 100mm is used. This value appears to be adopted in all such calculations in Australia.
3. The initial soil storage is taken to be zero (0 cm) if the location of interest is dry in January, 10 cm if it is wet, and 5 cm if it is temperate. The choice of initial storage value has little effect for long record.
4. *TMI* for a location is defined to be the average of the annual *TMI* calculated for that location (Richards, 11). For locations with meteorological records of less than 20 years, *TMI* may not give a true reflection of the location's climate.

4.4 Other Proposed Methods

Alternative methods have been proposed in recent years for *TMI* calculation, which generally use long-term monthly average climatic values instead of monthly values for the period of record, that is, *TMI* of an average year, not the average *TMI*. It is important that actual data is used, as *TMI* is essentially an index of extreme climatic conditions, aligned with the risk-based principles of the Standard. These alternative methods inevitably produce misleading *TMI* values. This will be discussed further in an extended paper.

5. *TMI* MAP OF SYDNEY

A *TMI* map, presented in Figure 5, has been plotted from *TMI* results from 40 stations in the Sydney region where both temperature and rainfall data are available for at least 5 years. *TMI* values from 23 other locations, where only rainfall data are available, are also used. At these locations temperature data from similarly placed locations are used in the computation. Where the record length is between 5 years and 20 years, the statistical spread is considered for contouring purposes. For reasons of space the results are not tabulated here, but will be reproduced in a separate paper.

The map indicates that the coastal fringes in the Sydney Basin are of a wet coastal climate, becoming more arid with distance from the coast. The Blue Mountain ranges, however, has a cooler alpine climate.

The relationship between climatic zones and the value of H_s is reproduced in Table 2.

Climatic Zone & Description	<i>TMI</i> range	H_s
1 Alpine / Wet coastal	> +40	1.5m
2 Wet temperate	+10 to +40	1.8m
3 Temperate	-5 to +10	2.3m
4 Dry temperate	-25 to -5	3.0m
5 Semi-arid	< -25	4.0m

Table 2. Climatic Zones (AS2870-1996)

From Figure 5, it is demonstrated that the climate of Sydney varies from the wet coastal to the drier temperate, with H_s varying from 1.5m to 2.3m.

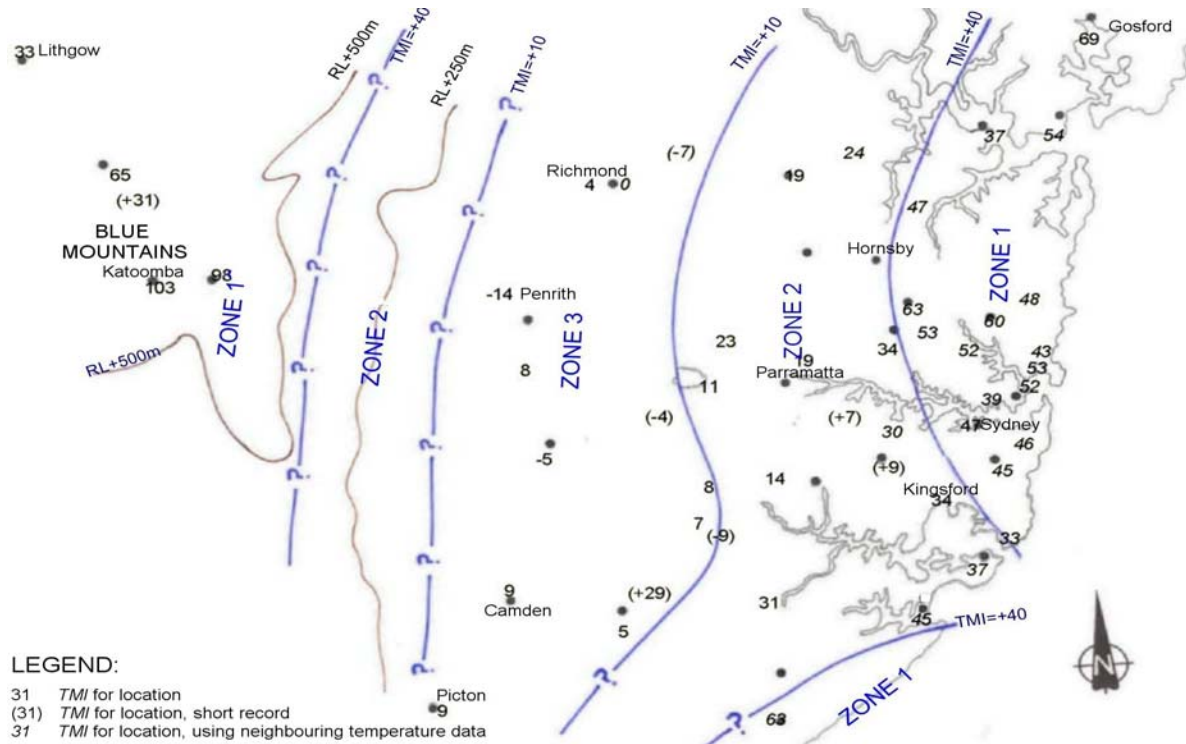


Figure 5. *TMI* Map of Sydney

5.1 Coffey and Partners Study, 1985

It is the authors' experience that the current Standard works well in Sydney when correctly applied in investigation, design and construction. It is therefore important not to adopt a new H_s value without consideration to the rest of the "package".

Coffey (4) performed a detailed study of reactive soils in Plumpton Park, Quakers Hill and Minchinbury, which includes calibration of reactivity with measured soil movement. The calibration was based on a H_s of 1.5m, which is a widely accepted value for Sydney, and a higher Δu value of 1.5 pF was given as a result of the calibration. This calibration was adopted in AS2870.

According to Figure 5, these three sites have a H_s value of 1.8m. As the Standard is a "package" where H_s , Δu and α combine together to estimate y_s , a different, lower value of Δu may be considered for Sydney as the result of a re-calibration. This will avoid yielding higher y_s for the area and producing conservative footing designs.

6. CONCLUSION

The use of *TMI* as a tool for estimating climate-derived H_s values is investigated in this paper. *TMI* values calculated for 63 locations in Sydney are provided, and a contour map produced for use in site classification in Sydney, in accordance with AS2870-1996.

7. REFERENCES

1. AITCHISON, G.D. AND RICHARDS, B.G. (1965). "A Broad-scale Study of Moisture Conditions in Pavement Subgrades throughout Australia", *Moisture Equilibria and Moisture Changes in Soil beneath Covered Areas*, Butterworth, Australia, pp.186-236.
2. BARNETT, I.C. AND KINGSLAND, R.I. (1999). "Assignment of AS2870 Soil Suction Change Profile Parameters to TMI Derived Climatic Zones for NSW", *Proc. 8th ANZ Conference on Geomechanics*, Hobart, AGS, pp.149-155.
3. CAMERON, D.A., WALSH, P.F. AND WYNN-JONES, M. (1989). "Site Classification Data from an Experimental Cover in Sydney's West", *Fifth National Local Government Engineering Conf.*, Sydney, pp.118-122.
4. Coffey and Partners Pty Ltd (1985). "Sydney Swelling Soils Study", Report S7032/2.
5. FITYUS, S., WALSH, P. AND KLEEMAN, P. (1998). "The Influence of Climate as Expressed by the Thornthwaite Index on the Design Depth of Moisture Change of Clay Soils in the Hunter Valley", *Proc. of the Geot. Engineering and Eng. Geology Conf.*, Newcastle, AGS, pp. 251-265.
6. FOX, E. (2000). "A Climate-based Design Depth of Moisture Change Map of Queensland and the Use of such Maps to Classify Sites under AS2870-1996", *Australian Geomechanics*, Vol. 35, No. 4, pp.53-60.
7. FOX, E. (2002). "Development of a Map of Thornthwaite Moisture Index Isopleths for Queensland", *Australian Geomechanics*, Vol. 37, No. 3, pp.51-59.
8. McKEEN, R.G. AND JOHNSON, L.D. (1990). "Climate-controlled Soil Design Parameters for Mat Foundations", *J. Geotechnical Engineering*, Vol. 116, No. 7, pp.1073-1094.
9. MITCHELL, P.W. (1979). *The Structural Analysis of Footings of Expansive Soil*, Newtown, 152p.
10. New South Wales Builders Licensing Board (1985). *Classification of N.S.W. Soils for Housing*, NSW BLB, Sydney, 42p.
11. RICHARDS, B.G. (2004). pers. comm.
12. RICHARDS, B.G., PETER, P. AND EMERSON, W.W. (1983). "The Effects of Vegetation on the Swelling and Shrinking of Soils in Australia", *Geotechnique*, Vol. 33, pp.127-139.
13. RUSSAM, K. AND COLEMAN, J.D. (1961). "The Effect of Climatic Factors on Subgrade Moisture Conditions", *Geotechnique*, Vol.XI, No.1, pp.22-28.
14. SMITH, R.L. (1993). "Estimating Soil Movement in New Areas", *Extending the Code beyond Residential Slabs & Footings*, Melbourne, AGS.
15. Standards Association of Australia (1985). *DR85108, Draft Australian Standard for Residential Slabs and Footings*, 65p.
16. SAA (1986). *AS2870-1986, Residential Slabs and Footings*, 52p.
17. SAA (1988). *AS2870 Supplement 1-1988, Commentary, Residential Slabs and Footings*, 38p.
18. SAA (1988). *AS2870.1-1988, Residential Slabs and Footings Part 1: Construction*, 36p.
19. SAA (1990). *AS2870.2-1990, Residential Slabs and Footings Part 2: Guide to Design by Engineering Principles*, 28p.
20. Standards Australia (1996). *AS2870 Supplement 1-1996, Residential Slabs and Footings-Construction- Commentary*, 48p.
21. SA (1996). *AS2870-1996, Residential Slabs and Footings- Construction*, 71p.
22. THORNTHWAITE, C.W. (1948). "An Approach toward a Rational Classification of Climate", *Geographical Review*, Vol. 38, No. 1, pp. 55-94.
23. WALSH, P. AND CAMERON, D. (1997). *HB28-1997, The Design of Residential Slabs and Footings*, Standards Australia, 158p.

Christchurch Liquefaction Hazard Mapping

Brent Clough: BE (Hons) Civil, GIPENZ; NZ Geotechnical Society; ISSMGE

A liquefaction study of Christchurch City was undertaken for the Canterbury Regional Council (Environment Canterbury) to create a set of liquefaction hazard and ground damage maps based on actual soil strengths from pre-existing data. The study required research and data collection from records held at Environment Canterbury and various Christchurch City Council offices, as well as contractor/consultancy information to obtain borehole and CPT records. The key information such as soil type, strength and water table levels was entered into a liquefaction database specifically designed for the project. Calculation of the liquefaction hazard for Christchurch utilised the information contained and a program embedded within the database (using the method of Youd & Idriss⁽¹⁾, 2001). The project created liquefaction hazard and ground damage maps at compilation scales of 1:25,000.

INTRODUCTION

In recognition of the earthquake threat to Canterbury and specifically Christchurch, Canterbury Regional Council (Environment Canterbury) initiated a programme in 1996 to assess the risk posed by earthquakes, identify mitigation options, and implement measures to ensure that the level of risk in Christchurch is acceptable.

Earthquake induced liquefaction (where saturated soils lose their strength when shaken) is recognised as a significant potential hazard for many parts of Christchurch City. Liquefaction maps published previously have shown the hazard as a function of soil distribution across the City, but have not taken into account the strength of soils. This may have exaggerated the hazard (as some soils of liquefiable grain size may have been too dense to actually liquefy). This liquefaction study and mapping incorporates the recorded strengths of soils in Christchurch, as well as investigating sensitivity to earthquakes of various magnitudes and water table changes.

LIQUEFACTION OVERVIEW

Liquefaction is the process in which loose saturated soils lose strength when cyclically loaded by earthquake vibrations. This causes soils to temporarily behave in a fluid like manner, similar to a “quick-sand”. Susceptibility to liquefaction requires certain criteria to be met before liquefaction can occur. Such conditions lead to liquefaction occurring more frequently in recently (Holocene) deposited saturated sand and silty sand soils. Such deposits are often found near rivers and in coastal areas such as deposits found in the central and eastern side of Christchurch.

CHRISTCHURCH AND ITS GEOLOGY

Christchurch city is located on sandy Holocene deposits along the coastal margin, alluvial outwash gravels to the west, and on the northern slopes of the adjacent loess covered volcanic Port Hills of Banks Peninsula.

Originally the site of Christchurch was a mixture of: swamp lying behind beach dune sand; estuaries and lagoons; and gravel; sand; and silt from the river

channel and coastal floodplain of the Waimakariri River. The Avon and Heathcote rivers meander through the city to form the main drainage system. These rivers form an estuary beside the coast where longitudinal movement of the dune sand enclose the estuary. The remnants of the older beach dune sands occur up to 6km inland from the coast.

Christchurch has developed as a service centre for agriculture, horticulture, orcharding, and market gardening on the fertile soil of the Canterbury Plains. Groundwater in the gravel aquifers underlying the northern Canterbury Plains provides a plentiful supply of high quality water for domestic, industrial, livestock and irrigation requirements.

Christchurch is adjacent to the tectonically active Alpine Fault region of New Zealand. Consequently, the geology and tectonic setting of the area indicate that future large earthquakes may have a major impact on the city, especially in terms of liquefaction.

STUDY METHODOLOGY

The liquefaction study undertaken has included:

- Research of consent records at Environment Canterbury which may have included soil information (e.g. borelogs), other than that already contained in Environment Canterbury’s Well Database⁽²⁾ (which does not include any soil strength data).
- Research of property files and other general records at Christchurch City Council for soil information (this primarily resulted in shallow borelogs that were required for building consent applications).
- Scanning of paper borelog records to create a digital record of all soil information encountered during the study.
- Research into soil information held by contractors and consultants, and obtaining their borehole and CPT records where this information is obtainable (many instances existed where the information was not made available due to legal reasons).
- Analysis of historical water table information of bores monitored by Environment Canterbury and the Christchurch City Council to determine typical levels.

- Investigations into the level of seismicity of the city and the creation of typical earthquake scenarios.
- Creation of a liquefaction database to store key liquefaction information (such as soil type, soil strength and water table levels).
- Transfer of the key information (such as soil type, soil strength and water table levels) into the liquefaction database.
- Calculation of the liquefaction hazard based on the information contained within the database, via an automated programme embedded in the database.
- The creation of Liquefaction Hazard Maps and Ground Damage Maps (both using two different ground water elevation scenarios), compiled at scales of 1:25,000.

No fieldwork was undertaken to confirm or otherwise the soil and water table information used in the liquefaction hazard assessment.

LIQUEFACTION PREDICTION FORMULAE

The sensitivity of a soil to liquefaction could have been established by either using multiple prediction procedures to estimate liquefaction, or to vary the earthquake parameters used in analysis. In this study the earthquake parameters were varied to establish a soil's sensitivity to liquefaction. The prediction method employed is based on the Simplified Seed procedure (Youd & Idriss⁽¹⁾, 2001), which is a widely used method.

In this method, the onset of liquefaction occurs when the Cyclic Stress Ratio (CSR) exceeds the Cyclic Resistance Ratio (CRR). The simplified approach is defined as follows (more information on this procedure can be found in Youd & Idriss⁽¹⁾, 2001):

$$CSR = 0.65 \left(\frac{a_{\max}}{g} \right) \left(\frac{\sigma_{vo}}{\sigma_{vo}'} \right) r_d$$

where:

- $a_{\max}$ = maximum horizontal acceleration at the ground surface
- g = acceleration due to gravity (9.81 m/s²)
- σ_{vo} = total vertical overburden stress
- σ_{vo}' = effective vertical overburden stress
- r_d = stress reduction coefficient

and

$$CRR = \left[\frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200} \right] MSF$$

where:

- $(N_1)_{60}$ = corrected N-value
- MSF = magnitude scaling factor

DESIGN EARTHQUAKES

An Alpine fault, foothills fault and a local earthquake event type scenario were initially considered for the liquefaction analyses. The smaller local earthquake may not have sufficient strong ground motion cycles (e.g. 5 cycles) to cause liquefaction and therefore has not been adopted. The foothills earthquake scenario was considered, but was not chosen due to the return interval being around 2,000 years compared to the Alpine earthquake having a return period of around 300 to 500 years.

Three different ground accelerations, with peak ground accelerations of 0.12g, 0.20g and 0.34g for the Alpine event, have been used to obtain a sensitivity analysis for soil liquefaction. These represent return period events on the Alpine faultline of 50, 150 and 475 years respectively (NZS4203⁽³⁾). By choosing different levels of shaking, the trigger points for liquefaction can be determined. For example, where a soil liquefies due to a relatively low level of earthquake shaking, the liquefaction hazard is High, as the soil has a low resistance to liquefaction. Likewise, if a soil liquefies at a threshold of 0.34g shaking and not the lesser value of 0.12g or 0.20g, then the soil has some resistance to liquefaction and is classified as "Low Susceptibility".

A detailed discussion of the earthquake parameters used in this study is included in the Christchurch Liquefaction Study Report⁽⁴⁾.

SOIL DENSITIES

Soil densities will vary from site to site across Christchurch, with depth at each site and also as different soil types are encountered, such as peat, clay, silt, sand, or gravel. Furthermore, one sand deposit may have a different density to an adjacent sand deposit, likewise for other soil types. As no laboratory tests are available in the database to define the actual soil densities, only predictions of the densities can be made (usually based on soil strength). To simplify the prediction process, densities of 1.8t/m³ have been used for saturated soils, and 1.65t/m³ for dry soils in the database (non-liquefiable soils are less likely to have densities of this order). The densities used are typical for many of the liquefiable sands within Christchurch.

This generalisation may potentially over or underestimate the confining stress of soil layers and therefore the liquefaction hazard for a site may be misrepresented. However, the difference between actual and estimated overburden pressure is likely to be small and therefore the estimation has a small effect on the prediction process. More complex estimation of the soil density could be undertaken based on the soil type and strength, but this is considered unwarranted for the compilation scale of the maps (1:25,000).

WATER TABLE LEVELS

The water table level affects the confining stresses of soils and, more importantly, any soil that lies above the water table would not liquefy (although may still be able to be densified during an earthquake). Water table levels have been provided with some of the borelogs collected for the database, but not all of them.

Water table levels in Christchurch will vary throughout the year and from year to year depending on ground water balance. Therefore, any reported water table levels might not be typical. To account for this and to provide information for records without water table depth information, all of the borelogs and CPT records were assigned a water level by the same process for the purposes of liquefaction prediction.

To demonstrate the effect of ground water variations, two different ground water level scenarios were analysed as described below.

The first ground water level scenario used as the basis of assigning water levels was based on the map provided in The Geology of the Christchurch Urban Area - Brown and Weeber⁽⁵⁾, 1992. The map provided by Brown and Weeber shows the contours lines for the depth below ground surface of the water table. The contour lines generally run north south, with the Port Hills forming a boundary to the south. To the east of Linwood the ground water level is within 1m of the ground surface (due in part to the Avon-Heathcote Estuary and the Avon River). In the analyses it has been assumed that the area between the 1 and 2m contour has a ground water level of 1m, to provide a conservative approach (the same approach has also been used between the other contours, and the area east of Linwood has been assigned a water level equal to the ground level). This water level scenario is assumed to represent a high probable water level.

The second ground water table scenario used the average ground water levels calculated using actual data from Environment Canterbury's Wells Database⁽²⁾. The Well Database information was processed by looking at records from the last ten years and from wells with a maximum depth less than 30m (to focus on groundwater levels in the upper unconfined aquifer only). The dataset was also processed to include average data from January only (in order to provide a less conservative approach than the brown and Weeber scenario discussed above). The average (10 year) reduced level water table was then plotted for each well to check for ground water trends that may have be affected by pumping.

The map produced is in general similar to the Brown and Weeber map (with a shallow water table in the east of the city and deeper contours heading west), however the contours were more detailed. Typically the ground water levels are lower than those presented by Brown and Webber.

The water tables used for each of the boreholes and CPT's in the database were determined by interpolation between the contours, rather than the assigning integer water levels (as undertaken with the Brown and Weeber water table scenario).

LIQUEFACTION HAZARD MAP

Following the prediction procedure outlined above, a liquefaction hazard map was produced for Christchurch for both ground water level scenarios. A copy of the resulting map for the lower ground water scenario is shown in Figure 1. The compilation scale of the map was 1:25,000.

The potential for liquefaction is shown on the Liquefaction Hazard Map for Christchurch and this potential has been divided into eight classes:

1. **High Liquefaction Potential** (solid red areas): Areas in which earthquake peak ground accelerations of greater than 0.12g, but less than 0.2g, for a magnitude 8 Alpine fault earthquake, potentially cause some of the soils to liquefy (Scenario 1). Note also that some other soils at the same location may not liquefy or could have a lesser susceptibility to liquefaction.
2. **Moderate Liquefaction Potential** (solid orange areas): Areas in which the earthquake peak ground accelerations of greater than 0.20g, but less than 0.34g, for a magnitude 8 Alpine fault earthquake, potentially cause some of the soils to liquefy (Scenario 2). Note also that some other soils at the same location may not liquefy or could have a lesser susceptibility to liquefaction.
3. **Low Liquefaction Potential** (solid yellow areas): Areas in which only the highest earthquake peak ground acceleration of 0.34g, for a magnitude 8 Alpine fault earthquake, potentially cause some of the soils to liquefy (Scenario 3). Note also that some other soils at the same location may not liquefy or could have a lesser susceptibility to liquefaction.
4. **Liquefaction Not Predicted** (white areas within the city boundary): Areas in which an earthquake ground acceleration of 0.34g failed to predict liquefaction. However, it should be noted that due to the limited spatial extent of soil information in the database, these areas might contain liquefiable soils or could be subject to other forms of instability, for example, ground fissuring.
5. **Insufficient Information Available, but may have High Liquefaction Potential** (red hatched area): No information is available in the database to analyse the liquefaction potential. However, these regions may have areas where High Liquefaction Potential may occur as outlined in point 1. Subjective assessment based on the Brown and Weeber⁽⁵⁾ geological map.

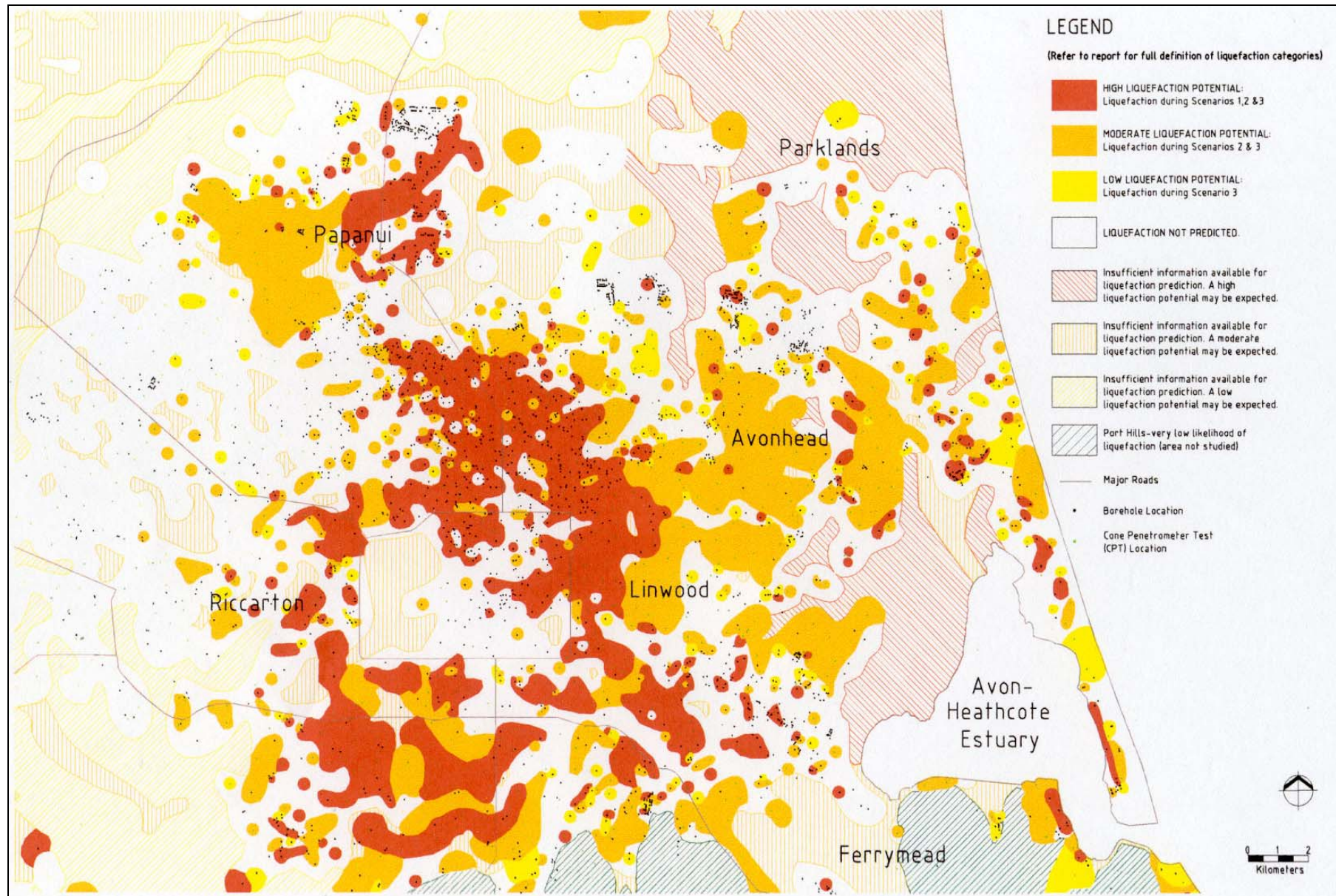


Figure 1: Liquefaction Hazard Map for Christchurch City based on the average January ground water levels over the last 10 years.

6. **Insufficient Information Available, but may have Moderate Liquefaction Potential** (orange hatched area): No information is available in the database to analyse the liquefaction potential. However, these regions may have areas where Moderate Liquefaction Potential may occur as outlined in 2 above. Subjective assessment based on the Brown & Weeber⁽⁵⁾ geological map.
7. **Insufficient Information Available, but may have Low Liquefaction Potential** (yellow hatched area): No information is available in the database to analyse the liquefaction potential. However, these regions may have areas where Low Liquefaction Potential may occur as outlined in 3 above. Subjective assessment based on the Brown & Weeber⁽⁵⁾ geological map.
8. **Port Hills – Very Low Likelihood of Liquefaction** (green hatched area): Areas not studied. However, it should be noted that these areas might contain small areas of liquefiable soils or could be subject to other forms of instability, for example, slope failure.

Notwithstanding the above determinations, different soil layers at a site may have different susceptibilities to liquefaction. The liquefaction algorithm assigns the liquefaction hazard for a site based on the weakest liquefiable soil. Hence the variability within a soil column is not presented on the Liquefaction Hazard Map.

Reporting of the liquefaction ground settlement, where the cumulative settlement within a soil column is calculated, provides an indication of the severity of liquefaction. (The settlement has been calculated for 200 of the data locations within the database (100 boreholes and 100 CPT's) and separate Liquefaction Ground Damage Maps for Christchurch for the two water table scenarios have been produced. These maps can be found in the Christchurch Liquefaction Study Report⁽⁴⁾.)

The Liquefaction Hazard Maps generated (one for each of the ground water scenarios) are similar in distribution of liquefiable deposits to the existing liquefaction hazard maps previously presented by CAE⁽⁶⁾ (1997) and Brown and Weeber⁽⁵⁾ (1992), in fact it predicts wider spread liquefaction on the eastern side of Christchurch. However, these new maps provide more definition to the extent and severity of the hazard.

In total, liquefaction susceptibility (hazard) was calculated at 2,194 boreholes and 885 CPT's in the study area. This is compiled from a database, which contains approximately 3,700 borehole sites that have been manually entered (with and without strength data), nearly 1,100 CPT locations (all with strength data), and around 4,700 records entered from Christchurch City Council (CCC) Microsoft Excel files (without soil strength data).

One of the main reasons for hazards not being calculated for every database entry was that there was no strength information to match to soil layers, hence calculations could not be undertaken (estimates could be made in the future, but this would be best undertaken on a manual basis). Coordinates for 1,305 sites could not be matched to the addresses supplied with the original data.

DIFFERENCES IN HAZARD MAPS FROM VARYING THE WATER TABLE

There is considerable difference between the liquefaction hazard maps produced using the two water table scenarios described above (Brown and Weeber⁽⁴⁾ water table levels, and the water levels obtained from Environment Canterbury's database⁽²⁾). The main difference is that there has been a general and widespread lowering of the hazard for the City when using the water well database due to the generally lower ground water levels. Many locations described as high hazard using the Brown and Weeber map have lesser hazards when using the database water levels.

Furthermore, many areas described as moderate or low hazard on the hazard map based on the Brown and Weeber water levels are not predicting a liquefaction hazard for the database water levels. Notwithstanding this, the pattern of liquefaction hazard being greatest on the eastern side of Christchurch has remained unchanged.

The lower water table resulted in a change of liquefaction susceptibility due to the:

- soils that may have been liquefiable in the zone through which the water table has been lowered are no longer liquefiable and therefore this will reduce the hazard (note however that these soils may still densify in a dry state), or
- lowering of the water table increases the overburden confinement pressures and therefore increases a soils resistance to liquefaction.

CONCLUSION

The Christchurch Liquefaction Study has utilised existing data from the Christchurch City Council, Canterbury Regional Council, and several private organisations to produce Liquefaction Hazard and Ground Damage maps for the City of Christchurch. The soil type, strength and water table data collected for each site has been entered into a specially prepared database and then analysed, using an embedded program (using the Youd and Idriss⁽¹⁾ method) within the database to provide a hazard coefficient for each site. These hazard coefficients have been used to form the basis of the Liquefaction Hazard maps.

The study has used the Alpine fault for its design earthquake. The magnitude of this earthquake has been varied to investigate the sensitivity of the various sites within the database to ground shaking.

As no laboratory tests are available for the majority of sites within the database, assumed soil densities of 1.8t/m^3 for saturated soils and 1.65t/m^3 for dry soils have been used for the analysis. For simplicity, these densities have been applied to the entire city. They are typical for many of the liquefiable sands within Christchurch.

The water table levels for Christchurch vary across the city and throughout the year. To investigate the city's sensitivity to water table variation, two water table scenarios were analysed. The first ground water level scenario used was based on the map published in The Geology of the Christchurch Urban Area - Brown and Weeber⁽⁵⁾, 1992. This approach is considered conservative.

The second water level scenario is based on actual water level records from bores monitored throughout the city by Environment Canterbury and the Christchurch City Council over the last 10 years. This provided a less conservative approach than scenario 1 above.

An example of the Liquefaction Hazard maps produced is shown in Figure 1. Eight different zones are defined, which show various hazard levels for the city.

There is a considerable difference between the liquefaction hazard maps produced using the two water table scenarios. The main difference is a general lowering of the liquefaction hazard due to the lowering of the water table (from scenario 1 to scenario 2).

ACKNOWLEDGEMENTS

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Special thanks also goes to Christchurch City Council, for their extensive assistance during the research stage of the project, providing unlimited access to their records, and geocoding assistance to determine borehole locations, and the various other organisations that provided data during the project.

Finally, the author wishes to thank Steven Christensen for his guidance and support throughout the project and the preparation of this paper.

REFERENCES

1. Youd, T. L. and I. M. Idriss (2001) "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", Jr. Geotech. & Geoenviron. Eng., April 2001, pp 297-313.
2. Environment Canterbury Well Database, February 2001.
3. NZS 4203:1992, "Code of Practice for General Structural Design and Design Loadings for Buildings".
4. Beca Carter Hollings & Ferner Ltd (2002), "Christchurch Liquefaction Study – Stage IV", prepared for Environment Canterbury, ECan Report # U02/80, Beca Job # 3812464.
5. Brown LJ and JH Weeber (1992), "Geology of the Christchurch Urban Area", IGNS, Scale 1:25,000.
6. Centre for Advanced Engineering CAE (1997), "Risk & Realities A Multi-disciplinary Approach to the Vulnerability of Lifelines to Natural Hazards based on the Work of the Christchurch Engineering Lifelines Group", November 1997, University of Canterbury, Christchurch.

Geotechnical Design of the Rapahoe Bridge No. 1 North Abutment

Richard Cole, Tonkin & Taylor Ltd

Tranz Rail Ltd proposes to build a replacement rail bridge over the Grey River at Greymouth on the West Coast of New Zealand. The north end of the proposed bridge lands at a skew to the existing cut rail line bench. Consequently, a pile is required to support the north abutment and retaining walls are required, perpendicular to the railway alignment at the abutment and along the outer edge of the railway formation north of the abutment, to extend the railway bench on fill. The preferred option for the retaining walls consists of precast panels founded on a reinforced concrete beam cast against and anchored to the rock face. The post tensioning of the anchors and friction between the uneven rock face and the concrete beam provides vertical support.

This paper presents details of the geotechnical design of the north abutment including the investigations undertaken, the ground conditions encountered, selection of the abutment location, options considered, selection of design parameters, load cases considered and detailed design. The design included finite element analysis of the retaining wall.

INTRODUCTION

Tranz Rail Ltd proposes to build a replacement rail bridge over the Grey River at Greymouth (Figure 1). This will provide a reliable, high quality link between the Rapahoe Branch Line and the Midland Line. The Rapahoe Branch Line is principally used to collect coal from the coal mines in the Rapahoe area. The Midland line takes the coal across the South Island to the deep water port at Lyttleton (Christchurch). The existing rail bridge was built in 1889 and has, in recent years, had extremely high and increasing maintenance requirements, as well as severe operating restrictions due to its deteriorating condition.

The stage is quickly being reached when it will no longer be viable to maintain the existing bridge in a state to enable its safe continuing use. Tranz Rail Ltd has therefore undertaken geotechnical investigations and design relating to construction of a new bridge, with a target date for commissioning the replacement bridge of mid-2005.

PROPOSED REPLACEMENT BRIDGE

The proposed new bridge is approximately 285 metres long. On the northern side of the river, the new bridge would leave the alignment of the existing line approximately 50 metres upstream of the existing bridge. The new line would then swing in an arc over the river in a south-easterly direction to rejoin the Midland Line approximately 150 metres upstream of the existing bridge on its southern side.

The bridge is likely to consist of 1.8m diameter reinforced concrete, circular single stem piers/piles, supporting a reinforced concrete hammerhead cap beam. The piles are likely to be founded on rock sockets. The superstructure is to comprise a reinforced and post-tensioned concrete U-beam spanning, simply supported between the hammerhead cap beams.

The north end of the bridge consists of a link span between the Number 11 pier and north abutment. The link span lands on a skew to the existing cut rail line bench. The proposed new alignment of the rail line requires the existing cut bench to be widened over a length of approximately 20m.



FIGURE 1 - LOCATION PLAN



FIGURE 2 - PROPOSED ALIGNMENT

SITE INVESTIGATIONS

The site investigations for the northern abutment design consisted of:

- Geological mapping of the rock face along the existing rail line cut bench at the proposed northern abutment location;
- Inspection by divers of the river bed below the proposed north abutment location; and
- Drilling of two boreholes from access platforms constructed below the existing rail line cut bench.

GROUND CONDITIONS

The observed ground conditions are summarised in Section on Figure 3. Photograph 1 show the rock face at the north abutment location. The riverbank and existing railway bench comprise exposed limestone rock with local areas of fill over-spill. Railway ballast blankets the railway bench.

The rock batters above and below the existing railway platform consist of light grey unweathered sand textured limestone. It is moderately strong to strong with widely spaced persistent joints. It is massively bedded with some thinner silty marl beds dipping into the face at 20°. The sandy limestone is more resistant to weathering with bed thickness ranging from 0.5 to 2.0m, while the marl beds are typically 0.5m thick. There are no persistent rock defects along the bedding layers.

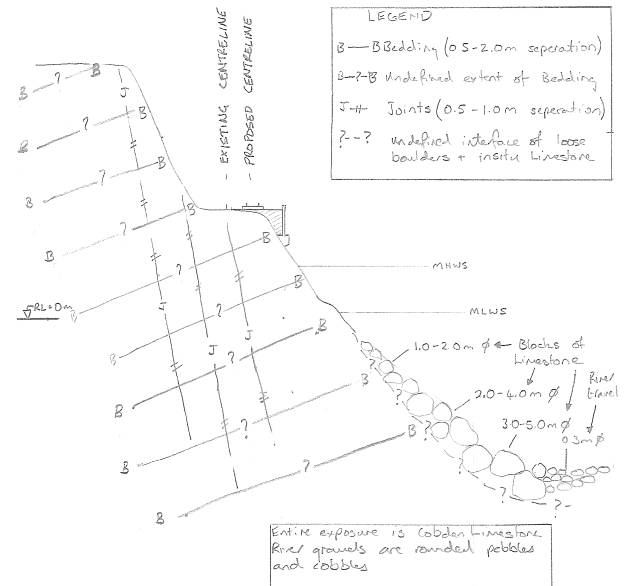


FIGURE 3 - GEOLOGICAL CROSS SECTION

Two major joint sets are present. The strike and dip of these joint sets are approximately 020/70°E and 290/80°N. The joints are very persistent but generally minor with limited soil and root infill. Near the water line the joints are more open and eroded into cavities in places. Joint spacings were closer and rock mass less competent southwest of the proposed abutment compared to the rock mass at the proposed abutment.

Cobden Cave is located 60m south-west of the proposed abutment. There is no evidence of similar extensive solution cavities in the direct vicinity of the proposed abutment. Solution erosion of joint surfaces in this area has resulted in joints being open (typically 50mm wide) extending 0.5m to 1.0m into the rock face. Solution erosion of a joint 5m southwest of the proposed abutment has resulted in a cavity of greater than 0.5m in width and extending 2m in from the general face of the rock surface.



PHOTOGRAPH 1 - NORTH ABUTMENT LOCATION

A joint controlled wedge failure of the rock face is evident near the proposed north abutment location. Adjacent to this the rock has dilated and relaxed and the rock batter is marginally stable.

Divers reported that below water level, they observed a batter slope at 35° to 45° extending to the riverbed at approximately 10m below mean sea level. This batter slope comprised boulders of rock ranging from 1m to 5m in size. The riverbed was observed at the toe of the slope with bed material comprising 300mm size rock cobbles. No rock exposure was reported. It is inferred that the boulders are overspill from the excavation of the road and railway benches and rock fall from the slope.

DESIGN LOADING

Load combinations considered as part of the ultimate limit state design included:

$$1.8EP + 1.8LL$$

$$1.35EP + 1.0EQ$$

where: EP=Earth Pressure
LL=Live Load
EQ=Earthquake Effects

The super imposed dead load for railway tracks, sleepers and ballast was 8.1kPa. This is equivalent to 450mm depth of ballast at 18kN/m^3 , which allows for overtopping of ballast.

The Tranz Rail Design Brief alternative axle case gave the highest live load of 230kN on axles spaced at 4.6m centres.

The seismic coefficient of 0.42g used in design was calculated in accordance with NZS 4203:1992 and the Transit New Zealand Bridge Manual.

DESIGN OPTIONS

The preliminary design was developed in conjunction with the Structural Engineer for the bridge superstructure. Assessment of the original location for the proposed abutment revealed that a retaining wall of greater than 12m height and extending below water level and through the rock boulder riverbank slope would be required. This was considered impractical. Also, at the location of the proposed abutment the geological mapping had identified that the rock slope was marginally stable.

Consequently, the abutment location was relocated north reducing the height of retaining wall required and avoiding the less competent rock identified. Pier 11 remained at its originally proposed location and the length of the link span between Piers 11 and north abutment was increased from 10m to 16m to accommodate the new abutment location. The abutment location selected required a maximum retained height of

5.0m. This retained height reduces to 1.5m over a distance of 10m to the north.

It is proposed that a ground beam cast onto the existing rail line cut bench supports the north end of the link span. A single pile will support the downslope end of the ground beam. The ground beam will resist lateral loads from the bridge superstructure through friction. Vertical post tensioned ground anchors are proposed to increase the frictional resistance between the ground beam and the underlying rock.

The proposed new alignment of the rail line requires the existing cut bench to be widened by nearly 2m at the proposed abutment location and tapering to no widening over a length of approximately 20m. To support the widening two retaining walls are proposed; one perpendicular to the rail tracks at the abutment location, and a second on the outer edge of the railway formation north of the abutment.

The following options for the construction of the abutment walls were been considered:

- Cast insitu concrete
- Sprayed concrete
- Precast concrete

The precast concrete option on a cast in-situ concrete ground beam was selected. The retaining wall is to be founded on a reinforced concrete beam cast against and anchored to the rock face. The post tensioning of the anchors and friction between the uneven rock face and the concrete beam provides vertical support. The retaining wall will comprise precast concrete slabs keyed into the in-situ concrete beam and anchored back. Granular backfill and subsoil drainage will be placed behind the wall.

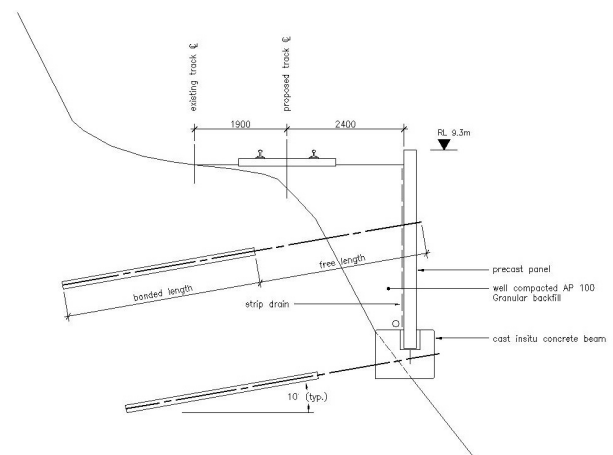


FIGURE 4 - PROPOSED WALL CROSS SECTION

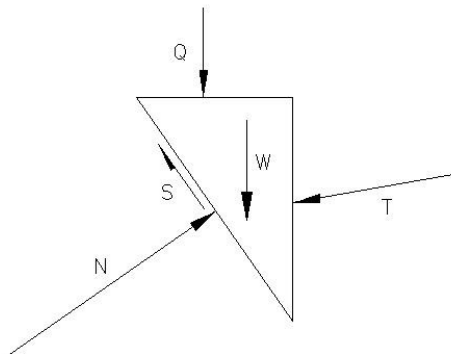
Ultimate grout to country bond (τ_u) used for the design of the anchors was 800kPa. A strength reduction factor of 0.5 was applied for ultimate limit state design and anchor testing is to be undertaken during construction. An

anchor free length of 2m beyond the rock surface has been provided to ensure that the bond length is beyond the open joints in the rock. The possibility of solution cavities within the rock cannot be discounted. Anchors would need to be extended through cavities if they are encountered. The anchors have been detailed with double corrosion protection consistent with BS 8081:1989.

WALL DESIGN

The wall was designed using limit state design principles. Limit state design requires that the design resistance effects (R^*) be greater than the design action effects (S^*)[1]. For this design the trial wedge method was used to assess the stability of the wall. The design actions were determined by assessing the forces acting on each wedge of soil. The weight of soil and live load were increased by the appropriate load factors. The design action is the resultant shear force on the base of the soil wedge (S).

FREE BODY DIAGRAM



FORCE DIAGRAM

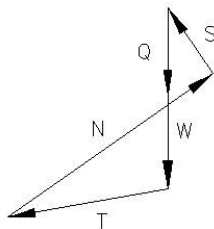


FIGURE 4 – Trail Wedge Analysis

The design resistance was assessed as the available friction at the base of the wedge (R) multiplied by the strength reduction factor (Φ). This was assessed to be:

$$\Phi R = \Phi N \tan f'$$

where

N = normal force on the base of the soil wedge

f' = internal friction angle

The minimum tension (T) required in the anchors was determined so that the R was greater than S . The minimum tension required was found to be 310kN/m. Figure 4 below summarises this analysis.

In the analysis $\phi' = 35^\circ$ was assumed being the angle of friction between the backfill and rock. The actual resistance along this base will be significantly greater this value as at the base of the wall the friction is between cast in situ concrete and rock.

The prestress force for the upper anchor was determined by calculating the “at rest” earth pressures on the precast panel and was found to be 80kN/m. The remainder of the 310kN/m anchor load required for stability of the wall will be applied to the lower anchor.

The proposed retaining wall also analysed using the finite element software PLAXIS Version 7.2. PLAXIS is a finite element code developed specifically for the analysis of soils and rocks. The PLAXIS analysis considered the following:

1. Initial Conditions
2. Excavation and construction of foundation beam, placement of wall panels, backfilling and prestressing of anchors
3. Live Load Case
4. Seismic Load Case

The PLAXIS analysis found a factor of safety greater than 2.4 for the live load case and 1.25 for the seismic load case. From the PLAXIS model the structural design actions for the precast panels were calculated for the two design load cases.

ROCK FACE STABILITY

Assessment of the major rock joints and bedded using stereographic projection indicated that stability of the rock face at the location of the proposed abutment should not be a significant issue as:

- The bedding dips into the rock face.
- The major joint sets are steeper than the rock face so that wedge failures are generally not kinematically possible.
- The rock anchors used to support the wall will also stabilise any potential wedge failures.

A joint controlled wedge failure of the rock face is evident on the slope above the proposed pier 11. The observed joint orientation at this location was different than that observed at the location of the north abutment. Assessment of these joints orientations indicated that a wedge failure was possible. Adjacent to this the observed failure the rock has dilated and relaxed and the rock batter is marginally stable. This loose rock will be scaled from the slope during construction. Detailed mapping of the excavated rock face is proposed for during construction. If the mapping identifies additional potential wedge failures then rock bolting will be undertake.

CONCLUSIONS

Tranz Rail Ltd proposes to build a replacement rail bridge over the Grey River at Greymouth on the West Coast of New Zealand. Tranz Rail Ltd has undertaken geotechnical investigations and design relating to construction of a new bridge, with a target date for commissioning the replacement bridge of mid-2005. The north end of the proposed bridge lands at a skew to the existing cut rail line bench. Consequently, a pile is required to support the north abutment and retaining walls are required, perpendicular to the railway alignment at the abutment and along the outer edge of the railway formation north of the abutment, to extend the railway bench on fill.

The site investigations for the northern abutment design consisted of geological mapping of the rock face, inspection by divers of the river bed and drilling of two boreholes. The rock batters above and below the existing railway platform consist of light grey, moderately strong to strong, unweathered sand textured limestone with widely spaced persistent joints.

The preferred option for the retaining walls consists of precast panels founded on a reinforced concrete beam cast against and anchored to the rock face. The post tensioning of the anchors and friction between the uneven rock face and the concrete beam provides vertical support.

The wall was designed using limit state design principles. An initial analysis was undertaken using the trial wedge method to assess the stability of the wall and to find anchor prestress loads. Finite element software was used to check the stability of the wall and calculate the design actions for the precast panels.

Assessment of the defects at the location of the proposed retaining wall indicates that stability of the rock face should not be a significant issue. A joint controlled wedge failure of the rock face is evident on the slope, south of the proposed abutment, above the proposed pier 11. Adjacent to this the rock has dilated and relaxed and the rock batter is marginally stable. Scaling and rock bolting will be undertaken to improve the stability of the rock face at this location.

ACKNOWLEDGEMENTS

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REFERENCES

1. Australian Standard AS4678-2002, "Earth-retaining Structures", February 2002.

Fault Tree Analysis and Risk Assessment of Unbound Granular Pavement Material Performance

Meera Creagh, Geotechnical Branch, Queensland Department of Main Roads (QDMR)

ABSTRACT

This paper describes the methodology issues considered in using Fault Tree Analysis to develop a risk assessment tool for unbound granular pavement material performance in Queensland. Example results from the tool are also presented. Granular pavements form a major geotechnical asset for Queensland Department of Main Roads (QDMR), and material underperformance is a significant concern. The uncertainty and economic consequences surrounding underperformance of granular pavements make them a prime candidate for structured risk assessment. The model was built using GoldSim, a simulation program that allows modelling of complex systems and quantitative probabilistic simulation. GoldSim is used to model a Fault Tree describing pavement performance factors and possible failure modes. The model uses semi-quantitative input ratings to describe the project scenario and the consequences for each failure mode, and outputs semi quantitative risk rankings for each failure mode. The model provides a relative measure of risk and is useful for comparing project scenarios and identifying high-risk areas for specific scenarios. The paper presents the model outputs for a particular scenario and compares them against actual performance.

1.0 INTRODUCTION

1.1 Risk in Main Roads

Queensland Main Roads are responsible for the construction, maintenance and administration of Queensland's most valuable physical asset: the state road network. It has a replacement value of about \$23 billion. The majority of roads administered by Main Roads are unbound granular pavements, forming a major geotechnical asset for Main Roads.

Pavement materials have a significant effect on pavement performance. Material problems generally have a disproportionate effect on the cost and performance of a project. Material failures are often significant in pavement underperformance [Wijeyakulasuriya, Ramanujam et al 2003(1), QDMR 2003 (2)].

The interaction of material source parameters, moisture sensitivity, and production and construction effects make material performance very complex. The combination of poor understanding of material properties, non-standard materials and water create a risky situation [Atkinson 1999 (3)].

Other issues that have contributed towards a poor appreciation of material performance risks include:

- Pavement loadings and design responsibilities are increasing, while budgets are shrinking [QDMR 2003 (2), Richmond 2003 (4), Skinner 2002 (5)].

- Design resources do not characterise pavement materials performance risks well [Wijeyakulasuriya, Ramanujam et al 2003(1), QDMR 2003 (2)].
- Current Main Roads risk assessment tools are designed to analyse diverse risks associated with project management, not specific performance risks.

These issues increase the risk of pavement underperformance due to material issues. This creates a need for some method suitable for managing and assessing these risks.

1.2 Pavement Performance

Poor pavement material performance manifests through different failure modes. Failure modes fall into two different groups: structural integrity, and functional performance. The distinction between failure types is relevant when considering the consequences of different failures.

Functional failures have cost, comfort and safety consequences. Structural integrity is the ability of the pavement to withstand loads. Table 1 shows the failure modes considered in the pavement material performance model.

Table 1: Pavement Failure Modes

Structural Failure	Functional Failure
Distortion (rutting, shoving, heave)	Skid Resistance
Cracking (crocodile, age, longitudinal)	
Disintegration (potholes, edge defects)	

Poor performance of pavements can generate safety issues, costs to the travelling public and costs to the organisation. Reeves [2001 (11)] estimates that poor performance can be linked to at least \$25 million of Main Roads annual expenditure, due to premature failures, additional costs during construction and increased maintenance and rehabilitation costs.

Hunt's [2002 (10)] study of pavement performance shows that underperformance has risen within the last ten years. Based on pavement roughness (a measure of pavement distress), it showed that pavements built in the last twenty to thirty years are meeting their design lives more consistently than their younger counterparts (Figure 1). This trend is noted by Atkinson [1999 (3)].

2.1 Risk Identification

Risk identification needs to specify the most significant risks. This step takes apart the complex system that represents pavement material influence on pavement performance. The system is divided into failure modes, and the base causes of these failures.

2.1.1 Failure Modes

Poor performance of the pavement material has several different manifestations:

- Distortion
- Disintegration
- Cracking
- Punching (seal aggregate penetrating base)

(The distinction between structural and functional failures is only relevant for assigning consequences).

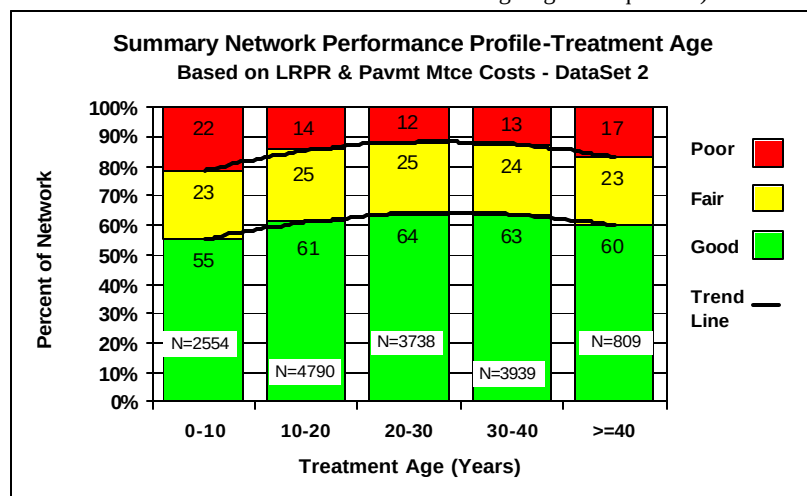


Figure 1: Pavement Performance [Hunt 2002 (10)]

2.0 RISK DEFINITIONS

Risk is the *chance* of something happening that will have an *impact* on objectives. It is measured in terms of *likelihood* and *consequence* [AS/NZS 4360 1999 (9)].

Risk Management is a systematic method of identifying, analysing, evaluating and controlling risks [AS/NZS 4360 1999 (9)]. It allows a more structured use of engineering judgement and experience in making decisions [Tsolakis, Thoresen et al 2002 (8)].

Risk Assessment is the overall process of risk analysis and risk evaluation. After identifying the important risks, a risk assessment tool helps analyse these risks to provide information to the risk evaluation.

2.1.2 Base Causes

The occurrence of these failure modes is controlled by a number of variables. These base causes have been grouped under the following headings in Table 2:

Table 2: Material Performance Factors

Performance Factor	Examples
Material Properties	Strength, reactivity, grading
Environmental Loads	Rainfall, temperature
Traffic Loads	Volume, % of heavy vehicles, growth
Cross Section	Drainage, layer thickness
Construction	Compaction, moisture
Maintenance	Reseal frequency, routine maintenance

2.2 Risk Analysis

Risk analysis aims to discriminate minor acceptable risks from major risks. The severity of a risk is determined by the relationship between likelihood and consequence [AS/NZS 4360 1999 (9)]. The risk analysis methodology controls both the way likelihoods and consequences are estimated, and how they are combined to estimate risk. Both of these must suit the risk problem and the user.

In this case, pavement material performance is a complex failure mode with many variables, so the analysis must be detailed enough to model this. The users will be Main Roads staff, who do not have the time, expertise or data to use sophisticated statistical models, so the risk assessment tool must also suit their resources.

2.2.1 Complex Failure Modes

The failure modes associated with poor pavement performance are complex, involving many different factors and interrelating subsystems. An example of this is the interaction between material properties and water. This is critical to the performance of the pavement, but both of these elements are dependent on many other factors, such as construction, climate, material production and material source properties.

The complex failure modes that pavements display have many component events occurring in sequence. Evaluating complex failure modes requires breaking these sequences down. The risk analysis method must be able to account for the interaction of these variables.

Current risk assessment tools in Main Roads are based on qualitative risk matrices. This is an inductive method, and is concerned with what types of failures are possible – *what may arise from a particular initiating event*. These methods are good for broad-brush risk assessments and overviews.

But simple overview methods lump together all the variables that affect an event's likelihood. This makes it difficult to judge the likelihood of events with many factors affecting their occurrence [Vick 2002 (12)].

More detailed examples of these methods, like Event Trees, are also difficult to apply to complex systems. This is because the different possibilities arising from subsystem failures (both singly and in combination) become too great [Vesely et al 1981 (13)].

These inductive methods produce many divergent paths, especially if many different factors are involved. For example, there are many different possible outcomes for a particular performance factor (causative agent) depending on how it combines with other performance factors. This makes a large, unwieldy event tree.

2.2.2 Analysis Method

The method of analysis defines how likelihood and consequence combine to form an estimate of risk, forming the framework for the analysis. Deductive methods focus on *how a specific failure can happen (eg rutting)*. This allows the dissection of the failure process to examine the interaction of these factors, especially where event trees or other inductive methods would be unmanageably large [Vick 2002 (12), Vesely et al 1981 (13)].

Fault Tree Analysis has been selected as the most appropriate way of analysing a complex event such as pavement failure. Fault Trees identify the key failures, in this case pavement material failure. These key failures are broken down into chains of sub-failures until the base causes are reached (the performance factors).

The Fault Tree forms a logic map of the system components and the interaction of different variables to generate the major failure of interest. This forms the analysis framework for the calculation of likelihoods.

Each subsystem failure is linked by Boolean logic – AND (if all base causes apply) and OR gates (if any base cause applies). Fault trees are quantified using Equations (1) and (2):

$$\text{AND: } P_{\text{AND}} = P_1 \times P_{i+1} \times \dots \times P_N \quad (1)$$

$$\text{OR: } P_{\text{OR}} = 1 - (1 - P_1) \times (1 - P_{i+1}) \times \dots \times (1 - P_N) \quad (2)$$

These equations assume independence between the base causes (performance factors). The performance factors have been dissected to a reasonably detailed level to avoid too much dependence between base causes.

The user attaches probability estimates (P_i) to the base performance factors, as these likelihoods are easier to judge than for complex failure modes. The fault tree calculates these likelihoods up the tree using Equations (1) and (2), to arrive at the overall probability of failure. The failure mode probabilities are then multiplied by the consequence of failure to get an estimate of risk. Figure 2 shows a major subsystem of the pavement material fault tree.

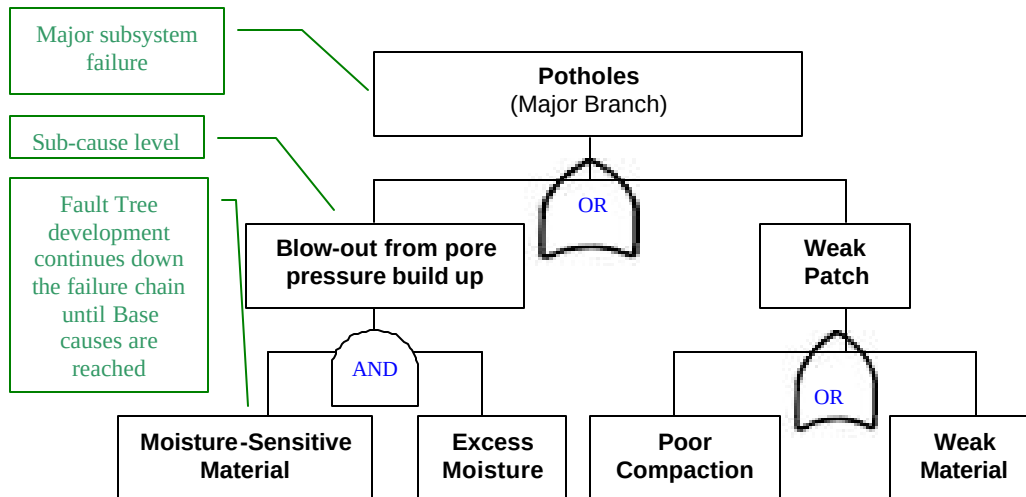


Figure 2: Example Segment of Pavement Material Performance Fault Tree

2.2.3 Expressing Probability and Consequences

This concerns the way estimates of likelihood and consequence are expressed. *Qualitative* descriptive methods are easy to use, but are difficult to apply in any detailed analysis of the failure process. *Quantitative* methods use numerical values to assess likelihood and consequence. However, it can be a difficult, data-hungry and expensive exercise to derive accurate numbers [Williams 1998 (14)].

The *semi-quantitative* approach allocates numbers to qualitative word rankings describing likelihood and consequence. These are a quantified expression of engineering judgment [Vick 2002 (12)].

The numerical conversions are based on a five point logarithmic scale, as per Williams' risk assessment model [Williams 2001 (15)]. These are shown in Table 3 below.

This avoids misinterpretation of numbers as probabilities, while also clearly discriminating between likelihood levels.

The semi-quantitative approach to risk analysis combines the flexibility, feasibility and ease of qualitative methods with the rigour and analytical advantages of quantitative methods.

The semi-quantitative inputs suit the analytical needs of the risk problem. It allows for numeric combination of many event likelihoods for analysis of the interactions between complex events and subsystems.

Semi-quantitative inputs also suit the tool's users. Main Roads staff do not have the time, expertise or data to produce statistical models. The semi-quantitative method suits the wealth of practical experience that characterises Main Roads staff. It allows staff to use practical experience and engineering judgement in a more structured way.

Table 3: Likelihood and Consequence Ratings

Likelihood		Log Scale
Rating	Description	
1. Rare	May occur	0.01
2. Unlikely	Could occur	0.033
3. Moderate	Moderate occurrence likelihood	0.1
4. Likely	Will <i>probably</i> occur	0.33
5. Almost Certain	<i>Expected</i> to occur	1

Consequence		Log Scale
Rating	Description	
1. Negligible	Dealt with through routine maintenance, negligible expenditure	0.01
2. Low	Threatens road's effectiveness, minor maintenance expenditure	0.033
3. Medium	Requires significant premature expenditure	0.1
4. Severe	Threatens road performance, needs major premature maintenance	0.33
5. Extreme	Permanently harms road and requires premature rehabilitation	1

2.2.4 Modelling

The Fault Tree is modelled in GoldSim, a software package provided by Golder Associates Pty Ltd. Likelihood and consequence judgements are transferred from an Excel spreadsheet to the GoldSim model. GoldSim calculates the risk rankings for the scenario.

3.0 RESULTS

3.1 Example Outputs

The Fault Tree model aims to assist in ranking the relative severity of risks for each failure mode for unbound granular pavement materials. This can be used to compare the severity of different failure mode for a specific project scenario. This can isolate the major risks for that particular scenario, and identify where funds can best be directed to reduce the overall risk. The result format for each individual scenario is shown in Figure 3.

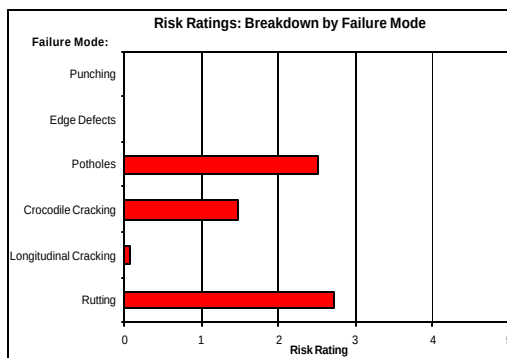


Figure 3: Risk Ranking

The risk for each failure mode can then be combined to give a global estimate of risk for the scenario. This is the cumulative risk for the scenarios, with the bar length reflecting the combined risk from each failure mode. Figure 4 compares scenarios where each performance factor is set to its extreme in turn. (All consequences were set to their extreme).

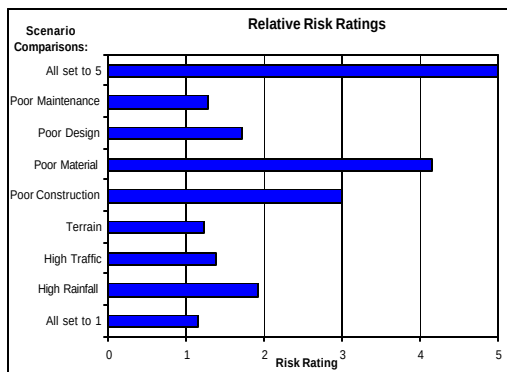


Figure 4: Risk Ranking Comparing Scenarios

3.2 Model Validation

The model is currently undergoing sensitivity testing, field testing and feedback from potential users. Part of the validation process has been comparing predicted to actual performance for different project scenarios.

A significant bypass project is used as an example here. The project featured a thick pavement with good quality material:

- strong (10% fines wet value = 340 kN)
- durable (degradation factor of 70)
- well graded and
- was crushed material with high densities

But the material had little or no plasticity, and the site suffered rainfall and the nuclear meters used for compaction control were poorly calibrated. Figure 5 shows that according to the model, this scenario has a high rutting risk and a medium risk rating for potholes.

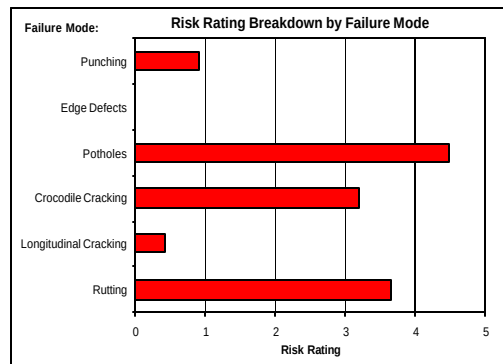


Figure 5: Risk Ranking for Test Scenario

The actual project suffered rutting and potholes before opening. Figure 6 shows Repeated Load Triaxial results for the material. High plastic strains are evident at the specified DoS limits, showing that this material was susceptible to rutting at these moistures.

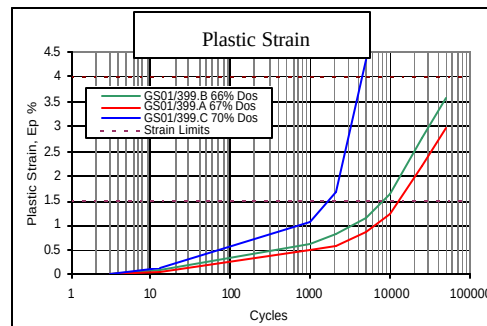


Figure 6: Plastic Strain for Test Scenario

These initial validation results indicate that the model gives a reasonable indication of high risk areas for pavement material related failures.

3.3 Model Limitations

Fault trees, like all models, are subject to the inaccuracies of modelling. These are generated by the assumptions and inaccuracies of representing a real, complex system in some logical virtual construct. But the sensitivity to model error can be minimised by using the model results for comparison studies, rather than using the absolute output to estimate risk.

The fault tree does not represent the entire system, only the area of interest to the analyst. The risk analysis is only valid for the failure modes that have been included in the fault tree, in this case, those associated with unbound granular pavement materials.

Any decision maker should beware of results that indicate very, very small probabilities of failure. This means that the system is unlikely to fail in the ways considered, but is far *more* likely to fail in ways that have not been considered.

4.0 CONCLUSIONS

The model shows that Fault Tree Analysis is a useful tool for risk analysis. It allows detailed examination of complex failures with many variables, such as unbound granular pavement material performance.

Semi-qualitative inputs are a flexible, powerful way of using engineering judgement to characterise likelihood and consequence.

The model provides individual risk rankings for different failure modes for a particular scenario. It can also be used to compare scenarios in terms of a global risk ranking. The model shows reasonable results so far, although model validation is still in progress.

The risk assessment tool will help designers consider the performance risks associated with pavement material. It is particularly appropriate for unbound granular materials as it allows the examination of failure modes that are not well covered in standard methods.

5.0 ACKNOWLEDGMENTS

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6.0 REFERENCES

1. WIJEYAKULASURIYA, C.V., Ramanujan, J., et al, "Characterisation of some crushed rock base materials from Queensland". in *9th ANZ Conference*. 2004.
2. QDMR, "Inspection of Pavement Practices: Victoria and New South Wales". 2003, QDMR, Brisbane.
3. ATKINSON, D., "NQ Asset Management: An Update". 1999, QDMR: Townsville.
4. RICHMOND, A. "Fitness for Purpose in Engineering Design". *Planning and Design Symposium*. 2003. Brisbane: QDMR.
5. SKINNER, D. "Project Cost Estimating: the past, the present and the future". in *2002 South East Queensland Region Symposium*. 2002. Caloundra: QDMR.
6. ANDREWS, C., "An Ageing Workforce - Does it affect DMR?" 2002, QDMR: Brisbane.
7. OLIVER, J., "The performance of sprayed seals". 1999, ARRB Transport Research Board.
8. TSOLAKIS, D., THORESEN, T., and MICHEL, N., "Economic Evaluation of Road Investment Proposals: Risk Analysis". 2002, Austroads: Sydney.
9. AS/NZS 4360, "Risk Management". 1999, Standards Australia.
10. HUNT, P.D., "Analysis of Roughness Deterioration of Bitumen Sealed Unbound Granular Pavements for use in Road Asset Management Modeling", in *Civil Engineering*. 2002, QUT: Brisbane.
11. REEVES, I. "Risk Management - Materials". in *Road System and Engineering Technology Forum*. 2001. Brisbane: QDMR.
12. VICK, S.G., *Degrees of Belief: Subjective Probability and Engineering Judgement*. 2002: American Society of Civil Engineers Press.
13. VESELY, W., et al., *Fault Tree Handbook*. Systems and Reliability Research. 1981, Washington: U.S. Nuclear Regulatory Commission.
14. WILLIAMS, D. "Risk-Based Methods to Facilitate Management and Design of Tailings Storage Facilities". in *Geotechnical and Environmental Risk Analysis*. 1998. Brisbane: Institution of Engineers Australia.
15. WILLIAMS, D.J., "Risk management strategies for the surrender of open cut coal mine spoil areas in the Bowen Basin coalfields, Final Report ACARP Project C8039". 2001, University of Queensland: Brisbane.

Geotechnical Design of Retention Systems for the Chatswood to Epping Rail Link, Chatswood, NSW

Craig Curnow, MIEAust. Golder Associates

A portion of the Parramatta to Epping Rail Link involved the construction of about 500m of cut and fill retaining structures, allowing two additional rail tracks to be assembled within the existing narrow rail corridor. The cut walls were cantilever pile retaining walls up to 7m high near residential buildings. Geotechnical design including the derivation of realistic soil and rock parameters, the use of finite element analysis techniques, and comparisons of predicted versus actual displacements. Construction of the embankment walls required temporary and permanent stabilisation measures to interface with reinforced earth walls for the retention of steep operating rail embankments, including innovative designs, and monitoring of the structures and embankment.

THE PROJECT

The Parramatta to Epping Rail Link involves the construction of 13.5 km of new rail line in an urban environment. The total project value is approximately \$850 million. The project principal is the Transport Infrastructure Development Corporation. The rail link is almost entirely underground (hard-rock tunnel), with the exception of the southern 1 km at Chatswood, which is above ground and utilises the existing rail corridor. Selected geotechnical design issues associated with preparatory works at this south end of the project form the basis of this paper.

The preparatory works were part of a design and construct (D&C) Contract to widened the useable area within the rail corridor for additional rail infrastructure. Transfield (now part of the John Holland Group) won the Design and Construct contract. Golder Associates was geotechnical consultant for the D&C team at tender and detailed design stages.

The bulk of the works included the construction of about 320m of cantilever pile walls up to 7m high to widen and retain an existing cutting (refer Figure 1), and about 180m of retaining wall to support and widen an operating rail embankment (Refer Figure 2). Three bridge overpasses (two road and one rail) and a short length of soil nail wall were also constructed, but are not discussed further in this paper.



Figure 1 – Cut Retaining Wall



Figure 2 – Embankment Retaining Wall

GEOLOGY & PHYSICAL SETTING

The site is underlain by Ashfield Shale, consisting of black to dark grey shale and laminite (thinly interbedded shale and fine grained sandstone). The Australian Geomechanics Society classification⁽¹⁾ for shale within the Sydney region was used to describe the rock. Briefly, this classification denotes Class V Shale as extremely low strength. Class IV, III, and II Shale represent a gradually improving quality of rock up to Class I Shale, which is medium to high strength with a defect spacing greater than about 600mm.

In the cut area the surficial residual clay is generally very stiff and about 1.5m to 2m thick, deepening to about 5m near the north of the cut section. Within the depth of interest the underlying shale grades from Class V to Class II. The thickness of Class V Shale varies from about 1m to 4.5m, depending on location, as does the thickness of the underlying Class IV and III Shale, which varies from 5m to 13m. Class II Shale is penetrated at depth. The depth to groundwater below the final excavation level in the cut area was inferred to vary from about 0m to 3.5m.

The northern end of the Contract area is situated in a broad local drainage depression, and is the location of the existing rail embankment. Some firm to stiff colluvium (0.5m to 1.5m thick) is present in the lowest elevations, which overlies stiff and very stiff residual soils derived from the Ashfield Shale. Class V Shale is present at depths of about 2m to 4m below the top of the natural soil layers. Groundwater was measured in boreholes at depths varying from 0.5m to 3.6m below natural ground surface.

In this northern end of the site the existing rail line is positioned on a fill embankment up to about 6m high. The embankment was constructed during the early 1900's from clay and shale fill, inferred to have been won from the rail cuts to the north and south.

REFERENCE DESIGN AND DISPLACEMENT CRITERIA

The Reference Design for the project detailed a cantilevered contiguous bored pile wall in the cut area. The wall is located on the property boundary, and ground anchors or other restraints were not permitted to extend outside the property boundaries. The adjacent property was mostly public roads, with the exception of one two-story residence near the south end of the project, and a brick commercial building, also at the south end of the project. During the tender stage it was identified that design efficiencies would likely be available with careful selection of geotechnical design parameters, and use of finite element methods in design of the pile retaining wall.

The reference design for the rail embankment widening called for a gravity retaining wall. Access for construction was restricted by the existing (operating) 1.75H to 1V rail embankment, a line of significant deciduous trees along the property boundary, and a deep swale drain at the toe of the embankment (Refer Figure 2). The tender design of a piled counterfort wall was modified during detailed design to a reinforced earth wall, as construction limitations and economies were recognised. This is discussed in more detail later in this paper.

Primary displacement criteria in the project Deed were 5mm settlement and 1/500 angular distortion for sensitive structures (i.e. homes and masonry structures), and 30mm settlement and 1/250 angular distortion for roads.

GEOTECHNICAL DESIGN PARAMETERS

Selected geotechnical parameters adopted for design of the cantilevered bored pile wall are presented in the following table. The elastic modulus values adopted are somewhat higher than those normally used for foundation design. However, it is important to make the distinction between the loading modulus used for foundation design and the unloading modulus that is considered more

appropriate for excavations. The unloading modulus is higher, as is evident from the results of field pressuremeter tests. The use of higher unloading moduli was supported by personal communications with Dr Chris Haberfield, who has been involved with recent developments in the understanding of the behaviour of fractured rock, which tends to be stiffer in unloading than previously thought. This is inferred to be a result of partial relaxation of the rock mass occurring naturally along the fractures in the rock, hence some displacement due to unloading will have occurred prior to excavation as a consequence of geological phenomena.

The rock mass strengths are based on the expected Geological Strength Index parameter ($GSI^{(2)}$) for the relevant rock types.

Soil/Rock Unit	Cohesion (kPa)	Friction Angle (degrees)	Horizontal Unloading Modulus (MPa)
Residual Soil	5	30	100
Class V Shale	20	33	390
Class IV/III Shale	30	40	600
Class II Shale	50	44	2000

Selected geotechnical parameters adopted for design of the embankment widening are presented in the table below. The Select Fill was used in the reinforced earth block, while the New Fill is the material above the reinforced soil block and existing embankment.

The moduli used for design are lower than those used for the soldier pile wall, as the stress conditions were primarily compressive. The properties of the existing embankment fill were assessed from back-analysis of the existing 1.75H to 1V slopes, assuming that the existing embankment had a slope factor of safety of at least 1.2. This was inferred from the lack of evidence of slope creep or past slope instability.

