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Geotechnical Advances and Challenges in Urban Development

PROCEEDINGS OF THE 2020 ONLINE AGS GOLDEN SYMPOSIUM

HELD

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November 13th 2020

Organising Committee: A. Parsa, C. Rujikiatkamjorn, S. Mirlatifi, H. Khabbaz, A. Hulskamp, AHMK Zaman, P. Rajarathnam, K. Kianfar

Editors: H. Khabbaz, A. Parsa, C. Rujikiatkamjorn and AHMK. Zaman

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PROGRAMME:

08:45 - 09:00
Opening Address

SESSION 1 - GEOTECHNICAL INSIGHTS INTO DEEP EXCAVATION IN URBAN AREA

09:00 - 09:35
Prof Robert Mair
Emeritus Professor at University of Cambridge
Geotechnical Insights into Circular Shaft Performance in Clay

09:35 - 09:55
Paul Hewitt
Technical Executive at WSP
Ground Response Due to Deep Excavations in Sydney Sandstone

09:55 - 10:15
Peter Sun
Geotechnical Engineer at Coffey
Use of 3D FEM Model to Assess the Impact of Deep Excavation on Existing Sydney Water Tank Stream

10:15 - 10:35
Q&A

10:35 - 11:00
Informative & Promotional Videos

SESSION 2 - ADVANCES AND CHALLENGES OF TUNNELING IN URBAN AREA

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Philippe Vincent
Managing Director at Menard Oceania
Cross Passages Ground Treatment for Melbourne Metro Tunnel Project

11:20 - 11:40
Helen Baxter-Crawford
Principal Engineering Geologist at SMEC
Why Aesthetics Needs Early Consideration in Tunnel Design and Excavation – A Case Study from the M8

11:40 - 12:00
Keith W.K. Kong
Geotechnical Principal at Coffey
How Wide Can a Rock Cavern be Opened?

12:00 - 12:20
Q&A

12:20 - 13:00
Informative & Promotional Videos

SESSION 3 - GEOTECHNICAL CHALLENGES IN TEMPORARY WORKS AND MAINTENANCE OF URBAN STRUCTURES

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Dr Chris Haberfield
Principle Geotechnical Engineer at Golder
Temporary Support of Deep Basement Excavations in Rock

13:35 - 13:55
Idy Li
Associate Principal at EIC
The Bridge Between Contractors and Consultants

13:55 - 14:15
Bosco Poon
Technical Director at GHD
Short and Cumulative Long Term Impacts of Subterranean Cut-off Walls and Excavation Dewatering on Adjacent Structures in Urbanised Areas

14:15 - 14:35
Antonio Ramirez Martinez
Design Manager Southern Region at RECO
Strengthening Reinforced Earth Walls for Increased Road Traffic in Urban Areas – Knox Road Duplication Case Study

14:35 - 14:55
Q&A

14:55 - 15:20
Informative & Promotional Videos

SESSION 4 - SAFETY AND SUSTAINABILITY IN GEOTECHNICAL ENGINEERING

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Theva Muttuvel
Principal Geotechnical Engineer at SMEC
Internal Compression of Fill Material Originating from Bringelly Shale

15:40 - 16:00
David Lees
Principal Engineer at Jacobs
Permeation Grouting in Sydney

16:00 - 16:20
Hashan Subasinghe
Geotechnical Engineer at Advisian (Worely)
Design of a Pipeline Protection Structure Over Compressible Ground

16:20 - 16:40
Paul A. O'Brien
Senior Geotechnical Engineer at Golder
Re-Engineering Old Foundations for A New Structure – Greenland Centre

16:40 - 17:00
Q&A

17:00 - 17:20
Closing remarks and thanks

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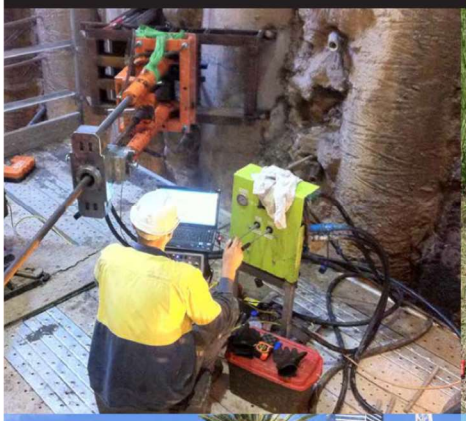

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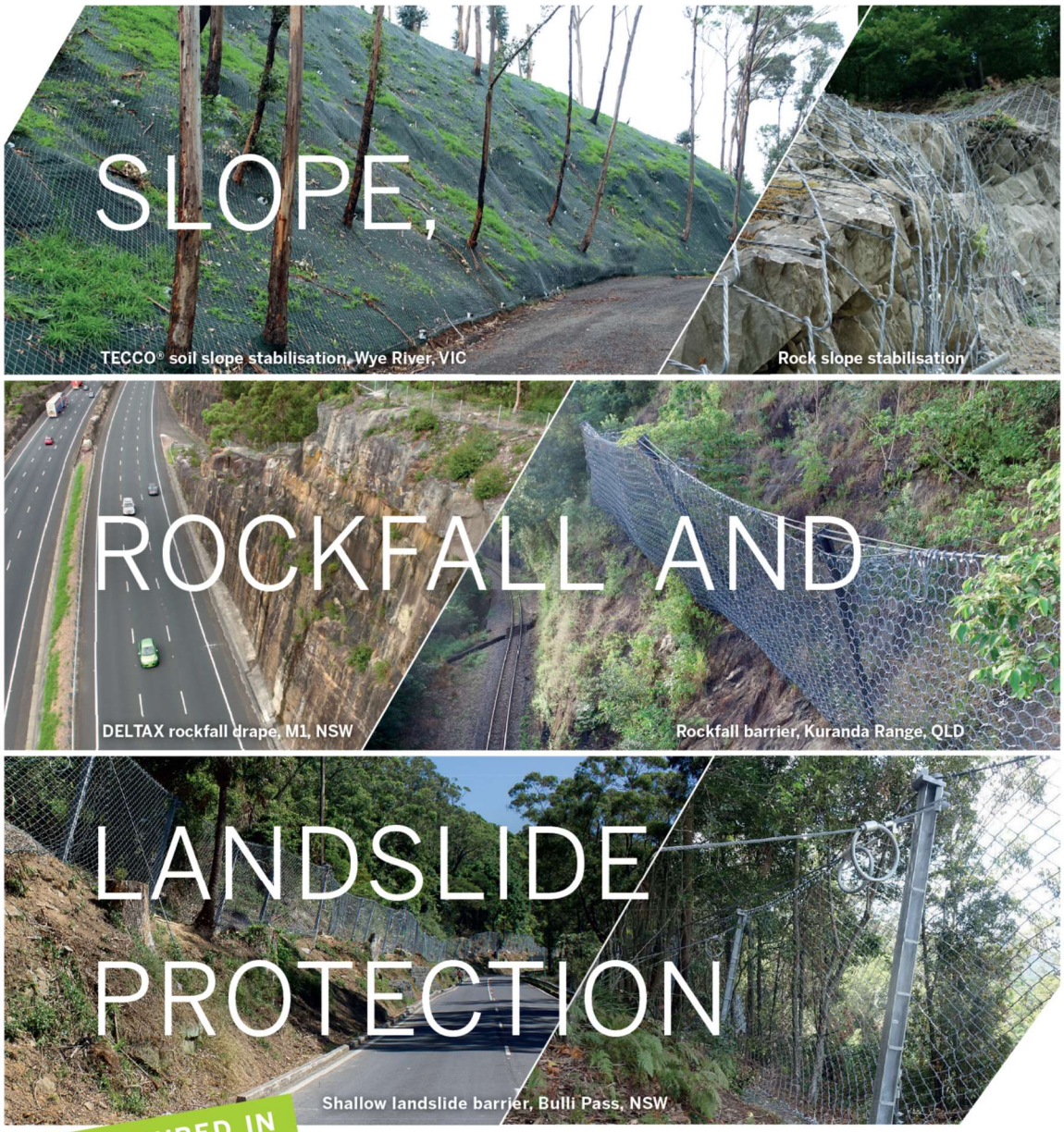

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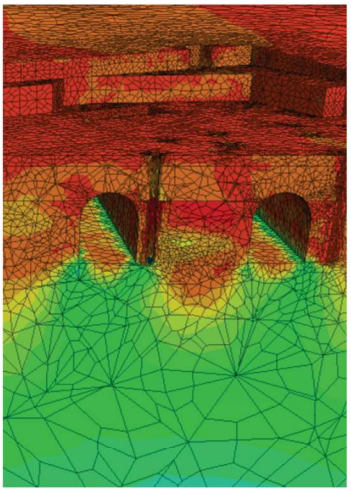

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PREFACE

This document contains the accepted papers submitted and peer reviewed for the 24th annual symposium organised by the Sydney Chapter of the Australian Geomechanics Society (AGS). This year the Australian Geomechanics Society marks its 50th anniversary. During the last five decades the AGS has stood as a beacon of the wider geotechnical community. Due to COVID-19 restriction, it is the first time that the presentations were delivered online to other geotechnical colleagues worldwide. It is hoped that the symposium will keep practicing geotechnical engineers, engineering geologists, and other engineering professionals informed of recent developments in this field. It also recognises the need to gather the experience of those practicing throughout Australia and to allow transfer of knowledge and sharing of their experiences.

These symposia continue to be one of the best forms for bringing together the key stakeholders of the Australian geological and geotechnical community. The main objective of the symposium, held on 13 November 2020, is to showcase state-of-the-art practices, new research findings and case histories that demonstrate geotechnical advances and challenges in urban development. The organising committee invites papers on geotechnical aspects of urban development, underground structures, restoration of geotechnical structures, innovative and cost-efficient solutions, groundwater modelling and application of smart technologies.

This symposium is the cooperative effort of many authors and qualified reviewers. The editors and organising committee wish to thank the authors, who have generously contributed their time to prepare the various papers and the colleagues of the authors, who have assisted with time, secretarial, drafting and other facilities. Appreciation is also extended to our sponsors for their support. Without them, the Symposium would not be possibly the best ongoing forum for the Australian Geomechanics and groundwater community.

Ali Parsa, Cholachat Rujikiatkamjorn, Sam Mirlatifi, Hadi Khabbaz, Adrian Hulskamp, AHMK Zaman, Puvaneswary Rajarathnam, and Kourosh Kianfar

On behalf of the AGS Symposium Organising Committee,
The Australian Geomechanics Society, Sydney Chapter

Annual Seminars of AGS Sydney Chapter

No.	Date	Topic	Chairman & Organising Team
1	1997	Pavement Design Beyond 2000	A Leventhal
2	1998	Recent Developments in Piling Practice in Sydney	P Andrews
3	1999	Flexible Retaining Walls: Design to Prevent Failure	P Andrews and P Hewitt
4	2000	Computer Methods	P Hewitt and J Carter
5	2001	Excavation Retention	T Walker and P Hewitt
6	2002	Landslide Risk Management	B Walker and T Walker
7	2003	Geotechnical Instrumentation and Construction Works Compliance Testing	G Scholey and T Walker
8	2004	The Engineering Geology of the Sydney Region – Revisited	G Scholey, M Parmar, G Young and G McNally
9	2005	Geotechnical Aspects of Tunnelling	H Buys and T Gourlay
10	2006	Soft Ground Engineering	H Buys, R Moyle and P Hewitt
11	2007	Engineering Advances in Earthworks	R Moyle, R Lindbeck and H Liu
12	2008	Foundation Design and Construction	R Moyle, R Lamont and B Ewers
13	2009	Geosynthetics – New Materials for Modern Infrastructure	B Ewers, H Buys and H Liu
14	2010	Seismic Engineering- Design for Management of Geohazards	C Rujikiatkamjorn, J McIlveen, R Lamont and M Haysler
15	2011	Coastal and Marine Geotechnics- Foundations for Trade	C Rujikiatkamjorn, J McIlveen, G Blumberg, J Smith and C Y Tey
16	2012	Advances in Geotechnical Aspects of Roads and Railways	H Khabbaz, C Y Tey, O Stahlhut and C Rujikiatkamjorn
17	2013	Retaining Structures: Recent Advances and Past Experiences	H Khabbaz, C Rujikiatkamjorn, M van Uden, C McColgan and S Mirlatifi
18	2014	Resilient Geotechnics	H Khabbaz, C Rujikiatkamjorn, S Mirlatifi, C McColgan and M van Uden
19	2015	Recent Developments and Experiences with Groundwater and Excavation	C Rujikiatkamjorn, S Mirlatifi, Katarina David, A. Hulskamp, and M van Uden
20	2016	Ground Stabilisation Techniques for Problematic Soils and Unstable Rocks	C Rujikiatkamjorn, S Mirlatifi, A. Hulskamp, T Mcwilliam and H Khabbaz
21	2017	Geotechnical innovations and challenges for transport infrastructure	S Mirlatifi, , C Rujikiatkamjorn, G Vorobieff, D Airey, A Parsa, A Hulskamp, H Khabbaz, H Buys and K Oj,
22	2018	Advances in Site Investigations, Monitoring and Instrumentation	A. Parsa, H. Khabbaz, S. Mirlatifi, C. Rujikiatkamjorn, A. Hulskamp, AHMK Zaman, and P. Rajarathnam
23	2019	Innovations in Geotechnical Construction and Design	A. Parsa, C. Rujikiatkamjorn, S. Mirlatifi, H. Khabbaz, A. Hulskamp, AHMK Zaman, P. Rajarathnam, M. Baringa, and T. Gourlay
24	2020	Geotechnical Advances and Challenges in Urban Development	A. Parsa, C. Rujikiatkamjorn, S. Mirlatifi, H. Khabbaz, A. Hulskamp, AHMK Zaman, P. Rajarathnam, K. Kianfar

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TEMPORARY SUPPORT OF DEEP BASEMENT EXCAVATIONS IN ROCK

C. M. Haberfield[1], A. L. E. Lochaden[1] & D. R. Paul[1]

[1]Golder Associates Pty Ltd, Melbourne, Australia

ABSTRACT

Deep basements have become common in our modern cities. Whilst the analysis and design of retention systems for deep basements in soil are relatively well established, the same cannot be said for deep basement retention systems in rock. In many instances the design of retention systems in rock are based on a soil mechanics approach and as a result often ignore unique aspects of rock mass behaviour that can significantly impact the performance of these retention systems. It is important that the characteristics of the rock mass are well understood and are quantified during the ground investigation. Some of the aspects of retention system design and analysis that are set out in this paper only became apparent following unsatisfactory performance of a retention system. Due to the sensitivity surrounding such unsatisfactory performance, it has not been possible to include the specific case studies. As a result, this paper provides an overview of the many aspects involved in respect to ground investigations, analysis, design and construction of deep basement retention systems in rock that are required to mitigate against unsatisfactory performance. This paper is restricted to temporary embedded wall retention systems that are installed to allow the construction of the permanent basement structure.

1 INTRODUCTION

Deep basements have become common in our modern cities. Height restrictions, local ordinances and building amenity often necessitate that new multilevel buildings include up to several levels of below ground basements for parking.

Basement construction is commonly undertaken by installing piles, diaphragm walls or similar around the edges of the planned excavation prior to the excavation commencing. As the excavation proceeds, rows of post tensioned ground anchors (or in some cases soil nails/rockbolts) or struts are installed to provide support to the piles/diaphragm wall. The ground anchors/struts are the main structural elements that provide temporary support to the piles/diaphragm walls to resist the earth pressures that develop as a result of removal of the ground from the excavation. Once the excavation is completed, the anchors and struts are usually replaced by basement floor slabs which form part of the permanent basement structure. The piles/diaphragm walls may be incorporated into the permanent structure or may be supplemented by permanent structural walls.

For most competent builders, the construction of the temporary basement retention system (hence forth referred to as retention system) is considered a high risk activity and is usually on the critical path for construction. Issues encountered during construction of the retention system can cause significant damage and construction delays and pose a risk to workers and the public. It is therefore paramount that the analysis, design and construction of the retention system are carried out competently and prudently with a full understanding of the ground related hazards that may be present.

Undertaking a basement excavation modifies a site from a usually stable situation into a potentially unstable one. The removal of the ground to form the excavation removes the support to the adjoining land and provides a space for which the adjoining ground can move or collapse into. Although not considered in this paper, the excavation can also locally change the groundwater regime which can also cause significant issues.

To limit movements and prevent collapse of a deep vertically sided excavation a retention system is required. The primary safety requirement of a retention system is that it prevents the walls of the excavation from collapsing. However, also of prime importance is that the retention system limits movement of the excavation walls and the surrounding land. This movement occurs as a direct result of the reduction of in situ stresses in the ground due to the excavation. Whilst the change in situ horizontal stresses is usually of most concern, as discussed below the reduction in in situ vertical stress can also be of significance. In general, the stiffer and more robust the retention system, the lower will be the reduction in in situ horizontal stress and hence the lower the ground movement due to the excavation.

A difficult design decision is often associated with what magnitude of movement is reasonable, and what is the lowest movement that is practically and reasonably achievable. This has a direct impact on the stiffness, robustness and cost of the retention system. A retention system that is too "light" could potentially result in unsatisfactory movement and damage to adjoining assets and hence significant cost implications with possible significant delays during construction. One that is too "heavy" has significant cost and program implications. The aim is to achieve a retention system that strikes a balance between these two extremes but includes a reasonable level of prudency.

Whilst the analysis and design of retention systems for deep basements in soil are relatively well established, the same cannot be said for retention systems in rock. For this reason, many designers use a soil mechanics approach to analyse and design retention systems in rock and as a result often ignore unique aspects of rock mass behaviour that can significantly impact the performance of a retention system in rock. It is important that the characteristics of the rock mass

are well understood and are quantified during the ground investigation. This paper sets out the authors' experience in respect to ground investigations, analysis, design and construction of deep basement retention systems in rock.

2 CRITICAL GROUND PROPERTIES FOR DESIGN

The primary aim of any temporary retention system is that it must be able to adequately resist the horizontal pressures which act or will act on the retention system as a result of removal of the ground within the excavation. These horizontal pressures include the in situ horizontal stress within the ground, ground water pressure and pressures due to nearby surcharges. Earthquake loads need to be considered for the permanent basement structure, but not usually for the temporary works.

The maximum change in in situ horizontal stress that can occur for a stable retention system is from the at rest state, K_0, to the active pressure state, K_a. It follows that in order to prevent collapse, the retention system must, as a minimum, be strong enough to resist the active (K_a) earth pressure as well as any other applied loads (due to groundwater and surcharges). The strength of the ground (which defines K_a), the groundwater conditions and applied surcharges are therefore key parameters in the design of a retention system.

However, a further key requirement is that a retention system also limit ground movements to tolerable levels. In the simplest case where there are no groundwater pressures or surcharges, horizontal movements occur if the horizontal stresses imposed by the retention system to the walls of the excavation are either less than or greater than K_0, with greater movements occurring the greater deviation from K_0. Vertical movements of the adjacent ground primarily occur as a result of the horizontal movements (resulting in settlement) but to a lesser extent also from the removal of vertical stress from within the excavation (resulting in heave).

Movements of the retention system walls towards the excavation are accompanied by a reduction in in situ stress from K_0 towards K_a (noting that for excavations that are self-supporting, $K_a = 0$). Greater than expected movements usually result from insufficient stiffness of the retention system (wall, membrane and supporting struts or anchors) and/or insufficient rows and/or insufficient post-tension force applied to anchors or struts (to maintain near K_0 conditions). Such inadequacy can occur in the design of the retention system, poorly constructed anchors or by not staging the construction works appropriately.

Movement of the retention system walls away from the excavation (i.e. in the passive direction) results in an increase in the in situ stress above K_0. This usually only occurs close to the ground surface in soft clay and is usually due to the design specifying excessive post-tensioning force in the upper row of anchors or struts.

The amount of movement that occurs depends on the stress change that occurs and on the stiffness of the ground. It follows that K_0 and ground stiffness are also key parameters for any retention system design.

An appropriate ground investigation therefore must be able to not only identify the subsurface stratigraphy and ground water regime but must also allow the strength, stiffness and at rest pressures in the ground to be reasonably quantified.

3 RETENTION SYSTEMS IN SOIL

The analysis and design of retention systems in soil are relatively well understood with comprehensive guidance for designers being provided in CIRIA C760 "Guidance on embedded retaining wall design". Key inputs into the analysis and design include soil strength and stiffness, at rest in situ horizontal stresses, ground water conditions (and wall strength and stiffness).

Appropriate ground investigations in soil allow a suitable geotechnical model to be developed which identifies the thickness and general characteristics of each soil unit within the geotechnical model. Each soil unit is generally assumed to be homogeneous and isotropic with no difference between intact and mass properties. Strength and stiffness properties are usually assessed directly from small laboratory samples, indirectly from in situ testing or from experience. Soil structure if it is present is usually included in the assessment of the strength and stiffness parameters (e.g. a lower strength may be assumed if a clay is fissured).

K_0 is rarely measured. However, a prudent estimate of K_0 in soils can usually be made based on knowledge of the soil stress history and the strength and stiffness of the soil. By definition, K_0 must be between the two failure states (active) K_a and (passive) K_p, and generally in most near surface soil deposits K_0 is much closer to K_a than to K_p. It is rare for soils within the depth range of most retention systems to have locked in horizontal in situ stresses close to K_p as the usual soil formation processes do not allow stresses close to K_p to develop. Even if they do develop, the low stiffness of the soil allows elevated stresses to dissipate. Nevertheless, because the strength of the soil is relatively low, the in situ horizontal stress in most near surface soil strata is also generally relatively low. That is, the in situ horizontal stress in soil that a retention system needs to support is usually relatively low.

Another aspect of retention systems in soil is that the retention system elements (piles, walls, anchors, struts) are generally much stiffer (and stronger) than the soil. As a result, it is usually practical to design and build a retention system in soil to limit movements to tolerable levels.

4 CHARACTERISTICS OF ROCK MASS BEHAVIOUR

Many reference books and publications such as CIRIA C760 provide the key aspects of analysis, design and performance of retention systems in soil. Most, if not all of these key aspects, are also applicable to retention systems in rock and as such, these publications provide a fundamental framework for the analysis and design of retention systems in rock. However, there are some important and significant characteristics of rock masses that are not usually considered when designing retention systems in soil. These are summarised below.

4.1 ROCK FORMATION PROCESS AND IMPACT ON ROCK MASS CHARACTERISTICS

All rock masses comprise planes of weakness or discontinuities (e.g. bedding, joints, faults, fractures etc.), referred to as rock structure. The structure and in situ stresses present within a rock mass are dependent on their formation process and geological history. For example, basalt which forms from the cooling of lava is prone to shrink through the formation process creating persistent vertical joints or columns with typically low horizontal in situ stress, whereas sedimentary rocks, which form through layering and consolidation of sediment, are more likely to have persistent horizontal bedding planes with higher in situ stresses. The stresses in the rock mass can then change through tectonic activity subsequent to emplacement of the rock. This can result in compression or extension of the rock mass leading to the development of further structure, including joints, folding and faulting, which develops in orientations influenced by the direction of the stresses applied.

Regional tectonic compression of a rock mass typically results in a stress anisotropy, with high horizontal stress in one direction and a lower horizontal stress in the orthogonal direction. These stresses typically result in deformation within the rock mass ranging from folds and shears through to faulting and jointing depending on the ductility of the rock mass. In brittle rock masses, reverse or thrust faults, which are essentially a failure of the rock mass once its passive resistance is exceeded can form with strike perpendicular to the direction of the principal stress. Similarly, normal faults and shears can form at an orientation perpendicular to the minor in situ stresses. The high displacement which occurs along the fault planes or slip between joints and bedding can serve to reduce strength along discontinuities to residual levels (e.g. slickensided bedding planes).

The in situ stresses, strength and stiffness characteristics of discontinuities are further altered by weathering, erosion and anthropogenic changes (human impact) such as excavation (e.g. mining).

4.2 ROCK STRUCTURE

The presence of discontinuities within a rock mass not only reduces the mass strength and stiffness of the rock significantly below that of the intact rock, but also introduces significant strength and stiffness anisotropy into the rock mass, with the strength and stiffness of the rock mass varying in magnitude and orientation depending on the characteristics of the discontinuities, including their strength, spacing, persistence, orientation, roughness and infill and the scale being considered. For example, the strength and stiffness of a rock mass with widely spaced discontinuities will be significantly greater for a 2 m deep excavation (due to the very blocky nature of the rock mass at this scale) than for a 20 m deep excavation (where the rock block size is small compared to the overall depth of the excavation).

There is often large variation in the discontinuity properties within the same rock mass. For example, bedding planes which tend to be persistent over tens if not hundreds of metres in sedimentary rock masses can have strengths varying from the intact rock strength to a friction angle of less than 10° on smooth, slickensided surfaces. Similarly, the spacing, aperture and infilling of joints can vary significantly with location and orientation and as a consequence, the stiffness of the rock mass also varies significantly.

This makes assessment of rock properties for the analysis of retention systems a difficult and imperfect task. The orientation, location, persistence and type of discontinuities within a rock mass will have different impacts on a retention system depending on the orientation (strike) of the retention system relative to the major discontinuities. For example, in a rock mass where the strike of the bedding is north-south and persistent bedding dips to the west, then the bedding is unfavourable and favourable for the east and west faces, respectively, of a retained excavation. However, whilst the orientation of the bedding can be identified from an appropriate geotechnical investigation, the location of the weakest bedding planes within the rock mass, and which have the greatest impact on the design and performance of the retention system, cannot usually be reasonably known. As a result, a designer must consider a range of potential defect locations and strengths that are critical to the performance of the retention system.

For example, in Melbourne Australia, the siltstone which forms the bedrock comprises siltstone with interbedded fine grained sandstone. Bedding planes can be persistent over hundreds of metres and with strengths that vary locally on the same bedding plane from relatively high (essentially intact rock strength) to very low (about 12° on slickensided planes). A relatively large number of laboratory direct shear tests have been undertaken on bedding plane joints in the siltstone over a number of years. The results of these tests generally indicate friction angles in excess of 35° with a reasonable lower bound value of about 23° which is similar to the residual strength of the rock. On rare occasions, testing has indicated friction angles as low as 12° for slickensided bedding planes. Such slickensides are relatively common but are generally restricted in their persistence. That is, the areas of slickensiding are generally not extensive, and hence in most situations do not significantly impact the performance of a retention system. However, on at least three occasions that the authors are aware of, the presence of an extensive slickensided bedding with a persistence of over at least 50 m resulted in either a batter collapse or excessive movement of a retention system requiring emergency remedial action.

One example where this problem has arisen was for an anchored soldier pile and shotcrete retention system constructed in a siltstone rock mass with a locally elevated groundwater table (due to a leaking service) and containing persistent bedding striking approximately parallel to the east excavation face and dipping out of the face at about 25°. One of the bedding planes (out of many hundreds) near to the base of the excavation (see Figure 1) was slickensided over a significant length (in excess of 50 m).

Figure 1: Photograph showing a slickensided bedding plane which caused movement of the retention system of the east wall of the excavation in excess of design

Direct shear tests on large samples obtained by coring into the rock mass at different locations along the sliding bedding plane shown in Figure 1 (with the bedding aligned along the axis of the core) indicated shear strengths ranging from 12.5° to almost intact rock strength. One test conducted on a sample obtained from the north wall of the excavation indicated a relatively high initial peak strength and then a very rapid reduction to a friction angle of 18°. This very brittle behaviour is shown in Figure 2 (results of a direct shear test under constant normal stiffness conditions with initial normal stress of 370 kPa and normal stiffness of 50 kN/mm). The very brittle behaviour makes the assessment of the appropriate bedding strength to be adopted for design difficult.

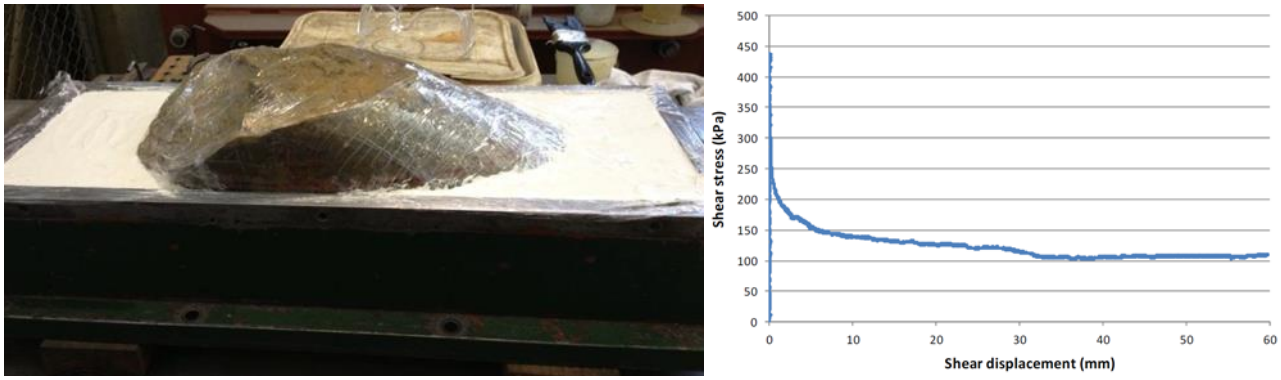

Figure 2: Photograph showing sample secured in bottom half of split shear box with plaster (left) and direct shear response showing very brittle behaviour (right)

In addition to the persistent weak bedding plane, an (unknown) sub-vertical weak structure was present within the rock mass at the northern end of the site which acted as a release plane allowing a large wedge of rock to rotate out of the face as shown in Figure 3. The locally elevated groundwater table and presence of a persistent slickensided bedding plane and sub-vertical weak structure were not known at the time of design and was considered in design.

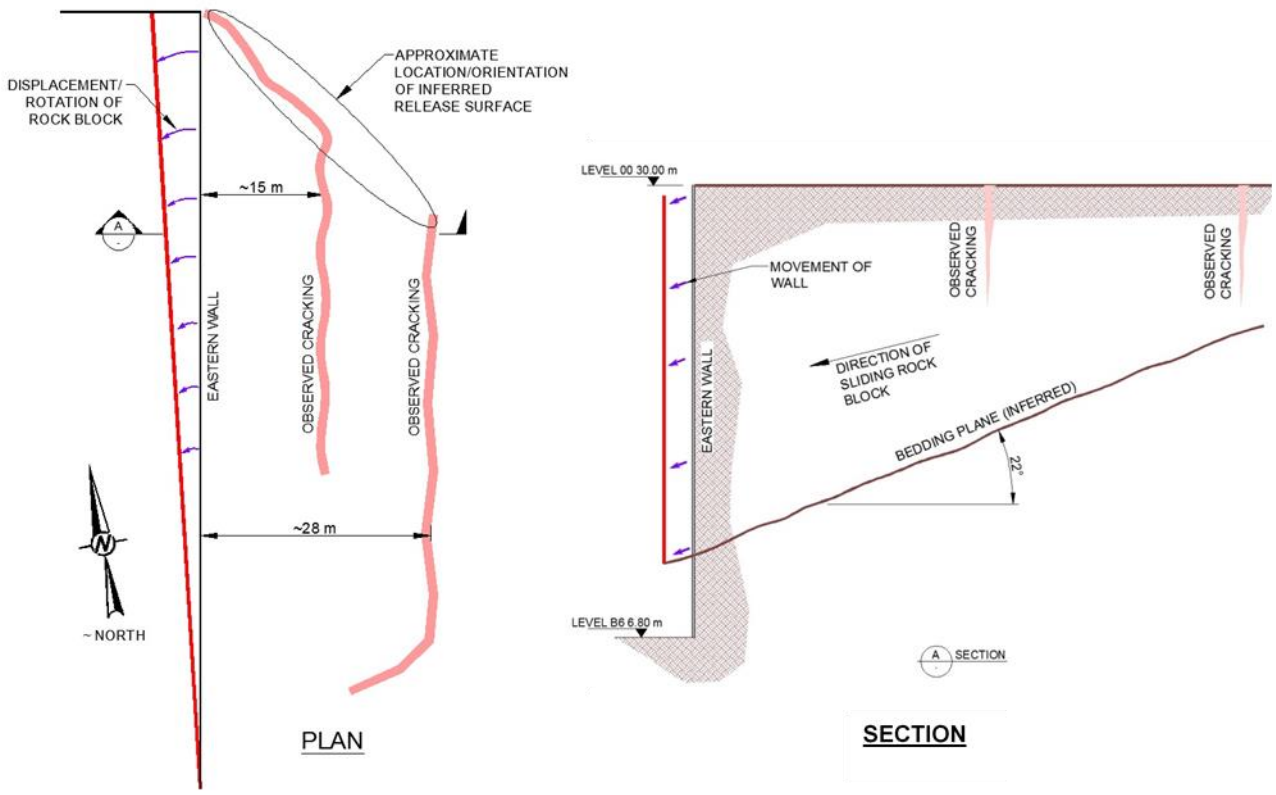

Figure 3: Mechanism of movement of east wall (plan, left and section, right)

The excavation at this site progressed without issue until the excavation was almost at full depth. The first sign of an issue was cracking and spalling of shotcrete between soldier piles at the very northern end of the east wall. Initially, this was difficult to explain as such damage would usually occur in the central portion of the wall. However, subsequent investigation indicated the spalling first occurred at the location of a subvertical release feature within the rock mass behind the north east corner of the excavation (see Figure 3). The first observation that a significant rock wedge had formed was provided by the soldier piles at the southern end of the east wall. Below a depth of about 20 m the rock was no longer in contact with the soldier piles, but above this depth the rock mass was in contact with the soldier piles. Closer inspection revealed the bedding plane on which sliding was occurring and this could be traced for at least 50 m to the north whereupon it was covered by shotcrete. As displacement monitoring was only being undertaken on a fortnightly basis there was no early warning of movement. Subsequent movement monitoring of the whole east face indicated that the wedge was moving as a block into the site. Emergency stabilisation measures (backfilling against the east face followed by installation of sub-horizontal bores to lower groundwater levels and installation of additional post tensioned rock anchors) were immediately employed. Movement of the rock wedge continued to occur until sufficient backfilling had occurred (where upon the east wall had displaced horizontally about 100 mm) and was not completely arrested until the additional rows of rock anchors were installed.

The above example focussed on bedding, which is often the most persistent and critical with respect to design of a retention system. However, other discontinuities of lower persistence (e.g. joints) can also have an impact. The orientation, spacing and persistence of such features relative to an excavation face defines the dimensions and geometry of rock wedges that are present and hence can be used to estimate the potential localised forces that need to be carried by the retention system (e.g. as may need to be carried by shotcrete between soldier piles, or for a larger wedge encompassing say a few piles only). Typically, the form of the rock wedges will change from one face of the retention system to another.

As a result of the rock structure the overall rock mass strength and stiffness is both anisotropic and scale dependent.

4.3 IN SITU HORIZONTAL STRESSES

The in situ horizontal stress present in rock masses is commonly developed through tectonic activity and remains locked in until excavation, weathering or erosion causes a change in the magnitude or orientation in the in situ stress field. In situ horizontal stresses can vary between active and passive pressures depending on the stress history of the rock mass. Faults, shear zones, erosion and weathering can all modify and lead to inhomogeneity of the stress field such that whilst there may be a consistent stress magnitude and orientation at the regional scale, at a local or project scale the in situ stress may vary, in some cases significantly from the regional stresses.

The highest in situ horizontal stresses generally occur in compressional tectonic settings and can be close to the passive resistance of the rock mass. The highest stresses will typically be present in the strongest and stiffest horizons within the rock mass. In a sedimentary rock mass for example comprised of siltstone and sandstone, the more competent (and higher stiffness) sandstone may attract higher stresses. More weathered ground might be encountered at shallow depth and may attract lower stresses, or there may be modification to the stress field around existing excavations. All of these factors can serve to produce local stress conditions different from regional conditions.

Undertaking excavations in areas with high in situ horizontal stresses can be problematic – not because of instability of the walls of the excavation which may be self-supporting, but because of the relatively large (and "stable") movements that can occur due to the relaxation in in situ horizontal stress caused by undertaking the excavation.

For example, the authors recently had experience with the excessive movement of one wall of a deep basement excavation in medium to high strength metamorphosed rock due to unexpectantly high in situ horizontal stresses. The retention system comprised anchored soldier piles supporting the upper weaker soil and rock units and pattern rock bolts below. The rock mass was blocky in nature with relatively high Geotechnical Strength Index (GSI, Hoek *et al*., 2013). The foliation with the rock mass, whilst reasonably persistent, was favourable for the excavation face that suffered the unexpected movement. Like the previous example above the works were progressing in accordance with the design performance until the excavation encountered high strength rock below about 12 m depth. Automatic hourly displacement monitoring of the walls of the excavation indicated very low horizontal and vertical movements up until the excavation extended below the toe of the soldier piles into medium strength rock where upon increased movements of the retention system, but still in accordance with design, occurred. As the excavation extended into higher strength rock the lateral and vertical movements of the retention system significantly increased. At the completion of the excavation to full depth the wall had moved approximately 125 mm horizontally with settlement behind the wall of up to 90 mm.

After the onset of the increased movements, it was thought that the movement may be due, at least in part, to the formation of a large unstable rock wedge. As a result, back filling was undertaken against the face, and on the recommendations of a review consultant, additional rows of rock anchors were installed over a portion of the face already excavated and over the full length of the face as excavation proceeded. Analysis undertaken to design the additional support indicated that if an unstable rock wedge was the mechanism causing the movements, then additional movements following the installation of the rock anchors should be minimal and a factor of safety of 2 was achieved on wedge sliding.

However, although we considered these remedial measures to be prudent, it was not clear that an unstable rock wedge had formed or was the cause of the movements. The movements that had occurred and our observations of the site were inconsistent with this mechanism, The movements occurred relatively uniformly over the full length (in excess of 100 m) and excavated depth of the wall and effectively ceased soon after that excavation flitch was complete. There was no evidence of cracking or distress in the shotcrete or exposed rock faces and no significant damage was observed in existing buildings and assets located immediately adjacent to the top of excavation (although these had been subjected to significant movements). Lift off testing of anchors indicated anchor loads that were not consistent with a wedge mechanism. In addition, subsequent survey monitoring of the area around the site indicated a relatively widespread movement consistent with those due to stress relief.

Through further investigation and due to previously unavailable information becoming available, it became apparent that very high in situ horizontal stresses were present within the rock mass in this region. Further evidence of high in situ stresses being the cause of the movement is obtained from the movement that continued to occur following the installation of the remedial anchors. In fact, movements continued at an increased rate with depth with each excavation flitch, which

was not consistent with the installation of the additional rows of anchors and the wedge hypothesis. The additional anchors appeared to have little if any impact in reducing the movements.

In ground conditions with relatively high in situ horizontal stresses, the anchor/strut forces required to limit lateral movement of the retention system can be impractical to achieve. For example, to balance an in situ horizontal stress of 1 MPa would require anchors on a 2.5 m x 2.5 m grid spacing each post tensioned to 6.25 MN. Under such circumstances, an option may be to accept that large "stable" horizontal movements are likely to occur and to design the retention system only to prevent instability rather than movement.

In ground conditions where the at rest pressure is close to K_p, any excavation will reduce the vertical effective stress acting on the base of the excavation and this may be sufficient to initiate passive failure of the floor of the excavation. Whilst such failure may not be directly observed during the works because of the ongoing excavation works (which removes failed material), it can result in high lateral movement of the otherwise stable walls of the excavation.

4.4 ROCK MASS PERMEABILITY

Most intact rock is relatively impermeable compared to the rock mass which is dissected with open fractures and discontinuities. In rock masses the groundwater flows are primarily restricted to the open fractures and discontinuities. As a result, only a relatively small volume of water is required to completely fill fractures. Groundwater levels can therefore rise quickly in rock with consequence for retention system design. In such instances, installation of sub-horizontal drains drilled into the excavated face to intersect primary water bearing fractures and features can significantly reduce the potential for adverse groundwater level rise impacting on the retention system.

5 CRITICAL GROUND PROPERTIES FOR DESIGN

As set out above there are a number of key geotechnical inputs that need to be quantified to enable a reasonable and prudent analysis and design of a retention system to be undertaken. These include the subsurface stratigraphy both within and outside the site, the characteristics (orientation, spacing, persistence) of the discontinuities within the rock mass, the strength of the discontinuities (especially bedding if present), the rock mass strength and stiffness, the ground water regime and the in situ horizontal stresses present within the rock mass.

A necessary starting point is to understand the geological and geomorphological history of site, the anthropogenic activity and its regional geological setting. An understanding of this information provides important insights into the likely subsurface conditions including the potential variability, the presence of significant geological features such as paleo channels, dykes, faults and folds and other rock mass properties including jointing patterns, weathering profile, the magnitude and direction of in situ horizontal stresses and groundwater conditions (including the potential for contamination plumes). A conceptual ground model would be developed at this point.

The scope of the geotechnical investigation can then be developed based on past experience in the vicinity of the site and the insights obtained from developing a conceptual ground model as set out above. Typically, an intrusive geotechnical investigation would be undertaken with boreholes drilled within the site and/or just outside the site boundaries. The number and depth of boreholes should be sufficient to reasonably and confidently define the subsurface geology (including different geological units and horizons of weaker, stronger and more weathered or fractured materials within each unit) and the groundwater regime. If there are known major geological features (e.g. buried paleo channels, major faults, dykes or shear zones) then these need to be investigated in sufficient detail to define their location, extent, geometry and material characteristics.

High quality drilling (e.g. triple tube core drilling) is required so that accurate information on rock mass conditions can be assessed without having to allow for artificial breaks in the core caused by the drilling. Even though auger drilling is often practical in very weak rock, it is preferable to commence coring as soon as is practical to obtain reasonable samples in the weak upper rock materials. Comprehensive borehole logs recorded by experienced and capable personnel and containing detailed information on discontinuities and other characteristics of the rock is essential. Auger drilling in rock provides little if any useful information and should be discouraged.

In rock masses for which the rock formation process indicates that persistent and pervasive discontinuities may be present within the rock mass (e.g. a bedded sedimentary rock), it is necessary to identify key features that could assist in estimating the shear strength of these discontinuities including persistence, orientation, spacing, infill and aperture, noting that these

will vary through the rock mass and sufficient data need to be obtained to assess the variability and to estimate reasonable and prudent characteristic values. Orientation and spacing information can be obtained from nearby rock exposures and by undertaking borehole imaging, but noting that as boreholes are generally vertical, discontinuity information from borehole imagery is unlikely to provide a reasonable representation of sub vertical discontinuities or to provide an indication of the persistence of the discontinuities. In some instances, it may be prudent to utilise inclined boreholes, especially when delineating major geological structures.

Although the strength of discontinuities is critical to the analysis and design of a retention system, it is rare, and often of little value to undertake strength testing of discontinuities. This is due to the usually wide variation in discontinuity strength at the scale samples are tested. It is usually sufficient for such information to be assessed from visual observation of the discontinuities in the core and past experience. Careful observation of the core is therefore required. Some slickensided bedding planes can be very thin (< 1 mm) and difficult to observe in the core (see Figure 4), and yet it only takes one such plane to be present within the excavation depth, be persistent, with an unfavourable orientation to cause significant stability issues for a retention system. However, it is not reasonably possible to assess the extent and persistence of such bedding planes from rock core, leaving the designer with a difficult decision as to what to include in the analysis of the retention system. This is where combining borehole indications with a thorough understanding of the geological setting is vital.

Figure 4: Photograph of section of core which contained a slickensided bedding plane which caused movement of a retention system shown in Figure 1

Recovered rock core should be tested to measure the uniaxial compressive strength (UCS) of the rock. This together with information on the rock structure and application of a suitable rock mass classification system (e.g. GSI, Hoek *et al*., 2013) is usually sufficient to provide a reasonable estimate of rock mass strength and a likely range of rock mass stiffness values for design, especially for higher strength rocks. Such approaches must be used with caution in weaker rock masses with GSI less than about 30. For these weaker and more fractured rock masses, additional down hole testing using a high pressure pressuremeter is recommended to provide a better estimate of rock mass stiffness. There is usually little value in undertaking pressuremeter tests in higher strength (e.g. UCS greater than 10 MPa unless the rock mass is highly fractured) massive rock masses as the stiffness of the rock measured in the test is likely to be greater than can be reasonably measured by the instrument and is usually of sufficient magnitude that it has little if any impact on the results of an analysis of the retention system.

In geological environments where in situ stresses are likely to be elevated, it may be prudent to undertake in situ stress measurements using borehole instruments that may utilise for example hydofracturing or stress relief methods. It is important however to correlate the in situ stress measurements with the regional geology and the mass stiffness of the rock at the location the measurements have been taken. This is because the processes through which the in situ stresses develop in the rock mass result in the stiffer horizons having a higher horizontal stress than the horizons with lower mass stiffness (even though they may be adjacent to one another). In situ stress distribution is also affected by topography and major structures such as faults.

Even in seemingly benign in situ stress environments, horizontal in situ stresses can be significantly higher than expected or indeed used traditionally in design and analysis. For example, for many years it was thought that the horizontal in situ stresses in the siltstone forming the bed rock below Melbourne were relatively low. However, recent stress measurements for a major tunnelling project indicate an anisotropic in situ horizontal stress field with at rest earth pressure K_0 of more than double that generally assumed in the past (e.g. in highly weathered siltstone typical measured values of K_0 are about 0.6 – 2.0 in both the east west and north south directions whereas corresponding values in slightly weathered rock are 1.5 – >4.0 and 0.75 – >2.0 respectively). The anisotropic in situ stress field is consistent with the regional tectonic setting in which the southern part of the Australasian Plate is subject to east-south east to north-north west tectonic compression.

Another example is in the much stronger rocks of the Neranleigh-Fernvale Beds underlying parts of the CBD of Brisbane. The Neranleigh-Fernvale formation comprises predominantly phyllite with intact strengths of slightly weathered, near surface material in excess of 30 MPa. The tectonic stress history in the area is one of north east to south west compression which has resulted in shallow angle reverse faults striking approximately north west to south east. Recent stress measurements at relatively shallow depth within the depth of deep basements indicate very high horizontal in situ stresses (several MPa) in the north east south west direction with measured K_0 values of 6 to 8 with the high stresses appearing to be related to major fault structures.

6 ANALYSIS

6.1 MODELLING ROCK STRUCTURE

It is clear from the preceding discussion that the rock mass structure needs to be appropriately accounted for in any analysis of retention systems in rock. There are many non-linear numerical modelling analysis software packages available that are capable of modelling the rock mass structure and other aspects of rock masses. These software packages adopt either a continuum or discontinuum approach. The continuum approach (e.g. PLAXIS 2D and 3D) tend to be the most widely used for retention system design. Discontinuum approaches (e.g. UDEC and 3DEC), although better suited to rock mass modelling are not routinely used for retention system design. This may be because it is usually difficult at design stage to adequately quantify the rock mass structure, and modelling of structural elements (e.g. piles and anchors) in such tools may be more limited. In any case, experience has shown that when a continuum approach is used appropriately, it provides a reasonable estimate of retention system performance.

When developing analysis models using a continuum approach, the methodology adopted to simulate the rock structure varies from engineer to engineer, but most methodologies can be categorized into two main types. Either the rock structure is included in an assessment of the mass strength and deformation properties of the siltstone (continuum model) resulting in lower strength and deformation properties, or alternatively less conservative parameters are adopted for the rock mass and a limited number of distinct discontinuity planes are included directly in the analysis model at critical locations (partial discontinuum model).

The available continuum based software packages offer a range of constitutive models suitable for use in both continuum and partial discontinuum approaches. These models range from simple Mohr Coulomb models to hardening models in which different loading and unloading performance can be modelled. All of these models generally assume the rock mass is isotropic with respect to strength and stiffness.

A continuum based constitutive model that allows anisotropic strength and stiffness to be modelled is provided by ubiquitous joint models (referred to as the Jointed Rock model in PLAXIS). This model effectively models the rock mass as containing an infinite number of weak planes in up to three different specified orientations. The strength and stiffness of the rock mass in the specified directions is assumed to be equivalent to that of the discontinuities in those directions. The intact rock is considered to behave elastically.

An appropriate choice of constitutive model and rock properties (including in situ stress) used in the analysis of the retention system is key to obtaining reasonable estimates of performance of the system. For this reason, it is the authors' opinion and experience that such analyses should be undertaken using a constitutive model that captures the important aspects of the rock behaviour along with "prudently conservative best-estimate" properties.

For a continuum model to provide a reasonable estimate of performance, the rock mass must behave as a coherent mass (or continuum) and its performance must not be dominated by planes of weakness. Whether this is realistic or not depends on the scale of the problem relative to the rock structure and the orientation of the rock structure. Hoek and Bray (2004) illustrate this in the example shown in Figure 5.

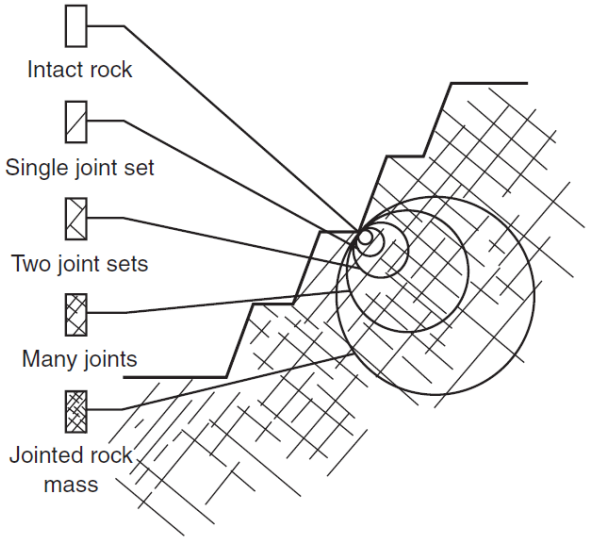

Figure 4.1 Idealized diagram showing transition from intact rock to jointed rock mass with increasing sample size.