Soil/Rock Unit	Cohesion (kPa)	Friction Angle (degrees)	Modulus (MPa)
Select Fill	0	36	50
New Fill	5	30	35
Existing Fill	2	30	15
Alluvium	2	28	15
Residual	5	30	35

GEOTECHNICAL DESIGN OF SOLDIER PILE RETAINING WALL

During tender, preliminary design was carried out using program WALLAP (Geosolve), using serviceability design parameters. This analysis indicated that the reference design of a contiguous pile wall could be

economised (as the structural capability of the piles was not being fully utilised) and a soldier pile wall with shotcrete panel infill was proposed. Depending on the wall height, expected ground conditions, and restrictions on displacement, the piles varied from 0.6m to 0.9m diameter, at spacings up to 2.4m (Refer Figure 3).

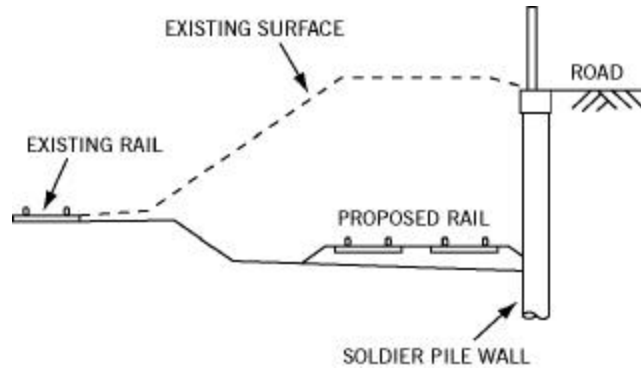


Figure 3 – Typical Section of Cut Retaining Wall

Detailed design of the soldier pile wall was trialed using more than one numerical analysis program. Sensitivity checks indicated that the soil/rock pressures behind the piles were well modelled by program *FLAC* (Itasca), which was then used to complete the analysis. The variance in results demonstrated the importance of not placing blind faith in the results of numerical analysis programs.

The analysis results were presented for two load conditions. Condition 1 represented the calculated serviceability displacements, without other external loads. Condition 2 included other loads such as rock wedge forces, and external loads such as wind loads on noise walls. For Condition 1, the analysis predicted displacements at the top of the soldier pile wall from 5mm to 25mm, depending on the spacing and size of the piles. Similarly, for Condition 2 predicted displacements ranged from 10mm to 35mm.

Detailed design led to other considerations, several of which are discussed below.

The project Deed specified a list of precedence for Design Codes. For piled retaining walls the Austroads⁽³⁾ design Code took precedence. The Austroads ULS design methodology required the geotechnical design parameters to be factored down for use in analysis, then the resulting structural forces factored up. The overall factor on design forces was a value of approximately 1.8. The reduced input parameters resulted in the peak bending moment occurring lower in the pile than for the case where unfactored geotechnical design parameters were used. This potential variability in the depth at which the peak bending moment could occur had to be clearly communicated to the structural engineer,

particularly as the Contractor requested an economic reinforcement design.

Concrete piles subject to bending can develop minor cracks, effectively lowering the pile stiffness. Hence lateral design of the concrete piles utilised a cracked-section modulus. The structural engineer advised us to use a cubic power formula to calculate an average cracked stiffness. The implication for design was that some iteration was required, as a more flexible pile attracted less load, which then changed the effective stiffness. This also had implications for displacements of the wall and supported ground.

The design pressures on the wall included forces from potential rock wedges. Whilst the force from a characteristic three-dimensional wedges could be resisted by the piles, consideration also had to be given to larger two-dimensional wedges arising from relatively continuous defects dipping out of the face at about 45 degrees. The risk of encountering such defects striking in this unfavourable orientation was considered to be low, but could not be discounted. There are some documented cases of encountering such wedges in Ashfield Shale, including one by Golder Associates in a recent basement excavation in Surry Hills. The impact on design meant that contiguous piles were designed to cope with the potential forces from such wedges. Where the spacing between soldier piles was greater than about 0.5m, geological mapping was required to confirm that no such defects were present. If a wedge was observed, it may have been necessary to install anchors. Such anchors would have extended outside the property boundary, which was not permitted. However, as the risk was considered low, and could only potentially impact public property, the Contractor and Principal accepted this risk management procedure.

Late in the detailed design stage the soldier pile retaining wall was affected by an issue regarding drainage provisions. The cut area had to be drained to the north, against the final grade, which meant a trench between about 0.5m and 1.8m deep had to be excavated near the toe of the soldier pile wall (refer Figure 4).



Figure 4 – Drainage Trench in front of Cut Wall

Relief of the passive restraint in front of the piles had not been designed for. Maintenance of the final track configuration meant that the drainage trenches were positioned about 3.7m from the wall. Our assessment considered this acceptable, provided the trench was constructed in sections no longer than 6m, backfilled with 2 MPa sand-cement mix (a similar strength as the surrounding rock), and geological mapping of the excavated shale was carried out to check for unfavorable joint orientations and potential failure blocks. Several response measures were available if potential failure blocks were exposed, including a reduction in the length of excavation, and temporary bracing.

RETENTION OF RAIL EMBANKMENTS AND GLOBAL STABILITY

The tender design of a pile counterfort wall was considered difficult to achieve in practice due to access constraints, and the Contractor sought to further assess the feasibility of a reinforced earth (RE) wall solution.

The RE wall solution required minimum levels of embedment (and strap length) to maintain global stability, potentially resulting in up to approximately 4.5m of excavation and temporary retention of the operating rail embankment. We suggested that it may be possible to satisfy global stability requirements by installing a row of soldier piles below the RE wall, thereby reducing the depth of excavation and height of temporary retention (Refer Figure 5). This solution was later adopted following a period of intensive design assessment, as discussed below.

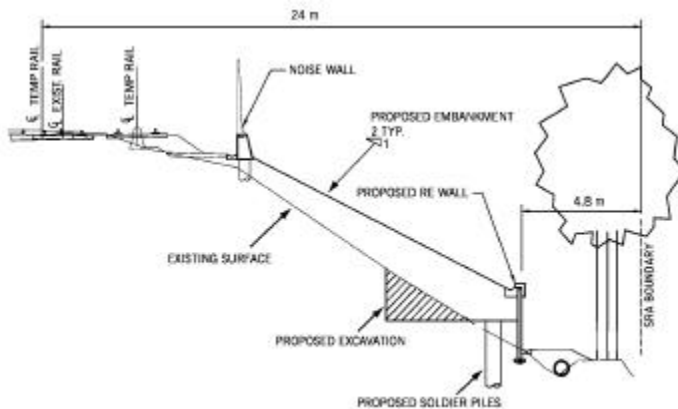


Figure 5 – Typical Section of Embankment Widening

To construct the proposed design, a temporary working platform 5 m to 6 m wide was required for machinery, access, and as a formation level for the reinforced soil wall. To achieve these widths, partial excavation and temporary retention of the existing embankment using sheet piles was proposed. To allow the sheet piles to be cantilevered without anchors or props, the level of the working platform for the reinforced earth wall was set to

limit the maximum height of sheet pile retention to 2.2 m.

The Ultimate Limit State (ULS) design methodologies adopted by the Design Codes specified in the Works Brief were incompatible with the design of the temporary sheet pile retention, as the factored (reduced) soil input parameters were less than the slope angle of the existing embankment (1.75H to 1V). As a result a working stress approach using engineering principles was adopted. A factor of 1.5 was applied to the forces calculated in the sheet piles for structural design.

The design of the sheet piles was carried out using program *WALLAP*. The results were supported by analysis using program *FLAC*, where the sheet piles formed a construction stage in the design of the permanent works. Larssen 6W sections (“back-to-back”) provided the required design capacities and were chosen by the Contractor for construction.

For the design of the permanent works, global stability of embankments and the use of soldier piles to improve the stability of embankments are also not covered by the Design Codes specified in the Works Brief. The stability analysis and design was thus carried out using engineering principles.

The stability of the proposed embankment was assessed using limit equilibrium methods and program *SLOPE/W* (Geoslope). The results of the analysis indicated that the factor of safety (FOS) against global slope instability (potentially undermining the reinforced earth wall) varied between about 1.2 and 1.4, if no measures were adopted to improve stability for the proposed depth of excavation. A design FOS of 1.5 was chosen for the permanent (operating) rail embankment.

Design of the stabilising soldier piles was carried out using two methods. The first assessed the additional restoring force (provided by the piles) required to raise the global FOS to 1.5. The magnitude of the additional force was assessed using *SLOPE/W*, for both circular mechanisms and wedge mechanisms. The bending moment and shear force profile in the pile was assessed by analysing the embedded pile below the point where it is cut by the critical failure surface, using the program *DEFPIC* (Sydney University).

The second design method used program *FLAC* to model the soil-structure interaction. The sequence of construction including installation of sheet piles, excavation, installation of soldier piles, construction of the RE block and construction of the final embankment was modelled in the *FLAC* analysis. In order to derive ultimate loads from the results of the working load *FLAC* analysis, the calculated working bending moments and shear forces were multiplied by a factor of 2, to

approximate the effect of an average load factor of 1.4, and an average material factor of 0.7.

Results of the *SLOPE/W* analysis indicated that the soldier piles would be required to provide an additional lateral force of up to 140 kN/m of wall in order to increase the global FOS to 1.5. Figure 6 presents a comparison of the factored bending moment and shear force profiles calculated using *FLAC*, and the *SLOPE/W* and *DEFPiG* combination. The results provided by the two methods were comparable, with similar peak bending moments and peak shear forces.

The design of the reinforced soil wall itself (including internal stability analysis) was carried out by Reinforced Earth Company. The design was based on AS4678⁽⁴⁾.

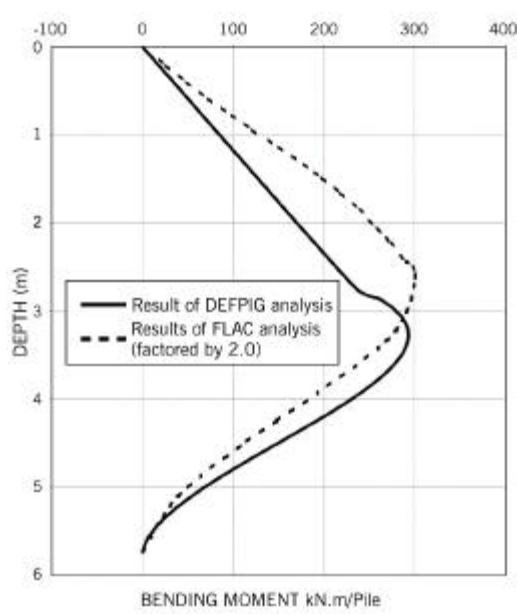


Figure 6 – Comparison of Predicted Pile Forces

The final design sought a solution that reduced the height of temporary retention, while not over-stressing the soldier piles, and adequately interfaced with the RE wall. The design specified 0.6m diameter bored cast in place concrete piles, at 1.8m centres. In most areas, to limit the height of the temporary cantilevered sheet piles to 2.2m, the level of the working platform was above the toe of the final RE wall configuration (see Figure 5 and 7). To maintain the continuity of urban design, the soldier piles were faced by the reinforced earth wall panels. The area between rear of the piles and the RE panels was backfilled with lean-mix concrete, to reduce the potential for plastic deformations associated with the soil squeezing between the piles. The select fill layer used to construct the RE wall was also placed behind the top 0.5m of the piles, to reduce the risk of sliding failure between the piles and RE block.



Figure 7 – Proposed base of RE block (left), and base of RE facing panels in front of piles (right)

CONSTRUCTION OBSERVATIONS

During the construction stage, geotechnical observations were carried out to confirm the design assumptions regarding ground conditions. For the cut areas and soldier pile wall these included:

- Logging cuttings of bored piers during drilling.
- Geological mapping of material exposed between piles. This included consideration of the design issue regarding potential earth pressures from relatively large two-dimensional wedges associated with continuous defects dipping at about 45 degrees. One such defect was noted, but the strike of the defect was approximately perpendicular to the cut face, and did not impact the design.
- Geological mapping of rock defects during excavation of the drainage trench in front of the pile retaining wall. Within the 6m long sections excavated, we did not observe joint defects continuous enough and of the correct orientations to form large passive failures.

For the embankment retaining wall, construction observations included:

- Confirmation of design bearing pressures for shallow footings and at the base of the reinforced earth wall.
- Proof rolling of subgrade.
- Assessment of unsuitable material.
- Observations of temporary sheet pile installation.

FIELD PERFORMANCE & COMPARISON WITH DESIGN

Monitoring plans were developed for both the pile retaining wall (cut) and embankment retaining wall (fill) were based on the “traffic light” reporting system, where “green” represents acceptable movements, “orange” indicates that further assessment of movements and a reduction in the rate of excavation/construction is required, and “red” means halt construction and implement remedial response.

Monitoring of the pile retaining wall indicated that measurable displacements continued for a period of about 3 months after the completion of excavation, which is

inferred to be the period required to effectively reach drained conditions and overall stress-strain equilibrium.

In summary, lateral displacements recorded at the top of the pile wall were:

- 7mm to 11mm in inclinometers.
- 6mm to 17mm from survey (repeatability was found to be typically $\pm 5\text{mm}$).

Comparisons with the predicted lateral displacements at the main locations chosen for *FLAC* analysis during design are presented in the following table.

Pile Config.	Predicted Displ. (mm)	Measured Inclinometer Displ. (mm)	Measured Survey Displ. (mm)
0.9m dia. @ 1.2m c/c	5 to 10 ^(a) 10 to 15 ^(b)	7	7 to 10 ^(c)
0.75m dia. @ 2.4m c/c	5 to 10 ^(a) 25 to 30 ^(b)	N/A	17
0.85m dia. @ 1.2m c/c	20 to 25 ^(a) 30 to 35 ^(b)	11	16
0.6m dia. @ 1.2m c/c	10 to 15 ^(a) 15 to 20 ^(b)	N/A	12

- (a) Load Condition 1 - serviceability displacements, without other external loads
(b) Load Condition 2 - includes other loads such as rock wedge forces, and external loads
(c) Two monitoring points in close proximity

As indicated above, the results of displacement monitoring were in good agreement with the predicted lateral displacements, justifying the use of higher unloading moduli for the fractured shale.

Monitoring of the embankment retaining wall included survey displacement monitoring of the temporary sheet pile wall during construction. Lateral displacements recorded at the top of the sheet piles varied between about 10mm and 25mm, which were within the “green” region of the surveillance criteria.

After construction, a program of precise survey monitoring of the completed wall was implemented. The survey targets were installed at the base and top of the RE wall, and on a concrete barrier at the top of the new embankment. As the predicted displacements were relatively low (the “green” region called for displacements less than 10mm in some areas), an accurate survey methodology was called for ($\pm 2\text{ mm}$ to 3 mm) to prevent unwarranted alarm over potential displacements in the “orange” region. The surveyor decided to install dedicated lugs in the concrete panels, to which a prism could be connected for survey.

Since the new rail lines became operational at the end of December 2003, displacements have been less than 8mm settlement at the top of the embankment, and less than 4mm lateral displacement at the face of the RE wall. These displacements are within “green” region of the surveillance criteria, and compare with predicted settlements at the top of the embankment of about 10mm to 15mm, and lateral displacements of the wall of about 5mm to 10mm.

CONCLUSIONS

Detailed design of the cantilevered soldier pile retaining wall incorporated several critical design issues, including the impact of applying Limit States approaches to geotechnical design, cracked section moduli for laterally loaded piles, and the use of higher moduli for fractured rock masses when applied to the unloading conditions in excavations. The validity of using realistic unloading moduli was demonstrated by displacement monitoring during construction. Management of geotechnical design risks associated with the fractured rock was achieved by geological mapping during construction.

The design of the embankment retaining wall presented significant design complications, including an operating rail line at the top of the embankment and limited construction access. To overcome these constraints, original designs were developed using soldier piles below a reinforced earth wall to improve the global stability of the final embankment, and to reduce the temporary excavation and retention required for construction of reinforced earth walls.

The successful design outcome also depended on cooperation between the Contractor, geotechnical engineer, and structural engineer.

ACKNOWLEDGEMENTS

The author would like to acknowledge the assistance of my colleagues at Golder Associates. Also thanks go to Transport Infrastructure Development Corporation, John Holland, Cardno MBK, Reinforced Earth Company, and Tony Phillips (geotechnical verifier).

REFERENCES

1. Pells, P.J.N., Mostyn, G., Walker, B.F., “Foundations on Sandstone and Shale in the Sydney Region”, Australian Geomechanics, December 1998, pp 17-29.
2. Hoek, E., Carranza-Torres, C., Corkum, B., “Hoek-Brown Failure Criterion – 2002 Edition”, 5th North American Rock Mechanics Symposium and 17th Tunneling Association of Canada Conference, April 2003, pp. 267-271.
3. “92 Austroads Bridge Design Code, Section 3: Foundations”, Austroads, 1992.
4. “AS4678–2002: Earth Retaining Structures”, Standards Australia, 2002.

Flexible Solutions in Flexible Ground, Te Ngaere Bay Slip Repair

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ABSTRACT

On Thursday 5th June 2003 a large crack appeared in Wainui Road above Te Ngaere Bay, part of the million dollar view tourist route in the Far North of New Zealand. The crack was initially sealed but continued to open up and by the 10th of June the road had subsided to the extent there was difficulty in traffic traversing the area. GHD was asked to urgently respond and report on the risk to road users and to properties below the road. The immediate risk was assessed and urgent safety measures arranged to avoid the road being completely cut.

Site investigations were implemented and a report outlining the causes of the failure, the ongoing risk, recommended remedial measures and estimated costs of repairs was prepared within two days. The recommended repair was a reinforced earth wall and deep subsurface drainage. Design and contract documentation was prepared and issued to selected tenderers and awarded 10 days after the initial crack had appeared in the road. Due to the rapid response and limited investigations undertaken, a flexible solution was chosen to allow modification and fine tuning of the design during construction resulting in an innovative and efficient solution.

INTRODUCTION

On Thursday 5 June 2003 a large crack appeared in Wainui Road above Te Ngaere Bay (Photo 1). The crack was initially sealed but continued to open up. By the 10th of June the road had subsided to the extent, there was difficulty in traffic traversing the area.

Concern was also expressed about the risk to dwellings below the road. GHD was asked to urgently respond and report on the risk to road users and to properties below the road.



Photo 1. Head Scarp of Slip on evening of 10 June 2003

SITE DESCRIPTION

The failure affected a 25m long section of the road (Figure 1). The failure was defined by several cracks extending across the road. The cracks had vertical displacements of up to 200mm. At the location of the failure the road appeared to have been formed on a fill embankment.

The toe of the landslide was 3/4 of the way down the bank where a prominent bulge has developed on the slope. There are two large blue gum trees on the slope, which had been back tilted. This indicated a deep-

seated failure on which might be the margins of a pre-existing landslide.

Several houses were located below the slip but our assessment indicated no distress of the house's structure or the surrounding ground.

There was evidence of past instability affecting the site. A two-basket gabion wall had been constructed to provide support along the outside edge of the road and a rock spoil buttress constructed along the uphill side of the road immediately adjacent to the recent slip site.

GEOLOGICAL SETTING

The site is mapped as being underlain by Waipapa Group greywackes and argillites with cherts, marbles and interbedded basic marine volcanics (Kear, D. and Hay, R.F. 1961). These greywackes are relatively massive but can be locally contorted, jointed, indurated siltstones with rarer sandstones. The regional dip is west along the eastern coastline, bedding is rarely seen.

This material typically forms weathering profiles consisting of several meters of soft, low permeability soils overlying rock. The greywacke, exposed in road cuttings adjacent to the site was closely jointed (50 – 100mm) with a dominant sub vertical joint set. Batter cuts in the area typically stand up vertically to 6m. The surficial soil veneer appeared to be relatively shallow.

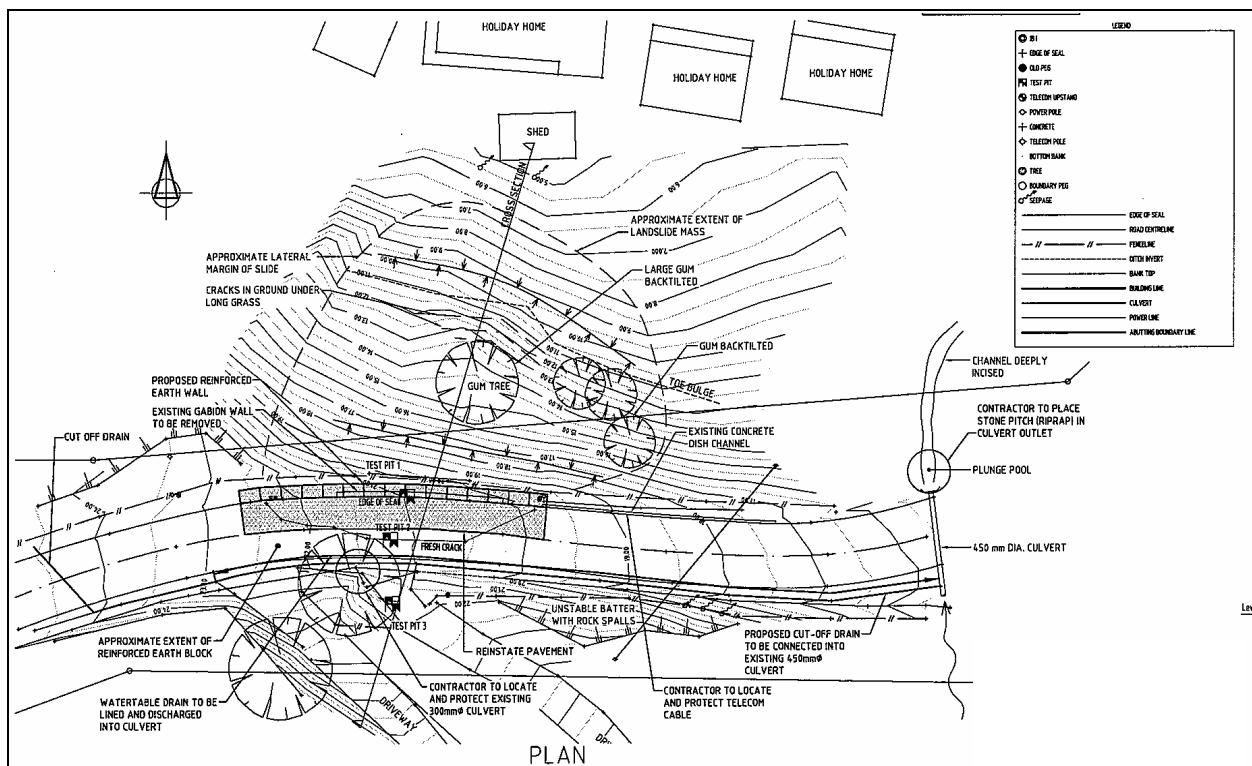


Figure 1. Site Plan

LANDSLIDE RISK ASSESSMENT

A landslide risk assessment was undertaken using the Australian Geomechanics Society Landslide Risk Management Guidelines. A Qualitative method of analysis was used.

Hazard Identification

The feature was identified as a landslide that had affected the road and adjacent properties. It was assessed as having the potential to develop into an earth flow and to impact on the dwellings. Although most of the dwellings are intermittently occupied, there were two permanent residents. The area affected was approximately 1200 square metres with an approximate depth of 3 metres. The volume of failed material was approximately 3500 cubic metres. The current rate of movement was slow but had the potential to be triggered into a fast earth flow with heavy rain.

Consequences of Failure

The consequences of failure were assessed as:

- Loss of road resulting in disruption to travelling public & residents.
- Danger to road users. Vehicles may drive into hole in scarp area.
- Damage to property. Landslide debris will fall onto private property.
- Damage to Dwellings – Landslide debris could impact dwellings or outbuilding.
- Public Outrage – Concern over danger and longer travel times.
- Political effects – FNDC could be criticised for not acting in timely fashion.

- Lives at risk – Motorist drives into hole. Catastrophic failure demolishes house with occupants inside (assessed as low).

The Qualitative Consequences to property were assessed as MAJOR.

Likelihood of Failure

Due to the continuing movement and widening of the tension cracks the landslide probability was assessed as almost certain. The Earth Flow probability was assessed as possible.

Risk Level Implications

The risk level matrix for the landslide case returns a result of Very High Risk

The Risk Level Matrix for the Earth Flow case returns a result as High Risk.

Due to the risk assessment returning a result of High to Very High Risk a rapid response was required with urgent remedial measures needed to avoid possible loss of the road resulting in damage to property and the public.

SUBSURFACE INVESTIGATIONS

Ground conditions were immediately assessed by 3 shallow machine-excavated testpits. Due to the remote location and urgency, the investigations were undertaken using a small 2-ton excavator with a 3m reach. Investigations locations are shown on the Site Plan, Figure 1. Subsurface conditions are summarised on the section (Figure 2).

RESULTS OF INVESTIGATIONS

The test pits encountered mixed, loose, wet, fill underlain by an apparent hard layer at least 2.8 meters down under most of the roadway width. After discussions with local contractors, it was revealed that the site used to be a gully where uncontrolled fill from local road widening projects had been dumped for many years.

CAUSE OF FAILURE

The failure below the road appears to have occurred as a result of saturation of the fill due to an extreme weather event resulting in elevated porewater pressures along the fill/natural ground boundary resulting in instability.

REMEDIAL OPTIONS

Movement was expected to continue if stabilisation measures were not implemented and further regression of the head scarp was likely. The road was unsafe in its current state with several cracks extending across the entire road. The cracks had vertical displacements of up to 200mm causing cars and trucks to bottom out across the depression. There was a high risk of further large-scale slope failure and regression of the head scarp occurring.

There was a real risk of losing the entire road, the toe could break out, and the failure material could reach as far as the back of the houses. With the run out at the base of the slope, it was anticipated that structural damage to the houses would result.

The client required a low cost flexible solution that could be easily modified during construction without huge cost increases, which also provide a long-term permanent solution.

Several options were considered for remedial works including soil cement columns and pole walls. Soil cement columns were discounted due to the unproven NZ track record in this type of situation and the dire consequences of failure. A cantilevered pole or steel H pile wall was also discounted, as the ability to drive the piles sufficiently into the underlying greywacke rock was unknown. It was likely to result in either damage to the H pile or a shattered rock socket with minimal strength possibly resulting in a toe kick out failure. There was no time or budget for additional field-testing to determine if sufficient embedment of the piles was achievable.

The remedial solution adopted was a geogrid reinforced fill in conjunction with deep drainage. Although reinforced walls were rarely used on the Far North Network, from experience, it was known to be a very practical and flexible solution that could be easily modified during construction without huge cost increases.

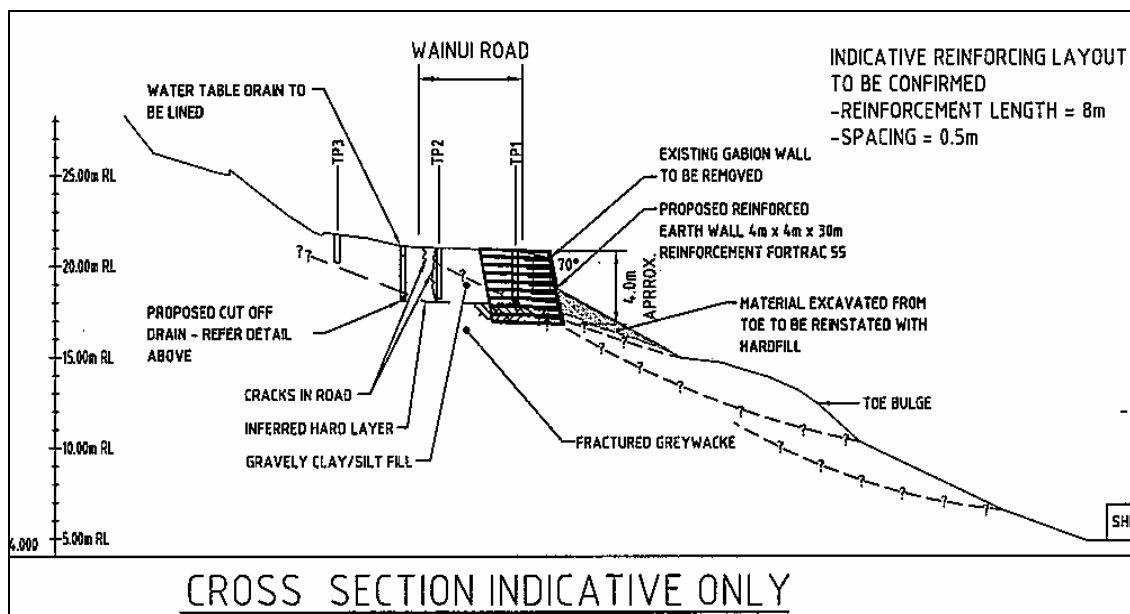


Figure 2. Section

Maccaferri software Macstars 2000 was used to design the wall (Figure 3). Due to the limited subsurface investigations undertaken and the possibility ground conditions would differ from those assumed several different wall heights were analysed.

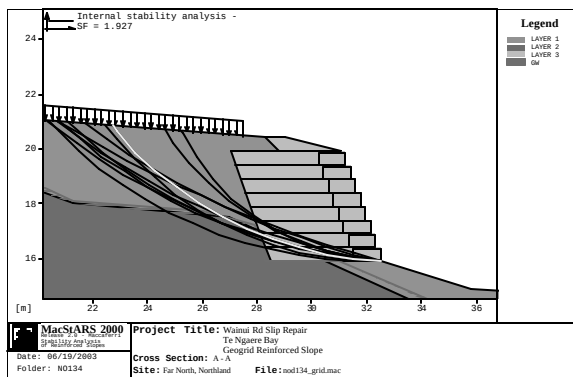


Figure 3. Macstars Reinforced Wall Analysis.

The reinforced earth wall would involve removal of the weak fill material underlying the road and replaced with compacted hardfill with a Fortrac 55 geogrid wraparound. The existing fill material was not reused due to high organic and clay content. The use of a locally available hardfill resulted in a higher strength fill, easy construction, lesser fill testing and monitoring requirements.

A major design consideration was to maintain at least one lane of traffic during construction, as there were no alternative detour routes close to the site.



Photo 3. One lane remained open during construction.

Drainage works would comprise deep subsurface drainage and surface runoff control. The aim of the drains was to intercept groundwater flow along the soil/rock boundary and divert surface runoff away from the slip area.

MODIFICATIONS DURING CONSTRUCTION

Due to the rapid response requested by the client the remedial solution was based on very limited subsurface investigations. Critical to the success of the remedial works and our cost estimate was adequate foundation conditions for the wall. It was therefore recommended that prior to works commencing, foundation conditions

be confirmed along the entire length of the wall with additional investigations.

Due to budget constraints it was decided additional investigations would be undertaken during construction to confirm the assumed ground conditions.

The design was initially modified due to a shortage of Fortrac 55 and Terramesh was substituted with 0.5m high gabion baskets used as the facing.

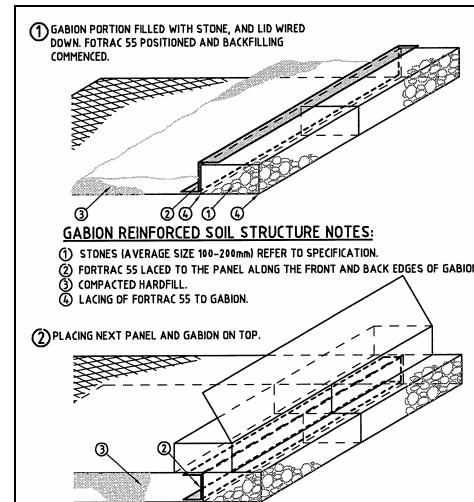


Figure 4. Gabion Facing

The design was modified a second time during construction when the assumed hard layer identified during the initial investigations at 2.8m below road level was discovered to be a layer of hardfill possibly associated with an old road.

Competent founding material was found to be a further 3m below this old road level. This was not unexpected and additional wall designs had been undertaken for just such an eventuality. The wall height was raised to 4.5m high with a 2m hardfill foundation pad excavated under the base. Groundwater was encountered in the base of the excavation and subsoil drains installed.



Photo 4. Completed wall

CONCLUSIONS

Due to the risk assessment returning a result of High to Very High Risk a rapid response was required with urgent remedial measures needed to avoid possible loss of the road resulting in damage to property and the public.

Due to the rapid response and limited investigations undertaken, a flexible solution was chosen to allow modification and fine tuning of the design during construction resulting in an innovative and efficient solution.

During construction, GHD staff were on site at critical times to inspect excavations and to vary the design of the remedial work to suit the final excavated profile.

Key features of this project included:

- The rapid response to the event– the total elapsed time from advice of the emergency to the letting of a competitively tendered contract for remedial measures was 10 days.
- An innovative but very practical solution – reinforced earth walls had been rarely used on the Far North Network.
- Minimum disruption to the road users.
- Value for money – the competitive tendering procedure resulted in a contract price below original estimates.

ACKNOWLEDGEMENTS

The author wishes to thank Bob McKelvey of GHD for his contribution and review of this paper.

REFERENCES

1. Kear, D. and Hay, R.F. 1961, Geological Map of NZ, Sheet 1 –North Cape, NZ Geological Survey, 1:250,000
2. Landslide Risk Management Concepts and Guidelines – Australian Geomechanics Society Sub-Committee on Landslide Risk Management *Australian Geomechanics* – March 2000.

Numerical Analysis of Large Adjacent Caverns for Sydney's Epping to Chatswood Rail Line

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Abstract: The Epping to Chatswood Rail Line is the NSW Government's largest publicly funded infrastructure project. It is currently being constructed through the Triassic age sandstones, siltstones and shales of the Sydney Basin. The project consists of twin, 12.5km, 7.2m diameter rail tunnels, 4 new station caverns and numerous shafts/tunnel service buildings. Macquarie Park, Macquarie University and Delhi Road Stations are 3 of the 4 stations along the route. The layout of each of these stations requires the construction of a number of large caverns (20m span) and shafts within close proximity of each other.

As part of the design process, a large amount of numerical modeling has been conducted to investigate the potential interaction between these large adjoining cavern structures. The presence of relatively high insitu lateral stresses has presented a number of challenges for the design process. This paper briefly describes a suite of three dimensional distinct element analyses (3DEC) conducted to investigate the behaviour of these large adjacent excavations. Particular reference is made to how these analyses were used as part of a larger design process.

INTRODUCTION

Major construction for the Epping to Chatswood Rail Line commenced on the 25th November 2002. Upon completion, up to 25km of rail tunnel, 4 new station caverns and numerous shafts/tunnel service buildings will be added to Sydney's current rail infrastructure.

This paper aims to present an insight into how, as part of the station cavern design process, three dimensional (3D) numerical analyses were used to enhance plane strain modeling in order to develop a fuller understanding of the likely interaction between the various adjacent caverns and shaft structures.

ROUTE GEOLOGY

The route of the Epping to Chatswood Rail Line lies within the Triassic age sandstones, siltstones and shales of the Sydney Basin. In particular, the three uppermost formations, the Wianamatta group (typically Ashfield shale), the Mittagong formation and Hawkesbury Sandstone constitute the encountered bedrock.

Figure 1 presents the stratigraphic sequence for these three formations (Herbert, 1983). In general, a movement up the sequence sees a progressive decrease in the encountered grain size, from the medium to coarse quartz grained Hawkesbury Sandstone to the claystones and siltstones of the Ashfield Shale. The Mittagong formation, being a transitional sequence between the adjacent formations, retains aspects of both depositional environments, with interbedded shales and fine to medium grained sandstones typifying this sequence. All three formations are near horizontally bedded and in general contain horizontal stresses significantly higher than the corresponding geostatic pressures (Enever et al., 1990, Pells, 1990, Enever, 1999).

AGE		STRATIGRAPHIC UNIT
MIDDLE TRIASSIC	WIANAMATTA GROUP	South of Hawkesbury River
		Bringelly Shale - Potts Hill Sandstone - Cobbitty Claystone Bed
		Minchinbury Sandstone
		Ashfield Shale - Mulgoa Laminite - Regentville Siltstone - Kellyville Laminite - Rouse Hill Siltstone
		Mittagong Formation
MIDDLE TRIASSIC	WIANAMATTA GROUP	Hawkesbury Sandstone

Figure 1: Stratigraphic sequence for the upper Sydney Basin formations (Herbert, 1983).

Whilst the proposed tunnel alignment is such that excavation is predominately within the Hawkesbury Sandstone, the proximity of the station caverns to the surface, as well as the gradational nature of the Hawkesbury/Mittagong contact has meant that the engineering properties of both the Mittagong formation and Ashfield Shale have had to be considered for the station cavern design.

CAVERN LAYOUT

Figures 2 and 3 present a plan view and typical cross section for the Macquarie Park Station Cavern. Whilst, strictly speaking, the set out of each station cavern is unique, the general layout of the platform and concourse cavern and the typical cross section for Macquarie University and Delhi Road Stations are similar to those shown here.

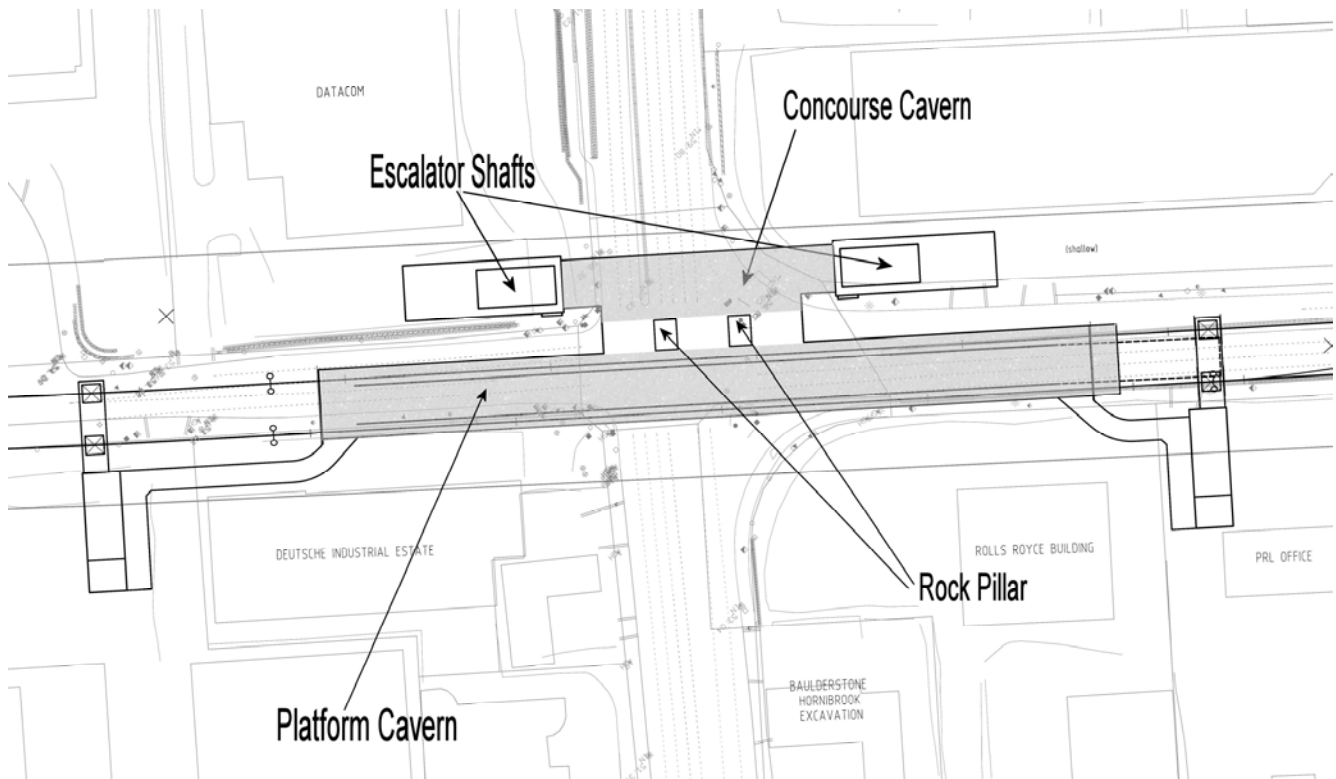


Figure 2: Plan view of Macquarie Park Station Cavern.

The main station structure is the 190m long, 15m high, and 20m wide platform cavern. When completed this cavern will contain the rail passenger platforms. Immediately adjacent to the platform cavern sits the smaller concourse cavern. This 50m long, 15m wide and 8m high cavern will contain ticketing booths and barriers. Access to the caverns from the surface is via two open shafts, which can be seen connecting to either end of the concourse cavern.

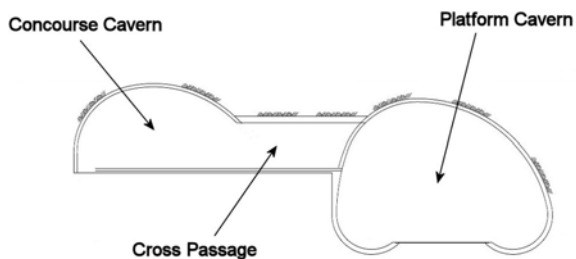


Figure 3: Typical cross section for the Macquarie Park, Macquarie University and Delhi Road Station Caverns.

ANALYSIS METHODOLOGY

As can be seen from Figures 2 and 3, the layout of the station caverns and the nature of the access tunnels between them is such, that modeling the structures using entirely plane strain conditions is challenging. In

particular, note is drawn to the pillars formed by the cross passages between the concourse and platform caverns and a narrowing of the access shaft length towards the base. Such features are intrinsically 3D and as such a modeling methodology that was capable of considering the 3D nature of the caverns needed to be developed.

Notwithstanding this, tight design deadlines in conjunction with the desire to use parametric studies in order to deal with the uncertainties inherent to the analysis of discontinuous rock (where the nature and location of the discontinuities is preponderant in determining the behaviour of the rock mass) meant that time taken to set-up, run and review the analyses was also of consideration.

In order to deal with these potentially conflicting demands, it was proposed that the modeling methodology involve a combination of 2D and 3D analyses. The influence of 3D restraint would be approximated within the plane strain models using various numerical techniques (e.g. material smearing). Limited 3D modeling could then be used to 'truth test' these plane strain approximations at different locations within the caverns via comparison of 'equivalent' 2D and 3D models.

In this way, the parametric studies carried out using the relatively rapid 2D plane strain analyses could be

enhanced using the results of the 3D analysis, with the 3D ‘truth testing’ process providing increased confidence that the adopted 3D approximation techniques were providing a reasonable approximation of the 3D interaction between the cavern structures.

ANALYTICAL PROGRAMS

The following three computer packages were selected for the analysis process:

- Examine^{3D} (Rocscience)
- 3DEC (Itasca)
- Phase² (Rocscience)

Each package utilizes an independent analysis method to calculate rock response (Examine^{3D} is a 3D boundary element program, 3DEC is a 3D distinct element program and Phase² is a 2D finite element program). A conscious decision was made to use independent analysis methods, because by comparing the results of ‘compatible’ analyses, some indication could be had as to the influence of the various simplifications and assumptions inherent to each specific method.

EXAMINE^{3D} (Rocscience)

Examine^{3D} is a commercially available 3D boundary element program. It uses infinite and semi-infinite elements to model the rock mass surrounding the tunnel. The method provides significant computational efficiency for 3D analyses, because only the model surface is discretised. As such, it was originally intended to be used for the majority of the 3D parametric variation work. Unfortunately however, the programs inherent assumption that the rock mass can be represented by an isotropic, homogeneous, linear elastic material proved restrictive for this application (see section titled ‘Effects of material layering’) and consequently use of Examine^{3D} was typically restricted to visualisation of the complex cavern facilities and basic validation/parametric studies. As an example of the programs visualisation capabilities, Figure 4 presents a view from inside the platform cavern, created using Examine^{3D} and a customized plotting program.

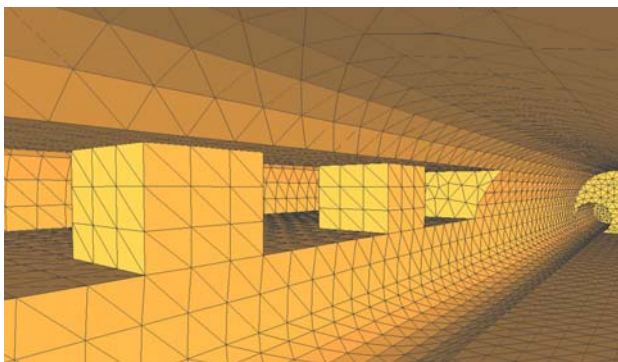


Figure 4: 3D view looking from inside the Platform Cavern, across to the concourse cavern.

3DEC (Itasca)

3DEC is a 3D distinct element program. The considered model is broken up into a series of deformable or non-deformable blocks, which then interact with each other according to various inbuilt joint behaviour models. A solution is obtained through the use of a ‘time-marching’ scheme to solve the equations of motion.

3DEC is well suited to model a rock mass where the behaviour of the discontinuities has a significant influence on the total rock mass behaviour. The model geometry is regularly updated during the calculation process, so the effects of large strains or concentrated displacements can be considered. Different material properties can be incorporated into the model and complex models (including staged excavation) can be developed.

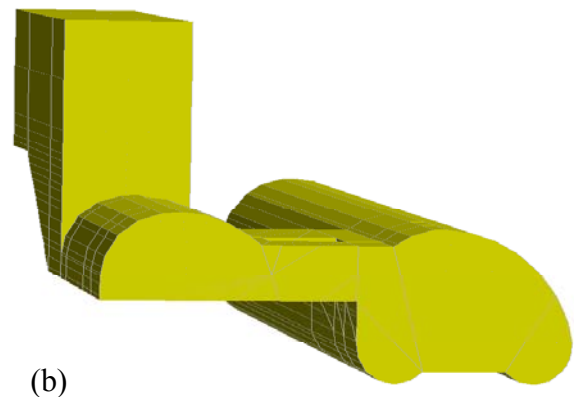
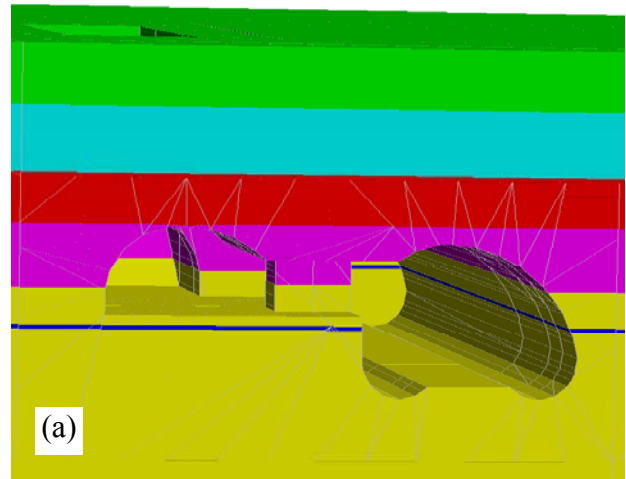


Figure 5: 3DEC model of the Station Cavern (a) blocks remaining following excavation of full cavern (b) excavated blocks.

As an example of the complexity of model attainable, Figure 5 presents one of the station cavern models developed for the project. As can be seen, both major caverns, the escalator shaft and the interconnecting tunnels have been specifically modeled (an axis of

symmetry across the station centerline has been used to reduce the required model size). Further, material layering (the colour layering shown in Figure 5 (a) indicates the modeled material layering) and horizontal bedding features (modeled as Mohr-Coulomb joints) have also been considered.

Whilst both analytically rigorous and capable of considering complex 3D models, for the type of model shown, 3DEC required considerable time for model development, analysis and result review (some of the considered models took up to four days to analyse). This meant that extensive parametric variation and option evaluation was not feasible solely using 3DEC. Instead the program was used to target specifically 3D issues (e.g. the validation of plane strain smearing techniques or the analysis of the cross passage pillar).

PHASE² (Rocscience)

Phase² is a commercially available 2D finite element program developed for the calculation of stresses and displacements around underground or surface excavations. The program is capable of modeling plane strain or axisymmetric models in jointed, layered, elastic or plastic materials with or without rock reinforcement (the formulation has 4 inbuilt bolt models and 2 inbuilt liner models).

Compared with the other packages Phase2 is relatively rapid for project set-up, analysis and review. The inbuilt staging devices mean that the program is versatile in its ability to consider various excavation options and to compare the relative effect of the considered permutations. This means it is well suited to assist with parametric studies, option evaluation and design queries.

As with any 2D finite element program, there are a number of simplifications inherent in the formulation. In order to assess the impact of these simplifications, a series of validation procedures were developed. Of particular interest to this paper are the 3D smearing techniques, which were used to approximate the effects of out of plane restraint (see section titled 'Estimation of 3D restraint'). Via this process, it was possible to perform and review a vast suite of analyses so that a broad understanding could be had of how the rock mass responded to excavation of the caverns and what factors principally drove this behaviour.

ANALYSIS RESULTS

In terms of the scope this paper, it is intended that the work presented herein provide an insight into how the problem of analysing an intrinsically 3D structure, in a limited time period was approached by the numerical modelers for this project. In keeping with this, results relating to two specific issues are presented, namely:

- 2D estimation of 3D restraint, and
- Effects of material layering.

These two areas have been 'singled out' because they encompass issues which impacted on how the proposed analysis methodology was eventually enacted.

2D ESTIMATION OF 3D RESTRAINT

Figure 5 (b) presents a 3D view of the 3DEC station cavern model excavated elements. As can be seen, the layout of the escalator shaft is such that, for the bottom half of the shaft, the length of the opening decreases with depth. Figure 6 shows a section through the 3DEC blocks remaining after excavation. The section has been cut approximately through the centre of the shaft, with all the blocks in front of the section hidden from view. As can be seen, the action of the sloping floor behind the section provides restraint to the shaft sidewalls in line with the section.

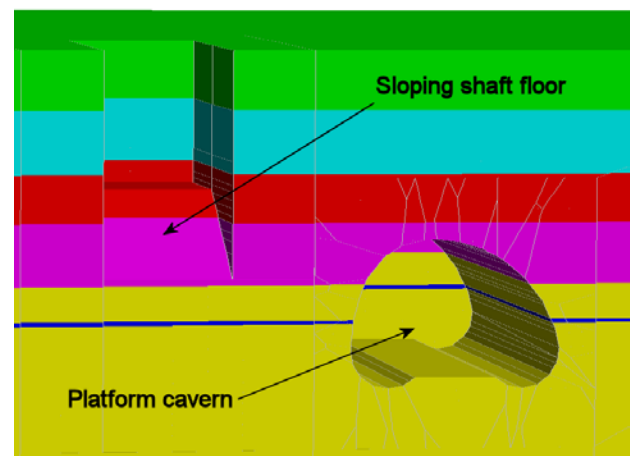


Figure 6: Section through 3DEC unexcavated blocks at escalator shaft mid section.

In order to develop a 2D, plane strain model of this section, it was necessary to approximate the action of this 3D restraint. This was achieved by a process of material smearing, whereby (instead of full removal) excavated material is replaced with a reduced stiffness material. The final smearing factors used were based on comparisons of the calculated displacements for the 3DEC model and a suite of Phase² models (each considering different smearing schemes). Displacements at a number of specific 'points of interest' within the two models (e.g. at the cavern crown or between the platform cavern and the shaft) were compared.

As an example, Figure 7 shows a comparison of the calculated horizontal displacement for the 3DEC model and the adopted Phase² model, at a location between the shaft and the platform cavern. Also shown is the final smearing scheme. As can be seen three separate smeared

zones were required. For the top half of the shaft, the length of the sidewall (approx. 50m) meant that no smearing was required to approximate the conditions at the query point. For the bottom half of the shaft two zones, using an 85% and 70% reduction in stiffness respectively, were required. These smearing factors were subsequently adopted for 2D plane strain analysis of this cavern section.

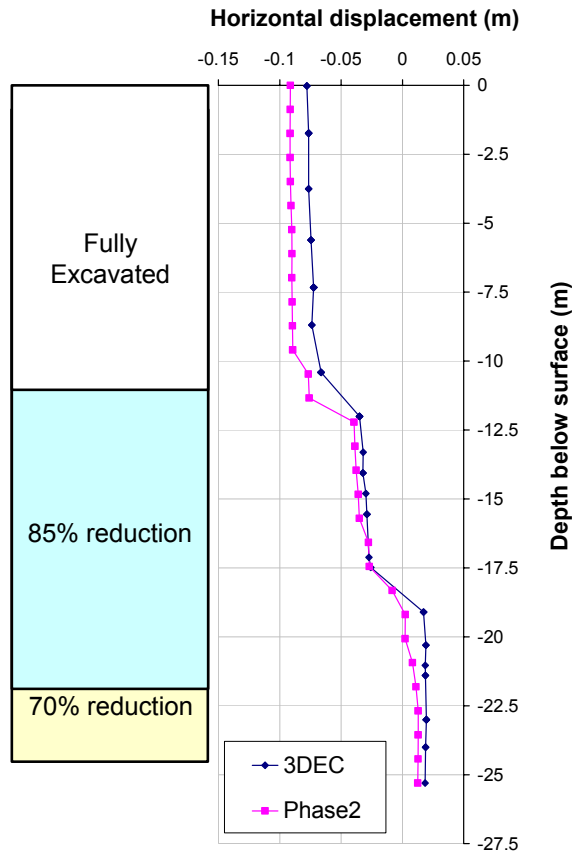


Figure 7: Comparison of the calculated horizontal displacement – 3DEC and Phase2 (with smearing).

EFFECTS OF MATERIAL LAYERING

It has been mentioned previously, that the intended role of Examine^{3D} was re-evaluated after initial program validation. Studies showed that the program's inbuilt assumption that the rock response could be approximated as a simple isotropic, homogeneous, linear elastic material, proved to be too restrictive for the intended purpose.

Figure 8 presents the results of verification work conducted using 3DEC. These results provide an example of the impact assuming a simple isotropic, homogeneous, linear elastic material had on the calculated behaviour. Figure 8 (a) shows the displacement vectors calculated using the simplified homogeneous material (this behaviour is essentially

identical to that displayed by the Examine^{3D} model). In particular, note is drawn to the heave calculated in the crown of the platform cavern and at the surface. Figure 8 (b) shows the effect of introducing some material layering so that the Young's Modulus of the rock mass increases with depth. As can be seen, the calculated displacement mode of the cavern is significantly different for the layered soil, with the cavern crown and ground surface now sagging into the excavation.

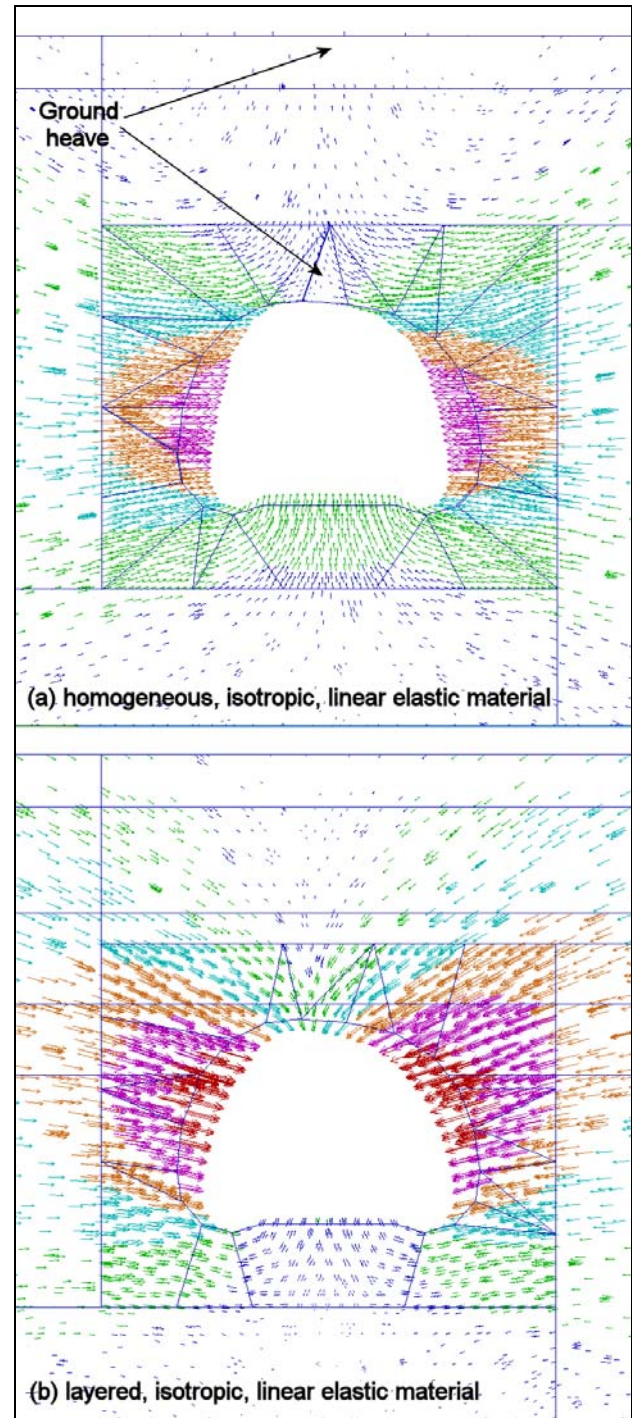


Figure 8: Vectors of total displacement calculated for (a) homogeneous material and (b) layered material.

Once the calculated surface heave was identified, a series of analytical investigations were conducted using all three analysis programs. These investigations sort to (a) reproduce the behaviour using independent analysis methods and (b) understand what was driving this mode of displacement. In summary, the results of this investigation showed that the ‘homogeneous’ displacement mode (Fig. 8(a)) was:

- Independent of the properties of the homogeneous material (material properties consistent with highly weathered shale and fresh sandstone both displayed similar patterns of soil displacement, with only variations to the magnitude of the displacements).
- Dependent upon relatively high horizontal insitu stresses. Surface heave did not occur in cases where the lateral stress was equal to the overburden stress (the results in Fig. 8 assume $\sigma_h = 0.5 + 2.5\sigma_v$ (MPa)).
- A result of mainly modeling the rock mass as a simple homogeneous material (Fig. 8 (b) shows that the introduction of basic material layering changed the predicted cavern crown behaviour considerably (i.e. from heave to sag)).

As these investigations showed, the oversimplification of the insitu geology had the potential to significantly alter the predicted behaviour.

CONCLUSION

Numerical modeling for the Epping to Chatswood Rail Line posed numerous challenges. This paper details how, in order to meet these challenges, an analysis methodology was employed which used detailed 3D modeling to enhance extensive 2D plane strain modeling, so that a broad understanding could be had of the response of the rock mass to the excavation of the station caverns.

As is explained, in areas where the out of plane cavern geometry was seen to be providing restraint to the in plane geometry, the results of 3D modeling were used in conjunction with 2D ‘material smearing’ techniques to approximate the action of the out of plane restraint. This system meant that numerous permutations (relating to either parametric studies, option evaluation or even responding to analysis queries) could be considered in a timely manner.

ACKNOWLEDGEMENTS

The above paper considers a small portion of a voluminous body of innovative work conducted for the Epping to Chatswood Rail Line. The author would like to gratefully acknowledge the tutelage and guidance provided by the project design team, in particular Mr.

Peter Stone and Mr. Kim Chan who are responsible for a lot of the work described herein.

REFERENCES

1. HERBERT, C, “Sydney Basin Stratigraphy”, 1983, p.7-26, in Geology of Sydney 1:100 000 sheet, Dept. of Mineral Resources.
2. ENEVER, WALTON and WINDSOR, “Stress Regime in the Sydney Basin and its Implications for Excavation Design and Construction”, 1990, Institute of Engineers, Australia, Australian Tunneling Conference, 1990.
3. PELLIS, P.J.N., “Stresses and Displacements around Deep Basements in the Sydney Area”, 1990, Institute of Engineers, Australia, Australian Tunneling Conference, 1990.
4. ENEVER, J.R., “Near Surface Insitu Stress and its Counterpart at Depth in the Sydney Metropolitan Area”, 1999, Australian Geomechanics, June 1999.

Slope Stabilisation-Site investigation, Design and Application of Deep Soil Mixing

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Abstract: State Highways in the Northland region of New Zealand are susceptible to instability due to slope failure. One of the main reasons is the unique properties of the Northland Allochthon that is underlying the area. Typical geomorphology within the Northland Allochthon is characterised by numerous shallow-seated failures due to low residual strengths. In some locations the problems have been compounded by inappropriate drainage measures or additional surcharging by placing fill on a subsiding road formation.

Deep soil mixing has been used as one of the remedial solutions to this problem. This paper presents a discussion and description of the regional geology and unique geotechnical characteristic properties of the materials. A case study is presented to demonstrate typical methods of investigation, interpretation and design of the adopted remedial works.

INTRODUCTION

Many slopes within the Northland Allochthon geology in the Northland region of New Zealand are known to be susceptible to instability and creep over time, partly due to the unique characteristic of the Northland Allochthon which has a low strength and a highly sheared fabric (Winkler [1]). Given the unique characteristics of this material, road instability due to slope failures are not uncommon. In some locations the slope problems have been compounded by inadequate drainage and inappropriate remedial work in the past such as placing additional fill on a subsiding road foundation.

Transit New Zealand embarked on Performance Specified Maintenance Contracts (PMSC) 002 which is the state highway maintenance contract that covers Northland. This contract places significant emphasis on maintenance/repair works innovation. SKM provides the professional services input to Encore who provides fixed price quote design repairs works to Works Infrastructure, who is the main contractor of PMSC 002. Encore is a joint venture of Work Infrastructure and Highway Stabiliser who brought the deep soil mixing technology to New Zealand.

This paper discusses developing deep soil mixing methods as a viable option to address the road slips. The regional geology and the unique geotechnical characteristic properties of the material are also discussed. A case study is presented to demonstrate the typical methods of investigation, interpretation and design of the adopted remedial work using deep soil mixing.

GEOLOGICAL SETTING AND GEOMORPHOLOGY

The distribution of the Northland Allochthon is shown on Figure 1. The Northland Allochthon stratigraphy is very distinctive due to the inversion of the normal stratigraphic sequence where older rocks (late cretaceous-90Ma) are found to lie on top of younger rocks (earliest miocene-25Ma) in many places. It is believed that large scale movement of the rock layer occurred with the layers sliding off from the top first followed by the second layer which came to rest on top of the first and so on. Consequently many shear planes have developed in the material producing the unstable formation (Isaac et al [2]).

Based on the age of original deposition (Winkler [1]), the Northland Allochthon is subdivided into three categories as follows:

- i. Tangihua Complex -submarine basaltic volcanics, about 90Ma to 65Ma
- ii. Mangakahia Complex – Variably calcareous and siliceous mudstones and sandstones that were initially deposited during Paleocene to Eocene times about 65Ma to 38Ma. It is also known as Onerahi Chaos.

- iii. Motatau Complex – Predominantly calcareous limestones, mudstones and sandstones that were deposited later during the Oligocene period about 25Ma to 38Ma

The Mangakahia complex is the most problematic group of materials from a geotechnical perspective. The soil mantle over this lithology consists of light colour, stiff to very stiff consistency, plastic silty clay (upper zone) and silty clays/clayey silts with gravel sized fragments (lower zone). (Winkler [1])

The geomorphology of Northland Allochthon is typically long, gentle to moderately grading broad hummocky slopes with hummocks generally extending all the way from the toe of the slope to the crest of the ridge, and side spurs. The hummocks are generally between 5m to 20m wide, tend to be oval and aligned parallel to the surface contours. Swamp reeds are found growing on the surface due to the very low permeability of the underlying soil (Figure 1). Another characteristic is the presence of numerous shallow-seated slope failures. Shallow failures have been observed on relatively gentle slopes of 5° to 15° , due to the low residual strength and ongoing creep movement of the in situ material (Winkler [1]).



Figure 1. Location of Northland Allochthon (Winkler [1]) and typical geomorphology of the Northland Allochthon

GEOTECHNICAL CHARACTERISTIC PROPERTIES

The Northland Allochthon soils typically comprise high plasticity, light coloured, clayey silts and silty clays with some gravel. These soils, in their undrained condition are usually stiff. However the residual friction angle is significantly affected by the presence of shear zones and sheared fabrics and is typically in the range of 10° to 15° . Winkler [1] has reported residual friction angles as low as 8° in this material. It is noted that the friction angle within the sheared zones for the calcareous and siliceous rocks are likely to be higher and typically in the order of 20° .

Northland Allochthon soils are highly plastic with a high shrink-swell potential. Liquid limit is highly variable and typically in excess of 80% and shrinkage limit less than 15%, indicating that the soil is highly reactive and is susceptible to creep. (Winkler [1])

A CASE STUDY: GALLIES SLIPS

The Gallies Slips are located on State Highway 12, approximately 90 km south east of Whangarei as shown on Figure 2. The slips have occurred in an existing road embankment. This stretch of the highway has previously been subjected to slope failures and unsuccessful slip repair work. The selected case study includes three slips with a total length of 50m over 190m of highway. The site is bounded on the north by water logged hummocky farmland and on the south by a farmhouse and some sheds (Figure 2). Figure 3 illustrates the extent and shape of each slip.

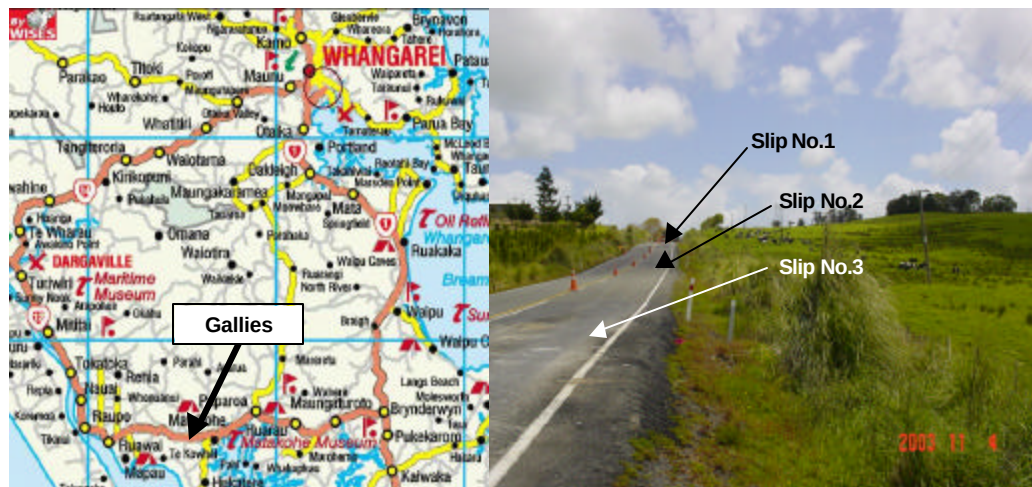


Figure 2. Location of Gallies Slip.

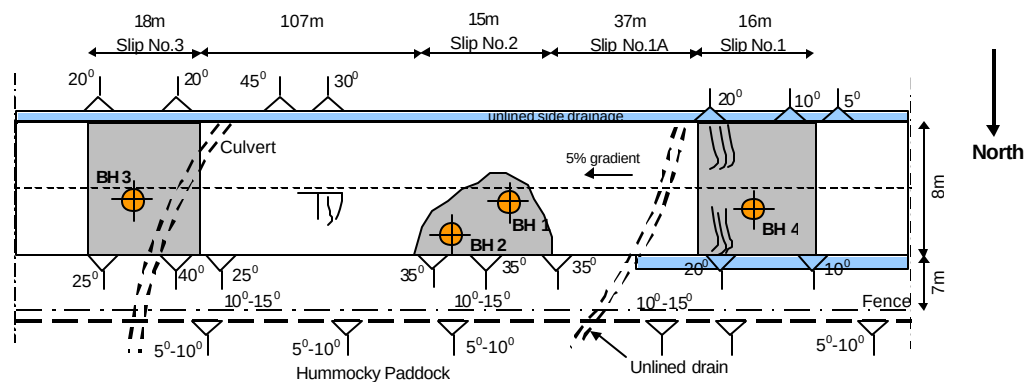


Figure 3. The geomorphology sketch of the site including borehole locations.

Previous slope failures have been re-surfaced at Slip No.1. Cracks that are perpendicular to the road are visible. The slip follows the slope of the road (approximately 5% gradient) for a distance of about 16m. The measured slope angle of the southern embankment batter ranges between 5° to 20° and the northern slopes are typically 10° to 20° .

Slip No.2 is about 37m downhill of the first slip. It is located on the eastbound carriageway and slightly encroaches into the westbound carriageway. An open crack is observed on the eastbound carriageway that starts at the edge of the road and extends to the centre line. This slip is approximately 15m long, and the embankment on the north is battered at approximately 35° . The measured slope angle below the embankment ranges between 10° and 15° .

Slip No.3 is approximately 107m downhill from Slip No.2. It straddles the eastbound and westbound carriageways. An open crack and minor settlement is visible within the westbound carriageway. The length of the slip is approximately 18m with embankment slope angles ranging between 20° and 40° . The measured slope angle below the embankment ranges between 10° and 15° .

This paper presents the remedial design for Slip No.3 in some depth. A similar remedial methodology was also successfully applied to slip No.1 and Slip No.2, but is not presented in this paper.

Site Investigation Performed and Results

Four boreholes were drilled in October 2003 to investigate the subsurface conditions. The locations of the borehole are shown on Figure 3. Undrained shear strengths were measured at approximately 1.5m depth intervals within the softer materials encountered. Push tube samples were taken from the soft sediments below the road fill. The recovered soil/rock cores were photographed. A selection of soil samples was subjected to geotechnical laboratory testing. A piezometer was installed in BH 2. The groundwater level in BH2 was recorded at 1.9m BGL 6 days after drilling.

The subsurface geology consists of road embankment fill ranging in thickness from 1.5m (BH1 and BH2) to 2.7m (BH3). The road embankment fill overlies soft to firm grey silty clays and light grey sandy silts. Bluish grey to grey mudstone of the Tangowahine Formation was generally encountered at depths ranging from 2.4m (BH4) to 7.35m (BH2) beneath the present ground level. The corrected undrained shear strength of the Tangowahine Formation clay/silt ranged from 50 to 150 kPa.

Slip Mechanism (Slip No. 3)

The geomorphology at Gallies comprises typical undulating Northland Allochthon landforms, which are suggestive of creep movement occurring on a global scale. It is likely that a majority of the existing hills are gradually creeping towards gullies and valleys.

Slip No.3 has occurred in a road embankment that has been constructed in a cut and fill slope. The rotational movement is unlikely to be the primary slip mechanism as no evidence of toe heave was observed. The depth of the fill material found indicates that settlement and slippage have been occurring for some time, and the road fill was probably re-graded and re-surfaced on more than one occasion in the past. These previous repair works has probably resulted in increasing the weight of embankment which causes further slippage. The on going creep movement reduces the strength of the soils and impacts on the bearing capacity of the subgrade.

High groundwater during wet periods and a lack of adequate drainage are likely to have contributed to the slip occurrence. Further, any leakage from the culvert crossing to the north of the carriageway would have contributed to destabilising the slope.

The pavement crack profile and geomorphology suggest that there is significant lateral creep movement towards the farmland, while some longitudinal movement is likely to be occurring as well. The slope angle to the north of the embankment ranges between 10° and 15° , which is sufficiently large to induce creep movement in the Northland Allochthon materials.

Deep Soil Mixing Column Design

It is judged that, Slip No.3 is likely to be developing multiple failure mechanisms. The conventional remedial measures such as gabion wall, soldier pile wall, stabilising berm and counterfort drains do not address all of the failure mechanisms.

Gabion wall and soldier pile wall do not address settlement and bearing capacity issues. A stabilising berm, although it is low cost could exacerbate settlement. Counterfort drains control groundwater and soft weak material is removed. However, it can cause disruption to traffic, unacceptable road roughness and presents a risk of clogging during operation. Counterfort drains was commonly used in the past because there were no other viable solutions at that time. In addition, formal consents are required for all of the aforementioned methods which would incur additional delay.

Deep soil mixing, not only addresses all the failure mechanism, but the speed of repair, minimal earthwork and minimum traffic disruption make it a cost effective alternative to other conventional methods. The fact that minimal sludge is generated during installation makes it an environmentally attractive alternative. Moreover, in almost all cases no consents are required.

Deep soil mixing columns are designed using numerical modelling with PLAXIS finite element software. Geotechnical parameters used in the design are derived from laboratory test results of the selected core samples collected during the site investigation. In some cases, geotechnical parameters are assessed from other site information based on the geology. An initial analysis is carried out to verify the geotechnical parameters used by simulating the actual slip failure in the model. Figure 4 shows a cross section of the slip and the geotechnical parameters used for the analysis.

Once the actual slope failure is successfully simulated, deep soil mixing columns are introduced into the model. The columns are modelled as soil using volume elements. Properties of the composite material (cement/lime soil mixture)

are derived from column strength test results combined with the geotechnical properties of the on-site material. Typically the unconfined compressive strength of a deep soil mixing column is about 1.5 MPa.

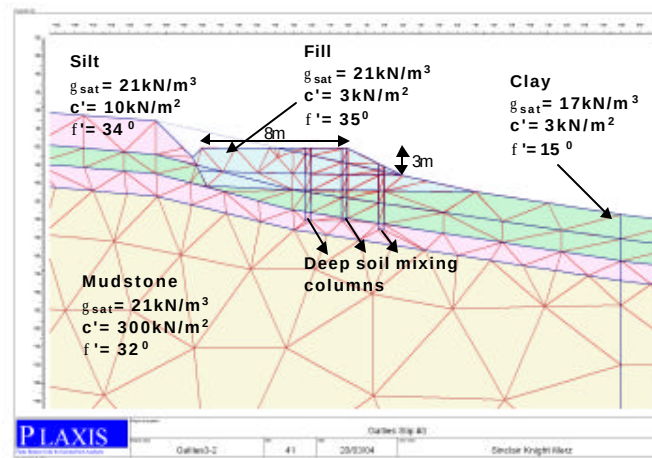


Figure 4. The cross section of slip (PLAXIS analysis model) and geotechnical parameters used on the analysis

The columns are positioned in such a way that a block of improved soil is created in the pavement and the slope below the road. This area is expected to remain stable even if further slip movement occurs below this area, so that the road structure will remain intact. The analyses are performed for normal groundwater conditions (expected groundwater during the winter months) and extreme events (full saturation of the slope and recharge-rapid drawdown analysis). The design factor of safety of 1.3 is required under the normal groundwater condition and 1.1 under the extreme events.

In this case, the analyses indicated that three rows of columns spaced at 2.5m were required to achieve satisfactory factors of safety under both groundwater conditions. The column length varies from 3.5m to 4.5m to maximise the columns' resistance to slope failure. The actual length of the columns are verified during construction. Figure 5 illustrates the simulated slip mechanism before and after the remediation under the extreme events. The calculated factors of safety are shown in Table 1.

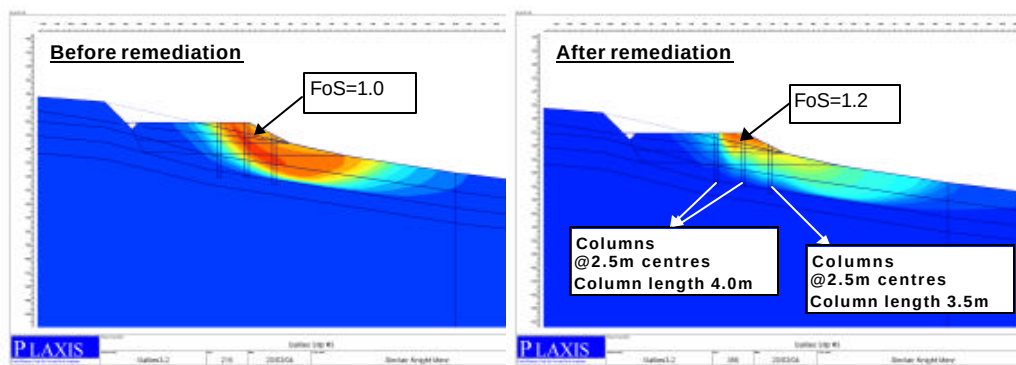


Figure 5. Slip mechanism and factor of safety before and after remediation – extreme groundwater conditions.

Table 1. The Factor of Safety of the Remedial Slope

Slope Structure	Normal Groundwater		Extreme Events	
Existing Slope	1.15		1.0	
	Achieved	Required	Achieved	Required
With Deep Soil Mixing columns @ 2.5 m	>1.5 , (1.3) ⁽¹⁾		1.2	1.1

Note: (1) FOS for slips down slope of the deep soil mixing columns, not involving the pavement. The FOS against the pavement failure exceeds 1.5.

The recommended column layout is shown on Figure 5 and Figure 6. The analysis also indicates that the columns are unlikely to fail by bending and shear under normal and extreme conditions.

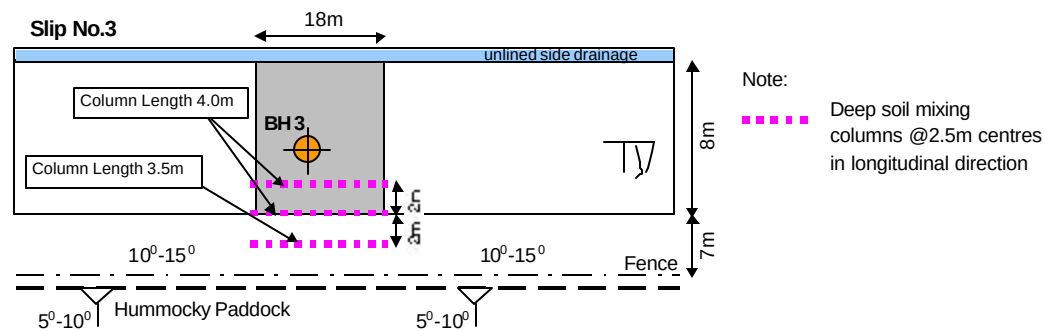


Figure 6. The Deep Soil Mixing column lay out

The factor of safety achieved is higher than requirement as shown on Table 1. The performance of deep soil mixing columns are regularly monitored on previously remedied sites. To date, no significant movement or instability has been reported.

Conclusion

Many roads in the Northland region have suffered from slope failure partly due to the Northland Allochthon that underlies this area. This material has unique characteristics due to the presence of many shear zones and sheared fabric. Numerous shallow seated failures within slopes of the Northland Allochthon have been observed on relative gentle slopes of 5° to 15° , which are attributed to the low residual strength and ongoing creep movement. The residual friction angle as low as 8° has been reported by Winkler [1].

Deep soil mixing technique has been successfully employed to address this problem. The factor of safety against slope failure has been increased significantly. However, it is noted that the effectiveness of this method is a function of column spacing and the type of material encountered on site.

Deep soil mixing has been opted due to its capability to address multiple slip mechanisms. The speed of repairs and minimal requirement for earthwork make it cost effective alternative. It is also environmentally friendly because minimal sludge is generated during construction and in almost all cases no consents are required.

The deep soil mixing remedial work is not expected to totally eliminate the ongoing creep movement up slope and down slope of the slip. However, the deep soil mixing remedial work, coupled with an effective drainage system, should mitigate the up slope and down slope creep movement to some degree.

Acknowledgment

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References:

1. Winkler, G.E., Geotechnical Engineering of the Northland Allochthon, Proc. of NZ Geotech. Society 2003 Symposium, 2003, 91-100.
2. Isaac, M.J., Herzer, R.H., Brook, F.J., Hayward, B. W., Cretaceous and Cenozoic Sedimentary Basins of Northland, New Zealand, Institute of Geological and Nuclear Sciences, 1994, 46-72.
3. Kear, D., Onerahi Chaos Breccia of Northland, N.Z. Journal of Geology and Geophysics Vol. 10, August 1967, 629-646.

Effect of Design Models and Test Numbers on the Design of Pad Foundations

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Abstract: It is widely known that there are significant uncertainties in the design of foundation systems. Specifically these uncertainties are categorised into model errors, uncertainties due to the spatial variation of soils and test and procedural effects. Geotechnical engineering professionals have aimed to minimise the magnitude of these uncertainties by adopting refined testing techniques, numerical design procedures and increased site investigation schemes. However the magnitude of the uncertainty still remains unknown. This paper introduces a method to quantify the effectiveness of undertaking site investigations of increasing scope as well as the use of varying testing techniques and foundation design models. The process involves simulating soil profiles using random field theory with nominated statistical parameters. This allows the soil properties to be known in detail at all locations – a situation that is not possible with real sites. Several design methods are employed using either complete knowledge of the soil profile or data derived from simulated site investigations. Designs are compared on a failure and over-design basis, where conclusions regarding the sensitivity of the design model and scope of site investigation are drawn. A Monte Carlo analysis is undertaken where numerous soil profiles are generated to develop probabilities of failure and over-design.

INTRODUCTION

A typical foundation design involves undertaking a site investigation of nominated scope and using the results from such an investigation to define mechanical soil properties. These properties are used in any one of a number of different design models, developed using differing theories, to estimate the settlement or bearing capacity of the soil and foundation. However, inherent uncertainties exist in this design procedure. Firstly, soils are inherently variable and consequently, site investigations do not always accurately characterise the actual soil conditions. This can severely affect the results of the estimated soil property and the resulting foundation design. There are also inherent uncertainties in the models used to estimate the response of a foundation system. This is a result of assumptions and simplifications in the theory used to develop the model and the model itself.

The framework of using random field theory [1] and a three dimensional finite element analysis to investigate the reliability of varying site investigation scopes has been introduced by Jaksa et al. [2]. The method involves simulating soil profiles based on random field theory, thus enabling all soil properties to be known at all locations in detail. A “traditional” design is undertaken using the results of a simulated site investigation. This is compared to an “optimal” design determined using the complete knowledge of the soil profile and a finite element analysis. Preliminary results have been published by Goldsworthy et al. [3] for a single pad footing. The research presented in this paper extends these results to investigate the reliability of various traditional design models using the results obtained from various site investigation schemes. This allows conclusions to be drawn about the effectiveness of the current available settlement models with respect to the magnitude of information gained from a site investigation scheme.

UNCERTAINTIES IN A FOUNDATION DESIGN

The typical stages involved in a foundation design procedure as identified by Bowles [4] & Holtz [5] are:

1. Establish the scope of the problem including the location of the site and loads;
2. Investigate the conditions at the site;
3. Design and undertake a subsurface exploration programme including in-situ and/or laboratory tests;
4. Determine relevant design soil parameters from the exploration programme in Step 3; and
5. Undertake the design of the foundation.

There are complex uncertainties in all of the above stages of a foundation design. Phoon & Kulhawy [6] identify the uncertainties inherent in the process of obtaining a soil design parameter as being due to:

1. Inherent soil variability;
2. Measurement errors; and
3. Transformation errors.

The uncertainties indicated by Phoon & Kulhawy [6], as shown in Figure 1, in addition to the uncertainty of the design model itself (Step 5), contribute to the overall uncertainty of the foundation design process.

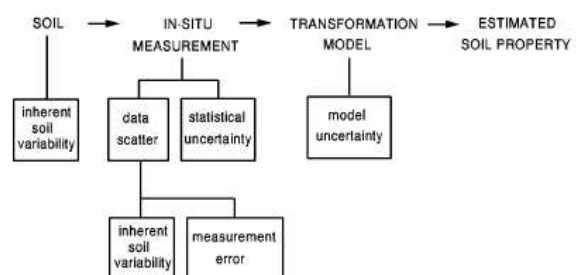


Figure 1. Uncertainty in soil property estimates [7].

Statistical methods have been used to quantify the uncertainty due to the inherent spatial variability of soils. The more common recent statistical methods include random field theory [1] and geostatistics [8], both of which involve modelling the spatial correlation of soil properties [9; 10]. In reality the spatial characteristics of a soil are rarely known for a foundation design and therefore, site exploration strategies are implemented to estimate them. The scope of these exploration strategies also affect the uncertainty of the process. Parsons & Frost [11], Azzouz & Bacconnet [12] and Subrahmanyam & Ng [13] have attempted to quantify the uncertainty due to limited site exploration strategies through the use of statistics and probability.

The contribution of measurement error to the overall uncertainty of a design process has been investigated by many authors [6; 14-18] and generally includes the effects due to sampling errors and testing errors. The transformation model uncertainties are sometimes difficult to distinguish from measurement errors, however Phoon & Kulhawy [19] and Ronold & Bjerager [20] have attempted to quantify these effects. The design model implemented also attributes to the overall uncertainty of the design process. This appears to be the least researched form of uncertainty and is one of the focuses of the research presented in this paper.

FOUNDATION DESIGN SCENARIO

A foundation system founded on six different soils based on the statistical parameters shown in Table 1 has been investigated. Analyses are restricted to serviceability criteria with a constant Poisson's ratio, requiring only the Young's Modulus, E , to be simulated by a random field. A foundation system of nine pad footings, evenly spaced and subject to column loads is modelled as shown in Figure 2.

Table 1. Statistical properties of soil profiles investigated

No	Young's Modulus (E)					ν
	Mean MPa	COV (%)	Scale of Fluctuation, θ			
			X	Y	Z	
20R	30	20	1	1	1	0.3
20M	30	20	4	4	2	0.3
20C	30	20	8	8	4	0.3
50R	30	50	1	1	1	0.3
50M	30	50	4	4	2	0.3
50C	30	50	8	8	4	0.3

The naming convention in Table 1 is based on the overall variability of the soil deposit (coefficient of variation of 20% or 50%) and the spatial variation of the field (random, medium or continuous). It is important to note the scale of fluctuation, which is a measure of the

distance within which soil properties are considered reasonably correlated [21].

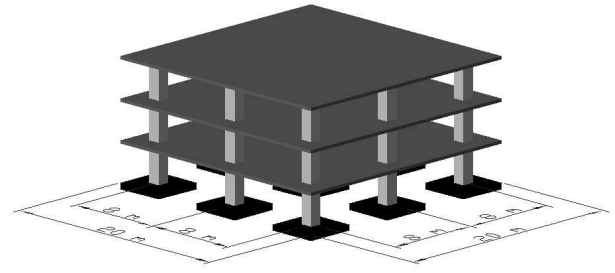


Figure 2. Layout of foundation system & design loads

The column loads applied to the nine rigid pad footings are indicative of a 20 metre $\times$ 20 metre, 3-storey building with a 3 kPa live load and a 5 kPa dead load.

FOUNDATION DESIGN TECHNIQUES

There are numerous foundation design models that are used to estimate the settlement of a foundation for design purposes. Many of these models relate the results of in-situ tests directly to the settlement of a footing. However, these models are notoriously uncertain [4] and models that are based on soil mechanics are generally preferred. The model proposed by Timoshenko and Goodier [22], is based on the theory of elasticity (Eq. 2). This model uses an averaged Young's Modulus, E_s and a uniform Poisson's Ratio, ν , with three influence factors I_1 , I_2 and I_f .

$$\rho = \frac{q d I (1 - \nu^2)}{E_s} \quad (2)$$

Schmertmann's [23] settlement relationship (Eq. 3) is recommended for use with granular soils, however Bowles [4] suggests it is suitable for most soils. The relationship uses two correction factors, C_1 and C_2 , which account for embedment and time effects, as well as an Influence factor, I_z , which idealises the strain profile into a triangle. Unlike the Timoshenko & Goodier method, Schmertmann's method integrates with depth, allowing effects from layering and varying Young's Modulus values.

$$\Delta H = C_1 C_2 \Delta p \sum_{i=1}^N \frac{\Delta z_i}{E_{s_i}} I_{z_i} \quad (3)$$

Numerous settlement models are based on linear elasticity (Eq. 4) theory. The models differ when determining the distribution of stresses throughout the soil mass. The "2:1 Method" was proposed by the US Army Corp of Engineers and involves scaling the magnitude of stresses in the soil at a ratio of 2:1. The resulting stress, q_v , at depth, z , is determined by Equation 5, where B is the width of the footing, L , is the length and Q is the applied load.

$$\Delta H = \sum_{H_i}^{H_{i+1}} \frac{\Delta q_i}{E_{si}} \quad (4)$$

$$q_v = \frac{Q}{(B+z)(L+z)} \quad (5)$$

The Bousinessq method develops a stress profile in the soil mass and Equation 5 is used to find the stress increment, q_v at depth z , for an applied load q_0 . The values, M , N , V and V_1 are all functions of the footing geometry, B & L and depth, z [4]. Alternatively, the Westergaard method [24] has the advantage of including Poisson's Ratio, accounting for varying lateral and vertical strains. The Westergaard equation is shown in Equation 6 and includes the a term given by Equation 7, incorporating Poisson's Ratio.

$$q_v = q_0 \frac{1}{4\pi} \left[\frac{2MN\sqrt{V}}{V+V_1} \frac{V+1}{V} + \tan^{-1} \left(\frac{2MN\sqrt{V}}{V-V_1} \right) \right] \quad (5)$$

$$q_v = \frac{q_0}{2\pi} \tan^{-1} \left(\frac{MN}{\sqrt{a}(M^2 + N^2 + a)^{1/2}} \right) \quad (6)$$

$$a = \frac{(1-2\nu)}{(2-2\nu)} \quad (7)$$

To obtain design values for use within the aforementioned design models, a simulated site investigation is undertaken. This site investigation consists of up to 8 test locations arranged as shown in Figure 3. It involves sampling from the random field representing the soil profile in a discrete manner. This is representative of a Standard Penetration Test (SPT) where soil properties are typically obtained at 1.5 metre depth intervals. It is important to note, at this stage, that no test uncertainty has been included in the results. Consequently, the results do not measure the effectiveness of the test method, rather the number and location of the test and the use of discrete sampling.

The dashed line in Figure 3 represents the boundary of the site investigation, while the solid line represents the site boundary. The naming convention used represents the scheme number, while the letters "RG" refer to a regular grid arrangement. The last number refers to the number of tests in the scheme. This convention is consistent throughout this paper.

It was concluded from non-compliance and over-design probabilities published by Goldsworthy et al. [3] that positioning a single test directly beneath the proposed footing location significantly influences the results. The test at the centre of the proposed footing location provides the best representation of the underlying soil conditions. However, few site investigations allow boreholes to be located at the centre of all of the proposed footing locations. Therefore, the test patterns

shown in Figure 3 are considered a more typical representation of most site investigation plans.

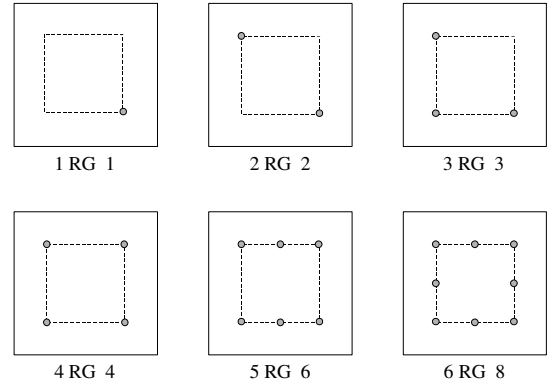


Figure 3. Site Investigation Schemes

Each design is compared with an optimal design achieved by utilising the completed knowledge of the simulated soil profile and a finite element analysis. The finite element analyses are three-dimensional and incorporate the stochastic variability of the soil properties.

EFFECTIVENESS OF SITE INVESTIGATION SCHEMES

Figures 4 and 5 show the probability of non-compliance and overdesign of the foundation system, designed with Schmertmann's settlement model for highly variable (COV=50%) soil profiles. Results from the soil profile with high COV are used exclusively for the remainder of the paper, as the uncertainty in the characterisation of the soil profile is proportional to the uncertainty in the properties of the soil profile. For the purposes of the research presented in this paper, non-compliance is defined as the design from the simulated site investigation being smaller than the design using complete knowledge of the soil profile or optimal design. Conversely, overdesign occurs when the design using site investigation data is greater than the optimal design. Future studies will investigate the use of the finite element analysis to analyse all designs to provide a better definition of non-compliance and overdesign.

Figure 4 shows an increasing trend of non-compliance probability for larger scope site investigations, however, the trend in Figure 5, suggests that the probability of overdesign reduces significantly with the increase in site investigation scope. This seemingly unusual trend in the probability of non-compliance is clarified in Figure 6, which presents the mean design area resulting from the Schmertmann model using the site investigation data and an optimal design using the complete knowledge of the soil profile. Figure 6 shows the mean design area reducing for increasing knowledge of the soil profile

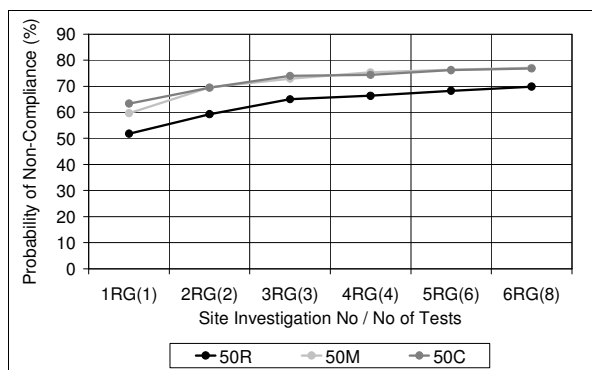


Figure 4. Probability of non-compliance for foundation system designed using *Schmertmann's* method

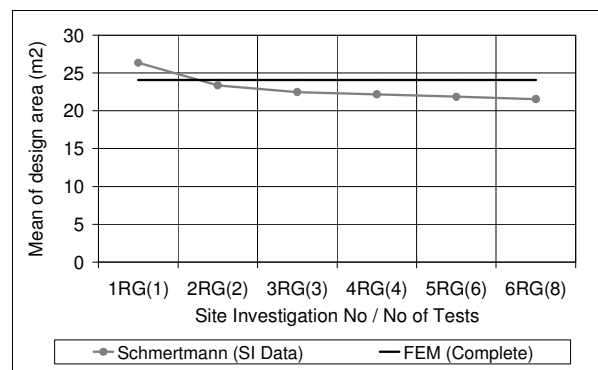


Figure 6. Mean of design area using *Schmertmann's* method and optimal design for soil profile 50C.

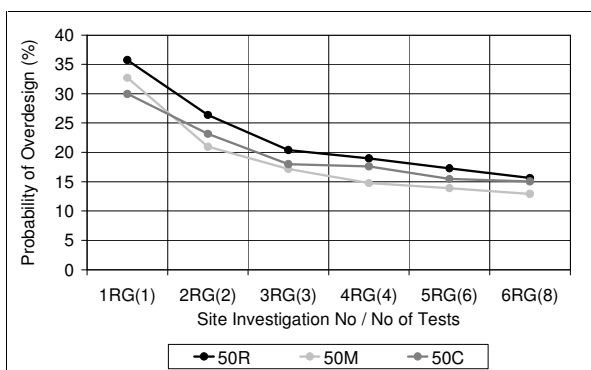


Figure 5. Probability of overdress for foundation system designed using *Schmertmann's* method

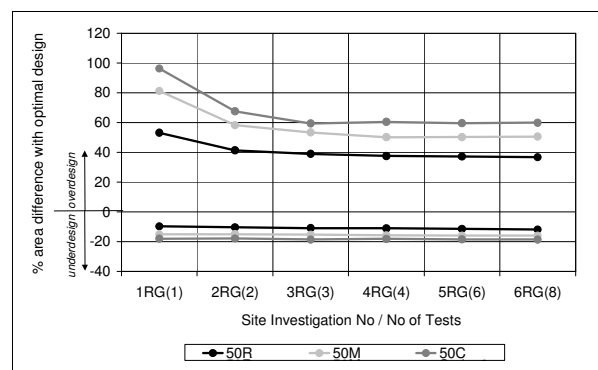


Figure 7. Design area comparison (underdesign & overdress) with optimal for *Schmertmann's* method

(larger site investigation scope), however for site investigation 6RG(8), where there is the greatest knowledge of the soil profile, there is a 6 m² difference in the mean design area. This represents the error associated with the Schmertmann model for this load and soil profile scenario. Hence, the Schmertmann model provides a less conservative design method where the resulting foundation design is less than the optimal design, leading towards non-compliance, as defined in this paper.

Figure 7 compares the error in the design area when non-compliance (underdesign) and overdress occurs for the Schmertmann model compared with the optimal design. Although Figure 6 indicates that the Schmertmann model is under-conservative and tends to underdesign foundations for this soil profile and design loading scenario, it appears from Figure 7, that when an underdesign occurs its magnitude is not large. Conversely, when overdress occurs there appears to be a large difference in the design areas. Figure 7 also shows a strong trend indicating that an increased site investigation scope will approach the optimal design as the difference between the underdesign and overdress lines narrow for increasing site investigation scope.

EFFECTIVENESS OF DESIGN MODELS

For each of the design methods, a reliability analysis was undertaken. This analysis involves investigating the differences in the resulting foundation design with the optimal design derived from complete knowledge of the soil profile. Figure 8 presents the mean design area of the foundation system for each design model and site investigation plan. The dashed line in Figure 8 indicates the mean design area resulting from the optimal design incorporating the complete knowledge of the soil profile. It is apparent from Figure 8, that all of the design models are less conservative showing smaller mean design areas than the optimal design for most of the site investigation schemes. This conforms with the conclusions drawn in the previous section. However, it is apparent from Figure 8, that the Schmertmann model and the 2:1 technique are more conservative than the other design models.

Figure 9 shows the uncertainty in footing design area for each model and site investigation plan. As knowledge about the soil profile increases (larger site investigation scope), the uncertainty in the design area is reduced. The trend for each of the design models appears to be the same, indicating this is a function of the soil profile and investigation strategy, rather than the model.

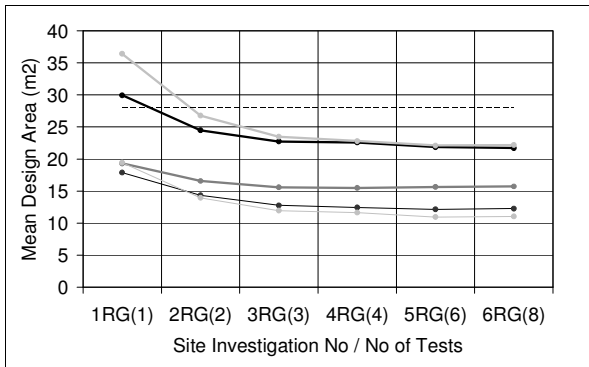


Figure 8. Mean of design areas for all foundation design models for soil profile 50C.

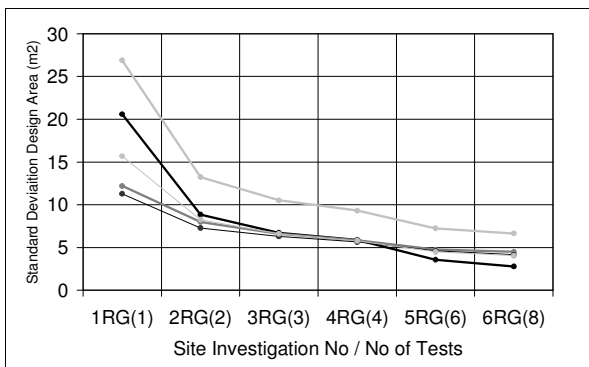


Figure 9. Standard deviation of design areas for all foundation design models for soil profile 50C.

Figure 10 provides the best comparison of design models, where the design area is compared with the optimal design area as a percentage error. The distance between the overdesign and underdesign line of each design method can be viewed as the uncertainty of the model, where a large distance represents large model uncertainty and a small distance indicates small model uncertainty. The model bias is measured by the distance the mean of the overdesign and undersign lines stray from zero. This is also a measure of the conservatism of each model.

Using these measures, it is apparent from Figure 10 that the Schmertmann and 2:1 models are very similar in both uncertainty and bias, while the other models (Bousinessq, Westergaard and Timoshenko) show similar behaviour. Figure 10 also further clarifies that the Schmertmann and 2:1 models are more conservative than the other models leading towards overdesigns of greater error than the other models. It is also concluded from Figure 10 that the Schmertmann model needs less information from a site investigation scheme to provide the best design. This is shown by the overdesign and underdesign lines reaching an asymptote faster than the other models.

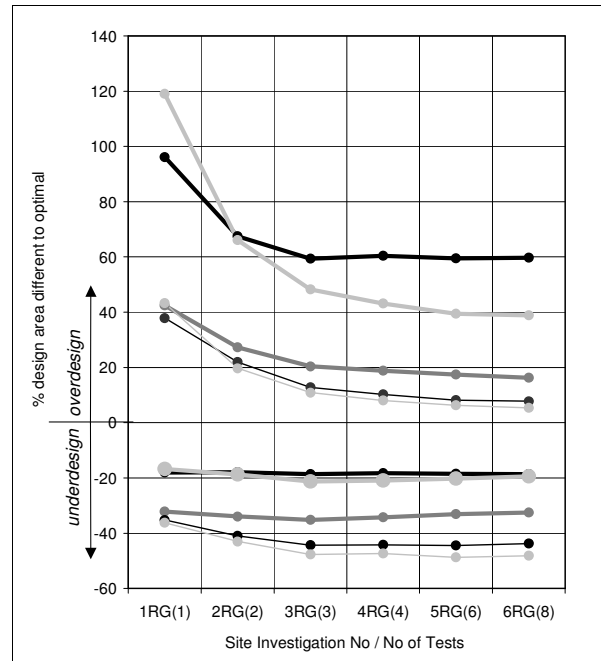
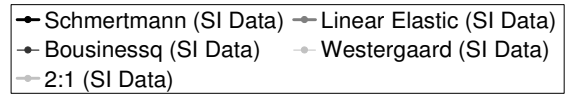


Figure 10. Design area comparisons (under-design and over-design) with optimal design for all design models for soil profile 50C.



CONCLUSIONS

The uncertainties in a foundation design due to a limited site investigation and the use of idealised and simplified design models have been quantified. It has been shown that an increased knowledge about the site from a larger scope site investigation will not only reduce the uncertainty associated with a foundation design, but will also reduce the possibility and magnitude of overdesign. The results also indicate that, although a greater knowledge of the soil profile reduces the probability of a foundation overdesign, it increases the probability of obtaining a design which does not meet all the design criteria (non-compliance). This is a result of the design approaching the optimal design and reducing the magnitude of overdesign. It has been shown that common design models have varying inherent conservatism, uncertainty and bias. It was also apparent that different design models approach their optimal design using different degrees of site knowledge. It is hoped that the results presented in this paper will assist geotechnical professionals to select site investigation plans and design models to suit the site conditions and design requirements.

ACKNOWLEDGMENTS

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REFERENCES

1. VANMARCKE, E.H., "Random Fields: Analysis and Synthesis", 1984, The MIT Press, Cambridge, Massachusettes, USA.
2. JAKSA, M.B., KAGGWA, W.S., FENTON, G.A., and POULOS, H.G., "A framework for quantifying the reliability of geotechnical investigations", 2003, 9th International Conference on the Application of Statistics and Probability in Civil Engineering, 1285-1291.
3. GOLDSWORTHY, J.S., JAKSA, M.B., FENTON, G.A., KAGGWA, W.S., GRIFFITHS, D.V., POULOS, H.G., and KUO, Y.L., "Influence of Site Investigations on the Design of Pad Footings", 2004, 9th Australia New Zealand Conference on Geomechanics, 282-288.
4. BOWLES, J.E., "Foundation Analysis and Design", 1997, McGraw-Hill, Singapore.
5. HOLTZ, R.D., "Stress Distribution and Settlement of Shallow Foundations", 1991, Foundation Engineering Handbook, Fang, H.Y., ed., Van Nostrand Reinhold, New York, USA, 923.
6. PHOON, K.K., and KULHAWY, F.H., "Characterization of geotechnical variability," 1999, Canadian Geotechnical Journal, 36(4), 612-624.
7. KULHAWY, F.H., and PHOON, K.K., "Observations on Geotechnical Reliability-Based Design Development in North America", 2002, International Workshop on Foundation Design Codes and Soil Investigations in view of International Harmonisation and Performance Based Designs, 31-48.
8. JOURNEL, A.G., and HUIJBREGTS, C.H., "Mining Geostatistics", 1978, Academic Press Inc, New York, USA.
9. FENTON, G.A., and VANMARCKE, E.H., "Simulation of Random Fields via Local Average Subdivision," 1990, Journal of Engineering Mechanics, 116(8), 1733-1749.
10. FENTON, G.A., "Simulation and Analysis of Random Fields", 1990, PhD, Princeton University, New Jersey, USA.
11. PARSONS, R.L., and FROST, J.D., "Evaluating Site Investigation Quality using GIS and Geostatistics," 2002, Journal of Geotechnical and Geoenvironmental Engineering, 128(6), 451-461.
12. AZZOUZ, R., and BACCONNET, C., "Optimal approach to the planning of a network of in situ soil test", 1991, 6th International Conference on Applications of Statistics and Probability in Civil Engineering (ICASP6), 652-659.
13. SUBRAHMANYAM, M.S., and NG, I.T., "Probability-based design of soil exploration programme", 2000, 8th International Conference on Applications of Statistics and Probability in Civil Engineering (ICASP8), 1109-1114.
14. SPRY, M.J., KULHAWY, F.H., and GRIGORIU, M.D., "A Probability Based Geotechnical Site Characterisation Strategy for Transmission Line Structures", 1988, EL-5507, Cornell University, Ithaca, New York, USA.
15. ORCHANT, C.J., TRAUTMANN, C.H., and KULHAWY, F.H., "In-Situ Testing to Characterize Electric Transmission Line Routes", 1986, In Situ '86 - Use of In Situ Tests in Geotechnical Engineering (GSP 6), 869-886.
16. LEE, I.K., WHITE, W., and INGLES, O.G., "Geotechnical Engineering", 1983, Pittman Publishing Inc, Massachusetts, USA.
17. ORCHANT, C.J., KULHAWY, F.H., and TRAUTMANN, C.H., "Critical Evaluation of In-Situ Test Methods and their Variability", 1988, EL-5507, Vol. 2, Electric Power Research Institute, Palo Alto.
18. KAY, J.N., KULHAWY, F.H., and GRIGORIU, M.D., "Assessment of Uncertainties in Geotechnical Design", 1991, 6th International Conference on Applications of Statistics and Probability in Civil Engineering (ICASP6), 683-692.
19. PHOON, K.K., and KULHAWY, F.H., "Evaluation of geotechnical property variability," 1999, Canadian Geotechnical Journal, 36(4), 625-639.
20. RONOLD, K.O., and BJERAGER, P., "Model Uncertainty Representation in Geotechnical Reliability Analyses," 1992, Journal of Geotechnical Engineering, 118(2), 363-376.
21. VANMARCKE, E.H., "Probabilistic Modelling of Soil Profiles," 1977, Journal of the Geotechnical Engineering Division, 103(GT11), 1227-1246.
22. TIMOSHENKO, S., and GOODIER, J.N., "Theory of Elasticity", 1951, McGraw-Hill, New York, USA.
23. SCHMERTMANN, J.H., "Guidelines for Cone Penetration Test: Performance and Design", 1978, FHWA-TS-78-209, US Dept of Transportation, Federal Highway Administration, Washington, USA.
24. WESTERGAARD, H.M., "A Problem of Elasticity Suggested by a Problem in Soil Mechanics: Soft Material Reinforced by Numerous Strong Horizontal Sheets", 1938, Contributions to the Mechanics of Solids - Stephen Timoshenko 60th Anniversary Volume, Macmillan, New York, USA.

Foundation Treatment for the Port River Expressway

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Stage 1 of the Port River Expressway is currently under construction, at the north western suburban fringe of Adelaide. These works include approximately 4 km of new road, and three grade-separated intersections incorporating embankments up to about 10 m high. The site for the Expressway is a Holocene estuarine plain that was partly covered by mangrove swamp, and comprises normally consolidated sand and clay. Areas of uncontrolled fill, including municipal waste are also present beneath the road alignment to depths of 4 m in places.

The project challenges have required the use of a variety of ground engineering solutions, including staged embankment construction with preloading, in-situ treatment with an impact roller, stone column installation, and conventional methods of excavation and replacement. A case study of the geotechnical solutions adopted and a comparison between the predicted and actual performance of the various parts of the earthworks is presented, including an assessment of the improvement of soil stiffness by the installation of stone columns.

INTRODUCTION

Stage 1 of the Port River Expressway (PREx) is currently under construction, with Stages 2 and 3 at pre-tender. The overall project will deliver a dual carriageway expressway linking National Highway 1 (Port Wakefield Road) to the industrial areas and container terminals around Outer Harbor. The expressway will bypass the historic heart of Port Adelaide.

The Stage 1 works extend from Salisbury Highway in the east to Francis St in the west. A location plan is shown on Figure 1.

Most of the alignment is low lying and comprises several metres of soft soils. Uncontrolled dumping of various materials (including municipal waste) has raised the existing ground levels in some sections of the alignment.

Grade separated intersections are being constructed where the PREx intersects South Road, Hanson Road and Eastern Parade. Embankment heights at the intersections will vary up to about 10 m above existing ground levels.

Along the general road alignment, the finished pavement level will be slightly above RL 3 m AHD, requiring placement of up to about 2.5 m of fill above existing ground levels.

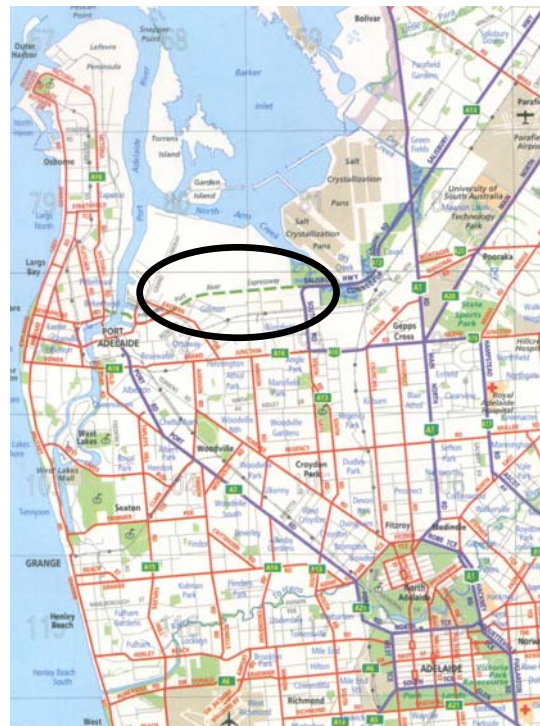


FIGURE 1: Location plan of Stage 1 of the Port River Expressway (Reference UBD, Universal Press Pty Ltd, 2003)

The Stage 1 works have been awarded under a Design and Construct contract, where the Contractor is responsible for maintaining the expressway for 10 years following practical completion. At the end of the 10 year period, the road must meet specified performance criteria when handed over to the State Government. The performance requirements include not more than 12 mm differential settlement over a 20 m length.

GROUND CONDITIONS

Prior to European development, the whole area for the Stage 1 works was covered by a Holocene estuarine plain, including mangrove swamps. Permanent groundwater is present at very shallow depths across the site, and lies almost at the surface in some areas (particularly during wetter months).

The engineering properties of the site are dominated by the Holocene estuarine deposits, known as the St Kilda Formation. These deposits typically comprise materials ranging from loose sand with shells, peat and soft to firm high plasticity clay. The soils typically have a high moisture content, and the organic, fine grained mangrove deposits are known to be highly compressible.

Underlying the St Kilda Formation, various different Pleistocene deposits are present. The Pleistocene deposits typically comprise very stiff (over consolidated) clay with some medium dense sand layers. The younger Pleistocene deposits comprise Pooraka and Glanville Formation soils at different parts of the alignment. The Hindmarsh Clay dominates the older Pleistocene deposits, and is expected to extend to a depth of about 90 m beneath the site.

FOUNDATION PREPARATION

General Alignment

Along the majority of the alignment, where the ground levels are typically being raised by not more than about 2.5 m, the foundation preparation comprised one of the following techniques:

- 1) Where little or no fill was present, and the ground was generally low-lying, the site preparation comprised removing the vegetation (but leaving the topsoil in-situ) and placing a “working platform”. The working platform typically comprised coarse, crushed recycled concrete (maximum nominal size 300 mm) up to about 1 m thick. The working platform was track-rolled during placement and proof rolled using a loaded articulated dump truck. Once a stable platform for construction traffic had been achieved, the embankment was constructed in a conventional fashion. Extensive use of re-cycled materials and blended site materials were used in the embankment construction.
- 2) Where municipal waste or deleterious fill materials were present, these materials were removed down to the natural soils. This involved excavations over large areas to depths of about 2 m below the groundwater level. The excavations were reinstated using coarse recycled concrete (similar to the working platform described above).
- 3) Where existing fill was present and it contained suitable material (no deleterious materials or high plasticity clay) the fill was largely left in place. The surface was stripped to a level corresponding to the top of the working platform, and rolled using an impact roller (square roller) to provide some compactive energy throughout the full thickness of the fill. Proof rolling was then performed with a loaded articulated dump truck. Any soft or weak areas were removed and reinstated with coarse recycled concrete. Any fill that had been stripped from these areas was generally re-used (and blended if necessary) in the general embankment construction.

In some areas where the existing fill had been left in place, and proof rolling indicated some minor surface deflections, plate load testing was used to better characterise the subgrade stiffness. In areas with marginally acceptable working platform stiffness, a biaxial geogrid was placed at the top of working platform level to improve the stiffness of the subsequent embankment layers.

All sections of the general Expressway embankment have been subject to preloading, prior to pavement construction. Design preload heights above the finished pavement level vary from 0.6 m to 2.5 m. The predicted preload duration varies from 1 month to 3 months. In all cases a 90 day “waiting period” has been adopted between removal of the preload and pavement construction, to reduce the anticipated magnitude of secondary consolidation of the pavement over the 10 year maintenance period to within the deflection requirements.

Eastern Parade Overpass

The Eastern Parade Overpass is a multiple span concrete bridge carrying the Expressway over the road corridor of Eastern Parade and a rail corridor. An elevation of the structure is shown by Figure 2.

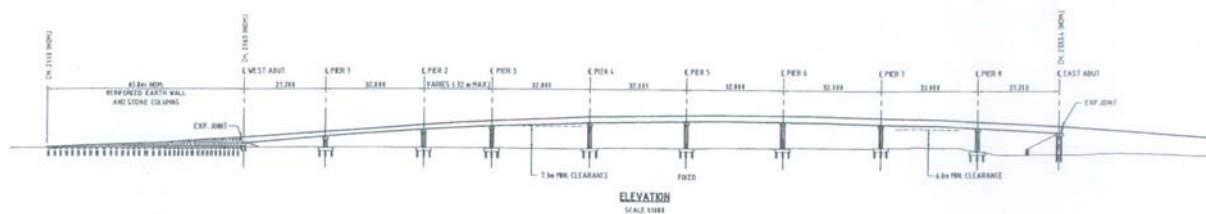


FIGURE 2: Elevation of the Eastern Parade Overpass (looking north)

Piled footings were required for the bridge piers. The abutments were also piled to limit differential deflection of the bridge deck. The piling system adopted was a driven cast-in-situ bulb base concrete pile.

Eastern Approach Embankment

The eastern approach embankment has a finished pavement level about 9 m above original ground levels. This embankment is underlain by typically 2 m to 2.5 m of the soft St Kilda Formation soils. A calcrete crust marking the upper surface of the Pleistocene Glanville Formation is present beneath the soft soils.

Space allows this embankment to be largely constructed as a conventional battered embankment.

The foundation preparation at the eastern approach embankment comprised construction of a geogrid-reinforced granular mattress 2 m thick.

Conventional embankment construction proceeded over the reinforced granular mattress, however, the rate of embankment construction was limited to a maximum of 1 m height each week to allow some drainage of excess pore water pressures from the St Kilda Formation soils. Piezometers installed at different depths within the foundation soils were monitored to ensure that pore water pressures remained at acceptable levels.

Once the embankment reached full height, an additional preload was placed at the top of the embankment and settlements were monitored to ensure that all primary consolidation settlements were complete prior to construction of the pavement. The rate of secondary consolidation was also observed during the preload period.

Western Approach Embankment

The western approach embankment was constructed in the middle of Francis Street, which was maintained as an operational public road throughout the overpass construction. Space restrictions required that this embankment be constructed with vertical sides. Reinforced earth retaining walls were selected to retain the embankment.

The western embankment was underlain by the Francis Street pavement and fill to a depth of about 2 m, and soft St Kilda Formation soils to a depth of about 6 m. In order to keep the imposed stresses at the base of the reinforced earth walls to relatively low levels, the embankment height was limited to about 3 m above the existing Francis Street pavement level.

Preloading of the embankment could not be performed after completion of the embankment construction as large settlements would adversely affect the reinforced earth walls. As such, the foundation preparation for the eastern approach embankment comprised installing a grid of stone columns, with a biaxial geogrid-reinforced granular mattress over the top of the stone columns. The stone columns were installed on a triangular grid, with spacings varying from 1.5 m centres to 2.5 m centres. The layout of the stone columns beneath the eastern embankment is shown by Figure 3. The columns were 0.8 m diameter, and extended to the top of the Glanville Formation, at a depth of about 6 m below the surface. The surface was then preloaded with a temporary preload embankment, and monitored for settlements. Following the preload period, the preload was removed and conventional construction of the reinforced earth walls and embankment proceeded.

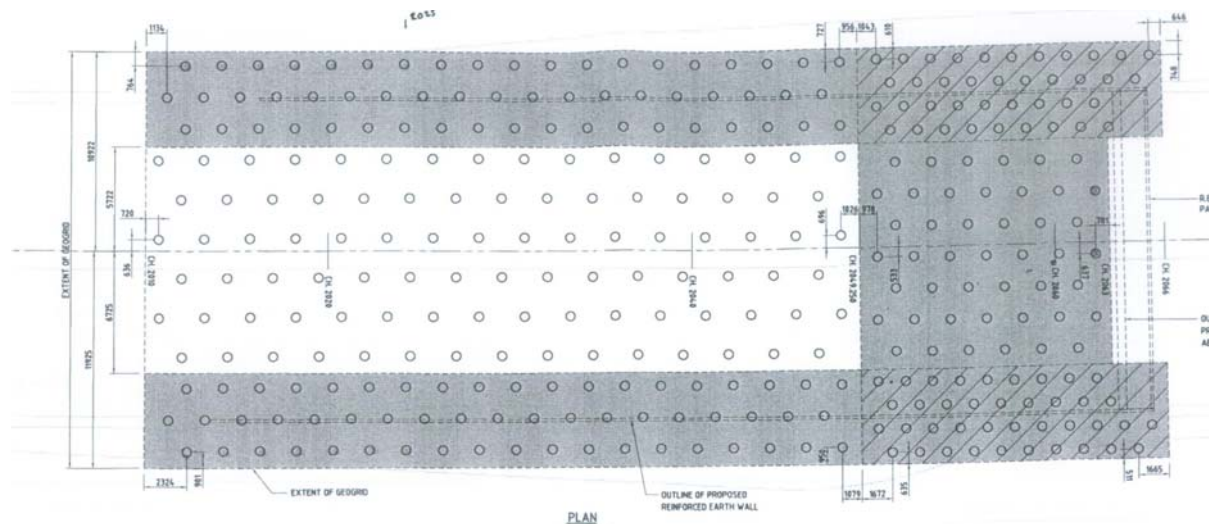


TABLE 1: MEASURED AND PREDICTED SETTLEMENTS

Element	Primary Settlement (mm)		Secondary Settlement (mm per log cycle of time in days)		Imposed Surcharge (kPa)	St Kilda Thickness (m)
	Predicted	Measured	Predicted	Measured		
General Embankment: Sections A, B and C (up to 2.5m high)	varies up to 70	varies up to about 70	varies up to 15	varies up to about 10	Up to about 50	Up to about 2
Eastern Parade: Eastern Approach Embankment	275	Approx 280	10 to 12.5	15 to 20	About 200	About 2.5
Eastern Parade: Western Approach Embankment (stone columns)	125	Approx 80	15	10 to 15	About 80	About 4
South Road and Hanson Road Embankments	<i>Still under construction at time of writing (no data)</i>					

Primary consolidation settlements occurred fairly rapidly, and were generally complete within about 1 month.

The secondary consolidation settlements presented in Table 1 have generally been recorded over the duration of the preload period only, which in most cases is less than 100 days.

Based on the data presented in Table 1, (and assuming that the consolidation settlements are due to compression within the St Kilda Formation) a summary of the compressibility of the St Kilda Formation is presented in Table 2.

TABLE 2: COMPRESSIBILITY OF ST KILDA FORMATION

Element	Primary Settlement ÷ Surcharge ÷ Thickness of St Kilda (mm/kPa/m)
General Embankment Sections A, B and C (up to 2.5m high)	Up to about 0.7
Eastern Parade – Eastern Approach Embankment	0.56
Eastern Parade – Western Approach Embankment (stone columns)	0.25

From the data presented in Table 2, an improvement factor of about 2.2 was achieved for the soil stiffness from the stone column installation. Relationships derived by Priebe (reference 1) suggest a design improvement factor of about 1.7 for the stone columns at 2 m centres.

CONCLUSIONS

A variety of alternative foundation treatments have been adopted during construction of Stage 1 of the Port River Expressway. The measured performance of the earthworks has largely been consistent with expectations, although the secondary consolidation characteristics of the estuarine soils at the Eastern Parade overpass appear higher than elsewhere along the alignment.

A grid of stone columns was installed under the eastern approach embankment to the Eastern Parade overpass. Based on the measured primary consolidation settlements of the embankment over the stone columns, the magnitude of ground improvement achieved by the stone columns appears to be greater than predicted by current design methods. From the limited data available during secondary consolidation, it is not clear whether the same magnitude of improvement has been achieved for the rate of secondary consolidation.

ACKNOWLEDGEMENTS

The author wishes to thank Transport SA, Bardavcol Pty Ltd and York Civil Pty Ltd for their permission to publish this paper.

The input and assistance offered by Mr Roger Grounds (Coffey's State Manager) during the course of the project and the preparation of this paper is also gratefully acknowledged.

REFERENCES

1. Priebe, H.J, "The design of vibro replacement", Ground Engineering, December 1995, Pages 31 to 37.

Gabion Baskets: An Effective Means of Erosion Protection

Melanie Henry, Tonkin & Taylor Ltd

Takarau Gorge Road and Makara Road, located to the west of Wellington New Zealand are subject to regular erosion damage from adjoining streams.

The geotechnical hazards present along these roads include slips both above and below the road, degradation of the streambed, erosion of the stream banks and loss of fine materials from beneath the carriageway. A variety of retaining and erosion protection structures have previously been built to protect the carriageway. Following a 100-year return period storm event in February 2004 failures of slopes and retaining structures occurred.

Gabion basket walls have become a preferred option for carriageway protection as they are flexible and, if adequately designed and detailed, have the capacity to withstand differential settlements and scour.

1. INTRODUCTION

Wellington City Council is the roading authority responsible for the upgrade and maintenance of the roading network in the Wellington area. This includes two rural roads which follow the alignment of streams and are at constant risk of erosion. Due to the low traffic volumes of these roads major realignments or constraining of the streams are difficult to justify. Retaining structures and erosion protection works are used to protect the road at critical locations.

The two roads are Takarau Gorge Road and Makara Road, located to the west of Wellington city as shown in Figure 1. They adjoin the Ohariu and Makara streams respectively. Both roads, one in a gorge and one in a valley, are susceptible to damage after heavy storms. In February 2004 a 100-year return period storm occurred.

This storm caused major flooding and damage in the area. The damage occurred to many unprotected banks both above and below the road and to existing retaining and erosion protection structures.

These failures and ways to prevent them are discussed.

The merit of using gabion basket walls to protect roadways is also presented.

2. ROAD SETTING

The Ohariu and Makara streams are approximately 2m wide and 0.3m deep in normal flow conditions. The water levels can rise 2m or more during flood events. The adjoining road surface levels are 1m to 3m above the normal flow water level.

Takarau Gorge Road provides access to farming and forestry blocks. The narrow road winds through a steep sided gorge adjacent to the Ohariu Stream. The road has typically been formed as a cut to fill road. The stream is constrained by the narrow gorge and continues to degrade. The road is susceptible to erosion by undercutting and over steepening of the banks.

The Ohariu Stream joins the Makara Stream where it exits Takarau Gorge. Makara Road follows a relatively straight alignment to the small coastal settlement of Makara Beach. In the wider valley the stream is unconstrained and meanders across the valley floor. The road is at risk of erosion where the stream adjoins and cuts into the banks on the outside of bends.



Figure 1 Map showing Takarau Gorge Road and Makara Road (TerraLink Ltd ⁽¹⁾)

3. GEOTECHNICAL HAZARDS

Takarau Gorge and Makara roads are susceptible to a number of geotechnical hazards. These are

- High cut slopes above the road
- Bank erosion
- Flood damage to the carriageway
- Internal erosion of fine material

Above the roadway there are cut slopes up to 10m high. The cuts are in rock, residual soils and colluvium and can have slopes of up to 60°. These often become unstable during heavy rainfall and slip debris can block the road. Large trees with shallow root systems on steep banks can become unstable during heavy rainfall events. Fallen trees can block the carriageway and also trigger larger slips.

Below the roadway unprotected stream banks are often eroded during flood events due to the flow patterns of the respective streams. The erosion leads to localised slipping and the loss of sections of carriageway. Photograph 1 shows a slip below the road that has been caused by the undercutting of the bank.



Photograph 1 Slip below road caused by undercutting of stream bank.

In Takarau Gorge the flood waters can flow over the road and cause localised scour, damage to unsealed shoulders and deposit flood debris.

Internal erosion can occur due to the migration of fine material with floodwaters. This can eventually cause severe damage to the pavement as illustrated in Photograph 2.

4. ROAD PROTECTION WORKS

Structures built to protect Takarau Gorge Road and Makara Road have usually been constructed as a reaction to damage caused by floods and slips.



Photograph 2 Damage to pavement due to the migration of fine material.

Large slips above the carriageway are not cost effective to prevent, however the carriageway can be protected from further damage by scaling works or promoting vegetation growth. Large slips that had occurred prior to the February 2004 storm and were remedied in this way showed no further signs of slipping during the storm event.

In both stream environments road protection structures are required that can withstand heavy traffic loads due to logging and stock trucks and allow for the other hazards described. The walls also need to be cost effective, relatively easy to construct in a streambed and be sympathetic to the surrounding environment.

The types of existing structure protecting the road from stream damage include:

- Rock revetments
- Pole walls
- Gabion basket walls

4.1 Rock Revetments

Revetments are a rock armour type protection. They consist of rocks stacked against the bank. The failure mechanisms observed in existing revetments along Takarau Gorge Road and Makara Road include the displacement of inadequately sized rock armour, localised scour at the end of revetments where there is a sharp transition between the natural bank and the revetment, and the loss of fine materials from behind the wall.

The advantage of revetments is that they are easy to construct below water level and are cost effective. They also have a natural look as they are made from natural materials and suit the rural environment.

Based on the observed damage the design of rock revetments need to consider the following:

- Armour rock large enough to resist movement
- Layers of filter rocks or a geotextile filter fabric to limit the movement of fines.
- A smooth transition from the natural bank to the rock structure to reduce scour at the ends of the structure.

4.2 Pole walls

Pole walls consist of posts, for example timber, railway iron or concrete, with timber lagging. The failure mechanisms observed in existing poles walls following storm events include overturning failures where the poles had been inadequately embedded into the underlying rock and undermining of the wall.

Photograph 3 shows a railway iron wall where localised scour has occurred beneath the wall. This has caused the backfill material to slump forming a hole. Localised scour can also occur at the end of the wall.

Based on the observed damage the design of pole walls need to consider the following:

- Potential scour depth to minimise the risk of undermining.
- Adequate founding depths for the poles.
- A geotextile filter fabric to limit the movement of fines.



Photograph 3 Undermined railway iron wall with loss of backfill material.

4.3 Gabion Baskets

Gabion baskets are usually 1m by 1m by 2m wire mesh baskets laced together and filled with very coarse gravel and cobbles. Gabion baskets are very flexible and can accommodate settlement of soft founding materials and erosion. To further protect

against erosion they can be constructed on reno mattresses. Reno mattresses are approximately 0.25m thick and constructed in the same way as the baskets. They extend under the wall and at least 1m in front. If the toe is eroded the reno mattress is flexible enough to form to the new bed contours and resist further erosion.

Despite this flexibility damage can occur to gabion walls during flood events. Photograph 4 shows a gabion wall that has been inadequately constructed. The wall was originally founded at streambed level. The stream has degraded, undermining a section of the wall and causing the wall to rotate. The design of gabion walls is discussed in the following section.



Photograph 4 Gabion wall where the middle section has been undermined and rotated.

5. DESIGN OF GABION WALLS

Gabion walls have become a preferable option for support of the Takarau Gorge and Makara Road carriageways adjacent to the Ohariu and Makara streams. These walls are cost effective and can be designed to accommodate significant settlement and scour. However the walls need to be designed for stability and correctly detailed so that they are not damaged during significant flood events.

5.1 Wall Stability

Gabion walls must be designed for stability in the same way as any gravity type retaining structure. This involves designing the wall to resist:

- Overturning
- Bearing
- Sliding
- Global stability

All design loads need to be considered. In this case large logging and stock truck loads need to be applied.

In gravity retaining structures the weight of the wall resists active soil and applied pressures. The walls along Takarau Gorge and Makara Roads can be up to four baskets high (4m). Often the desired carriageway width and the close proximity of the stream edge require the gabions to be steeply stacked.

To increase overturning stability a second layer of gabions can be incorporated into the design or a grid reinforcement system can be used in the backfill to increase the stability of the wall.

The wall should be founded in dense materials of adequate bearing capacity. A subsurface investigation will determine the depth of soft sediments below stream level. This allows the wall to be founded on dense materials and lessens the likelihood of differential settlements. This often results in the wall being founded below the current stream level. The flexible nature of the baskets means that they can withstand some differential movement and still remain fully functional.

Frictional and cohesive resistance along the base of the wall must be greater than the applied forces to resist sliding. A grid reinforcement system can also be used at the foundation level to increase sliding resistance.

The global stability of the wall design must be checked to ensure that the wall has a sufficient factor of safety against circular slip failures extending below the toe of the wall.

5.2 Detailing of Gabion Walls

In a stream the wall must also be detailed to resist:

- Scour beneath and at the ends of the wall
- Migration of fines from behind the wall
- Corrosion

There are two types of scour that must be allowed for in the design of gabion basket walls, the general scour caused by flow patterns in the stream and localised scour caused by the structure.

A hydrological assessment is required to assess flood levels and scour depths. In the case of Ohariu and Makara streams a scour depth of typically 1m has been assessed for design. To date gabion basket walls that have been embedded 1m below the

existing streambed level, or 0.5m below streambed level where the toe of the wall has been protected by a reno mattress, have been found to be adequate in large flood events.

Abrupt changes at the upstream and downstream ends of the wall can generate areas of localised turbulence and scour. A survey is often needed to determine the alignment of the new wall. This allows the curvature of the stream and the desired carriageway width to be incorporated into the design. Scour at the ends of the walls can be minimised by angling the end gabions back into the bank to create a smoother transition.

Filter fabrics are important to limit the migration of fines from the material behind the structure.

Gabion walls in aggressive environments such as the coastal location of Makara Road need to be protected from corrosion. The wire used to manufacture the baskets can be galvanised and coated with PVC.

6. CONCLUSIONS

Takarau Gorge and Makara roads are subject to damage from stream erosion.

Following a major storm event failures of slopes and retaining structures occurred. These failures illustrate the importance of design and detailing in a stream environment.

Gabion basket walls have become the preferred solution to protect the carriageway from erosion. This is due to their flexibility and ability to accommodate settlement. If reno mattresses are also used the wall is further protected from scour of the toe. Good design practices should include a detailed investigation of the ground conditions, a stability assessment of the wall and detailing to allow for the stream environment.

7. ACKNOWLEDGEMENTS

The author would like to acknowledge Mr John Forde at Wellington City Council for permission to publish this paper. Mr Stuart Palmer at Tonkin & Taylor is thanked for reviewing this paper and providing comment.

8. REFERENCES

1. TerraLink Ltd, "MapWorld TopoMap NZ V2.0.61", 2000.

Acid Rock Management Challenges

Reedy Creek Quarry

Gold Coast

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Abstract:

The purpose of this paper is to outline the geo-environmental challenges associated with the management of acid rock encountered at the Reedy Creek Quarry and key outcomes of a recently completed Acid Rock Management Plan. Rainfall runoff collected onsite is typically around pH 2.7-3.0 which is substantially below the quarry's licensed discharge limit, and has the potential to adversely impact on the downstream receiving environment. Acid drainage is due to rainfall interaction with acid rock containing pyrite, which is prevalent in localised sections of the quarry. Council commissioned consultants to undertake a geo-environmental investigation and to produce an Acid Rock Management Plan.

INTRODUCTION

The Reedy Creek Quarry is owned and operated by the Gold Coast City Council, and is located at Andrews, Gold Coast. It produces approximately 250,000 t/annum of high quality crushed rock and earth fill construction material products. It is an open cut quarry established to extract a hard and resistant band of quartzite and lower strength argillite (shale like) materials.

Quarry release water is discharged into Stoney Creek which is an ephemeral creek collecting runoff from the Pacific Highway and mostly developed upstream catchment. Other adjoining large privately operated quarries also drain into this creek system. Stoney Creek flows immediately downstream into the Burleigh Waters lake system, and the Mermaid Waters tidal canal, which ultimately discharges to the Nerang River.

The quarry location and Stony creek catchment are shown in figure 1.

Rocks encountered during extraction contain variable amounts of pyrite (iron sulfide) which oxidizes when exposed to the atmosphere to form deleterious and highly soluble acid salts.

A key environmental issue at the quarry is the management of acid rock drainage. Typical acid rock drainage collected on the site is around pH 2.7-3.0, requiring treatment prior to discharge to the downstream receiving environment. Council commissioned consultants to undertake a geo-environmental investigation and to produce an Acid Rock Management Plan (ARMP).

Operations at the Quarry are carried out under an Environmental Protection Agency (EPA) license. The license includes water quality performance criteria for waters released off-site, and other conditions to mitigate environmental harm. Release water is required to be within the pH range 6.5-9.0.

QUARRY GEOLOGY OVERVIEW

The pit floor of the Reedy Creek Quarry is presently excavated to RL 18m AHD. The floor consists of mainly weathered quartzite. A pit lake with sump level RL 9m AHD is located centrally in the pit floor. Up to 6 developed benches and sub-vertical faces are evident in the eastern high-wall which is at maximum RL 80m AHD.

Quarrying activities has resulted in the exposure of quartzites, argillites and carbonaceous shale in the eastern side wall (figure 2). The basic geological model of the eastern side wall generally consists of a thick band quartzite underlain by shale and metasilstone (figure 3).

As the next stage of quarrying involves extraction of the north eastern face, it was important the quarry ascertain the extent of potential acid forming rock in this location.



Figure 1: Reedy Creek Quarry Locality / Catchment Map

ACID FORMING ROCK

Rocks encountered during quarrying operations at the Reedy Creek quarry have been shown to contain variable amounts of pyrite (iron sulfide) which oxidizes when exposed to the atmosphere to form deleterious and highly soluble acid salts.

Any excavated/blasted face that has a high enough concentration of pyrite and/or other sulfides has the potential to generate and release acid. The occurrence of pyrite is highly variable within the quartzite and associated low grade metamorphic interbeds that comprise the rocks exposed in the Reedy Creek Quarry, with highest occurrence noted in beds of carbonaceous shale and overlying quartzite material in the presently exposed north eastern face of the quarry (figure 2).

Products supplied by the quarry are generally sourced from rocks that are resistant to weathering such as quartzite (rather than carbonaceous shale).



Figure 2: Picture of N-E Face with exposed carbonaceous shale (Acid Hot Spot)

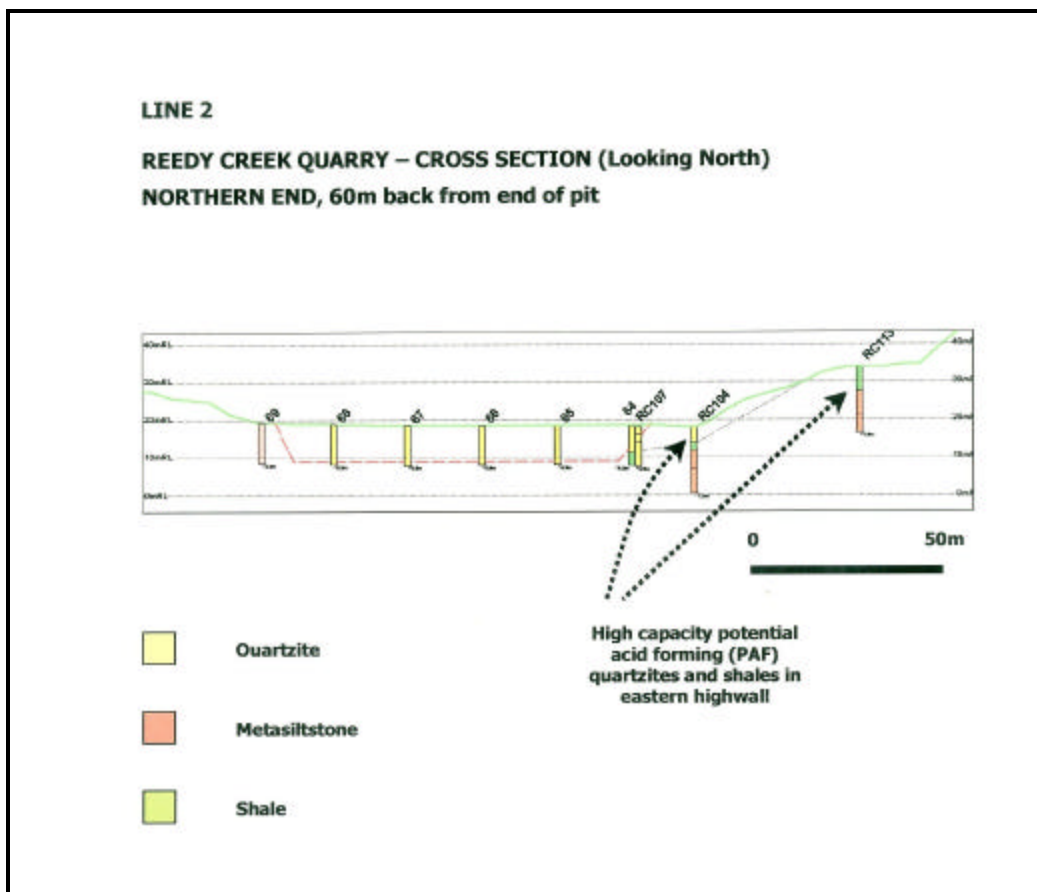


Figure 3: Typical Pit floor/ N-E Face Cross-section

The net acid producing potential of quartzite is generally lower than carbonaceous shale due to less pyrite content and the presence of minerals, which provide acid neutralization capacity (ANC).

The general acid production chemical reactions are similar to that for acid production in acid sulphate soils, where pyrite is typically encountered in low lying coastal areas.

ACID ROCK DRAINAGE

During rainfall events, surface water flows across and percolates through exposed and fissile surfaces of potentially acid forming rocks. If the exposed rock is acid forming and has insufficient natural acid neutralizing capability, then acidic rock drainage is generated.

Key Reactions to the acid production process are:

1. $\text{FeS}_2 + 3.5\text{O}_2 + \text{H}_2\text{O} = \text{Fe}^{2+} + 2 \text{SO}_4^{2-} + 2\text{H}^+$ (acid generating)
2. $\text{Fe}^{2+} + 0.25 \text{O}_2 + \text{H}^+ = \text{Fe}^{3+} + 0.5 \text{H}_2\text{O}$ (acid consuming)
3. $\text{Fe}^{3+} + 3 \text{H}_2\text{O} + \text{Fe}(\text{OH})_3 + 3 \text{H}^+$ (acid generating)

LATENT ACIDITY

Complete oxidation of Fe^{2+} (ferrous iron) to Fe^{3+} (ferric iron) in equation 3 can be slow in which case acid production may only become apparent after quarry water has travelled several kilometres off site. It is important the quarry consider latent acidity to ensure that released waters did not generate acid in the downstream receiving environment.

To account for potential latent acidity, total acidity tests are completed where samples are titrated with a strong base to a set pH (typically pH 8.3). The total acidity is reported in units of 'mg CaCO_3 /L'. A hydrated lime dosage amount is determined for the pit lake by converting the acidity measure into units of 'mg $\text{Ca}(\text{OH})_2$ /L'.

KEY OUTCOMES OF THE ACID ROCK MANAGEMENT PLAN

1. Non - Acid Rainfall Runoff Diversion

The quarry has implemented an on-site surface water diversion system to separate non-acid catchment runoff from contaminated acid runoff to minimise treatable acid rock drainage volumes. Having undertaken basic geological mapping the source of the acid rock drainage was determined and diversion drains installed.

2. Increased On-site Storage for Acid Rock Drainage

Pit lake capacity has been increased substantially to accommodate discharge for up to a Q20 average recurrence interval (ARI) storm event for runoff capture and treatment purposes. This exceeds EPA minimum requirements. This provides an extremely high level of lake overflow immunity. The large storage volume enables cumulative storage of acid rock drainage from many successive rainfall events. Pit lake treatment and pump-out is undertaken periodically to maintain a minimum residual storage capacity. Figure 4 shows the pit lake full to just below pit floor level.

3. Passive Acid Rock Drainage Treatment System

A passive acid rock drainage treatment system has been trialled. Highly alkaline cement waste has been profiled into pit floor perimeter berms to intercept and neutralise acid run-off, prior to it reaching the pit lake. Based on the long term observed buffering impact on sampled pit lake waters, the effectiveness of this passive approach appears limited. Upon exposure to the acid rock drainage the cement waste forms a surrounding precipitate. This limits further effective neutralisation.



Figure 4: Picture of full pit lake

4. Active Acid Rock Drainage Treatment System

Once collected in the pit lake, acid waters are laboratory tested for pH and other water quality parameters including turbidity, suspended solids and heavy metals. Based on the test results an optimum hydrated lime (calcium hydroxide) dosage is determined and applied to raise the pH to EPA licensed release standards. Additional laboratory testing is undertaken to ascertain the potential for latent acidity.

Based on successful trials, the quarry has determined that the application of hydrated lime in a diluted slurry form is the most cost effective active treatment option. Lime slurry is applied via a customised mixing tank / high pressure spray plant similar to a common hydro-mulching machine as shown in figure 5. Pit lake water is subjected to testing before and after treatment to monitor treatment effectiveness and to ensure water quality compliance with EPA license requirements, before release off site.

5. Handling, Use and Disposal of Acid Forming Rock.

A visual identification guide was prepared to assist plant operators identify potential acid forming rock. In conjunction with the guide, rock sample tests are completed to enable the quarry to confirm the locations of highly concentrated acid rock areas, which are referred to as 'hot spots'. Hot Spot material is separated and stockpiled into a bunded area where stockpile leachate is captured and directed to the pit lake for treatment.

The quarry is planning to trial blend the acid rock with various low cost neutralising agents eg calcium carbonate and cement waste products, to stabilise it for manufacture into inert earth fill construction products. Alternatively the acid rock can be safely disposed of long term submerged in the quarry pit lake or encapsulated with clay fill, where it is denied access to oxygen. Reliable identification and selective handling of acid rock was required to minimise the risk of contamination with other quarry products.

6. Ground water monitoring

Preliminary geological assessment has identified pit lake geology to comprise quartzite and argillite that is distinctly weathered with mostly intact/un-fractured rock, likely to provide good hydraulic isolation from the lower groundwater. The quarry plans to initiate a ground water monitoring program to ensure that there are no adverse environmental impacts associated with acid pit lake contamination of groundwater.



Figure 5: Picture of hydrated lime applied to Pit Lake

CONCLUSIONS

Rocks encountered during quarrying operations at the Reedy Creek quarry have been shown to contain variable amounts of pyrite (iron sulfide) which oxidizes when exposed to the atmosphere to generate acid rock drainage.

Trial of a passive acid rock drainage treatment system using fine cement waste in catch berms to intercept and neutralize surface flows has proven to be of limited effectiveness.

An active treatment system involving the in-situ neutralisation of collected acid waters in the pit lake by application of a hydrated lime slurry, has proven to be the most effective treatment option of acid rock drainage to date.

Latent acidity impacts have been considered to ensure that released waters do not produce acid in the downstream receiving environment, following release off site..

Implementation of the key outcomes of the Acid Rock Management Plan has enabled Council to mitigate potential environmental harm and better plan and manage extraction operations in quarry resources containing acid rock.

REFERENCES

1. Dobos and Associates, Short Course Notes on 'Treating Acid Drainage: Choosing and Implementing Cost Effective Technologies', 6th International Conference on Acid Rock Drainage, Cairns, 2003.
2. Butler Partners, Acid Rock Management Plan - Reedy Creek Quarry, March 2004.
3. Construction Services Branch, Gold Coast City Council, Reedy Creek Quarry Operations Manual, version 5/5/04.
4. Eco Roc, Report on Acid Rock Drainage, Reedy Creek Quarry, March 2003

Analysis and Monitoring of Stage 5 Deformations at QAL Red Mud Dam No. 2

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SUMMARY: Connell Wagner was commissioned by Queensland Alumina Limited (QAL) to implement a monitoring program for the Number 2 Red Mud Dam, located 14km south of Gladstone. The aim of the program was to obtain data associated with stability, deformation, pore water pressures and seepage during and after construction several downstream raises.

Dam safety inspections and subsequent geotechnical investigations in 2001 showed that deficiencies in the clay core and/or the presence of a granular layer had caused the water table to rise within the embankment. To address potential problems, modifications were made to the proposed stage 5 raising and the frequency of dam wall monitoring was increased.

This paper presents the analysis and assessment of monitoring devices before and during the Stage 5 raising. As part of the monitoring program, finite element models were developed at the monitoring locations to determine the likely deformations of the inclinometers due to the Stage 5 raising and operation of the dam. A back-analysis was performed by calibrating the deformation analyses with the monitoring results at Stage 4 to obtain reliable soil elastic parameters for the finite element models. These were then able to provide an indication of the expected future horizontal and vertical deformations at each of the monitoring sites. Using the inherent variability of the soil parameters, trigger limits were derived to indicate procedures to be followed if specified criteria is exceeded.

INTRODUCTION

Queensland Alumina Limited owns and operates a major alumina refinery at Parsons Point Gladstone. QAL currently pumps a by-product of the refinery process known as red mud from the refinery to the Number 2 Red Mud Dam (#2 RMD) on Boyne Island, 14km south of Gladstone.

The original number 2 Red Mud Dam was constructed between 1973 and 1976 and is formed by a combination of natural hillside and constructed earth embankments. Since 1991 the dams have been progressively raised by six 2m downstream stages. Geotechnical instrumentation were installed in June 1999 to monitor the dam wall response during the future construction stages, and operation of the dam

Dam inspections carried out in September 2001 showed wet spots on the slope of the embankments and ponded seepage on the upper berms. These inspections and subsequent investigations confirmed the presence of intermittent gravel layers within the clay core.

Based on the results of the inspections and investigation, modifications were made to the Stage 5 raising and the frequency of dam wall monitoring increased. Finite element models were created and calibrated with the results of the Stage 4 monitoring of inclinometers along the dam wall. To provide QAL with a set of procedures to follow based on embankment movement deformation analyses were carried out using the variability in soil parameters to establish a series of “trigger limits” at each monitoring location. The trigger limits, green, amber and red defined actions to be followed if the horizontal deformation is exceeded.

BACKGROUND

The Number 2 Red Mud Dam consists of three major constructed earth embankments and minor saddle dams that are wedged between metamorphic and sedimentary formations of the Curtis Island Group. The embankments have been progressively raised by six 2m downstream stages (originally designed in 1989) that have been continuously constructed as the mud level rises. The Stage 5 raising was completed in 2003. At the end of Stage 6 the embankment dams will be close to 20m above the natural surface.

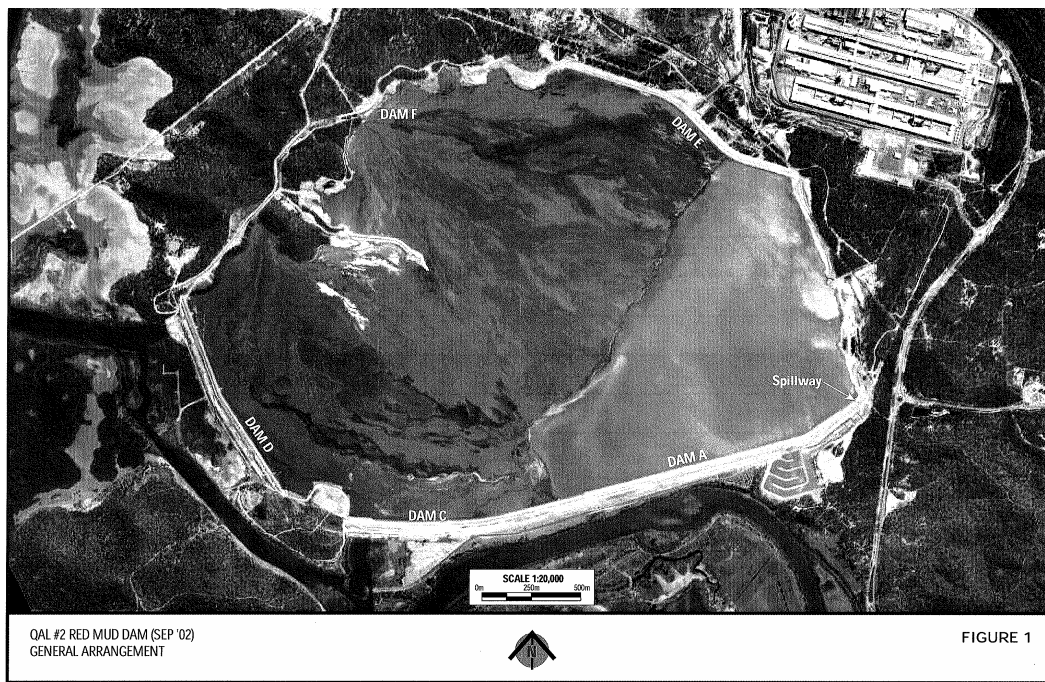


Figure 1 – QAL Red Mud Dam Number 2

Just prior to the Stage 4 dam raising geotechnical instrumentation was installed at critical sections along the dam walls. The monitoring equipment was installed to capture data associated with stability, deformation, pore water pressures, and seepage during the future construction stages, and operation of the dam.