Figure 5: Figure 4.1 of Hoek and Bray (2004) showing scale and rock mass behaviour

If the rock mass performance is dominated by one (or more) set of discontinuities then this set of discontinuities needs to be explicitly modelled in the analysis (partial discontinuum approach). Usually for an excavation face it is one set of discontinuities which is critical to the performance of the retention system (e.g. bedding), although other discontinuity sets might provide release planes or impact on the form of the failure wedge.

Haberfield (2017) provides an example of an analysis of a 25 m deep excavation supported by a retention system in a rock mass that contains weak persistent bedding striking north south and dipping at 25^o to the west. A summary of this example is included below.

On the east face of the excavation, the bedding is unfavourable and will dominate the behaviour of the rock mass and hence the bedding needs to be discretely modelled. However, on the north, south and west faces of the excavation, the bedding is either favourable or neutral and does not impact significantly on the performance of the retention system and hence need not be explicitly modelled in the analysis.

The analysis was undertaken using PLAXIS 2D for three different constitutive models, Hoek Brown, Mohr Coulomb and the Jointed Rock model.

The rock mass parameters for the continuum approach adopted for the north, south and west faces were calculated based on an estimate of the Hoek and Brown GSI properties (Hoek *et al*., 2013) of the siltstone for the scale of the rock mass under consideration, i.e. the full 25 m depth of the excavation. Equivalent Mohr-Coulomb parameters for effective rock mass cohesion and friction angle were calculated at a representative vertical stress. For the Hoek-Brown model, the GSI parameters were used directly. The same Mohr-Coulomb properties were used for the jointed rock model as appropriate, along with bedding plane friction angles of 35^o for the Serviceability Limit State (SLS) case and 23^o for the Ultimate Limit State (ULS) case (see below).

For the partial discontinuum approach, potential low strength bedding planes were included in the analysis using interface elements. The positions of the bedding planes were selected to represent the worst possible location with respect to the performance of the retention system i.e. at base of each excavation flitch prior to anchor installation. As for the jointed rock model, the SLS case assumes a bedding plane friction angle of 35^o, whilst the ULS case considered a minimum of three low strength bedding planes with a friction angle of 23^o are present. Figure 6 shows the PLAXIS model used for the critical east face (partial discontinuum model) of the excavation.

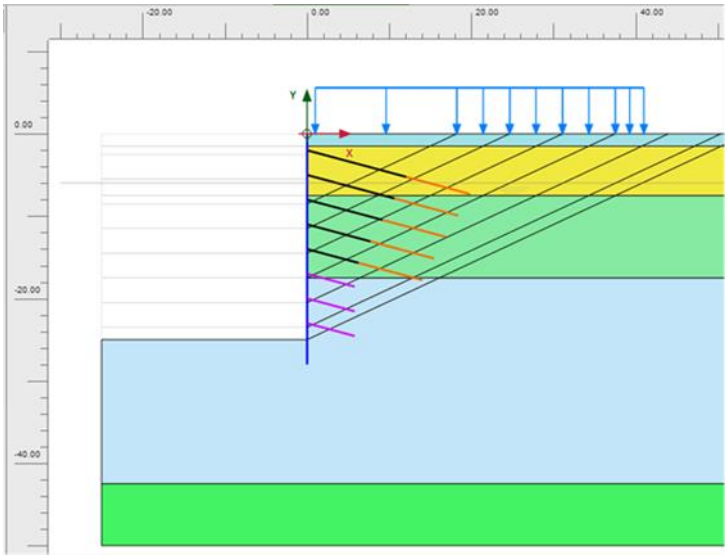

Figure 6: Partial discontinuum PLAXIS 2D model adopted for the critical east face

The Mohr-Coulomb rock mass parameters used in the discontinuum model were calculated based on an estimate of the Hoek and Brown GSI properties (Hoek *et al.*, 2013) of the siltstone for the smaller scale of the rock mass between the low strength bedding planes (i.e. about 5 m). This resulted in higher rock mass strength and deformation parameters than assessed for the Mohr-Coulomb continuum model. All other inputs into the analyses were identical.

As indicated in **Error! Reference source not found.**, the analyses using the different approaches and constitutive models showed significant differences in the results of the analysis in respect to the calculated lateral displacements, pile bending moments and shear forces and total anchor and bolt load. The jointed rock model failed to converge for both SLS and ULS analyses. It was concluded that a significantly more robust retention system was required to obtain convergence using the jointed rock model.

Table 1: Comparison of calculated results

Structural Reactions	Continuum		Discontinuum	
	Mohr-Coulomb	Hoek-Brown	SLS	ULS
Max. lateral disp. (mm)	29	48	13	20
Pile max. bending moment (kNm/m)	298	346	240	333
Pile max. shear force (kN/m)	260	297	660	860
Total anchor/bolt force (kN/m)	800	950	750	850

The different structural actions obtained for the east and other faces (i.e. between the partial discontinuum and continuum approaches) arise because of differences in the calculated deformation patterns. Figure 7 compares the relative deformation patterns for the partial discontinuum and continuum approaches assuming a Mohr Coulomb constitutive model.

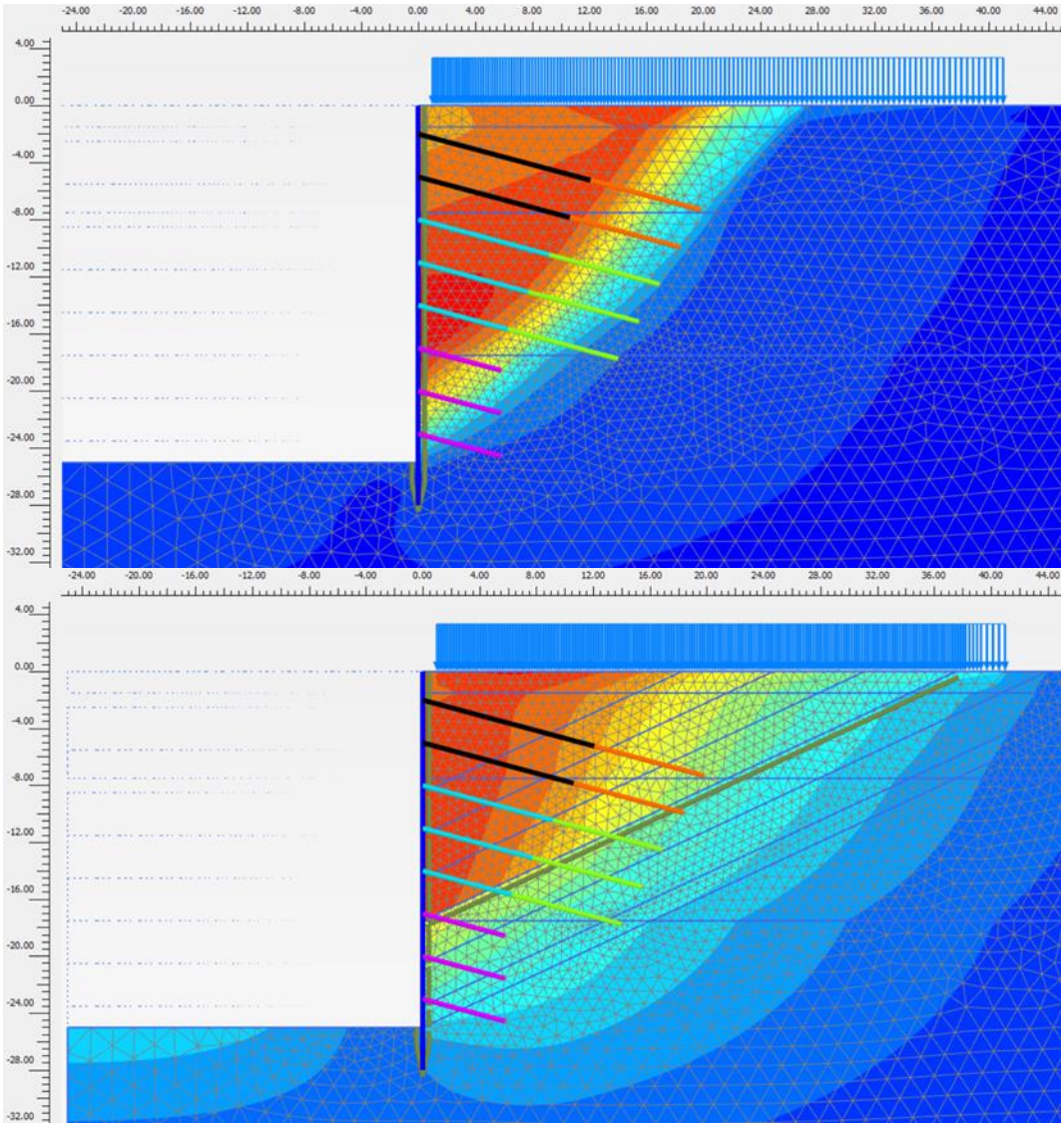

Figure 7: Comparison of deformation patterns using partial discontinuum (top) and continuum (bottom)

The different structural actions obtained using the different constitutive models arise because of differences in the strength envelopes adopted. In the authors' experience the Jointed Rock model generally provides a significant underestimate of rock mass performance in this application as evidenced by proven satisfactory performance of many retention systems designed using a Mohr Coulomb constitutive model. It is also the authors' experience that in general the Mohr Coulomb constitutive model indicates better retention system performance than provided by the Hoek Brown constitutive model. This is likely a result of the relatively high curvature of the Hoek Brown model near the zero stress origin (i.e. due to its near parabolic shape) compared to the linear Mohr Coulomb constitutive model. Whilst a near parabolic shape to the failure envelope appears to be reasonably representative of the strength behaviour of stronger rocks, weaker rocks (i.e. those with low GSI) tend to display a near linear strength envelope at lower stress levels that is closer to a Mohr Coulomb criterion (Johnston, 1985). This indicates that a linear strength envelope as provided by the Mohr Coulomb constitutive model may be better suited to weaker rocks and the Hoek Brown model to stronger rocks.

The range in results obtained from the different approaches and models used in this example has significant consequences for the design of the retention system. The continuum model results indicate significantly higher anchor forces, bending moments and displacements than the partial discontinuum model. It is therefore likely that a retention system design based on a continuum approach, in this case, would likely require larger diameter piles or piles with a greater percentage of reinforcement and higher capacity anchors. It is also likely that anchor lock-off loads may need to be increased to reduce lateral displacements. The continuum model also indicates significantly lower shear forces in the piles than calculated

using a partial discontinuum model. This may indicate that piles designed using a continuum approach have insufficient shear capacity.

It is not known which of the above models provides the most accurate assessment of the performance of the modelled retention system. However, the authors have used the partial discontinuum approach to design many retention systems in bedded siltstone and all but one has performed satisfactorily with movements generally within design expectations. The one exception was for an anchored soldier pile and shotcrete retention system constructed in a siltstone rock mass (as described above).

The example set out immediately above does not consider a high groundwater table nor the presence of a persistent 12^o bedding plane (the 2D plane strain nature of the analysis effectively assumes the presence of release surfaces at either end of the excavated face). Inclusion of both of these into the analysis will significantly affect the calculated results. The most likely outcome is that the retention system adopted above will not be satisfactory and anchor and pile support will need to be strengthened. Both the continuum and partial discontinuum approaches described above can satisfactorily model high groundwater conditions. However, the presence of a slickensided bedding plane cannot be reasonably modelled in the continuum models.

A question arises as to whether or not a retention system should be designed to accommodate the rare occurrence of an extensive, slickensided bedding plane. Such bedding planes are almost impossible to identify through ground investigation, but their inclusion in the analysis significantly increases the cost of the retention system, possibly making the proposed development uneconomic. It may indeed be impractical or cost prohibitive to design for low angled bedding with very low friction angles as the efficiency of post tensioned anchors in particular (but also applies to struts) in such conditions is significantly reduced. The approach that has been adopted by the authors is to not include such very low strength bedding planes in analysis, but to rely on monitoring and an observational approach, with the aim of identifying a potential problem early so that strengthening measures can be implemented if required.

6.2 MODELLNG IN SITU HORIZONTAL STRESSES

A common assumption in retention system design is that there is a uniform but increasing horizontal in situ stress field with depth, with the magnitude of the in situ horizontal stress related to the vertical stress by K_0. It is also common for engineers to adopt the maximum horizontal in situ horizontal stress and lower bound estimates of rock stiffness.

However, high in situ horizontal stresses are usually developed as a result of tectonic movements, wherein the rock mass is compressed in one direction. The impact on the rock mass is one of uniform strain with depth not uniform stress, and as a result, higher horizontal stresses will develop in the stiffer beds within the rock. It is not appropriate to adopt upper bound in situ stresses and lower bound rock stiffness values in an analysis of a retention system. Upper bound stresses occur within the stiffest rock, and appropriately high stiffness values should be adopted.

A more realistic method of modelling the initial in situ stress field is therefore not to use K_0, but to compress the rock mass in the horizontal direction by applying a constant displacement at one or both vertical boundaries of the analysis model to impose a uniform strain field on the rock mass. The magnitude of the boundary displacement applied depends on the width of the analysis model, but it needs to be set at a value that causes the known (or estimated) in situ horizontal stress to develop at the appropriate depth. Lochaden and Haberfield (2019) describe such an analysis that they carried out for a deep basement in horizontally bedded Sydney sandstone.

In situ horizontal stress fields within rock are usually anisotropic and this needs to be accounted for when modelling different faces of the excavation.

In situations where the horizontal stress field is very high, close to K_p, software packages based on achieving equilibrium (e.g. finite element methods such as PLAXIS) may not converge. This is because any reduction in vertical stress due to excavation results in passive failure at the base of the excavation. For such situations programs such as FLAC which do not rely on reaching internal equilibrium may be better suited.

7 DESIGN PHILOSOPHY

The above example highlights the importance of adopting realistic soil models and properties in analysis of retention systems. Results of such analyses can be seriously erroneous if inputs into the analyses are not reasonable and appropriate.

There are a number of standards, codes and local statutory rules which require various factors to be applied to soil and rock properties for the design of earth retaining structures. Many relatively common design approaches require application of reduction factors to elements that provide resistance (e.g. rock mass strength) and load factors to disturbing elements (e.g. rock weight). However for retention systems and retaining walls, rock weight also contributes to rock mass strength, and hence applying a load factor on bulk unit weight of the rock will add to the strength and resistance of the rock. This is clearly illogical if used in an analysis of a retention system.

The authors therefore question the practice of applying factors to reduce and increase strength parameters and loads (e.g. bulk density), respectively, in the analysis of retention systems for ULS considerations.

As stated above, it is the authors' opinion that analyses of retention systems should be undertaken using "prudently conservative best-estimate" properties. This applies to both SLS and ULS analyses, both of which should be undertaken. The SLS analysis should be undertaken assuming reasonably likely scenarios/conditions and prudently conservative properties. ULS analysis should be undertaken for the worst credible conditions but still using prudently conservative properties.

For example, an SLS analysis of a retention system should consider prudent, reasonably likely ground conditions, groundwater levels, loading and construction sequencing. The ULS analysis should consider credible worst-case conditions, e.g. accidental overdig, temporary high groundwater table (e.g. due to water main failure), unfavourable, persistent slickensided fissures or bedding, earthquake loading etc.

The structural reactions obtained from these analyses must then be factored up for design. Different factors should apply to the SLS and ULS reactions, and the maximum factored reactions adopted. Nevertheless, these analyses are usually still assumed to represent SLS conditions and therefore an appropriate load factor (e.g. 1.35) needs to be applied to the calculated structural reactions for design.

The guidance provided in CIRIA C760 (CIRIA, 2017) in relation to design philosophy is largely consistent with that set out in Eurocode 7 (British Standards Institution, 2004), i.e. adopting both Design Approach 1 (applying partial factors to increase permanent and unfavourable variable loads) and Design Approach 2 (applying partial factors to decrease soil strength and increase permanent and unfavourable variable loads). It is the authors' understanding that this approach may be revised in subsequent revisions to Eurocode 7.

Another design decision that needs to be made is the allowable movement of the retention system. A relatively light retention system may prevent failure but movements will be greater than if a stiffer retention system (comprising larger diameter piles and or higher anchor loads) is adopted. For many basement excavations in good quality rock, in situ stresses can be relatively high and batters (at least on favourable faces) can stand vertically without support, or at least with only limited support provided by rock bolts to mitigate localised failures. However, due to the high in situ horizontal stresses very high anchor loads are required to limit movements to normally acceptable levels. Under such circumstances, consideration should be given to providing minimal support provided larger than usual displacements can be tolerated. Such larger than normally accepted movements should be properly communicated so that they are not mis-interpreted as impending failure.

8 ROCK ANCHORS

Whilst the retention system wall (piles, diaphragm wall etc) are the main structural component of a retention system, the majority of the support is provided by the anchors and/or props. The piles primarily act as a "plate" to spread the anchor and prop loads over the excavated face. For this reason, the performance of the retention system is largely dependent on the performance of the props and anchors.

The authors' experience in Australia is that post tensioned anchors are preferred over the use of props and that very few retention systems in Australia are supported by props. However, the use of props is increasing due to issues associated with obtaining permission to install anchors beyond property boundaries.

The performance of post tensioned anchors is highly dependent on how they are installed and tested. In Australia, ground anchors are usually installed under a design and construct contract with contractors required to achieve stated loads via proof loading tests. However, the authors experience is that the quality assurance and quality control associated with ground anchor installation is often inadequate and requires improvement. Some anchor contractors believe it is not necessary to proof load every anchor, nor to keep detailed records of installation and proof loading, and as they are rarely

asked to produce this information they see no reason to record it. There is also a general misunderstanding with respect to bond length and free length, with a common practice being to "grease and sheath" the free length but grout the entire length of the anchor. If this is done then the proof loading test is of no value as the resistance over the full grouted length of the anchor is measured rather than just the resistance over the bond length.

The authors generally recommend the following in respect to ground anchors:

- due to the axial flexibility of anchors and the potential for "unzipping", the bond length of anchors should be no less than 3 m and no greater than 10 m to 12 m;
- for anchors installed along excavation faces unaffected by adverse, persistent discontinuities (e.g. bedding) a free length extending 1 m beyond a plane extending up from the base of the excavation at an angle between 45^o and 45^o + φ/2 (to be assessed by the designer on a case-by-case basis, where φ is the internal angle of friction), or above better quality rock that is assessed to be self-supporting, is appropriate;
- for anchors installed along excavation faces that are impacted by adverse, persistent discontinuities, the free length needs to extend beyond a potential bedding plane that could impact the wall. For example, the free length of the upper most row of anchors needs to extend beyond a potential bedding plane located at 0.5 m below the second row of anchors and so on for subsequent rows. At every stage of the excavation, there must be sufficient bond length beyond the critical bedding planes to support the potentially unstable wedge;
- excavation must not proceed more than 0.5 m below the level of any post tensioned anchors prior to their installation and tensioning;
- all post tensioned anchors are secondary grouted and the minimum free length is maintained to allow post tensioning of the anchors. This may require some grout from the initial grouting stage to be washed out prior to tensioning and the secondary grouting stage. For the reasons set out above, the authors consider sheathed anchors grouted over the full length are not satisfactory for post tensioning;
- the anchor contractor is to provide a detailed design and construction methodology statement for review by the designer prior to the commencement of anchor installation;
- all anchors are proof loaded to confirm that the stated loads are achieved. As a minimum all anchors to be proof loaded to 125% of the design working load and held for 15 minutes with records of the applied load and anchor elongation made at specific stages of the proof loading; and
- full quality assurance and quality control documentation, including proof loading results, length of free zone following wash out, length of grouted zone following wash out, anchor declination, anchor lock off load, etc to be provided for every anchor and be available to the designer if requested.

9 MONITORING

The appropriate undertaking of the ground investigation and analysis of the retention system does not negate the requirement for an appropriate movement monitoring system. The extent of such a monitoring system is dependent on a number of issues, including the magnitude of the retained height and the risk to adjacent structures and services. As a minimum, a monitoring system would likely include target markers at capping beam level which are manually surveyed. More sophisticated monitoring systems may include inclinometers, a series of target markers placed along the retained height as excavation continues, and monitoring of adjacent structures and services, with surveying of target markers undertaken at regular intervals using automated total stations. It is critical that target markers at capping beam level are installed prior to excavation works.

The monitoring system must be accompanied by guidance on appropriate trigger levels and the associated action to be taken should these movement limits be reached. Guidance should also be provided on the frequency of movement monitoring. Although often ignored by engineers, it may be useful to relate the trigger levels to the degree of excavation which has taken place at that time, such that early warning of movement which is not consistent with the design is possible. The results of numerical analyses which consider the staged construction sequence may be useful in this regard. However, the accuracy and precision of the monitoring instruments must be carefully considered. Furthermore, low trigger levels which are unwarranted are likely be exceeded, and to cause both program delays and frustration to those involved in the construction of the retention system.

10 CONCLUDING REMARKS

This paper summarises the authors' experience in the design of retention systems in rock, which differs significantly from those in soil. A number of the critical issues associated with the design of retention systems in rock have been discussed, including those related to the appropriate characterisation of the ground, design tools and design philosophy. Reference has been made to a number of case studies, including those with which the authors were involved and which did not perform as anticipated. Appropriate monitoring systems were critical in identifying the divergence of the true behaviour of the system from the anticipated behaviour.

11 ACKNOWLEDGEMENTS

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CROSS PASSAGES GROUND TREATMENT FOR MELBOURNE METRO TUNNEL PROJECT

Philippe Vincent[1], Alexandre Hubaut[1], Sin Ping Chan[2], Simon Auvergne[2]
[1] Menard, Sydney, NSW, Australia
[2] Bouygues Construction Australia, Melbourne, VIC, Australia

ABSTRACT

As part of the Melbourne Metro Tunnel project, there was a requirement for ground treatment between tunnels, specifically at cross passages where the ground was identified as not being sufficiently competent to safely execute underground activities such as the safe removal of precast concrete segmental tunnel linings at cross passages and excavation of the cross passages. Cross passages CP02, CP03, CP04, CP05, and CP15 are located within soft or mixed ground conditions which required ground treatment to control groundwater ingress and/or provide suitable ground control to allow stability of mined excavation between the tunnel tubes.

The proposed ground treatment principally consisted of overlapping jet-grouting columns forming a homogenous treated block installed from ground surface to depth of 30m. Further, when the cross passages treatment area intersect rock, rock-grouting treatment was required to reduce the permeability of the rock mass and of the interface between jet & rock to reduce potential water inflow to the future excavation within allowable limits.

The realization of those critical ground treatment required to overcome technical challenges which are detailed in the paper and include but not limited to; adapting the grouting parameters to accommodate for the high geological & geotechnical variability at each site, working near various urban assets including under and around a live rail line, maintaining alignments for both vertical and inclined columns, ensuring proper grouted diameters with sufficient overlap and remediation based on as-built 3D monitoring. To achieve this, the authors relied on a range of state-of-the-art trial at each treatment location and associated geotechnical testing, monitoring and mapping techniques some of which were used for the first time in Australia. In addition, a comprehensive regime of in-situ testing was carried out from the surface and from within the tunnel following and confirming performance of ground treatment to demonstrate that the project's water tightness and mechanical requirements were consistently met to mitigate the risks associated with excavation of the cross passages. The performance and quality of the treatment was ultimately confirmed during the excavation of the cross passages.

1 PROJECT OVERVIEW

1.1. INTRODUCTION

The $11 billion-project of Melbourne Metro is the greatest ever transport infrastructure project in Melbourne. Scheduled to open in 2025, the purpose of the metro tunnel is to free up space in the city loop to run more trains to and from the suburbs by taking the busiest train lines through a new tunnel under the city. The Project comprise twin 9-kilometre-long by 6.9m internal diameter tunnels providing North-South underground rail connection across Melbourne Central Business District from Western Portal near Kensington Road to Eastern Portal near Chapel Street, South Yarra. New underground railway station will be constructed at Arden, Parkville, CBD North, CBD South, and Domain. As part of the works, 26 cross passages are to be constructed at approximately 230 metres apart along the tunnel alignment. Cross Passages are constructed from below ground within the tunnels after the tunnel boring machines (TBMs) have passed through. These passages are an important safety feature that allow people (including emergency services) to move from one tunnel to the other in the event of an emergency as well as hosting maintenance facility for drainage.

1.2. TECHNICAL CHALLENGES

As part of the works, there was a requirement for ground treatment between tunnels, specifically at cross passages where the ground has been identified by the Designer, as not having the required parameters to safely execute underground activities such as the safe removal of precast concrete segmental tunnel linings and mined excavation of the cross passages under the threat of high water and earth pressure. Cross passages CP01, CP02, CP03, CP04, CP05 and CP15 are located within soft or mixed ground conditions which require ground treatment to control groundwater ingress and provide

suitable ground control to safely execute the underground works. This paper specifically presents the ground treatment design & construction at CP02, CP03 and CP04 (Figure 1).

Excavation of an underground cross passage is a high-risk activity and numerous associated accidents were reported in the literature. In case of an accident during the opening of a tunnel segment and the mined excavation between the tunnel tubes, the groundwater and soils might flow into the tunnel, causing large amounts of ground loss, and induce excessive ground movements which might cause permanent distortion to the tunnel shape and movement of the segments which make up the tunnel lining, ultimately damaging the completed tunnels. The influx of water/soil might also provoke piping and subsequent ground subsidence and formation of sink hole above the tunnels propagating to the surface. Such event causes delay & increase of cost for the project and may have significant impact on surrounding infrastructures.

In light of the above consideration, the designer had considered several complementary risk mitigation strategies for the safe excavation of the cross passages with due consideration that resulting discontinuity of the improved ground could create a path for the invading water. Additional measures include water leak test and probe grouting from the tunnel.

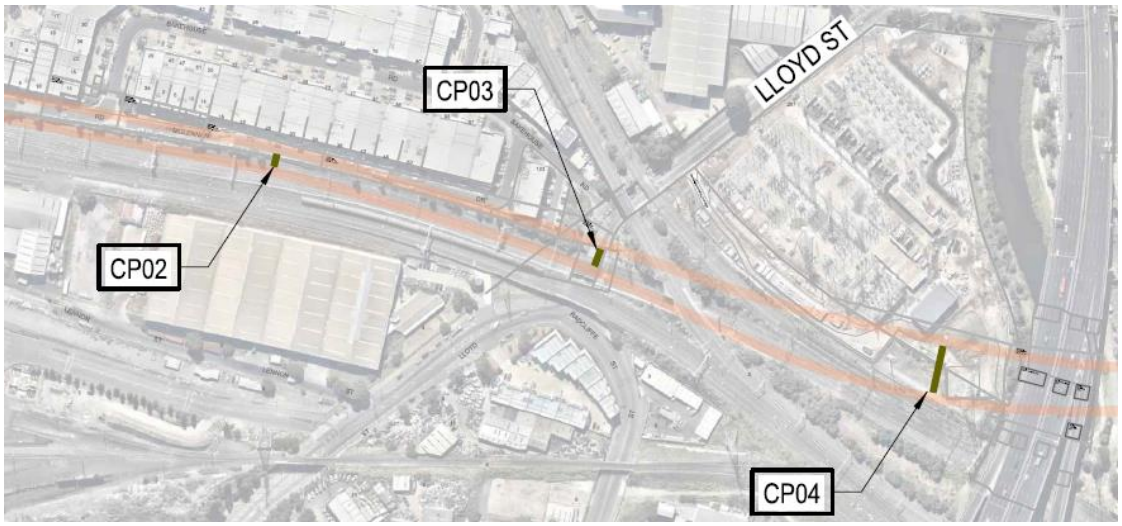


Figure 1: Key Plan

1.3. CONTRACT APPROACH

The Cross Yarra Partnership Consortium (CYP) consisting of John Holland, Lendlease and Bouygues Construction Australia have been selected to design and construct the Metro Tunnel (Tunnels and Stations PPP Project) for the State of Victoria (RPV).

In turn, and in order to perform the highly technical ground stabilisation works, the CYP consortium awarded Menard Oceania (MO) as specialist geotechnical sub-contractor for all the ground treatment works. The award process involved numerous consultations during which the scope of MO was adjusted several times in order to allow for appropriate risk sharing and mitigation measures to be included in the contract. Both CYP and MO worked in close collaboration to identify potential risks and effective measures and scenarios to deal with them. The award was based on the understanding that the limited amount of geotechnical information available would require detailed trialling at each CP.

2 CROSS PASSAGE GROUND TREATMENT DESIGN

2.1. GEOTECHNICAL CONDITIONS OF THE CROSS PASSAGES

The Silurian age Melbourne formation (also known as the Melbourne mudstone) forms the bedrock in the Melbourne area, it is comprised mainly of siltstone interbedded with sandstone and characterized by persistent bedding varying typically between 10 to 300mm thick. Above the bedrock tertiary deposits are observed on the western side of the project locally such as the Werribee formation (hard clay & dense sand/gravel mainly) and older volcanic (fractured basalt mainly). Then quaternary Pleistocene deposits such as Fishermens bend silty and Moray street gravel and Quaternary Holocene deposits such as the Coode island silt are the main geological formation impacting the ground treatment works.

Due to this complex geological context, each cross-passage location presents specifics and challenging ground conditions such as for example hard clay in CP2 underlying Older Volcanic, highly variable Melbourne formation rockhead in CP3 and soft clay in CP4 (Coode Island Silt).

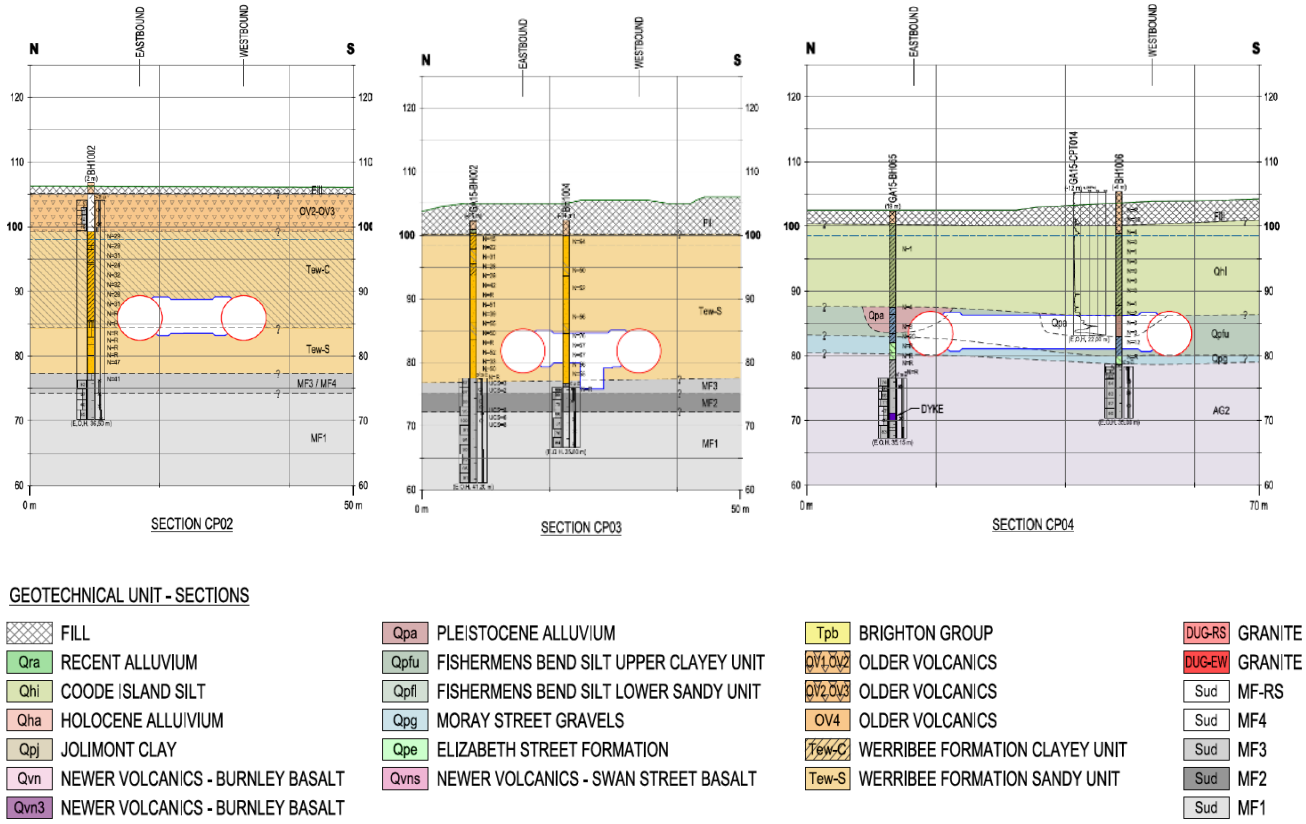


Figure 2 (a): Geotechnical cross sections at CP02 and CP03 and CP04

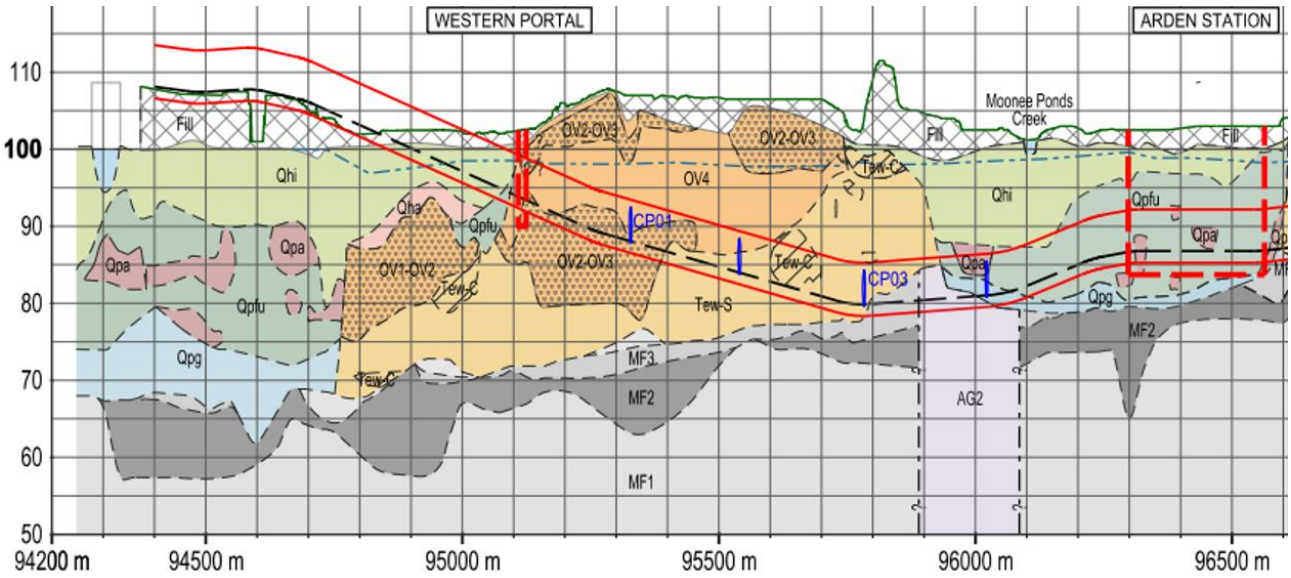


Figure 2 (b): Geotechnical Long Sections from CP1 to CP4

2.2. GROUND TREATMENT DESIGN

2.2.1 Design philosophy

The proposed ground treatment principally consisted of overlapping jet grouting columns forming a homogenous treated block installed from ground surface prior to TBM tunnelling with the intent to:

- provide sufficient strength and treated volume around the mined excavation between the tunnels tubes to allow safe execution of cross passages excavation works
- minimise the seepage of groundwater into the mined excavation between the tunnel tubes during execution of cross passages excavation works. The overall permeability of mass Jet grouting treatment is governed by defects. Such defects may play a minor role on the overall seepage processes if jet grouting is carried out in fine grained soils, whereas they may become relevant in granular soils because of the localised flows through the untreated soil portions.
- provide suitable durability for the intended life of the proposed improvement and until construction of the permanent lining (about 2 years after treatment block construction).

Further, when the cross passages treatment area intersect rock, rock grouting and interface between jet & rock treatment was required to reduce the permeability of the rock mass and interface to reduce potential inflow to the future cross passage excavation. The rock grouting was intended to target all joints, faults, bedding planes in order to create an acceptable seal to achieve the performance requirements.

2.2.2. Numerical Modelling

The stress and plastic yielding in the jet grouting block was assessed using Plaxis 2D FEM model. Stresses developed within the jet grouting block were assessed in 2D plane strain models for each CP. Two different load deconfinement ratios λ (0.3 and 0.7) have been considered in order to evaluate the interaction between the soil and the structure. Different values of the thickness of the ground treatment around the opening were investigated and the improvement of ground strength Rc to identify the values at which the plastification of the soil is sufficiently reduced.

The front stability for those CP was studied according to the methodology developed by Cornejo & al. During the design stage of the temporary support of the cross passages, the jet grouting block has been modelled with an increased cohesion and Young modulus to define the temporary support.

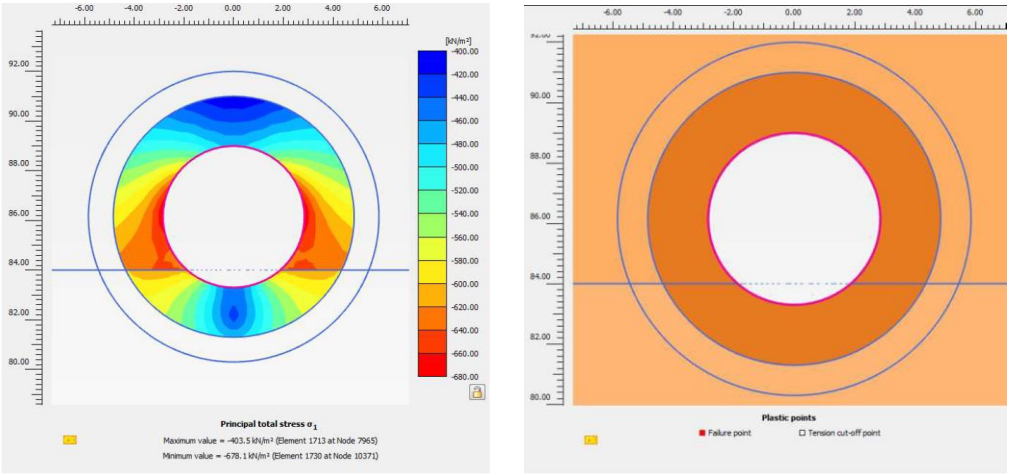


Figure 3: Principal total stress (λ = 0.3) Plastic points (λ = 0.3) following CP excavation CP02/CP03

At CP04, considering the length of the CP and the presence high hydraulic conductivity gravel layer above rock, the ground treatment consisted of unreinforced Continuous Flight Auger (CFA) secant pile walls made of weak concrete forming a 30.4m x 10.01m rectangular box and four cross walls installed to rock. In addition, jet grout blocks made of overlapping jet grouting columns were required to form plugs between the CFA cells and at the collar location between the tunnel and cross passages. Stresses developed within the jet grouted blocks were assessed in Plaxis 2D plane strain models.

2.3. GROUND TREATMENT DESIGN CRITERIA

Following detailed Finite Element Modelling, seepage and drawdown assessment, it has been found that the required thickness of the extent of the jet grouting treatment zone around the CP and TBM excavation vary between 2 and 3m depending on the cross passage with the requirement to achieve a permeability of 1.10-7m/s to 5.10-7m/s . The minimum required unconfined compressive strength at 28 days was set at 1.5MPa for CP02, CP03 and CP04. The minimum required Young Modulus of 225MPa was specified at CP04.

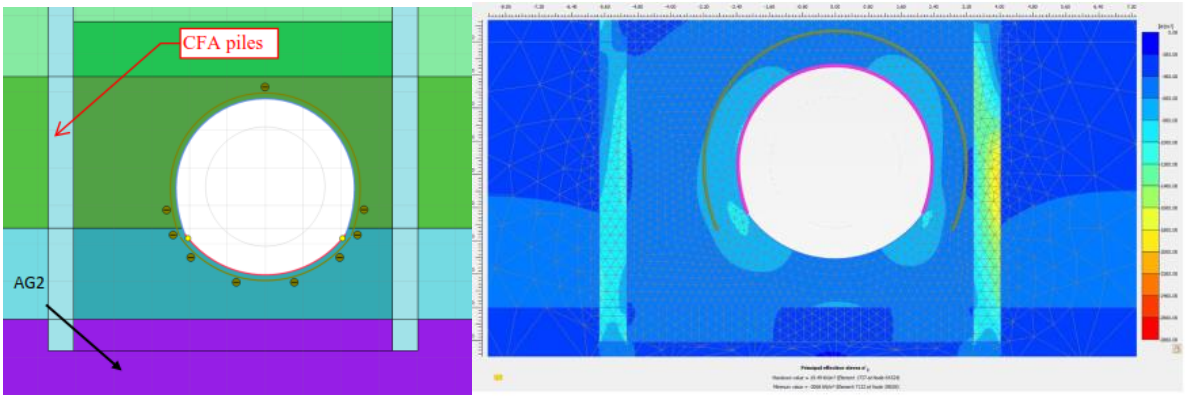


Figure 4: FEA model geometry and Principal effective stress following CP excavation at CP04

3 JET GROUTING GROUND TREATMENT

3.1. SITE LAYOUT CONSTRAINTS

The ground treatment was installed in a challenging urban environment. This requires handling of complex challenges for safe construction and to minimise impacts on existing buildings, structures, utilities and the public. The cross passage CP02 is located directly beneath operating rail lines requiring the ground treatment to be carried out from the surface with a drilling inclination angle up to 32 degrees. The jet grouting ground treatment block was constructed at the depth of 18 m to 27m. Live utilities including a major gas main was running through the working area requiring adjusting the location of the drilling entry points around an exclusion zone.

The ground treatment at cross passage CP03 was constructed at the depth of 18 to 26 m under a MTM rail bridge adjacent to a live road. A majority of the works were carried out under a 3.1m low headroom restriction in close proximity to electricity and water utilities. Inclined drilling for grouting works were required to avoid existing service utilities and the MTM rail bridge. The ground treatment at cross passage CP04 was as challenging requiring the installation of inclined jet grouted columns at the westbound plug due to overhead high voltage power lines and the operating rail lines.

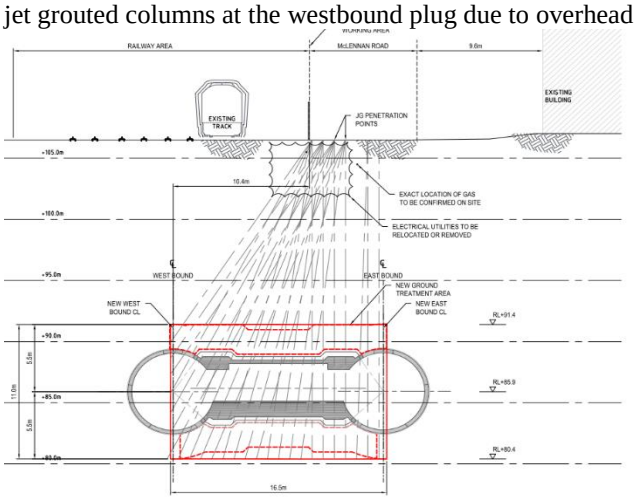


Figure 5: CP02 Cross section

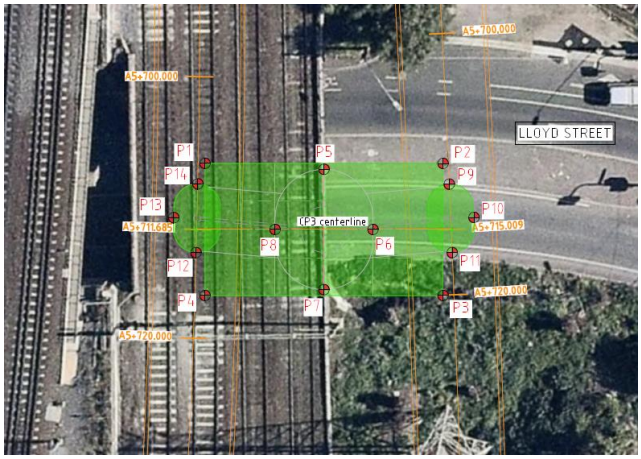


Figure 6: CP03 aerial view

The jet grouting batch plants, cement silos and high pressure grout pumps were containerised and established along the operating rail lines at the location of CP02 compound. Grout lines was run from CP02 site compound to CP03 work front.

The jet grouting operation was carefully monitored during all phase of the execution to prevent collapse of borehole and subsequent blockage which might lead to heaving and lateral ground deformation causing distress on the adjacent structures and buried utilities. A direct circulation of polymeric slurry was used in the drilling phase to sustain the borehole walls. During jet grouting, a visual observation of the flow and features of the spoil return at the collar was maintained at all times. The management of the spoil collection, storage and subsequent evacuation was particularly challenging considering the dense urban condition and tight working area. Following pumpability test of the generated spoil during field trial, Menard adopted a high suction capacity impeller pump to evacuate the jet grouting spoil from the rig to above

ground spoil bays established by CYP. The wet spoil was left to set over night and separate from excess water and later dispose of at a rate that maintains sufficient storage area to accommodate spoil delivery from the jet grouting activity.

Figure 7: Site set up at Cross Passage CP02 and CP03

Figure 8: Jet grouting operation along rail line at CP02

Figure 9: Batching plant set up at CP02

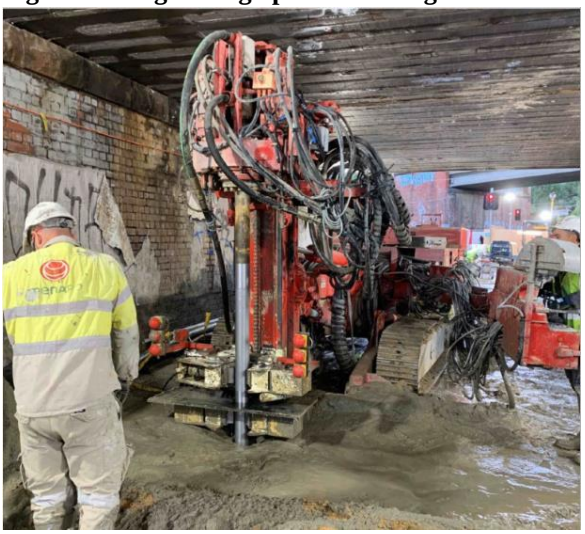


Figure 10: Jet grouting operation under rail bridge at CP03

Figure 11: Spoil bay area at CP03

3.2. CONSTRUCTION FIELD TRIAL

3.2.1. Overview

Considering the highly variable nature of the geotechnical conditions encountered, extensive field trials was performed at each cross-passage location to verify the relationship between jetting energy and associated jet grout column diameter. Pre-production test columns are generally jetted near the ground surface and exposed by excavation to allow for visual measurement and inspection of the columns in order to establish the jetting energy versus column diameter relationship. Due to the jetting depths involved on this project, the use of shallow columns was not practical because the near-surface soils in the available test area are made of fill material that is not representative of the deep Werribee Formation that is present on CP02 and CP03 and other geotechnical units encountered on CP04 and to be treated at the CP elevations. Also, the ground water pressure and the overburden stresses at depth differ substantially from the near-surface conditions and an effective correlation cannot be made between shallow test columns and the deeper production columns.

Considering near surface ground conditions are not representative of the production works and excavation and visual inspection of columns at the required depths is not practical, Menard proposed to use a combination of CYLJET electric cylinder method® (downhole geophysics) and cored borehole inspection. CYLJET® provided an overall understanding of the column continuity, and cored boreholes allowed to punctually confirm the CYLJET® result and provide visual information on the treated material as well as recovering samples for strength and permeability testing.

An additional method which allowed information to be obtained on the achieved diameter based on the observation of the effect of jet grouting within inspection holes placed at different distance parallel to the centreline of columns was trialled. The monitoring holes were positioned at variables distances around the injection holes to check if the jet is able to cover the respective distances at variable depths. It was proposed to insert painted small diameter pipes along the measuring verticals, which were then retrieved after treatment to observe if the jet action eroded the paint on the different pipes.

Finally, another control introduced relied on the analysis of the variability in density of the spoil return during the Jet grout columns sequenced installation. Following completion of the primary columns, density of spoil recovered during jetting of secondary columns is lower than that of the primary columns. This is because during the installation of the secondary columns, the volume of untreated ground is reduced by the area contained within the primary-secondary overlap. As a result, spoil density of secondary columns is closer to that of the grout density. A further decrease in density during installation of tertiary columns is also expected. A close monitoring of the evolution of the spoil permitted to assess the anticipated variations in column diameter and validate the consistency in overlap ratio throughout the extent of the treated block.

3.2.2. Construction Field Trial construction sequence

Construction field trials generally consisted of a minimum of three trial panels made of three overlapping jet grouted columns using different sets of injection parameters to adjust and select the most efficient jetting parameters which best suit the encountered ground conditions for production.

An initial CYLJET calibration hole was drilled and equipped with a PVC slot pipe in the untreated ground to measure its natural resistivity. Following the installation of primary column in each field trial panel, CYLJET readings were taken at the centre of the columns to measure the treated ground resistivity and assess the achieved column diameter through inversed contrast. Following installation and interpretation of achieved diameter on the primary columns, secondary and tertiary columns was installed at the condition that measured diameter of the primary columns were found satisfactory. Following a minimum waiting period of 7 calendar days after the installation of the overlapping jet grouting columns, vertical fully cored boreholes was carried out in the overlap zones and triple point to verify the column diameter and demonstrate the quality and continuity of the grout material in the installed columns.

3.2.3. CYLJET Electric Cylinder® Method

The Electric Cylinder® method developed and patented by SIXENSE consists of recording and analysing the potential differences generated by an induced electric current around a borehole. The electric monitoring field around the hole takes the form of a cylinder up to about 5 to 10m in diameter, depending on the electrical resistivity of the ground and instrumentation system employed. Deflections in the resistivity equipotential lines identify features such as voids, loose zones, clay beds, etc. The method was originally used to detect anomalies such as karsts, shear faults, groundwater leaks, etc., for example when probing ahead of a tunnel boring machine.

When applied through a jet grouted column, the CYLJET® method makes use of the strong contrast in resistivity between the ground and the column material containing cement. The influence of this jet grouted material mass distorts the equal resistivity contour lines. The electrical resistivities of the ground and of the soil cement forming the column being known, a computer can simulate the shape of the resistivity contour lines under the influence of columns of various diameters. It is then simply a matter of comparing various simulations with the actual instrumental field readings to find the diameter of the column.

In most cases, the resistivity of the ground in place is several tens of ohm-m, while the jet grouted column material registers only a few ohm-m. However, column resistivity increases sharply over time, as the material hardens. After a few days, the ground/column resistivity contrast diminishes, and eventually reverses when the column has fully hardened.

Due to the presence of saline groundwater at CP02 and CP03, measured ground resistivity was very low and comparable to that of the fresh jet grouted column material. Unlike the usual situation with fresh groundwater, it was necessary to wait several days or weeks for the grout to set and resistivity of the jet grouted material to rise well above that the ambient environment before measurements are made. At CP02, a 16 days setting period was required to allow the grout to reduce conductivity enough to differentiate from the charged ions in saltwater.

Thanks to use of this patented technology, the project team was able to refine the accuracy of the estimated column diameter to within +/-150mm but also and importantly to track down the evolution of JG column diameters with depth enabling to adjust the JG parameters based on the varying geotechnical profiles encountered.

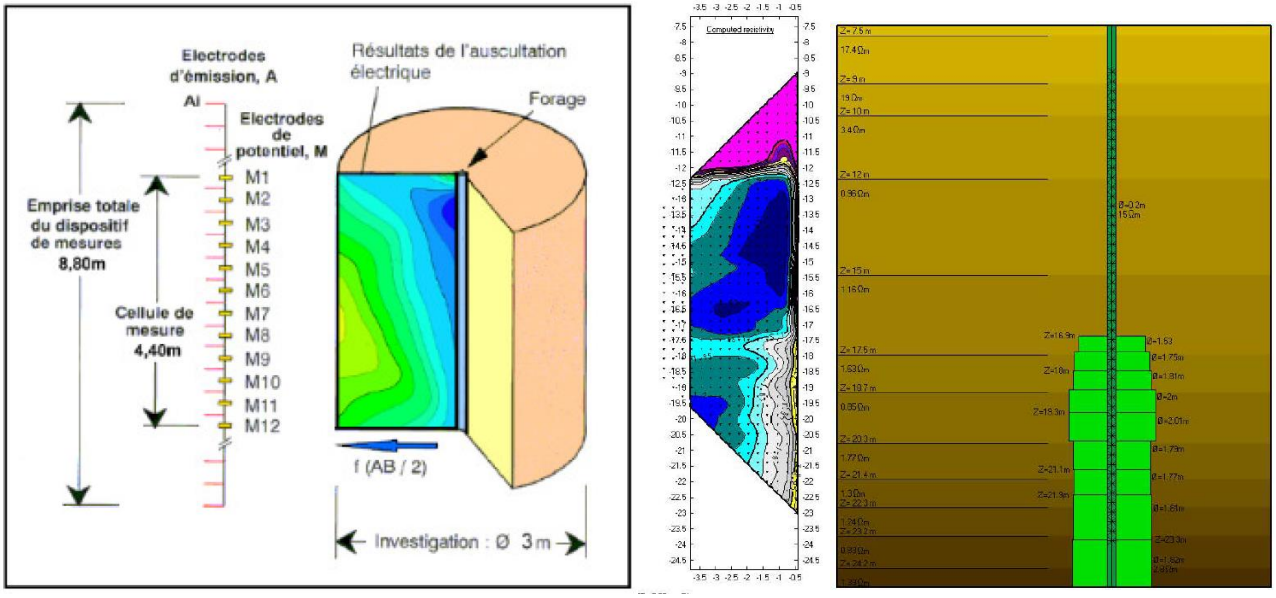


Figure 12: Jet grouting column measurement using CYLJET electric cylinder method® at cross passage CP03

3.3.4. Full depth Cored borehole and associated testing

The trialled overlapping jet grouted columns was fully cored at their intersections to verify closure of the columns and to confirm the achievement of a high homogeneity of treatment. Additionally, by drilling boreholes at different distance from the column axis, information on the average diameter and on possible defects of treatment was observed. UCS testing on retrieved core samples and laboratory permeability testing was carried out on retrieved core samples. In situ permeability testing on cored boreholes such as falling head tests and lugeon test (ie packet test) was carried out to assess the permeability of the ground treatment and validate the jet grouting parameters prior to production. Drilling deviation of installed jet grouted columns and cored boreholes was measured.

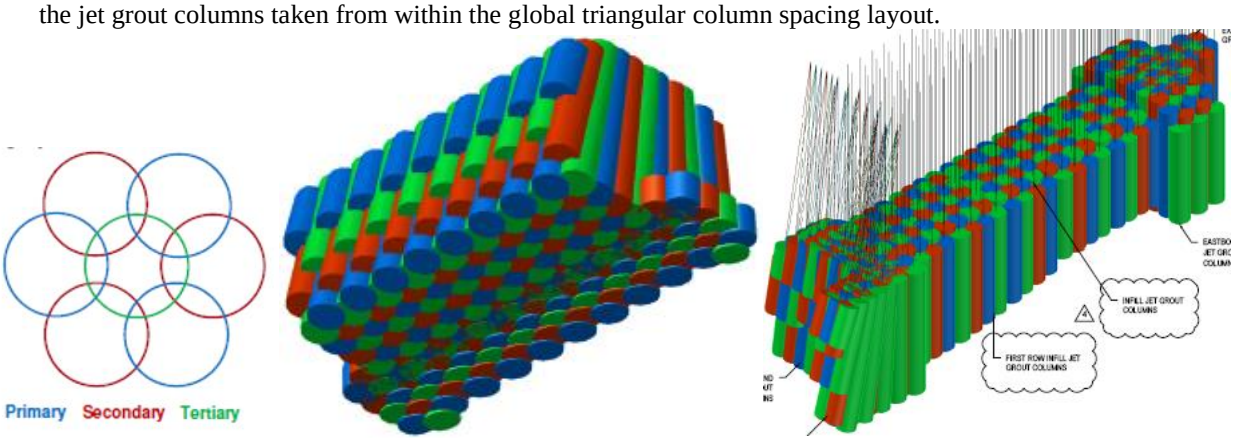
Figure 13: Cross Passage CP03 - Field trial panel D – vertical cored borehole at the triple points of columns

3.4. PRODUCTION PERFORMANCE TESTING

3.4.1. Jet grouting layout & design philosophy

In the case of the depths involved for the ground treatment of the cross passages, the potential drilling deviation associated with depths of 19-30m would result in an excessively tight column layout with high risk of shadowing effect. Given the complexity of the project, Menard measured drilling deviation at depth using the TIGOR system developed by Jean Lutz to control alignment and deviation of jet grouting columns. We note that this system allows to calculate the deviation of the drilling only when the drill bit returns to the surface at the completion of the JG column installation.
In order to control the ground treatment continuity and homogeneity, the following approach was adopted:

1) Prepare triangular pattern layout based on expected maximum deviation (ie. 0.5%)
   JG columns was installed using the primary/secondary/tertiary sequence with tertiary columns filling the remaining intersection between already jetted primary and secondary columns. The working pattern within the global layout consist of a hexagonal pattern of three primary columns and three secondary columns at the nodes of the hexagon and one tertiary column inside the six perimeter columns. Figure 14 (a) below shows a typical hexagonal pattern for the jet grout columns taken from within the global triangular column spacing layout.



(a) Hexagonal Pattern **(b) CP03** **(c) CP04**

Figure 14: Jet grouting Arrangement

2) Drill and install primary columns
3) Measure and record drilling deviation of the primary columns and then produce a 3D plan view of the as-drilled primary hole locations and primary JG columns.
4) Once this plan view was prepared and reviewed, the secondary column layout and work sequence was re-assessed. Where the drill-hole deviation precluded the use of the original design pattern, the column diameter and sequence was adjusted based on the as-built JG pattern.

a. When the drill-hole deviations are below 0.5%, no adjustment is necessary.
b. When the drill-hole deviations are above 0.5% in the order of 1%, the approach is to adjust the diameter of the secondary column and adjust drilling position.
c. Larger drill deviations can result in changing the jetting sequence, jetting adjacent columns fresh-and-fresh, and possibly installation of additional columns.

5) Drill and install secondary columns
6) Repeat steps (2) to (4) for tertiary columns.

A total number of 164, 199 and 290 jet grouted columns were installed at CP02, CP03 and CP04, respectively for a total of 15,752m3 of treated material. A total of 15,993 lm of drilling were carried out with a variable depth of 19 to 30 meters. The jet grout column diameters typically ranged from 1.8 m to a maximum of 2.0 m. The jet grouting works at CP02 and CP03 was carried out concurrently between March and September 2019. The jet grouting operation at CP04 was carried out between May 2019 and July 2020. Jet grouting works was carried out on double shifts to meet the stringent program requirement.

3.4.2. Selection and Monitoring of jet grouting parameters

The applied jet grouting energy per unit length of column E_n' was calculated at the nozzle in function of the grout density ρ_g, the grout flow rate Q, the number of nozzle M, the diameter of nozzles d and the average monitor lifting speed v_s.

$$E_n' = \frac{8\,\rho_g\,Q^3}{\pi^2 M^2 d^4 v_s} \quad (1)$$

Based on observation derived from JG field trials, the jet-grouting parameters were chosen to have a jet-grouting specific energy varying between 80 to 130 MJ/m depending on the ground condition and target diameter. Additional CPTs were carried out at CP04 to verify the strength of Coode Island Silt which SPT-N values were zero and subsequently assisted at defining the JG parameters. Triaxial test were also carried out on CP02 to confirm the strength of the hard clay Tew-C unit.

JG drilling and injection parameters were continuously monitored throughout execution by Jean Lutz automatic acquisition system, data logger and software to process data and to plot them on a computer screen. JG parameters were predetermined following validation of the construction field trial. Therefore, the aim of routine monitoring of jetting parameters was to verify the conformity of the injection parameters (Fluid flow rate and pressure, rotary speed, withdrawal speed, etc). Grout specific gravity and March cone viscosity was measured and recorded from grout samples taken directly from the batch plant agitators on a regular basis.

3.4.3. Continuity and Homogeneity of jet grout structure

Inclined cored boreholes were carried out in the jet grout mass treatment to demonstrate quality of the grout material in the installed columns. Core samples was examined for continuity and homogeneity of grouted material. Verification coring locations were selected by CYP Geotechnical and Design team and were selected in a potential gap in the treatment block at the intersection between columns, identified in the Menard as-built 3-D model.

3.4.4. Physical and mechanical properties of the jet grout material

The conformity of the ground treatment was verified by means of UCS testing on wet grab samples, UCS testing on retrieved core samples, laboratory permeability testing on retrieved core samples and in situ permeability testing on cored boreholes such as falling head tests and lugeon tests.

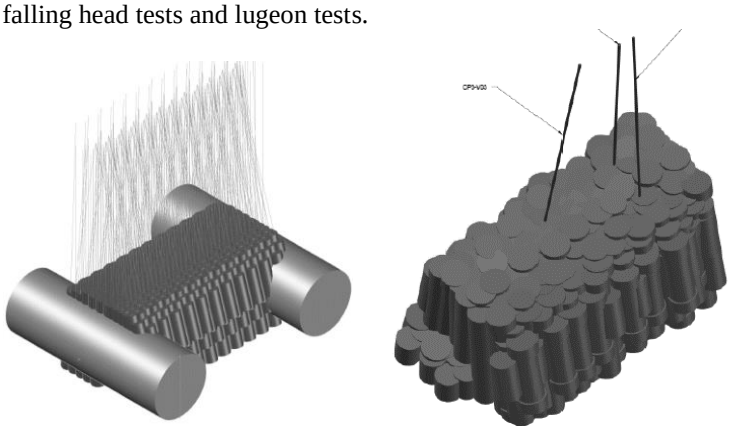


Figure 15: Cross passage CP03 Drilling Deviation monitoring and 3D as-built with inclined cored borehole

4 CROSS PASSAGES EXCAVATION

4.1. PROBE HOLE DRILLING & GROUTING PROGRAM

Use of probe hole testing prior to breakout of tunnels for construction of cross passages was adopted to avoid the consequences of unexpected encounter with high permeability zones or zones where pre-treatment has not been fully effective. A probe hole drilling & grouting program was established for each cross passage to avoid unexpected exposure to conditions which could be hazardous to the construction workforce or could give rise to water inflows that could induce progressive material washing leading to adverse environmental impacts or surface settlements.

Anticipated flow rates and associated trigger levels were nominated to indicate the potential for seepage to the completed cross passage in excess of rates which could give rise to adverse impacts. For cross passages in treated ground, probe holes which are at least 50% of the length of the cross passage yielding sustained flow from a single probe hole of less than 30% of the predicted/target total flow to the completed cross passage was considered satisfactory for opening of the excavation. Aggregate sustained flow to multiple probe holes of less than 50% of the predicted/target flow to the completed cross passage was considered satisfactory for opening of the excavation. Recommended trigger flow for single and multiple probe hole varied between 0.07-0.15L/s and between 0.1-0.2L/s respectively. Equivalent maximum permeability varied between 1.10^{-7} to 5.10^{-7}m/s.

Probe holes was drilled at angle varying between 2 to 16 degrees and depths ranged between 8 to 26 meters from inside the main tunnel. The probe hole testing campaign met the acceptance criteria set out by the CYP design team. The level of water inflow and permeability requirement of 2, 1 and 1 lugeon for cross passages CP02, CP03 and CP04 respectively was successfully met. No additional probing holes and pressure grout injections were required to meet the performance criteria.

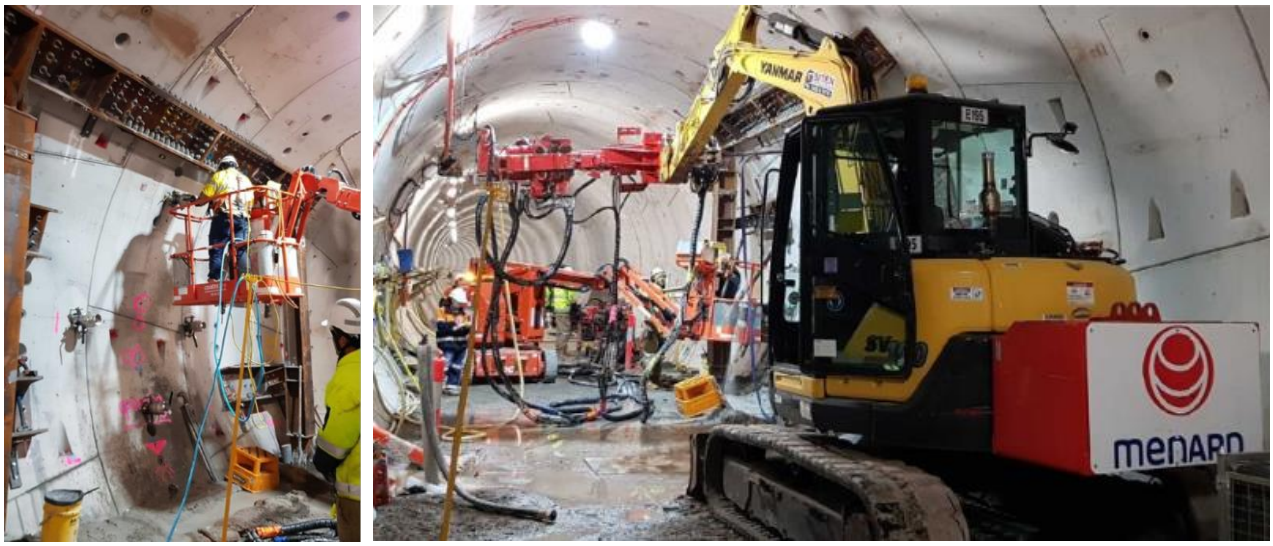


Figure 16: Probe grouting

4.2. REMOVAL OF PANELS AND EXCAVATION OF TREATED SOILS

Following installation of tunnel support to prevent distortion of the main tunnel, probe drilling and subsequent removal of precast concrete segmental tunnel linings at cross passages, mined excavation was carried out in section limited to 1500 mm using remote control demolition robots. The temporary support consisted of shotcrete of 200 to 250mm thickness and two layer of wire mesh.

4.3. MEASUREMENT OF AS-BUILT COLUMNS

Following opening of the cross passages, primary jet grouted columns at the crown was exposed by mined excavation and allowed for visual measurement and inspection of the achieved column diameter. The measured diameters were in line with the design. Exposed as- built columns were mapped and checked against the 3D as built models which confirmed the accuracy of the drilling deviation measurement.

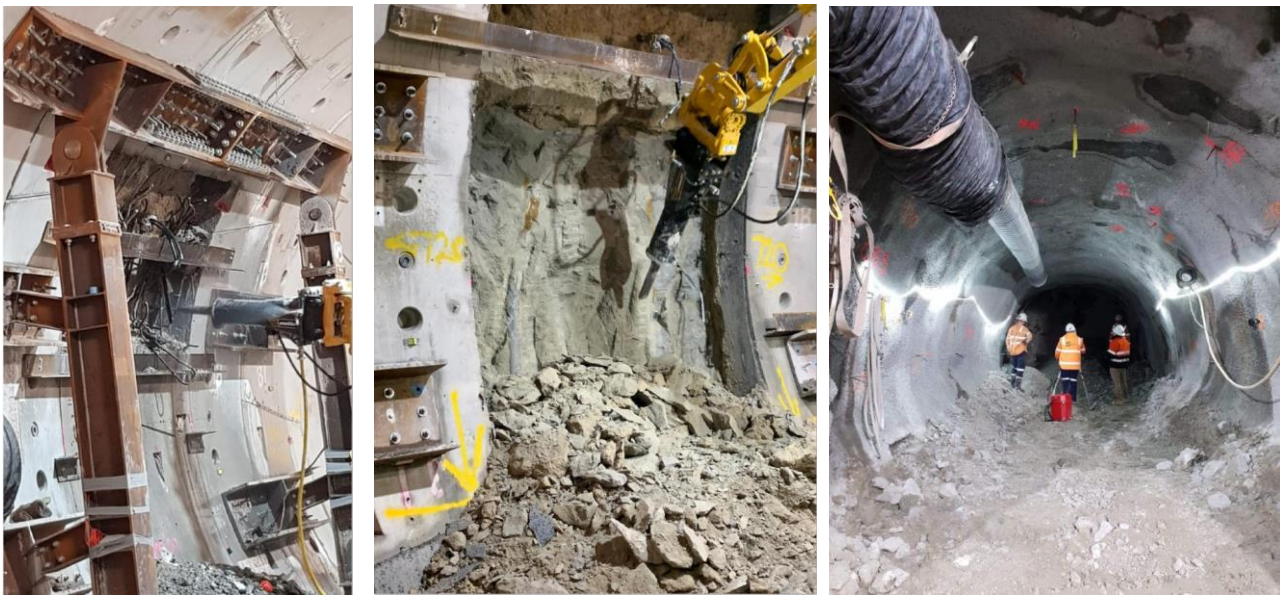


Figure 17: Tunnel support and excavation of CP using remote controlled excavator

(a) Exposed JG column at CP03 (CH5.8 at mid-crown) (b) Exposed JG column at CP04 (CH8.6 at mid crown)

Figure 18: Tunnel support and excavation of CP using remote controlled excavator

5 INSTRUMENTATION & MONITORING

As in the case of any large tunnelling project, the potential for ground movement exists where excavations would be undertaken as part of Melbourne Metro works. Ground movements may occur above and adjacent to Melbourne Metro works due to underground excavations and consolidation settlement of compressible soils due to groundwater drawdown. Buildings, utilities and civil infrastructure such as roads, rail lines and bridges may be subjected to the effects of ground movements (settlement) caused by the mined excavation of the cross passage portals and/or associated drawdown effects.

5.1. EXTERNAL TUNNEL MONITORING: SURFACE MOVEMENT

Considering the critical structures and utilities in vicinity of the ground treatment areas (Active railway, >100yr brick wall with shallow brick-foundation, >100yr large dia. Brick barrel sewer, gas mains and HV cable etc), intensive monitoring with close follow up against the highly restricted allowable displacement had been carried out covering the ground and structural displacement and change in groundwater level/pressure. Fluctuated background displacement with a range was identified, increasing the challenges on identifying the early warning of the displacement related to the ground treatment works. State of the art real time monitoring system including total stations network, automatic tiltmeter and pressure sensor piezometer, was adopted by CYP.

With the detailed design on the treatment parameters and sequence with review in according to the ongoing monitoring analysis and interpretation, the vertical displacement associated with the ground treatment works was strictly controlled to within +/-5mm (the min. design Amber Trigger) even at the time constructing the largely inclined column (columns

with max. 32 degrees inclination) at cross passage CP02. Generally stable groundwater level recorded during the ground treatment period. Negligible ground displacement was recorded during the cross passages tunnelling excavation with no unexpected change in groundwater level recorded. This indicate the treatment block has achieved the design required parameters and permeability, hence, ground movement and groundwater change were well within the design estimation.

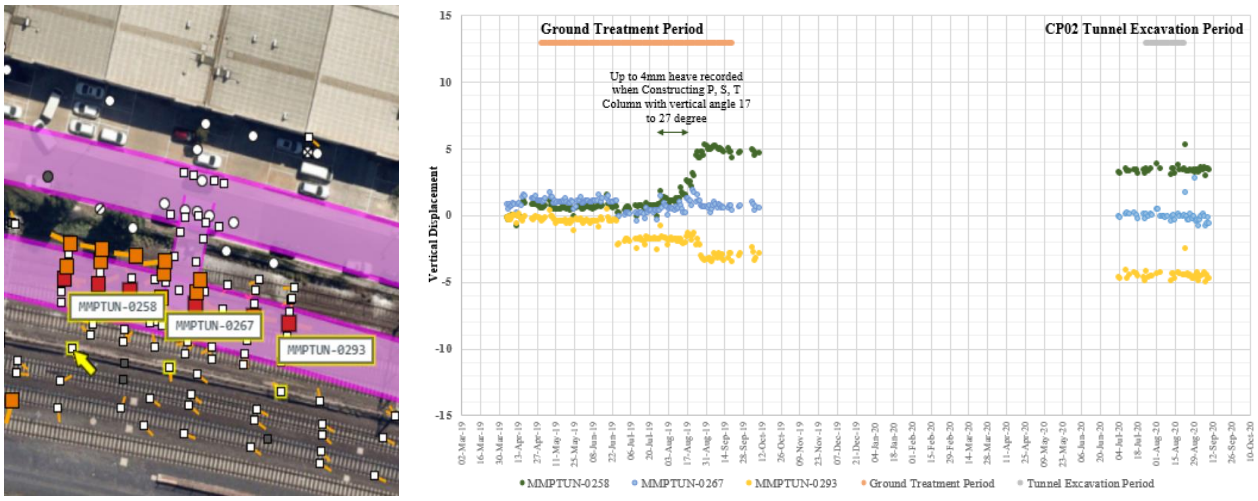


Figure 19: Vertical ground displacement during ground treatment and CP excavation at CP02 (automatic monitoring)

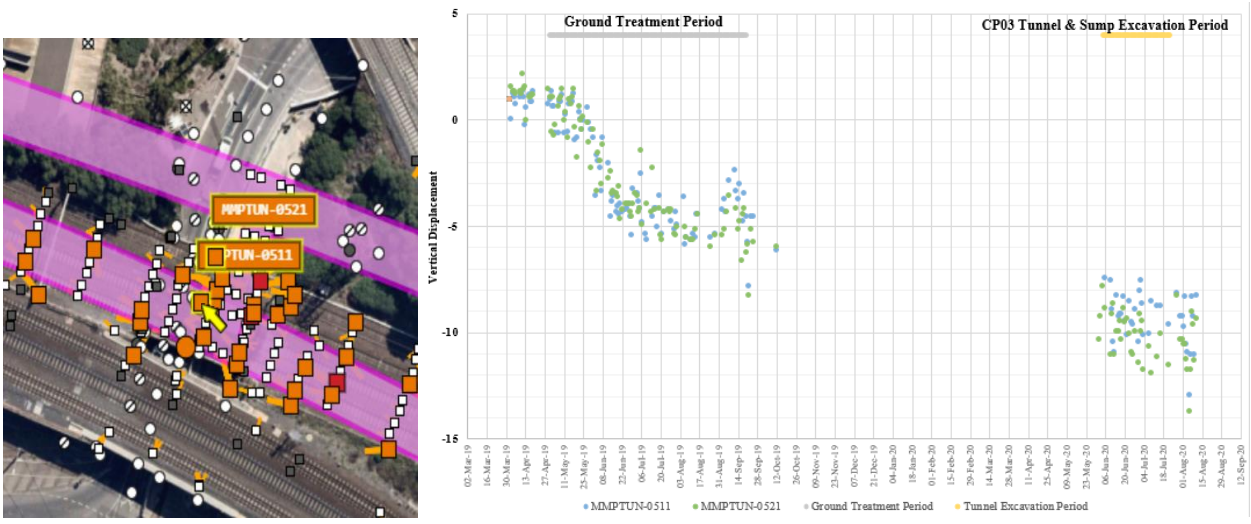


Figure 14: Vertical ground displacement during ground treatment and CP excavation at CP03 (manual monitoring)

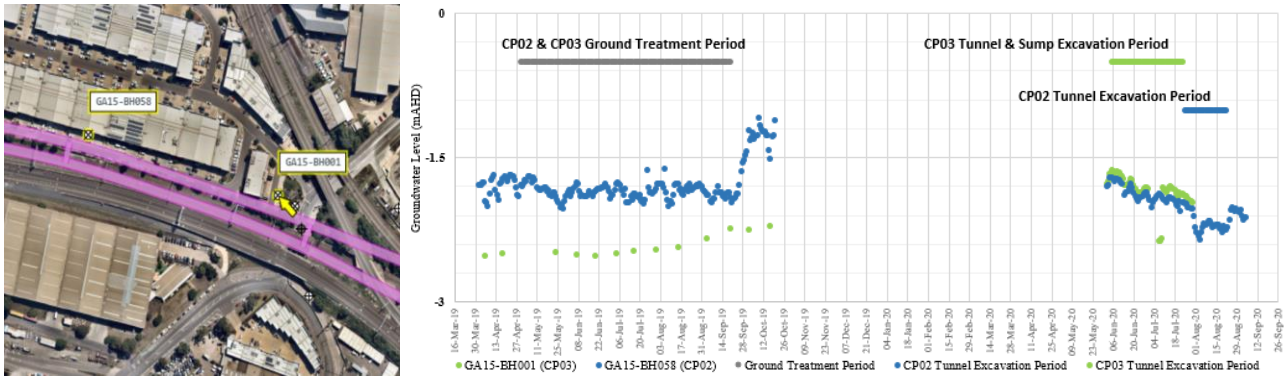


Figure 20: Change in groundwater level during ground treatment and CP excavation at CP02 and CP 03

5.2. IN TUNNEL MONITORING: PRIMARY LINING LOADING AND TUNNEL CONVERGENCE

Tunnel displacement monitoring points and strain gauges in the shotcrete lining were installed to monitor the performance during cross passage excavation. The opened ground conditions confirmed good quality of ground treatment with occasional minor water seepages which is completely manageable and within design allowable.
By analysing the loading recorded in the primary lining (Shotcrete layer) and tunnel convergence, the loading and convergence are both within any design trigger limits (Loading Amber Trigger: +/-1300kN; Convergence min. Amber Trigger: +/- 4mm). This indicates the treated ground strength achieve or even more favourable than the design assumption.

6 LESSONS LEARNT & CONCLUSIONS

The jet grout treatment carried out on the Melbourne Metro Cross Passages is a first of its kind for the Australian market and it gave the team the opportunity to implement some of the latest technological improvements available. The complex and variable geology required in-depth trial at each CP in order to define the specific range of parameters required at various depths. Further, the constraints imposed by adjacent infrastructure, buildings and services required the implementation of a range of real time monitoring of the ground behaviour as well as real-time digitalisation and 3D representation of the works performed in-situ. Indeed, it's only by applying this level of detail and frequency of measurement that the Contractor could ensure it had identified as many defects as possible and limit to a minimum the risks during excavation of the Cross passages. Notwithstanding this level of trialling, quality control and testing, it was agreed with the Main Contractor that additional investigatory probing and grouting would be carried out both from ground level and from the tunnel before excavation.

Whilst boring of tunnels through the treated ground and the consequent decompression of this ground may have left preferential flow paths along the contact between the tunnel lining and the treated ground, it was found that no cracks nor discontinuities in the improved ground was induced following the TBM drive.

The Main Contractor, it's Engineer, and the Subcontractor embarked on a collaborative partnership in which sharing of respective past experiences as well as in-depth technical assessment of the information available took place at Tender, Design and Constructions stages. This, in addition to a unique commercial arrangement that allowed for a fair distribution of the risks between all parties both during the works but also in the event of the occurrence of an incident set the foundations for the success of a project that probably could not have been built any other way.

7 ACKNOWLEDGMENTS

The author wishes to express his gratitude to Cross Yarra Partnership for providing the project information and Melbourne Metro Rail Authority for permission to publish.

Special consideration is also given to the whole team on the ground for the constant focus, very high level of professionalism and excellent quality of works carried out.

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THE BRIDGE BETWEEN CONTRACTORS AND CONSULTANTS

Idy Li

EIC Activities, Associate Principal – Geotechnics, idy.li@eicactiv.com, +61 403 417 680

ABSTRACT

For the past five years, I have been working as an employee of EIC Activities (a member of the CIMIC Group) as an in-house geotechnical engineering advisor providing technical support to CIMIC Group companies. This is a non-traditional role for a design engineer in the Australian civil infrastructure industry, as most design engineers work for consulting firms operating independently from construction companies and are engaged in project-specific contracts. Being in this unique role has allowed me to understand the operations of contractors, while building upon my technical skills acquired in the previous stage of my career as a geotechnical engineer in a more traditional role within a consulting firm. The perspectives I have gained from both forms of engagement have allowed me to effectively form a bridge between contractors and consultants.

One of the primary objectives of my current role is to contribute to creating value and minimising risk on projects and tenders. This objective influences the behaviours and approaches I take in my role. Using a number of case studies, this paper will discuss some of the actions and mindset my colleagues and I have adopted to fulfil this objective whilst bridging the contractor-consultant gap.

Innovation projects take place in addition to our existing project commitments to advance our knowledge in the pursuit of value creation and risk minimisation in the long term. Some innovation projects involve funded partnerships with universities to undertake research into challenges on construction projects. Such collaborations facilitate much needed dialogue between universities and the industry, and further cements our role as a bridge, in this case, between construction and academia.

1 INTRODUCTION

Conventional design and construct (D&C) contracts for large civil infrastructure works involve a head contractor signing an agreement with the Principal "project client" as the party bearing full responsibility for implementation of the project contract. The engineering skillsets required to implement such large-scale contracts are vast. Construction engineers or contractors will manage all aspects of delivery including, logistics, program management, and the physical construction of a project. Contractors frequently engage technical consultants to support delivery and address technical needs.

Conventional contractor-consultant engagements for D&C contracts in the civil infrastructure industry are formed on a project-by-project basis, whereby the consultant provides expert technical services to the contractor in a pre-negotiated, fee-for-service model. In return for the fee, the consultants' deliverables will be in the form of design recommendations and advice documented within reports and construction drawings. This engagement structure is drawn up diagrammatically in Figure 1 for D&C contracts. Note this paper focuses on D&C contracts only. Construct only and alliance type contracts will not be discussed.

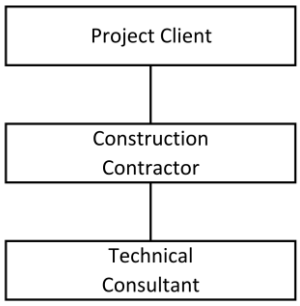

Figure 1: Conventional engagement model between contractors and consultants in "design and construct" contracts in Australian civil infrastructure industry

For the first ten years of my career, I was employed as a geotechnical engineer in consulting firms based in New Zealand and Australia. For large-scale projects, all consultant engagements with contractors were established in a similar format to that shown in Figure 1. This model is largely adopted on major civil infrastructure projects in Australia. From these projects and engagements I established the building blocks of my technical expertise.

I have since transferred away from conventional consulting firms, and for the last five years, I have been working with my current employer, EIC Activities (a member of the CIMIC Group), tasked with a geotechnical specialist advisory role. As in-house technical advisors, EIC Activities partner with CIMIC Group companies comprised of CPB Contractors, Leighton Asia, UGL, Thiess, Sedgman, Broad and Pacific Partnerships to provide engineering and technical advice and services. This engagement format is illustrated in Figure 2.

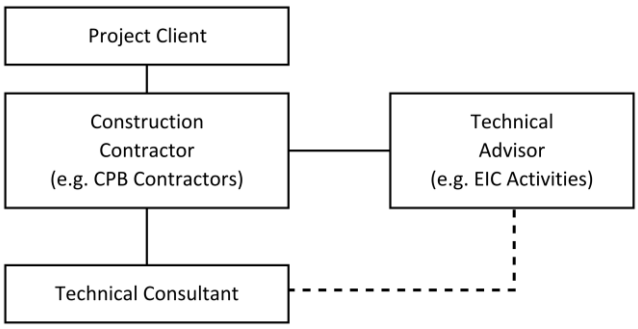

Figure 2: Engagement model with in-house technical advisor role supporting contractors

This paper will describe how I approach my unique role as an in-house technical advisor, allowing me to effectively form a bridge between contractors and conventional technical consultants to establish successful outcomes. I will be using a number of case studies to illustrate my perspectives.

2 PURPOSE OF TECHNICAL ADVISOR

As the party responsible for the implementation of a project, contractors have an obligation to ensure technical design solutions align with their motivators, such as constructability, cost, program and safety.

My role as a technical advisor is to facilitate design reviews with the contractor's core values and motivators in mind. I provide geotechnical expertise to support the contractor in ensuring the technical aspect of design and constructability are covered, allowing for a more holistic assessment of the solution from the contractor's perspective

Ultimately, the engineering and risk mitigation expertise provided by technical advisors, such as EIC Activities' is intended to provide contractors, such as CIMIC Group, with a competitive advantage for winning and delivering profitable projects that also generates value for clients.

3 GOALS OF TECHNICAL ADVISOR

3.1 VALUE CREATION AND RISK MITIGATION

One of the primary functions of technical advisors is to work alongside our contractor partners to create value and mitigate risk to establish a competitive advantage for winning and delivering profitable projects. Value creation can be defined as improving cost efficiency and can be realised through many avenues such as challenging assumptions, optimising whole-of-life solutions, using innovation to drive efficiency and productivity, and improving outcomes across the project and asset lifecycle in partnership with our contractors .

However, our motivations are not limited to reducing quantities and sizes to minimise expense. As geotechnical professionals we operate in a high-risk industry with many variables and unknowns. Being able to appreciate geotechnical uncertainties and construction constraints allows me to identify and quantify risks, thereby achieving the goal of "risk minimisation". Further, to attain the goal of project profitability one cannot focus solely on short term gains during the construction timeframe, but also potential risks in the long-term project life beyond the immediate construction and delivery.

3.2 COLLABORATION

The pursuit of value creation and risk mitigation is not a unilateral exercise which can be attained by the technical dimension alone. The true value of technical advisors is the ability to collaborate and integrate technical perspectives with the motivators held by the contractor to establish an optimal solution.

For example, I am often requested by contractors to provide a second opinion or alternate solution to existing designs. Upon receipt of the request, I will often seek clarification about the construction constraints, be it constructability, program, or budget and apply my technical design skills and experience to conduct an independent assessment. I will then engage in collaborative sessions with the consultant where we share our insights and opinions on the technical solution and construction constraints. This process ensures the solution generates the greatest value based on the motivators driven by the contractor.

Teamwork mentality is important to adopt with all the individual parties, including contractors and consultants. The purpose is not to find fault with others, but to provide the contractor with all the necessary tools to assess the solution from a technical perspective and foster dialogue between the technical design consultants and contractors.

Engineering design and construction of large-scale infrastructure is a significant and complex undertaking and it is important that all perspectives are considered. Often, as individuals we may not know all the limitations and earlier discussions which influenced the design to land where it has. It is our role as technical advisors to gather all the pertinent points and perspectives and bridge the technical expertise with the construction and design know-how to get to a best-for-project solution.

4 CAPABILITIES OF TECHNICAL ADVISORS

4.1 KNOWLEDGE AND EXPERIENCE

The fundamental requirement of a technical advisor is to have expert skills and knowledge in the appointed subject to complement the existing technical capabilities of a contractor.

However, technical skill alone will not suffice. Experience plays a large part in a technical advisor's role. It is the *integration* of technical solutions with the contractor's core engineering motivations to balance budget, program, risks and safety, which is paramount to the delivery of an optimal solution. For example, often, the most economical solution may not be feasible due to access constraints or program limitations. Experienced engineers have greater appreciation for the implications and integrations with stakeholders due to experience. Asking the right questions to uncover constraints is often key to finding the optimal solution, and that comes with experience.

It is for this reason, that the majority of the sector leaders performing advisory roles tend to have more than 25 years of experience, and are supported by a "top-heavy" structure, i.e. a large number of principal or senior engineers with 15 to 20 years of technical experience, each with their individual specialist areas of expertise, such as finite element modelling, seismic design, dam engineering, soft soils etc. An example of this is observed at EIC Activities.

4.2 CONSULTING BACKGROUND

In a similar fashion to my own career path, many of the technical advisors at EIC Activities have a background in consulting. Thus, we are familiar with the design processes, challenges, drivers and motivators held by most consultants. With this knowledge and our unique positions as technical advisors, we are well equipped to facilitate discussions around technical solutions between consultants and contractors.

5 MODES OF ENGAGEMENT

5.1 GENERAL

From my experience as a technical advisor, engagements on projects are varied and I am required to respond to the evolving needs of the business. For example, EIC Activities' technical advisory engagements and outputs vary substantially between each task.

I am often advising on numerous projects at one time, and the duration of my involvement may vary from a few days to a few months, sometimes more. The nature of my engagement is dynamic – I can be called upon to rectify issues quickly, or I may have a long-standing involvement on a project to undertake reviews on each design package. Technical advisors

can be engaged at all stages of a project, from the pre-tender stage through to construction and delivery. The duration and style of engagement will be guided by the needs of the contractor thus our response must be flexible to meet varying demands. The list below outlines the various modes and nature of engagement as a technical advisor.

- **Value engineering workshops.** In the early inception of a project, such as expression of interest (EOI), tender or commencement of detailed design, I have been requested to participate in value engineering workshops with a contractor where high-level discussions around alignment, and large-scale opportunities and risks are identified. These sessions allow the team to establish a holistic understanding of the most critical constraints and challenges on the project.
- **Design review.** At any stage during tender I may be requested to undertake high-level or detailed reviews of geotechnical related drawings or reports prepared by the design consultant. In this capacity, the contractor is typically expecting a check to ensure designs have been carried out in accordance with the specifications, and to identify potential opportunities or risks.
- **Ad hoc requests.** Ad hoc requests from contractors occur frequently. These will usually be initiated by the contractor who may be seeking additional technical insights and exploring whether the design is optimal or whether the solution carries excessive risk
- **Design management.** While I have not experienced this role myself, some technical advisors at EIC Activities are appointed by the contractor to perform design management duties on individual projects. This role allows us to have greater involvement throughout the design process.

6 APPROACH TO TECHNICAL ADVISOR ROLE

Some of the approaches technical advisors adopt are summarised below.

6.1 CONCEPTUAL FOCUS

As technical advisors, the success of our input can usually be achieved by interrogating conceptual design methodologies from a high level rather than undertaking an in-depth review. For example, if asked to challenge the design of a 500-metre-long retaining wall, a technical advisor may only undertake one or two independent assessments involving the critical wall heights and / or worst-case ground conditions. If the difference between the independent assessment and the original solution are significantly different, one would engage in discussions with the contractor and design consultant to understand the source of the differences and suggest whether a compromise or change to the solution would be required or accepted. In this example it is not necessary for the technical advisor to interrogate every drawing and design section along the 500m wall in detail; if the desired outcome is achieved by reviewing one or two critical sections, there is no need to review any more.

6.2 CONCURRENT DESIGN

One of the key imperatives of the consultants and contractors during the delivery phase is to settle on a workable design solution and progress it to completion as quickly as possible. A technical advisor's role is to explore potential alternative designs that can then be compared to the initial solution without disrupting the development of that initial solution. If the alternative proves to be better, then a change can be made. If not, then the design program is not affected.

So as not to hinder the delivery program, I will develop my alternatives concurrently, as the consultants develop and document their solutions. An example of concurrent design is shared in Section 7.1 below.

6.3 ALTERNATIVE PERSPECTIVE

In my role, I establish alternative perspectives for both the contractor and consultant and initiate dialogue towards solutions which would not have otherwise occurred. The contractor and consultant often have their own differing perspectives and by developing an understanding of both viewpoints, I am able to act as a bridge between both parties, as shown in the example documented in Section 7.2 below.

6.4 RISK BASED DESIGN

Very often, engineers are compelled to develop innovative solutions and / or multiple options for the purpose of assessing the cost variance between the differing options. However, there are instances where the pursuit of reducing costs will result in increased risk. There is an art to striking a fine balance between these two seemingly opposing set of motivators. As a technical advisor, I have developed alternative solutions and options for contractors which vary depending on the contractor's risk appetite. The risk assessment process involves the contractor being presented with a full set of the associated facts, opportunities and risks for each option which allows them to choose the most suitable

solution. This has allowed me to present more aggressive (cost effective, but higher risk) solutions when it comes to less critical works, such as smaller temporary works, but all solutions I present must ensure that the contractor is not exposed to undue risk and all safety requirements are met. To minimise the potential for unwanted consequences, the author develops contingency solutions in the event the risks are realised.

6.5 CONTRACTOR CONFIDENCE

The contractor's ultimate aim is to construct the most cost-effective and safe design which meets requirements. If upon review, technical advisors find no changes are necessary to the existing design solution, the contractor may be satisfied with this outcome, considering their advisors have provided them with confidence around the existing solution.

7 CASE STUDIES / OUTCOMES OF TECHNICAL ADVISOR APPROACH

7.1 CASE STUDY 1 – TRIFOLIATE SHAPED SHAFT OPTIMISATION

On the WestConnex M4 East project in New South Wales, EIC Activities were approached by the construction joint venture (CSJJV, CPB Contractors, Samsung and John Holland) to investigate the feasibility of optimising a ventilation shaft design in order to optimise program. The 60-metre wide by 47-metre deep shaft comprised an amalgamated form of three circular shafts, forming a trifoliate shape as shown in Figure 3.

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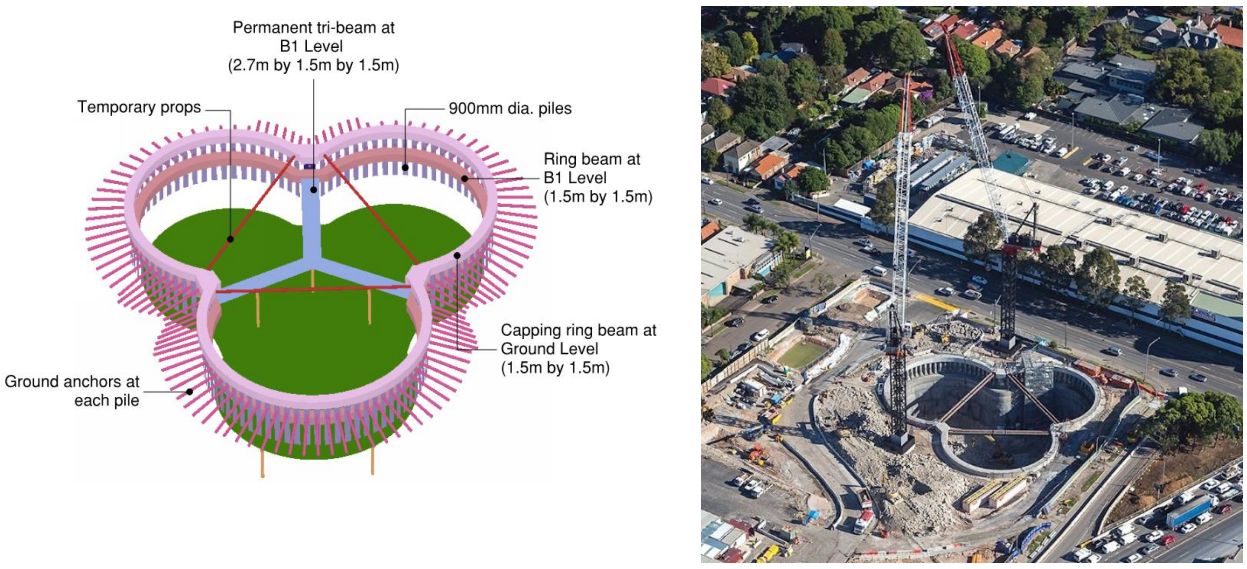


(a) EIC Activities' PLAXIS 3D model **(b) Aerial image of shaft construction**

Figure 3: Trifoliate-shaped ventilation shaft for WestConnex M4 East

The original retention solution proposed by the technical consultant included 900mm diameter reinforced concrete piles supporting the upper 10 metres of residual and medium strength rock, supported by two layers of temporary props and ground anchors in the short term. For the long term, permanent concrete tri-beams measuring 2.7 metres wide by 1.5 metres deep were proposed in place of the temporary props, at the ground level and at B1 level.

Given the shape of the shaft, it was clear that opportunities to optimise the strutting solution could be realised if three-dimensional soil-structure interaction modelling was adopted. However, the time and resources needed to undertake such modelling would be time consuming – and the benefits could not be quantified prior to the analysis.

Consequently, EIC Activities began pursuing alternatives using three-dimensional model simulations as construction continued to progress based on the original design. EIC Activities simulated the trifoliate structure in three-dimensional finite element software PLAXIS 3D, which allowed soil-structure interactions to be captured within this distinctly three-dimensional structure. Refer to Figure 3 for an illustration of the PLAXIS 3D model. Back analysis methods were used to calibrate the results of the three-dimensional model with those observed on site. The results from the three-dimensional

modelling concluded that the permanent tri-beam at ground level could be eliminated without any negative impact on the already-constructed elements and future structure.

This solution was not adopted due to program limitations, as the design change necessitated another round of lengthy design reviews. Nevertheless, this example demonstrates the benefits and potential opportunities which a technical advisor can provide without derailing an existing design schedule

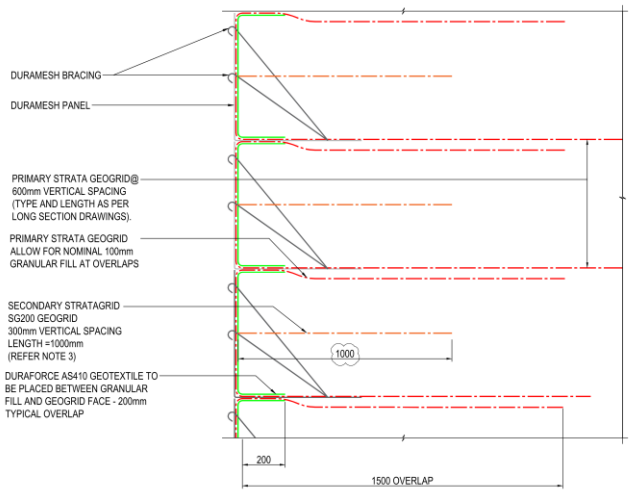
7.2 CASE STUDY 2 – MECHANICALLY STABILISED EARTH WALL CONSTRUCTION

On the Baypark to Bayfair Link Upgrade project in Tauranga, New Zealand, the consultant requested the contractor to remove a partially constructed MSE (Mechanically Stabilised Earth) wall, as the vertical spacing between the installed geogrid layers had exceeded the maximum vertical offset stipulated in the drawings by 100mm. The contractor raised an RFI (Request for Information) requesting the wall be accepted "as-is" as the deviation was considered to be within allowable construction tolerance. The consultant maintained the wall be taken down and re-built as the design solution was sensitive to the vertical offset. An example of an MSE wall on the project is shown in Figure 4.

I was contacted by the contractor for my opinion, whereupon I reviewed the design calculations in detail to assess the governing factors and elements of the design. I found that the "sliding" failure mechanism was governing the solution, and the consultant's comments were justified – the calculations showed little to no room for exceedance of the vertical geogrid spacing. However, I also recognised the resistance to "sliding" failure was also contributed to by the *length* of the geogrid upon which the sliding mechanism would occur. In my experience, installed geogrid lengths on site were likely to exceed the prescribed minimum design lengths as contractors tend to install beyond the minimal lengths to minimise handling and cater for rounding errors.

I confirmed my assumptions with the contractor and reviewed the as-built geogrid lengths which were found to exceed the minimum design lengths by up to one metre. Armed with this information, the contractor presented my findings and records of the as-built geogrid lengths to the consultant, who retracted the request to rebuild the MSE wall.

This example demonstrated the benefit I was able to provide through bridging the design considerations of the consultant and constructed conditions by the contractor.



(a) MSE geogrid drawing detail **(b) Partially constructed MSE wall**

Figure 4: MSE walls on the Baypark to Bayfair Link Upgrade project

7.3 CASE STUDY 3 – DRIVEN PILE SPECIFICATIONS

Construction of the Waikeria prison facility in New Zealand involved the installation of over 3000 units of 20-metre-long closed-end circular driven piles. A trial procedure including 22 piles spread across the site was undertaken with dynamic testing prior to production. The purpose of these trials was to establish the pile acceptance specification during production, such as a minimum target end-of-drive (EOD) set of 9mm. During production, the achieved end-of-drive sets for most of the piles were significantly higher than those recorded during the trial procedure, thereby failing the pile acceptance criteria. The consultant requested additional restrikes and pile tests, such as dynamic testing and pile wave analysis to verify the supporting capacity of the installed piles. All the additional testing proved the piles had the required capacity.