The No.2 Red Mud Disposal Dam is a referable dam and requires periodic inspections. Dam inspections carried out by Connell Wagner in September 2001 observed wet spots at the top of the dam wall and ponded seepage on the upper berms. The governing mechanism of the wetting was attributed to seepage through the clay core due to deficiencies in the clay core and/or the presence of a granular layer. A subsequent investigation commissioned by QAL confirmed the deficiencies of the dam core material and intermittent gravel layers.

Based on the inspection and investigation results the frequency of monitoring was increased and further seepage and slope stability analyses were carried out. The results of these analyses showed that the dam wall may have potential problems in the long term at the Stage 6 operation level with the current design at the time. Remediation measures, to improve the long-term stability, were proposed. The extent of the measures varied for each dam depending on the computed factors of safety, the mud deposition next to the dam wall, material properties, extent of seepage and consequences of failure. The remediation measures are detailed below.

- Construction of a seepage cutoff trench and lining at the upstream face of the dam
- Construction of a drainage layer enclosed in geotextile along the existing batters and benches that were constructed in the Stage 5 development on the downstream side of the dam
- Construction of additional stabilising berms at the toe of the dam embankment

The above modifications were incorporated into the design for the Stage 5 construction works. Periodic readings and analysis of inclinometer data was undertaken during the Stage 5 raising from 24/05/02 to 21/05/03. Finite element models were used to predict future movements at each location. This paper addresses the assessment of the inclinometer data, and the finite element analysis used to predict the future movements.

MONITORING DATA ANALYSIS

Monitoring instrumentation was installed before the 2000 Stage 4 raising. Inclinometers were installed to capture data associated with the stability and deformation of the dam. Table 1 details the inclinometers installed at each dam location.

Table 1 QAL Red Mud Dam No.2 Inclinator Locations

Location	Instrument Number	Comments
A750	INC 1	Located near wet areas on embankment
A1000	INC 2	Closest point to former channel of South Trees Inlet
C400	INC3	Alluvial area
C500	INC 4	Former channel
D1040	INC 5	Alluvial area
D1140	INC 6	Former channel

The inclinometer readings up to July 2000 showed no significant movements observed. The subsequent reports indicated that much of the data was distorted due to installation effects and/or physical or noise disturbances.

Readings obtained in May 2002 for the Stage 4 level showed some inconsistencies. Excessive movements were recorded at five of the six inclinometer locations. While there were obvious errors in the readings and its reliability, there was significant concern as to the movements which would have initiated the buckling of the inclinometer casing. A check reading was taken in July 2002 to confirm this was not an operation error.

Although the inclinometer readings showed excessive movements, inconsistencies in the plots clouded a definitive solution. Confusing or unexplainable results are common problems to all inclinometers. The commonly encountered errors that affect the quality of the inclinometer data are the so-called “zero-shift” error and rotation errors due to spiraling of the casing. When readings are obtained in a slightly inclined casing, a correction for the apparent drift is necessary. Rotation produces a more irregular pattern in the displacement-versus-depth profile. Both of these effects may mask shear movements occurring at discrete zones within the embankment.

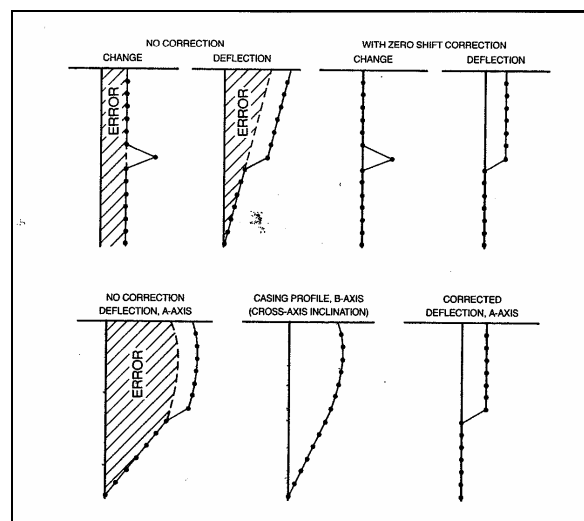


Figure 2 – Commonly Encountered Inclinometer Errors (Singh, B and Varshney, R, S, (1995))

The data collected for #2 RMD exhibited rotation and inclination errors, and several other anomalies that skewed the results. Much of the distortion was attributed to buckling of the inclinometer during the high dam operation level and subsequent raising in the Stage 5, which produced some consolidation of the foundation layers. To enable a sensible assessment of movements within the dam wall, the inclinometer data was analysed taking into account the following assumptions

1. Movements of 20mm within the XW Shale layer is physically unlikely
2. Oscillating movements of 20mm is unlikely to occur. This phenomenon may to occur in high permeability soils such as sand and gravel layers and also boulder fills reorienting itself during settlement
3. Considerable movement within the upper layers of the loose fill can be ignored
4. Movement parallel to the dam wall can be attributed to the rotation of the inclinometer in that the direction due to torsional effects rather than true movements in that direction. Therefore movement along the A axis for each location is considered to be of greater importance in the assessment

GEOTECHNICAL MODEL AND CALIBRATION

Using historical geotechnical data gathered from investigations prior to the installation of the inclinometers, deformation analyses were carried out to determine Stage 4 trigger limits. Trigger limits can be described as a set of procedures to be followed if specified criteria are exceeded.

Monitoring results observed in May 2002 showed that the Stage 4 trigger limits had been exceeded. However, as previously described, site inspections and investigation revealed that the clay core was not as competent as previously assumed. QAL implemented corrective measures during the Stage 5 raising but the Stage 4 trigger limits had to be refined to allow for the reduced strength parameters, higher phreatic surface and Stage 5 loading.

Based on the results of the 2002 geotechnical investigation and published references on soil properties correlation, the soil parameters for the Stage 4 calibration were determined. The deformation model was calibrated for the Stage 4 development at Section A750, A1000 and D1040. These sections were chosen since they provided the most consistent monitoring results, i.e. readings with the least errors. The calibration involved comparing the rationalised inclinometer readings with the results of the revised Stage 4 deformation analyses. Table 2 compares the maximum horizontal deformation (mm) from the inclinometer readings, Stage 4 analysis (using assumed soil parameters) and Stage 4 calibrated analysis.

Table 2 Comparison of Back analysis and inclinometer readings

Inclinometer	Max. Horizontal Deformation (mm)		
	1 (A750)	2 (A1000)	3 (D1040)
Inclinometer Reading*	30	15	15
Stage 4 initial (using assumed soil parameters)	6	6	14
Stage 4 calibrated (with high water table and lower soil strength)	16	15	21

*This reading neglects the obvious inconsistent reading in the top loose fill placed as a working platform during placement of the inclinometers.

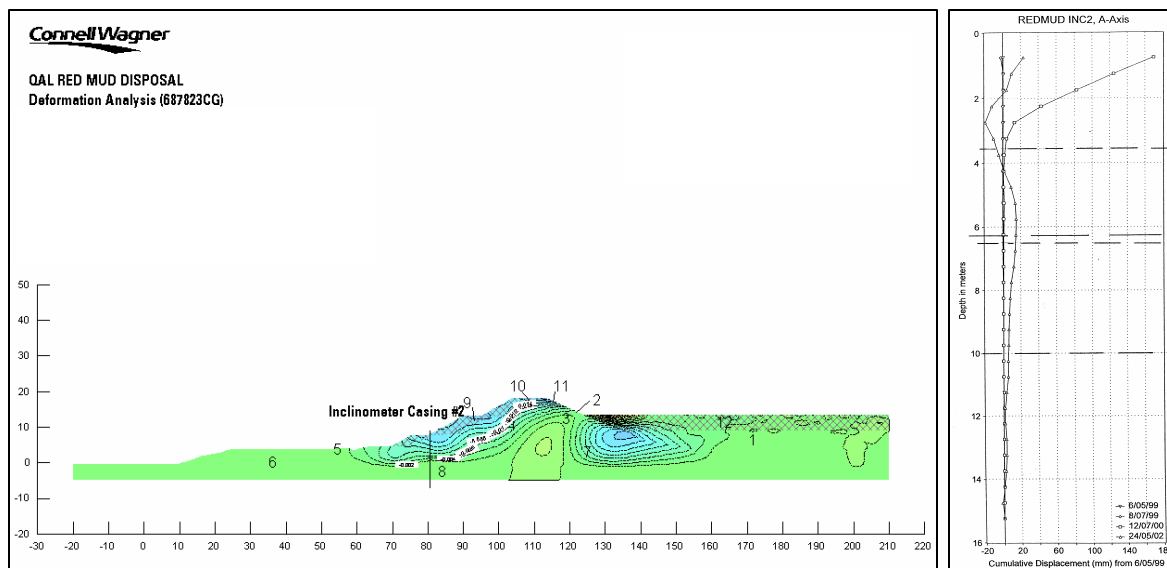


Figure 3 – Stage 4 Model Calibration

The comparison between the inclinometer readings and calibrated SIGMA/W model were shown to be in better agreement, although this result varies depending on the inclinometer. Further reduction in the soil strength parameters, to provide a closer calibration, would be unrealistic and furthermore not consistent with the results of the field investigations. Nevertheless, the above calibration was considered accurate enough to allow the calculation of the trigger limits for the Stage 5 raising.

DEFORMATION ANALYSIS and STAGE 5 TRIGGER LIMITS

To provide QAL with an indication of the likely horizontal deformation associated with the Stage 5 raising of the dam, further analyses were carried out using the calibrated Stage 4 soil parameters.

The analysis was carried out using the stress and deformation analysis computer program, SIGMA/W. Initial stress conditions for each section were determined using linear elastic soil properties. The deformation due to the addition of Stage 5 embankments and red mud has been calculated using Elastic-Plastic parameters. Table 3 presents a summary of the maximum expected deformations of each inclinometer following the Stage 5 raising, from the SIGMA/W analysis

Table 3 Maximum Expected Inclinometer Deformations

Inclinometer Location	Maximum Horizontal Deformation (mm)
INC1 (A750)	26
INC2 (A1000)	24
INC3 (C400)	8
INC4 (C500)	7
INC5 (D1040)	19
INC6 (D1140)	19

This was consistent with the appropriate methods (hand calculation) based on shear beam theory presented in Herzog (1999).

The results shown above provide an indication of the expected horizontal deformation of each of the inclinometers. The deformation analyses showed that the greatest horizontal deformations, due to the Stage 5 raising, are likely to occur along Dam A. These results were also supported by independent limit equilibrium slope stability analyses which indicated that Dam A possessed the lowest factor of safety for the Stage 5 raising.

The values extracted from the inclinometers provide the magnitude and rate of movement within the dam wall during raising. However, on their own, these results do not give an indication of the severity of the movements, particularly since the deformation is dependent on size of the raise and the subsurface profile. Therefore action limits (depending on the severity of movement) were adopted to compare against observed values.

To determine a set of “trigger limits” for the Stage 5 raising, a probabilistic approach was required taking into account the variability of soil parameters. The maximum expected inclinometer deformations were calculated using the average soil parameters. Therefore the results shown in Table 3 and shown in Figure 4 can be considered to be the “green trigger limit”; the most probable dam movement. To determine an “amber and red trigger limit” variance in the soil’s unit weight, undrained shear strength and elastic modulus were determined/estimated.

Kulhawy (1992) provides guidance on the variability of soil properties. Table 4 summarizes the coefficient of variance for the material properties used in the Stage 5 trigger limit determination.

Table 4 Inherent Variability of Soil Parameters

Property	Mean Coefficient of Variance (%)
Unit Weight γ , (kN/m ³)	7.1
Undrained Shear Strength, C_u (kPa)	33.8
Elastic Modulus (kPa)	33.8 (Inferred)

Note: Poisson’s Ratio, the soil friction angle and the coefficient of earth pressure at rest have remained unchanged

Since the data set from the previous investigations were not statistically significant and do not provide an accurate estimate of the soil’s variability, the values in Table 5 were adopted to determine the amber and red trigger limits. This assumes that the soil parameters can be described using a normal distribution. Table 5 provides a description of the trigger limits and the value of the soil parameters used.

Table 5 Description of Movement Trigger Limits

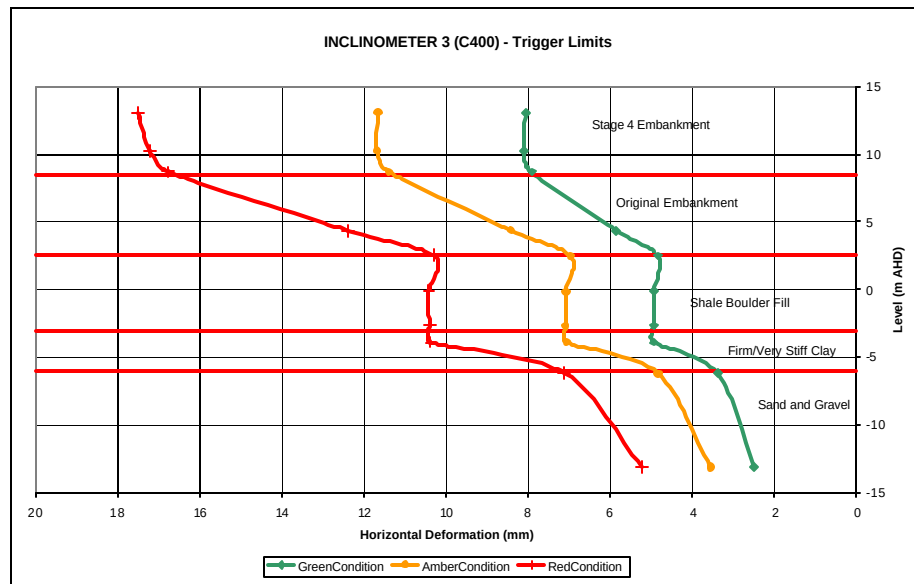
Trigger Limit	Movement Description	Soil Parameters
Green*	Less than the most probable movement prediction	Mean value
Amber	Between the most probable and upper bound movement prediction	Mean value reduced by one standard deviation
Red	Greater than the upper bound movement prediction	Mean value reduced by 1.65 standard deviations

Note: *Provides acceptable factors of safety from limit equilibrium analysis

Using the criteria adopted in Table 5, displacements corresponding to each trigger limit were determined for each dam/inclinometer section. Table 6 provides a summary of the trigger limits for the Stage 5 raising. These values were compared against actual monitoring readings during the Stage 5 raising.

Table 7 Stage 5 Trigger Limits for Monitoring Purposes

Trigger Limit	Action	Trigger Limit – Horizontal Deformation (mm)					
	Inclinometer Number	1	2	3	4	5	6
Green	No cause for concern	30	25	10	10	20	20
Amber	Movements causing concern -increase frequency of monitoring -Monitoring at 1 month intervals	45	40	15	15	30	30
Red	Increase reading frequency -initiate contingencies and actions -monitoring at 2 week intervals	60	50	20	20	40	40

**Figure 5 – Dam Section C400 Trigger Limits**

VERIFICATION

To monitor the lateral displacements of the embankment during the Stage 5 raising, Connell Wagner analysed and assessed inclinometer data obtained on 11/11/02, 13/01/03, 03/04/03 and 21/05/03 and compared it against the Stage 5 trigger limits.

During the monitoring period all significant movements were considered with particular attention being paid to the movements that occurred within the vicinity of the slip surfaces determined by stability analyses, i.e. the location/depth of the movement was compared with the location of the critical slip surface.

Correcting the inclinometer data for “zero-shift” and rotation errors and assessing movements within the critical slip surface only; a clearer indication of the dam movements was able to be derived.

The monitoring results during the Stage 5 raising show that dam wall movements ranged between 10mm and 30mm. These values did not exceed the “green trigger limit” criteria set out in the preceding section. The results shown above were considered to be in reasonable agreement with the results of the deformation analyses. Table 8 provides a comparison between the calculated and observed horizontal deformations.

Table 8 Observed versus Calculated Horizontal Deformation

Inclinometer	Horizontal Deformation (mm)		
	SIGMA/W Calculated	Green Trigger Limit	Observed
INC1 (A750)	26	30	30
INC2 (A1000)	24	25	15
INC3 (C400)	8	10	10
INC4 (C500)	7	10	10
INC5 (D1040)	19	20	15
INC6 (D1140)	19	20	15

The comparison between the observed and calculated horizontal deformations shows that the calibrated finite element model was able to accurately predict the movements for the Stage 5 raising; proving to be an invaluable tool during the monitoring period.

CONCLUSIONS

In summary this paper has shown that the combined use of inclinometers and finite element models in the monitoring of embankment deformations can be successful as long as an approach that carefully questions both the field and analytical results is adopted.

The monitoring of QAL Red Mud Dam No.2 has shown that inclinometers are sensitive instruments that require the end-user to analyse the validity of the results without concluding that the equipment is at fault. Problems that are common to all inclinometers, such as “zero-shift” and rotation errors, were exhibited in the inclinometers at Red Mud Dam No.2. These errors tended to accentuate and mask “real” shear movements in the embankment and as such a careful examination of the results was required.

The use of inclinometers as a tool to monitor the movements within the dam wall has been complemented through the use of finite element models that were calibrated against actual deformations. The predicted movements correlated with the limit equilibrium factors of safety and hand calculated estimates using shear beam theory. The results of the Stage 5 raising has shown that a carefully constructed finite element model that is calibrated against actual conditions can provide a reasonable assessment of dam wall movements.

ACKNOWLEDGEMENTS

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REFERENCES

1. Herzog, Max, (1999) “Practical Dam Analysis”, Thomas Telford, London
2. Singh, B and Varshney, R, S, (1995) “Engineering for Embankment Dams”, A.A. Balkema Publishers, Brookfield
3. Turner, K and Schuster, R, (1996) “Landslides Investigation and Mitigation”, Transport Research Board, National Academy Press, Washington
4. Connell Wagner Report, (1999) “Stage 4 Stability and Deformation Analysis”
5. Connell Wagner Report, (2000) “Inspection Report”
6. Connell Wagner Report, (2002) “Dam Safety Inspection”
7. Connell Wagner Report, (2003) “Stage 5 Stability and Deformation Analysis”
8. Connell Wagner Report, (2003) “Dam Wall Monitoring – Summary Report”

A Geotechnical Guide for Pump Stations Projects in the Sydney Region

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Summary: This paper provides a guide to geotechnical requirements at the investigation, design and construction stages for typical sewage pump station projects in the Sydney region. This guide is based on experience gained from over 230 pump station upgrades carried out as part of the SewerFix Pump Station Project (SewerFix) between 2001 and 2004.

Large, multi-site projects such as SewerFix provide an excellent opportunity to develop a systematic method of collecting and calculating geotechnical information. It is anticipated that the protocols developed during the project may be applied to other similar programs with a view to improved efficiency.

The paper also presents the benefits of conducting such a project under an alliance contract arrangement, which facilitates continual geotechnical input throughout the lifecycle of each pumping station and the project as a whole.

INTRODUCTION

The SewerFix program is a \$200 million infrastructure project involving the upgrade of more than 300 pumping stations over a four-year period. The program extended throughout Sydney, the Blue Mountains and the Illawarra region. Each pumping station site represented a project in itself, with varying scope, environmental and economic constraints. Several of the stations programmed for upgrade required capital expenditure exceeding one million dollars.

The aims of the project were essentially to:

- Improve operational reliability of the existing sewage pumping stations;
- Provide for future growth; and
- Comply with the Environmental Protection Authority's (EPA) licensing requirements for sewage transport systems, thus reducing sewage overflows and protecting public health and the environment.

The project was delivered to the client, Sydney Water Corporation (SWC), under an alliance or "friendship" contract. The alliance team comprised Bovis Lend Lease, CH2MHill, the Phillips Group, Sinclair Knight Merz, Sydney Water and Tenix Alliance. Sinclair Knight Merz (SKM) acted as design engineers under the contract with whom the author was engaged for the duration of the project design phase.

This paper describes the investigation, design and reporting methodology adopted throughout the SewerFix project. It will also look at the role of geotechnical involvement in the construction phase of the project and present a case study of a problem site and associated lessons learned.

Geotechnical input was provided on a total of 105 stations throughout the Sydney metropolitan, Blue Mountains and Illawarra regions. These sites covered a wide spectrum of geological environments with sub-surface conditions ranging from low lying alluvial flood plain environments through imported fill sites, shallow Hawkesbury Sandstone sites and steep, rocky sites.

GEOLOGY

The pumping station sites making up the SewerFix program can be categorised in terms of three geographical regions:

- Sydney Metropolitan;
- Illawarra; and
- Blue Mountains.

The extent of these regions and the site locations are shown below on **Figure 1**.

The geology of these regions has been subject to numerous studies and is generally well understood. Typically, the geology of the Sydney region comprises Triassic Sandstones and Shales with minor occurrences of Quaternary sediments, Branagan, (1).

The Blue Mountains region is generally made up of plateau, cliff and valley landscapes. The plateau and cliff features are typically sandstone above the sandstones, mudstones and coal measures of the Sydney Basin.

The geology of the Illawarra region includes the Hawkesbury Sandstone, Narrabeen Group, Wianamatta Group and the Illawarra Coal Measures. Unique geological units occur as intrusive and extrusive igneous features through sedimentary rocks (eg Gerringong Volcanics).

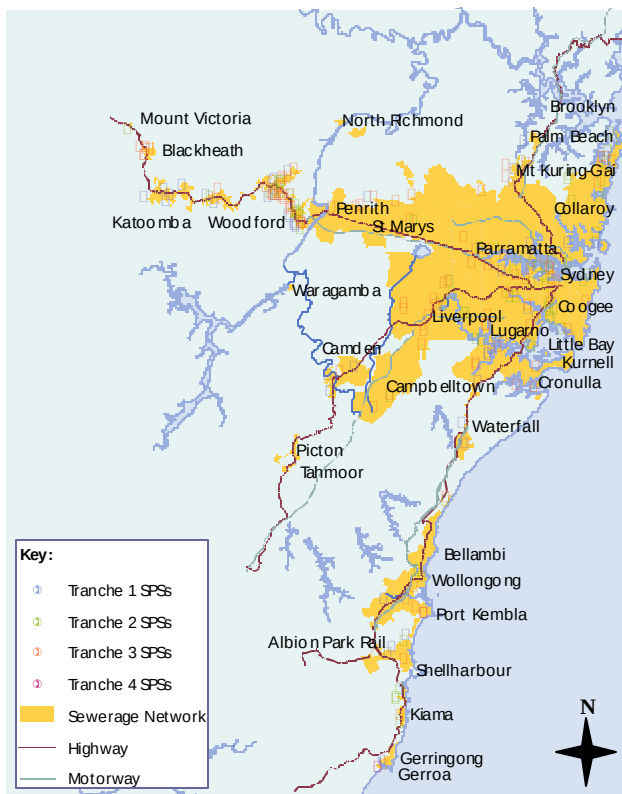


Figure 1 SewerFix Project Location Map

As a large proportion of pumping stations are gravity fed, they are commonly located in natural low points within a catchment; for example at the base or on the side of a valley, on the edge of a creek or drainage path, or on the coast. For this reason, alluvial sediments and imported fill formed major constituents in the geological environments encountered at many of the pumping station sites.

INVESTIGATION METHODOLOGY

Geotechnical Investigations

Geotechnical assessments were required at the majority of the stations undergoing civil upgrade works in order to identify geotechnical and environmental constraints and interpret ground conditions for structural design.

The geotechnical requirements for each site were scoped up according to the upgrade works proposed for each station. A site-specific investigation methodology was then developed in order to achieve these requirements and a geotechnical site investigation conducted within the environmental and community framework dictated by the program.

On a site level, typical geotechnical objectives were to:

- Characterise the subsurface geotechnical and groundwater conditions;
- Provide recommendations on possible footing types, founding levels and serviceability bearing pressures for any proposed structures;

- Assess likely excavation conditions, support requirements and uplift issues; and
- Provide advice on geotechnical and environmental issues such as site contamination and waste classification, acid sulfate soils, slope stability and effects on existing or proposed adjacent structures.

At the proposal stage the investigation requirements for a given pumping station site were broadly grouped into four investigation levels based on the scope of the proposed upgrade works. **Table 1**, below, outlines the investigation level, associated scope of works and typical investigation and testing methodology.

Table 1 Site Investigation Levels

Level	Scope of Works	Investigation and Testing
0	- No works; - Electrical upgrade.	-No fieldwork or testing; -Desk study if required
1	-Underground structures typically $\leq 2.5\text{m}$ deep; -Above ground structures; -Roads/pavements and turning areas.	<i>Site Inspection:</i> -Hand augered boreholes; -Dynamic Cone Penetrometer tests (DCPs); -Geological mapping. <i>Testing:</i> -Disturbed soil sampling (min 1 sample per soil/fill unit or 1 per m depth); -Acid sulfate soil screening, corrosion assessment; -Peroxide Oxidation Combined Acidity and Sulfate (POCAS) testing if required; and -Contamination Testing.
2	-Underground structures typically $> 2.5\text{m}$ deep but $\leq 3\text{-}4\text{m}$ deep; -Pipelines.	<i>Test Pit Investigation:</i> -Test pits excavated with rubber tired backhoe or similar; -Disturbed soil sampling; -DCPs; -Geological Mapping. <i>Testing:</i> -Disturbed soil sampling (min 1 sample per soil/fill unit or 1 per m depth); -Acid sulfate soil screening, corrosion assessment; -POCAS testing (if required); and -Contamination Testing.
3	-Underground structures typically $>3\text{-}4\text{m}$ deep; -Pipelines microtunnels	<i>Borehole Investigation:</i> -Cored or non-cored boreholes minimum 1, generally 2); -Disturbed soil sampling; -Undisturbed soil sampling (where appropriate); -Piezometer installation (where appropriate); -Geological mapping. <i>Testing:</i> -Disturbed soil sampling (min 1 sample per soil/fill unit or 1 per m depth); -Acid sulfate soil screening, corrosion assessment; -POCAS testing (if required); and -Contamination Testing.

Initial cost estimates for the project were based on the design of 74 stations. **Figure 2** below, shows the estimated site investigation level breakdown versus the actual breakdown.

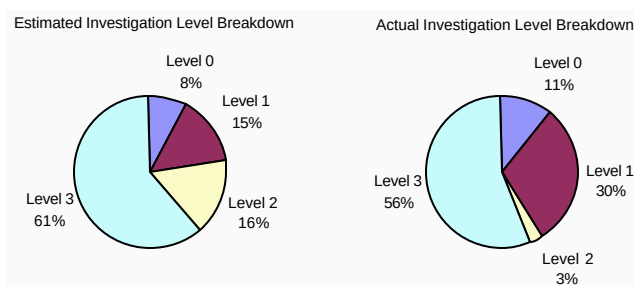


Figure 2 Estimated vs. Actual Investigation Levels

The final breakdown reflected changes in design methodology and a move from providing capital-intensive physical storage solutions such as emergency storage tanks to providing electrical and mechanical solutions such as generators and bypass arrangements.

Contamination Investigations

In order to streamline the investigation and design component of the project, environmental sampling and analysis was incorporated into the geotechnical scope of works. The main objective of conducting environmental sampling and analysis as part of the geotechnical investigations was to provide waste classification data for material to be excavated and disposed of off site as part of the upgrade works. In addition, the environmental sampling and analysis allowed for a preliminary assessment of potential site contamination issues, however it should be noted that detailed assessment of individual pumping station sites for the given land use and evaluation of potential human health risks were beyond the scope of the investigations and could potentially have led to very complex site issues.

The scope of contamination sampling and testing was developed in general accordance with the 1999 NSW EPA Solid Waste Guidelines and included testing at least sample of fill material and one natural soil sample for the following analytes:

- Heavy Metals (Arsenic, Cadmium, Chromium, Copper, Nickel, Lead, Zinc and Mercury);
- Total Petroleum Hydrocarbons (TPH);
- Benzene, Toluene, Ethylbenzene, and Xylene (BTEX);
- Polycyclic Aromatic Hydrocarbons (PAHs);
- Organo-Chlorine Pesticides (OCPs); and
- Polychlorinated Biphenyls (PCBs).

The results of these analyses were compared to the NSW EPA Health Based Investigations Levels for Commercial or Industrial Land Use, the Service Station Guidelines and the Waste Classification Guidelines. Samples returning results indicating a waste classification of solid or hazardous triggered additional Toxicity Characteristic Leachate Potential (TCLP) testing. Based on these results a waste classification was developed for the material tested.

REPORTING AND DESIGN

Reporting

As is the case in most construction projects, the aim of the geotechnical reporting was to provide the construction team and associated sub-contractors a clear and concise picture of the geotechnical conditions expected for a given site and highlight any potential geotechnical issues or constraints that may be encountered during the construction works. The level of reporting adopted for the individual pumping station sites was again a reflection of the scale of proposed upgrade works and subsequent investigation level. Typically geotechnical reporting for the stations was either in the form of an inspection or investigation report.

An inspection report was generally produced for Level 1 and some Level 2 sites. The inspection report format was developed to provide a concise overview of the potential geotechnical and environmental issues for small to medium scale upgrade works. The basic outline of a typical two to three page inspection report was as follows:

- Site Description and Proposed Works;
- Surface Conditions;
- Ground Profile;
- Geotechnical Hazards and Issues including:
 - Foundation Conditions;
 - Excavation Conditions, Support and Batter Slopes;
 - Acid Sulfate Soils;
 - Chemical Contamination and Waste Classification;
 - Structure Durability; and
 - Groundwater Issues.

A more comprehensive investigation report was generally produced for Level 2 and 3 sites, with the report format reflecting the increased scope of proposed upgrade works and subsequently the more detailed geotechnical investigation methodology. A typical investigation report presented an assessment of the sub-surface conditions at a given site, the results of any testing undertaken as well as a detailed discussion of potential geotechnical and environmental issues associated with the proposed upgrade works including:

- Foundation options/preparation;
- Excavation conditions and support;
- Lateral earth pressures on both temporary and permanent works;
- Uplift considerations;
- Groundwater issues; and
- Waste classification, acid sulfate soil and durability assessments.

All reports were published in portable document format (PDF) and were distributed and managed via the Bovis Lend Lease information network Project Web. Project Web provided a successful method of on-line document

control, streamlined the document approval process and allowed all project team members easy access to the available geotechnical information.

PROJECT MANAGEMENT

The project was executed in a number of isolated work packages or tranches with each having its own targets and drivers including cost, schedule and performance. An important feature of the project delivery under an alliance contract arrangement is the measurement of project success. The success of the SewerFix project and thus the shared rewards, relied on the alliance team exceeding best practices in all Key Performance Indicator (KPI) areas; cost, schedule, safety, community, environment, quality and legacy. Under the KPI structure, success is dependent on the whole alliance team attending to cost and non-cost areas with the same degree of priority.

Geotechnical costs were estimated at the proposal stage based on the four investigation levels. These costs were then averaged over the duration of the project to provide a method of tracking investigation costs over time as well as against completed sites. Actual costs were then tracked against this estimated rate of expenditure. **Figure 3** below, depicts actual costs versus estimated costs over the first year of the project to October 2002.

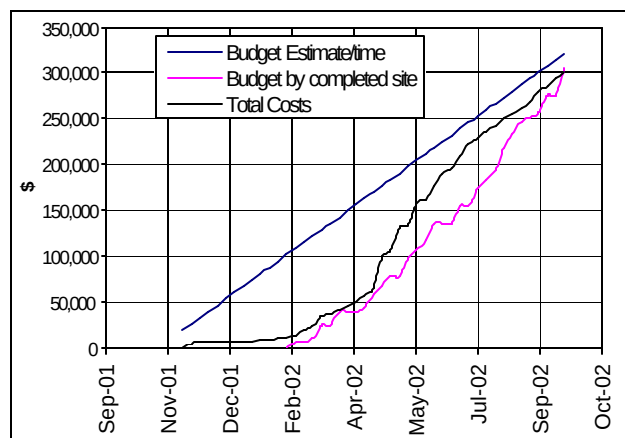


Figure 3 Geotechnical Cost Tracking

A total of about \$300,000 was expended on geotechnical and contamination investigation and testing at approximately 63 sites in the first year of the project. This equates to an average price of \$4,800 per site at 2001 prices with the more involved Level 3 sites having an average expenditure of \$6,000.

This method of tracking geotechnical investigation design costs became less realistic when the breakdown of investigation levels changed to include more Level 1 sites and a reduced number of Level 2 and 3 sites. This shift in design methodology resulted in less investigation and design money being spent to complete sites.

The generic investigation levels were also used in resourcing and programming of investigations works.

CONSTRUCTION

The principal permanent underground structures constructed as part of the SewerFix project were emergency storage tanks. These structures are constructed in order to provide additional physical storage at the pumping stations for utilization in the event of a breakdown or heavy flow situation.

Typically, the smaller emergency storage structures comprised pre-cast, reinforced concrete tanks, while larger structures were constructed in-ground using either a caisson or “bottom-up” approach. The construction method adopted for the larger tanks was dependent on a number of factors including:

- Subsurface and groundwater conditions;
- Space and elevation constraints;
- Surrounding land form, adjacent structures and potential instabilities;
- Site budget and construction schedule;
- Community issues and aesthetics; and
- Environmental issues including contaminated and acid sulfate soils.

Table 2, below summarises typical construction approaches used for emergency storage tanks.

Table 2 Construction Options for SewerFix Sites

Option	Advantages	Disadvantages
Caisson Structure	-Combines permanent and temporary works; -Allows for “wet” excavation; -Reduced risk of base liquefaction; -Can be low cost; -Minimal excavation footprint.	-Irregular ground profile can result in uncontrolled sinking and tilting; -Ground loss and inflow if constructed ‘dry’ below the water table; -Requires a semi-uniform rock level for seating; and -often poor groundwater cutoff.
Sheet Piles	-Provides good groundwater cutoff; -Can be vibrated in or ‘silently’ installed.	-Dewatering may be required; -Driving problems in blocky fill or talus material; -Careful dewatering to avoid base liquefaction and settlements.
Secant Pile	-Combines temporary and permanent works; -Can use piles uplift capacity; -Compressive arch effect of piles provides lateral support.	-Capital and labour intensive; -Good workmanship required for water tightness; -Large excavation footprint required.
Shoring Boxes	-No permanent works; -Low capital cost; -Widely available; and -Specialised plant not required.	-Large excavation footprint; -Limited to small excavations; -Internal bracing limits access to the excavation; -Minimal groundwater cutoff; -Potential for ground inflow.
Open Cut	-Easy to construct; -Specialised plant is not required.	-Large excavation footprint required; -Limited safe access to excavation; -Disposal of excavation material; -Limited to shallow excavations.

In addition, rock bolting and shotcrete support was implemented for deep emergency storage tank excavations in rock to allow tank formation.

Geotechnical Involvement

Due to the structure of the SewerFix alliance contract, geotechnical input was not limited to the design phase of the project. Geotechnical involvement in the form of foundation inspections and excavation stability/shoring assessments continued throughout the construction of the pumping station structures. This involvement provided an excellent opportunity to 'test' the geotechnical assumptions made during the investigation of each site and further refine the site-specific geotechnical models. This experience enabled confirmation of design assumptions and better control of construction risk, nomination of shoring and temporary works requirements, information which was in turned used to modify the methodology for the remaining site investigations, creating a 'feedback' loop rarely seen in typical design and construct projects.

PROBLEM SITES AND LESSONS LEARNT

Table 3, below comprises a list of the more geotechnical challenging pumping station sites and a summary of the relevant geotechnical issues:

Table 3 Problem Sites

Site #	Location	Description of Works	Geotechnical Issues
SP0477	Lidcombe	- Caisson construction	- Minor Ground inflow
SP0820	Blaxland	- Caisson construction	- Uneven rock level - Groundwater inflow - Ground loss around caisson
SP0515	Wombarra	- Caisson construction	- Slope instability - Talus material
SP1129	Millers Point	- Sheet piled excavation - Microtunnelling - Hand excavated shafts	- Subsurface gasworks structures - Dyke - Varying sandstone level - Highly contaminated

The above sites highlighted some important generic geotechnical issues for pumping station sites including:

- The need for adequate site investigation for caisson constructions, including a good idea of rock and soil profile and groundwater conditions;
- The provision of adequate resources (both monetary and labor) for geotechnical site investigations at pumping station sites to avoid knowledge gaps and construction complications; and
- The benefits of ongoing geotechnical involvement in the construction process to help manage any ground related problems that may arise.

CASE STUDY – SP1129, Millers Point

Introduction and Scope of Works

Site SP1129 is located on Hickson Road, Millers Point. The upgrade works for this station included:

- Construction of a new pumping station (SP1129);
- Installation of all associated pipework and connections;
- Decommissioning of existing stations SP0059 and SP0013 due to their location within commercial and residential development sites.

The scope of works at SP1129 included three main aspects:

- Construction of SP1129 using a top-down approach in combination with a sheet piled excavation;
- Microtunnelling approximately 500m of pipework beneath Hickson Road and construction of three associated transition pits; and
- A combination of hand excavated and microtunnelled connecting pipework connecting the new station to the Kent street network above.

Site Geology and Geotechnical Issues

Hickson Road is situated on the edge of reclaimed land overlying Hawkesbury Sandstone. Previous investigations conducted in the area suggested that Hickson Road is located at the base of a former sandstone quarry while the adjacent reclaimed area comprises a mixture of reworked alluvial material and man-made fill. A small dyke (<2m wide) strikes approximately east west across the site.

A number of significant geotechnical issues were encountered at the site. The area was historically used as a gasworks and subsequently a number of sub-structures including gas holders and tar tanks had been excavated into the sandstone bedrock. The sites industrial history also resulted in the soils, rock and groundwater underlying the area being heavily contaminated.

Geotechnical Investigations

The geotechnical site investigation for this site included the following aspects:

- Initial desk study of available geological information;
- Drilling two boreholes (one cored) at the proposed location for the new station;
- Drilling five boreholes along the diameter of the underground tar tank in order to establish its size and exact location; and
- Drilling eight cored boreholes along the length of Hickson Road following the proposed microtunnel alignment.

The geotechnical investigation was carried out in phases in order to adequately address the external issues posed by such a complex and sensitive site.

Problems Encountered

A number of problems were encountered at site SP1129 that consequently refined the process of geotechnical site investigation. These problems included:

- Varying rock level encountered at the new station location. Two boreholes and only limited cored information proved insufficient detail for sheet pile and excavation design.
- Sandstone encountered at the station location was found to be more fractured than suggested by borehole information. This created the need for additional excavation support and stability problems around the microtunnel entry points. A large proportion of the defects were vertical or near vertical and thus were not picked up in the vertical boreholes.
- Problems encountered in microtunnelling the last section of connecting pipework. Sandstone encountered had a higher proportion of clay than expected. In addition, microtunnelling was ceased in an area of soft/clayey material potentially an infilled underground structure or dyke feature.

Lessons Learnt

SP1129 and some of the construction problems encountered there provided a method of improving and tailoring major geotechnical investigations for pumping station sites. The lessons learnt from this site can be summarized as followed:

- Allow sufficient exploration at the proposed station location in order to adequately pick up changes in rock level across the tank structure/excavation. For major structures this should include a borehole in each corner of the structure and potentially one in the middle of the structure if significant differences in rock level are encountered. This is especially important for caisson structures.
- Detailed study of the sub-surface profile in the proximity of the station including any subsurface features, natural or man-made.

SUMMARY AND CONCLUSIONS

The multi-site SewerFix project provided a unique opportunity for developing a systematic approach to the geotechnical input on sewage pumping station upgrade sites throughout the Sydney metropolitan, Blue Mountains and Illawarra regions. This approach can be tailored to similar, multi-site infrastructure facilities. The main geotechnical features of the project can be summarised as follows:

- The investigation methodology involved grouping the sites into investigation levels (0-3) depending on the proposed scope of upgrade works. This investigation level was then tied into a standard scope of geotechnical and environmental sampling and analysis. Standardised reporting for the project again reflected the level of investigation adopted for each site.
- Average geotechnical investigation cost for the first year of the project was approximately \$1,800 per site at 2001 prices. The more complicated Level 3 sites had an average expenditure of \$6,000.

- Based on the experience gained from this project, a minimum of one borehole in each corner of the proposed underground structure should be adopted for pumping station site upgrades. This number should be increased if significant differences in ground profile are encountered or a caisson construction method is chosen.
- The generic contamination and geotechnical sampling and testing methodology developed for the SewerFix project can be easily modified and adopted in similar projects.
- The alliance nature of the project allowed for continued geotechnical involvement through the planning, design and construction phases of the program and promoted a “no blame” approach to design and construction.

ACKNOWLEDGEMENTS

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REFERENCES

1. Branagan D, " An Overview of the Geology of the Sydney Region ", in Pells (ed), *Engineering Geology of the Sydney Region*, Balkema, Rotterdam, 1985, 3-46.

Rock-Falls into Water and Potential Wave Damage

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ABSTRACT

This paper presents the findings of experimental work undertaken to investigate the potential for wave damage caused by rock-falls into partially water-filled open pit mines. A particular mine geometry has been used to model the situation in a wave flume, with a modelled rock-fall of 10,000 tonnes. The main findings of interest to the mining industry are: a) transfer of energy efficiency from a rock-fall to the water ranges between 5 and 80% depending on slide impact velocity; b) maximum wave run-up distance up slope is about 40 meters; c) available evacuation times for personnel located at a pumping station within the maximum wave run-up distance is about 37 seconds, which is insufficient.

1 INTRODUCTION

History provides us with several cases where large landslides and rock-falls have occurred and impacted into water bodies. The waves generated by these have caused widespread devastation and loss of life. Examples of such events have been well documented, and include a massive landslide into Vaiont Dam, Italy in 1963 and another at Lituya Bay, Alaska in 1958. The Vaiont slide destroyed several villages downstream and killed over three thousand people when the wave generated by the landslide overtopped the dam, whereas the Lituya Bay failure produced the highest vertical wave run-up ever recorded of 524m. A similar situation, though potentially on a smaller scale, may occur in any of the open pit mines currently in operation in Western Australia. Recently, failures of waste dumps into accumulated water have generated waves that have led to deaths.

During an open pit mining operation, ground and mining water accumulate at the bottom of the pit. The depth of water may be significant - greater than 30m, particularly after heavy rains and during a period of cut back. Access to the bottom of the pit is limited during these situations so a pumping station is often installed to manage and monitor the water level. However, should a slope failure which impacts with the accumulated water occur, there is a potential for a water wave to be generated that propagates across the pit water and runs up an adjacent slope. If the failure is large and impacts with enough momentum, then the wave generated may cause severe damage to the pumping station or other infrastructure near the water. Furthermore, personnel who may be located at or close to the water, such as during periods of pump maintenance will also be under significant risk.

At the end of the mining life, such pits may be considered for use as a recreational facility for boating or fishing. However, de-watering of the pits ceases, possibly leading to build up of pore pressures in the slopes and an increase in slope failures. This raises significant risk concerns for the regulators of the land in terms of Health, Safety and the Environment (HSE) implications, of which will not be obvious to the general public who utilise the facilities at a later stage.

This paper attempts to simplify the problem of rock-falls into water while ensuring that parameters inherent to the problem are not trivialized. Experimental work demonstrated in Figure 1, attempts to quantify the severity of damage caused by rock-falls into water. These are presented with the aim of providing the mining industry with a greater understanding of likely outcomes.

2 AIMS AND METHOD

The disciplines of physics, geology, geomechanics, fluid mechanics and mining engineering are involved in this problem. The areas least understood are processes involved with transferring energy from a rock-fall via water body, to wave run-up height. The specific aims therefore focussed on:

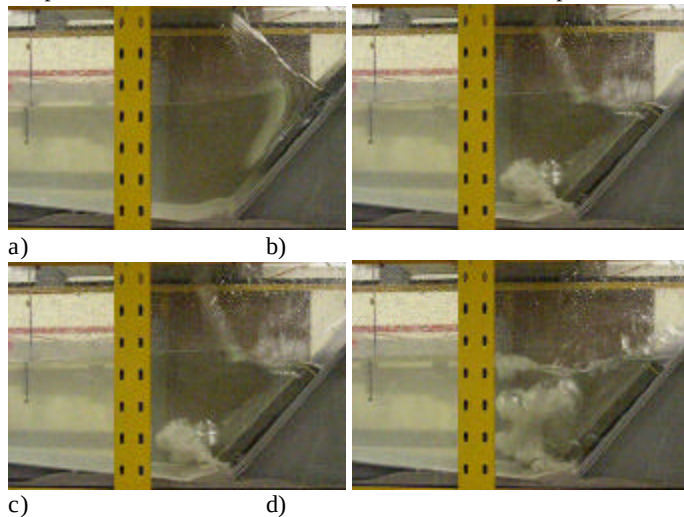


Figure 1 Model block entering water body

- 1 Construction of a scale model to facilitate experimental work, thus providing comparisons with previous research and theoretical predictions; and
- 2 Providing suggestions on possible methods to warning personnel of dangerous situations such as a monitoring buoy in the water which measures wave heights.

Although both theoretical considerations and calculations from empirical approaches were investigated, this paper focuses on the experimental work undertaken.

3 EXPERIMENTAL MODEL

3.1 OBJECTIVES OF EXPERIMENTAL WORK

The main objectives of the experimental work were to:

- 1 Identify the modes of wave generation: particularly for first and second waves;
- 2 Determine the relationship between the impact velocity of a rock-slide with the water and initial wave amplitude of the first wave (maximum amplitude);
- 3 Determine an evacuation time for personnel located at a pumping station; and
- 4 Determine the relationship between the impact velocity of the rock-slide with the water and the wave run-up distance.

3.2 PARAMETERS

Rock-falls are generally complex mass movements of material, not easily modelled in the laboratory. Block, piston and granular models are three methods used to model rock-falls or landslides. Such methods have been utilized in studies of historic slides. For the purposes of this research (particularly with application to the mining industry), a simple block model was chosen. Experiments considered a real-life geometry with the parameters shown in Table 1.

Table 1 Parameters considered in current research

Density of intact slide material (ρ_s)	2.8	tonnes/m ³
Still-water depth of water (h)	30	M
Maximum length of water body (x)	300	M
Failure slope angle (α)	50	Degrees
Run-up slope gradient (β)	1:10	-
Slide velocity range (v_s)	0.5 - 20	m/s
Slide tonnage	10000	Tonnes

3.3 SCALING LAWS AND PARAMETERS

A simplified model of the problem was created at a scale of 1:100 to enable a slide of about 10,000 tonnes to be modelled in the wave flume shown in Figure 2. The two major dimensionless parameters considered for scaling in open channel or free-surface flows are Froude (F) and Reynolds' (Re) numbers. With a model length scale of 1:100, the kinematic viscosity scale ratio would be 1/1000 which could not be satisfied. Therefore a 'distorted' model similar to an open channel or free-surface flow is needed, Young *et al.* (1).

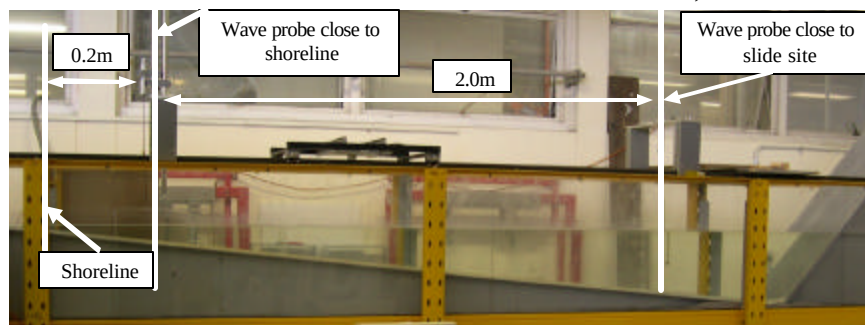


Figure 2 Wave flume used in experimental work

3.4 METHODS OF MEASUREMENT

The impact velocity was measured using a simple 'ticker-tape' measuring device which allowed measurement of velocity at 40Hz. This was a relatively quick, efficient and low-cost method, adequate for current requirements. The wave amplitude was determined both near the impact site and 0.2m from the model shoreline using wave probes. These provided information on variance in wave form and height, as well as the ability to measure the wave period and velocity.

4 RESULTS

Over 200 individual tests were undertaken in this research, to allow a reasonably complete and cohesive set of data to be generated.

4.1 RELATIONSHIP BETWEEN IMPACT VELOCITY AND INITIAL WAVE AMPLITUDE

The experimental results for wave amplitude at impact for 1st and 2nd waves are shown in Figure 3. The main observations that may be drawn are:

- Increase in impact velocity corresponds to an increase in amplitude of the first wave close to the impact site, with wave heights ranging from 1 to 2m; and
- Amplitude of the second wave close to the impact site is larger than the first wave, although there is significant scatter.

4.2 RELATIONSHIP BETWEEN SLIDE IMPACT VELOCITY AND WAVE PERIOD

Figure 4 shows the relationship between the slide impact velocity and the wave period, by considering the change in ratio of the first wave amplitudes. The main points of interest are:

- No noticeable change in the wave period for increasing slide impact velocities;
- Wave period of waves increase as they approach the shoreline, compared with near the impact site; and
- Wave period close to the shoreline decrease slightly in length with increasing impact velocity.

4.3 RELATIONSHIP BETWEEN SLIDE IMPACT VELOCITY AND WAVE RUN-UP HEIGHT

The relationship between the slide impact velocity and wave run-up height up the slope can be seen in Figure 5. Three key observations are made:

- Increasing impact velocity generally corresponds to an increase in wave run-up height up slope;
- For the range of velocities investigated, depending on the slide impact velocity, the wave run-up distance ranges between 30 and 40m; and
- There appears to be a plateau in the wave run-up height up slope for impact velocities greater than 14.5 m/s.

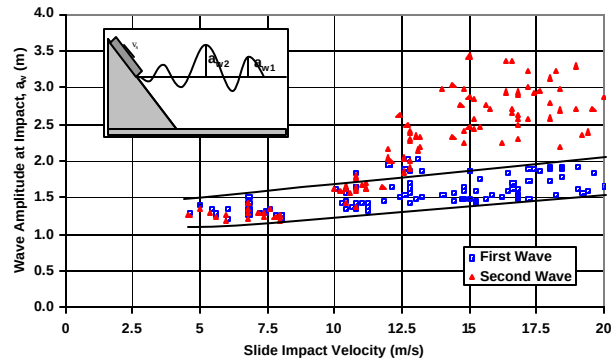


Figure 3 Initial wave amplitude vs. impact velocity

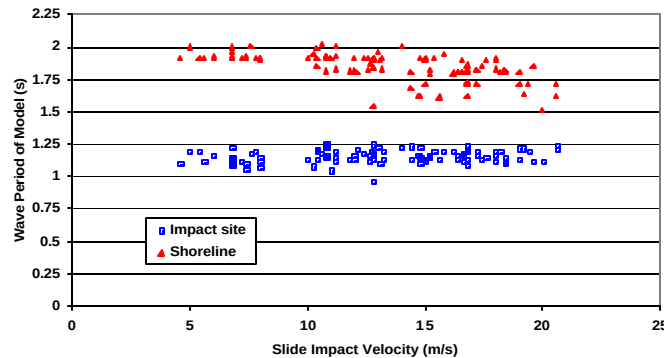


Figure 4 Wave period at impact site and shoreline

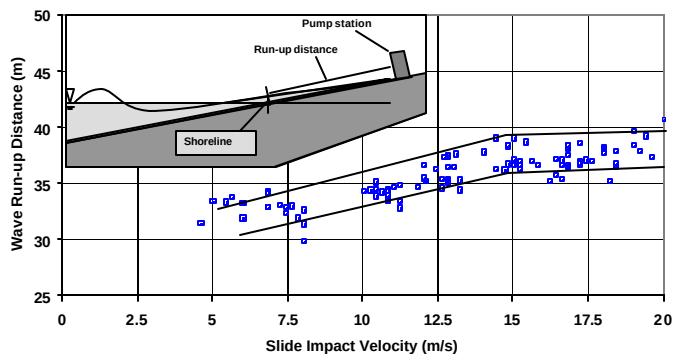


Figure 5 Experimental wave run-up height up slope

5 DISCUSSION OF RESULTS

The trend in results presented in Figure 3 are comparable to those from previous research, such as Davidson and McCartney (2). Fritz (3) considered a slide impact velocity range of 27 to 82m/s, and therefore it is suggested the current research may extend the understanding of slower slide impacts. The conclusions of Chaudhry *et al.* (4) were that “the heights of the generated waves were independent of the slide velocity”. This discrepancy may be attributed to different wave generation techniques. Chaudhry *et al.* (4) considered generation of waves by very low ‘impact force’ with the water, implying only slow impact velocities. Figure 1 clearly demonstrates the significant force of block impacting the water observed in the current investigation.

5.1 INVESTIGATION OF ENERGY TRANSFER

The principle result in Figure 3 is the relatively small increase in initial wave amplitude of the first wave for a relatively large increase in slide impact velocity. To investigate this observation further, a comparison between the energy of the slide and the first wave was undertaken. To determine the energy of the slide, it was assumed that at the impact of slide with water, all energy was kinetic, given by Equation 1:

$$E_k = \frac{1}{2} m v_s^2 \quad \text{Equation 1}$$

where E_k is the average kinetic energy of slide material and v_s is the slide impact velocity. To determine the energy of the wave, it was assumed that the first wave may be approximated by a solitary wave. Monaghan and Kos (5) and Fritz (3) also considered this as a reasonable assumption. The energy of a solitary wave may therefore be estimated using Equation 2:

$$E_{sol} = \frac{8}{3\sqrt{3}} \rho_w g \left(\frac{a}{h} \right)^{3/2} h^3 \quad \text{Equation 2}$$

where E_{sol} is the total average energy per unit length of wave crest and 'a' is the amplitude of the wave. Table 1 provides definitions for the other variables used. It should be noted that the generally accepted relationship given in Equation 2 does not include the slide width, b, and therefore has units of Joules per metre. To make comparisons with the energy of the slide at impact, the total kinetic energy given by Equation 2 is multiplied by the slide width to allow consistent units. Figure 6 demonstrates the efficiency of transfer of energy from the slide to the first wave for increasing impact velocities, with efficiency ranging from about 5 to 80%.

The energies of the slide and first wave were generated assuming that the energy of the first wave as approximately equivalent to that of a solitary wave. Figure 6 demonstrates the counter intuitive result that with increasing slide impact velocity, the efficiency of transfer of energy decreases. One possible explanation is that the energy is dissipated through turbulence and spray as illustrated in Figure 1. With higher impact velocities, the displacement of water in the form of an impact crater (as suggested by Fritz (3)) increases dramatically. By considering the efficiency of energy transfer from slide to water, it may be possible to explain why a relatively low increase in initial wave amplitude of the first wave for increasing slide impact velocity is observed. Furthermore, the efficiencies demonstrated in Figure 6 endorse the findings of Kamphuis and Bowering (6) and Monaghan and Kos (5) who suggest energy conversion from slide to water wave during impact ranging from 10 to 50%. Monaghan and Kos (6) propose that the energy transfer of 10% may present a lower bound. Figure 6 demonstrates that lower efficiencies are possible, suggesting the need for more detailed work in this area.

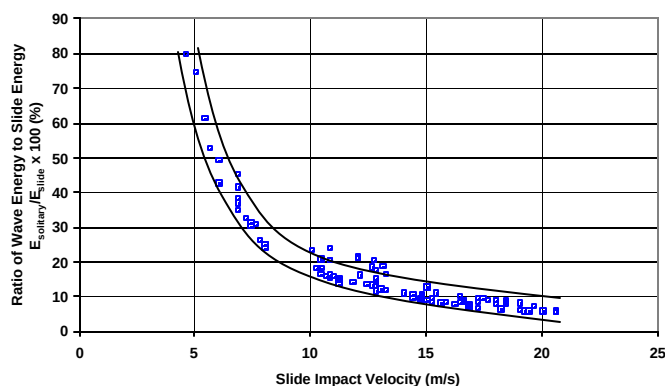


Figure 6 Efficiency of transfer of energy from slide to 1st wave

5.2 RELATIONSHIP BETWEEN SLIDE IMPACT VELOCITY AND WAVE PERIOD

From Figure 4, it can be seen that the wave period is independent of the slide impact velocity, demonstrated by the constant wave period. This follows the observations of Fritz (3) who concluded that there was a very poor correlation between the slide impact velocity and wave period. This has implications for the estimated time for evacuation of personnel. An approximation for the time taken by the wave to reach the shoreline is determined by calculating the velocity of the wave from the period. Figure 4 also demonstrates that the wave period increases close to the shoreline. Theoretically, this implies an increase in the wave velocity as the wave approaches the shoreline. However, current wave theories require the wave period to remain constant as the wave enters shallow water. By assuming conservation of energy, the wave length would decrease and the wave height increase due to the shallowing of the water depth.

An explanation of this discrepancy may be that the waves travel as a 'packet' (as shown by Figure 7), and thus the period of the first wave (measured in the current research) is not representative of the whole group. Further research is recommended to consider the waves as a 'packet', combined with Fourier transforms to better analyse the wave forms.

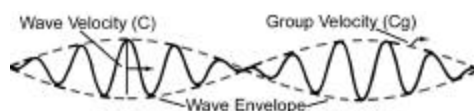


Figure 7: Representation of group velocity and 'packet' of waves (modified after Dean and Dalrymple (7))

5.3 CALCULATION OF EVACUATION TIME

To calculate evacuation times, the period of the wave close to the impact site was used to determine the initial velocity of the first wave. Assuming shoaling occurs, the group velocity can be predicted. The change in wave velocity with distance to the pumping station can be plotted, as demonstrated in Figure 8. This figure also shows the theoretical breaking depth for this wave, based on McCowan (8). Conservatively, the wave velocity after breaking was assumed to be equal to the breaking velocity, leading to an estimate of the time taken to travel the distance from the slide impact site to the shoreline of 37 seconds.

This is comparable to the results based on Fritz (3) which were also calculated, but not reported here. These can be considered a lower bound (conservative) result as the velocity of a broken wave is assumed to be constant from the point of breaking. In reality, the velocity of the breaking wave would decrease during run-up, therefore increasing the evacuation time. However, this lower bound indicates insufficient time for personnel to evacuate to safety. The use of wave buoys to provide warning signals is inadequate. It is therefore suggested that other methods, such as the construction of a windrow on the run-up slope, may be more appropriate. Further testing is needed to validate its effectiveness for wave energy dissipation.

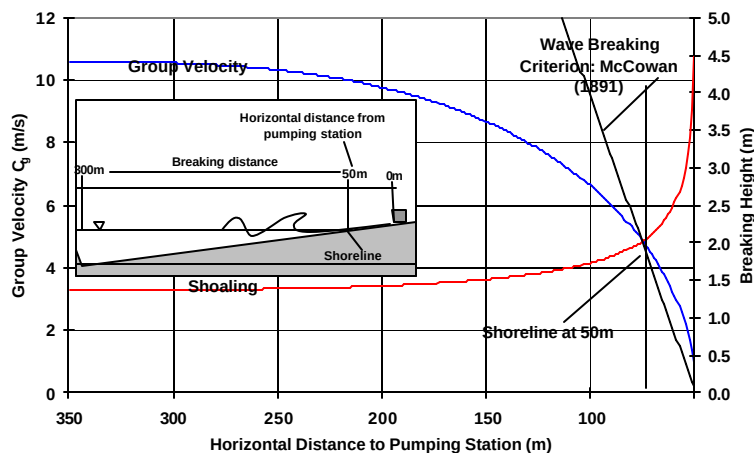


Figure 8 Shoaling, breaking and variation of group velocities

5.4 RELATIONSHIP BETWEEN SLIDE IMPACT VELOCITY AND WAVE RUN-UP DISTANCE

The observation that an increase in impact velocity corresponds to an increase in the wave run-up height up slope shown in Figure 5 was expected, and follows the findings of previous research. The plateau in the results may be an outcome of wave breaking that was observed for higher slide impact velocities during experimental work, and shown in Figure 9. Comparison with empirical wave run-up equations given by previous researchers (not presented here), show that the wave run-up of a nonbreaking wave is significantly higher than that of a breaking wave. This would be due to the amount of energy that remains within the wave, and which would be lost during the breaking process.

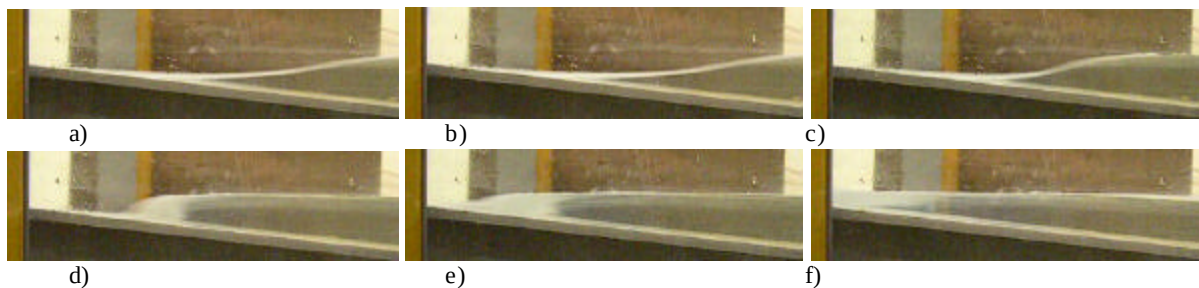


Figure 9 Photos from current experimental work showing wave breaking for high impact velocities (17m/s)

By considering the breaker type, it is of great interest to note that the plateau corresponds with the theoretical transition between a surging and breaking (plunging) wave (shown in Figure 10). Thus the change in wave run-up heights at a slide impact velocity of around 14.5 m/s (shown in Figure 5) may correspond to a change in wave breaker type. The relevance to the mining industry is that if a non-breaking wave is generated by a small rock-fall, then the resulting damage may be as significant as that caused by a larger rock-fall which breaks. Further work is required in this area to determine the magnitude of rock-fall that will generate waves that produce non-breaking waves.

In terms of providing information that is of use to the mining industry, it is demonstrated that a 10,000 tonne slide travelling at slide impact velocities of greater than 5 m/s are capable of generating waves that have a run-up distance of over 30 m for this slope angle.

This can be used in risk analyses to determining the location of structures such as a pumping station. If the example of a pumping station with maximum suction capacity of 50 m is used, then it is concluded that wave damage is probable if the pump's initial location is below 40 m, as demonstrated by Figure 5. However, as the water level decreases during the pumping process, the risks involved will decrease as well. This is based on the current assumption that a slide travelling at higher impact velocities is less likely than one impacting at slower velocities. This assumption needs further consideration before application in the mining industry.

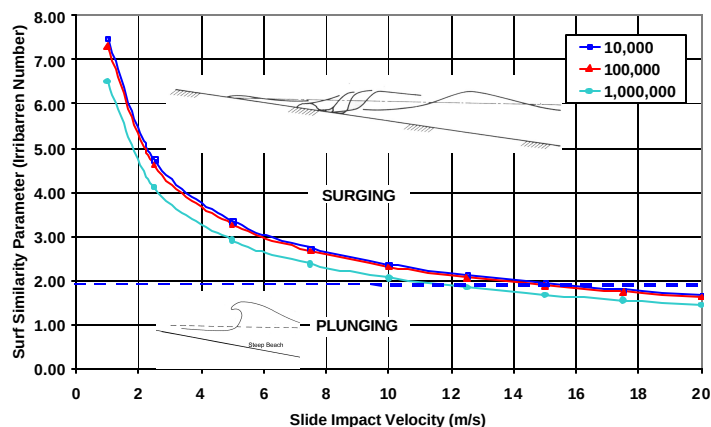


Figure 10 Breaking-type regions for waves of various slide tonnages

6 CONCLUSIONS

The potential for wave damage caused by the impact of rock-falls into water has been considered. The implications for open pit mines include:

- Transfer of energy efficiency from rock-fall to the water body ranges between 5 and 80% for high and low slide impact velocities respectively.
- Maximum wave run-up distance up-slope for a 10,000 tonne rock-fall is about 40m, corresponding with slide impact velocities between 14.5 and 20m/s;
- For a 10,000 tonne slide, the risk of structural damage and injury to personnel may be as significant for slower impact velocities, as for higher impact velocities based on wave breaker type; and
- The evacuation time for personnel located within 40m of the shoreline is inadequate. Devices such as a warning buoy do not give sufficient warning, and research into devices such as windrows to dissipate wave energy during wave run-up is proposed.

Although some solutions to the vast question of rock-falls into open pit mines containing water have been proposed, a great amount of research is still needed. The problem is multi disciplinary, and calls for collaboration between traditionally unconnected areas of research.

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8 REFERENCES

1. Young, DF, Munson, BR & Okiishi, TH (1997), 'A Brief Introduction to Fluid Mechanics', John Wiley & Sons, Inc
2. Davidson, DD & McCartney, BL (1975), 'Water waves generated by landslides in reservoirs', Journal of Hydraulics Division, ASCE, vol. 101, no. HY12, pp. 1489-1501
3. Fritz, HM (2002), 'Initial phase of landslide generated impulse waves' PhD thesis, VAW 178
4. Chaudhry, H, Mercer, AG & Cass, D (1983), 'Modelling of slide-generated waves in a reservoir', Journal of the Hydraulic Division, ASCE, vol. 109, no. 11, pp. 1505-1520
5. Monaghan, JJ & Kos, AM (2000), 'Scott Russell's wave generator', Physics of Fluids, vol. 12, no. 3, pp. 622-630
6. Kamphuis, JW & Bowering, RJ. (1970), 'Impulse waves generated by landslides', Proceeding of 12th Coastal Engineering Conference, ASCE, vol. 1, pp. 575-588
7. Dean, R.G. & Dalrymple, R.A. 1991, 'Water wave mechanics for engineers and scientists', Advanced Series on Ocean Engineering 2, World Scientific, Singapore.
8. McCowan, J (1894), 'On the solitary wave', Philosophical Magazine, Journal of Science, vol. 32, no. 5, pp. 45-58.

The Effect of Confining Pressure on the Behaviour of Railway Ballast Under Cyclic Loading

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During train loading, the railway substructure is subjected to loading amplitudes that can, over time, cause serious track alignment and stability problems. Railway substructure problems have become more serious and widespread in recent years due to the employment of faster and heavier trains. Railway authorities throughout Australia are faced with shortening maintenance cycles, and substantial increases in maintenance costs. With train speeds, loads and frequency expected to continue to increase, finding new and innovative ways to strengthen the substructure, in particular the ballast layer, is vital for the continued safe and economical use of both passenger and freight trains. This project investigates three possible ways in which the substructure can be strengthened; by a) altering the magnitude of confining pressure on the load bearing ballast, b) optimising the ballast particle size distribution, and c) employing the use of geosynthetics within the substructure as a form of reinforcement. In this paper, a large-scale triaxial apparatus is used to examine the effect of confining pressure on the degradation and settlement characteristics of ballast. It was found that increasing the confining pressure leads to a significant reduction in settlement but increased ballast breakage.

INTRODUCTION

The four most frequently encountered substructure problems on railway lines are ballast degradation (breakage), differential track settlement, subgrade pumping and ballast fouling. If any of these problems occur on a particular line, then it is usually the ballast layer that is most affected. The layout of the railway substructure is shown in Figure 1. The main roles of the ballast are to support the sleeper and prevent sleeper movement, distribute stresses at a reduced level to the subgrade, provide resiliency to the track structure, and facilitate drainage away from the track. If the ballast is unable to fulfill any of these roles, maintenance is usually the outcome. One such maintenance technique used to correct line and level is tamping. During tamping, a machine lifts the track to the required level and positions it laterally. The ballast under the sleepers is then squeezed into position using the tamping tines. Tamping is thought to be a main cause of ballast ageing [2], and as Figure 2 indicates, it is not very effective in the long term. Therefore, minimising the need for maintenance will save railway authorities financially and also lengthen the life of the track.

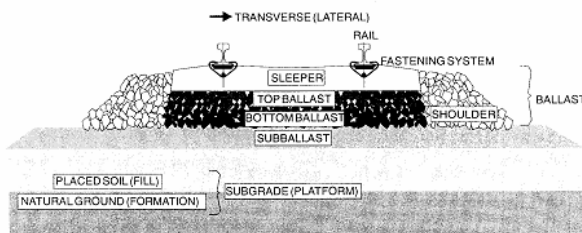


Figure 1 The Railway Substructure [1]

With these problems in mind, this paper presents the results from a series of triaxial experiments aimed at finding ways to improve the settlement response of the ballast layer, but at the same time ensuring that drainage and ride comfort are not compromised. The project is comprised of three main areas of study, but only the first is presented in this paper:

- The effect of the confining pressure on the degradation and deformation characteristics of the load bearing ballast (the ballast beneath the sleepers);
- The effect of ballast particle size distribution on track stability; and
- The potential use of geosynthetics within the substructure to prevent degradation, settlement and lateral spread.

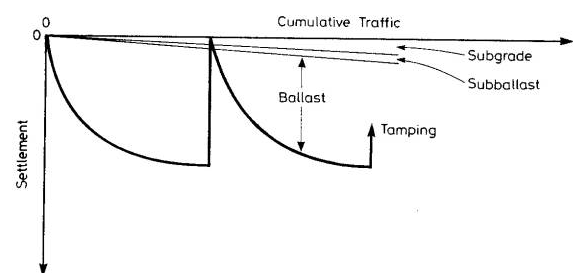


Figure 2 Track Settlement and the Influence of Tamping [3]

BEHAVIOUR OF GRANULAR MATERIALS UNDER CYCLIC LOADING

Settlement

Figure 2 shows that most settlement on a railway line is due to the accumulation of plastic strains within the

ballast layer. Permanent deformation within the ballast is the result of three phenomena, shear strain, volumetric strain accumulation, and ballast degradation. The ballast layer is prone to settlement for two reasons, a) it is loosened during maintenance (unlike the subballast and subgrade layers), and b) it is subjected to the highest stress ratios. Stewart [4] identifies that cumulative permanent strain is dependent on the permanent strain during the first cycle, the loading amplitude, and the number of cycles. Using variable amplitude triaxial tests, Stewart found that the maximum load determines the ultimate magnitude of plastic deformation.

A large number of researchers who have investigated the cyclic loading of granular materials (e.g. [5,6]) have found that the overall settlement response is governed by what is known as the “shakedown load”. Granular materials will either shakedown (the behaviour becomes purely elastic or reversible after a certain number of cycles), or will incrementally collapse (permanent strains increase rapidly, and the response is always plastic or irreversible). The shakedown load is the load below which behaviour becomes elastic. Ballast life is related to its resistance against incremental failure [7]. In a study by Brown [8], it was found that permanent and resilient strains reached equilibrium by 10^4 cycles.

Confining Pressure

The effect of confining pressure on the behaviour of granular materials under static loading is well understood. Figure 3 illustrates the relationships between volumetric strain and deviator stress in relation to axial strain under different confining stresses. For ballast and other granular materials, the peak deviator stress $(\sigma_1' - \sigma_3')_p$ increases with confining stress, but the peak principal stress ratio $(\sigma_1'/\sigma_3')_p$ decreases. As the confining pressure (σ_3') increases, the volumetric strain decreases and changes from dilation towards compression. Because dilation is suppressed, the breakage of particles is larger at high confining pressures [10].

Most papers investigating the effect of confining pressure on the cyclic behaviour of granular materials deal with undrained behaviour and liquefaction under earthquake loading. There are few studies that examine behaviour under drained conditions. Uzan [11] and Brown [8] found that under cyclic loading, axial strain decreases with increasing confining pressure. Raymond and Williams [12] conducted saturated drained repeated loading tests on ballast and found that the volumetric strain increases with confining pressure, although results were based on only two confining stresses. They indicated that the rates of strain during extension tests at $\sigma_3'=100$ kPa were approximately twice those at $\sigma_3'=50$ kPa.

Breakage

It has been reported that the primary cause of ballast contamination is from the breakdown of the aggregates themselves [13], and that this breakdown accounts for 40% of fouling material [1]. Ballast crushing not only leads to decreased track drainage due to the occupation of voids by fines, but also contributes to differential track settlement, lateral deformation and decreased shear resistance. Breakage is a common occurrence despite the fact that in-situ stresses are significantly smaller than the unconfined compressive strength of the parent aggregate.

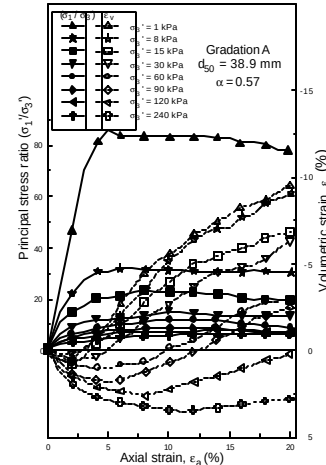


Figure 3 Behaviour of Railway Ballast under Static Loading Conditions [9]

It has been identified that the degradation of ballast particles can occur in three ways (Raymond and Dyaljee, 1979ab; Lees and Kennedy, 1975):

- The fracture of particles into approximately equal parts (this affects the long-term stability and safety of the track)
- The breakage of angular projections (influences the early settlement and track strength)
- The grinding off of small-scale asperities (introduces fines and causes fouling and reduced drainage)

Indraratna *et al.* [9] provide details on the steps involved in the deformation and degradation of ballast under cyclic loading. The first phase (initial 20000 cycles), often referred to as the stabilisation stage, involves the dynamic packing and sliding of particles relative to each other to form a compact aggregate. A consolidation stage occurs for the next 100000 cycles, during which the further rearrangement and packing of particles results in only minor ballast settlement. After this period, sharp edges and irregularities on the surface of the particles begin to break off at the contact points. Upon additional load increases and cycles, the relatively weak particles within the matrix, usually the large angular particles, start to break. Particle packing and settlement during this stage is much less than that corresponding to the stabilisation stage. Local crushing at one point causes a local redistribution of stress and a movement of particles

which results in crushing at a new location [16, 17]. This in turn produces differential settlement [18]. According to Jeffs [19], any further long-term breakage can be attributed to the additional deformation of larger particles and interparticle grinding processes.