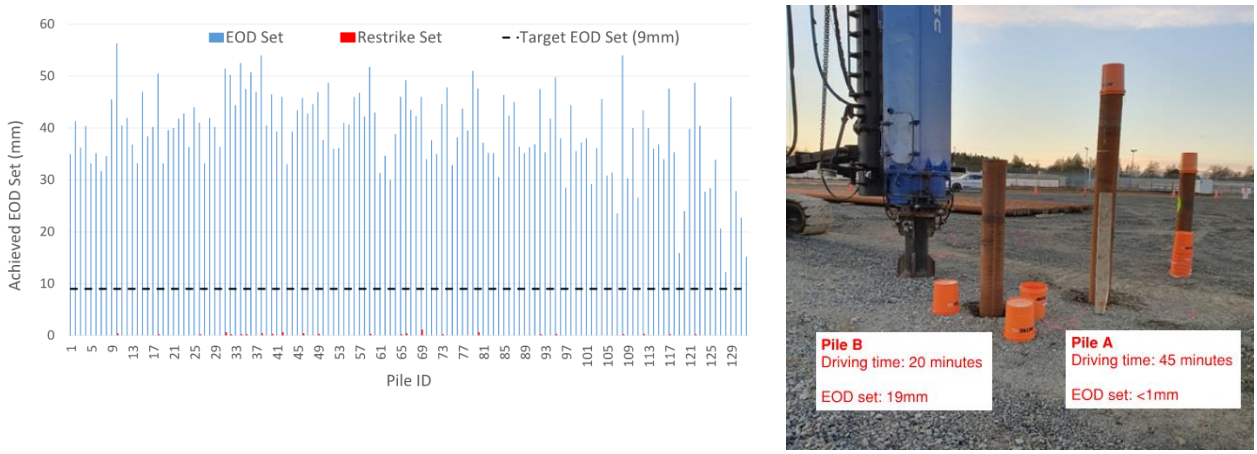
However, the time taken to undertake the additional restrikes and tests caused significant delay to the contractor's tight construction program.

I was requested by the contractor to review the situation and expedite the process. Initially, it was difficult to comprehend why all the piles were failing the acceptance criterion established during the trial procedure – the trial piles had been strategically located to cover most of the site, in an effort to capture varying ground conditions. The same piling crew had been engaged to undertake the trial piles and production piles; we interrogated the production piling records which were installed immediately adjacent to trial piles and there were no obvious differences between the piles, yet their achieved end-of-drive set differed substantially. At one location, the achieved end-of-drive set was 1.7mm for the trial pile and between 16mm to 40mm for the surrounding production piles.

Upon interrogation of the piling records, I noticed one observable difference between the production piles and trial piles – the driving times for the trial piles were substantially longer. On average, the production piles were installed in 13 minutes, versus 39 minutes for the trial piles. A slower rate of advancement was implemented during the trial procedures, as the piles were tested at multiple stages during the course of installation. By contrast, the no tests were undertaken on the production piles, until the desired termination level had been reached.

Despite the difference being only a matter of minutes, I postulated the difference in driving time had allowed the ground to achieve greater set-up i.e. increased pile capacity due to pore pressure dissipation, during the trial procedure. To verify this postulation, I requested the construction team drive two separate piles immediately adjacent to each other on site at a number of locations, each of identical length and size, but at varying installation rates. At each of the test locations, the piles driven at a slower rate achieved significantly smaller sets. For example, at one location, shown in Figure 5, Pile A was installed over 45 minutes, and Pile B installed in 20 minutes. The achieved EOD at the target pile length was <1mm and 19mm respectively. Thus, I concluded the reduction in driving time during production had not allowed the same degree of setting-up to occur as that during the trial procedure, causing the production piles to "fail" the acceptance criteria.

Once the discrepancy had been identified, I collaborated with the consultant to establish a revised set of pile acceptance criteria, including increasing the end-of-drive set to 16mm, which was tailored to the actual rates of installing the production piles. The revised criteria allowed a greater proportion of the piles to be approved without necessitating additional testing to verify the supporting capacity of the pile. Construction was able to progress at a quicker rate, much to the contractor's satisfaction.



(a) Summary of achieved EOD set for a building illustrating achieved EOD sets >>9mm

(b) Comparison of achieved EOD sets between piles installed at different rates

Figure 5: Waikeria Prison driven pile records

8 INNOVATION PROJECTS

Innovation projects take place in addition to our existing project obligations at EIC Activities to advance our knowledge in the pursuit of value creation and risk minimisation in the long term. Some innovation projects involve funded partnerships with universities to undertake research into existing challenges on construction projects. Such collaborations

facilitate much needed dialogue between universities and the industry, and further cements our role as a bridge, now between academia and construction.

For example, I am currently working on a research project in collaboration with University of Technology Sydney (UTS). The project was set up to investigate landfill behaviour due to the increasing number of projects involving construction around and upon existing landfill sites.

The innovation project aims to establish a more reliable set of testing procedures and variables for landfill engineering design and predictors of landfill behaviour. One aspect of the research involves the collection of landfill samples from a project site in NSW for testing in the university laboratory. Whilst on site, a large embankment has been constructed on the landfill and its performance is monitored closely with an intensive instrumentation schedule. The results of the laboratory testing will be compared against the site records to verify the reliability of the developed parameters established in the laboratory.

9 CONCLUSIONS

As an in-house technical advisor for a group of construction contractors, I use my technical experience and background to complement the contractor's existing capabilities such that holistic assessments can be made and optimal solutions can be developed. The main driving force in my role is creating value and minimising risk to the client and contractor so they are better able to achieve their outcomes. The mindset I adopt in my role is one of collaboration and teamwork with all the individual parties, including contractors, consultants and universities. My purpose is to foster dialogue between the technical design engineers and construction teams. I see myself as part of a team working towards a common goal of creating a successfully engineered structure. My role is to bridge technical expertise with the construction and design know-how to get there.

10 ACKNOWLEDGEMENTS

The author wishes to thank CIMIC Group clients and CPB Contractors construction teams from WestConnex M4 East, Baypark to Bayfair Link Upgrade, and Waikeria Prison for their efforts in delivering these projects, and their approval to have the content of this paper published. The author also wishes to thank my manager Dr Jeff Hsi for reviewing this paper and providing invaluable guidance to the Geotechnics team at EIC Activities.

INTERNAL COMPRESSION OF FILL MATERIAL ORIGINATING FROM BRINGELLY SHALE

Theva Muttuvel[1], Richard Kelly[2], David Malorey[3], Esteban Litvin[4] and Jubert Pineda[5]

[1]Principal Geotechnical Engineer, SMEC Australia Email: theva.muttuvel@smec.com Phone: +61 2 9900 7149

[2]Chief Technical Principal and General Manager Technical Excellence, SMEC Australia Email: Richard.Kelly@smec.com Phone: +61 7 3029 6625

[3]Technical Principal, SMEC Australia Email: David.Malorey@smec.com Phone: +61 2 9925 5558

[4]Engineering Services Design Manager, Ferrovial Agroman, Email: elitvin@ferrovial.com Phone: +61 2 8736 9600

[5]Senior Lecturer, University of Newcastle Email: jubert.pineda@newcastle.edu.au Phone: + 61 2 4921 7034

ABSTRACT

A rule of thumb handed down from senior to junior geotechnical engineers in Australia is that internal compression of embankment fill is 0.1% of embankment height and it has been widely used in performance specifications for major infrastructure projects around Australia. It is not clear where this rule of thumb originated from or on what basis it was developed. In Western Sydney this rule of thumb has been adequate for many years because the scale of earthworks in terms of fill thickness has been relatively minor. Large scale earthworks have started to occur over the past decade or so as more significant road and rail development has occurred. Recent experience with higher fills constructed in Western Sydney shows that internal compression strain rate can be greater than 0.1% per log cycle of time. Embankments to about 10 m height on a rail infrastructure projects on Western Sydney were constructed from Bringelly Shale. The fill materials and compaction were compliant with the relevant engineering standards. Comparison of topographic survey about 5 years after construction with design profiles indicated that they had settled between 0.05 and 0.25 m, subject to construction tolerances, and the internal compression strain rate varied between 0.5% and 6.3% per log cycle of time, adopting 1 year as the starting time of post construction settlement. Anecdotal evidence from road embankments of up to 14 m have identified similar magnitudes of settlement response from fill formed of the same Bringelly Shale materials. These values are much higher than the rule of thumb of 0.1% per log cycle of time. Though Bringelly Shale-based fill material has shown such a significant settlement issue, to the authors' knowledge, there are no references found in the technical literature that provides some guidance on assessing the internal compression of Bringelly Shale based fill material. Therefore, a series of laboratory tests have been conducted to understand the settlement behaviour of compacted Bringelly Shale. Compaction tests, particle size distribution, Atterberg limits, and small and large-scale compression tests of compacted fill material have been conducted. This paper summarises some of the findings of the laboratory tests and authors' view on the performance of Bringelly Shale fill material and future studies.

1 INTRODUCTION

Ongoing post construction settlement of compacted fill material originated from Bringelly Shale has been observed in major infrastructure projects in Western Sydney region, causing significant maintenance cost. Embankments to about 10 m height on a rail infrastructure project were constructed from Bringelly Shale. The fill materials and compaction were compliant with the relevant engineering standards. The measured settlement after five years indicates that the internal compression strain rate varied between 0.5% and 6.3% per log cycle of time as shown in Figure 1, assuming that the start time for settlement was 1 year. These values are much higher than a typical internal compression strain rate of 0.1% per log cycle of time for engineered fill. In addition, rates of settlement recorded for settlement markers installed after completion of construction were approximately linear with the logarithm of time. The strain at the location of the settlement markers was up to 1.2% of embankment height over the period of about four years since these markers had been installed after construction. The settlement and rate of settlement appear to reduce as the embankment height increases. This data contradicts the published literature that settlement is proportional to embankment height or applied stress (Hopkins and Beckham, 1998; Waddell and Wong 2005), but is most likely a result of higher fills existing adjacent to structures, hence are more heavily compacted than away from structures and utilise select fill rather than general fill.

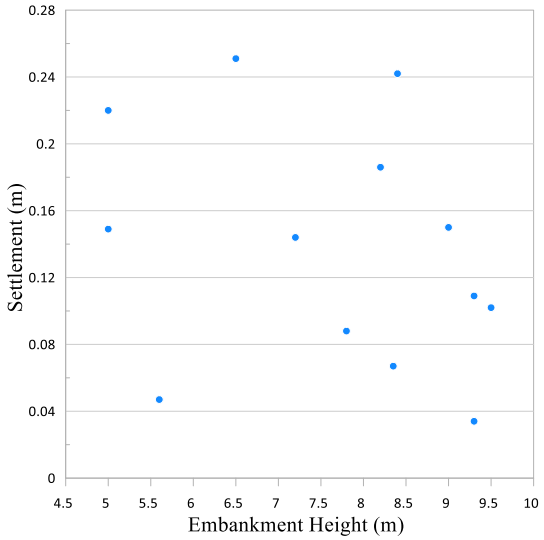


Figure 1(a): Change in embankment level over 5 years

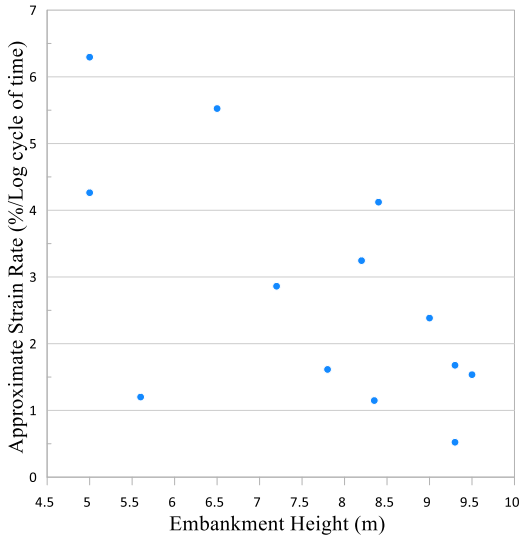


Figure 1b: Approximate strain rate over 5 years

Similarly, embankments constructed for major road infrastructure in Western Sydney have experienced settlement performance well beyond the rule of thumb magnitudes. In one example from two 14 m high embankments, 200 mm of surface settlement was observed in the 8 years post construction of the embankment. As these examples were located beneath pile raft foundations, no external vertical loading of the embankments was influencing the settlement behaviour. Samples extracted from this project were tested in the laboratory and compression of the order of 2 - 4% of the sample height were observed equating to the potential for the overall embankment compression to reach between 280 mm to 560 mm.

Western Sydney is underlain by Bringelly Shale. Bringelly Shale is interpreted by Herbert (1979) as a coastal alluvial plain sequence which grades up from a lagoonal-coastal marsh sequence at the base to increasingly more terrestrial, alluvial plain sediments towards the top of the formation. Lithologically, it comprises sequences that can be listed in order of decreasing volumetric significance as (1) claystone and siltstone (2) laminite (3) sandstone (4) coal and highly carbonaceous claystone.

Very little information is available in the literature regarding the engineering behaviour of Bringelly Shale fills. William (2004) and William & Airey (2005) provide basic properties of Bringelly Shale. They report that the mineralogy of Bringelly Shale comprises illite, mixed illite-smectite, kaolinite and quartz as major constituents and the slake durability of the material ranges from very low when weathered to medium when fresh. William (2004) also provides engineering parameters for intact rock and for reconstituted slurry material.

With the limited information provided in literature and authors' observations, Bringelly Shale particles break down due to changes in moisture content, causing settlement and such change may occur over a period.

Some evidence of long-term degradation of Bringelly Shale is provided in Figure 2. The figure shows a stockpile of Bringelly Shale, extracted from one of the brick quarries along Elizabeth Drive, that originally comprised cobble sized particles and has degraded over time to gravel size particles and finer. This degradation behaviour may contribute to the settlement of Bringelly Shale fill.

Figure 2: Observed degradation of Bringelly Shale

Pineda et al (2014a and b) studied the degradation of claystone due to environmental effects as those induced by stress relief (unloading) and the subsequent exposure of the rock to cyclic changes in relative humidity (RH). They demonstrated that the application of RH (suction) cycles causes a progressive degradation of the claystone. Degradation was quantified in terms of the accumulation of irreversible swelling strains (due to the expansion of clay minerals as well as micro-cracking), increase in the maximum water retention capacity, reduction in rock stiffness, increase in rock compressibility and permeability and reduction in shear strength parameters. Rock degradation increases with the number of RH cycles, increases further with the increase in the amplitude of RH, and reduces with the increase in the stress level. This is supported by observations from quarried batters in Western Sydney that experienced significant degradation of the formed surface in limited periods of time (circa 5 years) as illustrated in Figure 3 below.

Figure 3: Rock mass degradation on exposure in quarried batter

Research findings from Hopkins and Beckham (1998) in claystone and limestones indicated that major problems occurred in Shale fills because in many cases they tend to degrade from a hard, or indurated mass, to a fine-grained mass of soil. The degradation causes settlements and reductions in material strength. They reported that degradation of shale particles in embankments frequently occurred over a long period of time and many problems did not occur until several years after construction. They also reported that slope movement in shear has contributed to long term settlement and recommended to achieve a minimum long-term factor of safety of 1.5 to control settlement.

As discussed above, published literature indicates that shale particles shall break down when moisture movement either by direct wetting or change in humidity around the particles occur. For the purpose of this paper settlement caused by this mechanism has been classified as hydro-compression. It is expected that creep of engineered fill is likely to be minimal. However, it is difficult to differentiate creep and hydro-compression induced settlement in real world application. Therefore, for the purpose of this paper, both hydro-compression and creep has been termed as "internal compression".

Cox (1978) reports results from fully inundated oedometer tests performed on a claystone aimed at assessing how much the fill compresses or swells as a function of density and stress. Data from Cox (1978) is re-interpreted and presented in Figure 4. The data shows that the fill compresses at low density and low load as well as at higher density and higher load. As the density increases the fill has a propensity to swell at all loads. It is not clear whether these data include both hydro-compression and creep as the time-movement histories are not provided in the paper. Data from Waddell and Wong (2005) and unpublished compression test data from relevant construction projects are also plotted on Figure 4. Data from Hopkins and Beckham (1998) from a trial embankment constructed in Kentucky from non-durable shale are also shown in Figure 4. This embankment had internal compression of about 0.8% over about a 10-year period. However, the Hopkins and Beckham (1998) data plot in the swelling section of Figure 4. This demonstrates that accurate definition of compression or swelling is specific to a given material and possibly due to other factors such as moisture content with respect to optimum moisture content and clay mineralogy. Hopkins and Beckham (1998) also reported that relatively high embankment experienced internal compression even when material was well compacted. These settlements are considered partly due to slope movement and may be another reason why material classified under swelling zone in Figure 4 experience settlement.

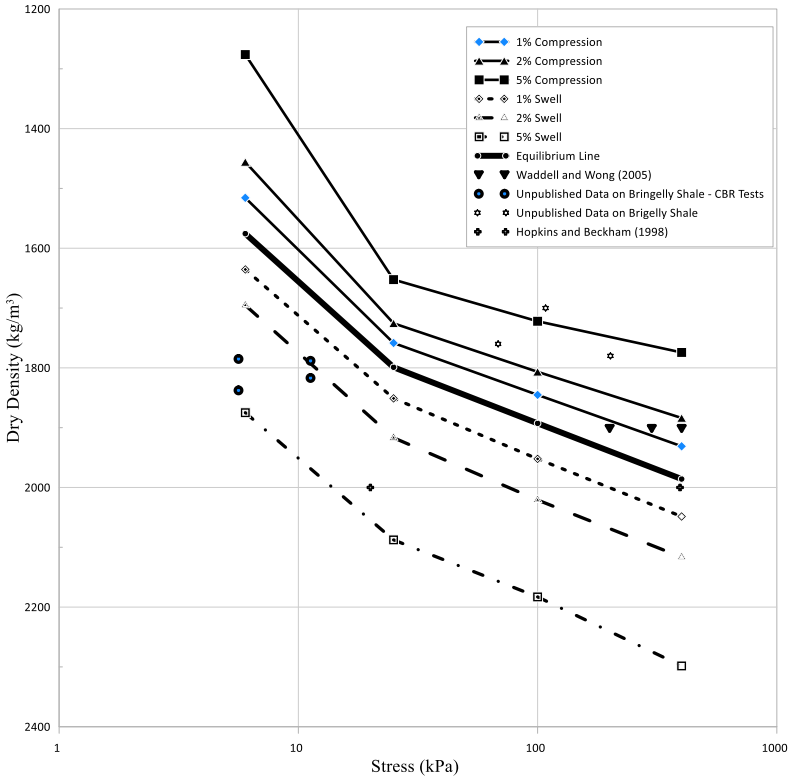


Figure 4: Re-interpretation Cox (1978) data

While Figure 4 appears to provide a means of estimating volume changes in fill it strictly only applies to the conditions adopted in the tests performed by Cox (1978). Different conditions would change the volumetric behaviour. For example, Alonso et al (2011) report soils compacted dry of optimum tend to have a more open void structure than soils compacted

at or wet of optimum which can collapse on subsequent wetting. Therefore, two samples compacted to the same density and subject to the same vertical load could deform differently depending on the compaction moisture content.

Cardoso et al (2012) reported experimental test data on a claystone used in construction of a motorway embankment in Portugal. They found, like Cox (1978), that there are zones of compression and swelling when the data is plotted as a function of void ratio and stress. Fills that are heavily compacted such that they exist in the over-consolidated zone with respect to a saturated normal consolidation line tends to swell when inundated and fills that are relatively loose with open void structure that exist above a saturated normal consolidation line tend to collapse. Cardoso et al (2012) explained this behaviour in the context of the unsaturated Barcelona Basic Model (Alonso et al, 1990).

Cardoso et al (2012) also reported that whether or not a material shrinks, or swells is a function of grading as well as density. They found that heavily compacted samples and samples where voids were filled with fines tended to swell whereas uniform grading tended to compress. Intermediate behaviour where swelling initially occurred followed by compression was found at intermediate density and grading states.

Waddell and Wong (2005) performed long term oedometer tests on weathered and fresh Ashfield Shale. Their samples were compacted to 98% standard maximum dry density at 1% dry of optimum moisture content. The samples were then loaded and allowed to compress without inundation for several months. The samples were subsequently inundated and monitored for several more months. Results from these tests showed that compression prior to inundation was a function of applied load recording 4% at 200 kPa and 6% at 400 kPa. Results presented by Waddell and Wong (2005) show that samples compressed on inundation. However, the more lightly loaded sample subsequently swelled. This behaviour is consistent with that observed by Cox (1978) if it is assumed that the lightly loaded sample exists in the swelling side of the line of equilibrium and the more heavily loaded samples exist on the compression side of the equilibrium line. Waddell and Wong (2005) interpret compression post about 100 days as linear with the logarithm of time and report creep strain rate as a function of vertical stress in MPa. The creep coefficient was larger for weathered shale than for fresh shale. The creep coefficient for weathered Ashfield Shale was reported to be 0.027 times the vertical stress in MPa. If this is applied for an embankment height of 5 m to 10 m constructed in the rail infrastructure project discussed previously, internal compression strain rate will be 0.27% to 0.54% per log cycle of time. Whereas the observed movement in this project is not following a typical trend of increasing in settlement with embankment height, the measured internal compression strain rate of Bringelly Shale based on monitoring data from this rail project is several times higher than that reported by Waddell and Wong (2005) for Ashfield Shale.

The literature suggests that whether or not a shale material compresses or swells, their magnitudes and time rates of volume change, is a function of its mineralogy, density, load, grading, compaction moisture content, and subsequent changes in moisture content. While trends in qualitative behaviour are reasonably clear, methods for quantification of the magnitudes of settlement and time rate of settlement for engineering purposes is less clear.

From an engineering point of view, the aim is to control the magnitude and rate of volumetric changes in a fill at least cost. Transport for NSW issued technical note TN033 in 2016 in which they noted that expansive material with similar properties to Bringelly Shale is considered unsuitable and should not be used in fills. The note requires conventional material suitability tests to be performed and allows the use of expansive materials blended with non-reactive soils for use as general fill. Expansive material can also be chemically stabilised to improve its properties. Both of these approaches are expensive and will increase the cost of construction in Western Sydney. An alternative approach is to use earthworks processes to limit deformations. Hopkins and Beckham (1998) report a process of compacting in maximum 200mm thick lifts, compacting at a moisture content of optimum +/-2% using a heavy rotary disc to mix soils, compact to minimum 95% standard maximum dry density using heavy compactors with a minimum of 3 passes of a static roller (minimum 27.4 Tonne) and 2 passes of a vibratory roller (minimum 24.9 Tonne). It is also recommended to apply adequate water to slake the material and promote breakdown and achieve majority of settlement during construction. The aim of this process was to accelerate breakdown of materials through mechanical means and through use of relatively high-water contents with an intention to keep the moisture content on wet of optimum as much practically as possible. In addition, they recommend a batter slope of 2.5H:1V to 3H:1V with a minimum factor of safety of 1.5 required to reduce the long-term settlement caused by slope movement with height more than 9 m. This method was reported to reduce long term settlements of embankments up to 20 m high from 0.3 m - 0.9 m in 10 years to 0.15 m - 0.18 m in 27 years. BRE (1998) provide similar recommendations for low rise buildings constructed on clay fills to reduce the potential for collapse compression. BRE (1998) recommend that fill be placed at densities ranging approximately between standard and modified maximum dry densities with air voids less than 5% (i.e. wet of optimum moisture content).

Although Bringelly Shale-based fill material has shown significant settlement at the rail and road infrastructure projects in Western Sydney as discussed above, to the authors' knowledge, no references have been found in the technical literature that provide some guidance on assessing the magnitude and time rate of internal compression of Bringelly Shale based fill material or how internal compression can be controlled.

Based on observations and literature discussed above, authors consider that internal compression of compacted Bringelly Shale fill is likely to be caused by moisture movement. This moisture movement can be:

- Direct wetting (Soaking): This is applicable to situation where free water movement occurs (e.g. earthworks in touch with water at bottom and top layers such as quarry filling and narrow embankments). This process will change overall moisture content of the fill.

- Moisture movement within compacted mass (i.e. at compacted moisture content – no soaking): This is applicable to situation where overall moisture content doesn't change due to external water movement (e.g. large landform constructed above ground surface). However, moisture content may not be uniform within compacted fill after placement (though overall moisture content of compacted fill is constant), hence moisture movement/humidity changes is likely to occur over time to reach a uniform moisture content.

Observed settlement of embankments in the rail and road infrastructure projects could be due to both mechanisms discussed above. Studying combined effects of these two mechanisms is very complex. Hence, a laboratory program has been developed to understand these mechanisms separately. A series of laboratory tests has been conducted to understand the settlement behaviour of compacted Bringelly Shale fill. Compaction tests, particle size distribution, Atterberg limits, and small and large-scale compression tests of compacted fill material have been performed. This paper summarises and discusses some of the findings of these laboratory tests.

2 LABORATORY TESTING PROGRAMME

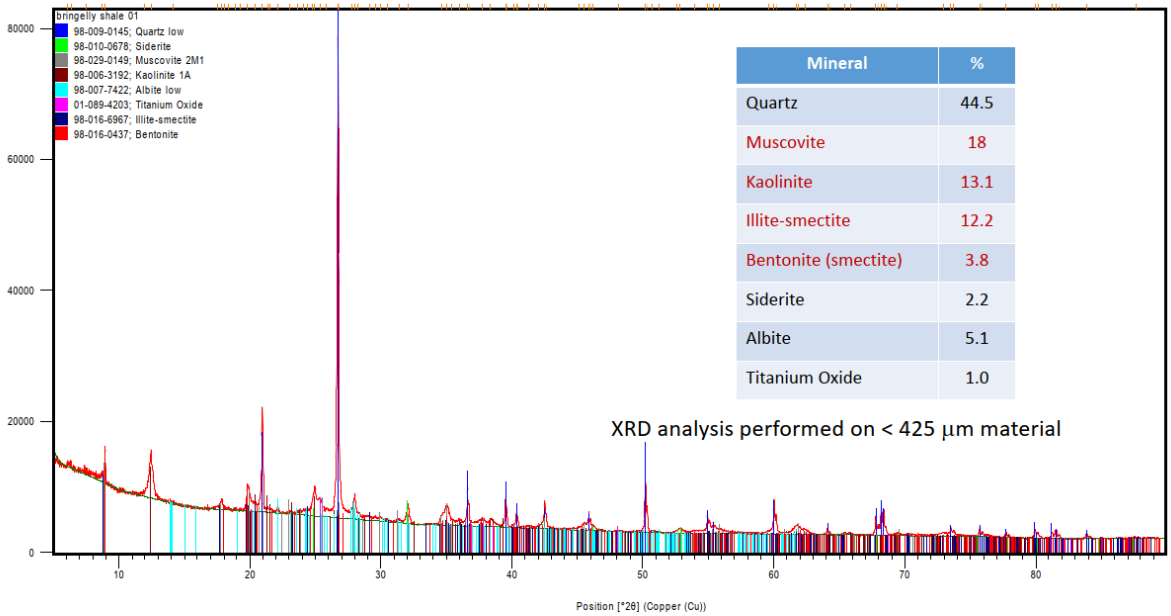
2.1 MATERIAL PROPERTIES

Shale material for testing has been sourced from a site in the Western Sydney and is classified as shown in Table 1. The shale was classified into BS2 and BS3 type weathered rock materials and residual clay.

Table 1: Material classification of Bringelly Shale

Strength Description	Point load index Is (50) MPa		Material Type	Material description
	From	To		
Residual clay	-	-	Soil	Soil derived from complete weathering of Bringelly Shale or dyke material
Soil Strength to Very Low Rock	-	0.03	BS3	Bringelly Shale: Interbedded/interlaminated sandstone and siltstone, and claystone
Low to medium	0.1	0.3	BS2	

A quantitative X-Ray diffraction analysis was carried out on BS2 material using rock powder sieved through 75 μm sieve. The X-Ray spectrogram is shown in Figure 5. A quantitative analysis indicates that it is composed by quartz (44.5 %), muscovite (18%), kaolinite (13.1 %), mixed illite-smectite (12.2 %), smectite (3.8 %), siderite (2.2 %), and albite (5.5 %). This represents around 47.1 % of clayey minerals. The results indicate less clay minerals than reported by William (2004).



Mineral	%
Quartz	44.5
Muscovite	18
Kaolinite	13.1
Illite-smectite	12.2
Bentonite (smectite)	3.8
Siderite	2.2
Albite	5.1
Titanium Oxide	1.0

Figure 5: X-Ray diffraction test conducted on BS2 material

Particle size distribution of residual clay, BS3 and BS2 are provided in Figure 6. In general, residual clay and BS3 consists of higher fine content than BS2. Particle size distribution of residual clay, BS3 and BS2 materials have been estimated according to the AS 1289.3.6.1. Particle size distribution curves are provided in Figure 6 where two PSD curves are shown for BS2. They represent the two different batches of samples used in the laboratory testing campaign. Sample 1 displays a fine fraction (< 75 µm) around 4% whereas it increases to 27% in Sample 2. Material BS3 and residual soil show fine fractions of 84% and 80%, respectively.

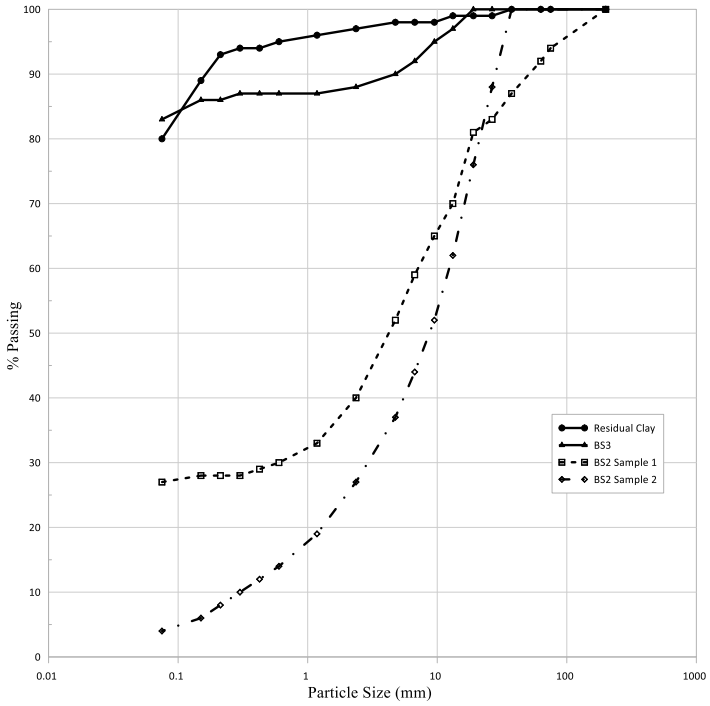


Figure 6: Particle size distribution of residual clay, BS3 and BS2

Liquid limit and plastic limit tests have been carried out to evaluate the plasticity of the tested materials. Atterberg limits were estimated following the procedure described in AS 1289.3.1.1, AS1289.3.2.1 and AS1289.3.3.1. Test results are summarised in Figure 7. Results indicate that plasticity of BS2 material is lower than BS3 and residual soil. This is attributed to the fact that BS2 has lesser fines than BS3 and residual clay. The residual soil classifies as high plasticity clay (CH) whereas materials from BS2 and BS3 are both low plasticity clays (CL).

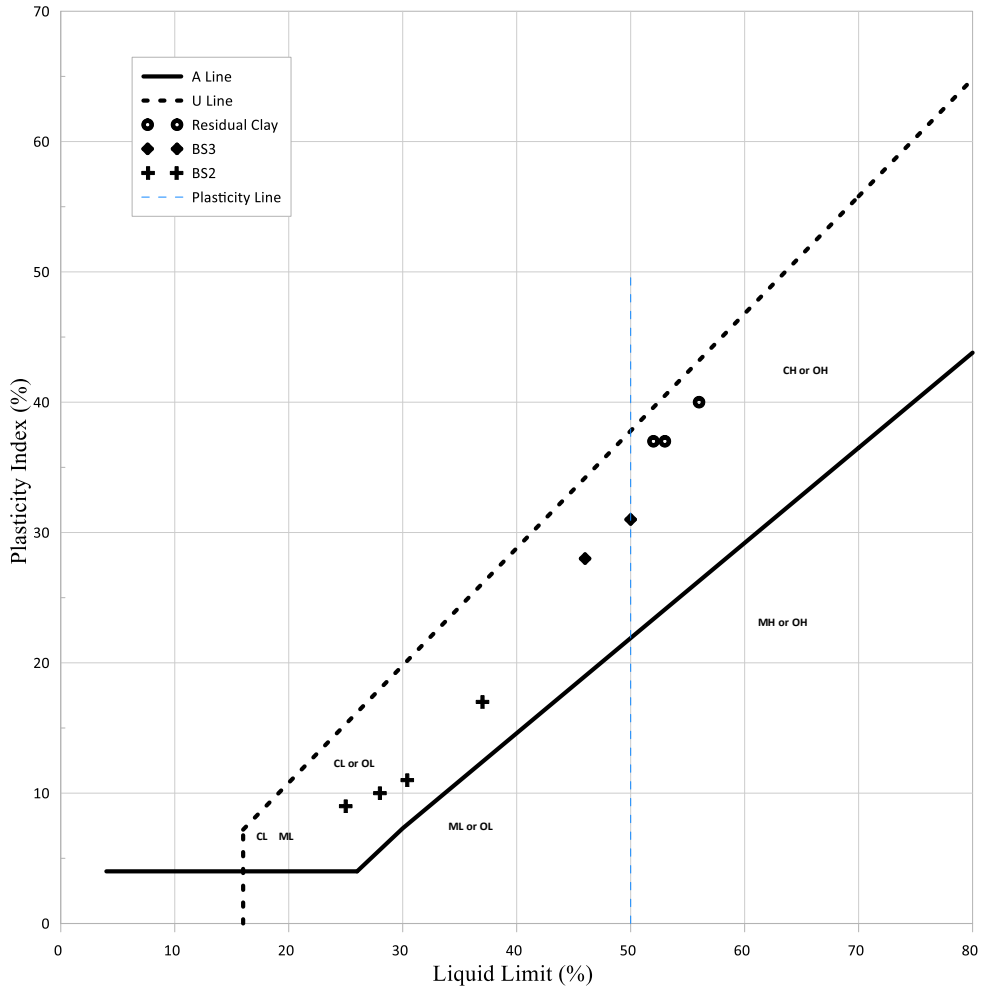


Figure 7: Atterberg limits of residual soil, BS3 and BS2

Standard and Modified Proctor compaction tests have been conducted on residual clay, BS3 and BS2 materials. Standard and Modified Proctor compaction curves were estimated following the procedure described in AS 1289.5.1.1 and AS1289.5.2.1, respectively to interpret maximum dry density (MDD) and optimum moisture content (OMC). The maximum dry density and optimum moisture content values are provided in Table 2. Maximum dry densities of BS3 and residual clay are lower than BS2 material while the optimum moisture content is higher than BS2 material. This is expected considering the increased fine content in BS3 and residual clay than in BS2.

Table 2: Compaction test results for residual clay, BS3 and BS2

Material Type	Number of tests	Compaction type	Maximum Dry Density, t/m^3	Optimum moisture content (%)
Residual Clay	3	Standard Compaction	1.74 ± 0.006	17.2 ± 0.60
	3	Modified Compaction	1.91 ± 0.010	11.95 ± 0.55
BS3	3	Standard Compaction	1.81 ± 0.035	15.35 ± 1.45
	3	Modified Compaction	1.99 ± 0.007	9.95 ± 0.45
BS2	2	Standard Compaction	2.09 ± 0.015	10.0 ± 1.5
	2	Modified Compaction	2.16 ± 0.020	6.65 ± 0.35

2.2 STUDY VOLUMETRIC BEHAVIOUR OF BRINGELLY SHALE

Two testing regimes have been carried out to study the volumetric behaviour of Bringelly Shale fill material a) under direct soaking (external water application) and at as-compacted moisture content (no external water application) as discussed below.

2.2.1 Testing Regime 1 – Study on volumetric behaviour under direct wetting

Two mechanical tests have been used to evaluate the influence of the compaction state, stress level and saturation state on the volumetric behaviour of compacted Bringelly Shale specimens. Small-scale and large-scale one-dimensional compression tests were used for this purpose. Small-scale tests were performed in a stainless steel oedometer apparatus with sample dimensions of 48 mm in diameter and 20 mm in height. In this device, top and bottom drainage lines are connected to sealed containers which may be filled with specific salt solutions to maintain a required relative humidity during the test. This feature reduces the possibility for moisture losses due to evaporation.

On the other hand, a 300 x 300 mm direct shear box device was used to evaluate the volume change of compacted Bringelly Shale in large scale tests. Squared specimens with 300 mmx300 mm plan area and 51.5 mm in height were compacted statically in the shear box until achieving the required density. The direct shear box was covered with plastic film to isolate the specimen from the atmosphere and reduce moisture losses due to evaporation.

To study the volumetric behaviour of compacted Bringelly Shale due to an increase in moisture content, a series of wetting upon loading tests has been conducted on material from BS2. Small scale and large-scale compression tests were carried out as summarized in Table 4. Samples for small scale tests 1 to 9 noted in Table 3 have been prepared by statically compacting material in standard oedometer cells to a required density. In all tests, specimens have been statically compacted using a displacement rate of 1 mm/min. After compaction, load has been applied incrementally to a required vertical stress before wetting the samples. To apply 30 kPa vertical stress on sample, load has been applied incrementally at 5 kPa, 15 kPa and 30 kPa. Similarly, to apply 300 kPa vertical stress on sample, load has been applied incrementally at 5 kPa, 15 kPa, 30 kPa, 80 kPa, 160 kPa and 300 kPa. Each load increment has been applied typically over a period of 24 hours. Samples were inundated for a minimum of 3 days and up to 6 days to observe swell and compression behaviour.

Various stress levels, compaction effort and moisture content with respect to optimum moisture content have been used to study the influence on volumetric behaviour.

Table 3: Hydro-compression laboratory tests on BS2 under direct wetting

Test ID	Applied stress at soaking, kPa	Maximum particle size	Actual compaction conditions
1	30	2.36 mm	Modified proctor, dry density of 2.07 t/m^3 and 7.2% water content (+0.5% of optimum)
2	300		
3	30		Modified proctor, dry density of 2.07 t/m^3 and 4.7% water content (2 % dry of optimum)
4	300		Modified proctor, dry density of 2.07 t/m^3 and 6.0% water content (0.7% dry of optimum)
5	30		Modified proctor, dry density of 2.07 t/m^3 and +10.9% water content (4.2% wet of optimum)
6	300		
7	30		Standard proctor, dry density of 1.94 t/m^3 and +12.7% water content (1.2% wet of optimum)
8	300		
9	300		Standard proctor, dry density of 1.94 t/m^3 and 10.6% water content (0.9% dry of optimum)
10	30	19 mm	Standard proctor, dry density of 1.97 t/m^3 and moisture content of 9.3% (wet of optimum)

2.2.2 Testing Regime 2 – Compression tests under as-compacted water content

As discussed previously, post-compaction moisture transfer within the fill could still occur as part of the moisture equilibrium process, leading to similar hydro-compression phenomenon as direct wetting. In order to explore this phenomenon, a series of small-scale compression tests and a large-scale compression test have been conducted (i.e. without inundation). As part of sample preparation for small scale compression tests, moisture content of in-situ material has been measured and compared with target moisture content for each compression test. Where the target moisture content for compression test was higher than in-situ moisture content, additional water has been added. Where the target moisture content for compression test was less than in-situ moisture content, in-situ material has been air-dried. Small-scale test samples were then compressed in a stainless steel oedometer apparatus with sample dimensions of 50 mm in diameter and 20 mm in height to achieve target density. The test specimen including the cell was covered with plastic film to isolate the specimen from the atmosphere and reduce moisture losses due to evaporation.

Small-scale samples prepared using particles passing through 2.36 mm and compacted to 100% standard maximum dry density (SMDD) or 97% of modified maximum dry density (MMDD) have been tested for an extended period. A summary of tests conducted at 100% standard maximum dry density is provided in Table 4.

Table 4: One dimensional compression tests on Residual clay, BS3 and BS2 material compacted to 100% standard maximum dry density – (Small scale samples)

Material	Stress Level (kPa)	Density t/m^3	Moisture content (%)	Period of test (Days)
Residual soil	200	1.73	15.8 (Optimum -2%)	22
	200	1.74	19.7 (Optimum+2%)	22
	200	1.74	16.5 (Optimum)	98
	100	1.72	19.1 (Optimum+1.2%)	19
BS3	160	1.82	16.4 (Optimum +2%)	96
	80	1.84	13.6 (Optimum)	19
	160	1.82	16.7 (Optimum +2.5%)	18
	160	1.82	14.7 (Optimum +0.8%)	18
BS2	160	2.09	9.0 (Optimum)	97
	80		9.4 (Optimum)	92
	160		11.8 (Optimum + 2%)	24
	160		7.5 (Optimum -2%)	24

In addition, one compression test with a large-scale sample (300 mm x 300 mm x 51.5 mm sample) and three compression tests with small-scale samples (compacted in oedometer cells) compacted to 97% modified maximum dry density and optimum moisture content have been conducted for BS2. A summary of tests conducted is provided in Table 5.

Table 5: One dimensional compression tests on Residual clay, BS3 and BS2 material compacted to modified maximum dry density at – (Small scale and large scale samples)

Material	Stress Level (kPa)	Degree of Compaction	Maximum particle size	Moisture content (%)	Period of test (Days)
Residual clay	100	Dry density of 1.9 t/m^3 (100% Modified proctor)	2.36 mm	12 (Optimum)	68
	200			11.5 (Optimum)	68
BS3	80	Dry density of 2.0 t/m^3 (100% Modified proctor)		9.9 (Optimum)	68
	160			9.8(Optimum)	68
BS2	50	Dry density of 2.1 t/m^3 (97% Modified proctor)		6.4 (Optimum)	43
	100	Dry density of 2.11 t/m^3 (97% Modified proctor		6.7 (Optimum)	42
	200			6.2 (Optimum)	43
	30		19 mm	7.1 (Optimum)	6

3 RESULTS

Results of the laboratory tests are summarised in the following sections.

3.1 TESTING REGIME 1 – STUDY ON VOLUMETRIC BEHAVIOUR UNDER DIRECT WETTING

A typical test result is shown in Figure 8 where volumetric strain is plotted against time. A small amount of compression occurred while loading prior to inundation. Swelling then typically occurred after inundation at most stress levels with the exception of a few samples loaded to 300kPa. Further swelling then occurred during unloading. The compression stage was generally completed within about 100 minutes while small amounts of swelling were still occurring up to 5,700 minutes.

The results of the testing programme are summarised in Figure 9 where volumetric strain is plotted against vertical stress as a swell-pressure plot for the inundation stage of the tests. Samples compacted to modified density swelled more than samples compacted at standard density. Samples compacted dry of optimum swell more than those compacted at optimum and greater moisture content. Swelling generally reduced as compaction moisture increased except for samples compacted to standard density and dry of optimum. Swell reduced as the vertical stress increased. Compression occurred in two of the samples compacted to standard density and one of the tests compacted to modified density when loaded to 300kPa.

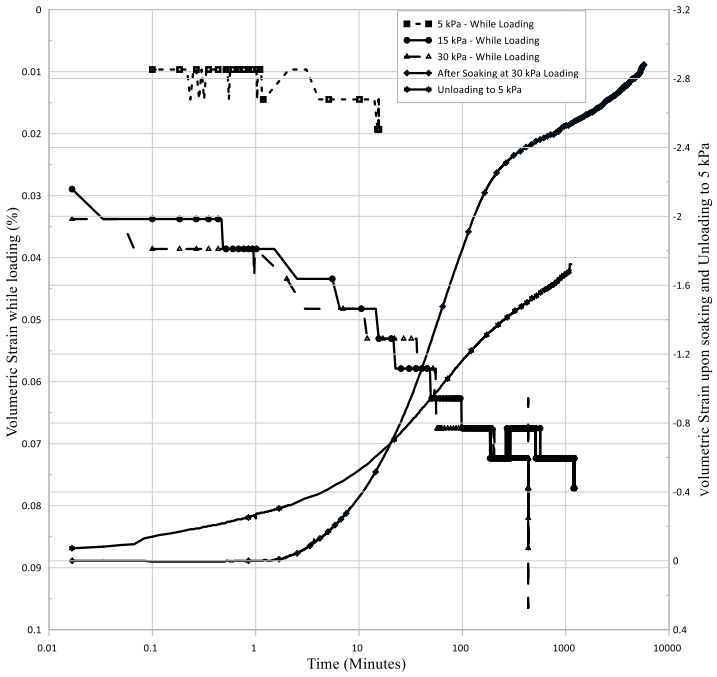


Figure 8. Typical test result

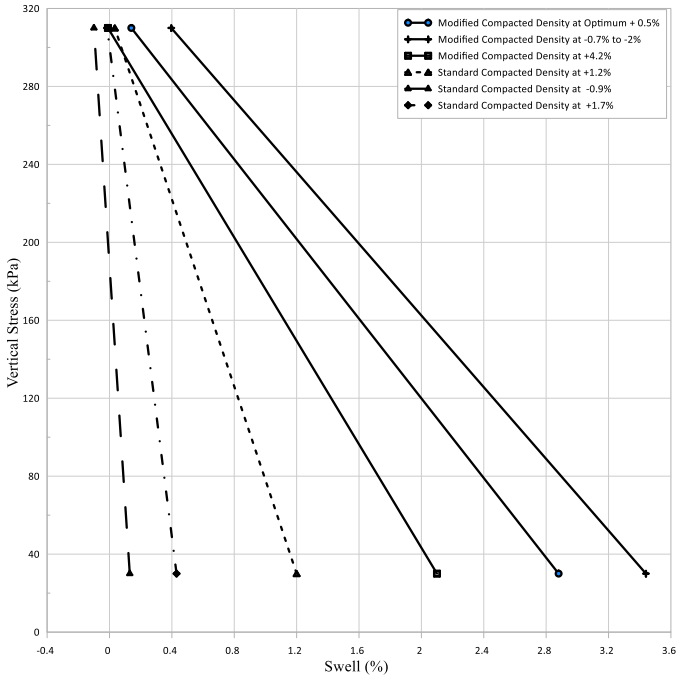


Figure 9: Summary of inundation testing

3.2 TESTING REGIME 2 – COMPRESSION TESTS UNDER AS-COMPACTED WATER CONTENT CONDITIONS

Time-strain plots for the tests on residual soil are presented in Figures 10 and 11. Four tests were performed up to 32,000 minutes and three tests were performed between 98,000 and 160,000 minutes. The tests performed to 32,000 minutes were compacted to standard density at different moisture contents. The strain at 32,000 minutes was greatest for the sample compacted wet of optimum and reduced as the compaction moisture content reduced. The test performed to 160,000 minutes with sample compacted at 100% standard density and optimum moisture content appeared to reach an asymptote compressive strain of 7%. The tests compacted to modified density initially swelled and then started to

compress. The swelling increased in inverse proportion to the applied load. The time to start of compression increase also in inversely proportional to the applied load.

The moisture content at the start of the tests was typically a few percent higher than the moisture content at the end of the tests. Some evaporation appears to have occurred and that could have contributed to the compressive strains and is considered as a limitation of the study. While evaporation complicates the interpretation of laboratory test data, it will occur naturally in the field which could contribute to real world performance.

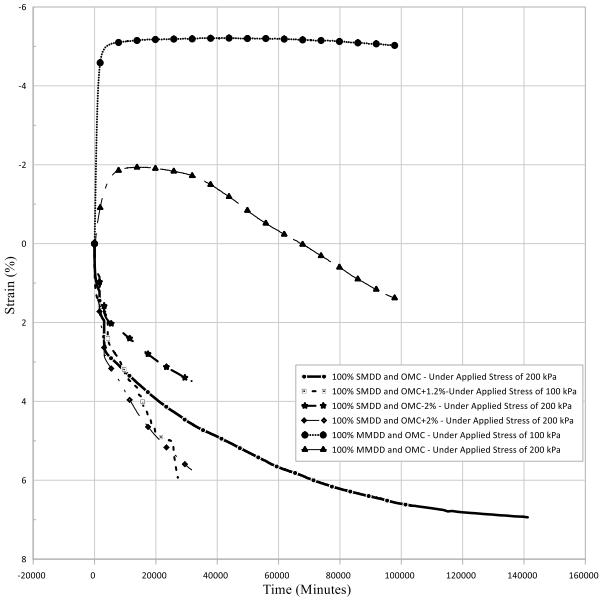


Figure 10: Strain versus time for Residual soil

The data in Figure 10 plotted against the logarithm of time shows that the rate of strain varies with time. The strain in the long-term test at 100% standard compaction has a shape broadly similar to a consolidation curve where the initial rate is relatively small then increases with the logarithm of time and then reduces at large times. The long-term tests at 100% modified compaction have shapes similar to unloading creep tests in soft soils. The time to the start of compressive strain varies with load and the rate of strain increases with time. At the end of these tests the rate of strain is approximately linear with the logarithm of time, however that might have changed if the tests were allowed to run for a longer period of time.

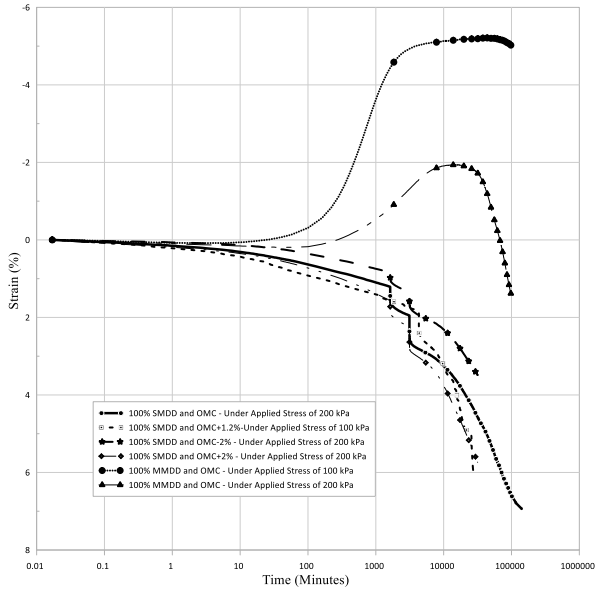


Figure 11: Strain versus log time for Residual soil

Time versus strain plots for BS3 and BS2 materials are shown in Figures 12 to 15. The data shows similar trends to residual soil. Compression increases as density reduces and as moisture increases, although all BS3 test wet of optimum and the BS2 test wet of optimum do not follow these trends. The magnitude of compression at 35,000 minutes of BS3 is less than that for residual soil but is similar to the BS2 sample. Swelling of the BS3 material compacted at 100% modified density is less than residual soil compacted at 100% modified density and compression of BS2 material occurred when compacted at 97% modified density. The trends of swelling / compression with load for the tests at modified compaction are the reverse of what would be expected. However, the magnitude of the movements is small and could be explained by slight variations in grading and density.

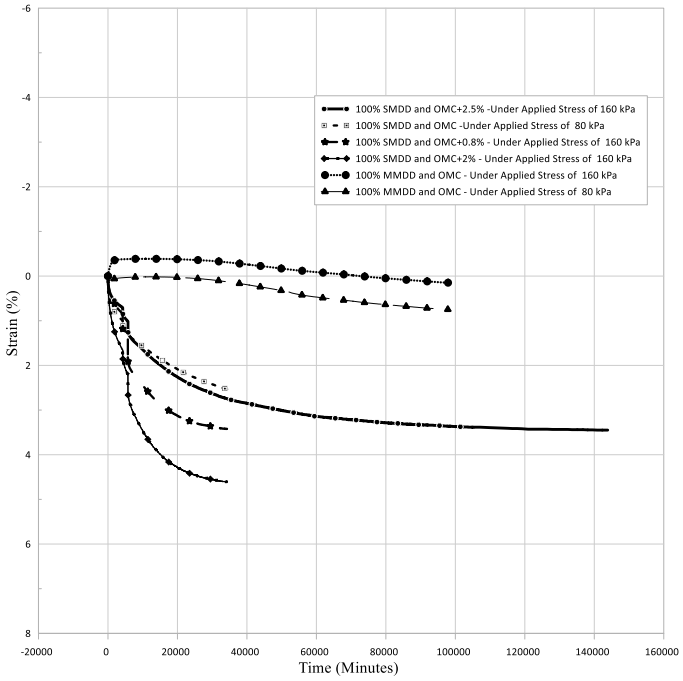


Figure 12: Strain versus time for BS3

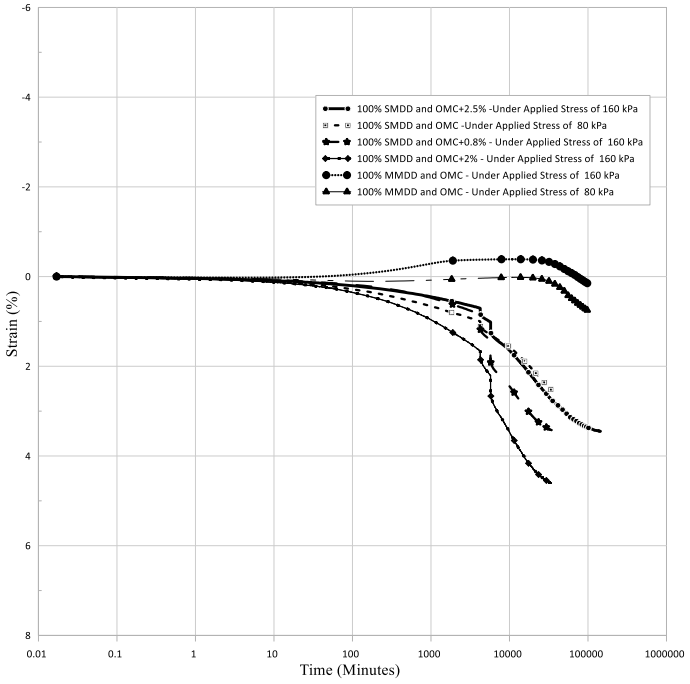


Figure 13: Strain versus log time for BS3

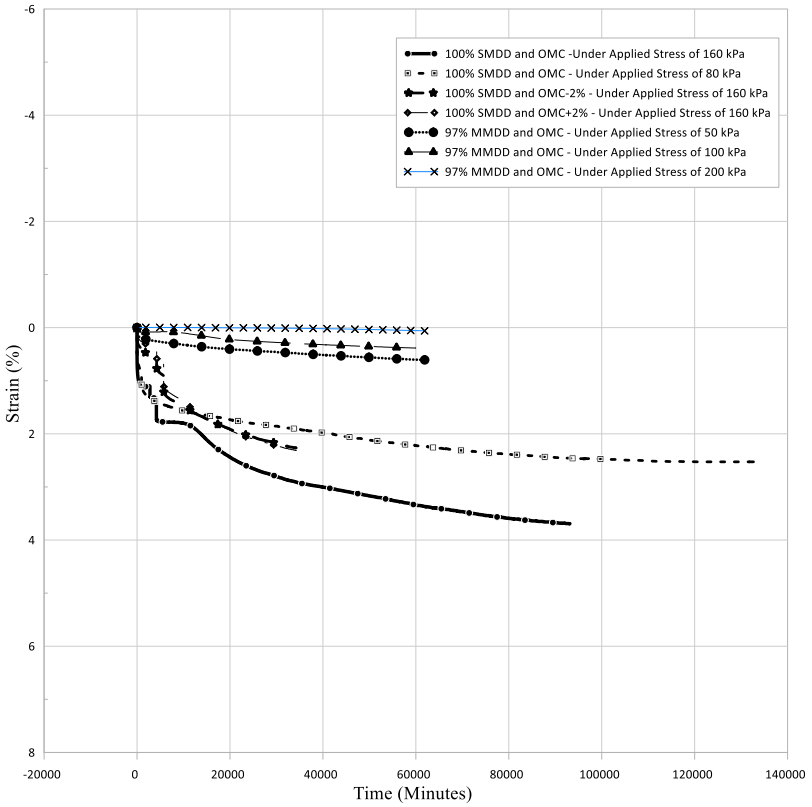


Figure 14: Strain versus time for BS2

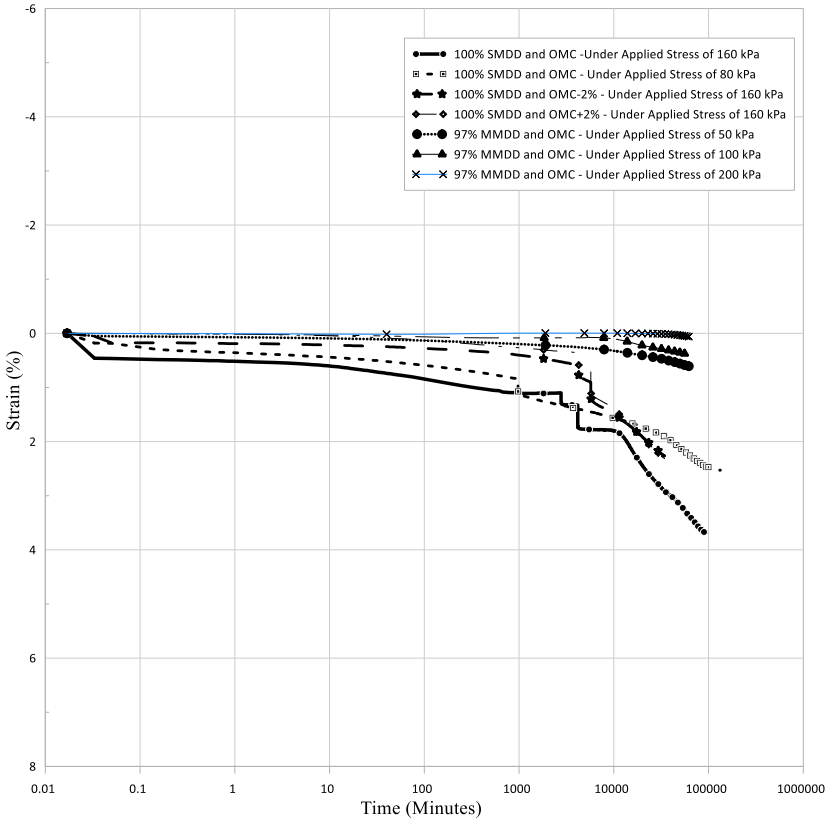


Figure 15: Strain versus log time for BS2

3.2.1 Rate of Internal Compression

The time-settlement behaviour of a compacted sample indicates that the rate of internal compression of fill appears to vary with time, density, stress and moisture content as discussed above for small-scale samples.

A typical settlement behaviour of a compacted sample of BS2 indicates that the internal compression of fill appears to occur at two different stages as shown in Figure 16 for small-scale samples at applied load of 160 kPa.

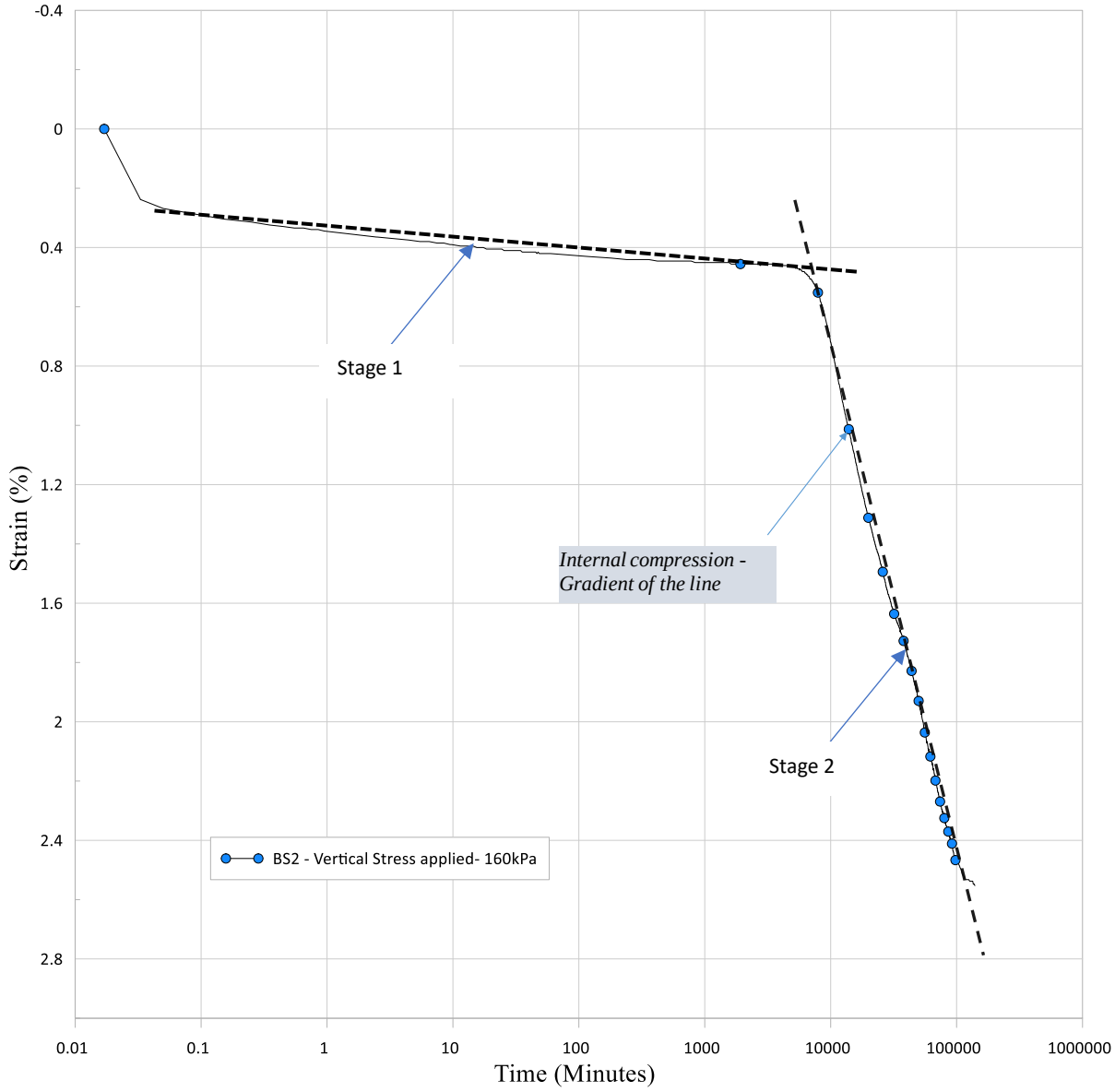


Figure 16: Interpretation of internal compression from compression test on material Type BS2

Authors' interpretation of Figure 16 is that the soil and shale particles act like heavily over-consolidated clays i.e. having high suction. The free water used during compaction moves through the soil and shale particles which causes softening of particle contacts as well as breakdown of particles. This softening of interparticle contacts, and breakdown of unsaturated particles with high suction causes settlement. Based on the compression test results, typical internal compression behaviour is a time dependent phenomenon and varying approximately linearly in log time scale at two different rates, namely Stage 1 and Stage 2 as shown in Figure 16.

Rates of internal compression at large time for BS2 samples compacted to 97% of modified maximum dry density is 2.11 t/m3 and standard maximum dry density of 2.1 t/m3 are shown in Figure 17. Rates of internal compression in percentage are approximately linear with applied stress and are 0.012 x vertical stress in kPa per log cycle of time. This rate is approximately 4 to 5 times greater than the value of 2.7 times vertical stress in MPa (in percentage) reported by Waddell

and Wong (2005) for weathered Ashfield Shale. Rates of internal compression for residual soil and BS3 soils are also shown in Figure 18. Trendlines fitted through the origin also show high rates of internal compression.

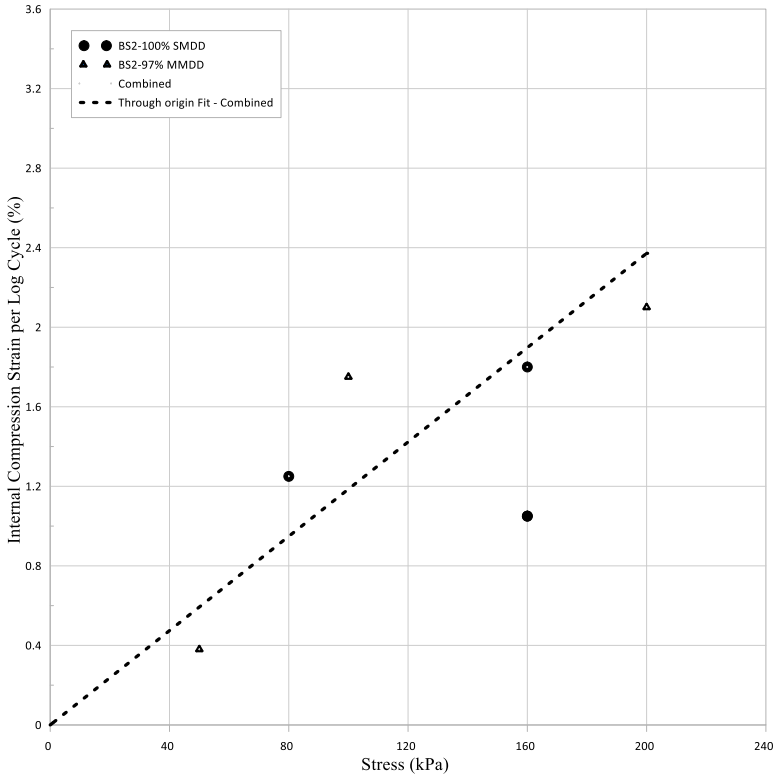


Figure 17: Internal compression strain rate with stress - BS2 (Small scale samples)

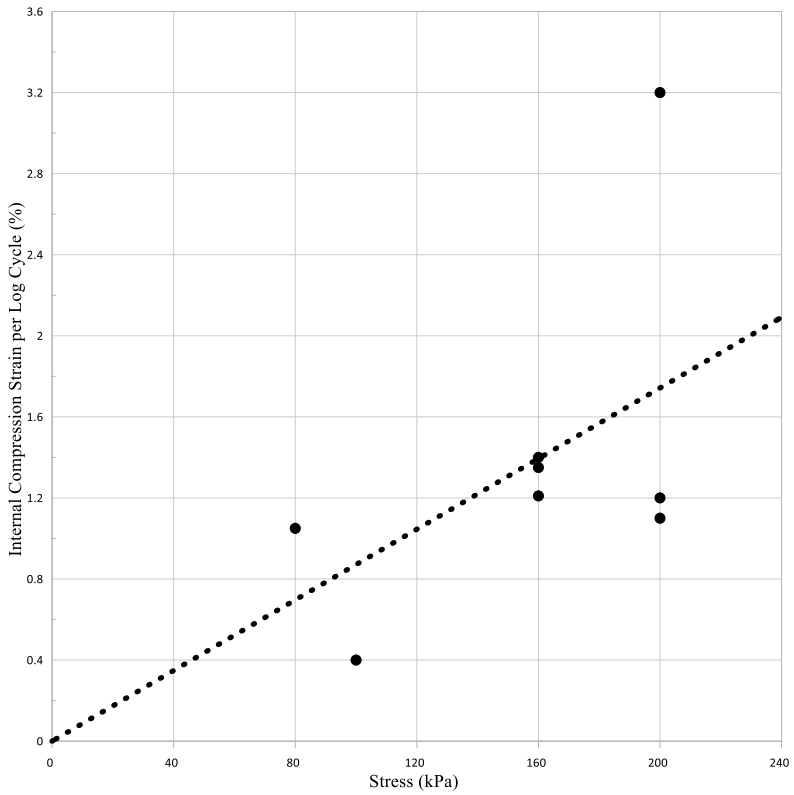


Figure 18: Internal compression strain rate with stress – BS3 and Residual Clay (Small scale samples)

3.3 LARGE SCALE TESTS

Time strain plots for BS2 material compacted to 100% maximum standard dry density and 100% maximum modified dry density at the optimum moisture content are shown in Figure 19. The samples were initially loaded to 30 kPa, allowed to compress for a period and then inundated with water. On loading, the test compacted to standard density compressed much more than the test compacted at modified density. On inundation the test compacted to modified density swelled slightly more than the test compacted at standard density but swelling was small in both cases. These data are compared with similar small scale oedometer tests in Figure 19. Densities and moisture contents for these tests are provided in Table 6 and it can be seen that they are similar. The compression on loading in the oedometer tests are broadly similar to the shear box test compacted to modified density but are much smaller than the test compacted to standard density. Swelling on inundation in the oedometer tests is much larger than swelling on inundation in the shear box. In general, results show that the behaviour of Bringelly Shale varies with the particle size distribution and scale of the laboratory test.

Table 6: Densities and moisture contents for shear box and oedometer tests on BS2 material

Test	Moisture content (%)	Dry density (t/m^3)
Modified Shear box	7.1	2.11
Standard Shear box	9.3	1.97
Modified Oedometer	7.2	2.07
Standard Oedometer	10.6	1.94

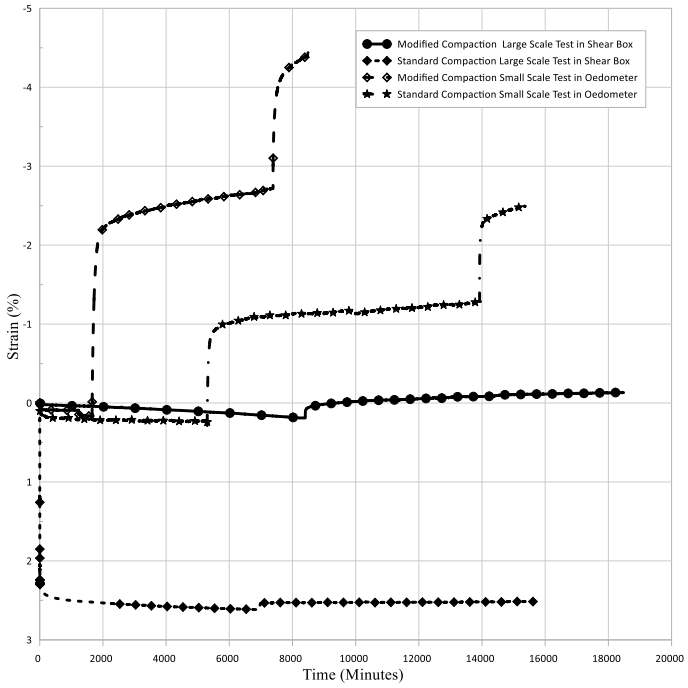


Figure 19: Comparison of oedometer and shear box inundation tests in BS2 materials

4 DISCUSSION

4.1 PREDICTING THE MAGNITUDE OF VOLUMETRIC CHANGE – WHEN INUNDATED

The results of the laboratory tests indicate that the volumetric changes in compacted Bringelly shale are a function of density, stress and moisture content (suction). This is in agreement with the behaviour of other compacted fills reported in the literature (e.g. Brandon et al., 1990; Noorany and Stanley, 1994; Cardoso et al., 2012). Predicting the magnitude of volume changes should take all of these factors into account. Use of a contour plot similar to Cox (1978) only takes stress and density into account but it has been shown that samples compacted to the same density with the same applied load have different magnitudes of volume change depending on their compaction moisture content and particle size. Similarly, attempting to develop strain contours in the compaction plane can capture effects of density and moisture

content but not the effects of applied stress. Unsaturated soil mechanics provides a framework that can take all three factors into account. Available experimental evidence can be studied from the Unsaturated Soil Mechanics perspective (e.g., Alonso et al, 1990).

A simplified version of the elasto-plastic Sheng, Fredlund, Gens (SFG) model (Sheng et al, 2008) has been adopted. It is assumed that the density and stress level adopted in the laboratory tests create a soil state that is over-consolidated in comparison with a saturated normal compression line and collapse type mechanisms do not occur. The SFG (2008) model is expressed in terms of normal compression lines but it also applies to swelling lines (Prof Sheng, personal communication).

In this model, volumetric strains are estimated from Equation 1 where $d\varepsilon_v$ is the increment of volume strain, κ_{vp} is the swelling line for saturated soils, κ_{vs} is the swelling line for unsaturated soils, P is the mean total stress and S is suction. The volumetric change has been assessed using one dimensional analysis. Changes in swelling line with suction are described in Equation 2. In Equation 2 S_a is the air entry suction.

$$d\varepsilon_v = -\kappa_{vp}\left(\frac{dP}{P+S}\right) - \kappa_{vs}\left(\frac{dS}{P+S}\right) \quad (1)$$

$$\kappa_{vs} = \kappa_{vp} \qquad S \le S_a \quad (2a)$$

$$\kappa_{vs} = \kappa_{vp}\frac{S_a+1}{S+1} \qquad S > S_a \quad (2b)$$

The parameters κ_{vp} and κ_{vs} were obtained from each individual inundation test as shown in Figure 20. The κ_{vs} parameters were taken as the secant line from initial stress to target vertical stress prior to inundation. The κ_{vp} parameters were obtained from the slope of the unloading line after inundation in each of the individual tests. Soil suction can be obtained using Equation 2 as a function of the stiffness parameters and the air entry suction. Volume strain is then obtained from Equation 1 adopting incremental steps in suction. A least squares data fit between measured and predicted parameters is then performed to obtain the air entry suction and optimise the predicted volume strain. A comparison of predicted swell during the inundation phase of the test with measured values shown in Figure 21 corresponding to an air entry suction of 39 kPa. There is reasonable agreement between measured and predicted swell. It is not known whether the adopted air entry suction of 39 kPa is accurate, however a low value is consistent with a granular material such as BS2.

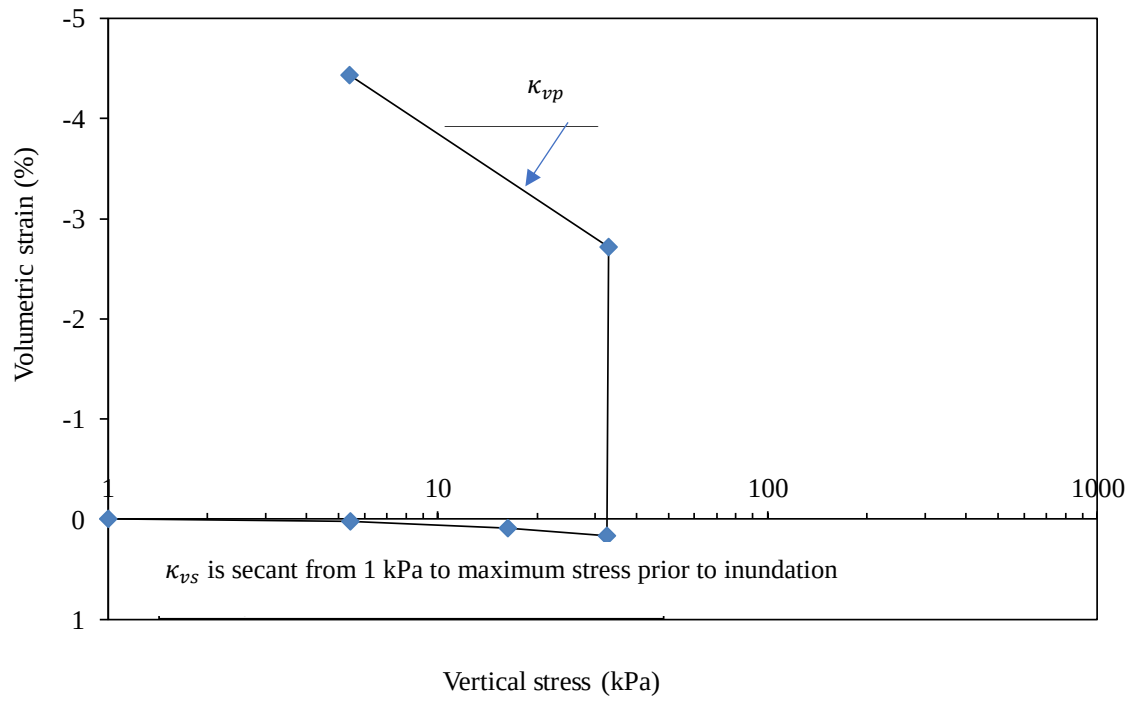


Figure 20: Typical test result

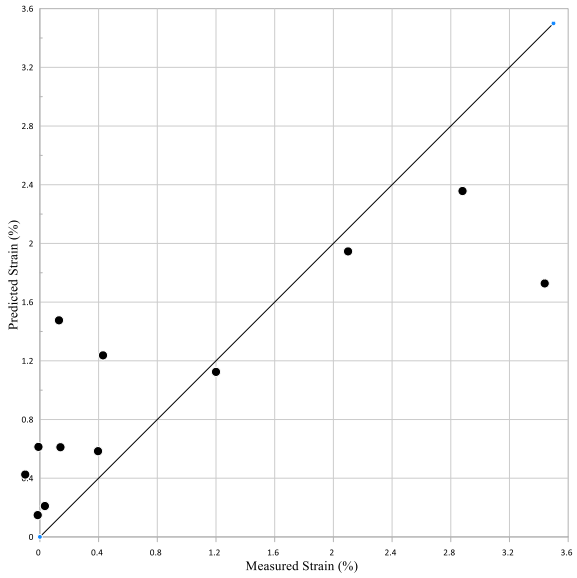


Figure 21: predicted versus measured swell

The inundation tests effectively provide the soil with access to a large amount of liquid water capable of inducing full saturation. This, in turn, leads to the samples to swell.

In principle, a suite of laboratory tests may be performed to facilitate prediction of the internal compression of Bringelly Shale fill. The laboratory tests would have to be performed at sufficiently large scale to capture the particle size distribution expected in practice. A soil water retention curve would be required to relate moisture content to suction and to obtain the air entry suction. One dimensional test where samples at field moisture content and target dry density are loaded to a target pressure and then inundated would provide the κ_{vp} and κ_{vs} values. The rate of compression could be estimated if the permeability of the intact soil particles was known.

The above approach can be used to predict settlement for a scenario where fills are likely to be inundated during the design life such as quarry fills.

4.2 PREDICTING THE MAGNITUDE OF VOLUMETRIC CHANGE – UNDER AS-COMPACTED MOISTURE CONTENT

As presented previously, laboratory test results at as-compacted moisture content indicated that Bringelly Shale fill settles with time (even without inundation). This scenario is applicable to a large landform with a minimal likelihood of change in overall moisture content due to external factors such as groundwater rise and rainfall.

When samples were prepared to study volumetric behaviour under as compaction moisture content (i.e. constant water content tests), required amount of water has been added when moisture content of in-situ material was less than target moisture content or air dried when the moisture content of in-situ material was higher than target moisture content. This process is generally similar to what will be adopted during construction. The test results show that samples compacted to standard density tended to compress while samples compacted to modified density either compressed, swelled or did both. Considering material preparation method (i.e. either air drying or adding water air dried material) it is likely that water content will be non-uniform within the sample. Moisture will therefore move within the sample to come into moisture equilibrium, leading to break down of shale particles. In order to study this behaviour in unsaturated soil mechanics discussed above, variation of moisture within a sample should be studied rigorously. In practice tests of this nature are unlikely to be performed unless the infrastructure being constructed on the fill is particularly expensive and sensitive to movement. A less involved method of estimating internal compression is to perform laboratory tests to obtain the rate of settlement with the logarithm of time at various applied stress levels and use the correlation to estimate compression over a time period. As an example, the BS2 data provided in Figure 17 has been used to estimate settlement over time. The method of calculation was based on the following:

- Fill is subdivided into 1m thick layers and the stress at the centre of each layer estimated assuming a bulk unit weight of 21 kN/m^3;
- Start time for calculations is assumed to be 1 year;
- Calculations were extended to 40 years; and
- Strain calculated using the conventional equation for creep in logarithmic time.

The assessed internal compression rate with respect to embankment height is plotted in Figure 21.

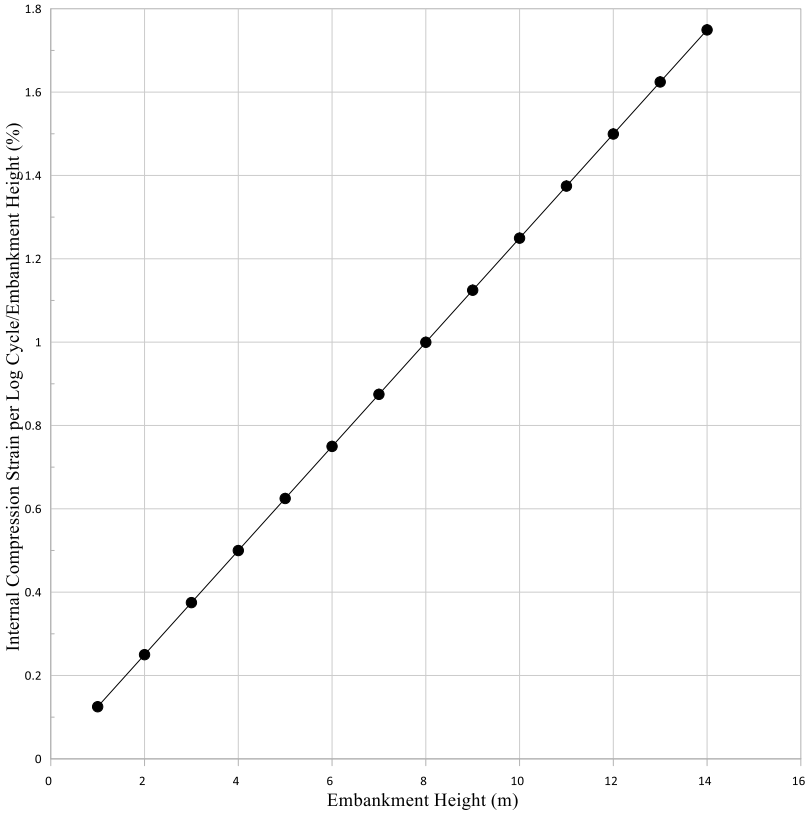


Figure 21: Strain estimated for Weathered Bringelly Shale

4.3 PRACTICAL MEANS OF CONTROLLING VOLUME CHANGES IN BRINGELLY SHALE IN DEEP FILLS

A typical cut to fill operation will start by removing residual soil and placing at the base of the fill then progressively winning and placing less weathered material. That means that the soils most prone to swelling and compression in the tests reported in this paper will be the most heavily loaded. In order to reduce internal compression of the fill the following process could be adopted:

1. Compact the lowest soil layers to modified maximum dry density. Increasing density has been shown to reduce compression;
2. Progressively reduce the compactive effort as the height of the fill increases and load applied to the fill layer reduces. However, higher compactive effort in the upper layers of fill provides some risk management as it would be more prone to swell hence counteracting any compression that occurs deeper in the fill.
3. Compact soils at optimum moisture content or wet of optimum. The intention is to maximise compression on loading by reducing the unsaturated stiffness of the soil and to maximise volume changes during construction rather than post construction. Compacting dry creates a stiff fill but long-term changes in moisture content and humidity will increase post construction internal compression.
4. Rigorously mix the water into the fill to maximise its distribution. The rotary hoe method adopted by Hopkins and Beckham (1998) aims to achieve a high degree of mixing.
5. Adopt thin layers and use heavy compactors in order to reduce the average particle size, maximise density and fill voids with particles.

These recommendations are essentially the same as those of Hopkins and Beckham (1998) except that compaction to higher density is recommended here.

4.4 LIMITATIONS OF THIS STUDY AND OPPORTUNITIES FOR FURTHER RESEARCH

This study has largely been based on limited number of small-scale laboratory tests. Data has been provided that shows fills constructed using a larger particle size distribution behave differently from the small-scale tests. Use of small-scale

tests to estimate rates of compression with the logarithm of time are unlikely to accurately represent the behaviour of fills constructed from a wider particle size distribution. Future work would be better performed using field scale particle size distributions.

An attempt to use the Unsaturated Soil Mechanics principles to explain the volumetric behaviour of Bringelly Shale upon direct wetting. Key parameters required by the theory have been inferred through correlating prediction and measurements. A thorough suite of unsaturated soil tests should be performed to investigate the use of the theory for fills constructed from Bringelly Shale.

The concept that there is a state of density, stress and moisture where the volumetric behaviour changes from compression to swelling does not appear to exist in the literature. Further study of this phenomenon would contribute to more accurate prediction of volumetric changes.

5 CONCLUSIONS

The results of this study have demonstrated that the internal compression of Bringelly Shale fill can exceed the rule of thumb strain rate of 0.1% per log cycle of embankment height, particularly when fills are high. A better estimate for higher fills can be obtained through large scale, long term, laboratory tests.

Methods for controlling the internal compression of fill constructed from Bringelly Shale have been developed. In-lieu of some directions provided in Transport for New South Wales (TfNSW) TN033 such as classifying expansive soil as unsuitable or use of chemical treatment and blending non-reactive material with expansive soil to use it as general fill, these methods could be adopted in order to allow greater use of Bringelly Shale hence reduce the costs of transport infrastructure construction in Western Sydney.

6 ACKNOWLEDGEMENTS

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PERMEATION GROUTING IN SYDNEY

David Lees
Principal Tunnelling Engineer, Jacobs
North Sydney, Australia

ABSTRACT

In recent years permeation grouting has developed in Sydney and is now a common choice for building contractors as a simple, cheap and un-intrusive form of ground improvement. There are many sand deposits across Sydney from beach sands, aeolian sand, Pleistocene and Holocene sediments and fill material. These materials may be treated by permeation grouting to reduce their permeability or increase their strength using sodium silicate or microfine cements. The system has been used extensively for underpinning structures, sealing behind or beneath retaining walls, or even creating retaining structures, or for the containment of contaminated ground. This presentation draws upon the application of permeation grouting on many projects across Sydney to explain the technique with different materials and testing regimes for many different end uses. Other examples are presented of the application of permeation grouting and its development on other projects across Australia and elsewhere in the World. The aim of this paper is to show how permeation grouting can be used in Sydney sand formations for different applications instead of more complex and more expensive systems.

1 INTRODUCTION

Permeation grouting is essentially filling of the pore spaces between particles in granular soils. It may be defined as the introduction of low viscosity solutions or particulate suspension into the ground under low pressure without disturbing the structure of the ground to reduce the permeability or increase the strength and stiffness of the ground.

1.1 HISTORY

Permeation grouting is probably the oldest form of grouting. The first recorded application of grout injection to stabilise civil structures is by Charles Berigny in 1802 for the Port of Dieppe in France, he went on to develop various injection processes in soft ground following the patenting of Portland Cement in 1824 (Glossop, 1960).

In 1856 Darcy published his studies on the flow of water through sand that formed the basis of ground permeability (Littlejohn, 2003). At the beginning of the 20th Century, Portier considered the possibility of grouting sand but argued that the unstable cement would be subject to filtration (Duvivier, 1908).

The use of sodium silicate had been known since 1886 (Karol, 1983). The application of sodium silicate grouting in sands to displace water and form a gel binding the sands together to form a sand mass with compressive strength of 3 to 5 MPa was developed by Joosten in the 1920s by injecting small volumes of sodium silicate in stages through a lance as it was withdrawn (Joosten, 1954). This system was then used on a number of projects in Europe. In England it was mostly used for underpinning with an exceptional example of stabilising the Thames gravels at the Bank Monument Station in 1933 (Harding and Glossop, 1940). 15,500 tonnes of sodium silicate was injected by Rodio for the Barrage of Bou-Hanifia in Algeria, and it was here that Ischy first used his invention of tube-a-machette to enable successive injections in the same hole (Ischy and Glossop 1962).

With regards to injection of cement grouts, Kolbrunner and Blatter noted in 1941 that it was only possible if the pre size diameter was greater than 0.1 mm and the associated permeability was about 1×10^{-3} m/s (Kennedy, 1961). Glossop and Skempton (1945) then presented a review of soil classification based on particle size and limits of applicable geotechnical processes.

The first successful use of finer ground cements by Carlson in 1944 to seal cracks in concrete using cement of < 40 micron to seal crack widths of 0.03 mm. Machis carried out some experiments of cement grout penetration into sands and gravel in 1946 and concluded that "*the cement ceases to pass into pores when the pore becomes small enough for two or three cement particles to bridge across it*" (Littlejohn, 2003).

Lord reports successful grouting of sands with cement with particle size <30 microns in 1955 and noted the need for higher mixing speeds and longer mixing time (Kennedy, 1961). King and Bush (1961) stated that the groutability criterion $N= D_{10}/D_{95}$ should be greater than 8 because filtering started at a value of 6.

Ultrafine grouts with a maximum particle size of 10 microns and an average of 4 microns was introduced by Shimoda and Ohmori (1982) capable of penetrating fine sand with a permeability of 3.75×10^{-6} m/s. The injection of weak

microfine cement into sand was first reported in 1989 by Arenzena *et al* (1989) with low pressure in fine sands to achieve penetration distance of at least 0.5 m. The introduction of silica fume in cement grout to reduce bleed for grouting sand was noted by Ata and Vipulanandan (1997) to increase compressive strength by 100%.

1.2 PERMEATION GROUTING IN AUSTRALIA

In Australia permeation grouting in sand has been common practice in Perth for many years. The technique was introduced in Australia for underpinning of the Titles Office and Town Hall during construction of the R&I Bank building in Hay Street in 1969. For this application permeation grouting using a ligno-sulphonate based chemical grout was used (Osborne, 2003).

From the late 1970s extensive chemical injection was carried out in the base of red mud storage areas at Alcoa Kwinana to reduce liquor seepage from the storages. This work was undertaken with an imported grout known as AC400. This represents the most extensive application of grouting techniques in the Perth region to affect ground improvement – in this case a reduction in permeability (Osborne *et al*, 1993).

Until the mid to late 1990s, chemical grouts were used, mostly for small scale applications. Specific applications of the process were strengthening of supporting soil beneath existing footings to accommodate additional load and grouting of a zone of soil adjacent to a proposed excavation to support the excavated face and/or adjacent buildings. The technique was used in conjunction with soil reinforcement (soil nails or anchors) either as temporary support during the excavation phase of the installation or where the anchors are installed to provide horizontal restraint to the grouted face (Osborne, 2003).

Sodium silicate was used extensively in Sydney in the 1990s, mainly for underpinning and sealing behind retaining walls. The first application of microfine cement for permeation grouting in Sydney was for Capella Apartments in 2004.

2 GEOLOGY OF SOIL DEPOSITS IN SYDNEY

Compared to the aeolian sands of Perth, the Quaternary soils of the Sydney Basin are more variable consisting of both aeolian and alluvial deposits. These can be divided into two groups – namely the Botany sand sheet and the Nepean and Hawkesbury floodplain. The upper sediments are of Holocene age, with the stiffer alluvium at depth of Pleistocene age.

The Botany sand sheet formed from the wind-blown deposition of marine sand swept constantly north along the NSW coast. It starts as a medium-sized sand on the coast and becomes finer further inland. The Hawkesbury–Nepean and Georges River floodplains are composed of sands, silts, cobbles and clay deposits.

3 MATERIALS

3.1 SODIUM SILICATE

Sodium silicate is a two-component grout that typically has very low viscosity but will often expunge water after gelling by a process called syneresis. With relatively short gel times down to a few minutes, Babcock (2016) states that sodium silicates are commonly used as temporary solution for water control or structural support, but that sodium silicates can be sensitive, bordering on unstable and have an estimated life span of just a few years. The main advantage of sodium silicate grouts is its low viscosity and low cost, and, apart from the high pH (typically 10.5 to 11.5), there are small problems with work safety and health (BASF, 2011).

3.2 MICROFINE AND ULTRAFINE CEMENTS

Normal GP cements have a typical particle size of more than 40 microns, microfine cements are defined as having particle size less than 20 microns and ultrafine cements typically have particle size less than 10 microns (Norwegian Tunnelling Society, 2011). This is presented pictorially by BASF (2011) as presented in Figure 1.

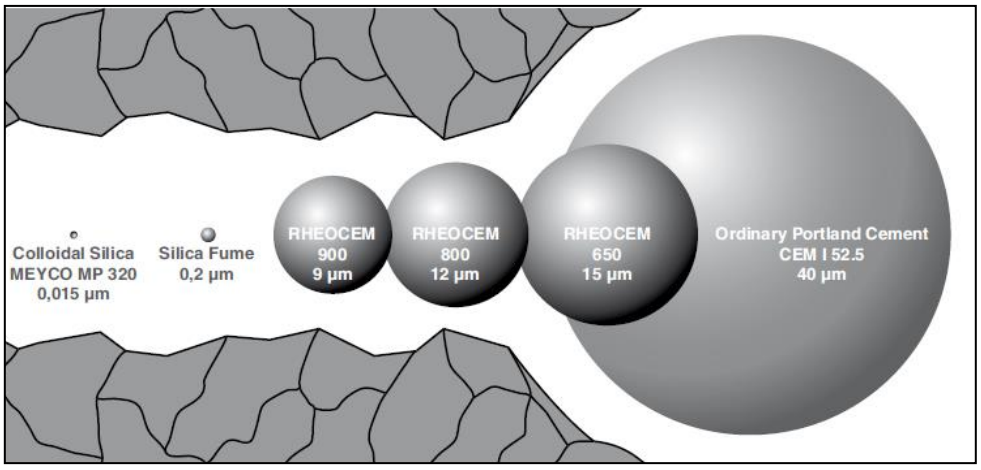

Figure 1: Comparison of particle size for cement products (BASF, 2011)

According to Roald *et al* (2001) the use of microfine cement has improved grout penetration to achieve a permeability less than 0.1 Lugeon, whilst producing more stable grouts with reduced shrinkage.

4 METHODOLOGY

Determining the right material for the ground conditions is the first step. As identified by Schwarz and Krizek (1994) during the injection process, preferential flow paths allow the migration of cement particles into the soil and micro structural packing undoubtedly varies within the pore spaces of the grouted sand. The amount of permeability reduction achievable in a particular situation depends on several factors including the hydraulic conductivity of the ground, but also the viscosity of the grout, the surface tension, grout pipe spacing, injection sequencing, quality control and stability of the cured grout.

Typically in Sydney, a weak microfine cement grout is used with a water-cement ratio of about 4:1 with added superplasticiser and silica fume to reduce bleed. The grout is injected at low pressure through 20 mm diameter lances that are pushed into the ground with a water flush. A fixed volume is then injected to form a bulb of grouted ground, then the lance is lifted a fixed distance and injected again. The height of the lift and volume of the grout injected will determine the effectiveness of the grouted column of sand and the spacing to adjacent injections. Adjacent installations should be separated into primary and secondary holes with secondary installations carried out after the grout has achieved its initial set, to avoid connection to the previously grouted hole.

In very dense sand it may be necessary to insert the grout lance with compressed air. It is important that the lance is sealed into the hole otherwise the easiest path for the grout is up the outside of the grout lance, this is also why the injection pressure must be limited. Also, if too high injection pressure is used then the soil may be hydro fractured, creating grout lenses rather than bulbs of grouted soil.

Quality assurance is critical to success. If the correct volume of grout is injected at a certain level without hydro fracture or grout return to surface, then the grouting will be successful. Site testing can be carried out with water injection through the grout lance. If the ground takes water, then it should take a week low viscosity microfine cement grout or sodium silicate grout.

5 TESTING

Typically, permeation grouting is ideal for sandy soil conditions where the grout travels and infiltrates a wider area. However, without a comprehensive site investigation the ground is a hazard, and, as Littlejohn (1982) stated, it is essential to provide a detailed geotechnical classification of the different ground types encountered together with their location and thickness, along with hydraulic gradient and chemical properties of the ground water.

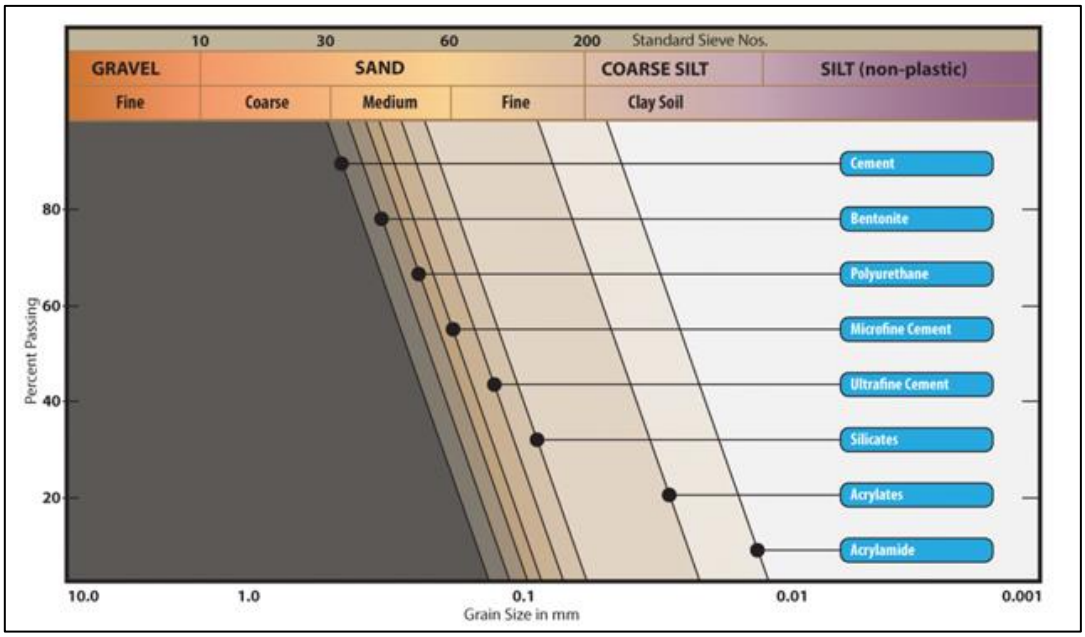

Figure 2: Permeability of different soils and selection of grout type (Babcock, 2016)

Grout types for different soils was presented by Babcock (as presented in Figure 2), but there will always be some variability in the response to, and behaviour of, soils with grout, even with soils of the same hydraulic conductivity and grouted strengths (Shodhganga, 1977). Although a soil's initial hydraulic conductivity is the best indication of the amenability of the soil to permeation grouting, it is not an all-encompassing descriptor. Hydraulic conductivity is a measure of a soil's ability to transmit water, not grout, which has different properties. A silt with coarse sand layers, may have the same transmissivity or hydraulic conductivity as a silty sand, but the two materials will respond very differently to grouting.

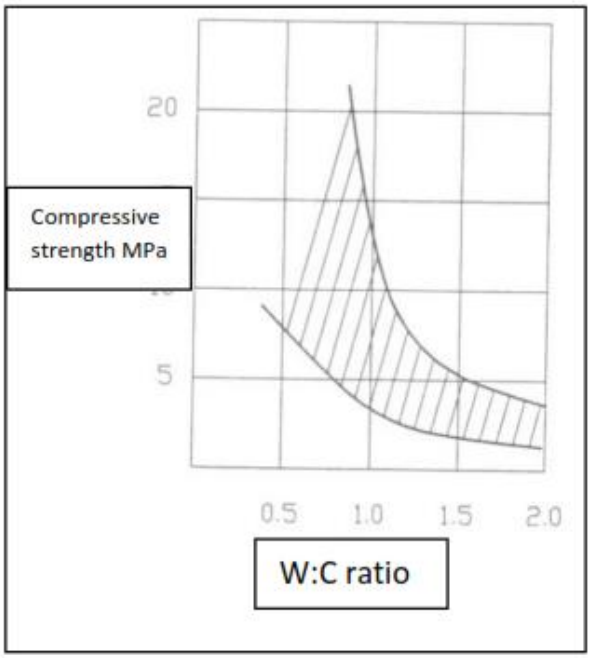

Figure 3: Compressive strength of grouted sand after 28 days (Bonzel and Dahms, 1972)

Figure 3 shows the strength development of poorly graded sands as presented by Bonzel and Dahms (1972,) and shows that the strength of the grouted soil is clearly dependant on the water-cement ratio.