Breakage Quantification

Several methods are available in the literature to quantify the degradation of granular materials. Two methods have been found to be suitable for the analysis of railway ballast, namely those proposed by {Marsal 1967 #1190} for rockfill and {Hardin 1985 #490} for sands. Hardin defines breakage using three parameters, the breakage potential B_p (Equation 1), the total breakage B_t (Equation 3) and the relative breakage B_r (Equation 4). All breakage is referenced to the largest silt size, 0.074 mm. However, in the current study, all breakage is referenced to 1 mm due to the large size of the ballast particles, and breakage is quantified in terms of B_r .

$$B_p = \int_0^1 b_p df \quad (1)$$

where b_p is the potential for breakage of a particle of size D given by Equation 2.

$$b_p = \log_{10} \left[\frac{D(\text{mm})}{\text{reference size}(\text{mm})} \right] \quad (2)$$

$$B_t = \int_0^1 (b_{p0} - b_{pl}) df \quad (3)$$

where b_{p0} and b_{pl} are the values of b_p before and after testing, respectively.

$$B_r = \frac{B_t}{B_p} \quad (4)$$

PREDICTION OF THE RELATIONSHIPS BETWEEN SETTLEMENT, CONFINING PRESSURE AND BREAKAGE

It is hypothesised that the confining pressure will be related to the breakage and settlement by the relationships shown in Figure 4. Figure 4 shows that at low confining pressures, breakage is significant. This is because at small confining pressures, ballast particles have a low coordination number, resulting in large stress distributions at the contact points. Particles are also free to move and slide over one another resulting in the weak corners sustaining the deviator stresses. As the confining pressure is increased, it is thought that breakage will decrease to a minimum value. This decrease in breakage will be due to an increase in the coordination number, and a reduction in particle rearrangement. Behaviour in this region is characterised by a stabilisation zone where further increases in breakage will be due to the continued shearing of corners and weak points. Both settlement and

breakage remain approximately constant within this zone. Breakage is more pronounced at high confining pressures and this is largely due to the suppression of dilation. Particles will no longer be able to slide past one another due to the high lateral restraint. This process will continue until the point A is reached which signifies the onset of severe breakage.

It is well known that settlement decreases as the confining pressure increases due to the inability of particles to rearrange and change position. However, in Figure 4, once point A is reached, the magnitude of particle crushing significantly influences the settlement characteristics. The breakage corresponding to point A is characterised not only by the breaking of corners, but also the fracture of particles.

It is quite clear from Figure 4 that high confining pressures restrict settlement but result in increased breakage. Therefore, one of the aims of this study is to identify the range of confining stresses that define the stabilisation zone, so that both breakage and settlement criteria can be satisfactorily met.

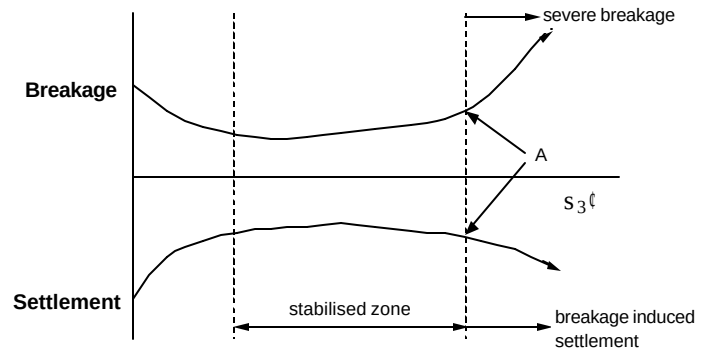


Figure 4 Hypothesised Effect of Confining Pressure on Breakage and Settlement

TESTING PROCEDURE

When studying the behaviour of railway ballast, scaled down aggregates do not give an accurate indication of the behaviour of full sized particles. For this reason, the use of large-scale equipment is necessary. A large-scale cylindrical triaxial apparatus (Figure 5) that can incorporate samples of size 300 mm diameter and 600 mm height has been utilised for ballast testing. A newly installed hydraulic dynamic actuator capable of applying loads up to 150 kN and frequencies of 60 Hz is used to simulate the train loading.

Tests were carried out on latite basalt, a quarried aggregate from Bombo in NSW. The basalt was prepared according to the particle size distribution shown in Figure 6. Also shown in Figure 6 is the distribution specified in the Australian Standard for Railway Ballast [22]. Triaxial specimens were compacted inside a 6 mm rubber

membrane using a vibratory hammer, with a final achieved bulk density of approximately 15.3 kN/m^3 ($G_s=2.7$ and $e_0 \approx 0.82$). Following this, samples were saturated under a back pressure ($B > 0.96$) and consolidated overnight.

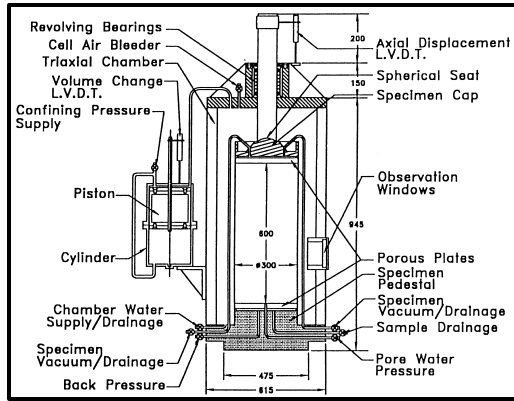


Figure 5 Large Scale Cylindrical Triaxial

Following consolidation, the appropriate confining pressure was applied, and an initial static load equivalent to the mean deviator stress was applied. The first 20 cycles were applied at a frequency of 1 Hz, and the remaining cycles at 20 Hz. A total of 500000 cycles were employed. Data in relation to settlement and volume change was recorded at regular intervals for the entire test. The ballast was resieved after the test to quantify the amount of particle breakage.

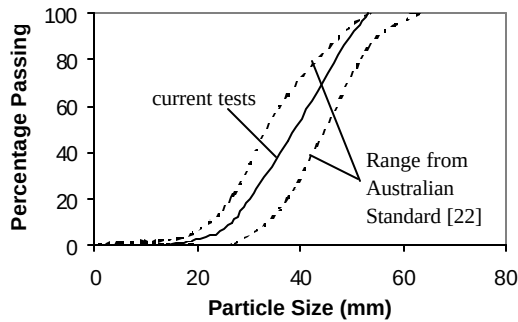


Figure 6 Particle Size Distribution used in Tests

Loading Characteristics

Six triaxial compression tests were conducted at effective confining pressures ranging from 1–240 kPa. The confining pressures supplied to the load bearing ballast are usually within the range 21–83 kPa [23]. The load was cyclically varied between two compressive stresses of 45–245 kPa. The 245 kPa pressure corresponds to the contact pressure between sleeper and ballast that would result from a static 25 tonne axle load, assuming 50% of the wheel load passes through the rail seat, and the effective contact area outlined in Jeffs and Tew [24] for

concrete sleepers is utilised. Traditionally, this static load is multiplied by a dynamic impact factor to take into account dynamic forces due to train speed, and track and wheel irregularities (for example, see [25]). No dynamic impact factor was applied in this case, so the loading scheme represents the lower bound stresses that would be applied by a 100 tonne freight wagon. The upper bound case will be examined in future testing. However, the load scheme applied in the present study is also equivalent to an average passenger train (50 tonne carriage) multiplied by a dynamic impact factor of 2. The lower pressure (45 kPa) was chosen to represent the effective weights of the rail and sleepers. The testing frequency (20 Hz) corresponds to different train speeds depending on the assumed distance between train axles. For instance, in the case of the NSW Millennium train with axle spacing 2.3 m, the simulated train speed is 165 km/hr.

RESULTS AND DISCUSSION

The axial strain results for the cyclic triaxial tests are shown in Figure 7, and, as expected, the axial strain decreased with an increase in confining pressure. The settlement characteristics are best described by Equation 5, where ϵ_a is the axial strain, N is the number of cycles, and A and B are regression coefficients. After approximately 25000 cycles all samples have reached the maximum axial strain values, and following this point behaviour is essentially elastic (shakedown has occurred for all specimens, and settlement is within the stable zone shown in Figure 4). Further increases in axial strain can only occur with increases in the deviator load or frequency.

$$\epsilon_a = A + B \ln(N) \quad (5)$$

Figure 8 shows the relationships between the confining pressure and the maximum values of axial, volumetric and radial strains achieved at the end of 500000 cycles. The change in axial strain with effective confining pressure can be described by Equation 6, where C is a coefficient (equal to 30 kPa in this case) and $R^2 = 0.95$.

$$e_a = \frac{C}{\sqrt{s_3}} \quad (6)$$

Comparing the axial strain curve in Figure 8 to Figure 4, it is clear that in the experiments, point A was not reached i.e. there was no large scale crushing of particles. This signifies that the deviator stress will need to be increased in future tests (to the upper bound case detailed previously) so that larger levels of breakage will be attained and point A can be determined.

Figure 8 also shows that for confining pressures greater than 30 kPa, the volumetric strain increased with increasing confining pressure. The volumetric strain for the 1 and 8 kPa samples were both highly dilative, but the 1 kPa result has not been shown due to difficulties encountered in measuring the volume change at such a low confining pressure. Radial strains were strongly expansive at small confining pressures, and at 240 kPa the radial expansion due to the cyclic loading was virtually cancelled out by the compression caused by the large confining pressure.

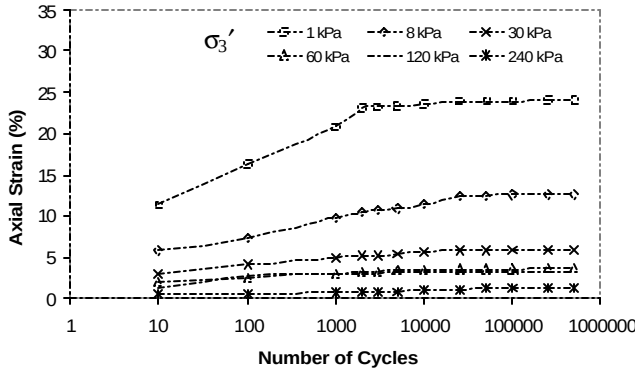


Figure 7 Axial Strains at Different Confining Pressures

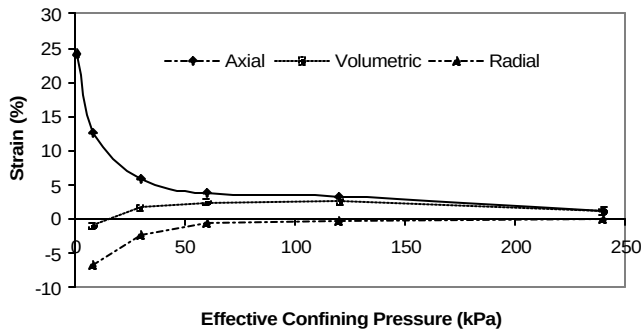


Figure 8 Relationships Between Confining Pressure and Maximum Values of Axial, Volumetric and Radial Strains

Ballast Degradation

The degradation results expressed in terms of the relative breakage B_r (Equation 4) are shown in Figure 9. Breakage is significant at low confining pressures (as predicted in Figure 4), and there appears to be an optimum confining pressure at around 30 kPa. Once again, however, particle breakage was too low for the critical point A to be reached. For this reason, these results need to be verified under a larger deviatoric loading scheme. Rapid axial strains were achieved for the 8 kPa and 1 kPa tests (Figure 7), which is largely responsible for the increased breakage at these confining pressures.

APPLICATIONS TO THE RAILWAY INDUSTRY

The main conclusion to be drawn from the current test series in relation of train loading is that passenger trains, or freight trains travelling under steady state conditions, do not cause significant ballast degradation. Even though normal pressures as high as 480 kPa were applied to the ballast, only the breakage of angular corners was achieved. There was no sign of particle fracture into approximately equal parts. It may be concluded that there can only be two possible causes of ballast degradation on a railway line, breakage caused by heavy freight trains with impact loading, or breakage caused by maintenance procedures, in particular tamping.

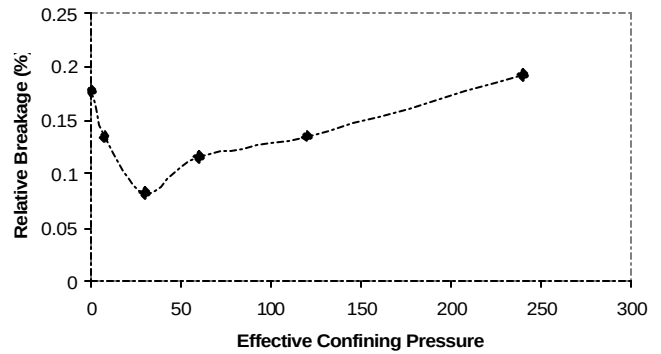


Figure 9 Effect of Confining Pressure on Relative Breakage

FUTURE TESTING

This research has shown conclusively that a maximum dynamic deviator stress of 245 kPa is insufficient to cause large-scale ballast damage. Several railway authorities provide values for the maximum contact pressures that are allowed between the sleeper and ballast. According to FIP [26], most railway authorities allow pressures of 400–500 kPa on the ballast layer. Pressures larger than this cause excessive ballast degradation and unsatisfactory track settlement. The Australian Standard for Prestressed Concrete Sleepers [27] states that ballast pressure should not exceed 750 kPa for high-quality, abrasion-resistant ballast. For this reason, the next set of experiments conducted on the cylindrical triaxial will apply double the current deviator stress (490 kPa) and the same range of confining stresses (1–240 kPa).

Following an increase in deviator stress, layer/s of geosynthetics will be placed within the ballast sample to investigate the effectiveness of geosynthetics in reducing breaking and settlement. Using another piece of large-scale equipment (cubical triaxial rig, samples 600×600×800 mm), previous studies at the University of Wollongong have shown that when geogrid-geotextile composites are placed at the subballast-ballast interface, reductions in settlement and lateral strain are observed. It is thought that by placing such geosynthetics within the

cylindrical ballast sample, the lateral confinement will be increased, and this will affect settlement and breakage characteristics.

Testing is also continuing on the large-scale cubical triaxial. The effect of altering the lateral confinement (whilst maintaining plane strain conditions in the longitudinal direction) is under examination. A quantification of the effect of geosynthetics incorporation on the degree of confinement is also under consideration.

CONCLUSIONS

According to the results of experiments on ballast using a large-scale triaxial rig, it can be concluded that:

- the axial strain decreases with increasing confining pressure under cyclic loading
- the deviator stress applied was below the shakedown limit and did not cause the fracture of particles
- the volumetric strain was dilative below 10 kPa, and at higher confining pressures increased with confining pressure
- minimum ballast breakage was achieved at around 30 kPa
- higher values of deviator stress are required to determine the value of the critical point A (see Figure 4).

ACKNOWLEDGEMENTS

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REFERENCES

1. SELIG, E.T. and WATERS, J.M., "Track Geotechnology and Substructure Management", Thomas Telford, London, 1994.
2. SCHMUTZ, G., "Ballast and Recycling Spent Ballast", Rail International, 2000, 31 (7), 2-7.
3. BROWN, S.F. and SELIG, E.T., "The Design of Pavement and Rail Track Foundations", Chapter in *Cyclic Loading of Soils: from Theory to Design*, Blackie and Son Ltd, New York, 1991, 249-305.
4. STEWART, H.E., "Permanent Strains from Cyclic Variable-Amplitude Loadings", J. Geotech. Eng., 1986, 112 (6), 646-660.
5. SHARP, R.W., and BOOKER, J.R., "Shakedown of Pavements Under Moving Surface Loads", J. Trans. Eng., 1984, 110 (1), 1-14.
6. COLLINS, I.F., and BOULBIBANE, M., "Geomechanical Analysis of Unbound Pavements Based on Shakedown Theory", J. Geotech. Geoenv. Eng., 2000, 126 (1), 50-59.
7. LEKARP, F., and DAWSON, A., "Modelling Permanent Deformation Behaviour of Unbound Granular Materials", Construction and Building Materials, 1998, 12 (1), 9-18.
8. BROWN, S.F., "Repeated Load Testing of A Granular Material", J. Geotech. Eng. Div., 1974, 100 (GT7), 825-841.
9. INDRARATNA, B., IONESCU, D., and CHRISTIE, D., "State-of-the-Art Large Scale Testing of Ballast", Railway Technology for the 21st Century, Conf. on Railway Eng., 2000, 24.1-24.13.
10. INDRARATNA, B., SALIM, W., IONESCU, D., and CHRISTIE, D., "Stress-Strain and Degradation Behaviour of Railway Ballast Under Static and Dynamic Loading, Based on Large-Scale Triaxial Testing", Proc. 15th Int. Conf. on Soil Mechanics and Geotech. Eng., 2001, 2093-2096.
11. UZAN, J., "Granular Characterization for Mechanistic Pavement Design", J. Trans. Eng., 1999, 125 (2), 108-113.
12. RAYMOND, G.P. and WILLIAMS, D.R., "Repeated Load Triaxial Tests on Dolomite Ballast", J. Geotech. Eng. Div., 1978, 104 (GT7), 1013-1029.
13. CHRISMER, S.M. and READ, D.M., "Examining Ballast and Subgrade Conditions", Railway Track and Structure, AREA, 1994, 39-42.
14. RAYMOND, G.P. and DIYALJEE, V.A., "Railroad Ballast Load Ranking Classification", J. Geotech. Eng. Div., 1979, 105 (10), 1133-1153.
15. LEES, G., and KENNEDY, C.K., "Quality, Shape and Degradation of Aggregates", J. Eng. Geology, 1975, 8, 193-209.
16. SOWERS G.F., WILLIAMS, R.C., and WALLACE, T.S., "Compressibility of Broken Rock and the Settlement of Rockfills", Proc. 6th Int. Soil Mech. and Found. Eng., 1965, 2, 561-565.
17. BILLAM, J., "Some Aspects of the Behaviour of Granular Materials at High Pressures", Stress-Strain Behaviour of Soils, Proc. Roscoe Memorial Symp., 1971, 69-79.
18. IONESCU, D., INDRARATNA, B., and CHRISTIE, D., "Behaviour of Railway Ballast Under Dynamic Loads", 13th Southeast Asian Geotechnical Conf., 1998, 69-74.
19. JEFFS, T., "Towards Ballast Life Cycle Costing", The 4th Int. Heavy Haul Railway Conf., 1989, 439-445.
20. MARSAL, R.J., "Large Scale Testing of Rockfill Materials", J. Soil Mech. and Found. Div., ASCE, 1967, 93 (2), 27-43.
21. HARDIN, B.O., "Crushing of Soil Particles", J. Geotech. Eng., 1985, 111 (10), 1177-1192.
22. STANDARDS AUSTRALIA, "Aggregates and Rock for Engineering Purposes", AS 2758.7-1996, Sydney, NSW, Australia.
23. ALVA-HURTADO, J.E., "A Methodolgy to Predict the Elastic and Inelastic Behaviour of Railroad Ballast", 1980, Report No. OUR80-240D, Dept. of Civil Eng. Uni. Of Massachusetts, Massachusetts, USA.
24. JEFFS, T., and TEW, G.P., "A Review of Track Design Procedures, Volume 2- Sleepers and Ballast", BHP Research Melbourne Laboratories, Melbourne, 1991.
25. BROADLEY, J.R., JOHNSTON, G.D., and POND, B., "The Dynamic Impact Factor", Railway Engineering Conf., Sydney, Institution of Engineers, 1981, 87-91.
26. FIP Commission on Prefabrication, "Concrete Railway Sleepers", Thomas Telford, London, 1987.
27. STANDARDS AUSTRALIA, "Railway Permanent Way Material. Part 14: Prestressed Concrete Sleepers", AS 1085.14 -1997, Sydney, NSW, Australia.

A Consultant's Perspective on Design and Build Tunnelling Projects in Hong Kong

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ABSTRACT

While aspects of geotechnical design are particularly challenging in the Hong Kong Special Administrative Region (HKSAR) with regards to building in the physical environment, a consultant's role in the management of design and build tunnelling projects can be as equally demanding.

This paper will discuss the prescriptive nature of the HKSAR government approval process and the impact that this can have on innovative design. On Design and Build (D & B) projects the process of concurrent design puts even greater demands on consultants acting as the intermediary between the contractor and government approval departments.

Aspects of managing business relationships and working within the government approval system will also be discussed within the context of the authors experience on the China Light and Power HK Limited (CLP) Tsz Wan Shan cable tunnel project.

INTRODUCTION

Job Description

As part of the ongoing upgrading of the power supply network for CLP, construction of two new cable tunnels at Tsz Wan Shan and Kwai Chung in the HKSAR were proposed. Dragages HK Joint Venture (DJV) were appointed as the contractor to undertake construction of the cable tunnels and associated works. The discussion in this paper will focus on the 650m long bored tunnel at Tsz Wan Shan.



Fig 1. Tsz Wan Shan and Kwai Chung Tunnel Locations

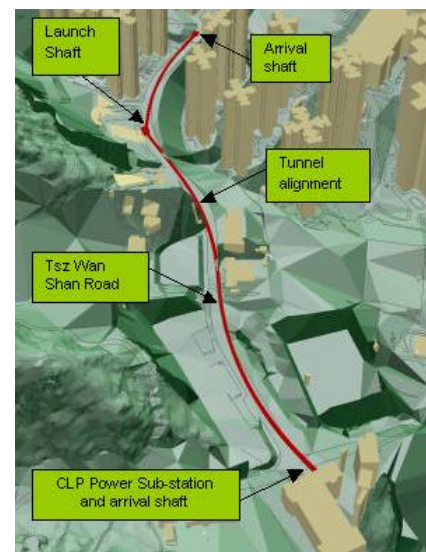


Fig 2. Tsz Wan Shan Cable Tunnel Route

The tunnel is being excavated using a Bessac Tunnel Boring Machine (a boom head TBM with pressurised face) and lined with a pre-cast concrete segmental lining. The Tsz Wan Shan cable tunnel runs either at the toe or along the crest of a number of registered slopes, and the low cover combined with highly variable soil stiffness and ground water level have had major influences on the design.

The temporary works for the launch and retrieval shafts have been formed by a combination of sheet-pile, pipe-pile, soldier-pile and composite-pile walls, with the composition changed to suit site constraints and avoid existing utilities where required.

Relevant Parties

Other than the Client and Contractor as previously discussed, parties relevant to the Tsz Wan Shan cable tunnel project are; Ove Arup & Partners Hong Kong Ltd (Arup) acting as the contractors design consultant, Atkins China Limited acting as the Clients Engineer, and Maunsell Group in the role of Independent Checking Engineer (ICE).

Tsz Wan Shan Cable Tunnel Design Development

In accordance with the conforming design at tender, a cut and cover proposal was submitted by DJV along the entire length of the proposed route. Due to the constraints of cut and cover works with regards to the impact of extended design programme, public disturbance and environmental impact, particularly so in the HKSAR, the client encouraged alternative bids. The extended design programme was a key factor as the government approval process in the HKSAR only permits cut and cover works to be carried out on small sections of roadway at a time to minimise public disturbance. A pipe jacking tunnelling alternative was submitted by DJV as part of the tender design.

DJV subsequently were awarded the D & B tunnelling works based on the proposed pipe jacking alternative. However, during scheme design, the pipe jacking option was replaced by a more cost effective segmental lining. The selection of this method of construction came as a result of a series of front end value engineering meetings which facilitated open discussion and input from all parties involved in the D & B works.

Significant capital cost savings were generated by selection of the TBM over cut and cover method as DJV were able to use the Bessac machine on an associated tunnelling project in the region, in similar 'soft ground' soil conditions, prior to the commencement of works on the Tsz Wan Shan tunnel project.



Fig 3. Bessac TBM



Fig 4. Segmental Tunnel Lining

HKSAR DESIGN APPROVAL PROCESS

While there are many government departments in the HKSAR with which a tunnelling D&B team may interact, it is through the Buildings Department (BD) that approval must be obtained. Specialist consultation to BD is also often provided by other government departments. One such department that the design consultants on D & B tunnel projects must liaise with is the Geotechnical Engineering Office (GEO).

Buildings Department

Any party that intends to carry out building works in the HKSAR is required by law to appoint an authorized person, and where necessary a registered structural engineer, to prepare and submit plans for the approval of BD. There are however exceptions to this in the form of BD approval exemptions, permitted to government projects and some quasi-government projects such as MTRC and KCRC railways. In many of these cases BD consultation is required but approval is not. This has been common in the past given the relatively few number of private sector D & B tunnelling projects in the HKSAR.

Geotechnical Engineering Office

GEO is the government geotechnical arm of the Civil Engineering Department (CED), which provides recommendations to BD on geotechnical aspects of tunnelling and shaft design.

Outlined in the flow chart below are the stages in the BD approval and consent process.

Detailed Design Approval Process

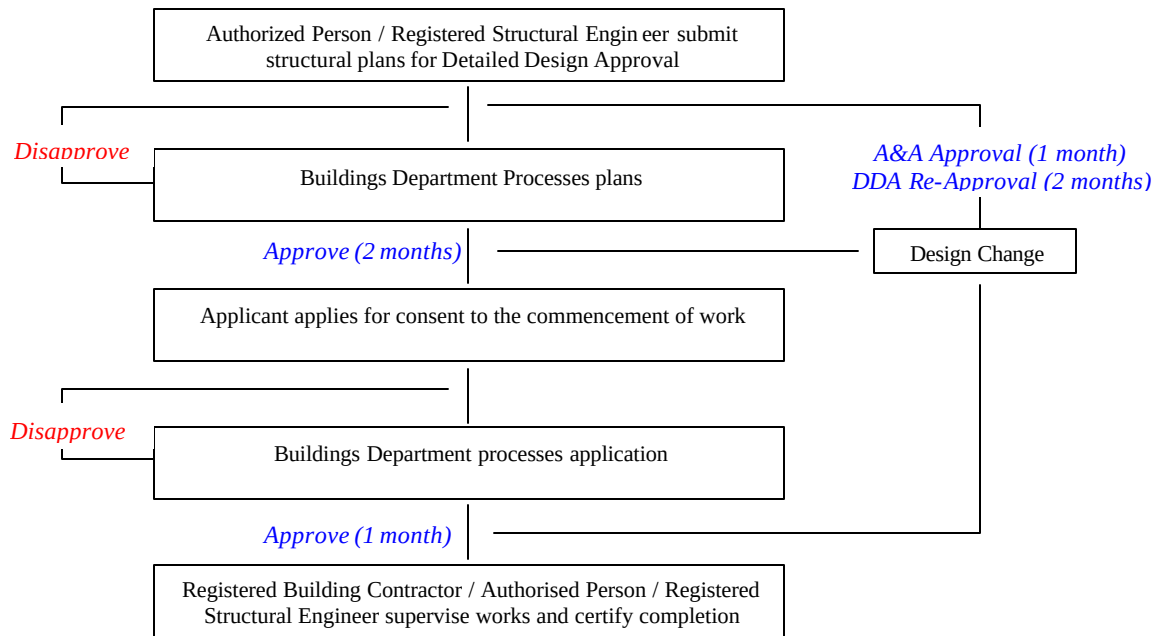


Fig 5. Buildings Department Design Approval Process

While the BD design approval process outlined above appears straight forward, it is the duration of each approval and consent stage that can impact on the desirable efficiencies of a D & B project. The first stage of the approval process, Buildings Department Detailed Design Approval (DDA), requires a 2 month processing period. Should the initial application be rejected, application for approval must be resubmitted for a further 2 months of processing. Future Alterations and Additions (A&A), referring to minor design changes, must be submitted to BD in the form of an amended submission. These require a 1 month processing period. Significant changes to the approved design require a complete re-submission for DDA. Re-submission for DDA similarly requires a 2 month processing period.

Applications for Consent to Commence Works

As indicated in Figure 5, application for consent to commence works requires a 1 month processing period by BD. However, as in the example of Tsz Wan Shan shaft design and construction, multiple applications for consent are required. Application for consent to commence installation of piling works is first required. Consent to commence excavation following these works must be applied for separately and application for consent for this stage may not be carried out until completion of the previous stage of works, as built drawings are submitted, and any other requirements are fulfilled such as pumping tests.

This clearly indicates the lengthy timeframe and requirements of the approval process and how this is not ideally suited to the tight programme restrictions of a D&B contract. While it appears that there are no obvious alternatives to this process, it is a practical matter of making allowances for time delays and modifying the design and construction management process to accommodate.

On a D & B project however, it is sometimes difficult to predict when a design change will be required and it is very easy for a 2 month period to be wasted on an unforeseen amended submission followed by a reapplication for consent to proceed. This may come about for example if design piling depths are not reached during installation for shaft Excavation Lateral Support (ELS) works. ELS refers to the temporary works involved in shaft design and construction. Failing to reach design pile tip elevations will call for an amended submission under BD guidelines and a reapplication for consent to proceed.

One of the reasons for the HKSAR's tight engineering controls imposed by govt departments is believed to have been brought about largely by "sins of the past" of corrupt building contractors and the break down of the certification process. A recent example of this was the practice of "short piling" by some contractors in the late 1990's. This has led to a prescriptive design approach which impacts on a design consultant's ability to develop innovative, challenging and efficient designs. To this end, BD release highly conservative design guidelines (Practice Notes for Authorised Persons – PNAP (1)) which are expected to be strongly adhered to. Similarly conservative design guidelines are produced by GEO in the form of the Geoguide Series (2).

RELATIONSHIP MANAGEMENT

Arguably the most important aspect of D & B contracts in the HKSAR is relationship management. While the importance of symbiotic management relationships between the client, contractor and designer in the D & B team is discussed and accepted on an international level, the importance of building strong working relationships to work within the approval system that governs private sector projects in the HKSAR is paramount.

In this design environment, where it is essential to minimise the likelihood of future changes in design, the necessity for close communication between the client contractor and design consultant, particularly at the front end of a D & B project is imperative. Early involvement of design consultants and contractors, and effective communication of client requirements, are important to close out as many design issues as possible and set as many aspects of the design as possible to avoid unnecessary re-engineering.

Whilst open and effective communication and the development of Relationship Management is essential at the front end, it should also be maintained and developed throughout the life of the D & B project. Major design changes, or continual minor changes, have the potential to greatly effect the economics of a project and cause work to be held up due to the timely BD approval process. While it is accepted that the concurrent nature of D & B contracts may not be economical within the HKSAR approval system, effective relationship management along with effective project planning both in terms of programme and finance, should minimise its impact.

DEVELOPING A CONTRACTOR/CONSULTANT DESIGN AGREEMENT

Fundamental to the detailed design consultancy is to establish a fair design agreement and clearly document the roles and responsibilities of the relevant parties in the D & B team throughout the project life. Considerable time spent at the front end on the elements outlined below will lead to significant avoidance of conflict and cost savings throughout the project.

Clear Scope of Works/Services

At the outset of a D & B project it is important to agree upon, and document, a clear scope of works. This should be fundamentally based on a detailed list of client requirements. The design consultant should also document their 'scope of services'. It is however important to emphasise to the contractor that the 'scope of works' is not the same as 'scope of services'. It should not be assumed that the consultant's ability to carry out certain services means that these services are included as part of the scope of works. These services should not be assumed to be part of the original design agreement. This is an element that has appeared on the Tsz Wan Shan and several other projects.

Designer/Contractor Obligations

The obligations of the Client, Contractor and Consultant should be outlined in detail prior to any consideration is given to the commencement of work.

One element of the design that is not often considered and often results in becoming the obligation of the consultant is the responsibility of miscellaneous minor tasks, or 'loose ends'. While this may be considered a minor concern, on smaller D & B projects this has the ability to eat away profits for the consultant who has notably smaller margins than the contractor. Such examples of miscellaneous minor tasks include the detailing of cabling brackets, as-built drawings, Operation and Maintenance Reports, and the outlining of rebar schedules.

When detailing the agreement it is important that the design consultant does not agree to the terms, or accept the obligation of performing 'the most economical design' or providing a design that is 'fit for purpose'. An efficient and economical design to an international standard, and produced within the time and fee constraints, is all that can be promised to a contractor. Agreement to most economical design is an unrealistic design agreement, particularly in the HKSAR where:

- i) there is not the fee nor the time luxury of carrying out highly economical and innovative design; and
- ii) there is not the ability of a design/contractor team to make economical variations to designs that require rapid approval to proceed.

At the completion of any job the contractor will have no problem in seeking out secondary advice on the "economical" nature of an 'as built' design when the limitations of the design process are forgotten.

As discussed by Garvey (3), "an adequate design in time is generally more useful and economic than a 'perfect' design at some unspecified future date". This could not be more accurate in the context of the HKSAR BD approval system.

Change Order Processing

It is important that early in the D & B process the Contractor and Consultant come to an agreement and document an efficient method for handling Change Orders (CO's). Further to this it is important that both parties appreciate that regardless of the demanding nature of carrying out concurrent design on D & B projects, particularly when facing the additional time pressure demands of rapidly carrying out amended designs to submit for BD approval, it is imperative that the contractor and consultant agree on variations prior to the commencement of additional works.

In the case of the Tsz Wan Shan Cable Tunnel a process for handling CO's was initially not well documented between the contractor and the consultant. The heavy demands of negotiating the BD approval process, where the needs to progress to maintain the design programme led to, in many cases, early progress on design variations prior to any documented agreement on the status of the work, and subsequently outstanding claims.

During the course of the project the contractor and consultant came to an agreement on the arrangement for CO payments. Three options were available for agreement prior to the commencement of works:

- i) Lump Sum agreement
- ii) Time basis
- iii) Lump Sum with BD Approval Commission

Option iii) entails an agreement for a portion of payment that represents the cost of performing the design work for the consultant. A further amount, which is agreed upon prior between contractor and consultant, for time spent in the approval process, is paid upon BD approval. This option was commonly adopted to act as an incentive for the consultant to progress through approval process as quickly as possible. Options i) and ii) were outlined in the design agreement and option iii) was used in practice on site for efficient handling of design changes with BD.

Design/Deliverables Programme

It is essential, particularly in the HKSAR, to include a detailed deliverables program into the design agreement. Clear indication of milestones and the end date will be beneficial for all parties in fairly processing CO claims as well as effectively monitoring progress.

Document the Assignment of Liability

The assignment of liability should be clearly documented in the project contractual documents. Typically the liability for D & B contracts resides largely with the contractor, as is the case for DJV on the Tsz Wan Shan D & B contract. The greatest element of risk in a D & B tunnel project is unforeseen ground conditions and the design consultant should ensure that this is clearly defined in the contractual documentation as a contractor liability. Resulting design changes should be clearly identified as in or out of the designer's scope of works to allow the risk to be assessed.

BENEFITS OF DESIGN AND BUILD CONTRACTS

Outlined below are the typical benefits that are associated with D&B projects. Where relevant, discussion is provided on how these common benefits are affected by the HKSAR D & B environment.

1. It can provide both flexibility and innovation in design – This is one of the greatest benefits of D & B projects as it takes advantage of the rapid and free flow of information between the Client, Contractor and Consultant. As previously discussed this flexibility and innovation is limited in the HKSAR due to the government buildings approval process;
2. An efficient design can be provided – While the D & B project in the HKSAR may not be considered 'efficient' with regards to operating within the approval system, it is actually the efficiencies within the D & B project setup that minimise the effects of the approval process on the project programme;
3. It is possible to provide a more buildable design as a result of early contractor and consultant involvement. This was found to be particularly true in the case of the Tsz Wan Shan cable tunnel;
4. The overlap of design and construction programmes leads to a considerably shorter delivery time – While this is true it can also lead to inefficiency in the HKSAR where construction works are placed on standby due to disruptions in the approval process or amendments to design;
5. It provides a single point of contact for the client to the contractor by combining contractor/sub consultants into a single group;
6. The risk is largely transferred from the client to the contractor. This is beneficial in that the contractor is in the best position to manage it's own risk. The main interface is now between the contractor and the designer;
7. The contractor's own plant, preferred methods and expertise are maximised.

FURTHER CONSIDERATIONS FOR THE DESIGN CONSULTANT

Factor into the Bid the Cost of BD Liaison

As a design consultant in the HKSAR, with the responsibility of being the interface between BD and the contractor on D & B projects, it is essential to factor into bids the cost of time spent liaising with BD. This can be quite hard for consultants to quantify given the variability of BD involvement. It is advisable to attempt to come to an agreement with the contractor that this is an element of design that should be done on a time basis.

Documenting Meetings

For a design consultant it is important when managing D & B projects to develop common practice in the documenting of meeting proceedings, particularly in the case where instructions to proceed are involved. In D & B projects the informal agreements that are common place have a tendency to lead to conflict when claims for CO's are submitted to the contractor. Further to this, unrecorded meetings allow for parties to walk away with a variety of interpretations on the actual outcome of the meetings.

Use of Option Reports

These are reports/sketches issued by the designer which clearly outline to the contractor a series of options on a particular design issue. Option Reports are very effective at obtaining rapid responses from the contractor rather than permitting the decision making process to be avoided and the design issue to remain unresolved. Option Reports are an effective tool for design consultants on D & B projects when it comes to discussing CO's as the designer is clearly covered with sufficient documentation.

Establishing a Joint venture system for Profit/Loss share

A joint venture system for Profit/Loss share was not established for the Tsz Wan Shan D & B project. However the benefits of this option can be seen in that:

- i) It would encourage the free flow of information within the D & B team and increase the efficiency of the design process;
- ii) It would provide encouragement for the design consultant in managing the BD design approval process.

Caveat Tender Drawings

Contractors carry out pricing based on tender drawings. For this reason it is important that the design consultant ensures that they sufficiently caveat tender drawings. Further to this, in protecting the contractor from a rapid escalation in costs during the detailed design, it is important that the design consultant does not pursue an unrealistically aggressive design at tender which will be un-buildable in detailed design within the bounds of the BD approval guidelines in the HKSAR.

CONCLUSION

This paper outlines a consultant's perspective on the key considerations and aspects of the management of D & B projects in the HKSAR. Consideration has been given to the fundamental aspects of D & B projects and this has been discussed within the context of the government design approval system and the authors experience on the Tsz Wan Shan Cable Tunnel project.

The development of Relationship Management is arguably the most important element in a D & B contract, particularly in the HKSAR. Other essential aspects of the D & B process that should be given considerable attention, particularly at the front end of projects, is the detailing of a clearly defined design agreement. It is recommended that particular attention is given to defining the scope of works, developing a clear design/deliverables program, and coming to an early agreement on and clearly detailing an efficient process for handling CO's.

ACKNOWLEDGEMENTS

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REFERENCES

1. Buildings Department, Practice Notes for Authorised Persons – (PNAP), www.info.gov.hk/bd/english/documents/index_pnap.html, last updated 6th April 2004
2. Geotechnical Engineering Office, Geoguide Series www.ced.gov.hk/eng/publications/publications_f.htm, last updated 8th April 2004
3. Garvey, Edward, "Design – Build Contractors Design Management", Copenhagen Metro Inauguration Seminar, 21-22 November 2002, pgs 95 – 101

Geotechnical Investigation & Construction Monitoring of Reclamation Project Harrington Waters Estate, Harrington, Australia

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This paper describes some of the geotechnical and construction aspects involved with a large-scale reclamation project at Harrington, which is situated over a deep alluvial profile containing soft / loose compressible soils. The combined residential and golf course development comprised dredging of one million cubic metres of sand from the adjacent Manning River to raise site levels, and facilitate ground improvement by means of preload to limit post construction settlements. Construction monitoring of the preload, together with the hydraulically placed fill which contained a high proportion of pyritic silt, was successfully undertaken to meet both the geotechnical and environmental objectives of the project.



1 INTRODUCTION

Harrington Waters Estate is nestled on the banks of the Manning River on the mid north coast of NSW, Australia.

Site development comprised dredging of one million cubic metres of sand from the adjacent Manning River in order to raise site levels, and enable the creation of about 560 residential allotments together with the construction of several lake systems, golf course and bridges.

The earthworks project commenced late 2002, and was approximately 60% complete as of April 2004. Pavements, services and bridges have been constructed on fill areas during this period.

Douglas Partners Pty Ltd (DP) have been involved with the project and Project Developers (Roche Group) from the early 1980's and have conducted geotechnical, environmental and groundwater investigation and design, together with construction monitoring during dredging and fill placement.

The site is situated over a deep alluvial profile, which extends beyond depths of 30 m and includes very loose sand and soft to firm clay

deposits. The upper profile was also assessed to contain potential acid sulphate soils and is susceptible to primary consolidation and creep.

This paper includes some aspects of investigation design and monitoring associated with the development.

2 SITE INVESTIGATION & GROUND CONDITIONS

Several phases of site investigation were undertaken to characterise the site and allow assessment of engineering soil properties for design and construction purposes.

The two main phases of geotechnical investigation program included:-

- Investigation of river sediments for dredging;
- Investigation of subdivision.

2.1 Assessment of river sediments

An investigation was undertaken within the Manning River to assess the suitability of river sediments for use as hydraulically placed fill and assess the presence of potential acid sulphate soils (PASS).

The investigation within the river comprised:

- Vibro-coring at a frequency of about 2 bores per hectare up to about 6 m depth below river bed levels (Total 110 bores);
- Grading / hydrometer testing & acid sulphate soil testing;

The above investigations indicated a fluvial profile consisting of sands with interbedded layers of clay, silty sand and clayey sand. Grading and hydrometer testing of the sediments indicated that a significant proportion (approximately 50%) of the dredged soil would contain a fines content of greater than 15%.

Laboratory testing was undertaken to assess PASS, which included screening tests and detailed laboratory analysis of the river sediments. The testing indicated that potential acid sulphate soils were present, however, there was no clear correlation between PASS with soil type, reduced level, depth or aerial distribution. The intensity of PASS, however, was, observed to be generally lower for sands with minor fines content.

2.2 Assessment of subdivision

The soil conditions were particularly suited to cone penetration testing (CPT) for the land-based investigation, particularly to assess the variability in the soft clay and loose sand profile. The CPT was supplemented with boreholes and test pitting to provide correlation with CPT interpretation, and enable samples to be recovered for laboratory analysis.

The land investigation revealed the site was generally underlain by the following:

- | | |
|------|--|
| CLAY | very soft to firm silty or sandy clay (up to 3.6 m deep) but interbedded with sand or clayey sand layers; |
| SAND | generally loose to medium dense (occasionally very loose), indurated sand layer also present, with clayey or silty zones (up to 8 m deep); |
| CLAY | further layers of clay generally firm to stiff and ranging between 0.2 m and 3.3 m thick; |
| SAND | medium dense to dense, to depths of up to 25 m; |
| CLAY | stiff to very stiff to depths of 33 m or greater. |

The main geotechnical feature of the site with respect to the development was the presence of weak strata, comprising very soft to soft silty clays, sandy clays and very loose sand.

Engineering soil parameters used for primary consolidation analysis such as the vertical coefficient of consolidation (c_v) and coefficient of compressibility (m_v) were derived from CPT/CPTU dissipation testing. Secondary consolidation parameters (creep factor) were based on methods presented by Mesri (1973).

3 GEOTECHNICAL ANALYSIS

The primary objective at the site was to determine ground improvement requirements (where necessary) to limit post construction settlements to enable a Class M (AS 2870) site classification for residential allotments. In addition, due to the high proportion of fines identified within the proposed dredge area, an assessment was required to determine options to incorporate the fines with site filling, without compromising the required Class M site classification.

Settlement analysis was undertaken at each CPT location based on the proposed loading by the dredged sediments, typically compacted to a density of 20 kN/m³, and the imposed load by single and double storey structures, (typically about 15 kPa).

The post construction settlements were estimated based on fill-induced settlement, plus the slab induced settlement, over a design life of 50 years.

Settlement analysis indicated post construction settlements ranging between 5 mm and 150 mm over a 50 year period without ground improvement.

An example of a time-settlement plot is presented in Figure 1 below.

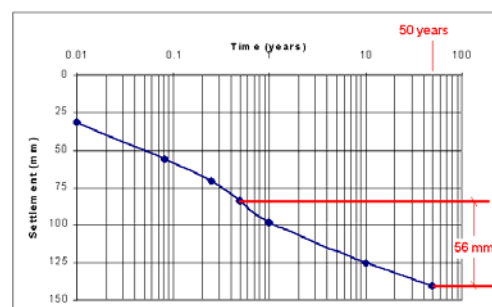


Figure 1: Typical Settlement Plot

The CPT results were also used to assess the likelihood of liquefaction of the very loose slightly silty sands at the site. The assessment was based on the results of the CPT and the methods presented by Lee *et al* (2001). A peak horizontal ground coefficient of 0.10g was adopted, based on AS 1170.4-1993.

A typical liquefaction probability plot is provided in Figure 2 below:

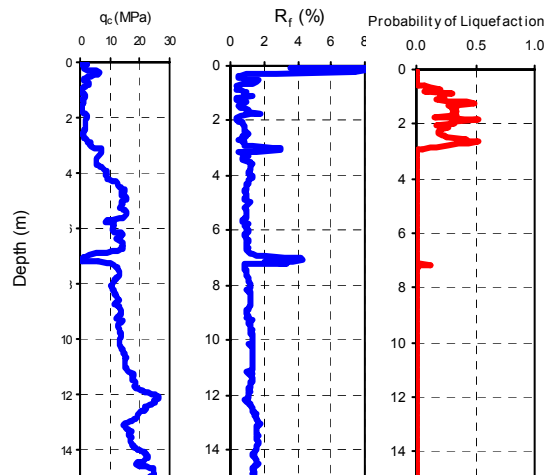


Figure 2: Typical Plot showing potential of Liquefaction (CPT 452)

The results indicated that liquefaction was unlikely to occur during an earthquake of magnitude $M = 6$ or less.

3.1 Ground Improvement Requirements

Based on the settlement analysis undertaken at each CPT location, the site was divided into geotechnical zones according to the magnitude of post construction settlement as described below:

- Zone A - Post construction settlements would preclude the adoption of Class M site classification without ground improvement.
- Zone B - Post construction settlements were commensurate with a Class M site classification. The majority of the development fell within Zone B.
- Zone C - Zone C was dominated by a sand profile and lower estimated settlements.

Additional testing using the CPT was undertaken following the preliminary analysis to better define the depth and lateral extent of

clay strata, (ie limit the size of the Zone A area requiring ground improvement).

Several options for ground improvement were considered for Zone A to improve post constructions settlements. Preloading by means of sand fill to a height of at least 1.5 m above the proposed fill level, however, was assessed to be the most feasible and practical ground improvement technique.

It was considered that the fines generated from the dredging process could be blended with the placed sand at a ratio of up to 15% dry weight, without compromising the required geotechnical parameters of site fill. In addition, due to the lower predicted settlements assessed within Zone C, it was considered these areas could accommodate up to 200 mm thickness of fines, subject to mixing with the base layers of sand fill, and remain a Class M site classification.

4 CONSTRUCTION MONITORING

4.1 General

Dredged slurry was pumped from the Manning River and discharged into bunded areas within the subdivision.

Ground conditions within the Manning River were suited to a bucketwheel cutter suction dredge. The hydraulic slurry was transported from the extraction site to the subdivision via 300mm ϕ pipeline. The dredge initially produced a discharge of about 300 m³/hr but was upgraded to 600 m³/hr for the second half of the project.

When the hydraulic slurry was discharged at the dredge head, sand materials with some fines were initially deposited. The remaining fine sediments discharged to adjacent designated sediment ponds.

A synthetic flocculant was added by injection to the hydraulic slurry at the dredge head to assist with the settlement of fines and improve overall tail-water quality.

The tail-water flowed from a series of sediment ponds, via a fixed weir arrangement, to a management pond where water was tested to confirm compliance prior to discharge to the Manning River.

4.2 Construction Monitoring Objectives

The main objectives during the monitoring program was to manage acid sulphate soils during filling operations, monitor tail-water quality prior to discharge, monitor preload, and undertake geotechnical testing of the placed fill to confirm a Class M site classification.

4.3 Acid Sulphate Soils

Daily acid sulphate tests were undertaken on the dredged sediments and tail water in accordance with an Acid Sulphate Soil Management Plan prepared by DP for the site.

Results of testing indicated that the PASS conditions within the dredged sediments were associated with the finer sediments (i.e. silt/clay), which were segregated during the dredging process and transported to designated sediment ponds. The sand materials deposited, therefore, did not require acid sulphate soil treatment (neutralisation).

Detailed testing of the fines indicated that treatment was required.

Upon raising site levels, the finer sediments within designated sediment ponds were placed over existing sand filled areas for treatment. Prior to placement of the finer sediments, a layer of lime was spread over the surface of the sand as a precautionary measure to counteract the generation of potential acid leachate.

The PASS sediments were treated by full lime treatment. Dozers and excavators were used to thoroughly mix agricultural lime with the soil.

Once neutralisation was achieved, clean sand was mixed with the treated sediments for re-use as engineering fill.

Tail-waters generated from the dredging process were also monitored and treated (ie pH adjustment), if required, prior to discharge to the Manning River.

Monitoring of the tail-water indicated that pH adjustment was generally not required.

4.4 Preload Monitoring

Preload monitoring during construction was undertaken to determine when the preload fill could be removed to allow development of the residential sites.

Installation of settlement plates was the preferred method to monitor the preload. Once the results of the monitoring indicated that the design settlement criteria was achieved, the preload was removed, and the excess sand utilised as fill on the adjacent area.

CPT's were used in preload areas following preloading to assess undrained shear strength within the clay profile and confirm whether sufficient strength gain and hence primary consolidation had occurred within the clay layers.

The strength gain with the clay after preload was based on a relationship between the increased effective overburden stress and undrained shear strength.

An example of strength gain measured in the clay as a result of preloading is presented in Figure 3 below.

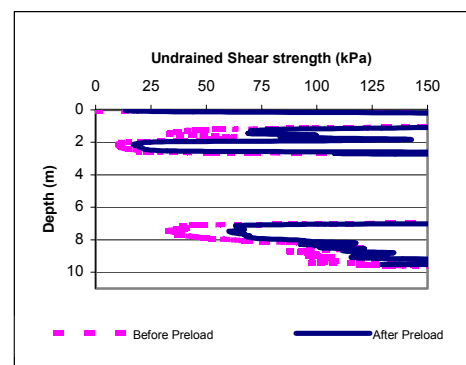


Figure 3: Strength Gain in Clay from Preloading (CPT 318)

The results of CPTs showing the strength gain within the clays, together with data from the settlement plots, confirmed the clays had undergone sufficient primary consolidation and allowed the preload to be removed.

5 CONCLUSIONS

A multi-disciplinary approach to investigation and construction monitoring has successfully been conducted for the above development which included geotechnical considerations (ie preloading, settlements, foundation and subgrade conditions), and environmental issues (treatment of PASS, and dredge tail-waters prior to discharge).

6 ACKNOWLEDGEMENTS

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7 REFERENCES

Douglas Partners Pty Ltd, 'Geotechnical Assesment, Proposed Dredging, Manning River, Harrington', *Unpublished report, Project 31360, September 2001.*

Douglas Partners Pty Ltd, 'Stage 4, Harrington Waters Estate', *Unpublished report, Project 18672L, 2003.*

Lee, D.H., Juang, C.H., Ku C.S., (2001) Liquefaction performance at the site of a partially completed ground improvement project during the 1999 Chi-Chi earthquake in Taiwan, *Can. Geotech. J. Vol.38, 2001, 1241-1253.*

Mesri, G. (1973) Coefficient of secondary compression. *J. Soil Mechs Fds. Div. Am. Soc. Civ, Engnrs 99, SMI 123 –137.*

Prediction of Primary and Secondary Consolidation of a soft soil in the Sydney Basin

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Summary: This paper presents an approach to predicting the short and long-term consolidation behaviour of a soft alluvial soil in the Sydney Basin. Laboratory testing data, Cone Penetration Testing logs and borehole information were used to characterise the subsoil profile and gain an appreciation of the subsurface variability across the site. Settlement plate, extensometer and piezometer data assisted the tracking of consolidation of the subsurface layers during the application of preload. An assessment of predictive modelling techniques was undertaken and a method developed by Handy adopted to assist in the estimation of time to completion of primary consolidation. This curve-fitting approach to actual data allows the calculation of the practical completion of primary settlement and assists in the prediction of secondary settlement over the life of the proposed development.

INTRODUCTION

The site was a former Industrial Storage Facility located on the bank of the Parramatta River. The proposed development comprises 1 to 2 storey residential development and associated infrastructure. The soft alluvial subsurface across the majority of the site meant that significant improvement of the foundations was required before development could proceed.

The earthworks included 3 main stages to stabilize the soft alluvial sediments across the majority of the site. Firstly, a drainage system was installed to assist in the dissipation of excess pore pressure. Imported regrade fill was then placed and compacted to raise the site above the 1 in 100 year flood level. Finally a 2m layer of surcharge material was placed on top of the regrade fill for a finite period of time to accelerate the rate of settlement. The surcharge layer was left on the regrade layer until an acceptable amount of primary settlement had occurred.

The site investigations, design of the drainage system and preload provisions were carried out by others, prior to URS Australia's engagement to oversee the following:

- installation of drains;
- placement of regrade and surcharge fill; and
- general progress of the preloading.

DRAINAGE SYSTEM

To facilitate drainage of the soft alluvial soils, synthetic wick drains were installed vertically across the site to provide a vertical pathway for the release of porewater, occurring as a result of the consolidation of the soft alluvial soils. These wick drains were installed on a 1.0m, 1.5m or a 2.0m triangular grid depending on the expected settlement and the proposed development of the area. The depth of installation was to the base of the soft soil layer, determined by previous investigations within the area and by resistance of the installation rig. The uppermost 1 to 2 metres of silty sandy material was generally considered to have sufficient permeability to act as a drainage layer for the porewater released from

the wick drains. Trench drains with a free draining aggregate were also provided as a secondary drainage path.

IMPORT FILL

The earthworks predominantly involved the placement of imported regrade fill to raise the levels of the foundations to above the 1 in 100 year flood level. This fill consisted of virgin excavated natural sandstone or shale placed in a controlled manner. Additional temporary surcharge fill was placed to facilitate consolidation of the underlying natural soft soils.

INITIAL INVESTIGATIONS AND LABORATORY TESTING

Preliminary geotechnical site investigations were undertaken across the site to characterise the subsurface profile of the proposed development area. Investigations onsite included the following:

- 7 Boreholes drilled to depths ranging between 8.1 and 16.8m;
- 11 groundwater and sampling wells;
- 9 test pits excavated to depths ranging between 0.8 and 2.9m;
- 52 electronic friction cone penetrometer tests (CPT); and
- geophysical survey of the substrata.

Selected samples of the alluvial soils obtained from boreholes were tested in a NATA approved laboratory. The following tests were undertaken:

- Moisture content;
- Atterberg limits;
- Linear shrinkage;
- One dimensional consolidation testing;
- Point load index;
- 4 day soaked CBR;
- Particle size distribution;
- Triaxial testing;

- Chemical tests (chloride, sulphate, nett acid producing potential); and
- Physical tests (pH, conductivity, TDS).

GEOTECHNICAL CONDITIONS

Reference to the 1:100,000 Geological series sheet 9130 (Ed 1 1983) and the initial geotechnical field and laboratory investigation indicates the floodplain area of the site is underlain by silty to peaty quartz sand silt and clay with ferruginous and humic cementation in places. The northern area of the site is underlain by Hawkesbury sandstone, comprising medium to coarse grained quartz sandstone with minor shale and laminate lenses.

The subsurface profile of the site is characterised by two main regions. The profile of the northern region typically consists of topsoil/fill overlying residual clays. Subsurface investigation indicated sandstone bedrock or shale bands lay between 0.8 and 2.7m below ground level. The profile of the southern region of the site typically consists of 1-2m of granular fill overlying alluvial sediments from the Parramatta River. Subsurface investigation indicates the soft alluvial clay deposits range in thickness from 0 to 13m underlain by residual soils and sandstone bedrock. The alluvial deposits are typically interbedded with sandy and silty lenses and bands. Across the site there sediments are highly variable, both vertically and laterally – typical of a fluvial depositional environment.

CONSOLIDATION MONITORING

Prior to installation of the wick drains, extensometers and vibrating wire piezometers were installed at 14 locations across the site to monitor the settlement of the subsurface layers and to observe the groundwater response associated with the settlement. Settlement plates were installed prior to placement of any fill, to allow each plate to stabilize. The plates were used as the primary method for tracking the settlement, as they provide a cost effective coverage of the settlement profile of the site. Where possible, the extensometers were used as a comparison for nearby settlement plates and to observe the settlement behaviour of each layer of material identified. The piezometers were useful as one of the indicators of the completion of primary consolidation (the return of elevated porewater pressures to stable levels indicating the completion of primary consolidation).

PREDICTIVE MODELLING APPROACH

There were two main requirements for predictive modelling of settlement during the project. The principal reason was to assess the time to completion of the primary settlement, to gain an indication of when the preload could be removed to allow the next stage of development to commence. This allowed for timely scheduling of the following works, for example installation of services and infrastructure.

The secondary reason for predicting the time for completion of settlement was to determine if the design surcharge load applied to the regrade fill was likely to give a satisfactory time for completion of primary settlement and to see if alternatives should be considered. Such alternatives include increasing the amount of surcharge to achieve an acceptable amount of settlement within a certain time period or employing other ground improvement techniques such as impact rolling.

PREDICTION OF PRIMARY SETTLEMENT

A number of methods were adopted throughout the course of the project to predict the total amount of settlement and the time to completion of primary settlement. These methods included:

- Hand calculations – hand calculations were undertaken throughout the project using information obtained from CPT logs and boreholes for the profile of the subsurface the laboratory testing. Approximations were and consolidation parameters obtained from made at many locations due to the limited laboratory testing undertaken and the expected variability of the subsurface profile over relatively short distances.
- Asaoka Graphical Settlement Prediction method - The Asaoka method was used during the early stages of the project to obtain an indication of the time to completion of primary settlement. It provides a simplistic approach whereby settlement observations ($p_0, p_1, \dots, p_n$) taken at regular time intervals are plotted on a graph in the form (p_j, p_{j-1}). Where the straight line fitted to these points intersects the $p_j = p_{j-1}$ line, is the point of practical completion of the settlement. (Asaoka, 1)
- Handy method – An approach that applies a first order rate equation for predicting the time to completion of the primary settlement. Similar to the Asaoka method of predicting settlement, it does not require a thorough understanding of the mechanisms involved.
- TCON – incremental analysis approach to predicting settlement. TCON is a computer driven program used to model consolidation behaviour. It requires thorough knowledge of the consolidation parameters, the stratigraphy of the subsurface, the loading history of the soil and the applied loading to the subsoil.

Prediction of settlement evolved throughout the project through review of a number of methods and observation of the performance of each method with respect to the onsite filling process and observing the most suitable method.

Initially, the expected amount of settlement was calculated at each of the settlement plate locations. The

stratigraphy was interpolated from nearby CPT or borehole logs and an equivalent layer of “significantly compressible” material was identified. This step was undertaken to simplify the analysis due to the limited amount of data on the consolidation parameters of the subsurface material. Consolidation parameters were assigned to the “significantly compressible” layer after assessment of the laboratory testing data and considering the likely variable subsurface profile across the site. A best estimate approach with error bounds was adopted when assigning consolidation parameters. The loading profile at each settlement plate location was determined and the expected settlement calculated.

To calculate the time taken for this settlement to occur, the TCON program was used. TCON analysis enables the operator to enter the subsurface profile, wick drain installation information and an incremental loading profile - typical of the application of compacted regrade and surcharge fill in the field. Problems arose with lack of accuracy due to the number of parameter inputs required and the uncertainty of the values of the parameters. The model required labour intensive frequent calibration to model the actual settlement behaviour onsite and there was no guarantee of its accuracy.

The Asaoka graphical settlement prediction method and a method presented by Handy were identified as predictive modelling techniques where a complete understanding of the mechanisms involved is not required, so long as the mechanisms driving the behaviour are consistent. They both model behaviour whereby an asymptotic approach to an end condition is required. The graphical Asaoka method, as described above, requires settlement to be measured at regular time intervals. The method presented by Handy allows each settlement data point to be used to develop a relationship between the observed settlement and time.

Taking in to account the accuracy of knowledge of the input factors including consolidation parameters, the extended loading profile and the parameters associated with the drainage system, the Handy method was deemed to be the most appropriate method for tracking and prediction of primary settlement. It did not require accurate knowledge of the consolidation parameters, was an appropriate tool for the settlement mechanisms involved and provided a reasonable and appropriate level of accuracy for estimation of the settlement.

HANDY MODEL

The Handy method is based on Terzaghi’s consolidation rate equation based on an analogy between the flow of water out of soil under pressure and the thermodynamics of heat flow (Terzaghi et al, 2).

$$\log(e - e_p) = -(k_t t + C_t) \quad (1)$$

where,

- e = void ratio,
- e_p = final primary consolidation void ratio for a particular applied pressure;
- t = consolidation time;
- k_t = constant; and
- C_t = constant.

“The equation is solved by substituting trial values for e_p until a linear plot is obtained as indicated by the regression coefficient. An equation for void ratio versus time then may be written from the results of the linear regression analysis.” (Handy, 3).

SETTLEMENT MODELLING DIFFICULTIES

The first order rate equation presented by Handy can be used to obtain a good prediction for the completion of primary settlement, however it does not take into account the process of secondary settlement. Secondary settlement is the ongoing consolidation the soil undergoes at constant effective stress, due to the rearrangement of soil particles.

In the field, it is difficult to separate primary settlement from secondary settlement especially when considering thick layers of consolidating clay. Some regions near the drainage surface may have undergone 100% of the primary settlement and be commencing secondary settlement, whereas regions further from the drainage surface may still be undergoing primary settlement (Holtz & Kovacs, 4). While the wick drains installed across the site reduce the effective drainage path for thick soil deposits, the different settlement mechanisms still occur simultaneously.

During the initial stages of primary settlement, the majority of the settlement occurring within the soil layer is primary settlement. Terzaghi’s first order rate equation analogy provides an accurate prediction of the likely time to completion of the primary settlement. As time continues, the soil elements closer to the drainage surface complete the primary settlement stage and enter the secondary settlement stage. The proportion of the observed changes in total settlement that is due to primary settlement gradually reduces. Towards the end of primary settlement, the majority of the change in total settlement may be secondary settlement. As this occurs, the application of a first order rate equation to the observed settlement becomes less valid, as the mechanisms controlling the settlement gradually change.

To account for this change in conditions controlling the settlement of the soil, consideration was given to the following factors when modelling the primary settlement:

- The total amount of settlement expected;
- Expected rate of secondary settlement at the completion of primary settlement;

- The time taken for primary settlement to occur; and
- The thickness of the equivalent layer of compressible material.

When the Handy model presented is used to model the observed settlement, the effect is for a conservative estimate of the time to completion of primary settlement to occur. The greater the secondary settlement component of the total settlement, the more conservative the estimation is likely to be. This was taken into account when assess the time at which surcharge fill could be removed.

It was important to estimate the expected rate of secondary settlement at the completion of primary settlement to ensure that the settlement occurring was indeed secondary settlement. The following section details the process adopted for prediction of secondary settlement.

PREDICTION OF SECONDARY SETTLEMENT

Estimation of the secondary settlement across the site was important during the project to predict the settlement rate at the completion of primary settlement and to gain an indication of the potential settlement of any structures constructed onsite over their service life. The total expected secondary settlement and the potential for differential settlement was estimated to assess the potential impacts on the foundation systems and to develop design recommendations for the site to manage potential ongoing settlements.

Factors influencing the amount of secondary settlement a structure is likely to undergo include the following:

- Time to completion of primary settlement;
- The design time period of the structure (design life);
- Consolidation parameters (primary and secondary);
- Surcharge effort; and
- Layer thickness.

Discussion of how each factor influences secondary settlement is presented below.

TIME TO COMPLETION OF PRIMARY SETTLEMENT

As discussed, the time to completion of primary settlement influences the total amount of secondary settlement observed post surcharge removal as primary and secondary settlement occur simultaneously. The longer the time period for primary settlement to occur, the greater the amount of secondary settlement, prior to surcharge removal.

In a laboratory analysis, the duration of the primary consolidation stage is typically small due to the loading control available. As a result, only a limited amount of secondary settlement can occur during the primary settlement phase. In the field, where compacted fill is

placed in relatively thin layers (typically 300mm), the rate loading is limited to how fast the fill is placed, compacted and tested. The staged placement of compacted fill over large areas can extend over months, resulting in a comparatively long time for primary settlement to occur. As the primary settlement phase is over an extended time period, there is equally an extended period of time over which secondary settlement can simultaneously occur.

To assess the magnitude of secondary settlement, the ratio t/t_p is calculated, where t is the total time and t_p is the time for completion of primary settlement. In laboratory analysis of secondary settlement, t/t_p can be typically very large, resulting in secondary settlement after surcharge removal being quite significant. In the field, where t_p is typically longer, secondary settlement after surcharge removal is likely to be less significant. “Typically, for the useful life of the structure, the value of t/t_p rarely exceeds 100 and is often less than 10.” (Terzaghi et al, 2)

Assuming a typical design life of 50 years, t/t_p for this project was below 100, with t_p 's ranging from 6 months to 18 months. Areas with larger values of t/t_p typically had thin equivalent layers of compressible material. For these areas, while secondary settlement may have a greater proportion of the total settlement, the total settlement is still likely to be small.

SECONDARY COMPRESSION INDEX

An appreciation of the behaviour of the secondary compression index, C_{α} , with respect to time is needed to predict the long-term secondary settlement of a structure. Terzaghi, Peck and Mesri (2) provide a discussion on the potential behaviour of C_{α} in post surcharge secondary compression, identifying the possibility of the index increasing, remaining constant or decreasing with time. However, C_{α}/C_c remains approximately constant in both recompression and compression ranges at any time during secondary compression, permitting the secondary settlement behaviour to be predicted.

For geotechnical materials ranging from granular soils to peat, C_{α}/C_c ranges from 0.01 to 0.07 respectively with inorganic clays and silts typically 0.04 ± 0.01 . A compressibility index ratio of 0.05 was adopted for secondary settlement predictions as the subsurface profile typically consisted of a thin layer of peat underlain by a deposit of varying thickness of soft clay. (Terzaghi et al, 2)

A secant, C_{α} , is identified for practical analysis of secondary settlement (see Figure 1). The prediction methodology is detailed below.

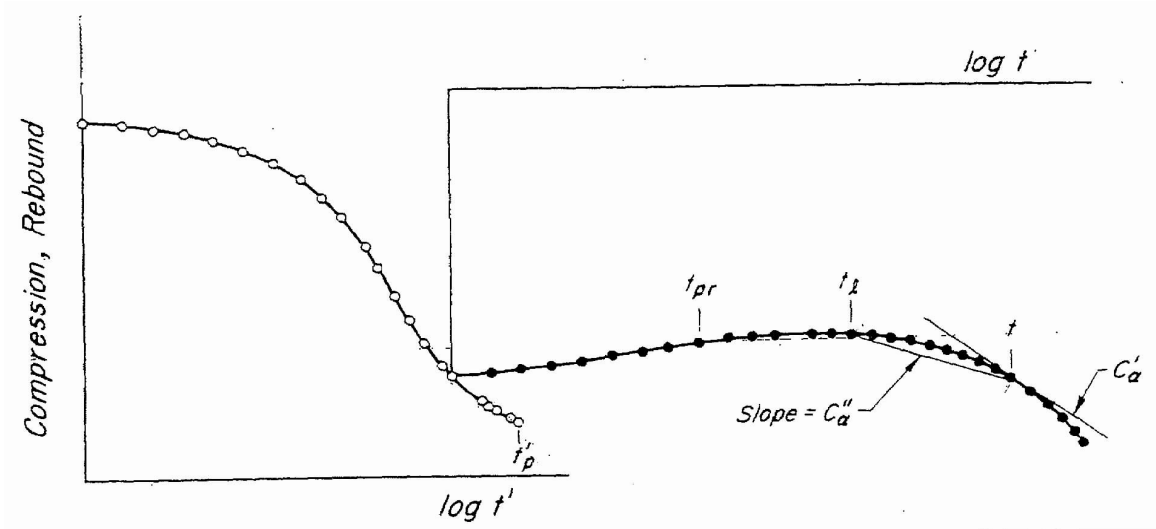


Figure 1: Settlement Behaviour - Surcharge Removal (Terzaghi et al, 2)

SURCHARGE

Surcharging was undertaken to reduce post construction settlement and to accelerate the rate of primary settlement.

The removal of the surcharge leads to rebound of the subsurface, with both primary rebound and secondary rebound occurring. The primary rebound time, t_{pr} , and secondary rebound time, t_l , depend on the rebound characteristics of the soil as well as the permeability and drainage boundary conditions. The value of t_l is determined from the empirical correlation between t/t_{pr} and R_s as presented below.

SECONDARY SETTLEMENT PREDICTION PROCEDURE

The following procedure identifies the methodology used to predict the likely secondary settlement at various settlement plate locations across the site.

STEP 1

Initially, the soil profile parameters were defined at a number of locations. The level of the water table, the thickness of the equivalent compressible layer and the unit weights of the existing and imported soil elements were estimated. These parameters were then used to determine the effective stress on the midpoint of the equivalent compressible layer.

STEP 2

The Effective surcharge ratio, R_s' , was then calculated.

$$R_s' = \frac{S'_{\text{vertical_surcharge_stress}}}{S'_{\text{vertical_post-surcharge_stress}}} - 1 \quad (2)$$

(Terzaghi et al, 2)

STEP 3

The primary rebound time after surcharge removal was then calculated.

$$t_{pr} = \frac{Td^2}{C_{vs}} \quad (3)$$

For areas without wick drains,

C_{vs} was obtained from laboratory oedometer testing;

$d = 0.5 \times \text{thickness of compressible layer}$

$$T = \frac{(p/4)(U_{av}\%/100)^2}{[1 - (U_{av}\%/100)^{5.6}]^{0.357}} \quad (4) \quad (\text{Das, 5})$$

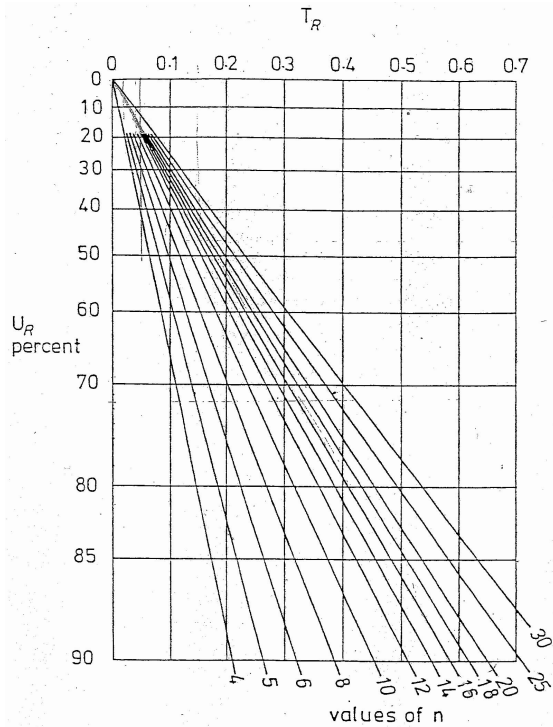
For areas with wick drains,

C_{vs} was obtained from laboratory oedometer testing

$d = 0.525 \times \text{triangular wick drain spacing}$

T was obtained from Figure 2.

Figure 2: Graph relating T, U and n



(adapted from Richart, 6)

STEP 4

The secondary rebound time after surcharge removal is then calculated from the following empirical equation.

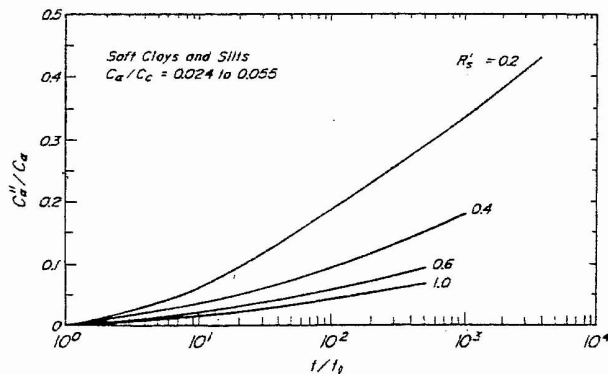
$$\frac{t_l}{t_{pr}} = 100R_s^{1.7} \quad (5)$$

(Terzaghi et al, 2)

STEP 5

The secant C_{α}'' was then estimated from Figure 3.

Figure 3: Post Surcharge Secondary Compression Index for Deposits of Clay and Silt



(Terzaghi et al, 2)

STEP 6

The total post surcharge secondary settlement was then calculated using equation 6.

$$S = \frac{C_a''}{C_a} \frac{C_a}{C_c} C_{ec} L_0 \log \frac{t}{t_l} \quad (6)$$

(Terzaghi et al, 2)

CONCLUSIONS

The Handy method presented is considered to be the most appropriate method for predicting the time to completion of primary settlement for field analyses, where there is limited knowledge of the subsurface profile and material characteristics. The approach models future settlement based on real data recorded and does not require accurate knowledge of the subsurface profile, the loading profile or the consolidation parameters of the soft sediment layers. The Handy method should be used in conjunction with calculations based on traditional consolidation theory, to ensure the settlement behaviour observed is relatively consistent with expectations.

It is acknowledged that predictions based on the Handy model become less accurate and more conservative as completion of primary settlement approaches. The gradual increase in secondary settlement as a proportion of the total settlement leads to an overestimation of the total primary settlement. The error associated with the increasing proportion of secondary settlement can be managed, provided the secondary settlement at the completion of primary settlement has been assessed.

Careful consideration should be given to the amount of secondary settlement any developments on site will undergo settlement amounts can be significant. Substantial differential settlements can occur as a result of secondary settlement over the design life of a structure and appropriate foundation design is important to reduce the potential for the occurrence of structural damage. Significant factors influencing the settlement of the structure include the time taken for primary settlement, the design life of the structure, the surcharge effort, the layer thickness of the consolidating material and the consolidation parameters of the subsoil material.

ACKNOWLEDGEMENTS

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REFERENCES

1. Asaoka, A., "Observational Procedure of Settlement Prediction", Soils and Foundations, Vol 18, No. 4, December 1978, pp 87-101.