Table 1: Increase in compressive strength of grouted sand with time (Bonzel and Dahms, 1972)

Age of sample (days)	Compressive strength (%)
2	10 to 30%
7	30-60%
28	100%
120	120%

This research also showed that about 60% of the strength is attained after just 7 days, and that even though 100% strength is achieved at 28 days, the strength of the grouted soil continues to increase up to 120% after 120 days as presented in Table 1. This was further investigated in Perth in 2001 where a 2 m deep spear was grouted in 5 stages and excavated 3 months later to show compressive strengths of 13.5 MPa (Multigrout, 2001).

Furthermore, the permeability decreases with elapsed time, it becomes almost constant beyond 15 days of curing period for lower cement contents (i.e. 4 % and 10 %), but at higher cement contents (e.g. 25 %), the permeability goes on reducing even after 15 days (Kainrath and Adam, 2015). Hence one can presume that reduction in permeability is directly related to the hydration of the cement.

The actual amount of cement controls the strength of the grouted soil. The amount of cement within the grouted soil changes with the void ratio of the soil and the degree of saturation. It can therefore be concluded that whilst the water:cement ratio is a parameter to control the strength of the soil, on its own it is an inappropriate parameter to predict the strength and the stiffness of grouted soil.

Testing to prove the effectiveness of sand that has been treated by permeation grouting is not easy. For a project for TransGrid to provide support to Cable 41 in Arncliffe Street, Tempe, the effectiveness of the grouting was established in a trial using cone penetration testing (CPT), which showed that the column of permeated soil extended to about 700 mm diameter from the point of injection through the 20mm diameter lance. The CPT also indicated layers where there was a reduction in ground improvement, considered to be due to poor grout penetration due to layers of organic material.

A property in Bay Street in Brighton Le Sands was built on a raft, however it was determined that this raft was needed to be underpinned by permeation grouting down to a depth of 7 m due to the deep basement on the adjacent property. To prove the effectiveness of the grouting it was decided to try to take some small diameter cores (>25 mm) of the material by diamond drilling after 7 days. But the material disintegrated whilst drilling. This was considered to be due to the high shear forces developed in the coring and the effect of this on the cemented sand particles. Finally, it was determined to excavate a section of the underpinning and determine its effectiveness by using a hand-held soil penetrometer.

Victoria Park stowage track alterations in Perth required cutting the existing cut slope to a steeper angle. To enable this excavation permeation grouting was carried out along the slope with some sections also being soil nailed and shotcreted in order to form steep batter slopes. Larger diameter cores (75 mm) were extracted after 10 days using a hand-held Hilti drill and samples were then tested with a hand-held soil penetrometer to determine the unconfined compressive strength.

In conclusion, it is considered if quality cores are need for laboratory testing at an early age then these should be taken using a triple tube core barrel.

Testing of low mobility grout columns installed after the earthquake in Christchurch was carried out using cross-hole seismic testing (Hwang *et al*, 2017). The cross-hole testing involved pushing two dummy CPT cones, one on either end of a linear array which were then used to transmit vertical impulses to the bottom of the rods where the cone tips acted as sources for simultaneously generating both constrained-compression (P) and shear (S) waves in these small-strain tests. At the same time, a custom-built, 3D-receiver in the shape of a CPT cone was pushed using CPT rods in the centre of the array composed of 3 geophones and acted as the receiver for each seismic source. The spacings between each source rod and the middle receiver rod ranged from about 1.4 to 1.8 m. The testing began at a depth of 40 cm and continued in 20 cm increments, usually to a final depth of about 5 m. Both seismic source rods and the 3D-receiver rod were pushed to the same depth for each measurement. The zone across the grout columns was expected to show increases in stiffness due to the presence of grout as well as the densification of the surrounding soil. The results showed zones of improvement at all three grouted test panels, but the improvements was highly variable and showed lack of continuity over the entire depth. Increases in stiffness greater than 50 % were expected and seen on some records. Values less than 50 % were considered could either be due to the presence of partial grout columns in the soil or the densification of the soil and indeed this was identified when the columns were exposed.

6 SELECTED PROJECTS

6.1 CAPELLA APARTMENTS

Figure 4: Basement excavation at Capella Apartments

Capella apartments are located in Kensington at the junction of Doncaster Avenue and Anzac Parade and comprises 164 apartments over nine levels in two low-rise developments plus retail space and basement parking. The basement construction was facilitated by sheet piling into the Botany Sands and with a high natural ground water level around the site, the site was dewatered to enable basement excavation. However, some sheets of the retaining wall had to be stopped at a higher level due to old brick sewers crossing the site. These gaps in the retaining wall were injected with microfine cements at a 4:1 w:c ratio through 20mm injection lances to stabilise the sand. However, when excavated after 28 days of curing, in some place a completed seal around the sewer had not been achieved. This is considered to be due to a flow of water along the sewer around the perimeter due to cracked and damaged pipes where the grout was washed away and the ground was poorly treated in these areas.

Conclusion: Permeation grouting cannot be carried out in flowing ground water.

3.2 BEACH STREET, COOGEE - NSW

Figure 5: Grouted sand underpinning Beach Street, Coogee

Development of the basement area beneath a property in Beach Street, Coogee required shallow footings to a depth of 1.5m to enable underpinning. The aeolian sand was treated by permeation grouting with 20 mm diameter lances through the floor boards of the ground floor with stable ultrafine grout with water-cement ratio of 4:1 and micro-silica in rows with primary holes 1m apart and then secondary holes injected on consecutive days. After two weeks the basement was excavated in small sections and the grouted sand replaced with concrete to provide the final underpinning of the building and allow completion of the excavation of a new basement level.

Conclusion: Permeation grouting in Sydney sand increases the strength making it suitable for underpinning large structures.

3.3 ALEXANDRIA POST OFFICE SITE

Development of the Post Office site in Alexandria required the stabilisation of contaminated ground due to historic industrial usage of the site. Due to site confines it was necessary to treat one corner of the property with permeation grouting to reduce permeability and demobilise any contaminated groundwater. The natural ground condition was Botany sand and a stable ultrafine grout with water-cement ratio of 4:1 and micro-silica was injected at 0.5m spacing in primary, secondary and tertiary holes over consecutive days to create a grouted block.

Conclusion: Permeation grouting can reduce permeability and therefore limit dispersion of contaminants in ground water.

3.4 UMINA WOOLWORTHS

Figure 6: Permeation grouting to limit settlement due to sheet piling at Woolworths Umina

Sheet piles were installed to provide a retaining wall for basement excavation in Umina through aeolian sands. The site included prefabricated housing along one boundary and a low-level brick building founded on shallow footings in the south east corner as part of the public library complex. During the installation of sheet piles adjacent to the library complex, settlement occurred creating cracks in the brick walls. Therefore, the grouting crew was mobilised to carry out permeation grouting along the interface between the sheet piles and the brick walls to provide underpinning to the shallow foundations. The grout was injected through 20 mm diameter lances installed at 45 degrees to treat the material beneath the shallow footings. The primary holes were spaced 1m apart and secondary holes were installed in between on consecutive days.

When the sheet piling was continued after permeation grouting was completed and the grout allowed to achieve some strength over 7 days - the settlement after was not only reduced but the cracks created earlier were seen to close.

Conclusion: Permeation grouting can be used to limit consolidation of soils due to vibration.

3.5 MINNEAPOLIS CRESCENT, MAROUBRA

A new development in Minneapolis Crescent, Maroubra required an excavation for basement parking. A retaining wall was created by permeation grouting around the site for a distance of 40 m to enable excavation to bedrock at a depth of 3 m below surface.

Figure 7: Grouted sand for 3m deep excavation at Minneapolis Crescent, Maroubra.

The ground conditions consisted of Botany Sands with minor layers of peat and Coffee Rock (a layer of iron enriched sand with higher compressive strength). The sand was stabilised with interlocking 500 mm diameter columns of grouted sand 4 m wide, by installing 20 mm diameter lances with primary holes 1 m apart and secondary holes installed on consecutive days. The depth of the permeation grouting was limited due to a hard layer at the base of the excavation through which the hand installed spears could not penetrate.

The bulk excavation was successfully completed but when the hard layer was later removed by hydraulic hammer some of the permeated columns of sand became detached. Anchors were installed through the permeated ground to provide tie backs and stabilise the sides of the excavation.

Conclusion: Permeation grouting can be used to stabilise ground to be used as retaining walls for excavations but the excavation but may need to be further supported to maintain stability.

6 CONCLUSIONS

This paper has shown that permeation grouting is a suitable process for many applications of ground improvement in Sydney. However, its application is limited by the soil conditions and whilst ideal in the aeolian sands, it may not be appropriate in the fluvial deposits which are richer in silts and organic matter. However, it has been shown that the process can be used for underpinning and construction of small retaining walls, and retention of contaminated ground water.

It is considered that if constructed properly, with good quality assurance, then the contractor can have great confidence in the effectiveness of the result. However, testing the result is not easy. CPT testing has been used to show ground improvement but if this is not carried out immediately after grouting then the ground will become too hard to test. Coring with diamond cores in small diameter has been shown to be ineffective as the shear force of the coring causes a breakdown of the cemented sand particles. It is considered if quality cores are need for laboratory testing then these should be taken using a triple tube core barrel. Further, samples of grouted sand have been shown to have increased strength characteristics up to 120 days after grouting.

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HOW WIDE CAN A ROCK CAVERN BE OPENED?

Keith W.K. Kong [1] and Jurij Karlovšek [2]

[1]*Geotechnical Principal, Coffey Services Australia Pty Ltd, Abbotsford, VIC, Australia*

[2]*Lecturer in Geotechnical Engineering, University of Queensland, Brisbane, QLD, Australia*

ABSTRACT

The largest known cave, Sarawak Chamber, which is in the Gunung Mulu National Park, Malaysia, is a remarkable natural wonder and measures 700m by 400m and over 70m in height. A cave of this size can fit the entire city streets and skyscrapers. Theoretically, empirical rock and structural mechanics principles could be used to interpret these physical phenomena of large span rock-opening structures. On this basis, hypothesizing a jointed-rock cavern could be supported by reinforced rock arch using rock reinforcement techniques formed by application of confining pressure via the bolts or cables, utilising the inherent strength of the rock to support the rock load above the opening. This reinforced arch is considered as a continuous arched-structural beam all along the cavern. Generally, rock caverns and tunnels are designed mostly to be arch shaped, either circular or parabolic. However, the function of arch shape, geometry, span and ratio of the rise of arch for rock excavations have seldom been discussed, together with rock reinforcement applied on the opening periphery. An investigation initiative is proposed to examine the maximum span of the opening with respect to the critical load of the arch for the opening; properties of rock mass; and other variable coefficients related to geometry of the arch. This is done by using the rock reinforcement method together with the Dinnik's equation, which is a simplified rule with variable coefficients for the analysis of the elastic arch when the loss of stability occurs. A numerical model is conducted in this paper to get a preliminary concept and agreement on this combination applications between rock mechanics and structural design. A desktop investigation initiative is outlined at the next stage for further verification of the concept on how wide a rock cavern could be opened.

1 INTRODUCTION

For the past few decades, some local governments like Hong Kong, Singapore and Scandinavia region, suggested to build the city below ground in order to release more surface land area for building a better living environment. However, creation of underground space has its physical constraints either in soil or in rock. Construction of an underground space by cut-and-cover method in soil, the earth retaining structure cannot go deeper because there is a serious issue with supporting the soil's outward pressure and the limitations of construction equipment. For rock caverns, the effective opening span is controlled by rock mass structure and quality, rock strength and insitu stress fields even it is with or without rock pillar support.

Interesting is that natural caves with span over 80 m can be found around the world. The largest cave "Sarawak Chamber" (formerly the Mulu Caves are in the Gunung Mulu National Park in Borneo, Malaysia) is a remarkable natural wonder with size of about 700 (L) by 400 (span) meters and at least 70 meters high (Figure 1). With this size of cave, it is capable of fitting entire city streets and skyscrapers. In concept, it is believed in that insitu reinforced rock arch is able to form a similar large span of arched or fixed beam support (White and White, 1969 and Davies, 1951) for rock cavern.

Figure 1: One of spectacular views of Sarawak Chamber (Mulu National Park, 2020).

Well understood that arched cavern provides a more stable structure then a flat roof cavern. As of structural mechanics, arches and arched structures have a wide range of applications in bridges, dams and can also be applied to building

structures. For a structural unit, an arch is a member shaped with supported loads in which these loads are transmitted to support primarily by axial compressive thrusts in the arch. The member must be sustained by supports capable of developing lateral as well as normal reaction components.

With reference to rock masonry arch and structural arch stability theories then applied on rock excavations using reinforced rock arch design that is based on the behaviour of rock bolts to act as reinforcement of the rock arch around the opening appears to be applicable in concept. Kong (2018) study found that the rock reinforcement approach for hard rock openings provides thrust capacity equivalent to the internal support by means of improving the strength of the rock mass at the excavation boundary by application of confining pressure via the bolts. Therefore, the rock arch formed by the tunnel periphery (i.e. arched roof and walls) is considered to be a supporting arch capable of sustaining a thrust at the arch ends.

Rock caverns and tunnels are designed mostly to be arch shaped either circular or parabolic. However, the function of arch shape has seldom be studied and discussed, together with rock reinforcement strategy applying on the rock openings. Imagining the reinforced rock arch (for the rock cavern) as an arched structure that is under the compressed load (i.e. the rock load), the critical load (of this compression load) is the maximum load at which the structure is still stable. The critical load for the arched structure is related to geometry of the arch such as arch lift, span, shape (e.g. circular of parabolic) and the properties of rock mass quality and strength around the opening.

The following sections give an initial insight in understanding the function of the reinforced rock arch how feasible acting as an arched structure to build a large span hard rock opening; in order that a desktop study proposal for further investigation is outlined.

2 ARCH STRUCTURES

Arched structures are often used as supporting elements for bridges or other long-span structures. Majority of these cases of arch members are considered as a purely under compression in a static condition. Thousands of years ago, large span arched structures such as masonry arch bridge (for example of the Gutach Bridge built in 1900 with a main span of 64m, see Figure 2) had been used for several hundred or thousand years and these structures (or bridges) can still be found around the world. In twentieth century and beyond, structural arched bridge with span over five hundred metres can be engineered and built (under different site conditions and terrain constraints; e.g. the New River Gorge Bridge built in 1977 with main span of 518 m over the New River Gorge near Fayetteville, West Virginia in US).

Figure 2: Gutach Rail Bridge built in 1900 with a main span of 64m (Structurae, 2020).

Arch bridges or structures work by transferring the weight of the bridge and its loads to the supports primarily by axial compressive thrusts in the arch. In addition, the member must be sustained by supports capable of developing lateral and normal reactions components. Having a given loading, the arch shape must be chosen to avoid the introduction of bending moments.

This arched structure is a simple beam although the induced stresses constitute bending moments as well as axial thrusts. Of particular concerns, the criteria for a credible arch system are explicit: (i) the supports to be capable of sustaining lateral thrust, and (ii) the structural member to have an axial shape such that these trusts are indeed developed under load.

Apart from the stone (or rock) masonry arch configurations, the modern engineered arched structure or bridge is considered as a monolithic or elastic structure. These structures are analysed using elastic theory (Kirchhoff, 1876; Timoshenko & Gere, 1961; Dinnik, 1946; Pippard, 1948; Pippard & Chitty, 1951) whereby the entire structure is assumed

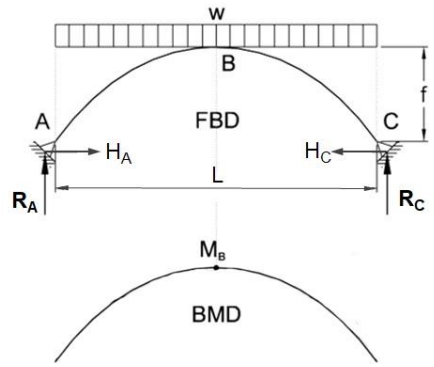
to be a monolithic elastic unit where the true crown thrust is determine using equation that take into consideration of the deflection or elastic distortion of the material under load.

2.1 STRUCTURAL ARCH BEHAVIOURS

The basic arch beam equation is shown on Figure 3 below. The principal of arch is when the load is uniformly distributed to them, the forces resolve into axial compressive stresses and thrust at the fixed base or support points (if adopted) then the base cannot perceive and transmit the bending moments. The fundamental feature of arched structures is that horizontal reactions appear even if the structure is subjected to vertical loads only. These horizontal reactions HA = HC are called a thrust (Figure 3). This phenomenon is known as arch action.

Given the equation, as the height or rise of the arch decreases, the outward thrust increases and as a result, maintaining the arch action and preventing the arch from collapsing, and thus internal ties or external bracing must be employed. The efficiency of an arch can be demonstrated by comparing it with a beam of the same span under the same loading conditions. However, some structural response behaviours of arch are considered important for supporting uniform distributed static load, they are: -

- Fixed arch: statically indeterminate structures to the third degree, the end points require solid restraints. These types of arches are the most ridged but are sensitive to relative settlement at the supports as well as any additional forces created by changes in temperature.
- Two-hinged arch: statically indeterminate to the first degree and although are not as rigid as fixed arches, they are somewhat insensitive to relative settlement at the supports.
- Three-hinged arch: statically determinate and are not affected by settlement or the additional forces created by temperature changes.
- Parabolic arch: because both the shape of the arch and the shape of the bending moment diagram are parabolic, when the arch is subjected to a uniform load, the bending moment at every section of the arch is theoretically zero. The arch will be under pure axial compression.



$$R_A = R_C = \frac{w \cdot L}{2}$$

$$H_A = H_C = \frac{w \cdot L^2}{8f}$$

$$M_B = 0$$

Notation:

FBD = free body diagram

BMD = bending moment diagram

A, B & C = points of interest on arch

f = height of arch from supports, m

H = horizontal reaction load (N) at bearing point

L = span length under consideration

M = maximum bending moment, Nm

R = vertical reaction load (N) at bearing point

w = load per unit length, N/m

Figure 3: Simple arch beam equation.

2.2 ELASTIC ARCH AND STABILITY THEORY

Robert Hooke was the first to carry out scientific research on arches. His main finding was about the shape of an arch which could stand on its own. He provided his finding in an anagram (Hooke, 1675) which in English reads; "As hangs the flexible line, so but inverted will stand the rigid arch". In later years, Pippard et al. (1936; 1938; 1941; 1948; 1951; 1952; 1968) worked on arch bridge problems and conducted a series of experiments on model arches and concluded that the voussoir arch behaved elastically within certain limiting loads. He also demonstrated that the collapse of arch bridges was due to formation of hinges as the result of cracking. He then derived several equations using strain energy methods to calculate collapse loads based on the assumption that the arch ring was made of a homogeneous linear elastic material.

The mathematical basis of stability theory of arches was first implemented by Kirchhoff (1876). The earliest systematic analysis of equilibrium stability was performed by Bryan (1888). Southwell (1914) continued further studies of the general stability theory of equilibrium of elastic bodies. Significant contributions (from Russia and German) solving the problem of stability of arches added by Timoshenko (1961), Morgaevsky (1939, 1961), Dinnik (1946), Chudnovsky (1952). Experimental work on the subject was contributed by Gaber (1934), Dinnik (1946), Pavlenko (1966).

Stable equilibrium state means that if the structure, under compressed load is disturbed from an initial equilibrium state and after all disturbing factors are removed, then the structure returns to the initial equilibrium state. This is concerning

to the elastic structures. If a structure consists of plastic or elasto-plastic elements, then a complete returning to the initial state is impossible. However, equilibrium state is assumed to be stable, if a structure even tends to return to the initial equilibrium state. In case of rigid bodies, the stable position of a structure in a stable equilibrium form of a deformable state is assumed that the acting compressed load is less than the critical one.

The critical force 'q_{cr}' is the maximum force at which the structure holds its initial equilibrium form (the structure is still stable), or minimum force, at which the structure no longer returns to the initial state (the structure is already unstable) if all disturbing factors are removed.

Unstable equilibrium state means that if a structure under compressed load is disturbed from an initial equilibrium state and after all disturbing factors are removed, then the structure does not return to the initial equilibrium state. In this case, the acting compressed load is larger than the critical one.

Change of configuration of a structure under the action of compressed load is called a loss of stability of the initial form of equilibrium or a buckling. If the compressed load is a static one, then this case is referred as the static loss of stability. If a structure switches to other state (as a result of loss of stability) and remains in this state in equilibrium, then this new equilibrium state is called the adjacent form of equilibrium.

The state of a structure that corresponds to a critical load is called the critical state. The switching of a structure into a new state occurs suddenly and as a rule leads to the collapse of a structure. The theory of static stability of structures is devoted to methods of calculation of critical loads.

For the analysis of stability of the uniform parabolic arches with different boundary conditions, arches are subjected to vertical loads only in all cases. The parabolic uniform symmetrical arch is subjected to vertical uniform distributed load q within span of an arch (Dinnik, 1946). This calculation derived by Dinnik is a homogeneous differential equation of third order with variable coefficients with respect to bending moment M which arises in the arch when the loss of stability occurs, and the parameter of critical load is $q^* = q \cdot R_0^3 / E \cdot I$. (where R_0 is the radius of curvature at the crown of the arch).

The Dinnik equation (Dinnik, 1946) shows that the critical load of arch is proportional to $E \cdot I/(span)^2$. For a parabolic arch subjected to vertical load uniformly distributed on a horizontal projection, it may be calculated by formula:

$$q_{cr} = K_v \frac{EI}{span^3} \tag{1}$$

$$H_{cr} = K_h \frac{EI}{Span^2} \tag{2}$$

where: q_{cr} is the critical vertical load of arch; H_{cr} is the critical horizontal reaction of arch; K_v and K_h is a coefficient (see Table 1); E is an elastic modulus of arch; I is the moment of inertia of arch; and $span$ (or L_s) is the arch span.

Table 1: Critical-load and horizontal reaction parameters for uniform elastic arches in pure compression

f / L_s	Three-Hinged Arch		Two-Hinged Arch		Fixed Arch	
	$(q_{cr} \cdot L_s^3) / (E \cdot I)$	$(H_{cr} \cdot L_s^2) / (E \cdot I)$	$(q_{cr} \cdot L_s^3) / (E \cdot I)$	$(H_{cr} \cdot L_s^2) / (E \cdot I)$	$(q_{cr} \cdot L_s^3) / (E \cdot I)$	$(H_{cr} \cdot L_s^2) / (E \cdot I)$
	K_v	K_h	K_v	K_h	K_v	K_h
0.1	22.5	28.1	28.5	36.3	60.9	76.2
0.2	39.6	24.8	45.4	28.8	103.1	64.5
0.3	47.3	20.6	46.5	20.6	120.1	50
0.4	49.2	14.1	43.9	14.1	117.5	36.7
0.5	43	9.6	38.4	9.6	105.3	26.3

3 REINFORCED ROCK ARCH

Reinforced rock arch is a treated ground to improve the strength of the rock mass at the rock opening boundary by application of confining pressure via the bolts. This supporting arch is capable of sustaining a thrust (i.e. rock load) at the arch ends as shown in Figure 4.

As revealed by Kong (2018), Lang (1961) developed one of the long-lived empirical design rules that were established on the basis of a range of laboratory, field and theoretical studies, used for pattern rock-bolting of permanent excavations during the construction of the Snowy Mountains Hydro-electric Scheme in Australia. These empirical design rules have

been reviewed by Brown (1999) and later re-modelled by Hoek (2007) using patterned bolts to stress up aggregate in a frame model.

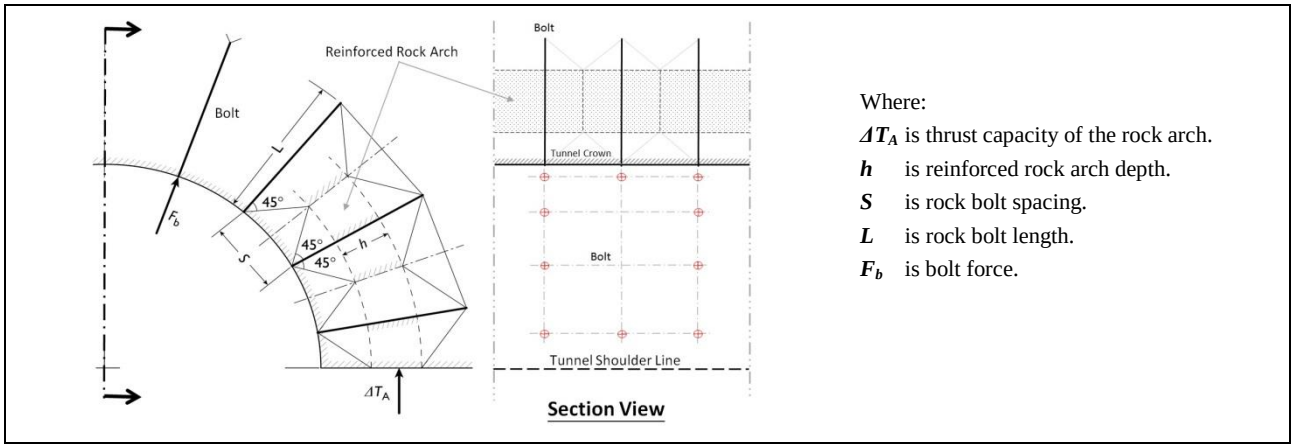
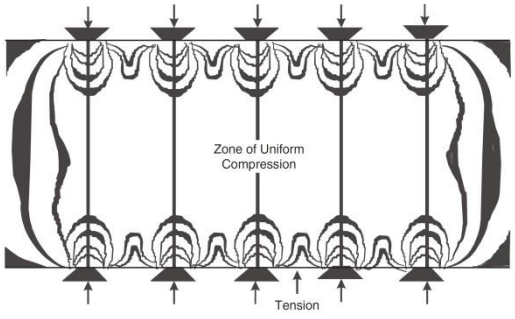
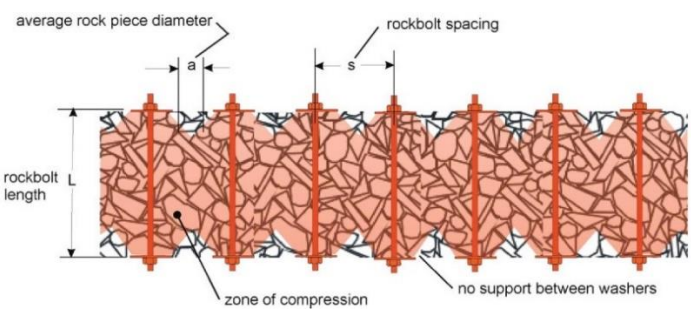


Figure 4: Geometry of tunnel bolting and rock arch thickness (excerpted from Kong, 2018)

Hoek's (2007) model (based on Figure 5(a) of Lang 1961) demonstrated that a zone of compression is induced in the region shown in red [Figure 5(b)]. This will provide effective reinforcement to the rock mass when the rock bolt spacing $\boldsymbol{S}$ is less than 3 times the average rock piece diameter, and the rock bolt length $\boldsymbol{L}$ will be approximately of 2$\boldsymbol{S}$ to 3$\boldsymbol{S}$ (particularly for closely spaced joints rock mass). The "zone of compression" of rock mass has been treated to become isotropic in strength due to the effects of tensioned pattern bolts. An axial pre-stress is developed due to Poisson's effect of normal stress on account of the bolt's pre-tension. This pre-stress can stabilize the rock beam effectively as in the case of pre-stressed continuous concrete beam, which agrees with the Bischoff and Smart (1997) suggested model of reinforced rock arch. If a highly compressible feature such as a fault or a clay seam, or a highly fractured rock mass [i.e. Q (Barton et al., 1974) < 0.1 or Geological Strength Index (GSI) (Hoek, 1994; Hoek et al., 1995; Hoek and Brown, 1997; Hoek et al., 1998) < 30 as suggested by Martin et al. (2003) crosses the compression ring, it is possible that the required compression could not effectively be developed, and that the reinforcement will be inadequate.



(a) Photoelastic stress pattern of bolting (Lang, 1961).



(b) Theoretical zone of compression by bolting (Hoek, 2007).

Figure 5: Illustration of "zone of compression" formed by stressed pattern bolt in media

Based on Bischoff and Smart's (1997) model, the thrust capacity of the rock arch is simply determined by the ratio of unconfined compressive strength to tensile strength, with the provided bolt pressure. The thrust capacity of the rock arch ($\boldsymbol{\Delta T_A}$, half span of opening, in MN) is expressed as:

$$\Delta T_A = \frac{\sigma_c}{\sigma_t} \cdot \frac{F_b}{S^2} \cdot (L - S) \tag{3}$$

where: $\boldsymbol{\sigma_c}$ is the unconfined compressive strength (UCS) of rock mass

$\boldsymbol{\sigma_t}$ is the tensile strength of rock mass

(other notations refer to Figure 4)

According to a Mohr-Coulomb criterion, (σ_c / σ_t) can be rewritten as $q = \tan^2(\pi/4 + \phi/2)$; where ϕ is the rock mass friction angle. ΔT_A can be equivalent to the Q support pressure (Barton et al., 1974) and could be considered as of the critical load of reinforced rock arch. Given the equation, the bolt length and spacing is directly controlled by the UCS and tensile strength of rock mass around the opening. In terms of rock strength, a stronger rock mass may require shorter bolt lengths and contrarily a weaker rock mass may need longer bolt lengths in place.

Kong (2018) commentated that comparing the Eq. (3) to Q-bolt-length equation [i.e. $L_{Q\text{-}system} = 2 + (0.15B / ESR)$, Barton et al., 1974], it is hard to get agreements or close relationship to determine the design bolt length using both approaches. For most cases when adopting closer bolt spacing, the estimated bolt length using Eq. (3) is shorter than the Q-bolt length. Contrarily in a wider bolt spacing arrangement, the estimated bolt length using Eq. (3) is longer than Q-bolt length. With confidence of more than 40 years' practical experience using Q-system and the above findings of bolt length relationship, it is recommended the design bolt length should be determined using both approaches whichever longer bolt length is estimated.

3.1 ROCK LOAD OF UNDERGROUND OPENING

Determination of rock support load of underground openings from a classical assumption made by Terzaghi (1946) to a well-recognized Q-system (Barton et al., 1974) has been briefly discussed by Kong & Garshol (2015). For a short note, Barton et al. (1974) advocated that the support pressure [i.e. roof support pressure (MPa) = $2\, Jn^{1/2}\, Q^{-1/3} / (30\, Jr)$; Jn and Jr refer to Q-system's notations] is independent of the opening width in rock tunnels. Verman (1993) also suggests that the support pressure is practically independent of the tunnel width, provided support stiffness is not lowered. This may be attributed to the dilatant behaviour of rock masses and joint roughness which prevent loosening of the rock mass. Goel et al. (1996) also studied this aspect of the effect of tunnel size on support pressure and found that there is a negligible effect of tunnel size on support pressure in non-squeezing ground conditions, but the tunnel size could have considerable influence on the support pressure in squeezing ground conditions.

Those researchers also concluded that Terzaghi's rock load class (and modified by Deere et al., 1970) has found limitations as summarised below:

(i) Terzaghi's method provides reasonable support pressure for small tunnels (tunnel span of B < 6 m).

(ii) It provides over-safe estimates for large tunnels and caverns (with span/diameter of 6 to 14 m) and

(iii) The estimated support pressure values fall in a very large range for squeezing and swelling ground conditions for a meaningful application.

One of the directive case examples of a 62 m span Norwegian Olympic Ice Hockey Cavern (Barton et al., 1994) showed that Q-system defined support pressures may be more suitable to assess rock load acting on the tunnel/cavern roof, even for larger spanning underground openings. Taking these into account (by exclusion of some underground mining excavation methods), most of the civil tunnels or caverns are constructed in arched roof shape that can be circular or parabolic in form. By consideration of the reinforced rock arch theory utilising the inherent strength of the rock mass in combination with the structural arch stability analysis using the critical load of arch equation (Eq. 1) and Q support pressure. Accordingly, this hybrid support system is proposed based on desktop study using computer program to examine the validity of the background concept and theory.

4 PRELIMINARY LARGE SPAN CAVERN STABILITY ASSESSMENT

To start with the cavern stability assessment, a set of assumptions should be pre-accepted based on the current engineering practices and design standards. Numerical modelling [such as Discrete Element Method (DEM) or Finite Element Method (FEM) equipped with joint networks function] is also be used to facilitate the prediction of boundary deformation of openings. In this assessment, FEM numerical plane-strain model of PHASE² (Rocscience, 2011) is utilised to stimulate the engineering behaviours of the opening. In the models, elastic-plastic analyses are performed with rock support interaction, and Hoek-Brown failure criterion (Hoek & Brown 1997) is adopted in rock material. Joints network are also included in the model. Kong et al. (2016) reported that PHASE² demonstrates the validity to carry out the analysis taking into account any engineering problems and ground conditions such as variable principal stress fields in hard or soft rocks with or without joints network.

In this assessment, 3 joint sets (parallel deterministic network) joint network model with 50m long and 0.85 factored persistence and 1m spacing of each joint set are assigned for the assessment. Regarding Lang (1961) and Hoek (2007) when using the rock reinforcement approach for hard rock opening, Kong (2018) suggested that no rock joints network will be included within the reinforced rock arch compression zone in the PHASE² model. It is because the "zone of compression" of rock mass has been treated to become isotropic in strength with the effects of tensioned pattern bolts forming a continuous rock arch. The general set up of PHASE² model is illustrated in Figure 6.

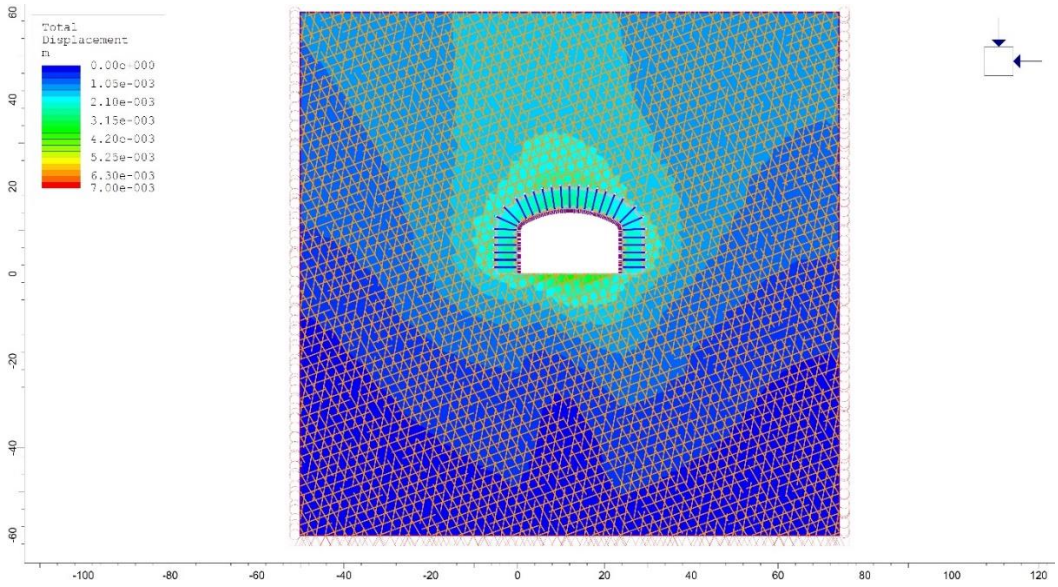


Figure 6: Typical set up of PHASE² model (with joints network) for this assignment

As discussed above, determination of support pressure of the opening will be estimated based on Q-system (Barton et al., 1974) support load equations that are able to provide a high confident level of estimation for the required supporting load. Also, the limit state design by factoring the required support load to 1.5 is considered in the assessment according to Eurocode 0 (BSI, 2010). This support pressure will then be used to estimate the maximum span of the opening using Dinnik's equation that assuming the support pressure is a critical load for the reinforced rock arch. It is considered that three hinged arch's critical load coefficient is more sensitive than two-hinged or fixed arches. Hence, three hinged arch's critical load coefficient (see Table 1) is adopted to determine the potential maximum opening span of rock cavern.

This comes with a set of assumed rock design parameters such as Q and/or GSI value, UCS_{intact} of rock, shear strength of rock joints, design load of rock bolts or cables, etc. Insitu stress 'k' ratio (i.e. σ_h / σ_v) = 59.9/z + 0.664 (Kwong & Wong, 2013; where z = depth (in m) below ground surface) is adopted for the model.

It is understood that the perimeter deformation of an underground excavation is one of key aspects to be check for the cavern stability. The governed vertical deformation limit of the rock arch beam of span/250 will be adopted with reference to Eurocode 2 (BSI, 2014) or AS 3600 (Australian Standard, 2018) for this assessment.

For the model, an elasto-plastic rock material is assumed. Other parameters inputted to the model are listed in Table 2. For this example, the estimated support load based on assumed Jn = 12 and Jr = 1 for the roof (using Q-system) is 313 kPa. The cavern span could be opened by 132 m [using Eq. 1 with critical load of 313 kPa x 1.5 (i.e. a partial factor based on BSI, 2010)] for the ratio of rise/span of 0.2. Thus, rock bolt length = 17.23m is calculated based on Q-system's recommended bolt length.

Table 2: Input parameters for PHASE² model

Unit weight of rock	26 kN/m³	Joint roughness coefficient (JRC)	6
Poisson's ratio	0.25	Joint wall compressive strength	20 MPa
UCS_{intact} of rock	40 MPa	Residual friction angle of joint	35 degree
Intact rock modulus	22275 MPa	Joint normal stiffness	4183 MPa
Rock mass modulus	3557 MPa	Joint shear stiffness	1673 MPa
Bolt capacity (Ø28mm high yield bar)	185 kN	Rock mass quality	GSI = 40 (approx. Q = 0.4)

4.1 FINDINGS OF THE CASE MODEL

The result of the model reveals that the cavern with such a large span opening is stable with a maximum roof and mid-wall deformation of 320 mm and 100 mm is predicted respectively. This magnitude of deformation is less than the current design standards of span/250 (i.e. 132/250 = 528 mm for roof and 50/250 = 200 mm for wall). Also, no yielded bolts are determined in the model. The model output image is shown on Figure 7.

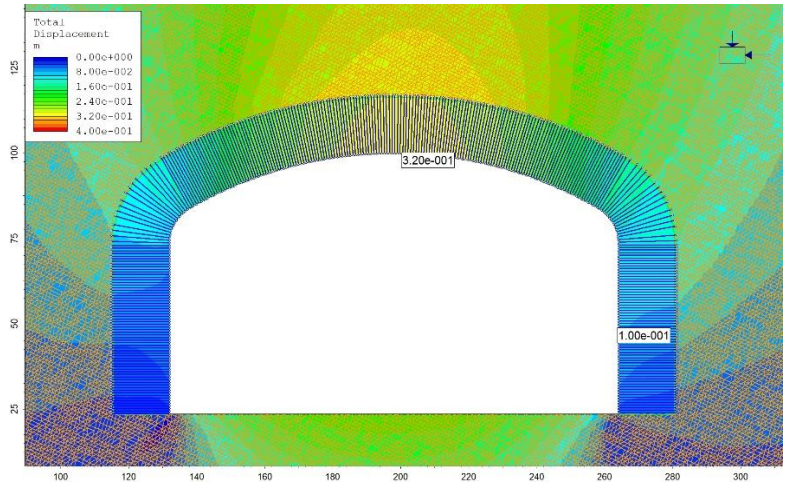


Figure 7: Result of perimeter deformation to the 132 m span opening in jointed rock

Moreover, in terms of isotropic media, hence, no joints network is included in the model, the modelled roof deformation of the opening is 142 mm and mid-wall is 67.5 mm as shown on Figure 8. This deformation is much less than the anisotropic model (see Figure 7 of the above). Based on these two models, a key message promotes that it is important to include joints network for jointed rock opening especially for rock mass quality such as GSI > 30 (Martin et al., 2003) or Q > 0.1 in order to obtain a more reasonable result and ground (or rock mass) responses.

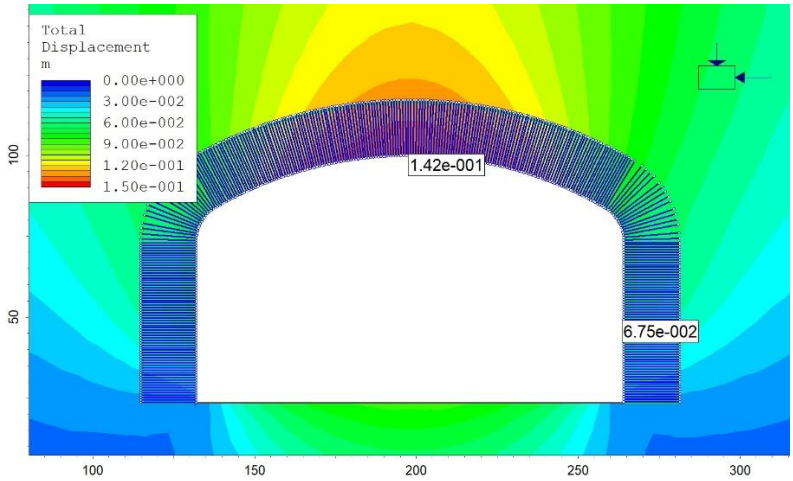


Figure 8: Result of deformation to the 132m-span opening in isotropic media

Based on this preliminary case study in a rock mass of $UCS_{(intact)}$ = 40 MPa; Q = 0.4 and a ratio of rise/span 0.2 for the roof arch shape assumptions, the reinforced rock (arch) support using patterned rock bolts can be considered as a continuous rock arch beam to support the rock opening. This reinforced rock arch could be determined by adopting structural arch beam theory and its structural stability requirements.

Although this model appears to be agreed with the above prescribed assumptions, however, there are large amount of uncertainties still existence and lots of arguments need to confirm in order to get a solid conclusion that this hybrid system by considering both rock mechanics and structural mechanics is valid. These uncertainties comprise the limit of the rock strength, depth of opening (with consideration of insitu stress ***k*** ratio), thus detailed examination is required to be carried in the next stage study.

5 CONCLUSIONS

It is no doubt that rock reinforcement method has been used in many underground rock excavations works successfully. This method can be able to utilises the inherent strength of the rock to support the rock load above the opening. Hence, minimizes the use of structural concrete lining for the opening can be proposed to provide sustainable and cost-effective solutions (Kong, 2018), as the rock reinforcement method reduces in construction time and resources that no formwork or rebar fixing are required for permanent lining. With confidence of more than 40 years' practical experience using Q-

system, the support rock load using Q support load may be feasible to be using in this concept for determining a maximum span of opening.

Further to Kong (2018) study when using rock reinforcement approach, it is evident that the application of confining pressure via pattern bolts suffices to improve the strength of the rock mass; especially for rock strength greater than 40 MPa (that it is based on most of the structural concrete strength of 40 MPa to be used for tunnel lining) and Q could be greater than 0.1 or GSI > 30 at the underground opening. These assumptions could be considered in the next stage of study because it is considered valid as the support points of the rock arch are subject to vertical and horizontal reaction loads induced by the roof support load of opening. At these load support points, rock failures (e.g. bearing or shear failures) could be occurred leading to the overall stability of the arch cannot be sustained.

The preliminary concept of the reinforced rock arch using structural arch beam's behaviours for underground rock excavation appears to be valid, especially for estimating the maximum span of opening under certain rock mass and ground conditions. In order to reinforce this concept and its validity, a range of rock mass and ground conditions are required to be further investigation by obtaining a broad picture and relationship of this hybrid system by consideration of rock and structural mechanics. Thus, a further investigation initiative is outlined in the following section.

6 OUTLINED FURTHER INVESTIGATION

Thirst for continuing demand of usable land to build a better living environment is one of key challenges for many cities and countries such as Hong Kong, Singapore, Finland, etc. The use of the underground space such as development of rock caverns will be one of the solutions to increase land supply. The initiative of this study attempts to deduce a rule or method of identifying a rock cavern with the potential maximum opening span under certain of rock and ground conditions for the development during the feasibility and detailed design stage.

Using the assumptions as discussed in the above, the further investigation will be based on the desktop study using FEM computer modelling (i.e. PHASE²) to assess the relationship between:

(i) reinforced rock arch by rock bolting/cabling and structural arch's equation (i.e. Dinnik equation's critical load of arch);

(ii) the function of arch shape, geometry, span and ratio of the rise of arch (i.e. the ratio of rise/span of 0.1, 0.2, 0.3 and 0.4 to be examined with respect to Table 1) for rock excavations;

(iii) two rock mass strengths of moderately strong (e.g. 40 MPa) and strong rock (e.g. 70 MPa) with respect to maximum span of opening;

(iv) two rock mass qualities of Q = 0.2 (GSI = 35) and Q = 1 (GSI = 45) with three joint sets (parallel deterministic joints network), and 1m and 1.5m spacing respectively to be include in the model;

(v) 60 m and 250 m rock cover with variable insitu stress using an equation [for example: $\boldsymbol{k} = 110.0/\boldsymbol{z} + 1.29$ (Kwong & Wong, 2013), where $\boldsymbol{z}$ = depth (in m) below ground surface]; and

(vi) Additional models for stratified rock mass conditions with horizontal and 45-degree inclined rock structures might be included in the study.

Based on the above conditions, at least 32 models will be assessed; also, detection of boundary conditions/deformation of the opening, and stresses change at cavern crown and shoulder will be examined. Also, a range of failures or instability evaluation criteria could be adopted to assess the stability of underground openings, such as displacement criteria including 'Roof Sag Ratio', 'Wall Convergence Ratio' and 'Floor Heave Ratio' (Zhang & Mitri 2008); and the depth (or extent) of yielding zones (Perras & Diederichs, 2010, 2016; Vlachopoulos & Vazaios, 2018 and Abdellah, 2017) into the rock mass surrounding tunnel opening. These criteria will be used to check the FEM models in this study.

Q-system (Barton et al., 1974) defines that Q-values between 0.1 and 1 is classified as a very poor rock quality. For Q = 0.1, the rock mass is highly fractured in nature and more joint sets with closely spacing need to be considered. However, it appears to be difficult by including more closely spaced joint sets in PHASE². As reported by Kong et al. (2016), the PHASE² programme will take a considerably longer time to finish, or some undetermined "no response" situations may be encountered. Therefore, this study will focus on Q-value between 0.2 and 1.

The assessment results could be able to come up with conclusions and findings, such that the key domain of this investigation is to modify the critical load prediction (based on Table 1 above) of structural arch for rock reinforcement with respect to different shape of arch rise. The proposed new equation (i.e. the critical load of reinforced rock arch) could be a function of opening span, rock mass modulus, reinforced rock arch size as well as some other factors related to in-situ stress ratio and material strength reduction.

Also, the result of this study possibly could give a reference guideline for land use planning for underground cavern development, and/or mining excavation development in a feasibility and concept stage.

Attention is drawn that although the concept of the study and investigation models are a computer simulation, however, in reality of all cases of underground construction, one of important strategies is to ensure the performance and ground responses especially of large span openings; on-site deformation monitoring should be necessary for further validation in which Eurocode 7 Part 1 (BSI, 2013) the observational method could be performed to offer a substantial set of advantages in terms of avoiding the "calculation trap" or "uncertainty", but this is a crucial alternative solution during construction. If this maximum span opening estimation method is finally be assessed to be valid and potentially to be adopted for construction, the detailed design and assessment process shall come with the observational method. By doing this, it could give a complete cycle to evaluate the results between the analyses and the monitored performance.

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DESIGN OF A PIPELINE PROTECTION STRUCTURE OVER COMPRESSIBLE GROUND

Hashan Subasinghe[1] and Doru Bobei[2]

[1]*Senior Geotechnical Consultant, MIEAust Advisian, Worley Group*

[2]*Principal Geotechnical Consultant, MIEAust, Ph.D Advisian, Worley Group*

ABSTRACT

This paper outlines the main geotechnical challenges associated with the protection of an existing major oil pipeline due to construction of an arterial highway within an area of compressible soils.

A detailed ground characterisation was carried out to understand the performance of foundation soils under embankment and traffic loading. This consists of a thorough interpretation of shear strength and consolidation characteristics to inform the design of a piled concrete slab protection structure. The design methodology was developed with the following three (3) key project drivers in mind:

- A solution that adopts piles to act as "settlement reducers" instead of a rigid piled alternative. The benefits of this approach are viewed through the lens of eliminating the formation of a so-called "hard point" within the road alignment measured at the pavement level.
- Mitigation of embankment fill stresses directly impacting on the performance of the pipeline.
- From a project perspective, the design aimed at achieving potential cost savings during the construction phase.

A 3D finite element model was developed in PLAXIS 3D to model the soil-structure interaction, coupled with a pipe stress analysis in ABAQUS 3D to model the soil-pile-pipe interaction and pipeline performance during primary and secondary consolidation.

1 INTRODUCTION

The current trend in Australia is to build additional capacity within the existing road network through re-development over suburban land. This will achieve improved transit times for communities, as well as ensuring the uninterrupted freight of goods between the main capital cities. Many of these future road developments will involve building high fill embankments over existing critical hydrocarbon pipeline routes which provide a supply of hydrocarbon for many industrial uses within the urban fringe of cities. The extent of the protection structure and pipeline alignment is shown in Figure 1.