2. Terzaghi, K., Peck, R.B., Mesri, G., (1996) "Settlement during Secondary Consolidation Stage," Soil Mechanics in Engineering Practice, Third Edition, pp108-110.
3. Handy R.L., "First-Order Rate Equations in Geotechnical Engineering", Journal of Geotechnical and Geoenvironmental Engineering, Vol, 128, No. 5, May 1 2002.
4. Holtz, R.D., Kovacs, W.D., (1981) "An Introduction to Geotechnical Engineering", Prentice Hall.
5. Das, B.M., (1987) "Advanced Soil Mechanics", McGrawHill, p. 511.
6. Richart, F. E., (1959), "Review of the Theories for Sand Drains," TRANS. ASCE, Vol.124, 1959.

Joint Inversion of Phase Velocity and Apparent Resistivity using Genetic Algorithms: A Synthetic Model

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Abstract: Independent inversions of apparent resistivity or phase velocity have assisted in obtaining global subsurface physical parameters such as resistivity and S-wave velocity. Restrictions on the attainability of actual observations, makes the inversion of observation data to a true model difficult. A joint inversion of Rayleigh wave phase velocities and apparent resistivity is proposed using genetic algorithms, in order to obtain appropriate model parameters from insufficient data in shallow geophysical exploration. The inversion of synthetic data supported the effectiveness of the method.

INTRODUCTION

Apparent resistivity and Rayleigh wave phase velocity tests, used in the analysis of soil properties, have many applications in the evaluation of natural resources, environmental and civil engineering problems.

For instance, Resistivity tests can detect water bodies and ground fractures through the current's identification of air as a resistor and water as a conductor. Known as a Vertical Electrical Sounding (VES) test, current is passed between two stationed poles and the dispersion recorded at various distances from the central current circuit. In doing so a direct relation between depth, spacing and apparent resistivity at the respective distances can be observed.

Rayleigh wave phase velocities are highly effective in the assessment of soil stiffness. As a

phase velocity wave continues through its medium, with consistent shape of peak, trough, or zero-point these test readings give valuable information regarding the layer stiffness and thus the dispersion of the Rayleigh waves (Haskell, 1953) at varying subsurface depths. Wave amplitude also diminishes exponentially with depth, measurable by the period and amplitude of the observed surface wave.

Forward modeling is the standard mathematical process of interpreting a model into graphical data. That is to say, taking a three-layer soil model with given layer thickness, resistivity, and phase velocity values, and interpreting the phase velocity over period, or resistivity over tested spacings and therefore relative to depth. Where it is difficult to know first these model parameters of thickness, resistivity, and phase

velocity, data from the abovementioned tests can be taken to then generate the model. This reversed mathematical process is known as an inversion.

The aim of inverting apparent resistivity and phase velocity data separately, (an independent inversion) obtained from field observations which generates subsurface parameters, is to minimise misfit between the observed data and theoretical values calculated in forward modeling. However, as various obstacles are often encountered in the field, the data observed on site may at times be incomplete, parameters obtained may consequently be far from those of the actual soil profile, this profile of which the inversion is seeking to attain to is herein referred to as the 'true' synthetic model.

In belief that the joint inversion of the two elements would compliment one another in the case of incomplete data, the assessment of joint inversion effectiveness was undertaken, focusing specifically on the applicability of a joint inversion to insufficient data. In this study, having proposed the joint inversion procedure, data of a synthesised model will be used to then investigate its applicability in shallow geophysical explorations.

METHOD

Rayleigh wave propagation is assumed in a layered subsurface model, with the phase velocity represented as a function of the S-wave (secondary, rotational wave) and P-wave (primary, dilatational wave) velocity, thickness, and density of each layer (Ohta and Goto, 1976). Once frequency-dependent phase velocities are obtained, through the optimisation of influential S-wave velocity and thickness, upper and lower search parameters for the two are determined in order to

minimise misfit, ϕ_p , as defined by

$$f_p = \frac{1}{N_p} \sum_{i=1}^{N_p} |c^c(f_i) - c^o(f_i)| \quad (1)$$

where N_p is the number of observed phase velocity data, $c^c(f_i)$ and $c^o(f_i)$ are the calculated and observed phase velocities at frequency f_i .

Apparent resistivity, observed on site at various electrode spacings, can be assimilated to the one-dimensional model assumed in the resistivity inversion. Resistivity and thickness of each layer are determined inversely, likewise from within assigned search parameters, for which the misfit used, ϕ_r , is

$$f_r = \frac{1}{N_r} \sum_{i=1}^{N_r} |r_a^c(a_i) - r_a^o(a_i)| \quad (2)$$

where N_r is the number of observed resistivity data, $r_a^c(f_i)$ and $r_a^o(f_i)$ are the calculated and observed resistivities at electrode spacing a_i .

As resistivity units, measured in ohms, are directly unrelated to that of phase velocity units of ms^{-1} , the use of a dimensionless misfit is necessary. In this study, the following dimensionless misfit in joint inversion, ϕ_j , was used.

$$f_j = p f_r + (1 - p) f_p \quad (3)$$

Throughout the study, the weighting factor p was taken to be 0.5, highly applicable to the fundamental three-layered theoretical model. Particularly useful in the case of insufficient site data, the p factor weighs the contribution of each test's results to the other in the joint inversion's profile modeling process, where stronger test results would call for a higher value of p .

In conducting a joint inversion, Genetic Algorithms (GA) were used. These proved to be a successful tool in independent inversions

(Yamanaka, H and Ishida, H; 1995) and were seen as applicable to an apparent resistivity and phase velocity joint inversion.

Genetic Algorithm variables included the probability of cross-over and mutation of individuals in each generation which were set at 0.7 and 0.3 respectively, as is generally characteristic of genetics. Mutation discourages premature convergence towards the optimal models within the misfit function, whilst the probability of crossover can be assimilated to genetic inheritance. In this sense the cross-over function contributes coded aspects of the parenting individuals to individuals of the successive generation. The additional element of elite selection ensures the generation's strongest individual is successfully carried into the following generation, assisting in attaining the potential, optimal model resultant.

For each inversion, independent and joint alike, models with a 1%, 3%, 5%, 10% misfit variation between the models identified as closely attaining to the true synthetic model, referred to as 'good-models', were evaluated. This was in search for the optimum misfit percentage with maximum number of good-models along with a minimum standard deviation. As the results found minimal variation with respect to the misfit percentage, 5% was selected for all inversion bringing a common percentile to the misfit analysis.

NUMERICAL EXPERIMENT

A simple three-layer subsurface structure was adopted for the initial model (model-bb1). The simple model was chosen to keep the parameters to a minimum in order to allow the results to show the appropriate fundamental conclusions. The

eight given parameters for model-bb1 include a phase velocity, and resistivity parameter for each of the three layers along with thickness parameters for layers N1 and N2, where the half layer N3 is of infinite thickness. Depicted below in Figure 1, the given values are shown below in Table 1.

Ground Surface		0m
Water-table	N 1	2m
	N 2	12m
Half layer	N 3	

Figure 1

Three-layer, subsurface synthetic model.

Table 1 Given parameters of true model

Layer No.	Phase Velocity Vs (m/s)	Resistivity r (ohms)	Thickness h (m)
N1	200	100	2
N2	200	1	10
N3	600	10	*

*Half layer of infinite depth, shared interface with N2.

A Forward modeled computation of resistivity and phase velocity using the given parameters for model-bb1, produced a set of synthetic observed data which was then applied to all inversions.

Figures 2 and 3 show the forward modeled true functions of resistivity and phase velocity respectively, along with the complete and limited observed data used in the inversions. Inversions involving the complete set of twenty observations implied a full inversion, where as, of those twenty observations six were selected representing insufficient data and therefore a limited inversion, as typically obtained on site.

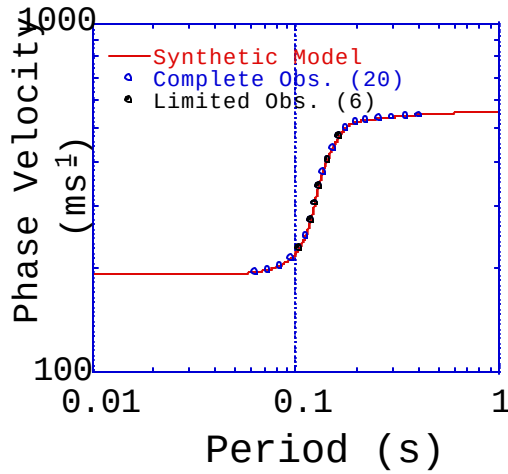


Figure 2

Points of observations taken from the synthetic phase velocity data used in complete and limited inversions.

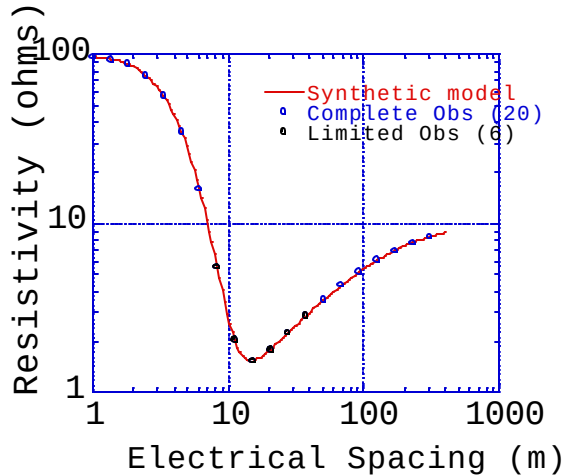


Figure 3

Points of observations taken from the synthetic resistivity data used in complete and limited inversions.

In a twenty observation full inversion of both apparent resistivity and phase velocity, a greater accuracy was achieved by the independent inversions, when compared with the jointly inverted models. Although the variance proved limited, it is recommended that independent inversions be used for profiling with complete

observations. Results obtained herein, are in reference to inversions carried out with the limited six observations.

As with an increasing subsurface depth, data for apparent resistivity becomes more difficult to obtain. Therefore, the limited observations chosen reflect those obtainable in a real situation. Similarly for the phase velocity inversion, the six observations chosen are within the limited period domain of 0.11s and 0.17s, assumed in the numerical experiment in making synthetic data.

Figure 4 shows the comparison between the synthetic, observed data and calculated apparent resistivity for models from independent and joint inversions.

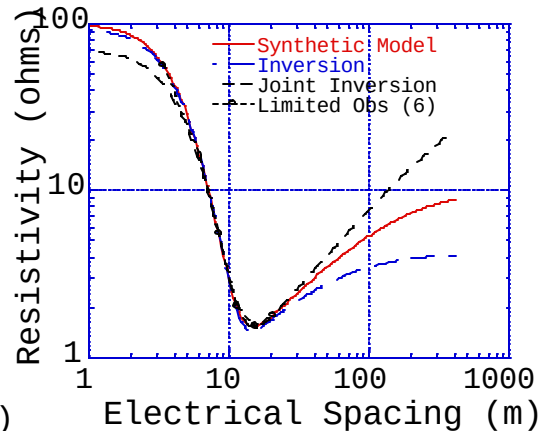


Figure 4

Comparison of synthetic resistivity with theoretical values for inverted models.

Table 2 Parameter Comparison: Resistivity

Layer	True Model		Independent		Joint	
	ρ	h	ρ	h	ρ	h
1	100	2	93.5	2.06	69.709	2.16
2	1	10	0.455	3.51	0.937	10.54
3	10	**	4.154	**	56.969	**

Table 2 shows that the parameters obtained in

the joint inversion for both layer depths and resistivity of N2 has achieved higher accuracy with respect to the synthetic model, in comparison to those obtained by independent inversion, particularly in the model's mid-depth layers.

Figure 5 below shows the comparison between the synthetic observed and calculated phase velocities of independently and jointly inverted models. Limited to six observations within the period range of 0.11s to 0.17s, the upper and lower bounds are exposed to significantly larger discrepancy outside this period range.

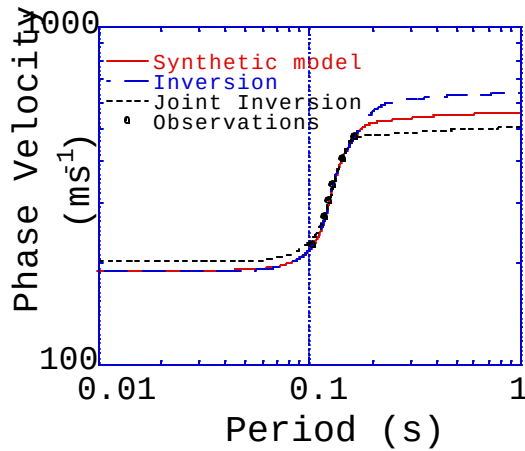


Figure 5

Comparison of synthetic phase velocity with theoretical values for inverted models.

Model	True	Independent			Joint	
N	Model	Vs	h		Vs	h
1	200	2	198.24	9.60	211.86	2.16
2	200	10	231.09	3.26	212.14	10.54
3	600	**	685.44	**	542.23	**

Table 3 shows S-wave velocity and thickness from inversion of the phase velocities for both individual and joint inversions.

The effectiveness of the joint inversion in this instance of limited data can be seen as the joint inversion achieves a greater accuracy in the phase velocity profiling, with respect to the true model, especially for phase velocities of higher periods.

A significant contributing factor to the success of the joint inversions is relative to the reduction of unknown parameters. Specifically, the layer thickness parameters h1 and h2 are represented twice with each independent inversion, but only once in a joint inversion:

Independent Inversion Unknown Parameters:

$$(\rho_1, \rho_2, \rho_3, h_1, h_2)$$

$$+ (Vs_1, Vs_2, Vs_3, h_1, h_2) = 10$$

Joint Inversion Unknown Parameters:

$$(\rho_1, \rho_2, \rho_3, Vs_1, Vs_2, Vs_3, h_1, h_2) = 8$$

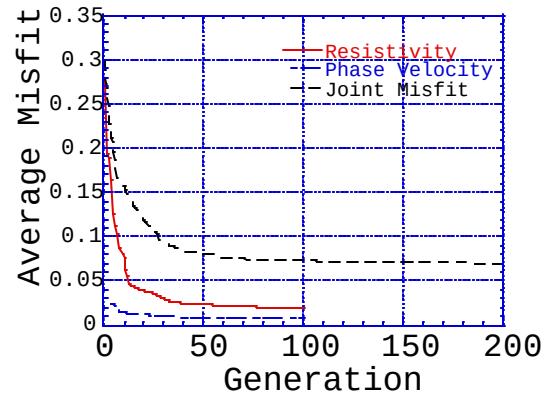


Figure 6

Comparison of misfit variations against generations in joint and independent inversions, for a misfit of 5%.

Shown in Figure 6 the final average misfit of the independent apparent resistivity and phase velocity inversions are lower than that of jointly inverted models. However, it is considered that the joint inversion results are within acceptable margins as the misfit stabilises, indicating the attainment of an optimum model of minimum

misfit for the 95th percentile.

CONCLUSIONS

Parameters obtained from the independent inversions aligned more effectively than those obtained through the joint inversion of apparent resistivity and phase velocity, for a profile modeled on a full set of observations.

Conversely, as anticipated the parameters generated by the joint inversion of a limited resistivity and phase velocity data set, attain more accurately to the true model.

Parameters of the joint inversion align more effectively overall, in comparison to those of the independent inversion, particularly in obtaining layer thickness parameters, resistivities of mid-depths and phase velocities of higher periods. Showing a greater effectiveness in attaining these parameters, the results confirm the credibility and applicability of joint resistivity and phase velocity inversions as a tool in shallow geophysical exploration.

As was to be expected in the case of decreased number of unknown parameters in the joint inversion, the final misfits of independent apparent resistivity and phase velocity inversion misfit functions are lower. However, as the joint inversion's 5% misfit function shows a steady misfit average around the 50th generation, it suggests the 95th percentile optimum model of minimum misfit is achieved.

Restricted to one jointly inverted model the depth of outcomes are limited, however, having chosen limited observations within small domains for the joint inversion, the average misfit and inversion results show an optimal model is being derived through the joint inversion of resistivity and phase velocity.

FURTHER RESEARCH

Investigation into a joint inversion of resistivity and phase velocity using observations obtained on site would be highly complementary to the analysis carried out on this synthetic model. Allowing a comparative analysis of a theoretical model versus the practical site application, the success of a joint inversion on limited data would be challenged further, uncovering any obstacles imposed by the true limits of on-site observations.

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REFERENCE

- Haskell, N:** 'The Dispersion of surface waves on multi-layered media', (in) Bulletin of the Seismological Society of America, Vol. 43, pgs17-34, 1953.
- Ohta, Y and Goto, N;** 'Estimation of S-Wave Velocity in Terms of Characteristic Indices of Soil', (in) Butsuri-Tankō Vol.29, No.4, 1976. (Japanese)
- Yamanaka, H and Ishida, H** 'Application of Genetic Algorithms to an inversion of Surface-wave Dispersion Data', (in) Bulletin of the Seismological Society of America, Vol. 86, No.2, pgs436-444, 1996.

Parameters Influencing the Shear Behaviour of Clay Infilled Rock Joints

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Abstract

The rock masses are heterogeneous and contain discontinuities. The discontinuities existing in rock masses are normally filled with fine material such as clay. The presence of clay infill in discontinuities has a major influence on the stability of rock mass.

The shear behaviour of infilled joints is significantly controlled by several parameters such as the type of joint and infill material, surface roughness and thickness of infill, drainage condition, the degree of overconsolidation etc. It is expected that the difference in shear strength between normally and overconsolidated states may be very large and the degree of overconsolidation of clay infill probably exceeds even the most overconsolidated deposits. Furthermore, a real danger is in the underestimation of pore pressure change, softening, and swelling. Therefore, the effects of pore pressure and the degree of overconsolidation on clay filled rock joints are analysed carefully with proper laboratory simulation.

Introduction

The shear strength of rocks and rock joints plays a key role in the field of Civil Engineering and Geo-sciences. Generally, rock masses contain discontinuities in the form of joints, faults and other such features with varying degrees of strength along their plane of weakness. The presence of discontinuities in rock masses not only decreases the shear strength, but also makes it more deformable and more permeable.

In nature, the discontinuities existing in rock masses are normally filled with foreign material. The choice of correct shear strength parameters becomes more difficult for joints in hard rocks, if they get filled with weak or loose material. These infill materials found in rock joints may be related to the origin of fracture itself with subsequent tectonic action or related to environmental conditions (e.g. infill material transported by water).

The typical infill material existing within joint interfaces can be divided into the following four categories ([1] Lama):

- Loose material brought from the surface such as sand, clay etc.,
- Deposition by ground water flow containing products of leaching of calcareous or ferruginous rocks,
- Loose materials from tectonically crushed rock, and
- Products of decomposition and weathering of joints.

The joint infill may exist in the form of partially loose to completely loose cohesive or non-cohesive materials under fully saturated or partially saturated condition. Any clay infill in a sloping discontinuity makes rock masses more slippery when it becomes wet and thereby promotes

failure. Thus, the presence of clay infill in discontinuities has a major influence on the stability of rock mass. The thickness of infill may vary from few microns to several meters. An increasing degree of complexity is introduced into the problem, when the thickness of clay infill is approximately less than the roughness amplitude of the joints.

Various research studies have been conducted in the past under different test conditions to investigate the shear behaviour of infilled rock joints. Those studies report that the shear behaviour is significantly controlled by several parameters such as joint type, the roughness of joint wall, thickness of the infill, stress history and pore water pressure.

Research on infilled rock joints

The majority of research on infilled rock joints has been empirical. Systematic research has been conducted under drained condition using direct shear tests (Constant Normal Load condition (CNL) and Constant Normal Stiffness condition (CNS)) to study the effect of surface roughness, different kinds of infill material, thickness of infill etc. on the behaviour of infilled rock joints. However, limited studies have been carried out on infilled joints under undrained condition, and no systematic research has been executed to model the overconsolidation effect.

Factors affecting the infilled rock joint behaviour

1. Type of joint and infill material

The shear behaviour of infilled joints is significantly influenced by the properties of rock joint such as compressive strength of rock and rock joint, roughness

amplitude of joint wall, the degree of weathering etc. and the type of infill material (eg. different types of clay, silt, sand etc.). In the past, laboratory tests have been conducted on natural joints and model joints such as concrete bricks and gypsum plaster bricks to investigate the effect of infill and thickness on the shear behaviour of infilled joints ([2] Indraratna et al., [4] Toledo & Freitas, [1] Lama, [7] Ladanyi & Archambault, [8] Kanji). Since it was impossible to use natural joints for systematic repetitive testing, model joints were produced precisely to match the same geometry and without much variation in the material properties. Their results vary with the type of joints and infill material. It is expected that some complication arises due to the infill that consists of widely graded material, for instance rock breccia down to clay.

2. Joint surface roughness and thickness of infill material

The infill thickness (t) and surface roughness amplitude (a) are the most important parameters controlling the shear strength of infilled joints. Several investigations have revealed the shear strength of infilled joint reduces with increasing infill thickness and eventually reaches to a value, approximately equal to the shear strength of infill alone at a critical (t/a) ratio ([2] Indraratna et al., [4] Toledo & de Freitas, [5] Papliangas et al., [6] Phien-wej et al., [1] Lama, [7] Ladanyi & Archambault, [8] Kanji, [9] Goodman). This critical (t/a) ratio varies with the type of joints as well as the type of infill material. Some critical (t/a) values for infill from test results are shown in table 1. Indraratna and Haque [3] have performed the tests under Constant Normal Stiffness (CNS) condition for bentonite clay filled joints. Figure 1 and Figure 2 illustrate the effect of infill thickness and surface roughness on the shear behaviour of the joints.

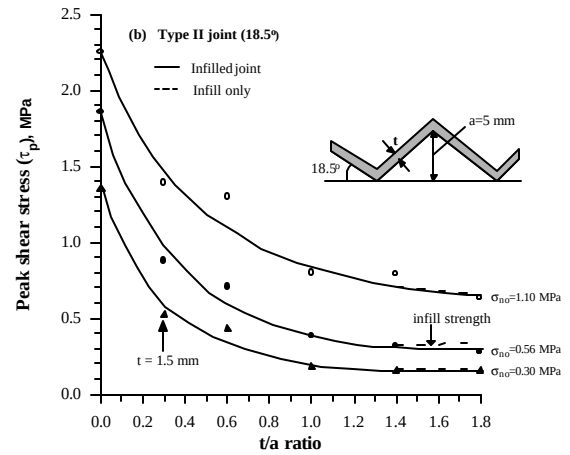
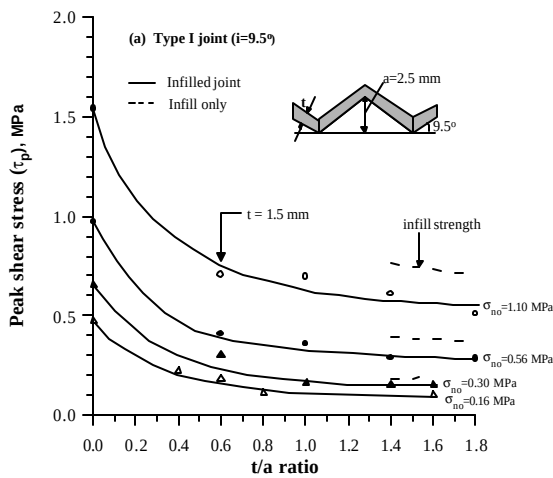


Figure 1. The variation of Peak shear strength with t/a ratio, for the type I and type II joints, (Indraratna and Haque, [3])

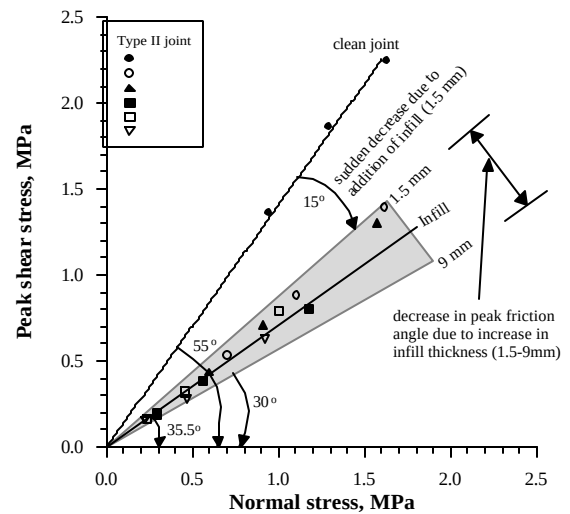
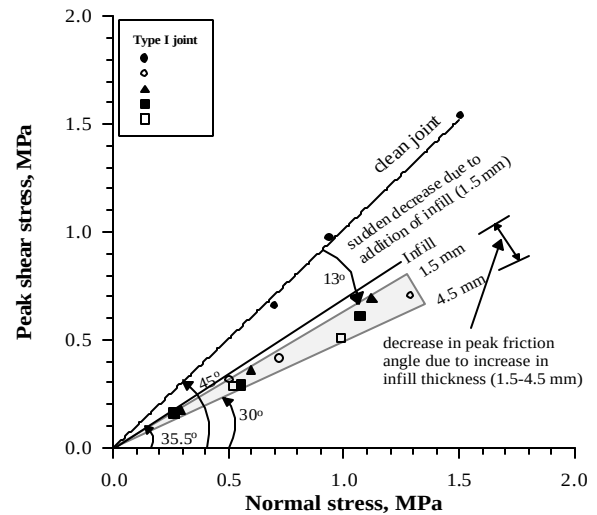


Figure 2 The effect of infill on the shear behaviour of rock joints, (Indraratna and Haque, [3])

Table 1. The Critical (t/a) ratio for different types of infill material and model rocks

Researchers	Type of model rock	Type of infill	Critical (t/a) ratio
Indraratna et al., [2]	Saw-toothed gypsum plaster block	Bentonite clay	1.4
Toledo and Freitas, [4]	Toothed Penrith sandstone	Clay	1
Papaliangas et al., [5]	Toothed, plaster cement bricks	Kaolin	0.6
		Fuel ash	1.25-1.5
Phien-wej et al., [6]	Toothed gypsum bricks	Dried bentonite	2
Ladanyi and Archmbault, [7]	Toothed concrete bricks	Kaolin clay	1
Goodman, [9]	Plaster-celite cast toothed joints	Crushed mica	1.25

3. Drainage condition, strain rate and pore water pressure

Drained or undrained tests are employed for effective stress or total stress analysis respectively. In all testing, the aim is to simulate the field conditions as closely as possible. The undrained shear strength of normally consolidated clay is always less than that of drained shears strength. But the undrained shear strength of overconsolidated clay may be smaller or larger than the drained strength depending on the value of overconsolidation ratio. For lightly overconsolidated clay, the volume tends to decrease during shear and undrained shear strength is less than the drained strength. However for higher values of the overconsolidation ratio, the clay tends to increase in volume during shear, the pore pressure correspondingly decreases and the undrained strength exceeds the drained value. The strong negative pore pressure associated with unloading would tend to draw water into the discontinuity causing the infill to soften and swell, whereupon the shear strength would steadily be reduced ([10] Barton).

Research has been carried out ([4] Toledo & Freits) on clay infilled joint at different strain rate under direct shear test condition. They concluded that the slower strain rate represents the drained condition whereas the faster represents the undrained condition (Figure 3).

As discussed by Lane [11], pore water pressure changes are sometimes critical to the safety of the engineering works on rocks, e.g. all kinds of dams, navigation locks,

power houses, spillway excavation etc. Several important failures of hydraulic structures and of natural slopes have been attributed mainly to the action of water pressure

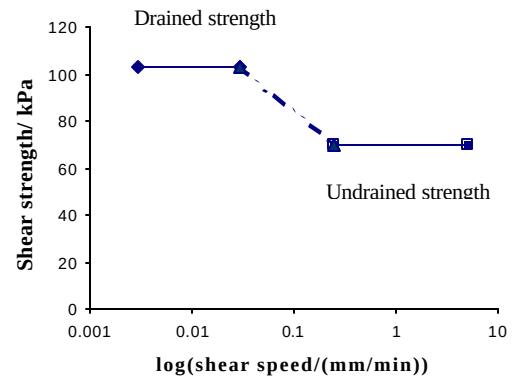


Figure 3. The effect of rate of shear on clay infilled rock joints (Toledo & Freits, [4])

inside the rocks and rock joints. Yet, experimental data on induced pore pressure on rock sample with joints and infilled joints are very scarce. The authors believe that this does not originate from the lack of appreciation of the problem importance but the difficulties in carrying out such elaborate experiments.

4. The degree of overconsolidation

In soil mechanics, the deposition-consolidation-erosion cycle which gives rise to overconsolidated clay is universally recognised, and the consequences are far reaching. The following discussion is directed towards the possible effects of overconsolidation on both infilled and unfilled joints intersecting the rock.

Increase in the shear strength of unfilled joints due to over-closure, an overconsolidation effect

In nature, the unfilled joints are never entirely closed and the degree of closure depends on the normal stress acting on them. It is possible for non-planar joints (particularly tension joints) to become mechanically over-closed. The shear strength of preloaded tension joints was found to be considerably higher than that of normally loaded joints ([12] Barton).

Barton [12] has investigated this effect on the natural rock joints using shear testing. Those samples were preloaded before shear testing under the lower engineering normal stress levels. However, the effective preconsolidation pressure, p_c , representing the undisturbed closure condition would not be known exactly. In an engineering approach to this problem, the historical preconsolidation pressure, p_c , had to be ignored and the problem simplified to an estimation of the ratio of overconsolidation caused by the progression from "undisturbed" to "post construction", for

instance in slope excavation. Here the post construction stress will be less than the undisturbed in situ stress, hence the overconsolidation effect. This degree of overconsolidation produced a significant over-closure effect in rough, unweathered model tension joints, which was reflected by a significant increase in the stability of the slopes between the two cases.

A set of preconsolidated direct shear tests performed by Barton [12], on model tension joints clearly demonstrates the potential increase in shear strength due to the overconsolidation effect. The tests have been performed for the overconsolidation ratios of 1, 4 and 8 as shown.

$$(1) S_n (\text{Preconsolidation}) / S_n (\text{test}) = 8/1$$

$$(2) S_n (\text{Preconsolidation}) / S_n (\text{test}) = 4/1$$

$$(3) \text{ Normally loaded } (1/1)$$

The test results show (Figure.4), there is an increase in friction angle of approximately 5° and 10° , when the model overconsolidation ratios are increased from 1 to 4, and from 1 to 8 respectively. Furthermore, Barton [12] concluded that the increased closure of the joints caused by high overconsolidation ratios necessitated larger dilation angles for the peak strength to be reached.

The effect of overconsolidation ratio on infilled joints

The infilled joints are overconsolidated, if the present stress is less than the preconsolidation pressure. It is probable that almost all discontinuities, filled or unfilled are in an overconsolidated state when exposed at the surface. There may be an additional overconsolidation effect between the stages “undisturbed” and “post construction”, since the undisturbed in situ effective normal stress may exceed the post construction effective normal stress particularly in the case of slope excavation.

The residual strength will be almost independent of the history of loading of the discontinuity. However, the history of loading has an enormous influence on the peak strength of infilled joints. Overconsolidated clay shows a marked difference in shear strength parameters between peak and residual due to reorientation of clay particles within narrow bands next to the shear surface. Furthermore, the dilation, accompanying shear in overconsolidated clay allows an increase in water content and consequent softening to occur ([10] Barton). The filled joints are classified as recently displaced joints and undisplaced joints to explain the importance of the overconsolidation effect on infilled joint.

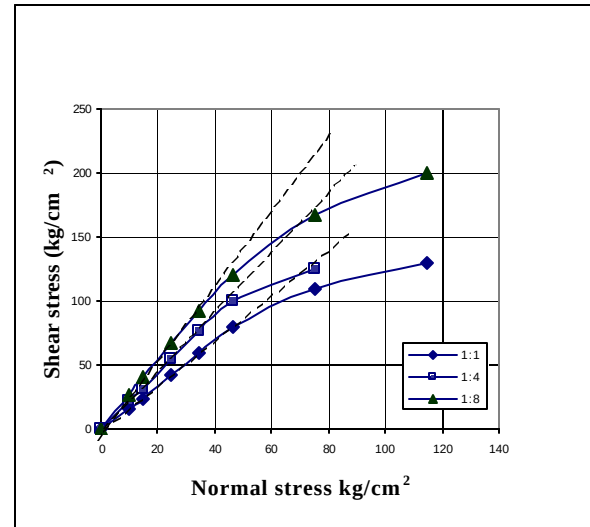


Figure 4. The effect of over-closure on unfilled joint, an overconsolidation effect (Barton [12])

In the case of recently displaced infilled joints such as shear zone and bedding plane slip, the shear strength will be close to the residual shear strength. Therefore, whether normally consolidated or overconsolidated is not of great importance. But, in the case of undisplaced infilled joints such as interbedded clay bands and hydrothermally altered fillings, the shear strength will be close to the peak shear strength. Therefore, it is important to consider the infill whether normally consolidated or overconsolidated to accurately analyse the behaviour ([10] Barton).

The influence of stress history, or paleo-overburden stress, on the shear strength of the clay-limestone contact was studied by subjecting the contact to confining pressures greater than the current *in situ* vertical stress ([13] Hatzor and Levin).

It has been shown that the paleo-overburden influences the frictional resistance of clay-limestone contact, from the test results of precompressed samples in conventional triaxial tests. Their test results justify that in the analysis of rock slope stability of clay filled discontinuities, the influence of paleo-overburden stress on frictional resistance must be resolved before the appropriate constitutive laws can be established for analysis.

Recent model on infilled rock joints

The effect of infill on the shear strength drop of rock joint is generally recognised ([7] Ladanyi and Archambault, [5] Papliangas et al). Recently, Indraratna and Haque [3] have predicted the strength drop (τ_p) (Figure 5) associated with infill thickness, relative to peak shear strength of clean joint using hyperbolic modelling. The tests have been carried out under constant Normal stiffness condition.

$$(t_p)_{\text{infilled}} = (t_p)_{\text{unfilled}} - \Delta t_p$$

Here, $(t_p)_{\text{unfilled}}$ has been modelled using energy balance concept, incorporating the Fourier series.

$$\text{And, the strength drop } (\Delta t_p) = s_{no} \left(\frac{t/a}{a \times t/a + b} \right)$$

s_{no} -Initial normal stress

a, b - Constants (Figure 6)

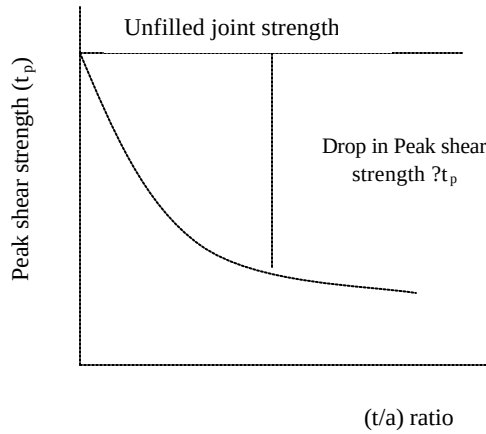


Figure 5. Peak shear strength drop due to infill, (Indraratna and Haque, [3])

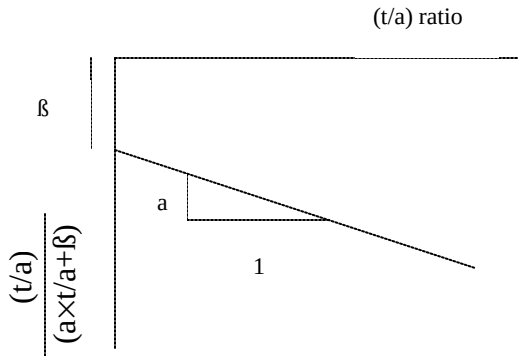


Figure 6. Formulation of hyperbolic model for the drop in peak shear strength due to infill, (Indraratna and Haque, [3])

Future research

In the past, most researchers have studied the behaviour of infilled rock joints using direct shear test condition without really simulating the actual stress path. Furthermore, tests on unconfined rock specimen were found to give

incomplete explanation of rock behaviour in situ. In addition, some of the most important aspects of the problem that need to be investigated clearly, are the effect of overconsolidation and pore water pressure development on shear and deformational characteristics of infilled joints. These effects cannot be monitored accurately in Direct Shear test condition

Therefore, the authors undertake this study with the main purpose of systematically investigating the above effects on the shear behaviour of clay infilled rock joints under triaxial condition.

Proposed experimental program

For this research program, carrying out triaxial tests using High Pressure Triaxial Apparatus (HPTA) at the University of Wollongong is recognised. Series of triaxial tests are carried out under Consolidated Undrained (CU) test condition to study the above effects. Since repetitive tests are to be conducted, model rock joints are cast. In this study, the effect of roughness and the thickness of infill are also analysed under triaxial condition. The infill material for the investigation is collected from a rockslide site at Kangaroo valley, NSW.

Analysis on infill material

The infill material collected from the site was stiff clay with the natural water content of between 14% and 16%, and the dry unit weight 18.16 kNm^{-3} . X-ray diffraction analysis of the collected clay revealed that it has 50.5% illite, 39.5% kaolin and 8.8% quartz. The Engineering Classification of the clay is Clay of Low plasticity (CL) with Liquid Limit (LL) 32-39% and Plastic Limit (PL) 21-22%.

Conclusion

The presence of infill significantly reduces the shear strength of joint, resulting in the reduced stability of entire rock mass. The shear behaviour of infilled rock joint is significantly controlled by several parameters such as roughness of the joint wall, thickness of infill, stress history, pore water pressure etc. The (t/a) ratio is used as an important parameter in the prediction of shear behaviour in most research. Any (t/a) ratio greater than the critical (t/a) ratio, the point at which the shear strength of the joint is at lowest value (in most cases, equal to the shear strength of infill alone), the infilled joint behaviour can be largely understood from well tested soil mechanics principal.

It is found that the pore water pressure effect is critical in the design of infilled rock joint. Moreover, it is recognized that there is a considerable difference in shear behaviour between normally consolidated and overconsolidated infilled rock joints. But no systematic research has been executed to analyse and model the above effects on infilled rock joint.

Therefore, a detailed study incorporating the above aspects of pore water pressure and overconsolidation effect is considered important for the better understanding of infilled rock joint behaviour.

Acknowledgement

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References

1. LAMA, R. D. "Influence of clay fillings on the shear behaviour of joints" Proceedings of 3^d international Association of engineering Geology, Madrid, 1978, Vol.2, Paper II-43, pp 27-34
2. INDRARATNA, B., HAQUE, A. & AZIZ, N. "Shear Behaviour of Idealized Infilled Joints under Constant Normal Stiffness" Geotechnique, 1999, 49, No 3, pp 331-355
3. BUDDHIMA INDRARATNA & ASADUL HAQUE. "Shear Behaviour of Rock Joints", 2000, Balkema
4. de TOLEDO. & de FREITAS. "Laboratory Testing and Parameters Controlling the Shear Strength of Filled Rock Joints" Geotechnique, 1993, 43, No 1, pp 1-19
5. PAPALIANGAS, T., LUMSDEN, A. C & HENCHER, S. R. & MANOLOPOULOU, S. "The Effect of Frictional Fill Thickness on the Shear Strength of Rock Discontinuities", International Journal of Rock Mechanics, Mining Science & Geomechanics Abstract, 1993, Vol 30, No 2, pp 81-91
6. PHIEN-WEJ, N., SHRESTHA, U B. & RANTUCCI, G. "Effect of Infill Thickness on Shear Behaviour of Rock Joints", Rock Joints, 1990, Balkema, Rotterdam
7. LADANYI, B. & ARCHAMBAULT, G. "Shear Strength and Deformability of Filled Indented Joints", The Geotechnics of Structurally Complex Formation, 1977
8. KANJI, M.A. "Unconventional laboratory tests for the determination of soil rock contact", Proc. 3^d Congr. Int. Soc. Rock mechanics, Denver, 1974, 2, pp.241-247
9. GOODMAN, R. E. "The deformability of joints. In determination of the in situ modulus of deformation of rocks" special technical publication (ASTM), 477, pp. 174-196
10. BARTON, N. "Review of Shear Strength of Filled Discontinuities in Rock" NGI Publication, OSLO 1974
11. LANE, K. S & HECK, W. J. "Triaxial Testing for strength of Rock Joints". Chief Geology, Soils & Materials Branch, Corps of Engineers, Missouri River Division, Omaha, Nebraska, 1964
12. BARTON, N. "Review of a New Shear Strength Criterion for Rock Joints" Engineering Geology, 7, 1973, pp 287-332
13. HATZOR, Y. H. & LEVIN, M. "The Shear Strength of Clay Filled Bedding Planes in Lime Stones- Back-Analysis of a Slope Failure in a Phosphate Mine, Israel", Geotechnical and Geological Engineering, 1997, 15, 263-282
14. RICHARD E. GOODMAN AND YUZO OHNISHI. "Undrained Shear Testing of Jointed Rock" Rock Mechanics, 1973, 5, pp 129-149
15. BARTON, N. "Rock Mechanics Review: The Shear Strength of Rock and Rock Joints", International Journal of Rock Mechanics, Mining Science & Geomechanics Abstract, 1976, Vol 13, pp. 255-279

Design of an Asphaltic Concrete Core Water Retaining Embankment in Iran

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ABSTRACT

A water retaining embankment was designed for the Miduk Copper Mine, near Shahr-e-Babak, in South-East Iran. This embankment forms part of a larger scheme for a down-valley thickened tailings storage facility which includes a tailings retaining embankment, decant system and associated emergency spillways. The water retaining embankment is located downstream of the tailings embankment. It is designed to collect bleed water and tailings surface and natural catchment rainfall runoff from the tailings storage facility. Water intercepted by this embankment is to be returned to the plant via an outlet pipe constructed through the embankment.

Selection of an asphaltic concrete core and the design of the water retaining embankment were based on two main criteria: the shortage of suitable core material and the associated costs of obtaining that material. This paper discusses the design process, including aspects of site selection, geotechnical investigation, material selection, stability analyses, seismicity, altitude and climate.

1 INTRODUCTION

The Miduk mine site is located 42 kilometres North-East of Shahr-e-Babak in Kerman Province, Iran. The general location is presented in **Figure 1** below.



Figure 1 - General Location in Iran

Australian Tailings Consultants (ATC) were contracted to undertake a feasibility study and then later design of the Tailings Storage Facility (TSF) for the Miduk Copper Project. Design of the TSF was completed in July 2003. Construction is currently underway.

2 PROJECT BACKGROUND

The Miduk Copper Project is a copper deposit which occurs within a fractured porphyritic granodioritic host

that intruded into a sequence of early tertiary aged andesitic volcanic rocks. The total ore reserve is calculated to be 170 million tonnes with an average grade of 0.8% copper. The processing is expected to produce 145.5 Mt of tailings over an expected 30 year mine life.

The project is managed by Olang Mining Services Company on behalf of the owner National Iranian Copper Industries Co (NICICO). Production is scheduled to start in late 2004.

3 WATER BALANCE ISSUES

Water is a valuable resource in Iran. The minesite intends to make-up water from a nearby bore-field. The purpose of the water retaining embankment is to intercept water from the TSF including rainfall runoff from the natural catchment and tailings surface, groundwater seepage, bleed water and consolidation seepage (from the tailings). For the minesite, the greater the volume of water collected by the water retaining embankment, the lower the demand on pumping from the bore-field.

Water intercepted by the embankment will be pumped back to plant. Under normal operating conditions this dam will not store water. It is estimated that the plant water requirement will be greater than the expected inflow into the water retaining embankment [Ref 1].

4 ALTITUDE AND CLIMATE

The Miduk minesite is located in a mountainous and arid region of Iran. Elevations around the site range from RL 2250 m up to RL 2842 m. Seasonal temperatures typically range from sub-zero in winter to the mid thirties in summer. The wet season is from November to April, with snow during the winter period. The average annual precipitation is estimated to be 250 mm.

5 SEISMICITY

Iran has a high seismic hazard risk. Three major faults exist in the Miduk region, Rafsanjan Fault to the east, Shahr-e-Babak Fault to the south, and the Anar Fault to the north. Seismicity data shows very few records of movement on these faults; this could be for several reasons including low level of activity on these faults compared with other parts of Iran, the relatively remote location, and poor seismograph coverage for this area (i.e. smaller earthquakes are not detected) [Ref 2].

Because the geology of the Miduk area is very young compared to most of the Earth's surface (Tertiary age or younger), the attenuation of the seismic waves with distance is high.

Two earthquake types were analysed for the Miduk site – the Operating Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE). The return periods for these two earthquake types were selected as the 1,000 years and the 30,000 years respectively [Ref 3]. The Peak Ground Acceleration (PGA) calculated for each of these earthquakes was 0.35g and 0.8g respectively [Ref 4].

6 SITE SELECTION

Site selection depended on several factors, these being the most efficient embankment alignment, the required storage capacity, and the results of the geotechnical investigation. The most efficient embankment alignment was selected to reduce material quantities and costs. The alignment also considered the suitability of the plinth alignment i.e. steepness of abutments. The embankment location was checked to see if the required peak event storage capacity was available.

7 GEOTECHNICAL INVESTIGATION

Investigations were undertaken in 2002 to characterize the site geology and available borrow material. These investigations included geological mapping of the proposed dam site, test pits in the proposed borrow areas, boreholes drilled in the proposed water retaining embankment site, and a review of the site geology.

The site investigation revealed bedrock consisting of andesite and andesite porphyry dykes, with surface deposits of alluvial and colluvial materials. A tectonised zone upstream of the right abutment and five minor faults were also noted during the site investigation. The faults encountered were analysed by an experienced geologist and an experienced seismologist. It was agreed by both that the faults in the vicinity of the dam were not active.

The alluvial and colluvial soils varied in depth from 0 m to up to 10 m. Typically the greatest depth of alluvials and colluvial soils were located in the gullies/drainage pathways.

The site investigation revealed groundwater flow in the order of 2 – 15 L/s through the alluvial soils (varies seasonally). Pressure packer testing showed the bedrock to have losses less than 2 Lugeons, however one borehole showed anomalous results of total loss. Those results were thought to be due to failure of the mechanical packer to seal the hole adequately.

8 EMBANKMENT SELECTION

Several embankment construction methods were assessed. The embankment types consisted of two main design types – a central core or an upstream membrane.

The central core options included:

- an impervious earth core, or
- an asphaltic concrete core.

The upstream membrane options included:

- a concrete deck,
- an asphaltic concrete membrane, or
- a geomembrane.

Concrete gravity dams were not considered due to the high seismic risk and presence of faults at the dam site.

The main advantage of a central core embankment over an upstream membrane is the reduced length of the core and grout cap/plinth and grout curtain, and subsequently reduced volumes of materials required. Other advantages included a simpler set out, ability to store water during construction, and the design feature of the core being buried and therefore not subject to damage or vandalism [Ref 1].

No suitable source of clay was located onsite during the borrow investigation. Because of this, the design of an impervious earth core embankment would be more complex. Large quantities of alluvial gravels and sands were available with fines contents varying between 4% and 36% [Ref. 1]. Earth core embankments can be constructed using moraine materials, but at Miduk the fine material exists in lenses within coarse alluvial material. A minimum fines content of 20% would be required for this type of embankment, and obtaining a consistent quantity of fines was considered to be difficult.

8.1 Asphaltic Concrete Core Selection

More than 70 asphaltic concrete core (ACC) embankments have been constructed to date, with the highest the Storglomvatn Dam in Norway standing 125 m [Ref. 5]. Typical core widths range between 0.4 m and 1.2 m depending on the final height of the embankment [Ref. 6]. The core is placed above a reinforced concrete grout cap/plinth. Most ACC embankments less than 50 m in height are constructed

with a vertical core which is supported laterally by a transition zone (grain size range of 0 to 200 mm), and then rockfill. The transition zone is not a filter; it merely provides lateral stability for the asphalt core during compaction, as well as providing drainage for the embankment.

The bitumen content typically ranges between 6 to 6.5 %. Excess bitumen is usually added to the mix to allow for absorption. Modified paving machines are used to place, level and partly compact the core and adjacent gravel material. The mix is compacted with vibratory rollers at temperatures of around 160 °C to 180 °C. Final air voids for the mix after compaction are typically less than 3 %. This gives an almost impervious core.

In 2002 ATC design personnel visited Norway for discussions with Norwegian designers and contractors to investigate whether an asphaltic concrete core dam could be successfully constructed in Iran. It was concluded that an asphaltic concrete core dam could be constructed in Iran using hand placement techniques.

ATC became aware of an ACC dam being constructed in Iran and arranged to visit the construction site. The site is located on the southern coast of the Caspian Sea. The construction of this particular embankment was using hand placement techniques. The plinth can be seen in **Figure 2**.

Construction of an asphaltic concrete core dam at Miduk would most likely be undertaken by a contractor with no prior construction experience with ACC dams. To ensure the correct construction of such an embankment, strict testing codes have been recommended as part of the design. During the design process it was assumed that hand placement techniques would be used, however, a Norwegian group have since been contracted to oversee the construction and provide a core placement machine.



Figure 2 – View of Meijoran Dam, Ramsar

8.2 Material Selection

Critical to the design of an ACC is the asphalt mix and compaction. The asphalt mix must match Fuller's curve: this is the grading which provides the densest mix. Material is also tested for suitability based on inter-particle friction, durability, wettability, plasticity index, size and gradation [Ref 7].

Local rock was tested as ACC aggregate and was found to be suitable.

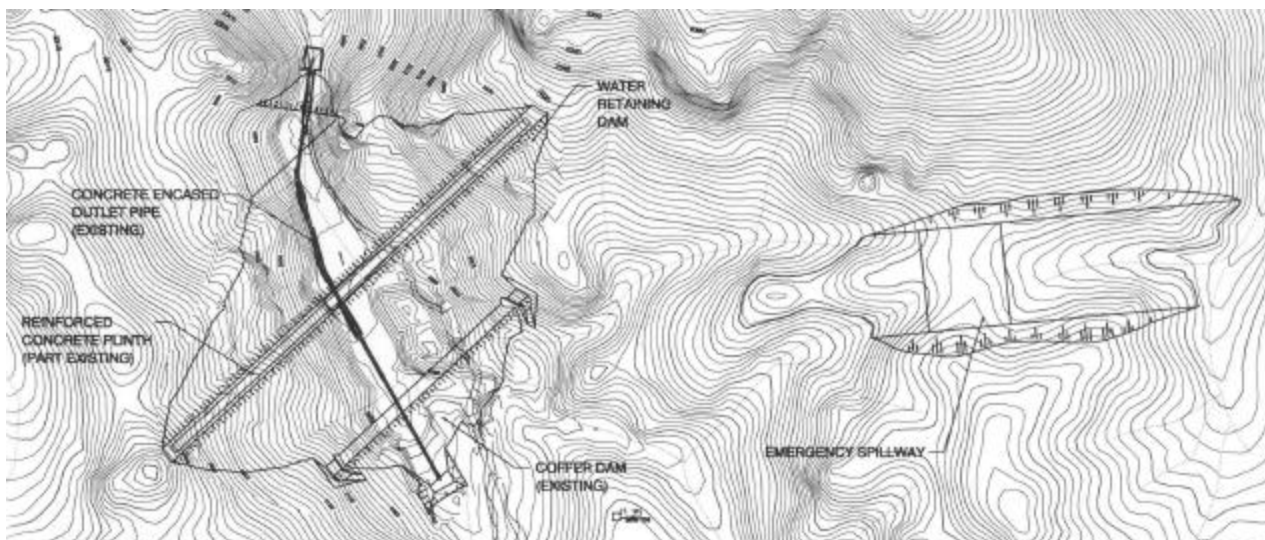


Figure 3 – General Layout

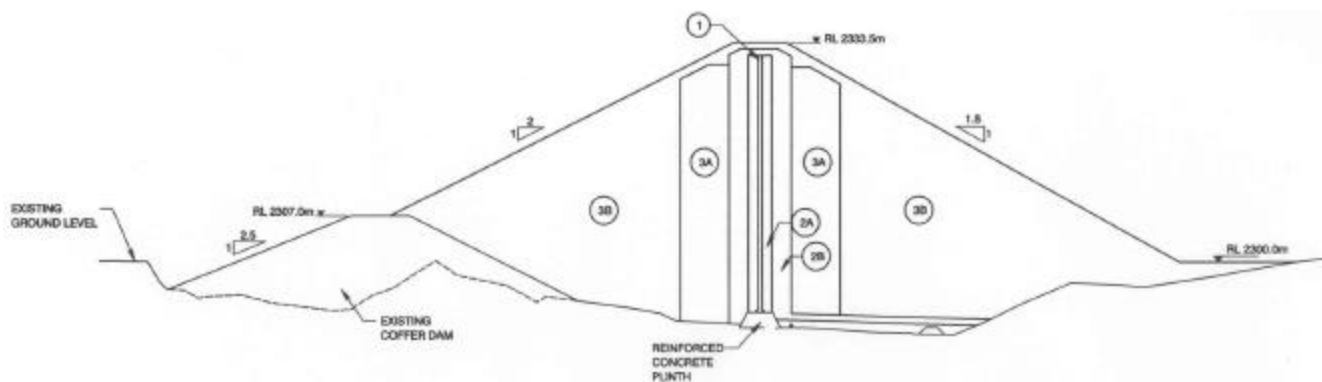


Figure 4 – Typical Section

9 DESIGN

The water retaining embankment was designed to store a 1 in 100, 72 hour storm rainfall event for the entire catchment (6.5 km^2). The rainfall intensity was estimated to be 4.8 mm/hr , and with initial and continuing losses, gave a required storage of $1.92 \times 10^6 \text{ m}^3$. The spillway design level was set at RL 2329.5 m. The spillway was designed to the east of the storage, and with a 50 m base width, and 1:1 batters.

Freeboard on the embankment accounts for wave run-up, and reservoir setup due to wind, the design flood depth in the spillway, and an additional 0.5 m safety margin. A 4 m freeboard was adopted. The design embankment crest became RL 2333.5 m. The final embankment height will be 43 m.

9.1 Outlet Pipe and Plinth

A 500mm diameter outlet pipe was designed to pass through the embankment. The size of the pipe was determined by the client on the basis of make-up water requirements. The outlet pipe is designed to recycle water from the water retaining embankment to the plant via a return water pipeline. The outlet pipe is encased in reinforced concrete and is founded on rock.

The plinth forms the foundation for the ACC. It is constructed of reinforced concrete and is also founded on rock. The layout of the outlet pipe and plinth is presented in **Figures 3**.

9.2 Cofferdam

The coffer dam is designed to RL 2307 m, and two spillways are constructed either side to RL 2305 m. Very little water is stored behind the coffer dam, it is constructed to provide protection to the works and to attenuate the flow of water during a storm rainfall event. The coffer dam spillways are designed for a 1 in 10 year rainfall event. The coffer dam is pervious, and is designed with a fine grained Zone 1 material and filter zones, with a rockfill outer shell. This shell would

provide protection to the coffer dam should the embankment be overtopped during a storm event.

9.3 Foundation Grouting

Foundation grouting was designed to consist of contact, blanket and curtain grouting. Contact grouting is undertaken to ensure a good seal between the plinth and the foundations. Blanket grouting is to extend to 10 m below the plinth, and curtain grouting to 30 m.

9.4 Water Retaining Embankment

The water retaining embankment was designed with a 0.6 m wide ACC. The core is widened at the base and at the abutments to ensure good contact with the concrete plinth. The embankment was designed with a crest width of 8 m, and an upstream batter of 2:1 (Horizontal: Vertical), and a downstream batter of 1.8:1. A berm was designed on the downstream batter at RL 2300 m to provide protection to the outlet pipe from rock fall where it passes through a steep gorge, as well as provide additional stability to the embankment.

9.5 Material Zones

All materials used in the construction of the water retaining embankment are to be sourced from hard, durable slightly weathered to fresh rock. A typical cross section is presented in **Figure 4**. The material zones are presented in **Table 1**.

10 STABILITY ANALYSES

Critical sections were selected for the water retaining embankment. Stability was assessed for various water levels. The stability of the structure was analysed using GEOSLOPE software.

Table 1 – Embankment Material Zones

ZONE	TYPE	MATERIAL	PLACEMENT
1	Asphalt Concrete Core	Max. particle size 19 mm aggregate, matching fullers curve, and bitumen.	200 mm compacted thickness.
2A	Core Support	Max. particle size 75 mm, aggregate matching fullers curve, non plastic material.	200 mm compacted thickness, 6 passes of a 1.5 tonne (static) smooth drum roller.
2B	Transition	Max. particle size 37.5 – 150 mm.	400 mm compacted thickness, 6 passes of a 10 tonne (static) smooth drum roller.
3A	Rockfill	Blasted rock, maximum particle size 300 mm.	400 mm compacted thickness, 6 passes of a 15 tonne (static) smooth drum roller.
3B	Rockfill	Blasted rock, maximum particle size 600 mm.	800 mm compacted thickness, 6 passes of a 15 tonne (static) smooth drum roller.
4	Drainage Gravel	Nominal 6mm aggregate.	400 mm loose thickness, 10 tonne (static), compact with vibration off.

Material strength parameters used in the stability analyses were as follows:

Table 2: Material Strength Parameters

Zone	Description	Depth (m)	Cohesion (kPa)	Effective Angle of Friction
1	Asphaltic Concrete		2000	-
2A	Granular Fill		-	38°
2B,	Rockfill	0 – 8.5	-	53°
3A,		8.5 – 22	-	49.5°
3B		22+	-	47.3°
4	Granular Fill (Drainage zone)		-	36°

The strength of rockfill is dependant on effective confining pressure, particle size distribution, strength of particles and degree of compaction [Ref. 5].

10.1 Factor of Safety against Sliding

Factor of Safety (FOS) against sliding was assessed and found to be approximately 2.

10.2 Static Stability

The static stability for circular failure was 2.6 for the critical upstream case, and 2.4 for the critical downstream case.

10.3 Seismic stability

10.3.1 Operating Base Earthquake (OBE)

The FOS was assessed for the OBE using the Corps of Engineers pseudo-static method [Ref. 8, Ref. 9]. This procedure sets the earthquake coefficient to 50% of the design acceleration, and drained strengths used are reduced to 80% of their original value. The method

states, that for a FOS of one or more, the maximum allowable deformation is one metre. The results of the analyses are presented in the table below:

Table 3: OBE Stability Results

Case	Water Level	FOS
Downstream	Maximum ⁺	1.46
Upstream	Maximum ⁺	1.27
	Half Height	1.29
	Operating Height*	1.34

*Crest Level of Cofferd Dam

⁺Full supply Level plus one metre

10.3.2 Maximum Design Earthquake (MDE)

The amount of deformation that occurs during the MDE was calculated using the technique presented by Makdisi and Seed [Ref. 9]. This technique requires the embankment response to the MDE, in particular the crest acceleration and natural period. These numbers were determined by Dr. E. Kavazanjian Jr. [Ref. 4].

The maximum crest acceleration, $\ddot{U}_{max}$ is 1.63g and the period, T_0 is 0.83 seconds. The procedure generally is as follows: a failure surface is selected, the yield acceleration for a FOS of one calculated, and the movement calculated based on the relationships presented in Makdisi and Seed [Ref. 10].

Trial surfaces analysed passed through the core and crest. A range of surfaces were analysed at various depths and water levels. The resulting displacements ranged from 0 m to 0.75 m. The maximum displacement was for a trial failure surface on the upstream face passing through the coffer dam, with a maximum water level.

This displacement is expected to occur as a zone of deformation. The major advantage of an ACC embankment is that the core is self healing. Because of

this, ACC embankments are well suited to seismically active areas.

11 CONSTRUCTION

Up to April 2004, all of the foundation stripping and part of the foundation preparatory works have been completed. Foundation stripping involved the removal of all the alluvial and colluvial materials, and completely weathered rock. The exposed foundation materials consisted of moderately weathered to faintly weathered and fresh andesite. Stripping was undertaken during the design process.

The outlet pipe is complete, more than half of the plinth has been constructed, and grouting is underway. The coffer dam has been constructed to protect the works during the wet season including emergency spillways. The plinth and some of the foundation stripping is can be seen in **Figure 5**.

12 CONCLUSIONS

Asphaltic concrete embankments have advantages in countries where water and clay is scarce and bitumen is readily available. The costs of constructing an ACC embankment are comparable to a concrete deck and clay core embankment. For seismically active areas such as Iran, an ACC embankment is ideally suited as it is self healing.

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Australian Tailings Consultants Pty. Ltd. and in particular Steven Murphy for the opportunity to participate in the design work for the Miduk Copper

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REFERENCES

1. Australian Tailings Consultants, "Olang Mining Services Company, Supplement to Draft Design Report, Design of Water Retaining Embankment, Miduk Copper Project", Reference 100056R33 Rev.1, July 2003, pg.1-21.
2. Seismology Research Centre "Review of Seismicity, Miduk, Iran, Preliminary Results", July 2001, pg.3.
3. Environmental System and Services "Design Accelerograms, Miduk, Iran", August 2002, pg.2.
4. Kavazanjian, Edward Jr. "Seismic Response Analysis Miduk Copper Project Water Retention Embankment" April 2003, pg.2.
5. Kjaernsli, B, Valstad, T. and Hoeg, K "Rockfill Dams – Design and Construction", Vol.10 Hydropower Development, 1992, pg. 111-113.
6. Committee on Materials for Fill Dams "Bituminous Cores for Fill Dams, State of the Art, Bulletin 84" CIGB ICOLD, 1992, p.49.
7. Creegan Patrick .J. and Monismith, Carl L. "Asphalt-Concrete Water Barriers for Embankment Dams" ASCE Press, 1996, pg.49-52.
8. ANCOLD "Guidelines for Design of Dams for Earthquake", August 1998.
9. Hynes-Griffin, M.E. and Franklin, A.G. "Rationalising the Seismic Coefficient Method", 1984.
10. Makdisi, F.S. and Seed, H.B. "Simplified Procedure for Estimating Dam and Embankment Earthquake Induced Deformations", J. Geotech. Eng. Div. ASCE, Vol. 104, No. GT7, July 1978, pg. 849-867.



Figure 5 – Water retaining embankment (under construction) March 2004.

Soil Desiccation from Vegetation and its Influence on the Cyclic Stiffness and Strength of Clay Soils

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Abstract: When engineers notice vegetation next to a structure, typically the first thought that comes to mind is usually a negative one. Anecdotal evidence from railway employees suggested the opposite; a noticeable improvement in rail track performance was observed where there were substantial stands of trees adjacent to the track. This perception led to a preliminary study conducted by the author and a colleague. This limited study showed an improvement in soil strength and stiffness at the sides and shoulders of the trackbed in the vicinity of trees on sites with highly plastic clays. The current study investigates trackbed properties, in particular the cyclic stiffness and undrained - unconsolidated (UU) shear strength directly underneath and adjacent to the track. Soil samples were taken from vegetated and non-vegetated sites containing expansive clay soils in western Victoria and Queensland. The vegetated and non-vegetated sites were selected within close proximity of each other to minimize variations in soil properties. This paper reports on the findings of the first half of the project, based on sampling in October on the Victorian sites and in November in Queensland. The results presented in this paper confirm the vegetated sites have improved trackbed strength and stiffness, mainly attributed from an increase in total soil suction (and decrease in soil moisture content).

INTRODUCTION

Vegetation and bioengineering techniques are rapidly developing for improving soil stiffness, slope stabilisation and erosion control. The feasibility of employing these environmentally friendly techniques to enhance the ballasted tracks conditions, particularly in areas with expansive soils, has yet to be investigated.

In Australia, rail corridors have been built on many areas over soft soil formations and upon stiff clays, which are very moisture sensitive (expansive soils which swell and soften on wetting and which harden and shrink as moisture is depleted). On soft soils, poor drainage of the rail formation may cause localised undrained failure of track. On expansive soils, the major problem appears to be the continual softening of subgrades from moisture permeating the ballast and collecting at the ballast-subgrade interface, as it can not readily evaporate in drier seasons. Softened subgrades also lead to accumulation of settlements with cyclic loading.

Although it is well known that trees can significantly reduce soil moisture, there has been a reluctance to adopt vegetation-based stabilisation by geotechnical engineers. One reason is partly due to the unknown performance of vegetation and possibly non-uniform patterns of soil moisture reduction.

The chief goal of the study is to quantify how, where and under which conditions the presence of vegetation leads to an improvement in subgrade cyclic stiffness and strength.

LOCATION OF TEST SITES

Sites were selected from known problem areas reported by track maintenance staff. These areas were narrowed down to where expansive clay soils were evident. A pedological map by Northcote^[1] (1975) was also used in the selection process to test on four distinct soil types (given in Appendix A in the site descriptions). The locations of test sites are given in Figures 1 and 2.

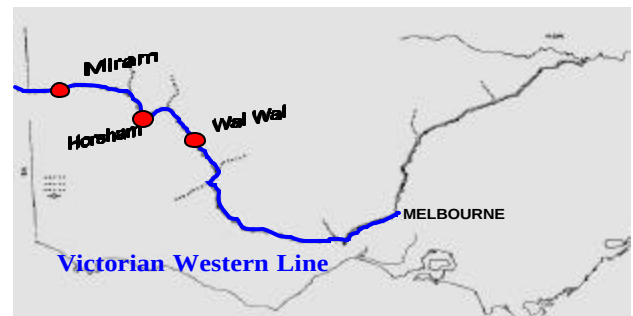


Figure 1 Victorian sites (Miram, Horsham and Wal Wal)



Figure 2 Queensland test site (Emerald)

METHODOLOGY

Thirty continuous cores in total (five at each site) were extended to a depth of at least 4.5 m as illustrated in Figure 1 at the eight test sites (four general locations, sites with and without trees). Atterberg Limits were conducted to assist the classification of soil types using the Unified Soil Classification System.

Total soil suctions were measured in the laboratory with a Dew Point Hygrometer according to AS1289.2.2.2-1992. Moisture content, Atterberg limits and Measurements for soil suction and moisture contents were taken from the continuous cores at 0.1, 0.4, 0.7, 1, 1.5, 2, 2.5, 3, 3.5, 4 and 4.5 m intervals.

Four 50 mm diameter push tube samples were collected to test trackbed subgrade shear strength, using the UU three stage triaxial test, performed according to AS 1289 (Standards Australia ^[2], 2000). Four 72 mm diameter samples were collected for testing to determine the differences in the cyclic stiffness. Push tubes samples were taken at nominal depths of 0.25 m, 0.5 m, 0.75 m and 1.0 m also shown in Figure 1.

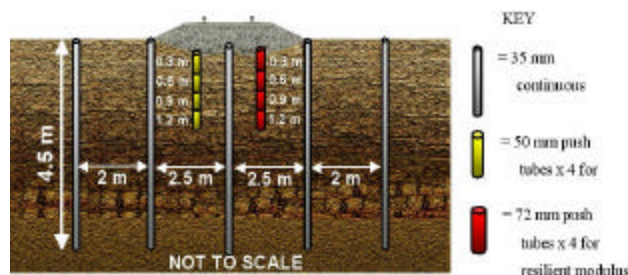


Figure 1 Cross section of sampling

The method used for resilient modulus testing was based on principles from AS 1289, Mundy *et al* ^[3], (1995) and the American Association of State Highway Officials (AASHTO) ^[4], 1994. The preparation of samples and procedure for the cyclic loading test used, including values for input parameters are documented in Potter and Londema ^[5] (2002).

RESULTS

The test results are divided into the following categories: moisture content and total soil suction, static triaxial and cyclic stiffness testing.

Moisture Contents and Total Soil Suctions

Figure 2 shows the comparison of moisture content profiles underneath the track centres of vegetated on non-vegetated sites for all locations.

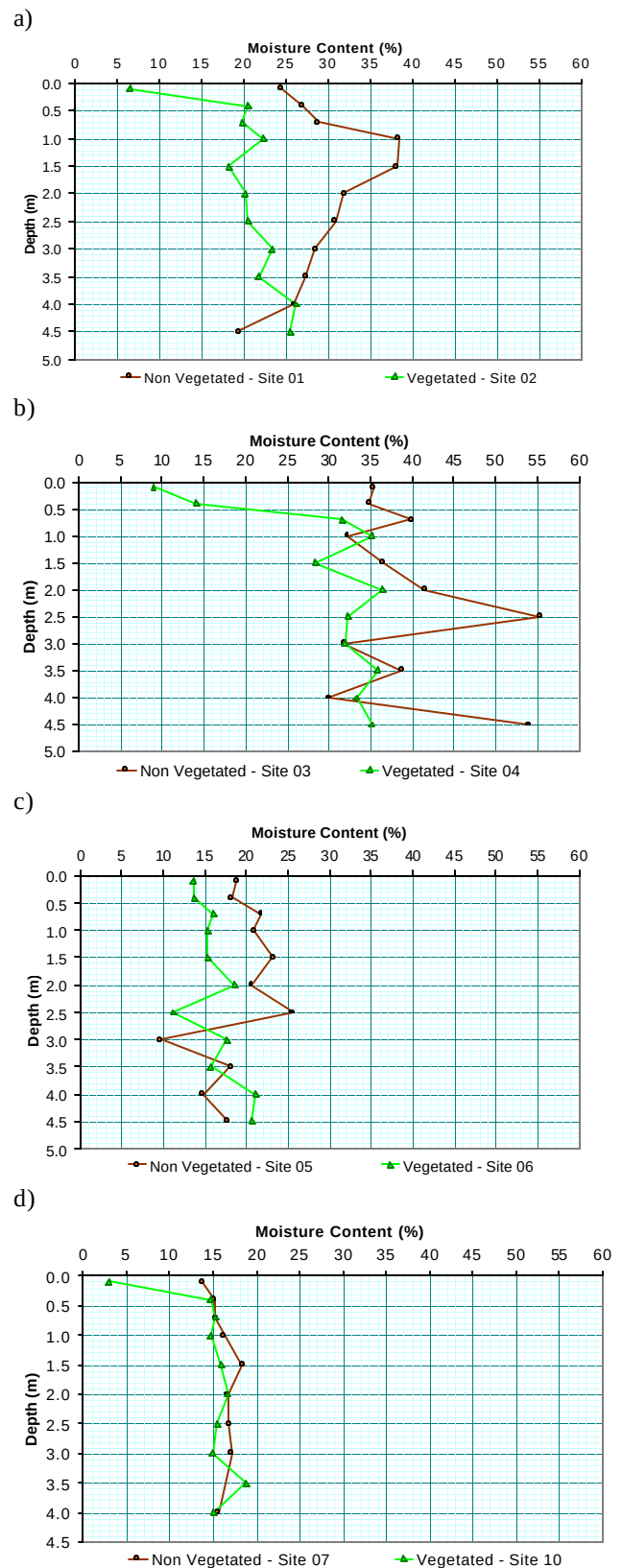


Figure 2 Moisture content profiles under track centres for a) Miram b) Horsham, c) Wal Wal and d) Emerald

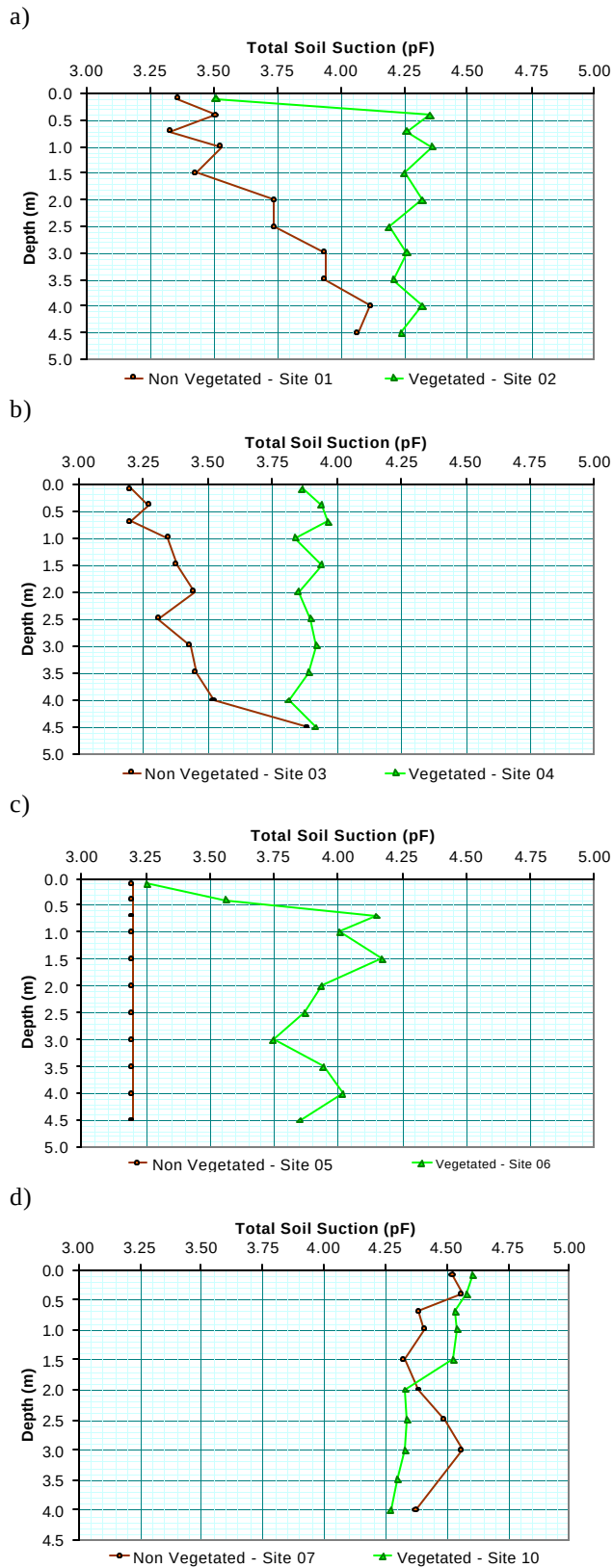


Figure 2 illustrated the vegetated sites had a strong trend of drier soil profiles down to a depth of three to four metres at the Victorian sites. Emerald was different, where only the top half metre was drier, then the profile was much the same. Total soil suction measurements revealed a much clearer trend in that the influence of the vegetation was still affecting soil suction level down to a depth of 4.5m. The Wal Wal non-vegetated site 05 indicated that the entire soil profile was near the point of saturation. The Wescor hygrometers have a limit of reading accurately only to a pF of 3.2, therefore any values recorded lower than this were recorded as 3.2pF. Again, Emerald proved to show only a slight increase in soil suction, however this occurred down to a depth of 1.5 to 2 m..

Static Triaxial Testing

The UU three stage triaxial test was used to calculate the shear strength of the top 1 m of subgrade directly below the ballast-subgrade interface. Unfortunately, not all samples were able to give a result due to the sample containing 'foreign matter' such as sub-ballast or natural intrusions such as calcium carbonate or tree roots. Other samples also failed early in stage one due to being brittle. Figure 4 shows the results of the successful tests.

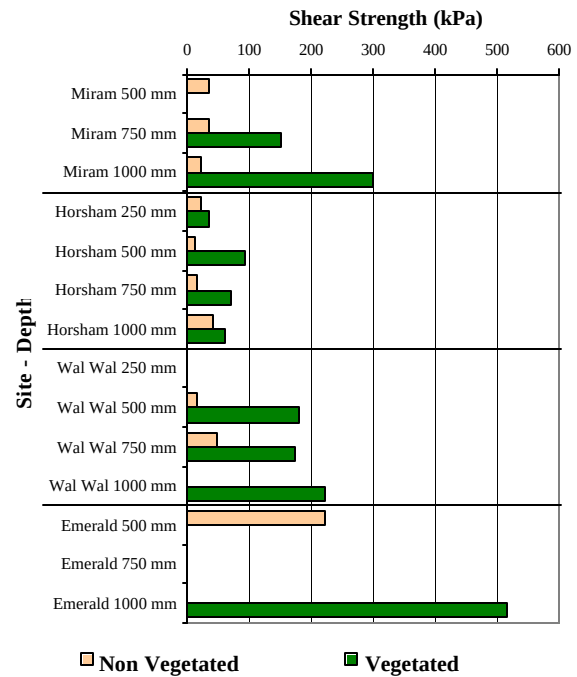


Figure 4 Shear strength (UU) comparison

Although there a quite a few missing results there is a definite relationship that exists. In all cases where a comparison can be made, the vegetated sites have higher shear strengths, with the Miram site at 1 m depth having more than a ten-fold increase (around 30kPa as opposed to 300kPa).

Resilient Modulus Testing

A comparison of resilient modulus is difficult to present concisely as it is not a single value, the range of values are dependent on input parameters such as deviator stress, σ_1 , and confining pressure, σ_3 . To overcome this, the author has adopted a modified version of Thompson and Robnett's 'break point modulus', Mr_{bp} (cited in Lee *et al* ^[6], 1997). The Mr_{bp} value is taken at the intercept where $\sigma_1 = 41.4$ kPa on the curve where σ_3 is closest to 20 kPa. An example of how a typical Mr_{BP} value is derived is demonstrated in Figure 5 below.

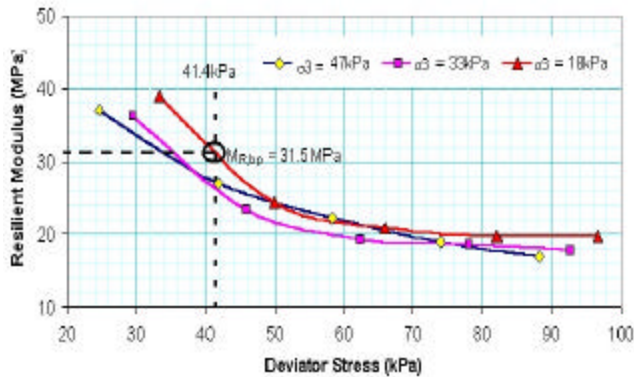


Figure 5 Example of a break point modulus value

The resilient modulus break point values were calculated and are presented in Figure 6 below. At a glance, it is easy to see that the vegetated sites all fared better than the non-vegetated sites.

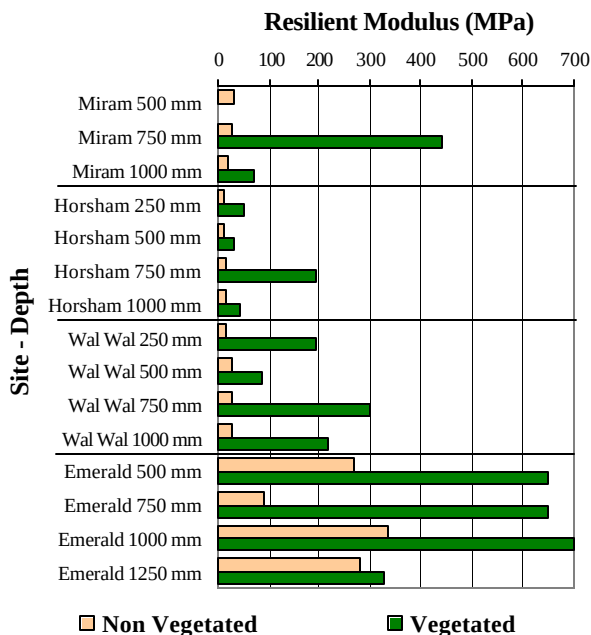


Figure 6 Cyclic stiffness comparison using Mr_{bp} values

Results shown in Figure 6 were not a complete surprise, as it is well known that resilient modulus improves with a higher suction value. It was however, very interesting to see such a dramatic increase (at least double the values down to 1 m) in resilient modulus at the Emerald sites, given that there was not a big difference in moisture content or soil suction.

CONCLUSIONS

The data presented gives a strong case that the presence of vegetation has benefits in improving the strength and cyclic stiffness of clay soils. It is however, only the first step in a series of understanding how to implement strategies using vegetation as a remediation tool. The next stage investigates the same sites again, and will focus on the seasonal variability on soil properties of the vegetated and non-vegetated sites.

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REFERENCES

- [1]. Northcote, K H, Hubble G D, Isbell R F, Thompson C H And Bettenay E., " A Description of Australian Soils ", 1975, CSIRO Australia.
- [2]. Standards Australia, , "AS1289; Method of Testing Soils for Engineering Purposes", 2000, New South Wales
- [3]. Mundy, M., Andrews, B., Stacy, W., Brimble, R., Gray, N., "Material Research and Development Report No. 66-1; Determination of Stiffness and Rutting Potential of Subgrade Soils for Pavement Design", 1995, Department of Transport, Materials and Services Section, South Australia.
- [4]. American Association of State Highway and Transportation Officials (AASHTO), "Standard Test Method for Determining the Resilient Modulus of Soils and Aggregate Materials, Designation TP46-94", 1994, Washington.
- [5]. Potter, W. and Londema, W., "The Influence of Vegetation on Rail Track Stability Along the Adelaide to Melbourne Corridor ", 2002, Honours Thesis, University of South Australia
- [6]. Lee, W., Bohra, N., Altschaeffi, A., White, T., "Resilient Modulus of Cohesive Soils", 1997, Journal of Geotechnical and Geoenvironmental Engineering, vol 123, No. 2, pp. 131-136.

APPENDIX A GENERAL DESCRIPTION, VEGETATION AND SOIL TYPE OF TEST SITES

SITE DESCRIPTION	VEGETATION PRESENT	SOIL TYPE (Northcote ^[1] 1975)
<u>Site 01 (Miram Non Veg)</u> About 2km west of the town of Miram – relatively remote	Has no trees within 100m, but sparse ground cover is evident at the shoulder	Red Duplex
<u>Site 02 (Miram Vegetated)</u> 150 m west along the track of site 01	8m tall trees are present on the south side of the track, located approx 5m away from the track shoulder	Red Duplex
<u>Site 03 (Horsham Non Veg)</u> On a section of corridor that runs parallel to the Western Highway	No trees within 2km, however some growth near the track shoulders consisting mainly of 100 to 200mm high weeds	Deep Grey Cracking Clay
<u>Site 04 (Horsham Vegetated)</u> Also on a section of track that runs parallel to the Western Highway. 2 km northwest of site 03	The vegetation is on one side only on the southern side closest to the Western Hwy, consisting of 15m high Red Gums and Sugar Gums, located 3m from the track shoulder.	Deep Grey Cracking Clay
<u>Site 05 (Wal Wal Non Veg)</u> Access to this site can be quite difficult in wetter months. Located in a remote section of corridor	A few remnant small shrubs near the boundary, however unlikely to be influential as the root system is assumed to have no influence beyond 2m given their small size	Yellow duplex clay with sandy lenses in the first 4m of the soil profile.
<u>Site 06 (Wal Wal Vegetated)</u> 50 m south along the track from site 05	Clusters of 10-12m tall trees exist on both sides of the track approx 10m away from the track shoulders.	Yellow duplex clay. Sandy lenses as above.
<u>Site 07 (Emerald Non Veg)</u> This site should be just clear of the hydrostatic influence zone of the dam located around the 7.5 km. Cotton fields are adjacent to both sides.	Graded about 2 years ago, this site contained 2-3m high shrubs. Presently it contains no vegetation within 3m of shoulder .A few small saplings exist about 7m away.	Black Earth
<u>Site 10 (Emerald Vegetated)</u> Located approximately 450 m west of site 07	A mature stand of 12-15m tall trees is present on the southern side only. The trees are approximately 7m away from the toe of the ballast.	Black Earth

Design and construction of a multi-anchored retention system within Ashfield Shale

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Excavation for a four-storey basement alongside a busy commuter railway line required both temporary and permanent support within the Ashfield Shale formation. Rail authority requirements were such that the new building had to remain independent from the retaining wall supporting the railway line. A multi-anchored wall was therefore adopted to permanently support the excavation.

The design of the permanently anchored retaining wall was carried out using allowable or 'working' stress methods based on the worst 'credible' design wedge load, taking into account surcharge and earthquake loading. A probabilistic analysis was used to determine the worst 'credible' design wedge load acting on the wall based on a review of geological data in the area. This paper briefly presents some of the considerations involved in the design of the permanent wall with emphasis on the methods used and experience gained during construction.

INTRODUCTION

The basement excavation covered the entire area of the site to depths ranging from 13 m to 17 m below existing surface levels, as shown in Figure 1. The permanent retaining wall supported the 100 m long eastern side of the excavation, which extended to a depth of approximately 17 m below the railway line.

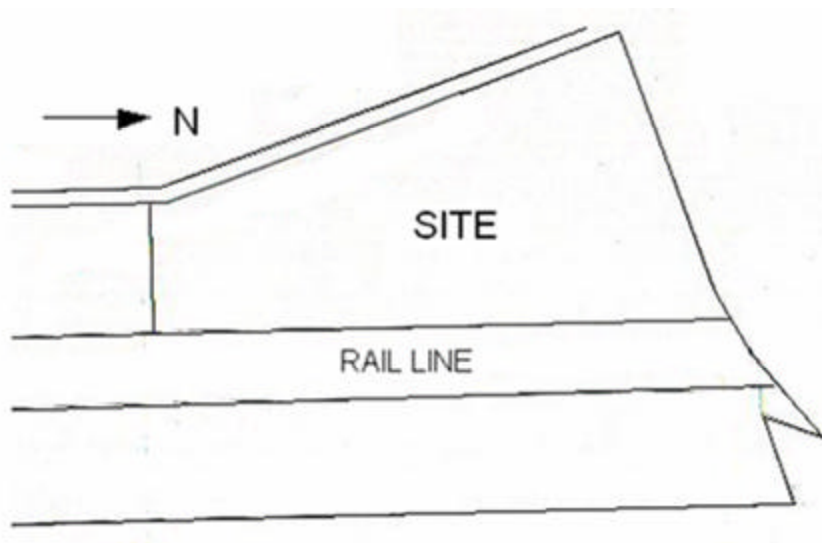


Figure 1 – Site Plan

The railway line originally ran along an open section of track on top of an earthfill embankment of 2 to 3 m height. Prior to initial excavation at the site, a Railway Enclosure Structure (RES) was built around the rail line for the length of track included in the development. The RES formed an above-ground tunnel structure of approximately 20 m width and about 10 m high over the rail corridor. The RES was constructed on beams supported by large diameter concrete filled steel piles founding at typical depths of 20 m to a level below the base of the proposed excavation. A photograph of the RES on the earthfill embankment prior to excavation is shown in Figure 2.

Site investigations revealed the following typical subsoil profile at the site:

Depth (m)	Material Description	Comments
Surface to 1.5 m	FILLING: Shale, crushed sandstone and clay fill	Original railway embankment
1.5 m – 3.5 m	CLAY: Hard, red brown clay	residual clay
3.5 m – 6 m	SHALE: Very low strength, light grey	Ashfield Shale
6 m – 14 m	LAMINITE/SHALE: Low strength, dark grey	
14 m – 18 m	SHALE: Medium to high strength, fresh	
Below 18 m	SANDSTONE: Medium strength	Mittagong Formation

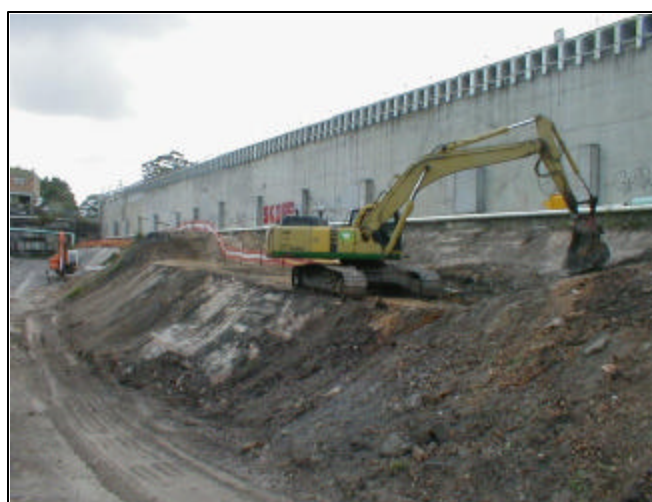


Figure 2 – RES and rail embankment prior to excavation

DESIGN AND SPECIFICATION

The retaining wall system comprised reinforced concrete wall panels of about 10 m length, spanning between the large diameter shoring piles, anchored to the excavation face with horizontal ties or inclined rock anchors. The ties comprised multi strand cables running below the RES structure and tied to the concrete panels on either side of the rail corridor.

Both lateral earth pressures and mobilised wedge loads were considered in the design of the RES wall, in addition to surcharge and earthquake loading. The mobilised wedge loading was found to be significantly more than the lateral earth pressures for retained heights of more than about 8 m, and this was the governing failure mechanism used to design the retaining wall.

The magnitude of the design wedge loading was determined using a probabilistic assessment of joint mapping data collected from the area. The worst ‘credible’ design wedge was assessed to be formed by the intersection of joints dipping at 45° with at least one joint having an incident angle to the strike of the excavated face exceeding 30° . The horizontal wedge load applied to the wall was calculated to be 875 kN/m. This process is described in greater detail in Andrews & Braybrooke 2001 (Reference 1) and Douglas Partners unpublished report No. 28994G ‘Review of Geological Data for Perimeter Shoring Design’ 2002 (Reference 2).

The vertical layout of horizontal ties and inclined rock anchors over the permanent wall is presented in the typical cross section shown in Figure 3. The final design consisted of three rows of horizontal ties and three rows of inclined rock anchors, with approximately 20 ties or anchors per row. The permanent anchor lengths were designed based on the allowable working bond stresses within the low and medium strength shale shown in Table 1, and a free length extending behind a 45° plane from the toe of the wall with allowance for debonding within the bond length over the life of the structure.

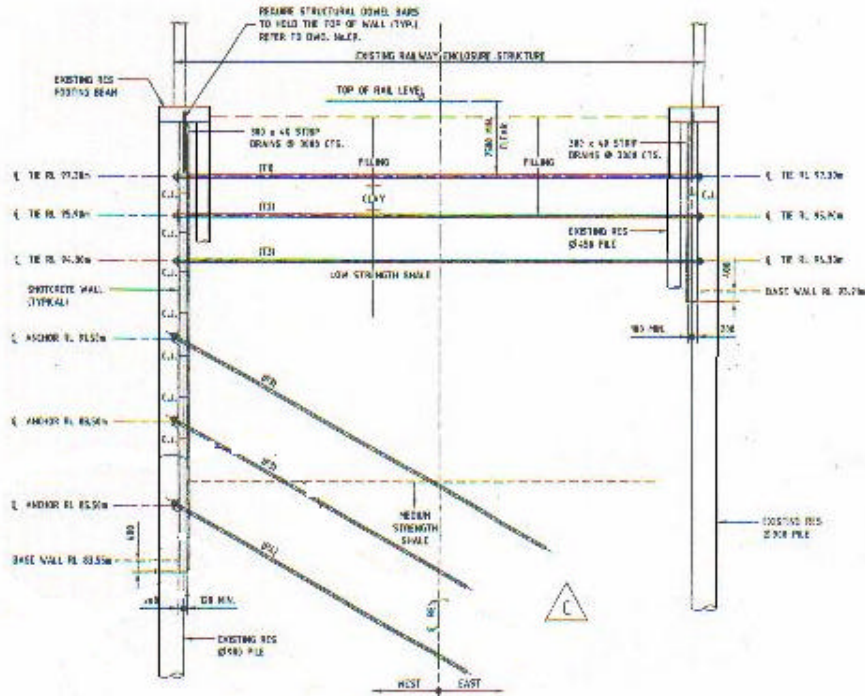


Figure 3 – Typical wall cross section

Class Shale	Strength	Design Working Bond Stress (kPa)
III	Low strength	200
II	Medium strength	500

Table 1 – Design Working Bond Stress

Most of the anchors had 7 strands (250 kN Ultimate Tensile Strength per strand) with a design working load of 875 kN. The anchor lengths ranged from 10 m to 16 m depending on the free length with the bond lengths ranging between 6 m and 8 m. A typical cross section through a permanent anchor is shown in Figure 4.

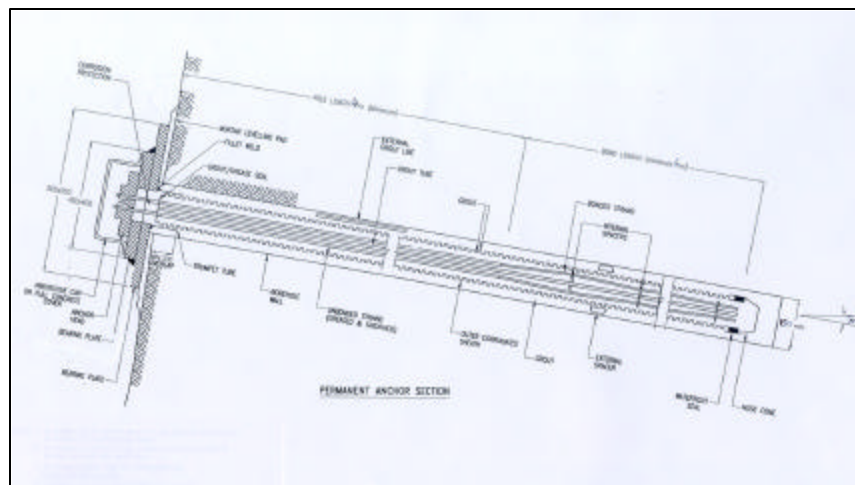


Figure 4 – Typical cross section through permanent anchor

The design specification called upon a design life of 100 years for the anchors supporting the permanent wall. Corrosion protection measures were incorporated for the anchors including an outer plastic sheathing to encapsulate the anchor strands and provide an additional barrier to corrosion through hairline cracking in the anchor grout. The anchors were also electrically isolated from the wall with isolation bearing plates beneath each anchor head to reduce stray currents.

CONSTRUCTION

Douglas Partners Pty. Ltd. (DP) was engaged to supervise the construction of the permanent retaining wall. This included face mapping and recommending lift heights through the soil/rock profile and supervision of the installation of the permanent anchors. The author was involved in this project from the beginning of the installation of the inclined rock anchors, discussed below.

Face Mapping

DP were required to be present during excavation for each panel of the RES wall to confirm that the conditions exposed were consistent with the design parameters and to advise on temporary stabilisation measures, where required.

Through the filling and upper clays this generally involved limiting the height of the lifts to around 1.5 m to reduce the risk of collapse of the temporarily unsupported face. Steel reinforcement and strip drains were placed prior to shotcreting each panel.

The faulting and jointing was mapped and measured for every panel exposed within the shale, allowing larger features to be identified and tracked to lower panels on the wall. A typical joint exposed within the shale is shown in Figure 5.



Figure 5– Typical exposed joint



Figure 6– Drilling of the anchor holes

Construction Methods

Lift heights of between 2.5 m and 3.5 m were adopted through the laminate/shale. Reinforcement was placed over the wall panel following final trimming of the face to ensure adequate concrete cover to the steel. Anchor pads were constructed at each anchor location prior to shotcreting.

The shotcrete was applied using a hand held nozzle fed by a concrete pump truck located at surface level. Drilling of the anchor holes commenced 1 day after shotcreting of the panels (Figure 6). Typically 3 anchor holes were drilled into each panel between the large diameter piles. The holes were inclined at 30° to the horizontal and extended to between 12 m and 16 m depending on the free length of the anchor to be installed. 150 mm diameter holes were used to allow for placement of the 125 mm diameter anchor assembly. Drilling resistance and cuttings returned by the compressed air from the drill bit were used to confirm the strength of the anchor bond material.

Following drilling, the holes were filled with water and the rate of water loss from the hole monitored. The rate of water loss was used to indicate the potential for grout leakage from the holes into the joints within the shale during the grouting stage and the need for the hole to be sealed with grout and re-drilled prior to the installation of the anchors. The water leakage observed in the drilled holes was relatively minor and grouting of the holes was only required for a few anchors.

Following the water test the water was flushed out of the holes with compressed air pumped into the bottom of the holes using a long tube. The permanent anchors were then inserted into the holes in preparation of the outer sheath integrity test. The outer sheath (a corrugated plastic tube) of each anchor was filled with water to detect leakage which would indicate defects within the corrosion protection sheath around the anchor. One anchor failed this test and was removed to reveal a crack in the outer sheath which was repaired, re-inserted and re-tested. Grouting of the anchors commenced once each anchor had passed the outer sheath test.

The grout mixture was prepared on-site to a predetermined water:cement ratio with a electric grout mixer. The grout was injected into the 15 mm diameter grout tubes extending out of the anchors to allow a grout column to be poured from the bottom of the anchor to the top. The grout was then injected into the outer tube to grout the void between the outer sheathing and the rock. The grout levels were inspected at regular intervals after grouting to monitor grout loss from the outer section of the anchors. Grout cubes were cast to monitor the strength gain of the grout with time and select a suitable time to commence stressing. An admixture was used halfway through the construction of the wall to accelerate the strength gain of the grout and allow for a quicker turnaround between constructed anchor rows.

Anchor Stressing & Monitoring

The anchors were stressed using the jack shown in Figure 7, which was lifted using a 7 tonne excavator. The anchors were proof loaded to 125 % of the design working load in 5 equal increments with the load held at each increment for 30 seconds and the final proof load held for 10 minutes to allow time for anchor-wall movements and some movement of the wall within the extremely low strength shale. Once the anchors had demonstrated that they could successfully hold the proof load they were locked-off at 5% above the design working loads. Lift-Off testing was carried out on each anchor to provide a reference force in each anchor to compare to future lift-off test results.



Figure 7– Stressing of Anchor

Following the immediate lift-off test the next panel below was excavated with the above procedure repeated until the full depth of excavation had been reached. A photograph of the completed wall is shown in Figure 8.



Figure 8– Completed Permanent Multi-Anchored Retaining Wall



Figure 9– Grease capped RES anchor

Lift-off testing was carried out on all anchors at 24 hours and 7 days after stressing with 15% of anchors tested after 28 days of stressing (i.e. 3 per row). This allowed the monitoring of anchor load with time to confirm that the anchors were holding the design loads.

Anchor Head Protection

During the construction of the building the final corrosion protection was applied to the anchors. This included secondary grouting and a grease filled cap. The secondary grouting was used to fill the void which is left behind the anchor plate during initial grouting of the anchors. The placement of a protective cap filled with grease protects the strands extending from the anchor head and allows for removal of the cap in the future for long-term lift-off testing over the life of the structure. A photograph of a capped anchor is shown in Figure 9.

CONCLUSIONS

The design of the permanent retaining wall was carried out using a probabilistic quantitative method using available jointing data within Ashfield Shale. Geotechnical supervision was maintained throughout the construction process with inspections on each individual anchor during the various stages of their installation. Corrosion protection measures were applied to the anchors to satisfy the 100 year design life of the anchors.

The experiences gained during the construction of this wall have increased the authors understanding of the factors involved in the design and construction of both temporary and permanent multi-anchored retaining walls.

ACKNOWLEDGEMENTS

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REFERENCES

- (1) 'Deep Excavations in Shale', P.V. Andrews & J.C. Braybrooke, AGS Mini-symposium "Excavation Retention" Sydney, 2001.
- (2) 'Review of Geological Data for Perimeter Shoring Design', Douglas Partners Unpublished Report No. 28994G, April 2002.

Rigorous Solutions of Classical Lateral Earth Pressures

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ABSTRACT: Rigorous plasticity solutions for lateral earth pressures are presented in this paper by applying advanced upper and lower bound methods. The techniques, which were developed at the University of Newcastle, use finite element formulations of the limit analysis theorems and have been successfully applied to a number of geotechnical stability analyses in the past few years. For the cases analysed in this paper, the numerical lower and upper bounds typically bracket the true limit loads to within a few percent, and are therefore useful in design practice. These accurate bounds can be computed efficiently with a standard personal computer.

INTRODUCTION

Limit analysis refers to the calculation of collapse loads and plays an important part in the practical design of many geotechnical structures. In recent years, there has been an increased geotechnical awareness of the potential benefits in using numerical limit analysis (Carter et al. 2000). Owing to the nature of many geotechnical problems, which typically involve complex geometry, inhomogeneous soil profiles and anisotropic strength characteristics, the task of estimating the ultimate load has never been an easy one. Because of its great technical importance, research on geotechnical stability analysis using advanced limit analysis techniques has long been one of the major research interests in Australia and many other countries. However, it was not until finite element formulations of the bounding theorems became available (Sloan 1988, 1989; Lyamin and Sloan 2002a,b), that the full power of limit analysis was harnessed and applications to a wide range of geotechnical structures began to grow. The development and application of numerical limit analysis methods is a major research theme of the geotechnical group at the University of Newcastle (Merifield et al. 1999; Shiau et al. 2003).

To date, limit equilibrium methods have often been used by practicing engineers to determine lateral earth pressures. Among these are the Rankine, Coulomb, and Log-spiral techniques. These limit equilibrium methods presuppose collapse is triggered by large relative movements along a predefined failure surface and that, simultaneously, the stresses on this surface reach limiting values which are governed by soil strength parameters. Once these two assumptions are made, the solution is found by invoking equilibrium and then optimising the geometry of the failure surface, either analytically or numerically, to give the lowest load. An inherent limitation of the limit equilibrium method is the need to define the general shape of the failure surface in advance, as this may affect the accuracy of the solution obtained. A typical example of this limitation is the assumption of plane failure surface in the calculation of passive resistance using Coulomb Theory. As shown by Chen (1975), it is not unusual for limit equilibrium solutions to be grossly unconservative in their estimates of the limit load, especially if the assumed form of the failure surface is physically inappropriate.

In this paper, the classical lateral earth pressure problems are investigated by using the modern limit analysis tools developed at Newcastle. Results are presented in the form of earth pressure coefficients with tables and charts provided for the purpose of design practice.

STATEMENT OF THE PROBLEM

The lateral earth pressure problem considered in this paper is illustrated in Figure 1. The back of the wall has an angle α to the horizontal and the surface of the backfill has a slope of β to the horizontal and a uniform surcharge q . In determining the passive earth resistance, a rigid retaining wall of height H is pushed horizontally into the soil. To induce active failure, the wall is allowed to move away from the backfill. The soil is taken to be c - ϕ Mohr-Coulomb material with a unit weight γ .

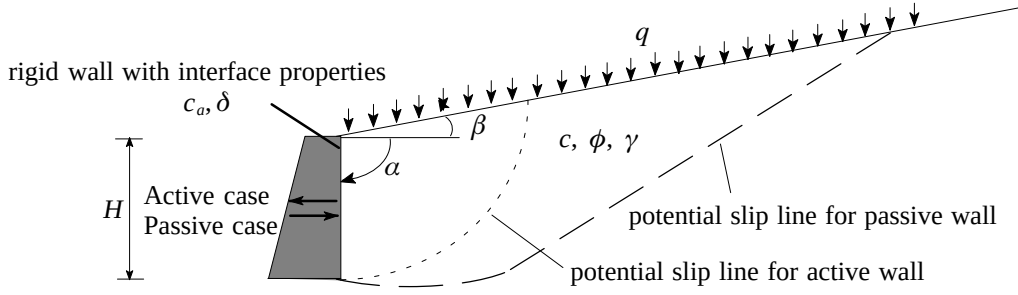


Figure 1. Problem notation and potential failure mechanisms

It is convenient to use soil-wall friction angle δ and adhesion c_a for representing the wall roughness. For a c - ϕ backfill, $\delta = 0$ and $c_a = 0$ indicates a perfectly smooth wall while a perfectly rough wall can be modelled by adopting $\delta = \phi$ and $c_a = c$. In a similar manner to the bearing capacity equation of Terzaghi, the total active and passive thrust acting on the wall, P_a and P_p , are defined in terms of earth pressure coefficients K_a , K_c , and K_q according to

$$P_a = \frac{1}{2}\gamma H^2 K_{ar} - c H K_{ac} + q H K_{aq} \quad (1)$$

$$P_p = \frac{1}{2}\gamma H^2 K_{pr} + c H K_{pc} + q H K_{pq} \quad (2)$$

Equations (1) and (2) are governed by the geometric parameters α and β , the soil-wall friction angle δ and adhesion c_a , the surcharge q , and the backfill frictional angle ϕ . Given the above parameters, the total active and passive thrusts are computed from the finite element upper and lower bound analyses. Accordingly, the coefficients of earth pressure can be determined from the equations and are presented in this paper.

FINITE ELEMENT LIMIT ANALYSIS

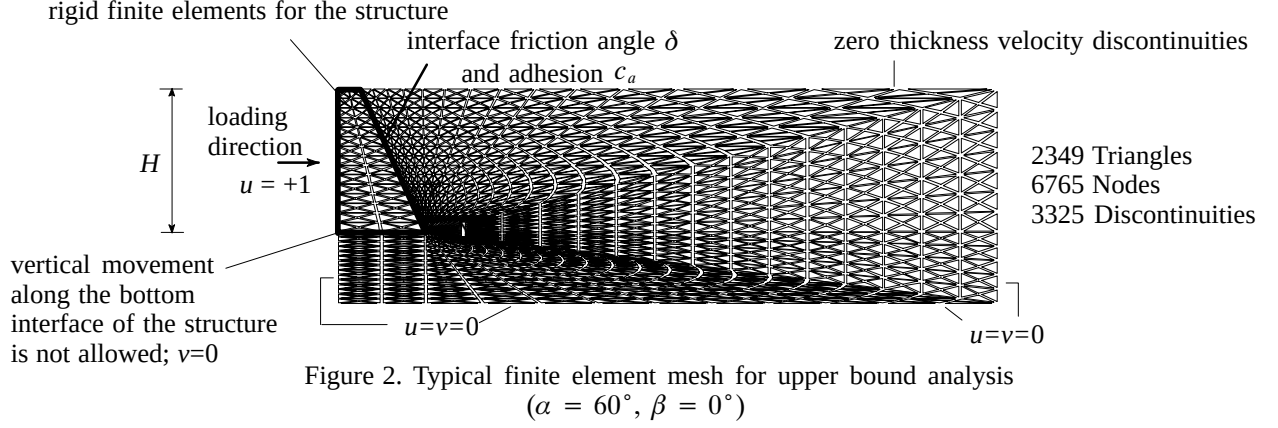
A very brief summary of the finite element formulations of the upper and lower bound theorems is introduced. Detailed descriptions of the techniques can be found in Sloan (1988, 1989) and Lyamin and Sloan (2002a, 2002b).

The lower bound limit theorem states that if any equilibrium state of stress can be found which balances the applied loads and satisfies the yield criterion as well as the stress boundary conditions, then the body will not collapse. Stress fields that satisfy these requirements, and thus give lower bounds, are said to be statically admissible. The key idea behind the lower bound analysis applied here is to model the stress field using finite elements and use the static admissibility constraints to express the unknown collapse load as a solution to a mathematical programming problem. For linear stress elements, the equilibrium and stress boundary conditions give rise to linear equality constraints on the nodal stresses, while the yield condition, which requires all stress points to lie inside or on the yield surface, gives rise to a nonlinear inequality constraint on each set of nodal stresses. The objective function, which is to be maximised, corresponds to the collapse load and is a function of the unknown stresses. For linear elements, this function is also linear. After all the element coefficients are assembled, the final optimisation problem is thus one with a linear objective function, linear equality constraints and nonlinear inequality constraints.

The upper bound theorem can be formulated in terms of finite elements by adopting a similar approach to that just described for the lower bound case. This theorem states that the power dissipated by any kinematically admissible velocity field can be equated to the power expended by the external loads to give a rigorous upper bound on the true limit load. A kinematically admissible velocity field is one which satisfies compatibility, the flow rule and the velocity boundary conditions. In a finite element formulation of the upper bound theorem, the velocity field is modelled using appropriate variables and the optimum (minimum) internal power dissipation is obtained as the solution to a mathematical programming problem.

Figure 2 shows a typical finite element mesh for upper bound limit analysis of the problem considered. This mesh comprises 6765 nodes, 2349 triangular elements, and 3325 velocity discontinuities. The bottom and right hand edges of the mesh are fixed since it is assumed that the failure mechanism is contained within the proposed grid. Note that this condition needs to be checked for each case and in some instances larger meshes are necessary to ensure that the optimal failure mechanism is captured correctly. To consider the effect of soil-wall interface, those nodes on the interface boundary are given a new material property which is different from the one adopted for the backfill soil. For cohesionless soil, a smooth wall is modelled by adopting a zero interface friction angle δ . Note that the maximum value of δ is equal

to the backfill soil friction angle ϕ and represents a perfectly rough wall. To induce passive failure, an upper bound solution is obtained by prescribing a unit horizontal translation ($u = +1$) into the soil. After the resulting optimization problem is solved for the imposed boundary conditions, the upper bound on the passive forces P_p is obtained by equating the power expended by the external loads to the power dissipated internally by plastic deformation. The relevant passive earth pressure coefficients K_{py} , K_{pc} and K_{pq} can then be found by direct substitution in Equations (1) and (2).



The lower bound mesh is not shown here and can be found in Shiau et al. (2004) and Lyamin et al. (2004).

RESULTS AND DISCUSSIONS

Typical Results

Numerical upper and lower bound analyses have been carried out for the case of $\phi = 35^\circ$ with a vertical wall and horizontal backfill. Results for the earth pressure coefficients (K_γ , K_c , and K_q) are presented for both the active and the passive cases in Table 1 and Table 2 respectively. The various factors, K_γ , K_c , and K_q are computed by putting $c = 0$ and $q = 0$, $\gamma = 0$ and $q = 0$, and $\gamma = 0$ and $c = 0$ in the calculations respectively.

ϕ	δ/ϕ and c_a/c	K_{ar}			K_{ac}			K_{aq}		
		UB*	LB*	AVG [#]	UB	LB	AVG	UB	LB	AVG
35	0	0.266	0.272	0.269	1.044	1.041	1.043	0.268	0.271	0.270
	1/3	0.245	0.268	0.256	1.118	1.099	1.108	0.248	0.262	0.255
	1/2	0.241	0.260	0.251	1.167	1.147	1.157	0.244	0.258	0.251
	2/3	0.242	0.257	0.249	1.226	1.203	1.215	0.243	0.257	0.250
	1	0.247	0.261	0.254	1.387	1.360	1.373	0.249	0.269	0.259

* LB and UB : Lower and upper bound results

[#] AVG: Averaged lower and upper bound results

Table 1. Active earth pressure coefficients from upper and lower bound analyses

Note that five sets of interface properties have been studied. For the active case, K_{ay} and K_{aq} decrease as the interface friction is increased, indicating that non-conservative design of an active wall can result if full friction on the soil-wall interface is assumed. It is interesting to see that the value of K_{ac} increases as the interface strengths are increased. However, this increase has an adverse effect on the total active thrust because of the negative sign in equation (1). For the passive case, K_{py} , K_{pc} , and K_{pq} increase as the interface strength parameters increase, indicating that the total passive resistance is increased by assuming a fully rough soil-wall interface.

ϕ	δ/ϕ and c_a/c	K_{pr}			K_{pc}			K_{pq}		
		UB*	LB*	AVG [#]	UB	LB	AVG	UB	LB	AVG
35	0	3.72	3.70	3.71	3.86	3.84	3.85	3.71	3.69	3.7
	1/3	5.58	5.00	5.29	6.45	5.75	6.10	5.50	5.00	5.25
	1/2	6.77	6.08	6.43	7.79	6.95	7.37	6.47	5.87	6.17
	2/3	8.17	7.32	7.74	9.12	8.29	8.70	7.45	6.85	7.15
	1	11.50	10.99	11.24	11.84	10.82	11.33	9.52	8.83	9.17

* LB and UB : Lower and upper bound results

[#] AVG: Averaged lower and upper bound results

Table 2. Passive earth pressure coefficients from upper and lower bound analyses

Figure 3 shows the velocity diagrams from the upper bound calculations for various values of δ/ϕ' . The plots clearly demonstrate the improved passive resistance that results from increasing the soil-wall friction and show the potential errors involved in Coulomb Theory with the assumption of plane failure surface. Interestingly, a typical Rankine solution is obtained for the smooth case ($\delta/\phi = 0$) with a planar failure surface oriented at an angle of $45^\circ - \phi/2$ to the horizontal backfill.

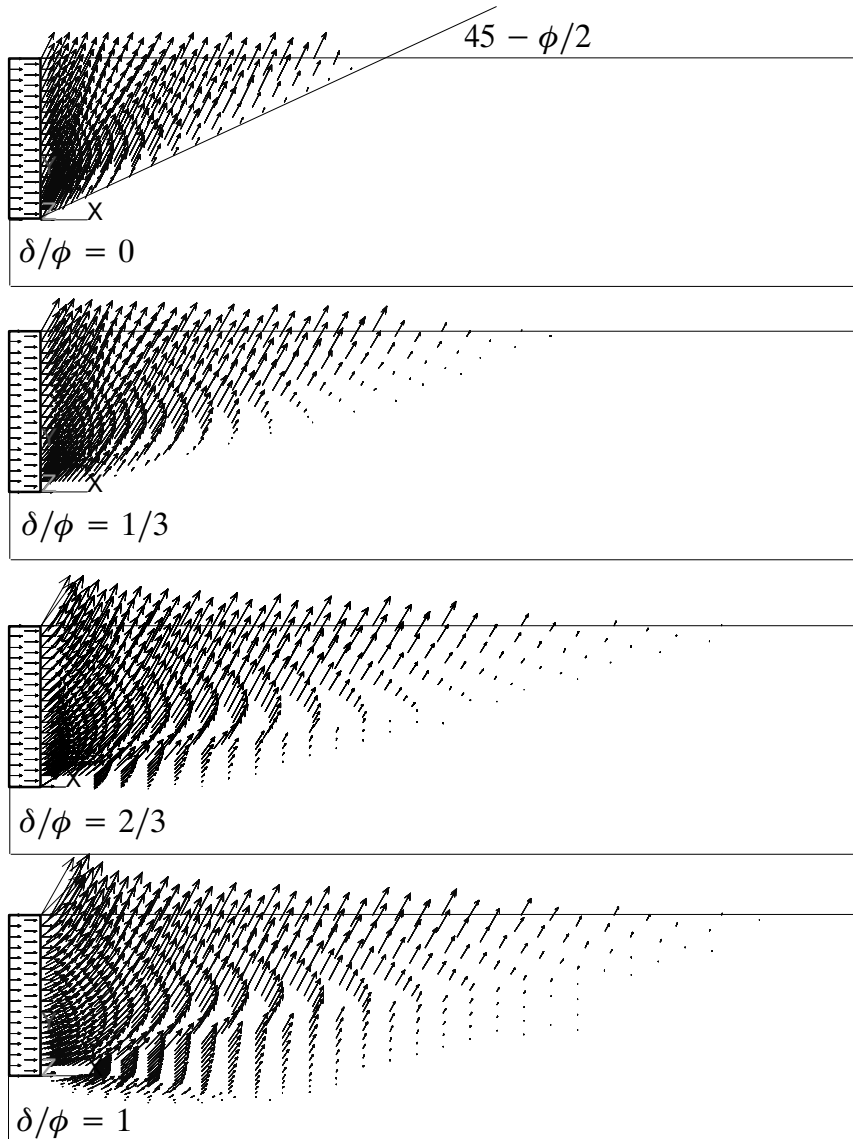


Figure 3. Velocity diagrams for various values of δ/ϕ
($\alpha = 90^\circ$, $\beta = 0^\circ$, $\phi = 35^\circ$, $c = 0$)

Note that the results presented here are for heavy walls as no vertical movement is assumed. If the weight of the wall is considered in the computation, the failure mechanism can be quite different (Shiau et al. 2004). A wide range of analyses have also been performed for passive cases with cohesionless backfill and rigorous bounding results are presented for various values of δ/ϕ in Figure 4. It is expected that any available method, based on the assumption of associated flow, should yield results which are located between our rigorous upper and lower bound solutions.

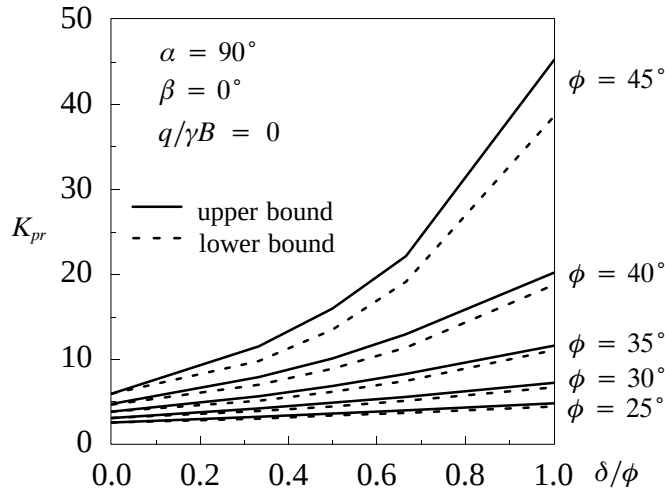


Figure 4. Typical upper and lower bound results for cohesionless backfill ($\alpha = 90^\circ$, $\beta = 0^\circ$, $c = 0^\circ$)

A Layered Example

A layered backfill has been analysed using finite element limit analysis techniques. As shown in Figure 5, The height of the wall is 6 m and the backfill consists of a drained sand overlying an undrained clay. Both perfectly smooth and perfectly rough walls are considered. For the smooth wall, the computation predicts an upper bound active thrust $(P_a)^{UB} = 166.35$ kN and a lower bound $(P_a)^{LB} = 173.21$ kN. Interestingly, hand calculation from Rankine Theory gives a value of $(P_a)^{Rankine} = 170.01$ kN.

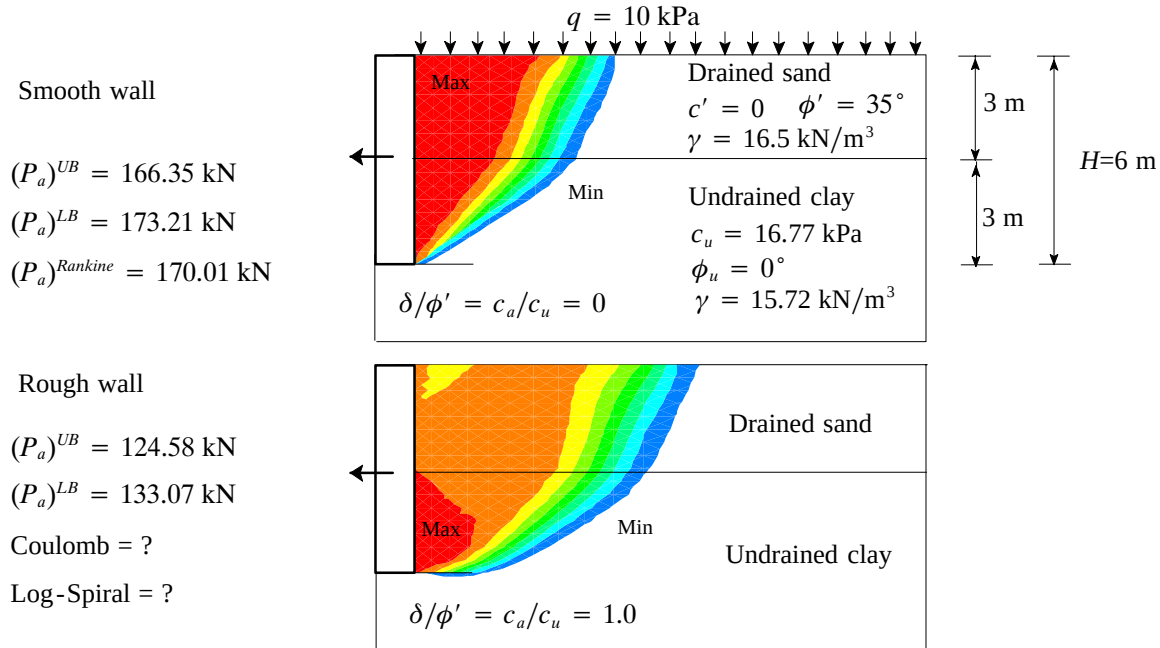


Figure 5. Velocity diagrams for active wall with smooth and rough interfaces

For the rough wall case, the upper and lower bound active thrusts were found to be $(P_a)^{UB} = 124.58$ kN and $(P_a)^{LB} = 133.07$ kN. Owing to the effect of interface friction and adhesion, the active failure mechanism for the rough case is larger than that for the smooth case, resulted in a reduction of active thrust by a factor of 1.3. On the other hand, as shown in Figure 6, the results indicate an increase of passive resistance by a factor of approximately 1.15.

Interestingly, the boundary of the failure zone is non-planar for the active case (Figure 5), and has a complicated shape for the passive case (Figure 6). A question has thus been raised as to what is the potential error involved in using the

Coulomb method (which assumes a planar slip surface) or the log-spiral method for estimating the passive resistance of such a layered backfill. The importance of the bounding techniques can thus be easily understood from this demonstration.

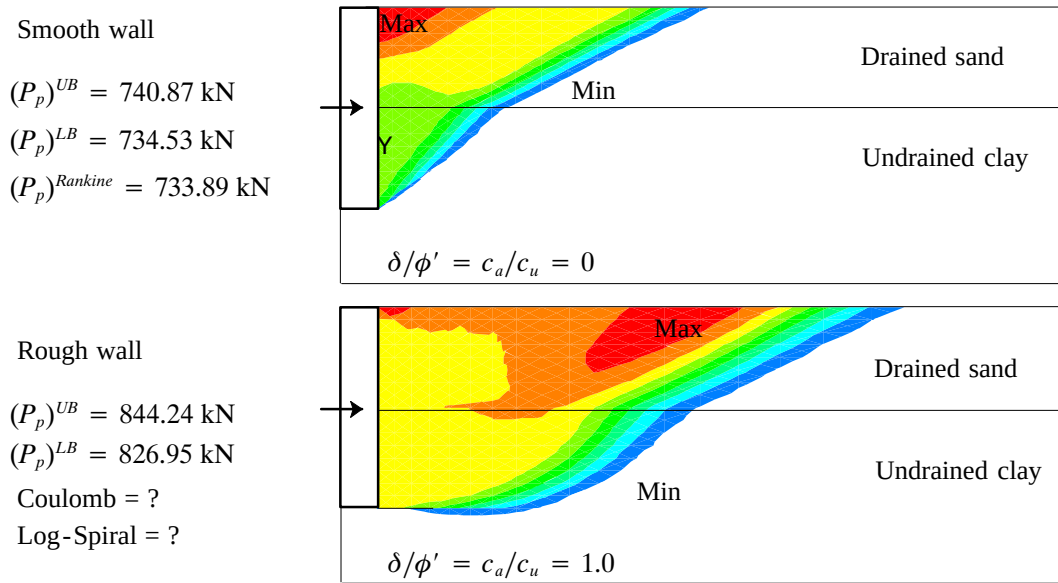


Figure 6. Velocity diagrams for passive wall with smooth and rough interfaces

CONCLUSION

The upper and lower bound solutions obtained for the active and the passive earth pressures bracket the true solution to within 10% or better. Results have been presented in the form of earth pressure coefficients with tables and charts provided for the purpose of design. It is expected that any available method, based on the assumption of associated flow, should yield results which are located between our rigorous upper and lower bound solutions. An interesting investigation regarding the velocity jumps across layered media can be made in the future.

REFERENCES

- Chen, W. F. (1975). *Limit analysis and soil plasticity*, Elsevier, Amsterdam.
- Carter, J. P., Desai C. S., Potts D.M., Schweiger, H.F. and Sloan, S.W. (2000). Computing and computer modelling in geotechnical engineering, Invited Paper, *GeoEng2000*, Melbourne, Australia, CD format.
- Duncan, J. M and Mokwa, R. (2001). "Passive earth pressures: theories and tests." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 127(3), 248-257.
- Lyamin, A. V. and Sloan, S. W. (2002a). "Lower bound limit analysis using nonlinear programming." *International Journal for Numerical Methods in Engineering*, 55, 573-611.
- Lyamin, A. V. and Sloan, S. W. (2002b). "Upper bound limit analysis using linear finite elements and nonlinear programming." *International Journal for Numerical and Analytical Methods in Geomechanics*, 26, 181-216.
- Lyamin, A. V., Shiau, J. S. and Sloan, S. W. (2004). "Calculation of active and passive earth pressure in cohesive-frictional backfills." (In preparation)
- Merifield, R. S., Sloan, S. W. and Yu, H. S. (1999). "Rigorous solutions for the bearing capacity of two layered clay soils." *Géotechnique*, 49(4), 471-490.
- Shiau, J. S., Lyamin, A. V. and Sloan, S. W. (2003). "Bearing capacity of a sand layer on clay by finite element limit analysis." *Canadian Geotechnical Journal*, 40, 900-915.
- Shiau, J. S., Lyamin, A. V. and Sloan, S. W. (2004). "Passive earth pressure in cohesionless backfill." (To submit).
- Sloan, S. W. (1988). "Lower bound limit analysis using finite elements and linear programming." *International Journal for Numerical and Analytical Methods in Geomechanics*, 12, 61-67.
- Sloan, S. W. (1989). "Upper bound limit analysis using finite elements and linear programming." *International Journal for Numerical and Analytical Methods in Geomechanics*, 13, 263-282.
- Terzaghi, K., Peck, R. B., and Mezri, G. (1996). *Soil mechanics in engineering practice*, 3rd Ed., John Wiley and Sons, Inc., New York.

Case Study: Seismic Stability of Underwater Retaining Walls

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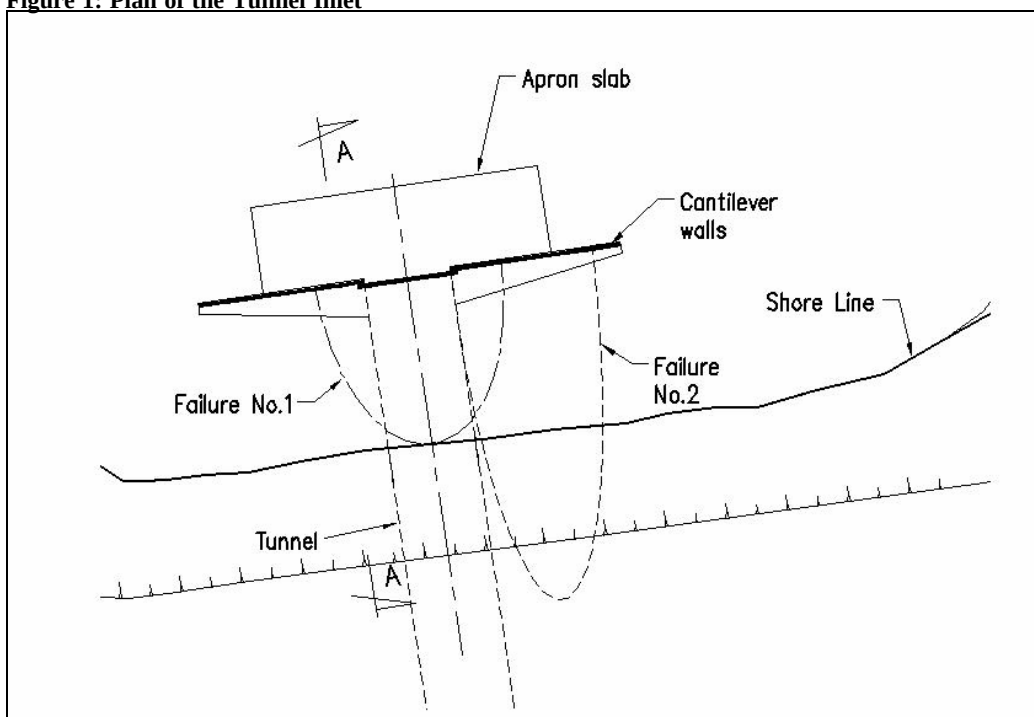
The upstream end of a reservoir's outlet structure is retained by 9 to 12m high retaining walls. The effect on retaining wall stability of a 3 to 4m thick layer of gravels deposited upslope of the walls, particularly retaining wall stability under seismic loading, has been assessed.

This paper details the method of stability analysis, modelling three-dimensional effects of the reservoir shoreline slope and wall geometries, modelling wall reactions, soil and hydrodynamic loads, and determination of seismic displacements. Different thicknesses of deposited surcharge material were modelled, as were different reservoir levels. Both drained and undrained conditions on the failure surface under earthquake loading were considered. Results are presented for stability, sensitivity, wall capacity and shoreline slope displacement.

1.0 Background

The upstream end of a reservoir's outlet structure is retained by 9 to 12m high retaining walls. Figure 1 below shows a plan of the outlet structure and location of the retaining walls.

Figure 1: Plan of the Tunnel Inlet



A review report recommended an updated assessment of retaining wall stability for the latest seismic hazard information and wall surcharge loadings. This assessment comprised assessing wall displacements required for first cracking, first yield and ultimate capacities and comparing wall displacements to slope displacements, which were assessed from three earthquake records using the Newmark Sliding Block Method. Further assessments were recommended comprising a more detailed assessment of the structural performance of the walls, and consideration of the three dimensional effects of the vent shaft, and apron on the shoreline slope failure geometry. The potential for tunnel blockage was also to be assessed.

This paper details the method of stability analysis, modelling three dimensional effects of the slope and wall geometries, modelling wall reactions, soil and hydrodynamic loads, and determination of seismic displacements.

Seismic performance of the walls was initially assessed by a force based approach, which computed earth pressures on the back of the walls for various failure geometries and compared earth pressures with wall structural capacities. Seismic earth pressures were determined using the Monobe Okabe equation. However this equation was not appropriate for the slope geometry and higher seismic coefficients, and a graphical approach (Culmann's method) was then used to calculate earth pressures on the back of the wall. The force based approach indicated that wall capacities were exceeded for relatively low return period earthquakes (about 150 years).

A displacement based approach was then considered as this approach recognises that:

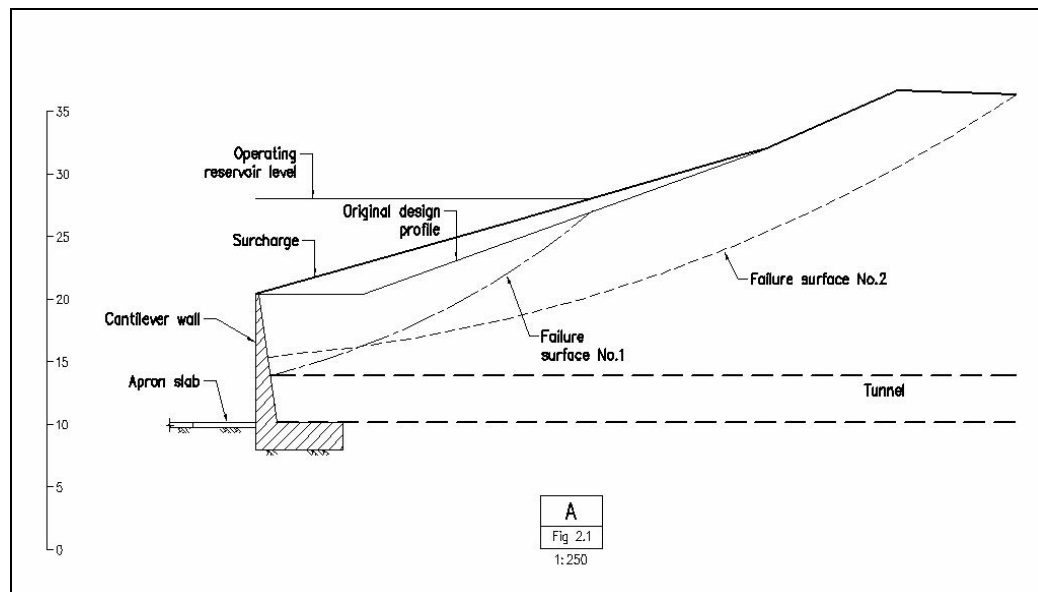
- the mobilised wall resistance is dependant on wall displacements
- the shoreline slope will displace a finite amount for the earthquake considered and not necessarily experience catastrophic failure.

Slope displacements are then compared with wall capacity displacements to determine the condition of the wall (i.e. first cracking, yield or ultimate capacity). This approach assumes that shoreline slope displacement will cause a similar displacement in the walls.

2.0 Wall, Shoreline Slope and Failure Surface Geometries

As-built drawings were used to define wall and slope geometries. Figure 2 shows a section (A-A) through the cantilever wall and slope.

Figure 2: Section A-A Through Cantilever Wall and Slope



The cantilever walls are restrained above their base by the apron slab, wall foundation embedment and slope. Wall displacement by rotation and sliding at foundation level is therefore prevented and the wall section above the level of the apron slab contact resists static and seismic loads. The counterfort wall and the upstream end of the tunnel are monolithic. The apron slab and shear key embedment at the upstream end of the tunnel prevent sliding of the tunnel and wall. Wall displacement by rotation and sliding at tunnel foundation level is therefore prevented and the counterfort section at the top of the tunnel and panels between counterforts resist static and seismic loads.

Two failure surfaces in the shoreline slope were used to partially model the three-dimensional effects of the slope and wall geometries and the vent shaft on shoreline slope failure. Figures 1 and 2 show the

location of the assumed failure surfaces. Failure No.1 was located behind the wall and constrained to mid slope with the maximum depth limited by the top of the tunnel. Failure No.2 (critical surface) was located behind the walls between the tunnel and abutments and extended to the top of the slope. The maximum depth of failure No.2 at the cantilever walls was limited by the sloping apron slab and represents a reasonable maximum depth for a three-dimensional failure surface behind the cantilever walls.

The computer program Slope/W was used to determine failure surface geometries given the above boundary constraints. Slope/W models the failure surface in a two-dimensional plane and published research suggests that factors of safety computed for a three-dimensional surface are always higher due to the restraining effects of the shear forces on the sides and ends of the sliding mass. Hence the factors of safety determined for failure surfaces No.1 and 2 are conservative.

3.0 Wall Structural Assessment

For the cantilever walls the critical section was defined as the part of the wall where fixity is assumed and the wall section can develop its ultimate strength. An equivalent wall reaction and location could then be determined. Two possible application points for the wall reactions were proposed. These were at the point where the average wall capacity was assessed which reflected the wall reaction pressure distribution at the wall base, and at 2/3 the depth of the failure surface from the top of the wall, which reflected the pressure distribution at the base of the failure surface.

Wall capacity displacements were then determined corresponding to first cracking, first yield and ultimate capacity (reinforcing steel tensile failure). A permissible wall crack width was used to define wall serviceability. Displacements midway between top of wall and the failure surface were used for comparing slope displacement and hence wall performance.

4.0 Upstream Shoulder Displacements

Seismic induced displacements of the slope were calculated using the Newmark method of double integration. This involved determining minimum factors of safety (static) and critical accelerations (earthquake pseudo static) for the two failure surfaces using Slope/W software and using the critical accelerations with earthquake time histories to determine slope displacements.

The Slope/W wall reactions and Newmark displacement analysis were iterated for each of the 150, 1,000 and 10,000 year earthquake records until the shoulder (seismic induced) displacement matched the wall displacement (established from structural assessment) for the Slope/W wall reaction used.

Four earthquake acceleration time history records from overseas were used since suitable local records are not available. These were:

- Valparaiso, Chile (Richter magnitude 7.4)
- Vile, Mexico (Richter magnitude 8.1)
- Landers, USA (Richter magnitude 7.0)
- Tabas, Iran (Richter magnitude 7.0)

The Chile, Mexico and Iran records were scaled to give peak ground accelerations for the site spectra 150, 1,000 and 10,000 year return period events, i.e. 0.2g, 0.39g and 0.75g respectively. The USA record represents the expected record from an earthquake on a nearby fault which contributes a significant portion of the total seismic risk at the tunnel location.

The displacement calculation included reversal of earthquakes, and upslope movement was not permitted.

5.0 Slope Stability Assessment

5.1 Load Cases

For each of the two slope failure surfaces, three slope geometries were considered to account for the varying levels of deposited surcharge material. These were the “No Surcharge” construction as-built case, the “Surcharge” case, and the “Intermediate Surcharge” case.

Three reservoir levels were also considered for the slope stability model. These were, low reservoir level, operating reservoir level, and high reservoir level. For each reservoir level, seismic hydrodynamic effects on the retaining walls and the slope were considered.

Density and strength parameters of the slope and surcharge material were obtained from the design records. For the slope stability model, three values for the effective friction angle for the slope material were considered. Slope and surcharge material parameters are presented in Table 5.1 below.

Table 5.1 Material strength parameters considered for stability model

Slope material			Surcharge material		
Density (saturated) (kN/m ³)	Effective cohesion (kPa)	Effective friction angle (degrees)	Density (saturated) (kN/m ³)	Effective cohesion (kPa)	Effective friction angle (degrees)
23	0	36, 38 and 40	18	0	28

5.2 Wall Reactions

The wall reactions for the cantilever and counterfort walls were modelled as horizontal line loads applied just inside the vertical face (i.e. inside the sliding mass) at the toe of the slope.

5.3 Modelling of Reservoir

When modelling a structure such as an underwater slope, problems can arise in stability calculations when the failure surface is partially submerged or in the case here for failure surface 1, completely submerged. For the case with failure surface 1 where the failure mass is completely submerged, the reservoir was modelled as a no strength layer and a piezometric line along to the reservoir surface was applied. A second analysis was then run which did not include the reservoir as a layer or include a piezometric surface, but modelled the slope and surcharge material unit weights as a submerged unit weights. Very little discrepancy (0.2% difference) in factor of safety values between the two methods was found and the no strength soil layer method was adopted for the remaining analyses.

5.4 Drained and Undrained Parameters

Both drained and undrained conditions on the failure surface were considered. Slope/W allows for the modelling of undrained conditions with drained input parameters by using the failure surface shear resistance for the critical static drained failure surface and converting this to an equivalent $\phi = 0^\circ$ shear resistance for the earthquake case.

5.5 Hydrodynamic Forces

Seismic hydrodynamic effects on walls and the slope were considered for each reservoir level. The hydrodynamic loads were applied as horizontal line loads applied at 2/3 wall height from the top of the walls and 2/3 the slope height below reservoir level. These hydrodynamic loads act in the opposite direction to wall reactions.

5.6 Sensitivity Analysis

Before selecting the critical load case for analysis a number of sensitivity analyses were carried out including varying the wall reaction application point, varying soil effective strength parameters, using drained and undrained parameters, and varying application point of hydrodynamic loads.

The sensitivity of Slope/W factors of safety to the level of application of the wall reaction line load was assessed. From the structural assessment of the walls two wall reaction locations were proposed. It was found that factors of safety were not significantly sensitive to the location of application of the line load and an average level of application was adopted.

Three effective stress friction angles for the slope material were considered ($\phi'=36, 38$ and 40°) and the sensitivity of these to displacements was assessed. It was found that displacements for the $\phi'=38^\circ$ and 40° were 20% and 40% less than the corresponding 36° displacement. The typical level of confining stress (i.e. vertical effective stress at the failure surface for operational reservoir level and surcharge) is 140 kPa and documented research on triaxial tests on similar size range materials showed that a friction angle of 40° or more could be expected at confining pressures less than 140kPa. Hence an effective stress friction angle of 40° was adopted.

Displacements for drained and undrained failure surface conditions were assessed and displacements for the drained condition were found to be 60% to 90% greater than for the undrained case. It was felt that using drained conditions might not be appropriate due to the low-permeability material that the failure surface predominantly passes through, and the earthquake loading being rapid. Undrained conditions were hence adopted for analysis.

The sensitivity of the analysis to hydrodynamic loads was assessed. It was found that for hydrodynamic loads on walls and hydrodynamic loads on walls plus the slope component, factors of safety were not significantly sensitive. It was also felt that applying hydrodynamic loads to such a small area of reservoir was somewhat conservative and hence the “No hydrodynamic” case was adopted.

5.7 Critical Load Case for Analysis

The load case selected for slope displacement assessment for failures 1 and 2 was intermediate surcharge, operating reservoir level, no hydrodynamic load, and 150, 1,000 and 10,000 year events for all 4 seismic records. The intermediate surcharge geometry was selected as it represented a profile that could be readily maintained by dredging. Operating reservoir level was chosen as an “average” condition between high and low reservoir levels.

The two critical cases were then modelled with varying wall reactions and corresponding critical acceleration coefficients determined. The wall reactions and critical accelerations were used in the Newmark displacement analysis and iterated for each of the 150, 1,000 and 10,000 year earthquake records.

6.0 Wall Capacity and Slope Displacement Results

The following table compares wall capacity displacements with slope displacements for the walls.

Table 6.0: Wall capacity and slope displacements

Wall type	Wall capacity displacements (mm)				Slope displacements (mm)		
	First cracking	Yield	Ultimate	Serviceability	150 year event	1,000 year event	10,000 year event
Cantilever walls	3 to 4	15 to 26	70 to 195	15 to 28	0	0	7 to 47 Average 28

7.0 Conclusions

After reviewing the wall capacity displacements, slope displacements, and volume of displaced surcharge deposits, the following was concluded:

- Performance of the cantilever walls under the 150 and 1,000 year seismic events is satisfactory
- Displacements for cantilever walls may exceed the serviceability displacement (0.4mm crack width) under the 10,000 year event but will remain below ultimate (collapse) displacements
- The calculated slope displacements will not result in collapse of the cantilever walls nor will they contribute to a slope failure.
- Tunnel inlet blockage by seismic induced displacement of surcharge material is unlikely.

Cover Trials on Tailings Storage Facilities in Arid Hypersaline Regions

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Abstract A field study was established in 2001 to evaluate a range of cover designs for the rehabilitation of tailings storage facilities in arid hypersaline regions. The success of a cover system in this environment is dependent on the extent to which the cover limits the upward evaporation-driven migration of salts from the tailings into the cover. A “discontinuous cover” (that is, cover mounds) is one particular system that was trialled. Aside from being a cost-effective alternative to placing a conventional continuous cover, the concept of a discontinuous cover system was built on the idea that water rising to the surface due to capillary action would not rise further than the low points between the cover mounds, as the cover materials are far coarser than the tailings. The sides and tops of the mounds would then in theory be suitable for the growth of vegetation, while the low points would become salinas where vegetation could not grow, mimicking a natural salt pan.

The current research has demonstrated that the development of a salt crust on the bare tailings surface has reduced the rate of evaporation from the bare tailings, to the point where there is evidence of lateral migration of salts from beneath the bare tailings surface into the adjacent cover mounds. This has resulted in higher salt concentrations within the cover mounds than within a continuous cover of corresponding materials and cross-section. This outcome questions the potential effectiveness of a discontinuous cover system for hypersaline areas where revegetation is the aim. Further trials are recommended to further understand the mechanisms limiting the success of the current discontinuous cover system design. A key element in developing a vegetated cover on hypersaline tailings in an arid climate is an effective and sustainable capillary break between the tailings and the growth medium.

INTRODUCTION

The Kalgoorlie mining region of Western Australia is underlain by hypersaline groundwaters, which due to the shortage of good quality water in the region are used as process water. As a result, the tailings formed on processing the ore have highly saline pore water. Highly saline tailings are analogous to the natural salt pans that exist in the region.

The high salt content of the tailings has a number of effects. The formation of a salt crust on the surface of the tailings on evaporative desiccation tends to close off further desiccation, while at the same time limiting dusting. However the salt crust also makes revegetation problematic.

Where tailings contain highly saline pore water, a continuous cover will require a capillary break to prevent saline water rising to the vegetation root zone within the cover. An alternative notion, which is being trialled as part of this study, was to construct discrete cover mounds to act as island sanctuaries for vegetation. In between the mounds the tailings are left bare. The mounds are limited in size, which was thought to limit the extent to which the highly saline tailings pore water would be drawn up into the cover. Infiltrating rainfall would then form a fresh water source above the saline pore water to supply any vegetation on the mound. The performance of the discontinuous cover system is discussed herein,

and compared with that of a conventional continuous cover system.

DETAILS OF TAILINGS COVER TRIALS

Study Objectives

The key project objectives pertinent to the cover trials at WMC Resources Mount Keith Nickel Operation (MKO) are as follows, with the third-listed objective being the key focus of this paper.

- To assess the rate and extent to which the high salinity of the tailings is migrating into the cover material, driven by soil suction effects, involving both solute (due to salinity) and matric (or capillary) suctions.
- To gauge and compare the performance of the different cover materials used in the cover trials; viz.: course quarry rock versus hard pit rock versus caprock versus oxide waste rock.
- To assess whether a “discontinuous cover system” presents a viable, cost-effective alternative cover solution.
- To compare and contrast the characteristics of the tailings cover trials and the state of a natural salt pan (Lake Miranda), located south of the mine.

Cover Trial Design and Philosophy

Figure 1 presents the general layout of the cover trials at MKO. On the western side of the central decant accessway, is a 60 m x 60 m continuous cover. This main pad is split up into four 30 m long square segments. This was done so that four different cover material types could be trialled simultaneously.

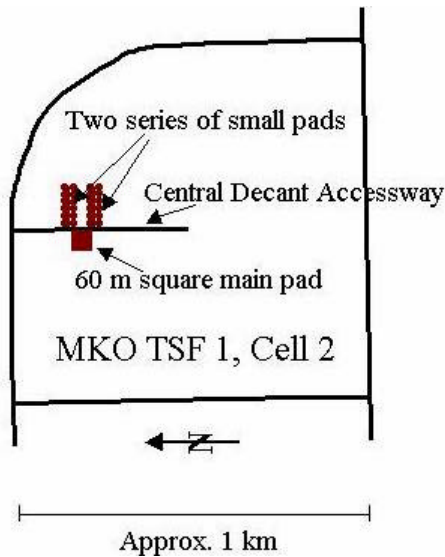


Figure 1: Cover trial layout at MKO TSF 1, Cell 2

The cover/tailings cross-sectional profile is shown in Figure 2. For a cover in an arid hypersaline region to sustain vegetation, a “capillary break” is required to prevent the upward migration of salts into the growth medium (topsoil), due to the excessive evaporative conditions. Hence it was deemed necessary to trial and compare a range of materials (typical of what would be available at most mine sites) to serve as a “capillary break.” The different materials being trialled comprise the following:

1. Coarse quarry rock
2. Hard pit rock
3. Caprock
4. Oxide waste rock

The nominal thickness of the capillary break layer for the current trials is greater than the theoretical height to which water could rise to under the effects of capillary action, for all of the materials trialled; this height being governed by the materials particle size distribution. It is important that the capillary break is much thicker than it theoretically needs to be, to allow for the infiltration of fines from the overlying topsoil/growth medium, which is inevitable without having some sort of filtering mechanism in place, such as a graded soil filter, or a geomembrane (which is generally prohibitively

expensive due to the large area involved). In addition to the above, the capillary break layer should be thicker than it theoretically needs to be to allow for the effects of root penetration by vegetation.

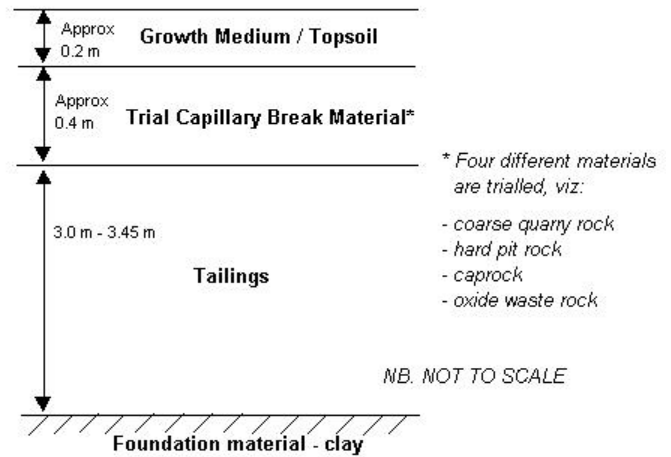


Figure 2: General cross-sectional profile of trial covers overlying tailings

The concept of a discontinuous cover system

As an alternative to placing a conventional continuous cover (with a necessary capillary break) on top of the TSF, the trials also involved a series of smaller pads, or “mounds” (a “discontinuous cover system”) on the eastern side of the central decant accessway (refer to Figure 1). The mounds are approximately 10 m in diameter and 10 m apart.

The original notion was that by not having a continuous cover, perhaps the need for a capillary break (to limit the uptake of salinity into the cover) could be avoided; however the mounds were still constructed with a capillary layer (as per the main pad), with a plan to gauge relative performance. In addition to this it was also expected that the “micro-climates” created by the mounds would facilitate revegetation. The mounds also had a range of slightly varied shapes to assess what works best from a wind erosion standpoint with regard to seed germination.

Sampling and Testing Methods

The cover trial testing involved hand augering through the full depth of tailings or natural soils (in the case of the natural salt pan analogue) to obtain closely spaced samples, and the sampling of the cover materials from the sides of test pits. The test holes were located to be representative of conditions around and under the trial covers. All test holes were taken through the tailings to

the underlying foundation at a depth of 3.2 to 3.8 m below the tailings surface.

The gravimetric moisture content (ratio of the mass of water to the mass of dry solids, expressed as a percentage) of the recovered samples was determined in the mine site laboratory, and selected samples were subjected to total suction (using a portable psychrometer), pH, Electrical Conductivity (EC) and salinity (Total Dissolved Solids, TDS) testing. The pH, EC and TDS determinations were carried out on a paste formed by adding distilled water to the moist sample in a nominal water (distilled water plus pore water) to dry solids mass ratio of about 5 to 1, as recommended by Hunt and Gilkes (1992). The distilled water had a neutral pH and negligible salinity. As a result, the pH of the samples was not affected by the addition of distilled water. The EC and salinity of the samples were determined on a solids dry mass basis. The amount of distilled water added was sufficient to ensure that all soluble salts on the tailings particles were dissolved, yielding the total dissolved solids (Hunt and Gilkes, 1992).

The EC values in $\mu\text{S}/\text{cm}$ were approximately twice the numerical TDS values in ppm. The EC may then be correlated with osmotic suction, which when subtracted from the total suction values gives the matric suction. Only the osmotic suction values are plotted herein, as a relative representation of salinity within the different cover profiles.