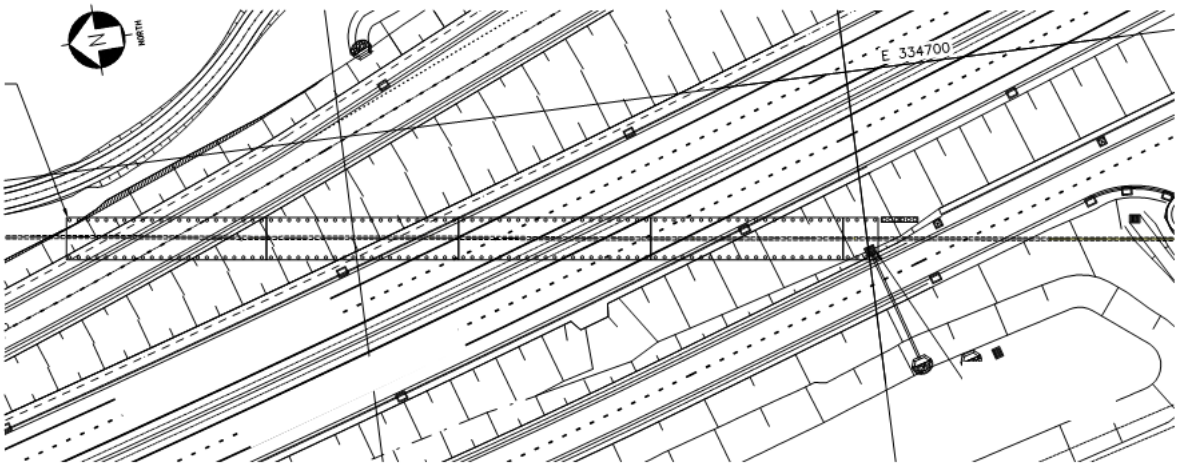

Figure 1: Alignment of existing pipeline and proposed freeway upgrade

The proposed road alignment consists of a dual carriageway, four lane highway, with fill embankments up to 7.5m high. The proposed pipeline protection structure consists of a concrete slab spanning across the buried pipeline, to be supported

by concrete piles. The slab shields the dead and live loads from the pipe and it also minimises the associated ground movement that could affect the pipe.

Prior to the construction of the protection structure, it was required that the affected section of the pipeline be exposed, inspected and recoated. The old coal tar enamel coating was to be removed, and a new high-build epoxy coating (HBE 95 or equivalent) would be applied. This new coating has the capacity to tolerate a higher vibration level arising from the piling and nearby construction activities.

2 GEOETCHNICAL CHARACTERSATION

3.1 REGIONAL GEOLOGY AND ENGINEERING GEOLOGICAL CROSS SECTION

The location of the existing oil pipeline sits within a complex and varied geological setting of broad downthrown areas bounded by uplifted fault blocks. These geological features are presented in Figure 2.

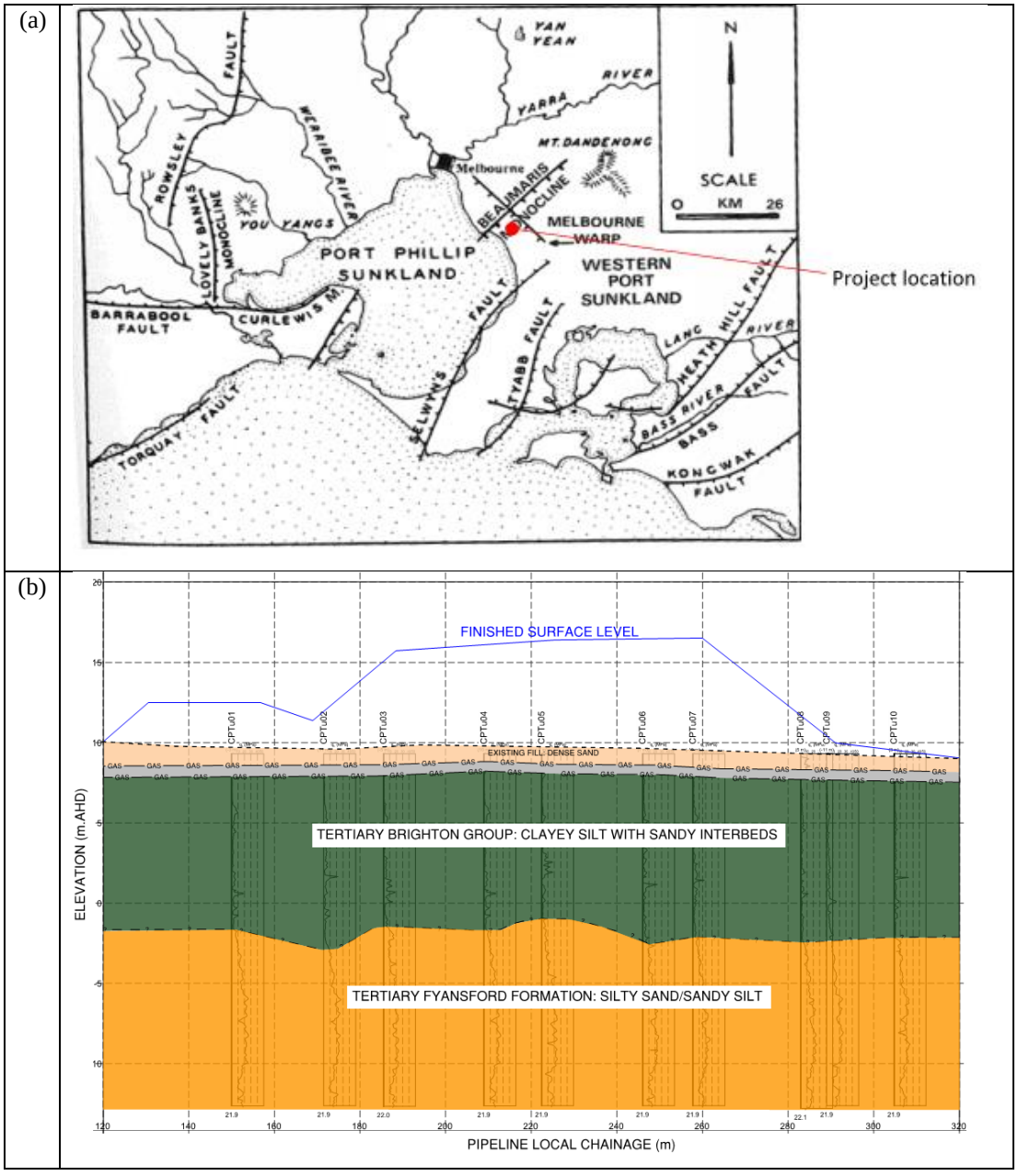

Figure 2: Published geological map and geological cross section, (a) project location, (b) finished surface level

The sedimentary soils developed in a depositional environment that was affected both by local tectonism and eustatic sea-level change, which resulted in a general fall in sea level and associated change from shallow marine to littoral to terrestrial environments.

Based on published geological maps, the subsurface conditions for the site are shown to comprise Quaternary-aged swamp, lagoon and wind-deposited dune sediments. These sediments overlie Neogene Period sediments assigned to the Brighton Group which comprises the Red Bluff Sandstone and Beaumaris Sandstone. Underlying the Brighton Group is the Fyansford Formation, which in turn is underlain by the Silurian Period Melbourne Formation.

An intrusive CPTu testing regime was carried out adjacent to the existing pipeline structure. The interpreted geological long section along the pipeline alignment is shown in Figure 2.

3.2 BASIC SOIL INDEX AND MATERIAL BEHAVIOUR

The behaviour of the Tertiary Brighton Group Clays and Silts were assessed to be critical to the deformation of the pipeline due to the embankment fill loading. The variation of soil index properties is an important consideration for understanding the general behaviour of soil materials. Atterberg limits for the samples taken in the Brighton Group were plotted on a Casagrande Plasticity Chart. In general, this material mostly behaves as a low to medium plasticity CLAY or SILT, although, some higher plasticity samples were also recovered. The material classification according to the USCS was compared to typical CPTu soundings carried out within the project footprint. The interpretation is presented in Figure 3 below for comparative purposes.

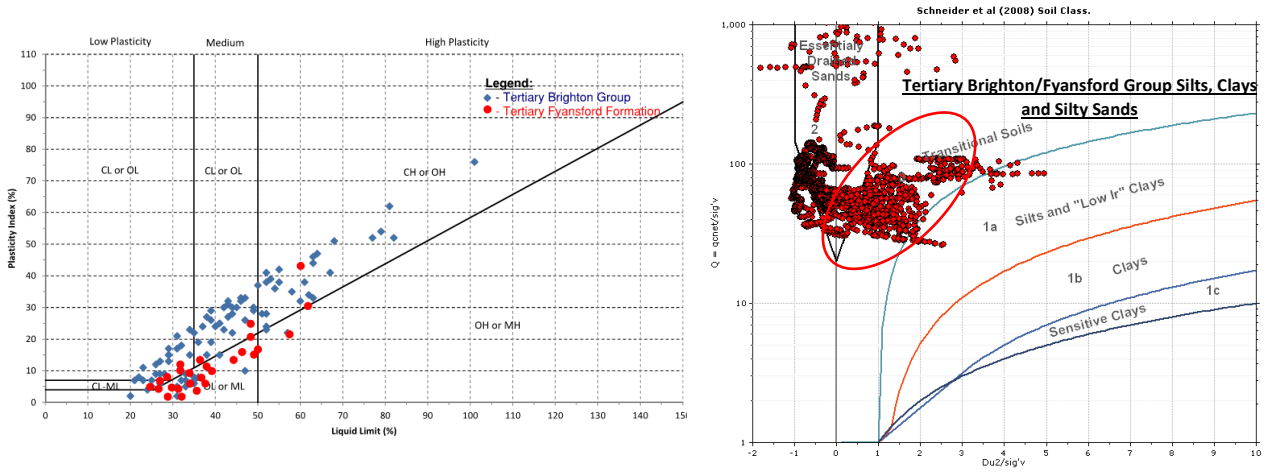

Figure 3: General soil index and behaviour

The Tertiary Brighton Group soil is clearly a transitional material but is likely to manifest in undrained behaviour. Subsequent sections discuss the interpretation of undrained shear strength, deformation and consolidation characteristics of the Tertiary Group Clayey Silts. For the underlying Fyansford Formation, the CPTu interpretation skews to the conclusion that this soil unit behaves mostly as a silty SAND mainly due to the normalised pore pressure ratio, B_q. However, the extensive laboratory testing campaign for the wider project footprint suggests that this material has a high fines content (>30%) and may also behave in an undrained manner. Subsequent foundation assessments have assumed that both soil units tend to behave as undrained, fine grained soils. Groundwater at 1.0m below existing ground level was considered in the following interpretations.

3.3 UNDRAINED SHEAR STRENGTH, DEFORMATION AND CONSOLIDATION PROPERTIES

Undrained shear strength (s_u) has been derived using the CPTu testing carried out within the project footprint. For the Tertiary Brighton Group soil unit, undrained shear strengths were determined using location specific CPTs, with s_u estimated by adopting the correlation by Lunne et. al. (1997):

$$s_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \quad (1)$$

where:

q_t	=	Corrected cone resistance
σ_{vo}	=	Total overburden pressure
N_{kt}	=	Cone factor

An empirical methodology was proposed by Ladd (1991) to provide a platform for a unified framework of correlation of soil strength with stress levels and soil compressibility characteristics. Mathematically such a relationship may be expressed as:

$$\frac{s_u}{\sigma_v'} = S \times OCR^m \tag{2}$$

where:

- OCR is the over-consolidation ratio of the soil.
- σ_v' is the vertical effective stress; and
- S and m are empirical derived coefficients to consider the effect of clay plasticity and soil compressibility characteristics. The relationships proposed to derive these coefficients are as follows:

$$S = 0.2 + 0.05 \times PI \text{ or simply } 0.21 \text{ for PI} = 20\% \tag{3}$$

$$m = 0.88\left(1 - \frac{C_r}{C_c}\right) \text{ or simply } 0.8 \tag{4}$$

In-situ Marchetti Dilatometer readings were carried out to calibrate the in-situ CPTu cone factor. The adopted N_{kt} value of 25 was found to calibrate well with the dilatometer results. It is noted that the N_{kt} = 25 adopted for this project is on the higher end of published data by Lunne (1997) and other researchers. This suggests that the soil is likely to be lightly cemented or structured. This conclusion is in line with the documented geological history of the site, as well as interpretation of previous stress history (by assessing the over-consolidation ratio), discussed in Section 3.4. Deformation properties have been interpreted from laboratory testing and in-situ testing CPTu testing. The interpreted strength and deformation parameters are presented below in Figure 4:

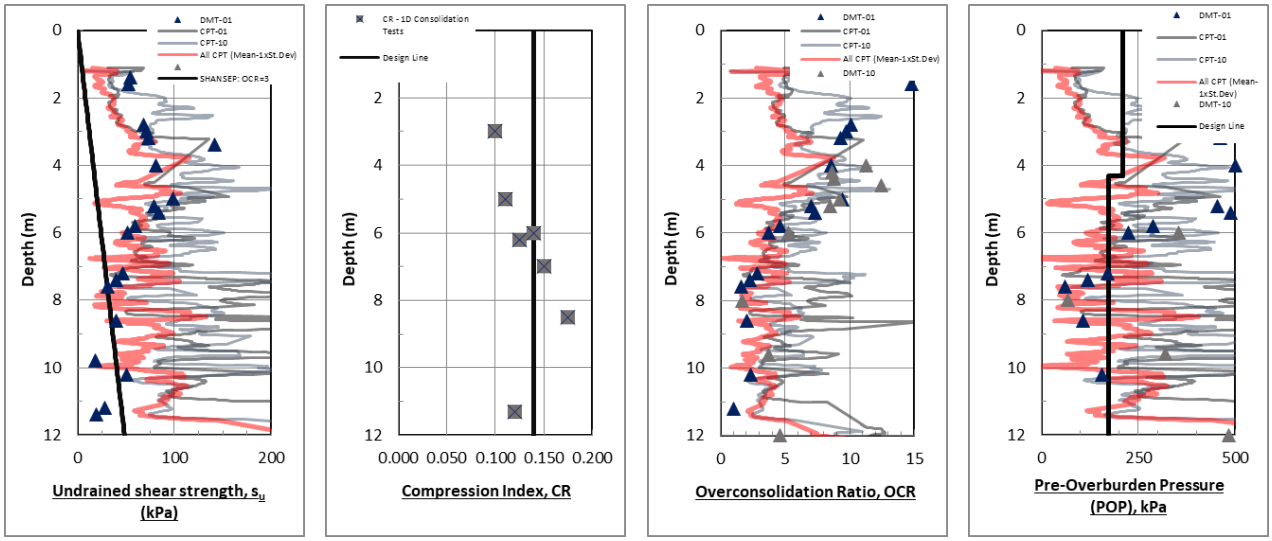

Figure 4: Interpretation of geotechnical parameters

- CR = Compression Index. Soil compressibility interpreted from Oedometer test results on undisturbed soil samples.
- CRR = Recompression index, adopted as 0.15xCR
- Cαe = Creep Index, adopted as 0.04xCR adopted based on creep consolidation testing and previous project experience.
- OCR = kQ_{t1} where Q_{t1} is the normalised CPT cone resistance and k = 0.15 (calibrated using in-situ DMT and oedometer results).

3.4 SUMMARY OF GEOTECHNICAL PARAMETERS

The adopted geotechnical parameters for the pipeline protection structure are summarised below in Table 1:

Table 1: Summary of geotechnical design parameters

Soil Unit	Depth to base of unit (m)	Bulk Unit Weight, γ_b (kN/m^3)	Undrained shear strength, s_u (kPa)	Drained friction angle, ϕ' (deg.)	CR	CRR	Secant modulus, E_{50}^{ref} (MPa)	Pre-overburden pressure, POP (kPa)	Vertical permeability, k_v (m/day)
Existing Fill	1.2	20	-	34	-	-	30	-	8.64
Tertiary Brighton Clay/Silt – Upper	4.3	18	50	-	0.14	0.042	-	210	0.0007
Tertiary Brighton Clay/Silt –Lower	12.0	18	50	-	0.14	0.042	-	172	0.0004
Tertiary Fyansford Formation	-	19	150	-	-	-	30	-	0.0008

3 FOUNDATION DESIGN

3.1 DESIGN PHILOSOPHY

The design of the pipeline protection structures forms a small part of the overall project upgrade, that is, the design of a major highway and all its associated works. As such, an important consideration for this design package is the interaction of the pipeline protection structure with the overall road project. The following key design considerations were of utmost importance when designing the protection structure:

1. Ensuring that the protection structure does not form a significant "hard point" along the final pavement level;
2. Accommodating fill long term settlement (residual primary consolidation and creep settlement) of embankments adjacent to the protection structure;
3. Cost effective design; and
4. Minimising pipe stress within the existing pipeline to acceptable limits.

With these design intents it was decided (through interaction of consultants and client's engineers) that the piles would be designed to behave more like "settlement reducers" rather than rigid piles (bridge abutment piles, for example). This approach proves to be very cost effective in this scenario as the structure is around 200m long and pile lengths could be significantly reduced. When adopting this approach however, it is prudent that detailed numerical analysis is undertaken to model the soil-structure-pipeline interaction. Subsequent sections discuss these assessments in greater detail.

3.2 DESIGN LOADS, STRUCTURE CONFIGURATION AND PILE FOUNDING LEVELS

The protection structure was designed by the structural discipline to minimise the embankment fill stresses developing large deformations within the foundation soils under the pipeline. The design entails the construction of a concrete slab supported by piles on either side along its entire alignment. It was proposed to adopt the Contiguous Flight Auger (CFA) piling technique to install:

- 600mm diameter piles under the fill embankment founded on the Tertiary Fyansford Formation Very Stiff to Hard Clay Unit; and

- 750mm diameter piles under the embankment sides to accommodate the lateral foundation loading.

A cross section of the proposed protection structure is shown below in Figure 5, with design loading summarised in Table 2:

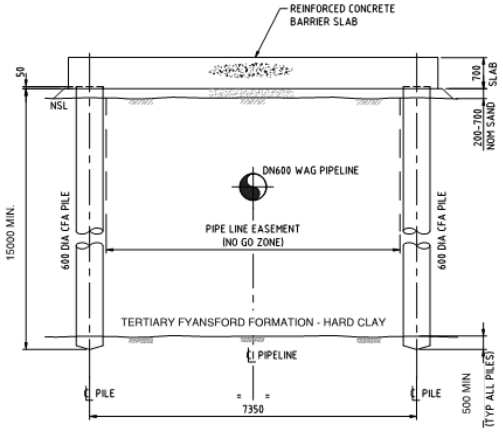

Figure 5: Proposed pipeline protection structure cross section

Table 2: Summary of structural loads

Load Case	Main Piles under Full Height
ULS Axial Force (kN)	1390
ULS Bending Moment (kN.m)	510
ULS Shear Force (kN)	470

3.3 COMPARISON OF SIMPLIFIED α METHOD AND CPT LCPC METHOD

The anticipated Factor of Safety (FoS) of the piles under compressive loading was assessed using the simplified approach and compared to the LCPC CPT method (Bustamante, M., and L. Gianeeselli, 1982). The comparison for 600mm CFA piles and the associated Factor of Safety is presented in Figure 6.

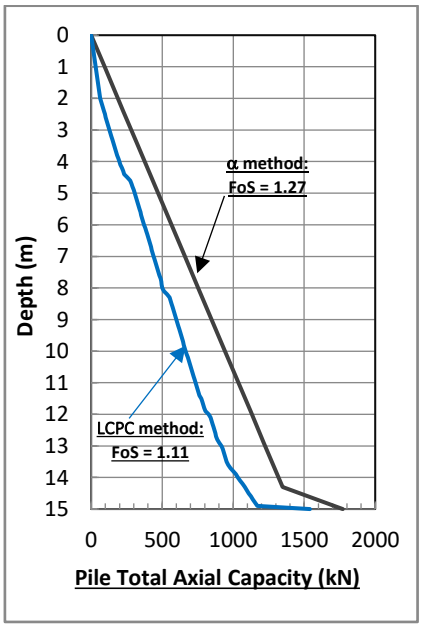

Figure 6: Comparison of α and LCPC method for pile geotechnical capacity

The contribution of end bearing (Q_b) is calculated based on $9xs_u$ for the Tertiary Fyansford formation, for both methods. It was found that the CPT method generally produced very high values of end bearing in this material, most likely due to its cementation. It is anticipated that these cementitious bonds will be destroyed within the region where effective end bearing can manifest, as a result of the CFA piling technique. On the contrary, the LCPC method predicted a lower shaft adhesion contribution compared to the α method, even though a conservative unified undrained shear strength of 50kPa was adopted for the Tertiary Brighton Group Clays.

4 NUMERICAL ANALYSIS

4.1 PIPELINE DEFORMATION ANALYSIS AND SOIL STRUCTURE INTERACTION: PLAXIS 3D

The pipeline settlement due to embankment loading and traffic loads over the proposed protection structure have been assessed using a full soil-structure interaction model using commercial software PLAXIS 3D. The model considers the following effects:

The 3D model mesh consists of 105253 finite elements and 158541 nodes. 10 nodded tetrahedral finite elements were used in the model. The model is normally fixed on all boundaries, except the maximum vertical boundary (fully free in all directions) and the minimum vertical boundary (fully fixed in all directions). The soft soil constitutive model was used to model the deformation of compressible soils, with parameters summarised in Table 1 adopted in the analysis.

The concrete slab was modelled as a plate element and CFA piles modelled using embedded beam rows with axial skin and base resistance derived according to the LCPC method for axial skin resistance and $9s_u$ for base resistance;

The staged construction loading conditions are summarised as follows:

1. K_0 procedure to determine in-situ stress conditions;
2. Installation of protection structure (plastic load step);
3. Embankment fill construction with a maximum fill thickness of 7.5m (plastic load step);
4. Application of a 10kPa traffic load (plastic load step); and
5. Consolidation of excess pore pressure (up to 95% degree of consolidation) to model the long-term primary consolidation of the pipeline and protection structure.

The 3D finite element model of the pipeline, the protection structure and the proposed embankment loading is shown in Figure 7.

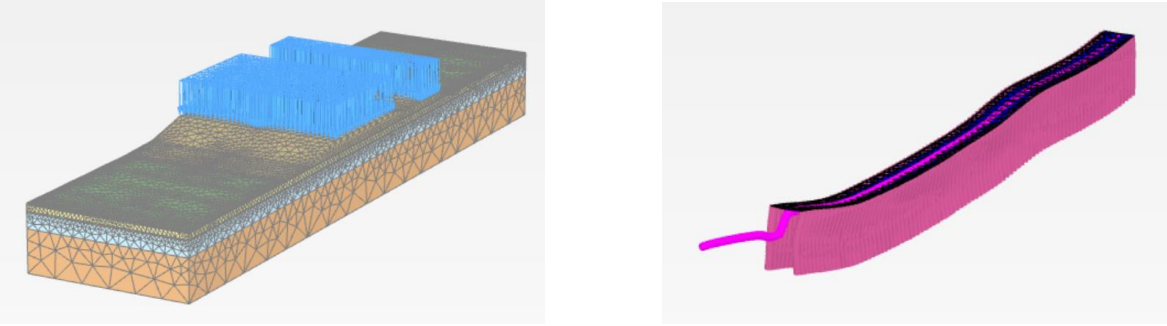

Figure 7: PLAXIS 3D soil-structure interaction model

The long-term secondary consolidation (creep) was assessed separately using a calculation methodology developed by Wong (2007), which was found to fit reasonably well based on oedometer testing results for the Tertiary Brighton Group Clays.

The results of settlement calculations to predicted long-term vertical deformation of the pipeline (including creep effects) is presented in Figure 8. The settlement diagram predicts a gradual change in settlement along the pipeline alignment. A maximum of 120mm settlement is anticipated to develop at pipeline chainage 210m.

The results of the 3D finite element analysis were validated with a settlement calculation using Terzaghi's One Dimensional consolidation theory. Due to the semi-rigid pipeline protection structure, it is difficult to ascertain exactly how much load is transferred to the soil. An estimate of 20% load transfer was considered, based on experience with load transfer to soil from concrete injected columns.

The magnitude of settlement assessed using 1D consolidation theory was **133 mm** for the 7.5m fill thickness and 10 kPa traffic load, assuming a 20% transfer of load to the soil. This demonstrates a reasonably fair comparison between the finite element analysis and the 1D hand calculation and validates the 3D finite element analysis.

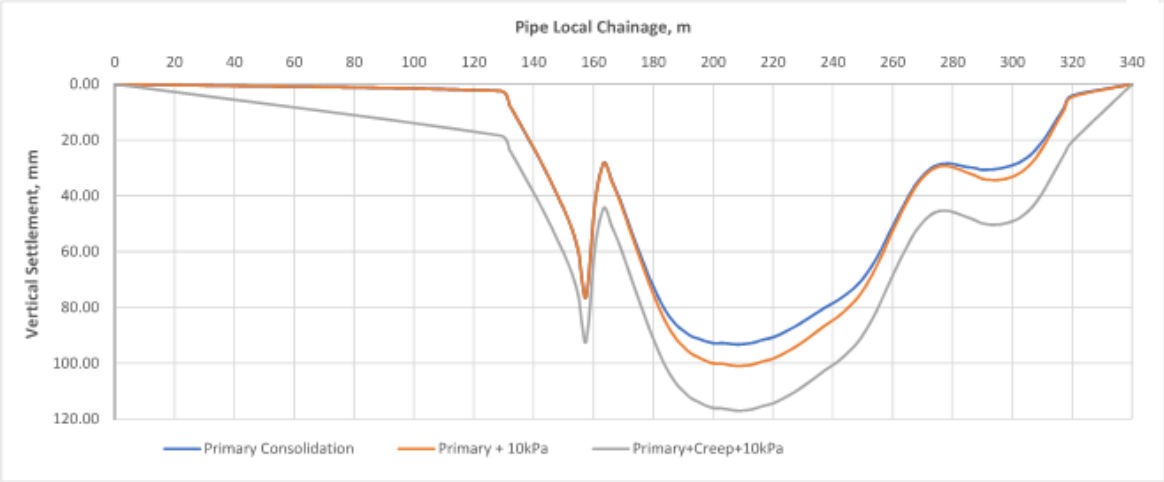

Figure 8: Vertical settlement profile of pipeline

The pipeline deformation profile is provided as input for the pipeline stress analysis for capacity checks. Of particular importance for pipeline stress analyses will be the sudden change in deformation gradient at pipeline chainage 160m. The sudden increase in settlement is caused by interfacing issues with the Freeway off-ram ramp which caused the protection slab to finish early under the embankment batter. A provision was only made to retain the fill batter with 750mm diameter CFA piles to restrict the lateral deformations impacting on the pipeline.

4.2 PIPELINE STRESS ANALYSIS: ABAQUS

The 3D ground displacement profiles computed in PLAXIS 3D were applied as prescribed displacement input to a Pipe-Soil-Interaction (PSI) model in Abaqus to determine the stress state in the pipe. The pipe element in this model can account for the internal pressure, thermal effects, Poisson's ratio effect of a restrained pipe, and soil deformation. The pipe-soil interaction was represented by a series of bi-linear springs along the pipe. The methodology and the advantages of using this approach to analyse pipe stress caused by ground deformation are detailed in Ho and Dominish (2005).

A typical pipe stress plot is shown in Figure 9. The computed pipe stresses were assessed against the requirements in AS 2885.1 to ensure they are compliant.

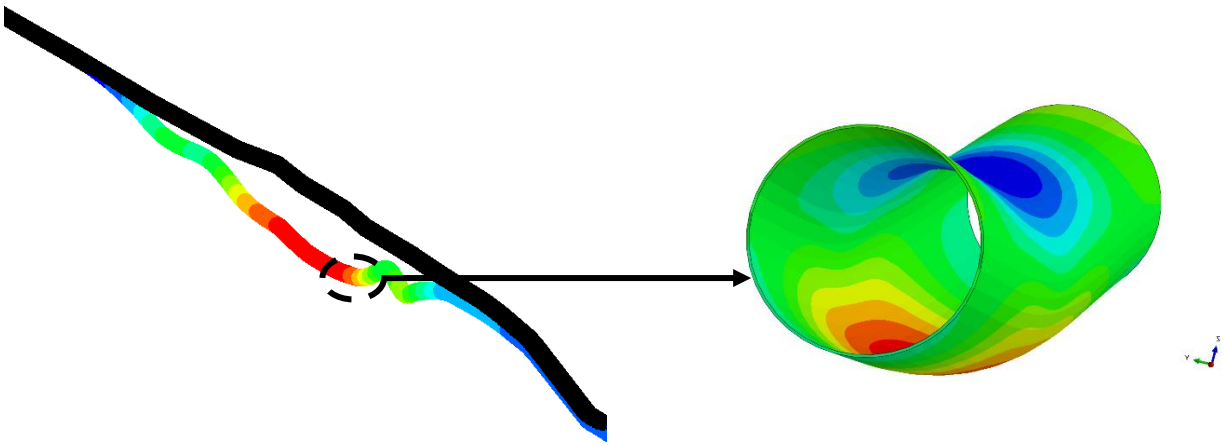

Figure 9: Results of pipe stress analysis

5 DISCUSSION AND CONCLUSIONS

In line with the above findings, the following recommendations were provided for consideration of the construction team:

- The design of the piled foundation system was found to limit the settlement development at the finished pavement surface level to alleviate concerns of potentially creating a “hard point” to exceed the grade change requirements outlined project scope of works at technical criteria;
- This project was delivered in close collaboration with the structural team to design the appropriate connection between the piles and concrete protection slab. A number of iterations were conducted with final design to include a “pinned” connection to prevent development of large shear stresses and bending moments exceeding the pile structural capacities;
- Pre-fabricated steel connections were specifically designed for this project to bring an innovative alternative to address the pile structural design considerations;
- The assessment of the piled protection structure on the basis of “settlement reducers” resulted in a cost-effective design for such a large structure. This design approach was found to eliminate the need for additional ground treatment at transition zones between the protection structure and general embankments;
- A detailed numerical analysis has combined the computational efficiencies of a finite element model (PLAXIS 3D) to predict the deformations of foundation soil materials, with a specialist pipeline stress analysis model carried out in Abaqus 3D.

6 ACKNOWLEDGEMENTS

The authors would like to thank Dr David Ho and Mr Alexander Link of the Advisian Advanced Analysis Group for their work on the project and assistance with inputs regarding the pipe-stress analysis.

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STRENGTHENING REINFORCED EARTH WALLS – KNOX ROAD DUPLICATION CASE STUDY

Antonio Martinez Ramirez [1], Diarmuid Moriarty[2] and Christopher Lawson[3]

[1]aramirez@reco.com.au (+61 (0) 437 276 851)
217 Arden Street. North Melbourne, VIC 3051 (Australia)
The Reinforced Earth Company - Design Manager Southern Region
[2]dmoriarty@reco.com.au (+61 (0) 448 088 261)
Level 4, 20 George Street. Hornsby, NSW 2077 (Australia)
The Reinforced Earth Company - Head Office Team Leader Design
[3]clawson@reco.com.au
Level 4, 20 George Street. Hornsby, NSW 2077 (Australia)
The Reinforced Earth Company – Engineering Manager (Ret'd)

ABSTRACT

In 1979, The Reinforced Earth Company (RECO) designed and supplied Reinforced Earth walls (REWs) at Knox Road bridge over main Western rail line at Doonside, NSW. The arrangement comprised a pair of abutments spanning over rail. In 1979, the REWs were constructed with two traffic lanes on the Eastern side (Stage 1), and with provision for a further two traffic lanes to be added on the Western side (Stage 2). Between 1979 and 2011, the Western side of the abutments were left in a relatively unfinished condition in readiness for the Stage 2 abutment to be constructed.

The original design of the REWs was carried out on the assumption that when the Stage 2 duplication was to occur, the same abutment beam seat, relative founding level and bridge loading would apply. However, the Stage 2 duplication design, developed in 2011, required a larger abutment beam seat and significantly higher bridge loads than the original design assumed. This meant that there was insufficient earth reinforcement capacity within the existing REW structure to safely support the new bridge loading.

To increase the existing capacity, additional galvanised metallic soil reinforcing strips (RE strips) were incorporated into the REWs. This increase in capacity was achieved by a combination of installing new reinforcing strip connections to existing panels and fabricating new facing panels. The work involved a staged construction process with the initial stage comprising removal and replacement of under-strength panels and retro-fitting with new reinforcing strip connections. The second stage of construction comprised conventional construction of the REW to the new finished surface level incorporating newly fabricated facing panels. Impact on existing road and rail users was paramount during the design and construction of the bridge. It was important to keep the existing road open as much as possible, and the reduce the amount of works in the Rail Corridor. This Case Study presents an innovative method of strengthening existing structures, while reducing costs and construction impacts on both road and rail users in an urban environment.

1 INTRODUCTION

Reinforced Earth™ (RE) technology was introduced to Australia in 1974 with the establishment of The Reinforced Earth Company in NSW. RE combines granular backfill with RE Strips and concrete facing panels to produce a coherent gravity structure, capable of retaining significant amounts of earth with a relatively thin facing. The arrangement used on the Knox Road design, both in 1979 and in 2011, was that of a 'true abutment', where the abutment beam seat bears directly on the RE fill, and is not supported on piles hidden by the RE mass and founded in competent strata beneath the RE. 'True' abutments (refer figure 1) were popular at this time but have become less so in recent years due to the requirements of codes such as AS 5100 Bridge Design (Standards Australia, 2017), which requires that bridges over rail be supported independently of the REW. Within Australasia, true abutments are still very popular in New Zealand and the design and supply of a true abutment has been completed in 2019 for the Gawler Freeway Upgrade project in South Australia.

The Knox Road Bridge over rail was a true abutment built in 1979 with provision for two future traffic lanes to be added at a later stage. These added lanes were designed in 2011, with completion occurring in 2013. Between the initial construction date and the duplication, there was a significant increase in bridge loads, with an increase in relative size and lowering the founding level of abutment beam seat.

A detailed analysis of the structure was undertaken in order to assess whether the new bridge loads could be catered for in the existing structure. Several important factors were found to mitigate the effects of the higher bridge loads. A number of these factors will be presented here:

- The method of design in 1979 involved using a lower bound estimate of the effective angle of friction. At the original design, 32.5 degrees was the design value while in 2011, during the design assessment, the value was

found to be a minimum of 39 degrees following shear box testing. The results of the laboratory testing are appended to this paper, which describes the material as a silty sand. Given that RE technology is a recent innovation in civil engineering, there were no design specifications in place in NSW at that time, such as Transport for New South Wales (TfNSW) Specification R57 (TfNSW, 2020) which is now used in most infrastructure projects in the state.

- The original abutment beam seat width was 2.4 m. In the 2011 design, the abutment beam seat was 3.2 m wide, which significantly increased the footprint of the beam seat on the RE fill below, thereby reducing the vertical bearing pressure below the 200 kPa threshold, as recommended by RE.
- During the design phase in 2011, a representative sample of RE strips incorporated in the 1979 works were excavated and assessed by RECO engineers. The site inspection revealed that no loss of the galvanic coating, nor loss of thickness of the structural steel section was observed. Therefore, this allowed the design life to be assessed assuming zero loss of section, effectively resetting the service life of the structure to zero years.

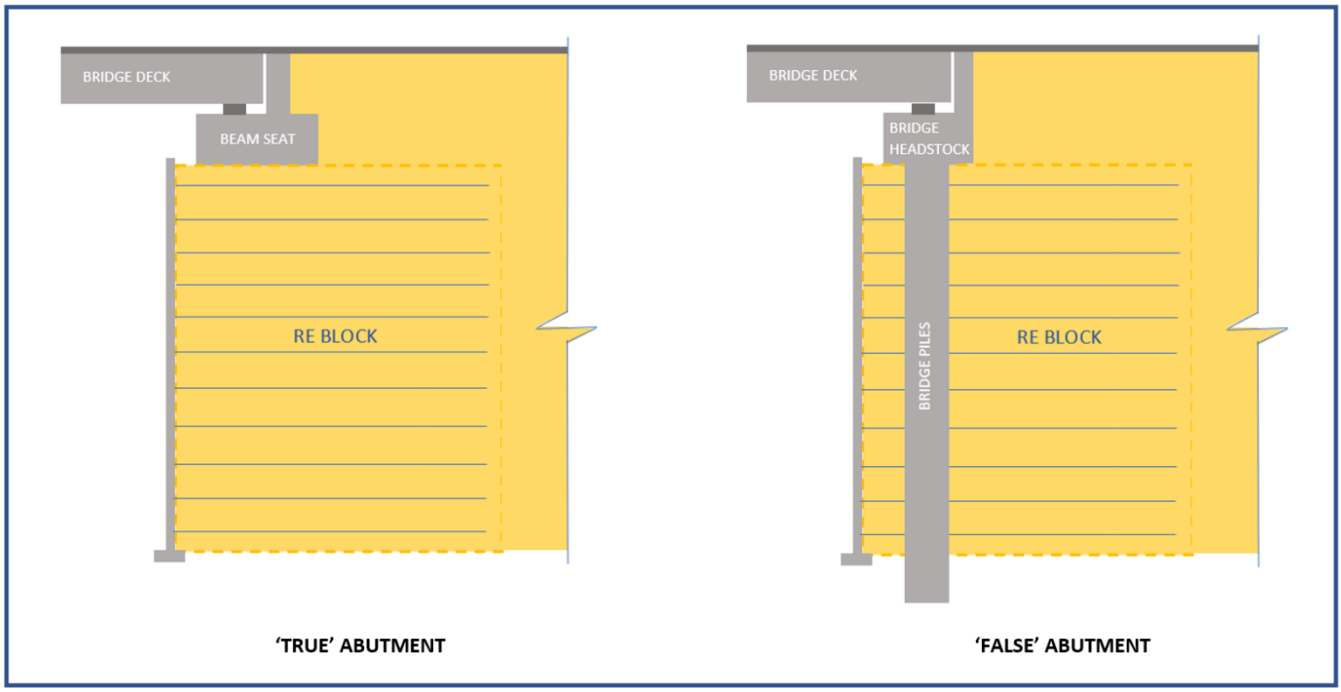

Figure 1: Diagrammatic representation of typical bridge abutment types

2 DESIGN METHODS

In order to assess the new bridge loads in 2011, three methods of design analyses were utilised. Firstly, a now-superseded Working Stress design approach that was originally used for the 1979 design. This method applies material strength reduction factors applied in conjunction with the unfactored applied loads. Maximum stress within the RE strips is taken as 0.6 of the nominal yield stress (equivalent to a FoS=1.67) and maximum allowable frictional resistance of RE strips is taken as 2/3 of the theoretical resistance available (equivalent to a FoS=1.5). Note the original 1979 design was carried out assuming the effective friction angle of select backfill Ø'=32.5° while the present design assessment is based on a nominal effective friction angle of select backfill Ø'=39°. In preparation for this design assessment, several samples of the existing select backfill material were removed and tested for direct shear strength following the AS1289 6.2.1 test methodology (Standards Australia, 2001), returning a favourable result which yielded excess capacity in the existing RE strips.

The second analysis method utilised is based on Limit State design principles to assess RE strips capacity (referred to as the internal stability). This analysis was included only as a comparative design analysis check using current design recommendations and guidelines. This design method would also satisfy the requirements of Australian Standard AS4678 – Earth Retaining structures (Standards Australia, 2002).

The third method was a Limit Equilibrium stability analysis using the program 'Stares'. This program was written at the University of Sydney (Balaam, 1999) for the analysis of REWs and is based on Bishop's simplified method of slices (Bishop, 1955). This method divides the soil above the assumed failure surfaces into "n" number of slices with vertical boundaries. However, the analysis method in Stares accounts for additional terms which take the action of the reinforcement strips into account. Thus, in addition to moment equilibrium between slices, it allows for both pull-out and rupture type failures of each strip. Corrosion is considered by reducing the net area of the strips to account for the predicted reduction in cross-sectional area at the end of the design life. The stabilisation provided by the earth reinforcements are

assumed to be entirely provided by the axial forces generated along their length. Thus, any contribution towards stability due to any shear forces in the reinforcements are ignored. The results of this analysis are expressed in terms of a factor of safety FoS against shear failure along a circular arc. This factor of safety is also assumed to be the factor of safety against rupture and pull-out failure of the reinforcements. This can be expressed as follows:

$$F_{REO} = F_{MAX}/FoS \quad \text{(Equation 1)}$$

where,

F_{REO} = Sum of the restraining forces generated in all reinforcing strips that intersect with the base of the slice being considered

F_{MAX} = Sum of the maximum restraining forces that each strip can provide which intersects with the base of the slice being considered. The force for each strip is the minimum of the force required to rupture the reinforcement and the force required to pull-out the reinforcement strip.

The simplified Limit Equilibrium method satisfies the laws of statics, which is a limitation in its' usefulness. Krahn (2001) has described the fundamental shortcoming of Limit Equilibrium methods being the failure to consider strain and displacement compatibility. He further postulates that a method of overcoming this is to use Finite Element computed stresses within a Limit Equilibrium framework, which will result in more accurate analyses within the familiarity of Limit Equilibrium methods. This type of analysis has not been completed for this structure. However, the use of three methods in analysing the structure is considered to provide counterchecks to support the final design conclusions.

For a bridge structure such as this, a minimum factor safety of, approximately 1.5, is sought. The material parameters adopted in the design are presented in Table 1.

Table 1: Reinforced earth wall design parameters

Parameter	Value	Unit
Soil Maximum (Minimum) Unit Weight in RE Block	21.5 (20)	kN/m³
Soil Unit Weight in General Backfill	21.5	kN/m³
Soil Friction Angle in RE Block and General Backfill	39	deg
Soil Friction Angle on Base of RE Block	36	deg
Maximum vertical spacing of RE strips	750	mm
Existing Strip Width (thickness)	60/40 (5)	mm
Existing Strip Minimum Tensile (Yield) Strength	410 (275)	MPa
New Strip Width (thickness)	45 (5)	mm
New Strip Minimum Tensile (Yield) Strength	550 (400)	MPa
Strip Friction coefficient at surface, f*max (f*min)	1.6 (0.80)	-
Strip Corrosion Allowance	0.75 per face 1.50 Total	mm

Further to the above, it is important to note that REWs are typically sensitive to horizontal loading. Where true abutment configurations are used, there are typically significant horizontal loading considered in the design. These loads typically act normal to the abutment beam, outwards towards the centre of the bridge. The original design catered for 22 kN/m SLS across the abutment beam seat, but in 2011 the loading proposed was more than double, at 45 kN/m SLS. The choice of a fixed restraint on Abutment B (Southern Abutment), and expandable arrangement at Abutment A (Northern Abutment) was made by the bridge engineer, Cardno. This additional loading was not able to be transferred to the RE wall, even considering the addition of new straps. Therefore, in order to avoid a very expensive solution, and specifically to avoid a far more extensive removal of the existing REW structure, new RE strips were attached to the rear of Abutment B beam seat to provide an earth anchor restraint to resist the horizontal braking loads. Abutment A did not have any horizontal loads at the abutment beam seat. The increased loads were provided by the primary designer, Cardno. It is unknown why the bridge loads differ so much between 1979 and 2011, although the requirements of AS5100:2004 are considered in the 2011 design. These requirements include consideration of dynamic loading allowances and minimum ultimate horizontal forces acting normal to the bridge. These considerations may not have been included in the original design.

For Abutment B, where a horizontal braking load is applied, the Working Stress and Limit State analyses did not consider this horizontal loading. It is assumed that all horizontal forces are resisted by the earth anchor RE strips attached to rear of abutment beam seat. However, the Limit Equilibrium analysis considers the horizontal braking force and the presence of the RE strips. The capacity of the RE strips to resist pull-out failure governs the design of these beam seat RE strips. The RE Strip pull out capacity is dependent on the frictional characteristics of an inextensible material and the physical characteristics of the soil backfill and is calculated directly from the vertical stresses obtained from the soil mass above and RE strip resistant length taken as the free length behind the abutment beam seat. The number of RE Strips required to resist the applied loading was calculated using basic principles of soil mechanics. At each layer, the frictional and tensile resistance of the straps was calculated for a number of load cases. To determine the number of straps needed across the abutment beam seat, for each load case the critical resistance per strap was divided into the total force acting across the abutment beam seat.

A critical assumption in the analysis is the Soil – RE Strip Friction Coefficient, f*. This friction coefficient is found to be above unity at low overburden pressures, being a maximum at the finished surface level and reducing to a value of tangent Φ' at about 120 kPa overburden, or 6 m below surface level. Historically, a maximum of 1.5 was adopted by RE entities worldwide, based on equation 2 below. However, as testing data has been obtained over the past 50 years, and as specifications have become stricter in terms of the physical characteristics of select fill, the original values of f* adopted would appear to be on the conservative side (Lawson et al, 2013; Jayawickrama et al, 2015) . The value of f* is related to the characteristics of the soil being used as backfill, but also related to the earth reinforcement being used. The ribbed strips which RE utilises contribute to the increased f* observed in the top 6 m of a typical RE wall. Testing of this specific backfill was not carried out during the design or reconstruction stage. For the Abutment B strip design, the selected value of f* at 2.8 m below the surface level is calculated using equation 3. The f^*_0 values used in 1979 and in 2011 are consistent, while the f* at 2.8 m is based on a Φ' of 36 degrees. This is considered prudent given uncertainty in the choice of f* under the instantaneous loading which would be expected in the event of a sudden braking force on the bridge.

- f^*_{max} at surface = f^*_0 = 2.1 x tan (36 degrees) = 1.5 (Equation 2)
- $f^*_{2.8\ m}$ = f*0 x (6 – 2.8)/6 + tan (36 degrees) x 2.8/6) = 1.14 (Equation 3)

3 EFFECTS OF ABUTMENT BEAM SEAT GEOMETRY AND LOADING

The bridge abutment beam seat was designed by the primary bridge designer, Cardno (NSW) Pty ltd. The 1979 and 2011 bridge loading and geometry is presented below. The significant increase in the vertical dead load is mitigated by the increased footprint of the abutment beam seat, which significantly reduces the pressure on the RE fill underneath to about 125 kPa for dead loading only, and 185 kPa when both dead loads and live loads are combined, considered to be within the capabilities of a typical REW to sustain.

Table 3: 1979 vs 2011 abutment beam seat

Parameter	1979 Value	2011 Value	Unit
Beam Seat Width	2.4	3.2	m
Vertical Dead Load	100	220	kN/m
Abutment Beam Seat Self Weight	120	120	kN/m
Superimposed Dead Load	0	17	kN/m
Vertical Live Load	80	177 (average)	kN/m
Horizontal Dead Load	0	0	kN/m
Horizontal Live Load	22	45 (Abutment B Only)	kN/m
Uniform Distributed Live Load at Road Level	24	22	kPa

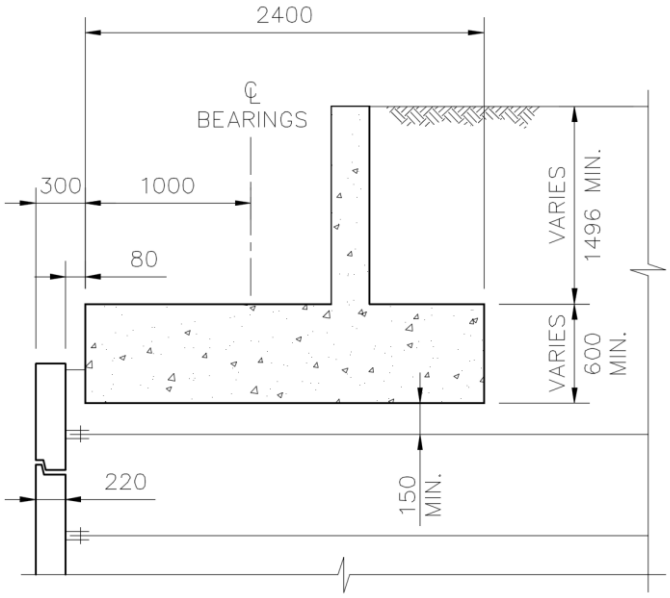

Figure 2a: Diagrammatic representation of 1979 abutment beam seat (not to scale)

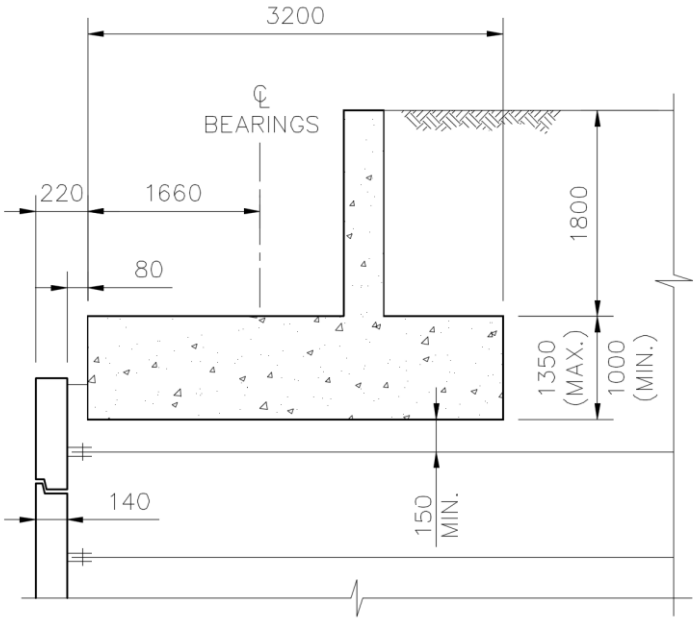

Figure 2b: Diagrammatic representation of 2011 abutment beam seat (not to scale)

4 DURABILITY ASSESSMENT OF EXISTING RE STRIPS

As part of the duplication, an inspection and assessment of the existing buried RE Strips was requested by Blacktown Council. The inspection and assessment were requested by Blacktown City Council with the aim of:

- Establishing the present condition of RE strips.
- Assess whether the existing RE strips remain were corroding in accordance with original design assumptions.
- Assess the anticipated design life of the existing REW sections which are required to be integrated with new sections of REW to support the duplication works.

Steel buried in soil will corrode slowly with time. The REW was originally designed in 1979 assuming a sacrificial allowance in the thickness of the buried RE strips. The design philosophy is to assume that a certain amount of steel corrodes and so this part of the strip section is ignored in the design at the end of the service period.