RESULTS AND INTERPRETATION

The test results to date generally present little variation in engineering performance for the different cover materials trialled. While from a capillary break standpoint the hard pit rock has performed better than the other materials trialled, it hasn't been completely successful as a capillary break, with salt still breaching this layer and migrating into the overlying growth medium. However, when the performance of the continuous cover is compared with the discontinuous cover for the same material type, a much more interesting disparity emerges, as discussed in the following subsections.

Salt Crust Development

The osmotic suction depth profiles for the caprock continuous cover system, as a function of time, are shown in Figure 3. At depth, there is little shift in profile. However, within the cover the osmotic suction varies largely between the test times. November 2002 presented significantly higher osmotic suction values at the surface, when compared with May 2002, which is likely to be attributed to the particularly dry period leading up to this investigation. A salt crust was noticed during the course of the November 2002 visit, and the

increase in osmotic suction for the surficial tailings from approximately 3,500 kPa in May 2002, to nearly 9,000 kPa in November 2002, confirms this crust development. The salt crust has continued to amass salt, climbing to an osmotic suction of 9,600 kPa in October 2003.

Wilson, Fredlund *et al.* (1997) discussed the significant effect that soil suction has on evaporative fluxes from soil surfaces, observing that the rate of actual soil evaporation declined when the total suction in the soil surfaces that were tested exceeded approximately 3,000 kPa (at Mount Keith, the osmotic suctions alone far exceed this value). Unique relationships for the different soil types were developed from their research, but they had very similar trends. It is this phenomenon that is also coming into play in the performance of the discontinuous cover system (or cover mounds), as discussed below (Wilson, Fredlund *et al.*, 1997).

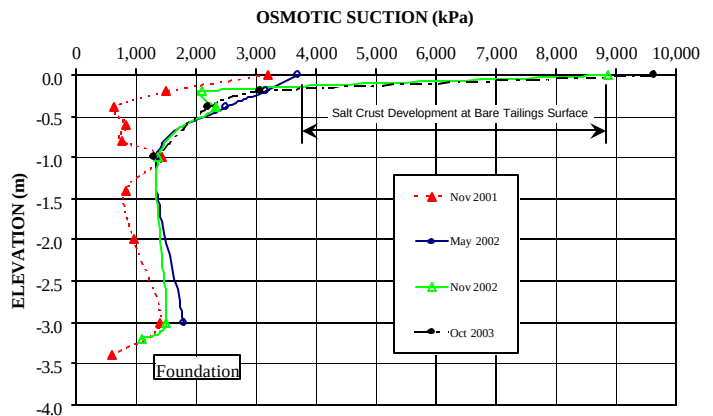


Figure 3: Osmotic suction depth profiles (main pad/continuous cover, caprock)

Cover System Comparisons

The caprock and hard-pit-rock cover types were both studied during the October 2003 investigation in order to compare the performance of the main pad (continuous cover) with that of the mounds (discontinuous cover). The covers were compared by way of moisture content, matric suction and osmotic suction. The moisture content and matric suction plots showed very little variation between the main pad and the mound for both cover types (Figure 4 presents the matric suction profiles for the caprock cover).

The osmotic suction plots tell a different story. The cover mound for both material types presented a much higher value of osmotic suction at the cover surface, than the main pad. Figure 5 presents the osmotic suction depth profiles for the caprock material. The hard rock cover presented similar trends to the caprock cover, for moisture content, matric suction and osmotic suction.

These results indicate that the salt is well and truly migrating to the surface of the mounds, but more importantly salt is migrating at a faster rate through the mounds than through the main pad.

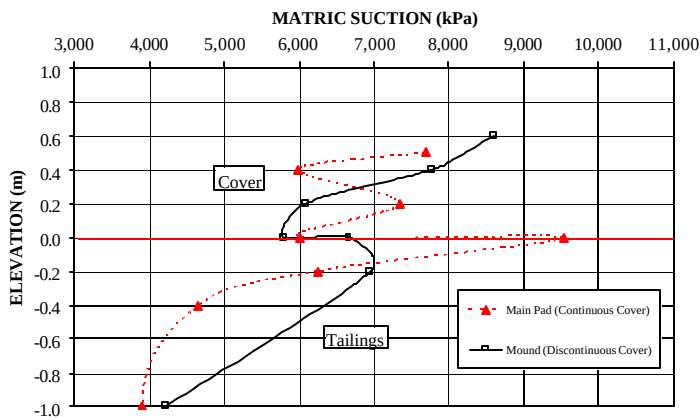


Figure 4: Matric suction depth profiles for the continuous and discontinuous cover systems (caprock) – October 2003

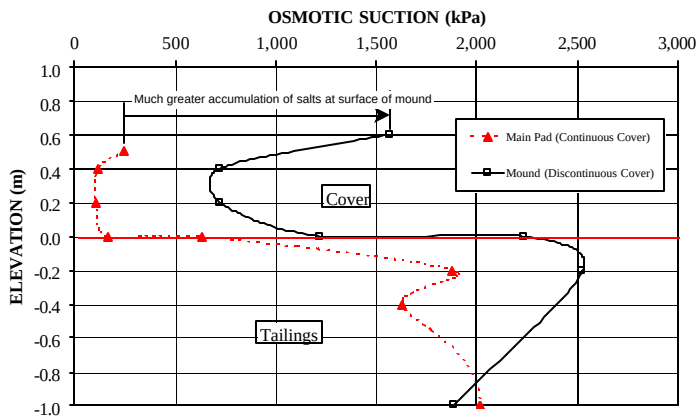


Figure 5: Osmotic suction depth profiles for the continuous and discontinuous cover systems (caprock) – October 2003

This behaviour goes against what was originally expected to happen, and is a very important outcome. There is a number of processes that could be driving this behaviour:

- The development of a salt crust has greatly reduced the actual rate of evaporation from the tailings surface (as suggested by Wilson, Fredlund *et al.*, 1997), to the extent that it has led to evaporation occurring preferentially through the cover system, as opposed to from the bare tailings.
- The fines from the overlying topsoil layer have tended to erode their way downslope over time resulting in a thin layer of finer material forming

around the full perimeter of the capillary break layer (refer to Figure 6). This renders the capillary break useless, since the sides of the mound provide an alternate access route for the saline pore water.

- The tailings underlying the salt crust, as well as the cover materials nearer the tailings interface, are much wetter than the cover materials near and at the top of the mounds. Earlier research by Thorpe and Barbour (1991) found that for a given water flux, the velocity of advective sodium (Na) transport increases as the water content decreases. The low water content in the topsoil results in the rapid movement of Na to the evaporative front where it is concentrated as a result of evaporation. Secondly, the lower water content of the surface topsoil allows more rapid increases in Na concentration as a result of diffusive transport or as a result of evaporative water losses (Thorpe and Barbour, 1991).
- A combination of any or all of the above.

Extensive numerical modelling is currently in progress, in an attempt to reproduce the behaviour observed in the field, and to subsequently provide further insight into the complex mechanisms at play.

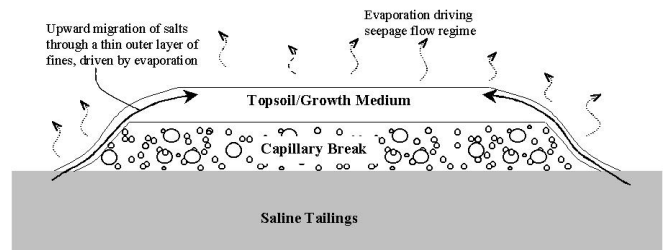


Figure 6: One likely mechanism responsible for higher salt levels in the cover mounds (cf. the main pad)

CONCLUSIONS

The results of the field investigations have questioned the potential effectiveness of a discontinuous cover system for hypersaline areas where revegetation is the aim. However, a discontinuous cover system may still be an option under the following circumstances:

- In areas where a salt crust is unlikely to form due to more manageable levels of salinity, or where salt leaching occurs either naturally (that is, by means of regular rainfall), or by manual means (which implies post-mining maintenance, which may be impractical).
- The situation depicted by Figure 6 is somehow avoided in the design and construction of the discontinuous cover system by incorporating an effective capillary break into the design.

The costs associated with placing a continuous “advanced” (that is, comprising a properly engineered capillary break, or multiple layers) cover system are very high. A discontinuous cover system, which is somewhat akin to an elevated salt pan, with some topographic relief, presents a potential cost-effective alternative. Accordingly, it is recommended that further field trials and numerical modelling be conducted to test the above conditions and move a step closer towards finding a viable and sustainable rehabilitation solution for TSFs in arid hypersaline regions.

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REFERENCES

Hunt, N. and Gilkes, B. 1992. *Farm Monitoring Handbook*. The University of Western Australia, Nedlands, WA.

Thorpe, M. B. and Barbour, S. L. 1991. "Key Factors Affecting the Movement of Sodium into Surface Amendments Used to Reclaim Saline Land." *Proceedings of the 44th Canadian Geotechnical Conference*, Calgary, Alberta, Canada. 9 pp.

Wilson, G. W., Fredlund, D. G. and Barbour, S. L. 1997. "The Effect of Soil Suction on Evaporative Fluxes from Soil Surfaces." *Canadian Geotechnical Journal*, **34** (1): 145-155.

Liquefaction Risk Assessment for the State Highway 2: Dowse to Petone Upgrade Project.

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The site of the proposed realignment of State Highway 2 from Petone overbridge through to the Dowse intersection, Hutt Valley, New Zealand is in a region of high seismicity. The alignment is subject to very large seismic design loadings (1000 year Return Period PGA of 0.65g, Maximum Credible Earthquake PGA 1.0g) due to its proximity to the active Wellington Fault (M_w 7.4). Following a site-specific seismic hazard assessment, the geotechnical assessment of the site included a liquefaction analysis addressing likely magnitude of induced ground subsidence and effects on bridge piles, MSE abutments/ ramps, and nearby commercial properties at Dowse. Results of the seismic assessment present significant challenges to bridge and MSE wall designers for this project in achieving compliance with Transit New Zealand design philosophy requirements. This paper discusses methodologies adopted for the liquefaction assessment, results, and implications for the design of the structures on the project.

INTRODUCTION

As part of Transit New Zealand's highway network in the Wellington region, State Highway 2 is a strategic corridor linking Wellington city, in the south of the North Island, with the Hutt Valley suburbs, where it continues over the Rimutaka Hills into the Wairarapa and northwards. The SH2 Dowse to Petone project extends 2.7 km from the Petone foreshore northwest in line with the Western Hills, through two major intersections at Korokoro and Dowse (Figure 1). As part of the projected realignment of this road, intersections at Korokoro and Dowse are to be replaced with interchanges allowing traffic to continue along the state highway unimpeded. To achieve this, a number of overbridges and connecting Mechanically Stabilised Earth (MSE) ramp structures are to be constructed to link the existing side roads to both the highway, and the nearby Hutt Road – a major local road that lies adjacent to SH2.

The significance of the proximity of the Wellington Fault, and the geological setting overall, was evident from the results of the site specific seismic hazard assessment carried out by Beca in 2003, indicating that regional seismicity was heavily dominated by the close proximity of this fault, and the other regionally significant earthquake sources such as the West Wairarapa Fault, which produced New Zealand's largest magnitude earthquake in historical times (M_w 8.5, in 1855).

The recent geological history of the site has included periods of fluvial deposition from periodic avulsion of the Hutt River, in addition to periods of marine deposition in response to absolute sea level changes. Although some of these deposits have densities that fall within the range specified in the literature as being "liquefiable" most are of medium dense consistency and have some silt content, placing them outside the bounds of what the literature has *traditionally* focussed on in terms of what is known to be "liquefiable" (i.e. loose, uniformly graded (very well sorted), medium grain sized sands). This is because of the high levels of acceleration anticipated.

How liquefaction may affect nearby engineering structures can be assessed from observation of these phenomena in past seismic events; sand boils, large scale ground settlements, loss of bearing capacity resulting in foundation failure, lateral spreading resulting in shoving/ sliding towards an unsupported margin, buoyant displacement of buried services causing disruption. This paper provides a brief appraisal of the liquefaction risk assessment process as it was applied to the project, and discusses some of the outcomes of the study.

THE GEOLOGICAL SETTING AND PROFILE

The geological setting of the Hutt Valley region is dominated by active tectonics associated with the Wellington Fault, which runs from Cook Strait, through the Karori Reservoir, Thorndon, along the northwest side of the harbour and the west side of the Hutt Valley. The Western Hills are the remnants of a degraded fault scarp, progressively uplifted and eroded along the length of the Wellington Fault (Figure 1). The rock that forms the Western Hills is of the Torlesse Complex, and is commonly interbedded greywacke sandstone and argillite siltstone. As a legacy of its complex history, these rocks have undergone extensive deformation associated with periods of tectonic extension and compression, involving warping, tilting, folding and faulting of the rock mass, leaving it extensively fractured and jointed, and locally described as a "fault breccia".

The fault scarp extends some 300 m below sea level adjacent to the shoreline at Petone (Begg and Mazengarb, (1)). The fault is projected steeply through Quaternary sediments to a surface expression some 250 m and 150 m east of the Western Hills, at Korokoro and Dowse interchanges respectively.

The fault rupture, estimated to have been caused in the past by a M_w 7.4 level earthquake, is such that the eastern wall of the fault moved vertically down 0.5-1 m, and horizontally to the south 4.0 m relative to the western wall.

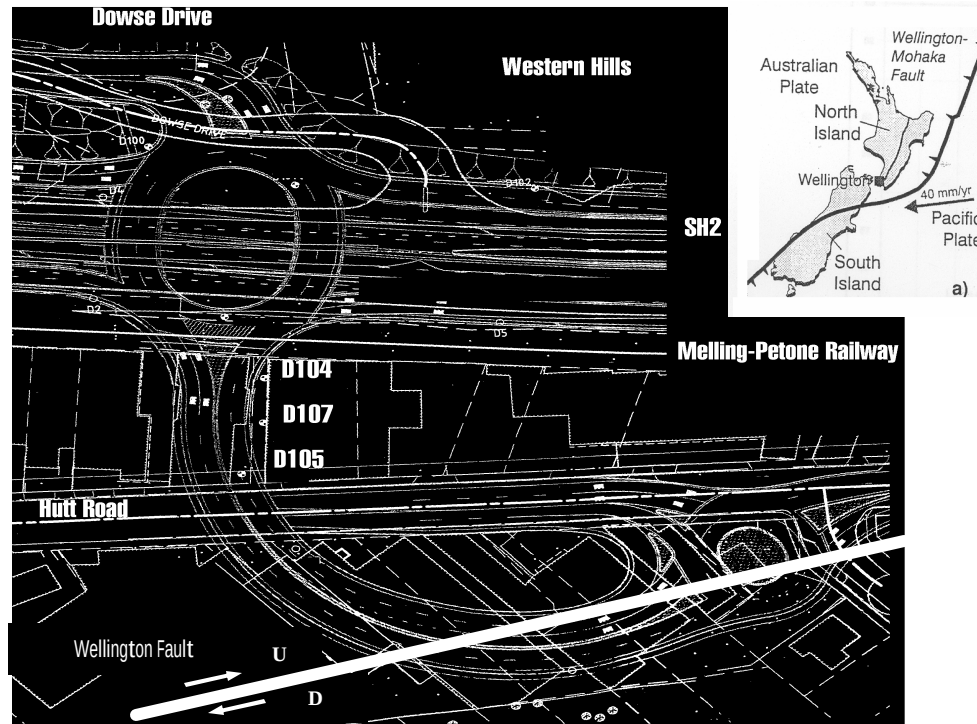


Figure 1: Location Plan for SH2 Dowse Interchange and connecting overbridge to Hutt Road. (1a) Inset: Tectonic plate boundary in relation to Wellington.

The location of the fault at the Dowse-Hutt Road site has been investigated by trenching (IGNS (2)). Findings from this study and mapping of surface expressions in the vicinity enable a confident positioning of the primary fault rupture zone to the east of Hutt Road, and probably crossing the proposed MSE ramp structure at Hutt Road.

The Hutt Valley floodplain is a tectonically controlled sedimentary basin, infilled with glacially derived alluvial gravels of greywacke origin, interglacial marine silts, and sands associated with sea level changes. This has resulted in distinct layers of fine-grained and coarse-grained materials. Near shore marine and beach sediments interfingering with fluvial and fan gravels deposited by the Hutt River and minor tributaries coming off the Western Hills, form the most recent deposits in the project area. Colluvium and rockslide debris occur to the west of the existing highway, and manmade fills surround and extend beneath the existing highway, most notably at Korokoro (5-6 m thickness).

GEOTECHNICAL CONDITIONS

The focus of the geotechnical investigations carried out for detailed design was on determining the founding conditions for the bridge piles throughout the project, and for the MSE ramp structures. Due to the soils on the project being largely medium dense greywacke-derived alluvial and colluvial soils, boreholes were favoured over CPTs.

During the preliminary phase investigations (1998), medium-dense marine sands were identified to the east of the Dowse Interchange in one borehole; however no laboratory testing was carried out, leaving the possibility of liquefaction as a design issue speculative and inconclusive. Thus during the detailed design phase investigations, additional boreholes were used to attempt to determine how extensive any potentially liquefiable materials were, and understand the likely impact this phenomenon would have on the proposed structures.

A summary of the vertical sequence of materials at Dowse to the east of SH2, towards Hutt Road, is presented in Table 1 (Beca, (3)).

Table 1
Dowse (East of SH2) Summary of Materials

Approx Depths (m)	Material Description	Corrected SPT N values ($N_{1(60)}$)	Comments
0-2	Silty Sands with some gravel (FILL/ disturbed natural ground)	20-50 (Typ. 37)	Varies in thickness and composition.
2-4	Gravelly Sands and Silts (ALLUVIUM)	22-50 (Typ. 42)	Largely sandy gravels extend from the highway east. Approx. 3 m thick.
4-18	Silty Sands (MARINE to MARGINAL MARINE/ ALLUVIUM)	13-50 (Typ. 20)	Sands with varying silt contents. Uniformly graded sands extend from immediately east of SH2, to east of Hutt Road.
18-?	Silty Sandy Gravels (ALLUVIUM)	50	Alluvial Gravels identified from east of Railway eastwards to eastern side of the Wellington fault.
-	Greywacke/Argillite basement rock	50+	Not encountered east of the Railway.

The sands identified between 4 and 10 m depth, are described as medium dense grey uniformly graded fine sand, occasional gravels and shells. SPT $N_{1(60)}$ range for this deposit: 13-25.

Correlations of the soil deposits encountered during the geotechnical investigations showed that the bedrock dips steeply away from the Western Hills, such that it is not encountered in boreholes to the east of SH2. Marine sediments were only encountered to significant depths (13.5m east of Hutt Road on the downthrown side of the fault), on the eastern side of SH2, indicating the on-lapping of the marine incursion across the site from east to west. Boreholes to the east of the Wellington fault were found to have the same marine deposits as identified west of the fault, however at these locations SPT testing recorded much greater densities (SPT $N > 30$). This difference may be in part to lower energy deposition where the marine deposits onlap, or is perhaps a function of directional earthquake energy effects.

LIQUEFACTION ASSESSMENT METHODOLOGY.

Site Seismicity and Liquefaction Assessment Philosophy

The liquefaction assessment methodology was adopted to meet the structural Design Objectives (Beca, (4)):

Objective 1: Withstand major earthquake shaking (assumed to be that which has a 10% probability of exceedence in the 100 year design life of the bridge structures in this project (i.e. approximately 1000 years) with a reasonable margin against collapse, and;

Objective 2: Withstand the most severe earthquake shaking (assumed to be equivalent to that from the MCE with a median plus one standard deviation predicted shaking) with a low margin against collapse.

For earth structures, including MSE with a period at or close to zero, the 1000-year return period (design base earthquake) Peak Ground Acceleration (PGA) is 0.65g. For MCE level of shaking, the recommended PGA for zero period structures is 1.0 g.

On finding evidence in the geotechnical profile for potentially liquefiable marine sands, a liquefaction assessment was carried out, to verify the layers would indeed liquefy under the design base earthquake, and attempt to quantify the likely effects on the proposed structures for the Dowse-Hutt Road sites.

Source Information and Parameters Used

The site-specific hazard report included graphs of recurrence of PGA for various magnitude ranges (Figure 2) and the response spectra for the MCE (Wellington Fault), which were used to evaluate the likelihood of liquefaction occurring.

Soil grading information for individual soil layers was based on nearest available grading test data in the borehole column. Some samples obtained from open nose SPT tests during drilling had to be combined to provide sufficient sample for testing, reducing the accuracy of the sieve grading test, but providing a ballpark figure for fines content. Hydrometer testing provided grading of the silt and clay portion of the samples, an important consideration for the liquefaction assessment of sands with some fines present.

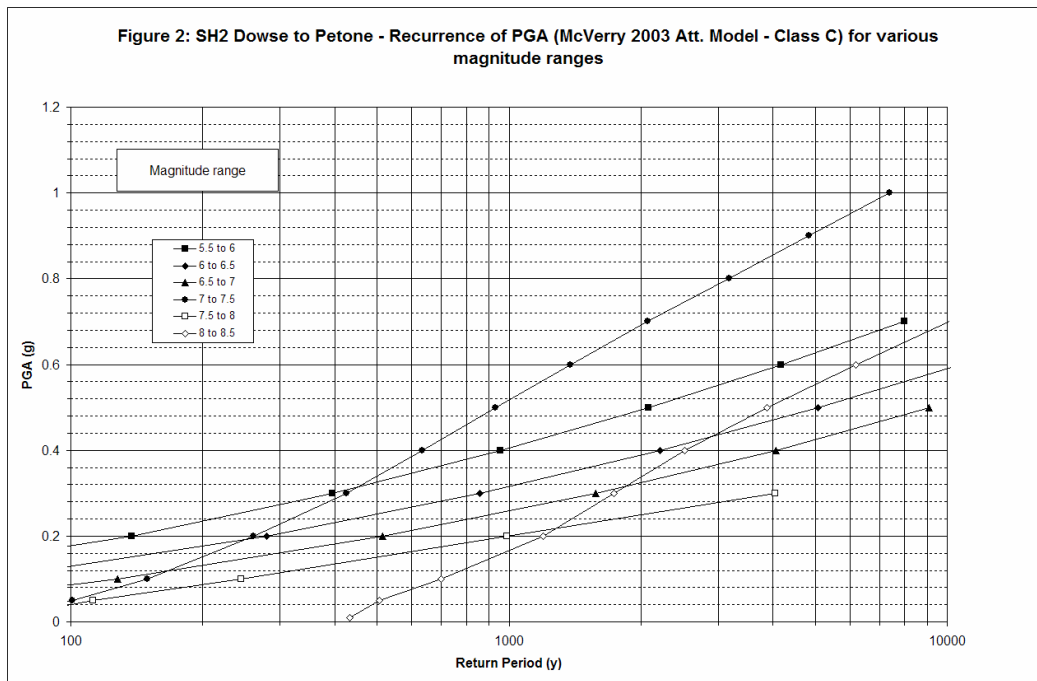


Figure 2: Recurrence of PGA for various magnitude ranges. Magnitude ranges 7-7.5 and 8-8.5 have much larger increases in PGA with return period, due to the weighted contribution provided by Wellington and West Wairarapa faults respectively. Source: Beca (4).

Assessment Method followed

There are many factors that affect whether a certain soil at a certain site, under a certain magnitude earthquake will liquefy. As part of the assessment, all of these factors need to be considered (Kramer (5)):

Historical Criteria

Evidence of past liquefaction at a site may well be evidence of susceptibility of soils to further liquefaction. There are no known reports of liquefaction damage to this site occurring during the M_w 8.5 Wairarapa earthquake of 1855, however much of the Lower Hutt Valley was an undrained wetland at the time, and any evidence of liquefaction may have been missed. Historical earthquakes also lessen the risk of further occurrences of liquefaction under the following criteria:

Geological Age and Origin

It is generally considered that older deposits have a lower risk of liquefying than younger deposits, as older deposits are more likely to have experienced similar seismic events, already resulting in densification of the soil. Weathering effects over time may lead to a cement-like bonding between grains, such that there is a higher amount of energy required to break the initial particle configuration in order to attempt to densify the deposit. At Dowse-Hutt Road, the proximity of the marine sand to the surface deposits, and our understanding of the geological history of the near surface deposits, suggest this marine sand was laid down in the Holocene (<6Ma), prior to shoreline regression to its current location at Petone. It may have experienced anywhere between 1 and 10 prior M_w 7.4+ prehistoric seismic events.

Particle Size and Classification

In general, the potential for a soil to liquefy decreases with increasing fines content in particular when those fines are plate-like minerals with high surface area such as clays. For this reason, Plasticity Index, liquid limit, and clay content are considered to be good test criteria for whether a particular soil will exhibit liquefaction or not. Non-plastic silts are known to liquefy. Using Tsuchida and Hiyashi (6) curves of liquefaction susceptibility based on grain size, some sands, over the depth range typically 5-7 m at Dowse-Hutt Road with non plastic silt fines in the range 13 - 43 % fall into the "easily" to "very easily" liquefy categories. (Figure 3).

Density

Beyond a certain density, the earthquake energy will be transferred sufficiently through the soil via direct grain-to-grain contact that the granular configuration of the sand will not attempt to recompact, thereby avoiding placing stress on water within the pore spaces. Also, beyond a certain depth (say a nominal 10 m), liquefaction will be highly unlikely to occur due to the additional vertical stress on the soil particles providing resistance to the effort to recompact the soil when shaken. In liquefaction assessment procedures adopted in the literature, SPT and CPT are the common tools for determining the relative density of the soil. The NCEER (Youd and Idriss (7)) method

suggests that soil in the “Dense” category, (SPT $N = 30+$) has an extremely low risk of liquefaction, regardless of the Cyclic Stress Ratio that may develop.

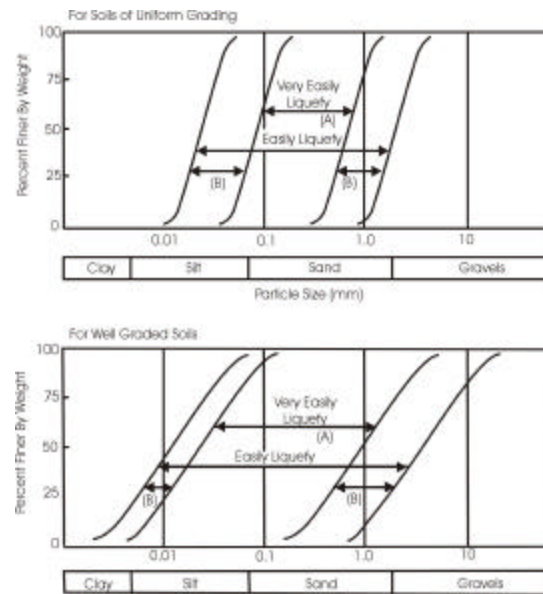


Figure 3: Tsuchida and Hiyashi (6) Grain size distribution curves for liquefaction susceptibility. The marine sands identified as being potentially liquefiable when plotted on the Tsuchida curves, fall within the Very-easily to easily liquefy categories for uniformly graded soils.

Calculation of Resistance to Cyclic Stress

Under Objective 1, the susceptibility of the soils to liquefaction was assessed using the probabilistic method outlined in the paper by O’Brien et al (8). The calculations adopted for assessing liquefaction were taken from the NCEER report. Corrected SPT data was used from boreholes at locations adjacent to the proposed structures at Dowse and Hutt Road sites. Magnitude Scaling Factors (MSF) were applied as per NCEER. The upper bound values of Andrus and Stokoe and lower bound values of Idriss were averaged for our calculations, and interpolated for the midpoint of ranges corresponding to those magnitudes provided in the site specific hazard report (Figure 2).

The probabilistic method is summarised as follows:

1. The soil stratum below the water table was subdivided into ‘sub layers’ of 0.5 m thickness.
2. PGA likely to cause liquefaction for various magnitude ranges was assessed for each sub layer. Earthquakes of magnitude less than 5.5 were excluded, as they are unlikely to cause liquefaction due to the short duration of shaking caused by these events. However, if low magnitude events were included the result would not change significantly, as the frequency of liquefaction from these events would be very low. This was calculated through solving to obtain a factor of safety (FOS) of 1.0 with corresponding MSF’s applied.
3. The return period associated with the PGA to cause liquefaction for a particular magnitude was assessed from Figure 2.
4. The Expected Annual Frequency was calculated by inverting the return period.
5. The return period for liquefaction for a particular layer is the inverse of the sum of the expected annual frequencies for each magnitude level.

The susceptibility of a soil layer to liquefy under Objective 2 was assessed by taking the PGA recommended by the site-specific hazard report, as being the spectral acceleration at a period of zero for the MCE. The liquefaction susceptibility in this case was assessed non-probabilistically as it is a definite single event case that was checked. This involved a calculation of the cyclic stress ratio (CSR) using the MCE PGA, to determine directly whether liquefaction occurred given the cyclic resistance ratio (CRR) determined by the properties of the soil. The PGA at which liquefaction was initiated for these deposits was also determined.

RESULTS OF LIQUEFACTION ASSESSMENT.

At Dowse interchange, the results of this assessment quantified the risk of liquefaction in terms of the expected return period for liquefaction to occur. Results from Boreholes D104, D105 and D107 (Refer Figure 1) demonstrated the susceptibility of the marine sand deposit (5-7m depth) to liquefaction. Despite the fact that at their location (between the railway and Hutt Road) the marine sand deposit is only in the order of ~ 2 m thick, this deposit (with similar and lower SPT readings) is observed in historical boreholes on the eastern side of Hutt Road at

up to 6.5 m thick. This indicates that potentially liquefiable soils cover a broad area, affecting the abutment ramps, and the piles of the proposed Hutt Road overbridge. These historical borelogs were not assessed beyond inspection as no grading tests were performed on samples preventing an accurate analysis.

Return periods for the occurrence of liquefaction in soil layers between 5 m and 7.5 m depth in boreholes D104, D105 and D107 average 335 years, with a minimum of 205 years, and a maximum of 680 years. A loose to medium dense silty sand at 9-10m depth in D105 had a liquefaction return period of only 120 years using this method.

Level ground settlements were estimated using the method of Ishihara and Yoshimine (9), and calculated to be in the order of 100-200 mm west of Hutt Road, with settlements to the east of Hutt Road potentially up to three times this amount. This ground settlement is expected to induce a downward force due to negative skin friction on the piles supporting the bridge piers and abutments. The load was assessed from the thickness of soil above the water table that would form a crust in the event of liquefaction. Lateral resistance provided by the liquefiable deposit was ignored under the seismic load case, as per the requirements of the Transit NZ Bridge Manual (10), which conforms with the findings of Ishihara (11).

Lateral spreading was considered as a possibility, however the nearest unsupported margin is the Hutt River over 1 km away, and borehole logs east of the Wellington fault show deposits too dense to liquefy, proving a restraint to lateral spreading.

CONCLUDING REMARKS

The SH2 Dowse to Petone project has provided a challenge in ascertaining the seismic affects on the project, complicated by the presence of the Wellington Fault transecting the site. In addition to the anticipated large fault movements, and high design seismic accelerations, liquefaction was found to be a risk that required special design consideration for the structures on the project.

As part of the assessment of liquefaction risk, a clear understanding of the site geology and how it affects the likelihood and extent of potential liquefaction was required, followed by an understanding of the impacts that liquefaction is likely to have on the design of the proposed structures, in terms of additional loads on piles, affects on MSE footings, level ground settlements etc. The use of probabilistic methods to evaluate liquefaction risk is considered important to maintain the integrity of the nature of the seismic hazard assessment data (being probabilistic).

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REFERENCES

1. Begg J., Mazengarb C. (1996) "Geology of the Wellington Area". Institute of Geological and Nuclear Sciences.
2. Institute of Geological and Nuclear Sciences, (1998) "SH2: Melling – Petone Upgrade Project. Location of Wellington Fault Adjacent to State Highway 2". Letter to Beca dated 6 March 1998.
3. Beca Carter Hollings and Ferner Ltd. (2003a), "SH2 Interpretive Geotechnical Report" Job # 3202860.
4. Beca Carter Hollings and Ferner Ltd. (2003b) "Seismic Hazard Assessment for SH2 Dowse to Petone." Job # 3202860.
5. Kramer S.L., (1996) "Geotechnical Earthquake Engineering", Prentice Hall, USA.
6. Tsuchida, H., Hayashi S. (1971), "Estimation of Liquefaction Potential of Sandy Soils", Third Joint Meeting U.S. Japan Panel on Wind and Seismic Effects, UJNR, Tokyo.
7. Youd T.L., Idriss, I.M. (1997) "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils": National Centre for Earthquake Engineering Research, Buffalo N.Y. (Technical Report NCEER-97-0022).
8. O'Brien M.T., Novakov D.R., Jury R.D. (2003) "Seismic assessment for industrial facility in Dunedin", *2003 Pacific Conference on Earthquake Engineering*.
9. Ishihara K., Yoshimine M. (1992) "Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes", *Soils and Foundations* Vol. 32, No. 1 pp173-188, Japanese Society of Soil Mechanics and Foundation Engineering.
10. Transit New Zealand (2000) "Bridge Design Manual."
11. Ishihara K. (2003) "Liquefaction-Induced Lateral Flow and its effects on Foundation Piles." *Geotechnical Society of New Zealand Proceedings of Tech Gps. Vol 30. Issue 1 (GM) 2003*

Retaining Wellington's Roads, Chaytor St Case Study

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Roads of Wellington city are often carved around steep greywacke rock terrain. Geotechnical engineers are faced with challenges to provide cost-effective solutions to retain the associated sidling fills, which often have marginal stability. This paper presents a case study of a tied back, concrete palisade wall that was constructed to retain a section of Chaytor St in Wellington.

The project was challenging given the significant depth of fill, site constraints and high seismic design accelerations. Significant effort was made during the development of the design to provide a cost effective solution that fulfilled both the requirements of the New Zealand Loadings Standard and the requirements of the client. The design that was adopted relied on the anchors resisting the majority of lateral load and avoided the more traditional approach of resisting part of the lateral loads through embedment of the wall piles into the underlying rock. The paper discusses the design philosophy of the tied back concrete palisade wall.

INTRODUCTION

During mid 2002 signs of creep and instability was evident by cracking and subsidence in the Chaytor Street carriageway.

Wellington City Council, the local road controlling authority, engaged Tonkin & Taylor Ltd to investigate the stability of sections of Chaytor St and then later design and supervise the construction of retaining walls to stabilise the outer edge of the carriageway. This paper discusses one particular wall that was constructed to retain part of the Chaytor St carriageway.

There were a number of design issues that provided a challenge in developing a cost-effective solution for the Chaytor St wall, these included:

- Significant depth of loose fill
- Site constraints
- High seismic design accelerations
- Steep topography
- 2m widening of the road for a bus lane

A number of retaining solutions were investigated however it was found that the most cost-effective solution for the site was the construction of a tied-back, concrete palisade wall. The constructed wall is shown in Figure 1.

The solution relied on the anchors resisting the majority of the lateral load. This was an alternative to the more traditional approach of resisting part of the lateral load through embedment of the piles into the underlying rock.

The preferred solution was developed in an interactive approach with the client. A variety of solutions, with differing levels of risk were presented to the client along with construction cost estimates. This allowed the client to make an informed decision based on the funding available and the level of risk that they were willing to

accept. This approach is particularly important to authorities that need to prioritise work and ensure that they gain the maximum benefit from their limited budgets.



Figure 1. Completed Tied-Back Concrete Palisade Retaining Wall.

SITE DESCRIPTION

Chaytor St is a principal arterial route linking the largest Wellington suburb of Karori to Wellington city centre. The road has been constructed as a sidling cut to fill, where material cut from the inside of the carriageway was used to fill the outside of the carriageway.

Chaytor St traverses a steep slope where the existing ground below the wall slopes away at up to 50 degrees. Below the road there are signs of historic instabilities.

During mid 2002 signs of creep and instability became evident in cracking and slumping of the carriageway. The existing wall was in a poor condition and was providing little support to the carriageway. On the basis

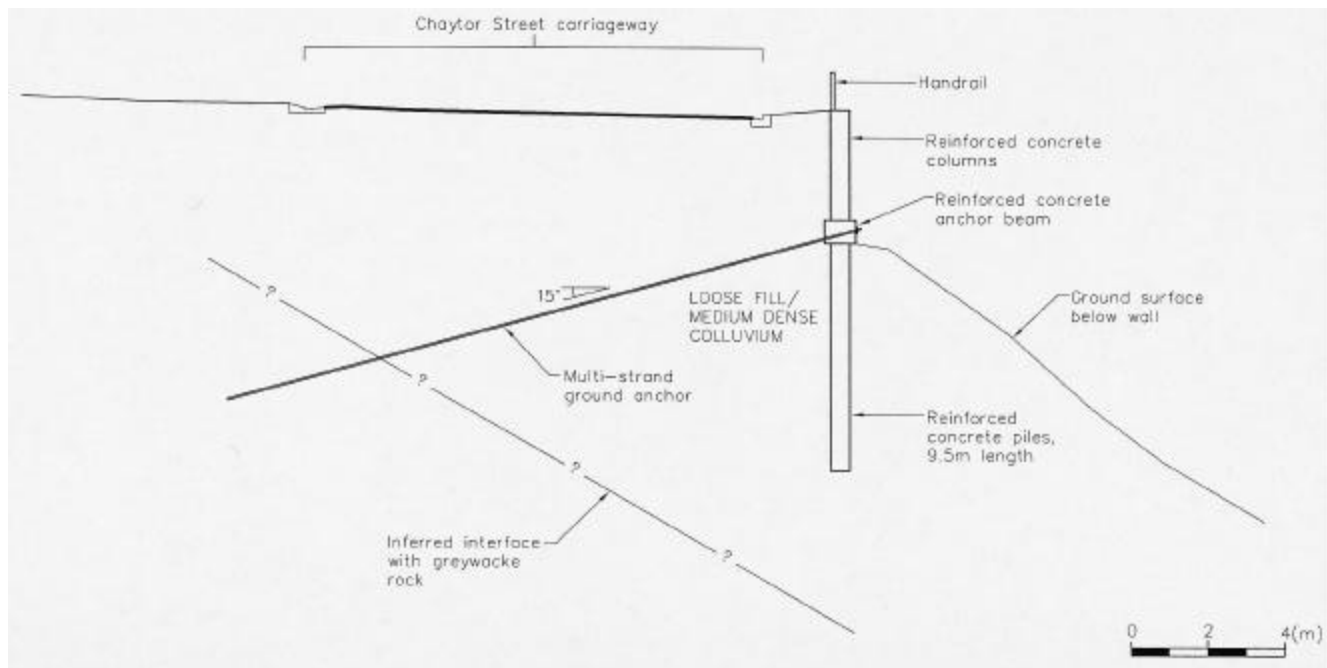


Figure 2. Typical Retaining Wall Detail.

of Tonkin & Taylor's recommendations Wellington City Council decided that the wall would be replaced over a 50m length.

SUBSURFACE CONDITIONS

An investigation comprising boreholes, cone penetrometer tests and geological logging of rock exposures was undertaken by Tonkin & Taylor Ltd. The investigations indicated that the proposed wall traversed an infilled gully. There was 15m of loose fill underlying the footpath at the centre of the valley alignment. The fill was underlain by approximately 3m of colluvium over greywacke rock.

A section through the centre of the infilled gully is shown on Figure 2.

SITE CONSTRAINTS

There were a number of site constraints that had to be assessed and allowed for during design. These included:

- The steep slope below the wall.
- The significant depth of loose fill.
- The distance behind the wall to greywacke rock. This meant that some anchors had to extend 20m from the wall through loose fill to encounter rock.

- Minimising the impact on traffic. Chaytor St carries a significant amount of traffic and it was necessary to keep both lanes open during construction.
- Minimising the impact on the local residential community.
- Widening of the road for a bus lane.

SEISMIC LOADING

The active Wellington fault passes within 200m of the site and dominates the seismic hazard for the site. The New Zealand Loadings Standard NZS4203:1992 requires that the wall be designed so that it remains stable following a 450-year return period earthquake. This is equivalent to a movement on the Wellington fault of M7.6 magnitude. Such a seismic event is expected to impose peak accelerations of up to 0.5g. In addition to the requirements of the New Zealand Loadings Standard it was agreed with the client that we would also assess the potential deformation of the wall and carriageway following a 450-year return period earthquake.

STABILITY MODELLING

The majority of the stability modelling was carried out using the finite element, soil modelling program Plaxis.

A site model was developed from the investigation information, topographical data and soil parameters

obtained through back analysis of the previous retaining wall and observed instabilities on the existing fill slopes.

It was assumed that the existing slope at the wall location would have a factor of safety in the order of 1.15 under static load conditions. This is consistent with the observed creep deformation observed at the site.

The soil parameters obtained from the back analysis of the previous slopes were then used in the design and stability modelling of the new wall.

A critical component of the design process was modelling the behaviour of the slope below the wall under ultimate limit state conditions. The static factor of safety of a slip below the wall was far less than the required 1.5 and the return period of a seismic failure was less than a 100-year event. Therefore the potential depth of material that could slip away below the wall was assessed for both the static and seismic ultimate limit state cases. The ground model was then modified by removing that depth of soil that was unlikely to provide passive support to the structure under the ultimate limit state case.

The stability modelling of the wall showed that the seismic case was likely to be the critical load case. The most probable failure under the seismic load case was a large rotational failure. A typical plaxis output is shown in Figure 3. The large rotational failure was one of the main considerations that influenced the selection of the wall design. To achieve a reasonable level of seismic stability the wall needed to be embedded below potential failure surfaces and anchors needed to develop load/bond beyond the failure surface.

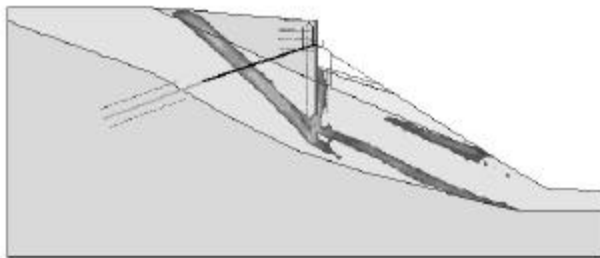


Figure 3. Plaxis Strain Modelling Output for a 450-year Return Period Earthquake (Ultimate Limit State Seismic Case).

During design optimisation it became apparent that the cost of a wall, where permanent displacement would be negligible following a 450-year seismic event, would be prohibitively expensive. Therefore alternative solutions were also presented to the client where the wall remained stable following a 450-year seismic event but minor displacements were expected. These solutions satisfied the New Zealand Loadings Standard but were not as robust, however the estimated construction cost of the

chosen design was half of that for a solution where negligible permanent displacements were expected.

DEVELOPMENT OF DESIGN

In the early stages of the design process many concept designs were considered for the retaining wall including; a gravity wall, a soil nail wall and a concrete palisade wall with tie back anchors extending to either greywacke rock or deadman. The level of risk, construction cost, impact on traffic and the impact on the residential community were then considered in the development of the final design.

The option of a gravity wall such as a geogrid reinforced fill embankment was considered as the fill at the site was reasonably granular in nature and would be suitable for compacted fill. However this option would have required substantial earthworks to found beyond potential failure surfaces under the seismic load case. The option would also have caused significant impact on existing services, traffic flow and the residential community.

Stabilising the slope through the use of soil nails was considered. This solution required the full length of slope to be nailed to provide adequate stability. Soil nails would need to be 10 to 15m in length to extend beyond the potential seismic failure surface. The number, length and installation options (through granular fill) did not make this solution cost effective.

A tie-back concrete palisade wall with multi-strand anchors extending into rock was chosen as the most cost effective solution. The wall had a clear retained height of 3.5m and piles at 1.5m centres extending 9.5m below the base of the anchor beam. The main benefits of this design over the other concept designs are:

- The wall piles have sufficient embedment to resist the potential global rotational failure under seismic shaking;
- Minimal impact on the traffic and residential community during construction;
- Minimal impact on services within the carriageway due to reduced amount of earthworks;
- The use of multi-strand anchors over the more traditional bar anchors was adopted to space anchors at greater centres, significantly reducing the total length of drilling required;
- The cost of the wall was significantly less than other solutions providing a similar level of stability improvement.

With this solution the wall is expected to remain stable following a 450-year return period earthquake with displacements in the order of 200 to 300mm. Following

the earthquake the road is likely to be passable and it is anticipated that the earthworks required to level out the seismic displacements are likely to be relatively quick and low cost. Furthermore the seismic hazard is comparable to other hazards along the road, such as rockfalls, that would need to be cleared prior to the road reopening.

In order to prevent displacements following a 450-year event the piles would need to be embedded in the underlying rock. Given the level of risk and low cost of remedial works following the 450-year seismic event the solution with the piles not extending to the underlying rock proved to be the most cost effective.

CONCLUSIONS

Geotechnical Engineers are faced with challenges to provide cost effective solutions to retain the roads of Wellington city that are often carved around steep greywacke rock terrain. The associated sidling fills are often marginally stable and the design of retaining solutions can be complex.

This paper presented a case study of a retaining wall along Chaytor Street, Wellington. An interactive approach with the client was successful in providing a cost effective solution to the client.

The case study shows a project where a variety of solutions were presented to the client. A clear description and estimate of risk was presented for each option along with the expected construction cost. This allowed the client to prioritise the wall against other works and make an informed decision on the most cost-effective option to Wellington City Council.

The case study is an example of a project where the final solution was not the most robust solution but proved to be the most cost-effective. The wall design satisfied the requirements of the New Zealand Loadings standard by remaining stable under the ultimate limit state cases. However by accepting displacements under the maximum design earthquake, while still ensuring that the wall remained stable, the cost of the project was half that of the more typical robust solution of embedding the piles into the underlying rock.

SELECTION OF SUITABLE GROUND IMPROVEMENT TECHNIQUES FOR SOUTHEAST QUEENSLAND SOFT CLAY

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Abstract: Soft estuarine clays are found in many expressway and motorway projects in Southeast Queensland. The strengths of these clays are very low and as such embankments failures can be encountered during construction. Several ground improvement techniques are available to increase the soil strength, and these techniques include the use of surcharge with stone column or vertical drains. When the coefficient of consolidation is low, the sensitivity and organic contents are high, preloading techniques with vertical drains may not be effective due to smear effects. Thus, chemical stabilisation may be the appropriate alternative ground improvement technique. The cement stabilisation improves the strength of soft clay by effectively aggregating smaller particles with additional chemical bonds. A laboratory study has been conducted to examine the influence of cement additives on Southeast Queensland soft clay. Unconfined compression tests showed that, 5% cement additives cured for 7 days increased the unconfined compressive strength when compared to untreated samples. Further, cement additives were also found to increase the pre-consolidation pressure in the oedometer tests. Thus, the initial studies conducted have demonstrated its potential use in shallow and deep stabilisation for Southeast Queensland soft clay.

INTRODUCTION

Soft compressible clays are found in many expressway and motorway projects in Southeast region of Queensland. Such sub-soil conditions can have considerable implications on the design of embankments and structural foundations. This is due to both low shear strength, and a tendency to consolidate and deform with time. Often the simplest solution to such unfavourable soil conditions is to find an alternate alignment, although this can be costly and impractical. As an economic alternative to structural foundations, ground improvement techniques are becoming more prevalent. Ground improvement in Australia primarily encompasses the use of granular stone columns, surcharge with vertical drains, and chemical stabilisation.

In the case of the sensitive soft clays found in Southeast Queensland, substantial remoulding takes place during the stone column or vertical drain installation, resulting in smearing of the drain surface, thus rendering them ineffective. When the coefficient of consolidation is low, and also when the organic content is high, preloading techniques may not be the most appropriate technique. Therefore, the use of cement or lime stabilisation, which entails mixing the chemicals with the soil has various advantages. These methods improve the engineering properties of soft soils by effectively aggregating smaller particles into larger cementitious clusters, subsequently increasing its stiffness and volumetric stability while accelerating consolidation.

Southeast Queensland clays have widely varying engineering properties, depending largely on the deposit's depth below the ground surface and the proximity to the water table (as shown in Figure 1). Based on the field shear vane, the undrained shear strength of soft/very soft clays is around 10-15 kPa. Natural moisture contents commonly vary between 60 and 120%. The liquidity

indices are generally in the range of 1.5 - 2.5, displaying high sensitivity. Compressibilities as high as $C_c/(1 + e_0) = 0.4 - 0.5$ have been observed in the laboratory. At these high compressibilities, strain rate effects can be significant. Macro fabric features have been variable, and in the worst cases $C_v = 0.1 - 0.3 \text{ m}^2/\text{yr}$, and $C_v/C_h = 1.0$ have been typical based on laboratory testing. Organic contents up to 10% have been observed, with high creep rates ($C_{ae} > 1\%$). Given the sensitivity of soft clays, the compressibility parameters are likely to be underestimated in the normally consolidated range. Piezocone dissipation tests are also masked by the remoulding of the clay caused by insertion of the cone (Wijeyakulasuriya *et al.*, 1999 [1]).

Cement, lime and fly-ash are the commonly used stabilisers for chemical stabilisation which were successfully executed by many researchers (Saitoh *et al.*, 1985 [1]; Bergado, *et al.*, 1996 [3]; Fahoum, *et al.*, 1996 [4]; Porbaha, 1998 [5]; Bergado, *et al.*, 1999 [6]; Holm, 1999 [7]; Hoyos, *et al.*, 2004 [8]). In addition, shallow stabilisation with chemical admixtures has long been used to improve the performance of base course and subgrade materials in the construction of road pavements. It is considered a cost-effective method of improving long term performance and reducing whole-of-life costs of heavily trafficked pavements (Austroads, 1998 [9]). The increased bearing capacity of cement-modified subgrade allows a reduction in the thickness of the base course required to ensure prolonged serviceability.

In this paper, the research focuses on the cement addition technique. General purpose cement, also known as ordinary Portland cement (OPC), is widely available throughout Australia, making it a feasible ground improvement medium in most cases. Cement can be applied to soft clays via one of two methods. Shallow mixing, where cement is added to the upper 200-300 mm of the subgrade using a stabilising machine over a relatively

wide area, is widely utilised as a rehabilitation technique to improve the strength of granular pavement materials found in road bases. Deep mixing is the process of mixing cement with clay using drilling equipment to form small radii deep cylinders, which act somewhat similarly to traditional concrete and steel piles.

This paper will explore the effects of cement additions to Southeast Queensland soft clays, with particular attention to strength gains and lessening compressibility. A laboratory testing program comprising Unconfined Compression and Oedometer tests has been undertaken to investigate the increase in strength and the reduction in the compressibility characteristics, resulting from differing magnitudes of cement addition.

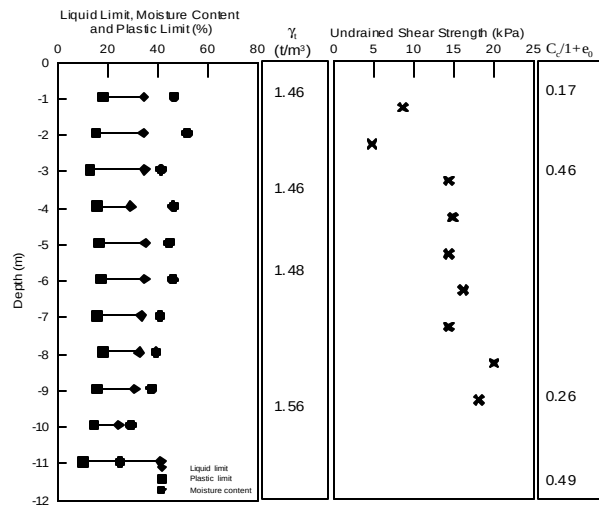


Figure 1(a) Geotechnical properties of clay in Gold Coast Highway (based on Wijeyakulasuriya *et al* (1999) [1]).

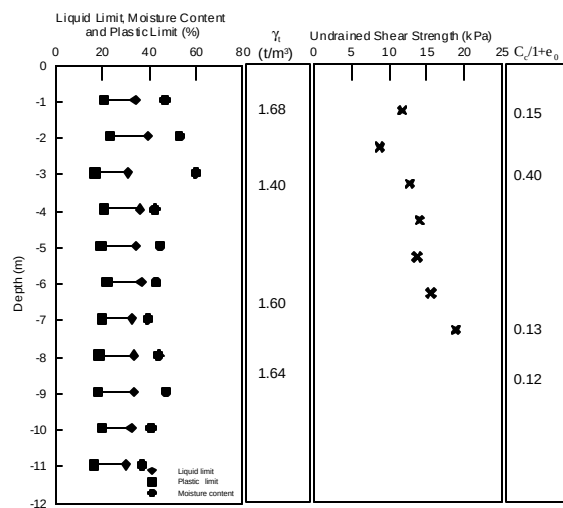


Figure 1(b) Geotechnical properties of clay in Sunshine motorway (based on Wijeyakulasuriya *et al* (1999) [1])

QUEENSLAND EXPERIENCES

Wijeyakulasuriya *et al* (1999) [1] presented the behaviour of three trial embankments founded on sensitive soft clays along the eastern coastal belt of Queensland. In Case Study A, the embankment at Mackay was built to

5m height by stage loading. The construction period was 6 months and the period of preload was 9 months. Case study B was at the Sunshine Motorway, to assess the feasibility of 2-staged construction within an overall construction period of 300 days. Case study C was at Coombabah Creek on the Gold Coast. The behaviour of these embankments is still being studied. In Case Study C, stone columns were used. In Case Study B, the section of the motorway traverses a swamp comprising very soft organic marine silty clay ranging in thickness from 4-10 m and underlain by sandy deposits. The height of the embankment was 2.8 m and was reinforced with high strength geo-synthetic at the embankment and foundation interface.

The embankment at the Sunshine Coast had a top width of 17 m on nominally 1V:2H batters. The berms were 1m high with a width of 5m on one side and 8m on the other side. The trial embankment comprised three 20 m sections, with the end sections installed with wick drains (Section A, 1m spacing and Section C, 2m spacing) and the middle section on undisturbed virgin ground as a control section (Section B). The settlements of the three sections shown in Figure 2 indicate that the wick drains have not accelerated settlements to any great extent. However the settlements in all sections were in excess of 1.5m. These measurements are now being further analysed.

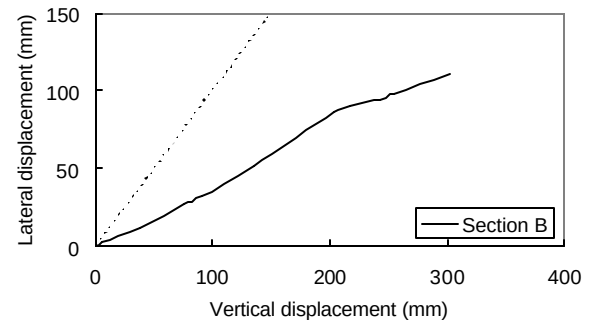


Figure 2(a) Variation in lateral displacement with settlement in Sunshine motorway trial embankment (based on Wijeyakulasuriya *et al* (1999) [1])

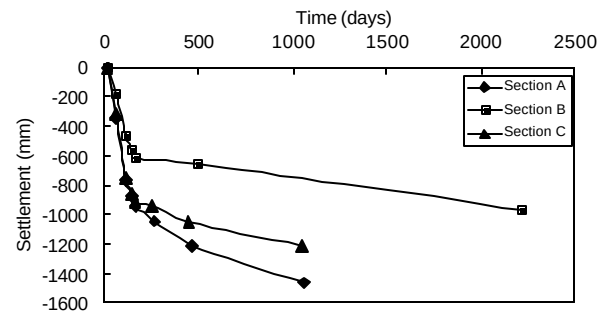


Figure 2(b) Time versus settlement curve for Sunshine motorway trial embankment (based on Wijeyakulasuriya *et al* (1999) [1])

The trial embankment at Coombabah traversed a swamp with soft clay up to 13 m. Here again the trial

embankment consisted of 3 sections. Section A had stone columns at 1m spacing and Section C had the stone column spacing as 3m. Section B was without any stone columns. The stone columns extended to 16 m. The height of the trial embankment was 2m with a top width of 12 m on 1V: 2 H batters. The stone columns were installed by the jetting process as used in vibro-flotation. The stone columns too were not found to be effective in reducing the settlements (as shown in Figure 3).

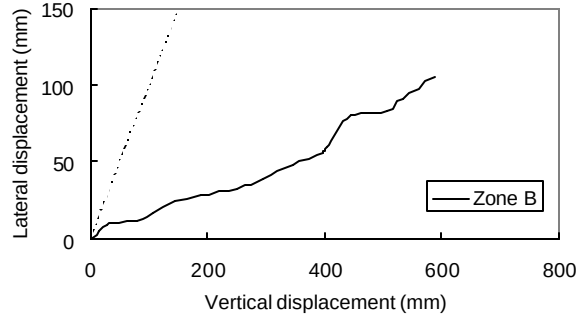


Figure 3(a) Variation in lateral displacement with settlement in Gold Coast Highway trial embankment (based on Wijeyakulasuriya *et al* (1999) [1])

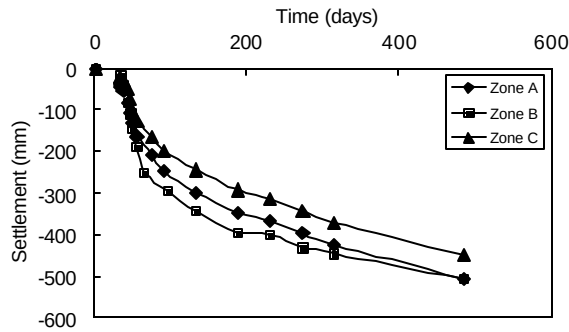


Figure 3(b) Time versus settlement curve for Gold Coast Highway trial embankment (based on Wijeyakulasuriya *et al* (1999) [1])

LABORATORY STUDIES

In this paper, the soil samples are taken from a site in Gold Coast Highway. This section of the highway traverses a swamp with up to 13.5 m of soft clay. The moisture content ranges approximately 110% at the surface and 60 % at a depth of 11m. The wet density (r_{wet}) of the clay was determined for several depths, and ranged from a low of 1.46 t/m^3 to a high of 1.56 t/m^3 . The dry density (r_{dry}) of the clay ranged from a low of 0.54 t/m^3 to a high of 0.96 t/m^3 . The density fo solid (r_s) of the clay ranged from a low of 2.61 t/m^3 to a high of 2.69 t/m^3 . The liquid limit is generally uniform with depth, at a value of 60% to 70%. The plastic limit is generally consistent with depth and ranged from 25% to 49 %. The classification of the soils is based on the Unified Soil Classification System. Using these grading (fine-grained soil, with 50% or more is $< 0.075 \text{ mm}$) and the

Atterberg limit given (liquid limit of 70%, and plasticity index of 35%), the soil can be classified as “inorganic clay of high plasticity (CH)”.

The physical properties of Southeast Queensland soft estuarine clay samples, both treated with cement and untreated, have been ascertained (as tabulated in Table 1). As expected, cement treatment increased total and dry densities, raised plastic and liquid limits, and reduced the water content and void ratio when compared to untreated clay. The addition of cement to this clay produced an immediate reduction in void ratio, as demonstrated below in Table 1. This is likely due to the interaction between cement particles and pore water to produce solidification, reducing water content and filling in the voids with the resulting hydrated cementitious material.

Table 1 Physical properties of cement treated clay

Ce- ment Con- tent (%)	Total Unit Weight (t/m^3)	Dry Unit Weight (t/m^3)	Moisture Content (%)	Liquid Limit (%)	Plas- tic Limit (%)	Void Ratio, e
0	1.255	0.711	90.72	59.8	23.0	2.727
5	1.351	0.765	86.93	109.1	31.8	2.464
10	1.459	0.828	77.92	121.5	44.3	2.188
15	1.483	0.845	75.26	125.0	69.2	2.112

Untreated specimens of Southeast Queensland soft clay were tested in unconfined compression. It is evident from the results that the natural clay was very soft, attaining a peak unconfined compressive strength of about 27 kPa. Further, cement treated specimens of Southeast Queensland soft clays were tested. Samples of 5, 10, and 15% cement addition ratios were tested at 7, 14 and 28 days after mixing. The results of these unconfined compression tests have been presented to explore the two parameters under investigation. Plots of normal stress and axial strain for sample sets of differing cement content at curing times of 7, 14 and 28 days are presented overleaf in Figure 4.

For samples tested at 7, 14 and 28 day curing times, a sharp rise in stress was observed, attaining a maximum value at approximately 1.25% strain. The rate of stress rise for initial stages of each curing time plot seemed to grow with increasing cement content. It can then be concluded that the stress-strain behaviour of different cement addition ratios is consistently improved, independent of curing time. Maximum stress was attained at a much lower strain for cement treated samples than for untreated samples. The effect of additional cement is noticeable at these peak values, with higher admixture ratios yielding higher strength. This effect can be quantified by comparing the peak axial stresses, as demonstrated in Figure 5.

The addition of cement produces dramatic improvement in strength of Southeast Queensland soft clay. The effect of curing time is observed to be much less significant than the cement content parameter in terms of the rate of unconfined compressive strength gain.

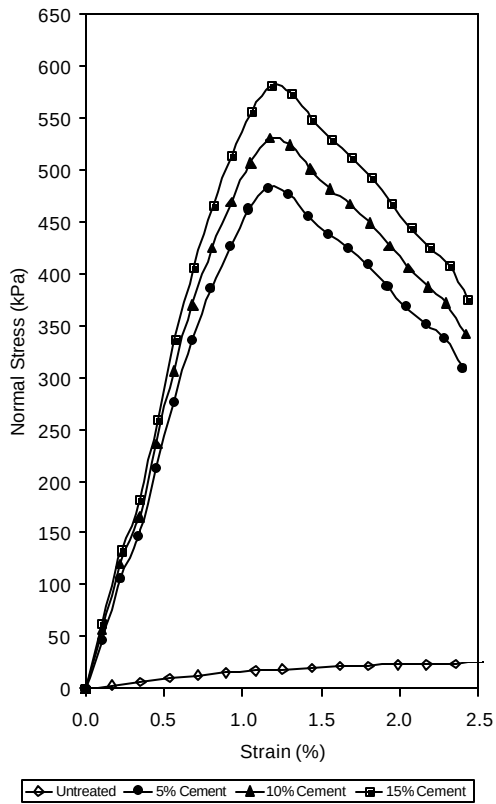


Figure 4(a) Comparison of stress-strain behaviour of treated and untreated clay (7 days curing period)

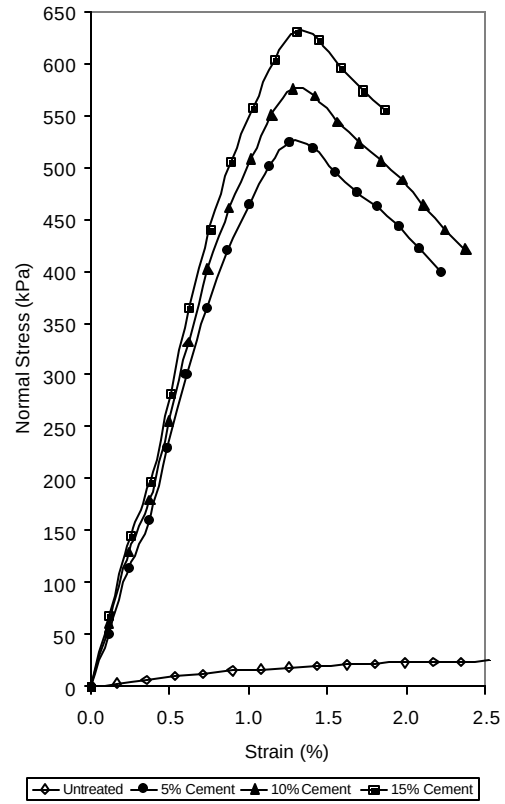


Figure 4(c) Comparison of stress-strain behaviour of treated and untreated clay (28 days curing period)

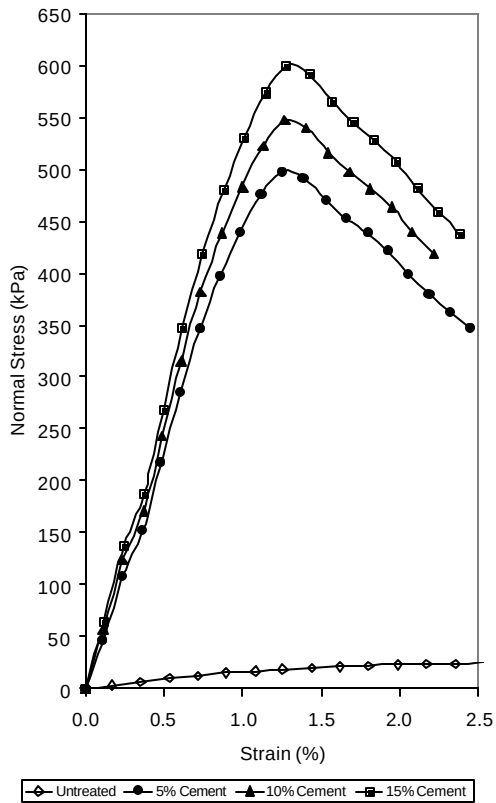


Figure 4(b) Comparison of stress-strain behaviour of treated and untreated clay (14 days curing period)

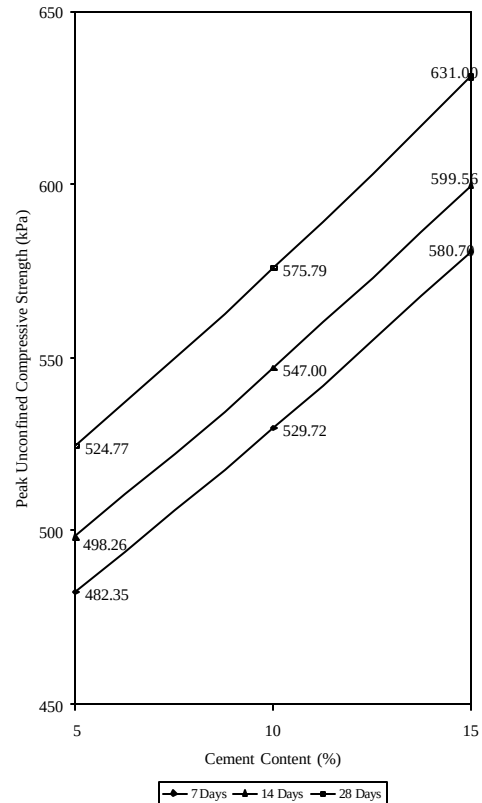


Figure 5 Relationship of Peak Unconfined Compressive Strength with different cement content.

Compressibility characteristics of untreated specimens of Southeast Queensland soft clay were tested using the oedometer consolidation test. The results of these tests are graphically presented in Figure 6. It is apparent from the results that, the natural clay was very compressible, attaining a peak compression index (C_c) of 1.79. Plots of void ratio and vertical stress for sample sets of differing cement content at curing times of 7, 14 and 28 days are presented in Figure 6. A dramatic difference in compressive behaviour is evident between the 5% cement sample and the higher ratios at all curing times explored. The slope of the $(e, \log s_v)$ plot for 5% cement is much greater than the same plot for 10 and 15% cement additions. This suggests that this sample was much more compressible than the higher cement content samples. The difference in slope became less extreme at higher curing durations, although still significant.

For the 10 and 15% cement samples, behaviour was very similar. A very low $(e, \log s_v)$ slope is observed for stresses up to 800kPa, indicating considerable compressibility improvement compared to untreated samples. It was observed that the samples treated with 10 and 15% cement displayed largely incompressible behaviour up to an applied load of 750 kPa. It was shown that curing time had very little influence on the rate at which it compressed. The addition of 5% cement did not improve the preconsolidation pressure of this clay.

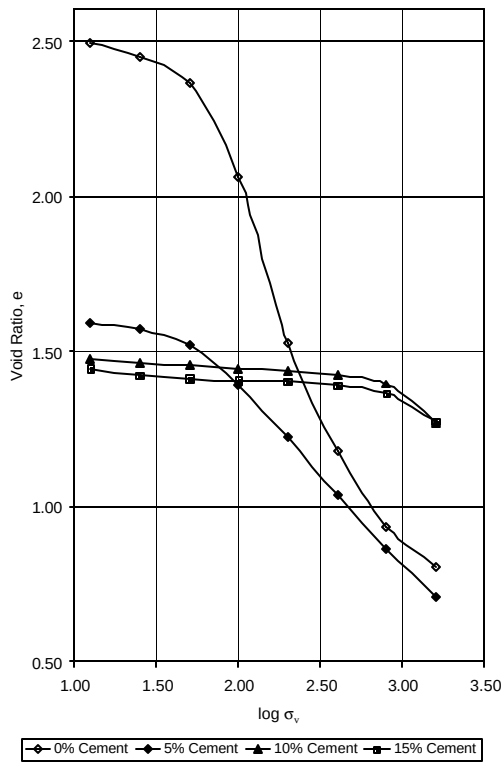


Figure 6(a) Void ratio (e) and $\log s_v$ relations of treated and untreated clay in oedometer test (7 days curing time)

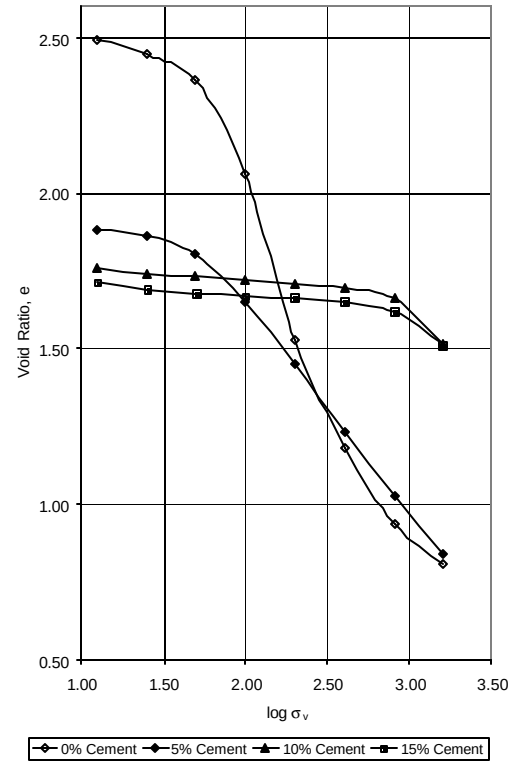


Figure 6(b) Void ratio (e) and $\log s_v$ relations of treated and untreated clay in oedometer test (14 days curing time)

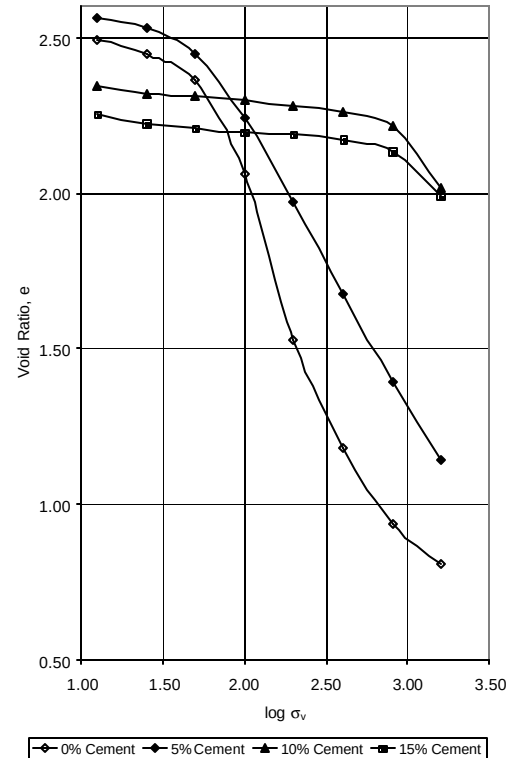


Figure 6(c) Void ratio (e) and $\log s_v$ relations of treated and untreated clay in oedometer test (28 days curing time)

CONCLUSIONS

Ground improvement techniques developed for roads, expressways and other infra-structure developments are now widely used in particular for embankments on soft clays. This paper summarizes some ground improvement techniques used in Southeast Queensland, and demonstrated the applicability of cement stabilisation.

Unconfined compression tests showed that 5% cement additions cured for 7 days produced dramatic increases in unconfined compressive strength when compared to untreated samples, from 27 to 482 kPa. Further cement addition and longer curing duration gave gradual strength benefits, with 15% cement samples cured for 28 days yielding an unconfined compressive strength of 631 kPa. Cement addition was found to increase the preconsolidation pressure and reduce the compression index of the local clay at ratios of 10 and 15%, while it was observed that 5% cement addition was ineffective in these terms. Preconsolidation pressures of 700 - 800 kPa were common for samples of 10 and 15% cement cured for 7 - 28 days. The average compression index of untreated clay over the 50 - 800 kPa stress range was shown to improve dramatically with 10% cement, from over 1.18 to less than 0.10.

Overall it was concluded that the addition of cement has favourable effects on the strength and compressibility characteristics of Southeast Queensland soft clays.

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REFERENCES

1. Wijeyakulasuriya, V., Hobbs, G. & Brandon, A. (1999). Some experiences with performance monitoring of embankments on soft clays. Proc. 8th Australia New Zealand Conference on Geomechanics, 783-788.
2. Saitoh, S., Suzuki, Y., & Shirai, K. (1985). Hardening of soil improvement by deep mixing method. Proc. 11th International Conference on Soil Mechanics and Foundation Engineering, Helsinki, Finland. 947-950.
3. Bergado, D.T., Anderson, L.R., Miura, N. & Balasubramaniam, A.S. (1996). Soft Ground Improvement in Lowland and Other Environments. American Society of Civil Engineers, USA. 234-304.
4. Fahoum, K., Aggour, M.S., Amini, F. (1996). Dynamic properties of cohesive soils treated with lime. Journal of Geotechnical Engineering, ASCE, 122(5), 382-389.
5. Porbaha, A. (1998). State of the art in deep mixing technology: Part I. Basic concepts and overview. Ground Improvement, 2(4), 81-92.
6. Bergado, D. T., Ruenkrirergsa, T., Taesiri, Y. and Balasubramaniam, A. S. (1999). Deep Soil Mixing Used to Reduce Embankment Settlement, Ground Improvement, 3(2), 145-162.
7. Holm, G. (1999). Applications of dry mix methods for deep soil stabilizations. In Bredenberg, H., Holm, G. & Broms, B.B. (Eds). Deep Mix Methods for Deep Soil Stabilization. Balkema, Rotterdam. 3-14.
8. Hoyos, L. R., Puppala A. J., and Chainuwat, P. (2004): Dynamic Properties of Chemically Stabilised Sulfate Rich Clay, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 130(2), 153-162.
9. Austroads (1998). Guide to Stabilisation in Roadworks. Austroads, Sydney, 11-37.