For the original structure, a total corrosion allowance of 1.50 mm (or 0.75 mm per face) was assumed in the original design. Figure 2 indicates the expected linear rate of loss of thickness over time. This allowance for corrosion is based on predictable and well-defined rates of steel corrosion in soil with chemically controlled parameters. In particular, the pH,

chloride, sulphide, resistivity and sulphate parameters of a soil backfill are controlled where RE Strips and backfill are in contact. Results of testing of three representative samples of existing backfill undertaken in March 1979 indicates the chemical and electrochemical parameters are low and well within the specification requirements of the time. Test results are reproduced below and compared to existing specification limits. The limits below are for REWs built in dry conditions and match the current TfNSW R58 (TfNSW, 2020) specification limits.

Table 4: Soil chemical results from 1979

Sample ID	pH	Chloride (ppm)	Sulphate (ppm)	Resistivity at Saturation ohm.cm
15607	8.3	30	25	1435
15606	7.1	18	31	3500
15605	7.5	17	Not Detected	1795
Current RECO Specification Limits	5 - 10	200	N/A – Sulphate test has been replaced by a Peroxide Sulphur test. Previous limit was 1000 ppm prior to 2014 R57 specification revision.	1000

A total of two (2), full-length, 40 mm wide x 5mm thick RE strips were excavated and removed from REW 4 (North Abutment). Each of the samples were assessed for average section thickness and widths, and thickness of galvanizing coating. The condition of these sampled RE strips was then assumed to be representative of all other RE strips within the structure in order to carry out the design analysis. It is important to note that although the initial RE strip sample size is small, the results obtained were consistent with the excellent chemical results from 1979. Furthermore, validation of the design assumptions was possible during the construction stages which included partial demolition of the REWs, and replacement of 1979 RE strips with new ones. Measurements of the galvanic coating carried out with a galvanising thickness meter confirms the average zinc coating thickness is well above the minimum 85µm thickness specified in 1979, with minimum values of 105 µm and 130 µm observed on the two samples respectively, and average values of 130 µm and 149 µm. The fact that the average measured thicknesses are so much higher than the specified minimum is not unusual based on our experience with hot-dipped galvanised products and suggests no degradation of zinc coating occurred between 1979 and 2011/2012.

5 RESULTS OF ANALYSIS

Four wall sections were analysed, two per abutment. For the Limit State and Working Stress analysis of the internal stability of the REW, any embedment of the REW was neglected. However, given that there is significant stabilising embedment in front of the wall in the form of a rail embankment, this embedment was considered in the Limit Equilibrium analysis. Results from both the Working Stress and Limit State analysis programs confirmed both Abutment A and B structures in their current condition, assumed to have been built in accordance with specifications, did not have adequate structural capacity to support the new proposed bridge duplication loading in 2011. In particular, the top sections of walls were found to have insufficient earth reinforcement attached to the panel to resist failure in rupture. Additional RE Strips were determined to be necessary to support the increased loading, and this additional capacity was required in the top three layers of RE Strips directly underneath the abutment beam seat. Panel modifications in the form of new physical connections and/or casting new panels were designed and detailed to provide the extra capacity.

For Limit Equilibrium analysis, the calculated Factor of Safety with the incorporation of the additional soil reinforcement capacity and new loads is presented below. A typical output of the Limit Equilibrium analysis is shown below, with the critical failure surface have an FoS of 1.54 in this case, with the new loads applied and panel/RE Strip modifications complete. Results from the 1979 design analysis are not available, and the values shown in the right hand column of Table 5 are the assumed minimum values which would have been sought in 1979.

Table 5: Summary for the limit equilibrium stability analysis to determine the factor of safety

Abutment	REW Height (m)	FoS After modifications and applying new loads	FoS Of Existing Bridge without modifications and with original loads
A	11.25	1.54	-
A	11.57	1.53	-
B	11.25	1.56	-
B	11.55	1.61	-
A/B	11.25	-	1.50
A/B	11.795	-	1.50

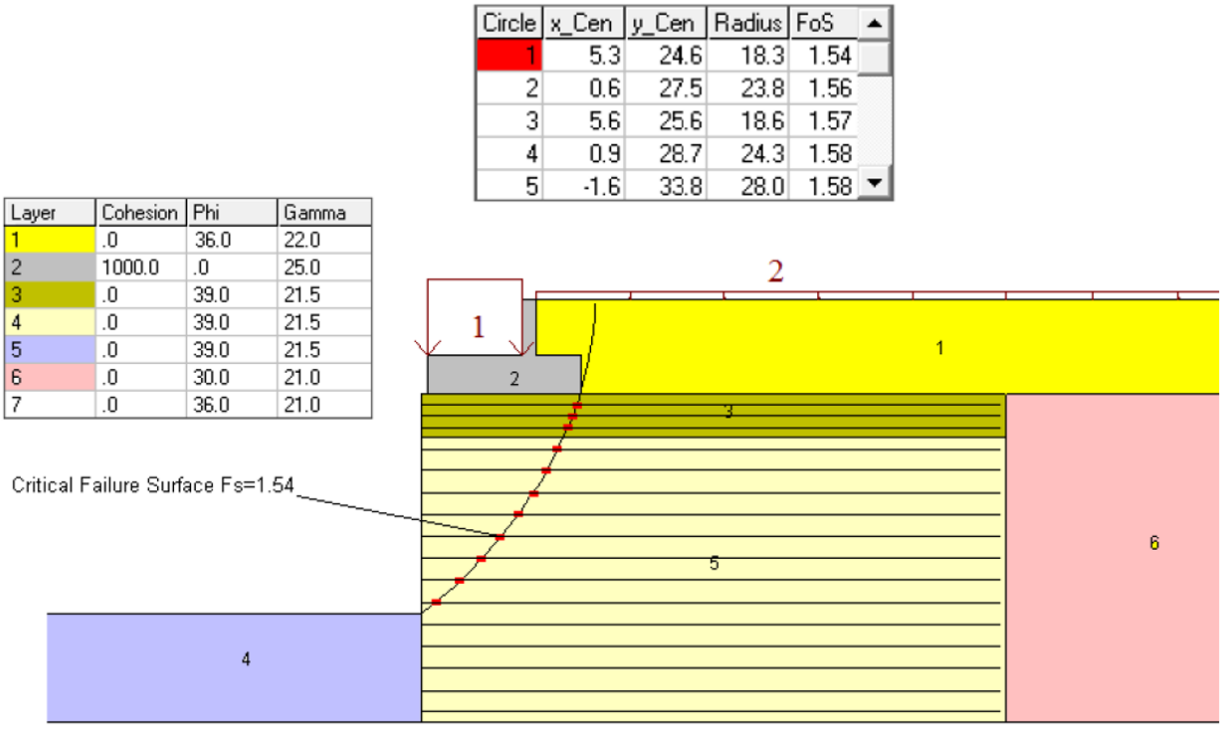

Figure 3 Typical limit equilibrium analysis output

6 CONSTRUCTION WORKS

The construction sequence after the 2011 design of the bridge extension can be briefly depicted as follows.

- Step 1 - Excavation and dismantling of redundant top RE strips from the original provision

  After carrying out the design of the new section of the bridge and RE walls, the majority of the existing panels (including the strips attached to them) that do not comply with the new design have to be removed. Dismantling of facing panels and RE strips is to be carried out in a top-bottom sequence down to the level were the strips are found satisfactory with the new design. Removal of panels and RE strips implies removal of the backfill as well.

- Step 2 - Strengthening on some of the existing panels to remain in the wall (additional RE strips and post-fixed connections to panels to be added). Refer to Figure 9 for post-fixed connection details.

Due to the staggered arrangement of the facing panels, there are panels that cannot be dismantled and will remain in the wall. However, there will not be sufficient cast in tie strip connections for all the additional strips required by the new design. Therefore, new post-fixed connections need to be added to the existing panels to remain in the wall. Both the reaming tie strips and the additional post-fixed ones will provide the facing panels with the required amount of connections for the RE strips numbers required by the new wall design.

- Step 3 - Installation of the new facing panels & RE strips and backfilling to the underside of the new beam seat

  This stage of the re-construction does not differ from a standard RE wall construction sequence. New facing panels are installed and RE connected to them in a sequence of backfill layers from bottom to top (underside of the new Beam Seat).

- Step 4 - Construction of the new bridge beam seat

  Forming of the new bridge beam seat as per new bridge design details. Installation of the new bridge beams and construction of the new bridge deck can be carried out right after the construction of the new beam or at a later stage.
- Step 5- Installation of new panels above the underside of the new beam seat.

  As step 3, this stage of the re-construction does not differ from a standard RE wall construction sequence and so, another sequence of new facing panels are installed and RE strips connected to them completing the backfilling of the wall up to the new road level.

One can refer to Figures 6, 7 and 8 below for a graphic description of the construction steps.

Figure 4: Bridge abutment in 2011 – view 1

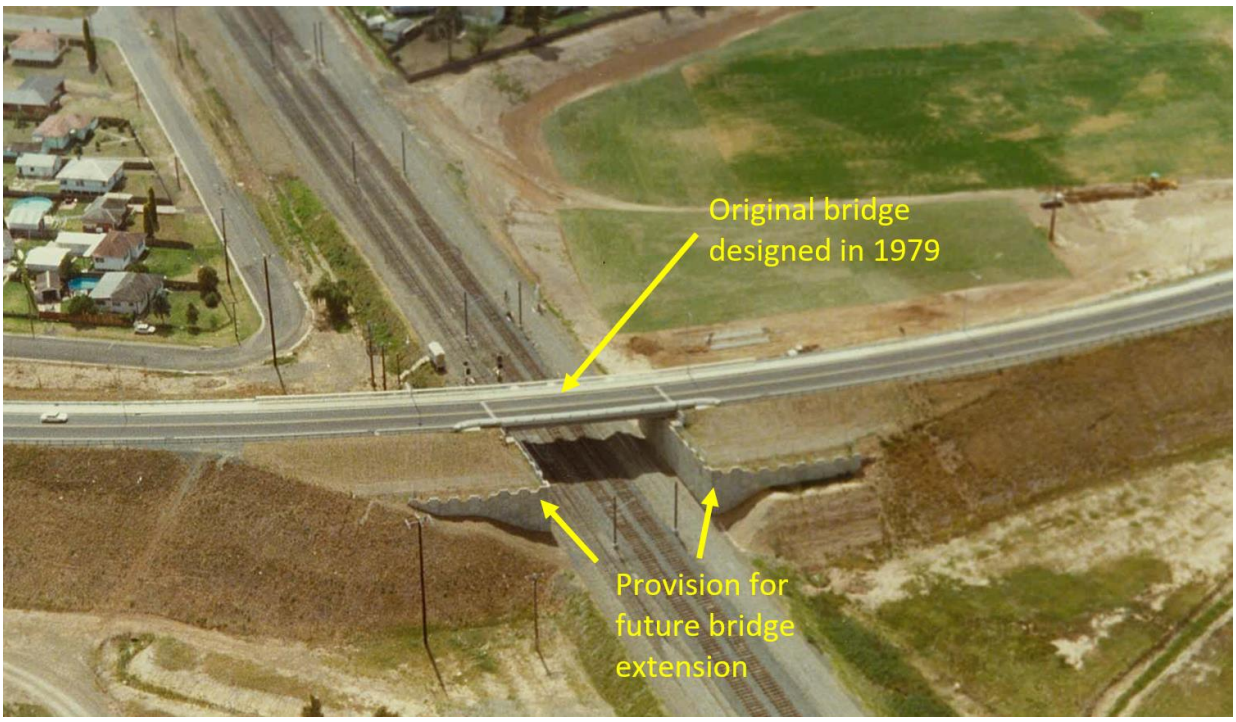

Figure 5: Bridge abutment shortly after completion in 1979/80 – view 2

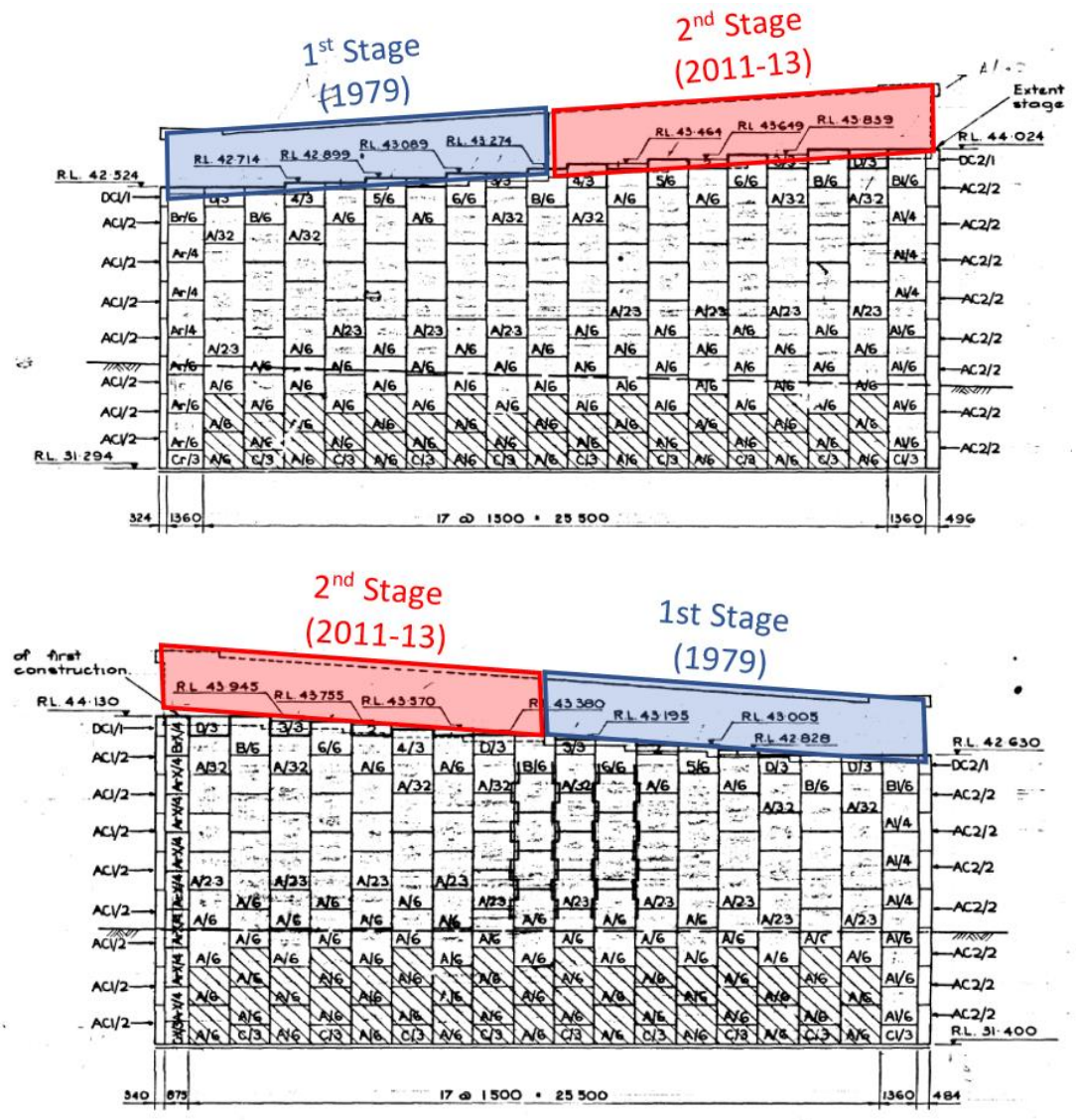

Figure 6: Abutment walls (rear view) - Original 1979 Design drawings

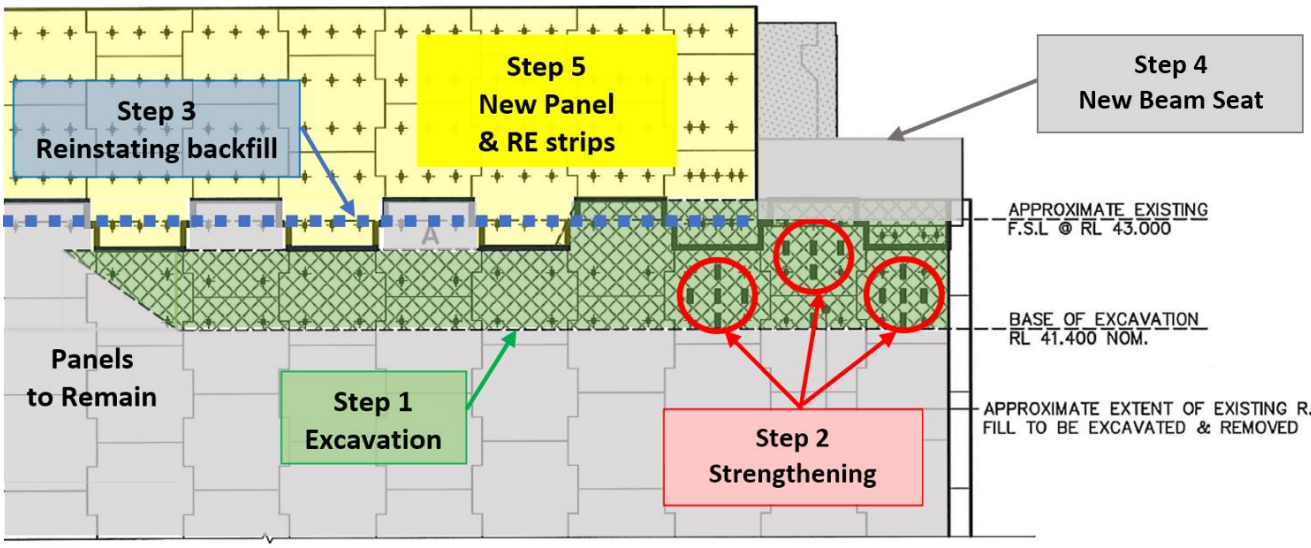

Figure 7: South West Wing-wall (extract from 2011 Design drawing) - Construction Steps

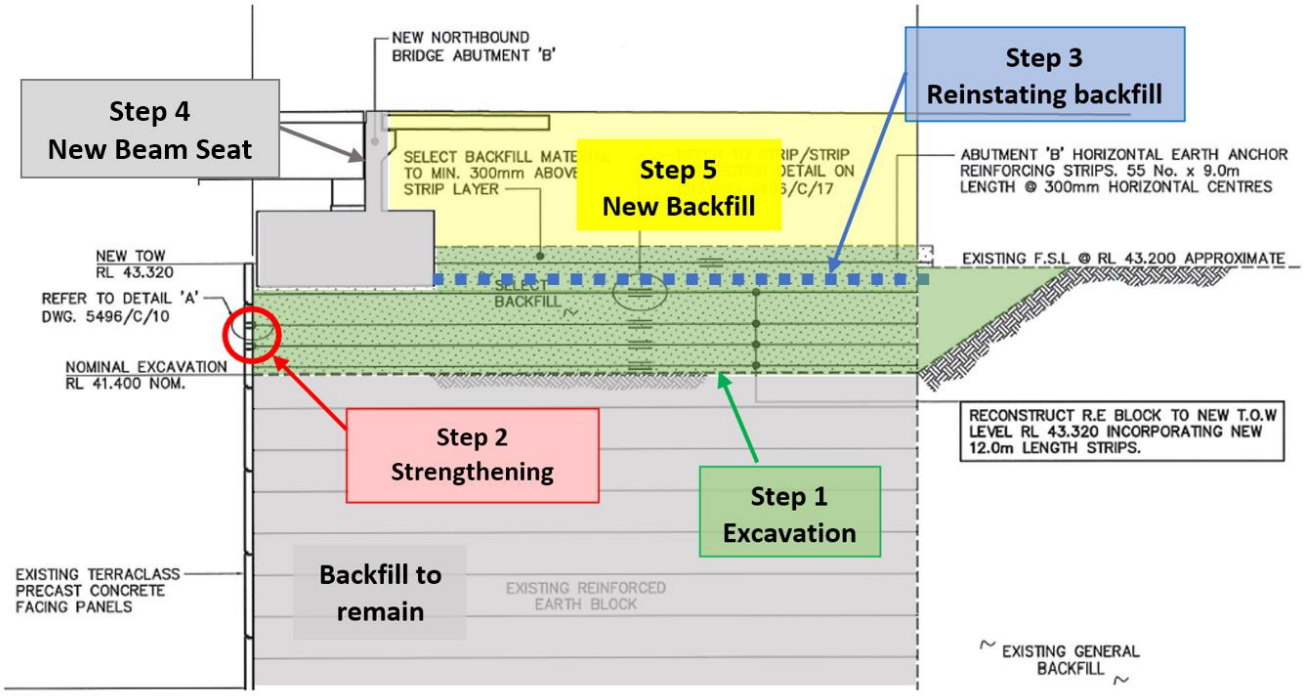

Figure 8: North Abutment (extract from 2011 Design drawing) - Construction Steps

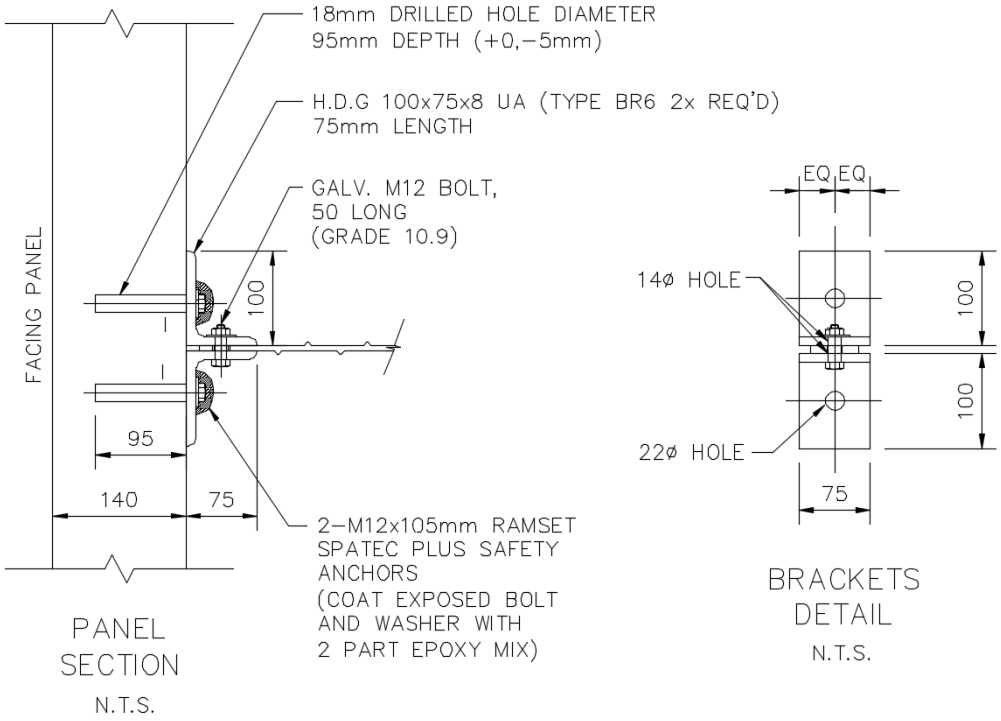

Figure 9: Panel Strengthening Detail – Additional Panel/RE strip connections

Figure 10: New reinforced earth walls and bridge, completed in 2013

7 CONCLUSIONS

The REWs at Knox Road bridge over main Western rail line at Doonside, NSW, were originally designed and built in built in 1979 with provision for future duplication. In 2011, the duplication was formalised but a significantly higher loading and beam seat arrangement was put forward by the bridge designers. RECO completed a detailed design and durability analysis in order to facilitate the duplication. This approach combined Working Stress, Limit State and Limit Equilibrium analyses in order to justify the new arrangement. Further optimisation work involved the utilisation of RE Strips on the new bridge beam seat in order to provide horizontal restraint to the significant bridge loads. The result yielded significant time and cost savings for Blacktown Council, and all work on the REWs was fully completed within the footprint of the walls in a populated urban environment, and outside of the adjacent rail corridor. Further to this, a critical contributor to the success of the project was the high-quality backfill which was used at initial construction. This material not only provided excellent shear strength, but also provided superb resistance to chemical attack. The use of this material allowed for the design life of the structure to be fully reset in terms of corrosion, but also contributed to the relatively minor adjustments required to justify the new design loads.

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APPENDIX A

SMEC TESTING SERVICES Pty Ltd
Phone:(02) 9756 2166 Fax:(02) 9756 1137
Email: smectesting@pacific.net.au
Unit 14/1 Cowpasture Place
Wetherill Park, NSW 2164

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DIRECT SHEAR BOX TEST REPORT

PROJECT: Material Testing
PROJECT No: 18383
REPORT No:11/1240
CLIENT: The Reinforced Earth Pty Ltd
DATE REPORTED:22/10/11
CLIENT ADDRESS:Level 2, 20 Georg Street Hornsby NSW 2077
SHEET 1 OF 2

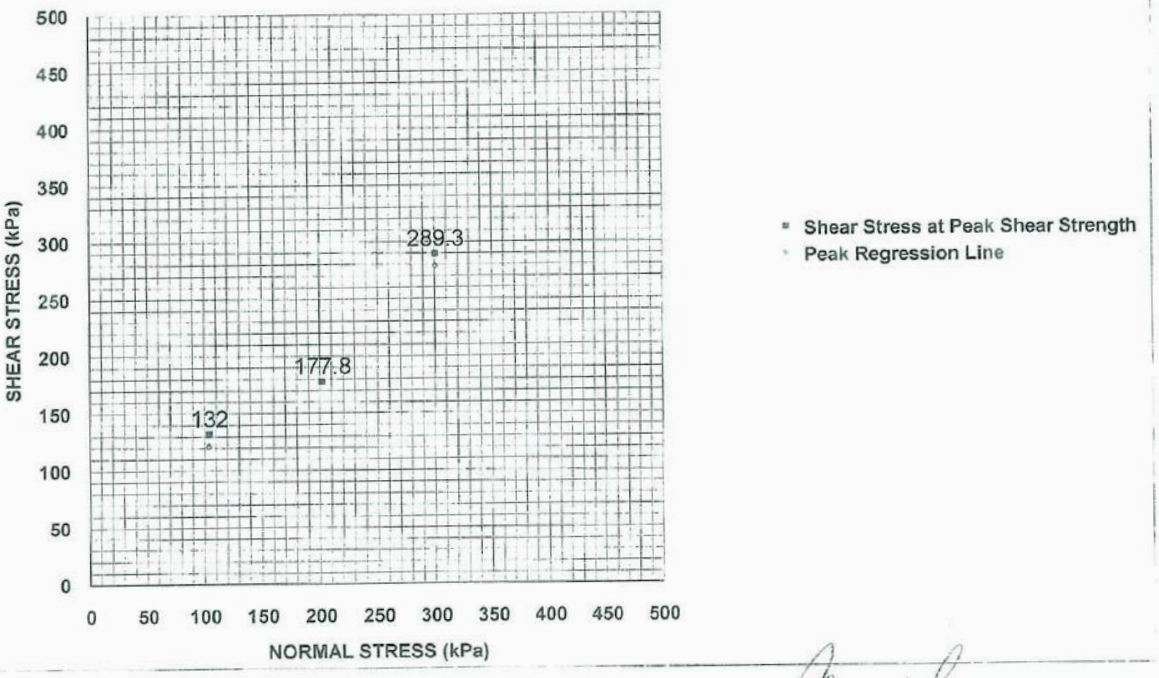
SAMPLING METHOD:Sample Supplied By Client
TEST METHOD:AS1289.6.2.2, 5.1.1, 2.1.1
DATE OF TEST:14/10/2011
TESTED BY:M.B.
STS No:98783
SAMPLE No:2
LOT No:Not Known
REQUEST No:PO12270
MATERIAL DESCRIPTION: Silty Sand, brown
DEPTH:Not Known
LOCATION:Knox Road
TEST TYPE: Peak / Remoulded
STRAIN RATE:0.218mm/min
SHEAR BOX SIZE: 113mm Dia
TYPE OF SPECIMEN: Disturbed
MAXIMUM DRY DENSITY(t/m3): 1.87
OMC (%): 16.7

STAGE No:	STAGE 1	STAGE 2	STAGE 3
NORMAL STRESS(kPa):	104.1	202.0	300.9
INITIAL MOISTURE CONTENT(%)	16.1	16.1	16.1
INITIAL DRY DENSITY(t/m3):	1.88	1.88	1.88
INITIAL DENSITY RATIO(%):	100.0	99.9	99.8
INITIAL HEIGHT(mm):	30.00	30.02	30.03
CONSOLIDATED DRY DENSITY(t/m3):	1.92	1.94	1.92
CONSOLIDATED HEIGHT(mm)	29.41	29.03	29.34
FINAL MOISTURE CONTENT(%)	19.9	19.4	19.1

TARGET COMPACTION (%): 100.0
MAXIMUM PARTICLE SIZE(mm): 4.75
AVERAGE EFFECTIVE ANGLE OF FRICTION(deg.)= 39.0
CORRELATION COEFFICIENT(r^2)=0.95

REMARKS:SHEAR STRESS IS CORRECTED FOR AREA CHANGE AVERAGE EFFECTIVE ANGLE OF FRICTION AND CORRELATION COEFFICIENT CALCULATED BY LINEAR REGRESSION ANALYSIS

UNCORRECTED NORMAL STRESS VERSUS CORRECTED SHEAR STRESS AT PEAK SHEAR STRENGTH



Approved Signatory:
James Hughes - Senior Geotechnician

SMEC TESTING SERVICES Pty Ltd

Unit 14/1 Cowpasture Place

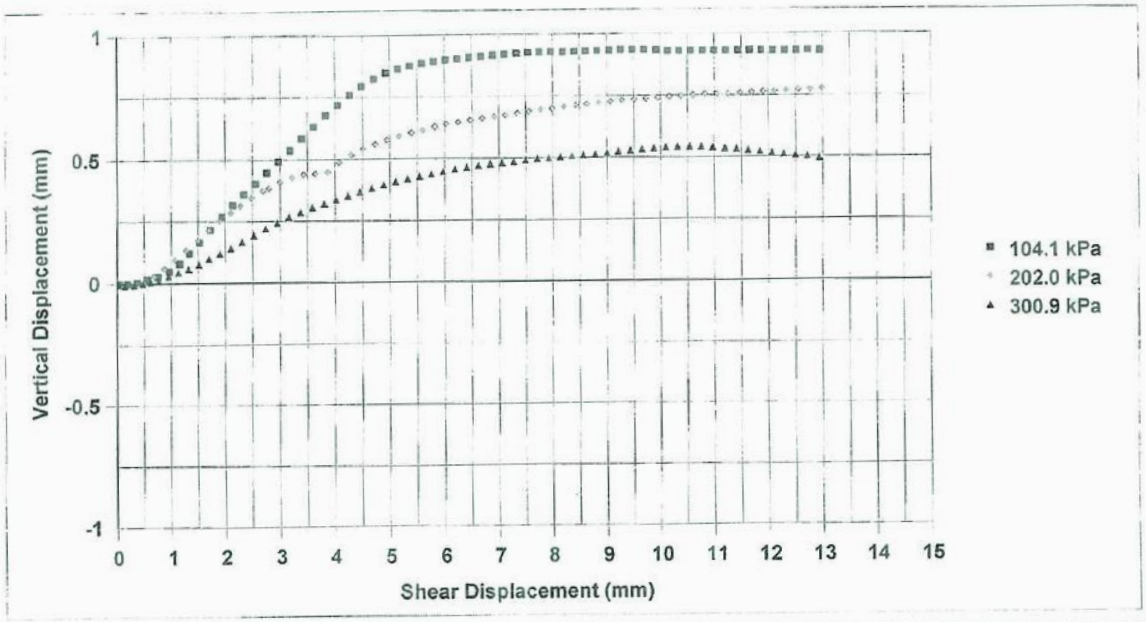
Wetherill Park, NSW 2164

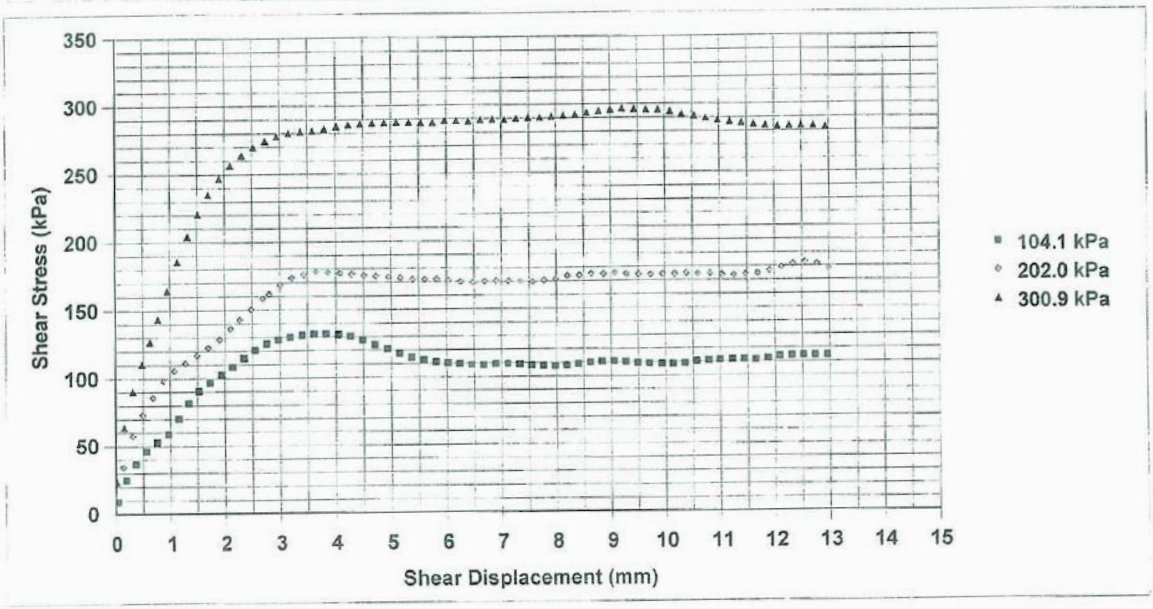
Phone:(02) 9756 2166 Fax:(02) 9756 1137

Email: smectesting@pacific.net.au

DIRECT SHEAR BOX TEST REPORT

PROJECT: Material Testing	PROJECT No: 18383	STS No:9878B
	REPORT No:11/1240	SAMPLE No:2
CLIENT: The Reinforced Earth Pty Ltd	DATE REPORTED:22/10/11	LOT No:Not Known
CLIENT ADDRESS:Level 2, 20 Georg Street Hornsby NSW 2077	SHEET 2 OF 2	REQUEST No:PO12270





USE OF 3D FEM MODEL TO ASSESS THE IMPACT OF DEEP EXCAVATION ON EXISTING SYDNEY WATER TANK STREAM

Peter Sun[1], Firman Siahaan[2], Jack Apostolou[3]

[1]Geotechnical Engineer, Coffey Services Australia Pty Ltd, Level 19, Tower B, 799 Pacific Highway, Chatswood, NSW 2067, Mobile: +61 0438 480 457, email: Peter.Sun@Coffey.com
[2]Senior Geotechnical Engineer, GHD, Level 2/29, Christie Street, St Leonards, NSW 2065, email: Firman.Siahaan@ghd.com
[3]Director, Turner & Townsend Thinc Pty Limited, Level 19, One Wharf Lane, 171 Sussex Street, NSW 2000, email: Jack.Apostolou@turntown.com

ABSTRACT

A residential-commercial development located in Alfred Street, Sydney (Circular Quay) was to be constructed adjacent to the heritage structure Tank Stream, currently owned by Sydney Water Corporation (Sydney Water), with a 6-level basement extending to approximately -17.3 m AHD (Australian Height Datum) resulting in an approximately 21 m deep excavation. The Tank Stream comprised a range of construction including brick oviform section, stone arch and twin concrete pipes employed to replace a damaged section. Due to the proximity of the proposed excavation to the Tank Stream, a geotechnical impact assessment was warranted. As part of the impact assessment work, a three-dimensional finite element numerical modelling (3D FEM) was carried out to assess potential impacts of the basement construction on the various components of the Tank Stream. The retention system was designed to satisfy a reasonably stringent criteria intended to keep oviform and stone arch components in compression. The paper discusses the development of the numerical model, the results of the analysis and the predicted ground movement arising from the construction and the measures taken to monitor and protect the Tank Stream. Inclinometer monitoring results are presented for comparison against the movements predicted from modelling.

Keywords: 3D FEM model, deep excavation, Tank Stream, geotechnical monitoring, ground movement

1 INTRODUCTION

The Tank Stream, currently owned by Sydney Water Corporation (Sydney Water), was the name given to a fresh water course which originally drained a catchment within the modern Sydney CBD, with the primary water course running northwards from the present-day Hyde Park to its termination in Sydney Harbour at the present-day Circular Quay. It was originally established in 1790 as Sydney's original water supply for the early European settlers of Sydney. With time, however, the watercourse became fouled with sewerage and rubbish, essentially converting it to an open sewer (Godden Mackay Logan Heritage Consultants, 2010). The natural channel was progressively enclosed with a stone and/or brick semi-circular drain from the 1840's through to the 1870's. Sections of the drain were redeveloped throughout the 1900's (and in 2000) with concrete/steel piping or boxes.

The Tank Stream was listed on the NSW State Heritage Register in 1999. The heritage protection is associated with the operational portion of the Tank Stream. The Tank Stream curtilage is 3 m from all surfaces of the operational Tank Stream and developments within 10 m of the Tank Stream structure should be approved by a suitable qualified structural engineer (Casey and Lowe Pty Ltd Archaeology and Heritage, 2013). Activities that may impact the operational Tank Stream within a 3 m buffer zone which extends from the outer face of the fabric of the operational Tank Stream, requires approval under the Heritage Act 1977.

The development is located at 1 Alfred Street and bounded by Pitt Street to the east, which has an approximate footprint area of 4,000 m^2 as shown in Figure 1. The length of eastern boundary of the site is approximately 60 m. The Tank Stream approaches the south-eastern corner of the site at a slight angle before it continues, approximately parallel to the eastern boundary. The Tank Stream changes in structural form from brick oviform to upstream stone-arch followed by twin concrete pipes before it changes back to downstream stone-arch near the north-eastern corner of the site. These portions of Tank Stream are connected via rectangular-shaped concrete structures (i.e. transfer chambers).

The development comprises the demolition of the existing buildings and the construction of two towers with a common 6-level basement extending to approximately -17.3 m AHD. Due to the proximity of the proposed excavation to the Tank Stream, a geotechnical impact assessment was warranted. In regards of this, a three-dimensional finite element numerical modelling (3D FEM) was carried out to assess potential impacts of the basement construction on the various components of the Tank Stream.

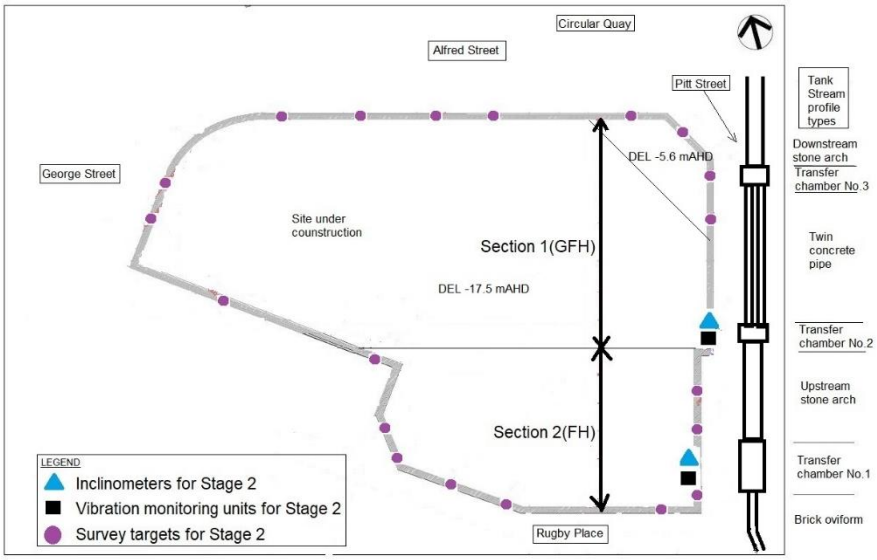


Figure 1: Plan showing the site, monitoring locations and outline of Tank Stream near eastern boundary (data sourced from Pile Design Solutions, 2019)

2 PROPOSED DEVELOPMENT

The existing buildings on site comprised of Gold Fields House (GFH) at 1 Alfred Street with 28 levels and two to three levels of basement, Fairfax House (FH) at 19-31 Pitt Street with 16 levels and one basement level and the Rugby Club (RC) off 31 Pitt Street with 6 levels and no basement. The existing GFH and FH basement perimeter retaining walls were to be left in place and re-used as permanent retention for the new basement construction, where applicable. The existing walls were expected to provide cut-off against the groundwater seepage. Construction of a secant pile wall was proposed to provide groundwater cut-off where the existing wall does not extend to the bedrock.

As shown in Figure 1, the existing basement wall along the eastern site boundary was categorized as Section 1 (GFH) which extends vertically down to about -5.1 m AHD and the outer edge of the wall is approximately 1.5 m away from the outer edge of the nearest portion of the Tank Stream and Section 2 (FH) which extends vertically down to approximately -0.5 m AHD.

The constructed shoring system along the eastern boundary and its main features are as follows:

- Along Section 1, a new concrete wall was cast in front of the existing shoring wall providing support for excavation down to -5.6 m AHD. Vertical props and rock anchors were designed to provide temporary lateral restraint to the system.
- Along Section 2, a new secant piled wall at a short distance in front of the existing basement wall was constructed with 'hard' piles socketed to a minimum depth of 1 m into Class III Sandstone (approximately -5.6 m AHD) or toe level of -6 m AHD, whichever is deeper and the 'soft' piles terminated to a depth of 0.5 m into the top of the bedrock. Horizontal props, rock anchors and walers were designed to provide temporary lateral restraint to the shoring system.
- For both of the above sections the design required rock bolts were to be installed during the basement excavation phase below -5.6 m AHD.

Within the majority of Section 1 and the whole Section 2, the bulk excavation is to be carried out to -17.6 m AHD. In the vicinity of north-eastern corner of Section 1, the bulk excavation is to be undertaken to -5.6 m AHD.

3 DEVELOPMENT OF 3D FEM MODEL

The assessment was undertaken using three-dimensional Finite Element Method (3D-FEM) within a commercially available program PLAXIS 3D. About half of the site along Pitt Street was modelled and the adopted 3D FEM model is shown in Figure 2. Soil layers were modelled using a hardening soil model and rock layers Mohr Coulomb criteria by taking into account appropriate in-situ stresses. The majority of structural elements including Tank Stream have been represented by volumetric elements in the model.

The adjacent nearby development of Lendlease Circular Quay (LL-CQT) is located to the south/southeast of the site and is being constructed by Lendlease Building Pty Ltd (Lendlease). It is understood that LL-CQT development comprises varying basement levels with two basement levels along Pitt Street end of the site, south

of Rugby Place. An interaction between two project sites has been analysed previously and presented in Coffey Services Australia, Pty Ltd (2018a). A simplified adjacent LL-CQT construction has been included in this paper.

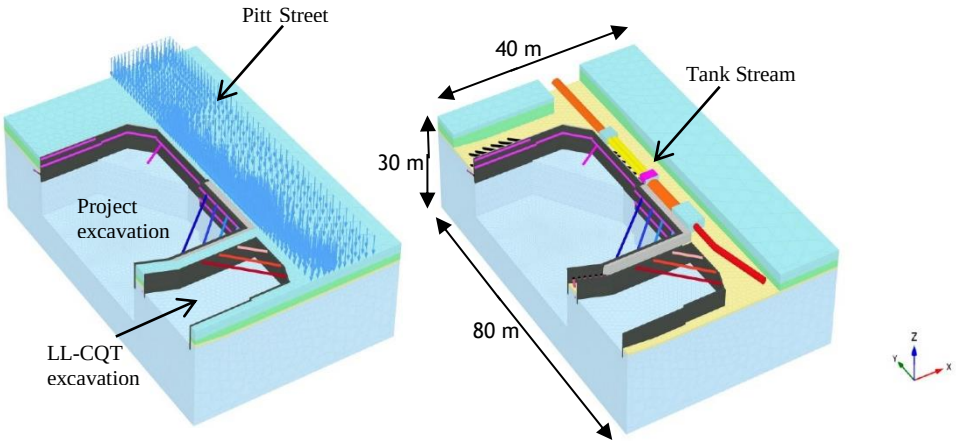


Figure 2: 3D FEM model showing surface view at the end of bulk excavation (left) and with some soil layers hidden to display Tank Stream (right)

3.1 TANK STREAM MODEL

In vicinity of the eastern site boundary the Tank Stream comprises brick oviform, upstream stone arch, twin concrete pipes and downstream stone arch. These components were separated by transfer chambers. Figure 3 presents a rendered image of Tank Stream and the modelled various sections. The dimensions of various components of the Tank Stream and adopted model properties for these components are tabulated in Table 1.

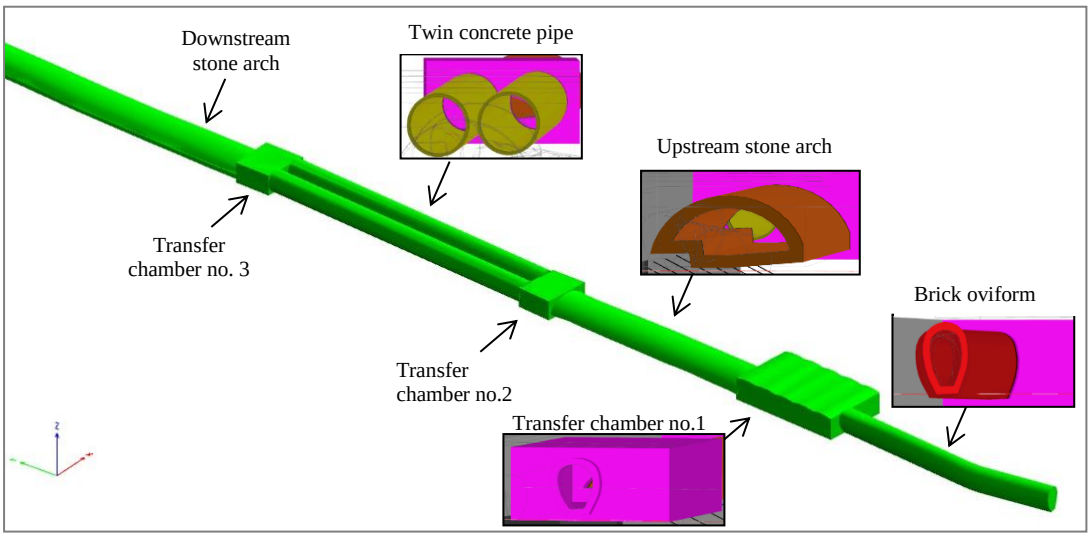


Figure 3: Alignment of rendered Tank Stream and modelled components

Table 1: Adopted dimensions of Tank Stream components and model properties (Coffey Services Australia Pty Ltd., 2018b)

Components	Length (m)	Internal height/width (m)	Thickness (m)	Unit weight (kN/m^3)	Young's modulus (GPa)	Poisson's ratio (v)
Brick oviform	4.0	1.68/1.14	0.23	24	4.9	0.15
Transfer chamber no.1	7.5	2.00/4.00	0.30	24	21.9	0.15
Up/Downstream stone arch	15.0	1.32/3.08	0.37	24	10.1	0.15
Transfer chamber no.2	2.5	2.00/3.00	0.30	24	21.9	0.15
Twin concrete pipes	20.5	1.37	0.08	24	21.9	0.15
Transfer chamber no.3	2.5	2.00/2.80	0.30	24	21.9	0.15

3.2 GEOTECHNICAL MODEL

The subsurface profile adopted in this assessment was based on the geotechnical investigation report for the proposed site (Coffey Services Australia, Pty Ltd, 2015) and Coffey's experience in the neighbouring sites in Circular Quay. The adopted subsurface foundation profile and design parameters are listed in Table 2.

Table 2: Subsurface foundation profile and design parameters (Coffey Services Australia, Pty Ltd, 2015)

Units and descriptions	RL at top of unit (m AHD)	Weight (kN/m^3)	Strength Parameters		Stiffness Parameters			At-rest earth pressure coefficient K_0*
			Effective cohesion c' (kPa)	Effective friction angle (°)	Elastic modulus E_v (MPa)	Un/reloading modulus E_{ur} (MPa)	Poisson's ratio, ν	
2-Alluvium	-0.7	18	0	28	8	24	0.3	0.53
3a-Sandstone**	-2.1 to -3.1	22	30	35	300	N/A	0.25	
3b-Sandstone***	-2.8 to 3.8	23	1000	40	2000	N/A	0.25	

Note: * In-situ stresses for Units 3a and 3b were assessed as part of the initialisation phase of 3D FEM to obtain the stress regime described in Pells (2002); ** Extremely to moderately weathered sandstone; *** Slightly weathered to fresh sandstone

The original groundwater level at -0.5 m AHD has been adopted in the model. The general groundwater flow was assessed to generally move inland within the Circular Quay area. This was possibly influenced by the recently completed and ongoing developments within the area where temporary dewatering and permanent water take have been undertaken.

3.3 LOADING AND BOUNDARY CONDITIONS

Surcharge load of 20 kPa has been applied starting from 0.5 m offset from the outer edge of the existing wall. This load represents traffic load on the footpath and adjacent Pitt Street, which runs parallel to the eastern boundary of the site. It was assumed that this load has been in place for a considerable amount of time in the past.
Four displacement boundaries were applied on the rock layers to compress the rock body during initialisation of stresses. The top and bottom boundaries were set to roller boundaries. Displacement boundaries were applied for major (north-south) and minor (east-west) directions followed by removal of top boundary and bottom boundary being changed to "fixed" boundary.

3.4 ANALYSIS METHODOLOGY

In order to incorporate the in-situ stresses within Hawkesbury Sandstone, proper at-rest earth pressure coefficient K_0 values were adopted using equations (1-3) by Pells (2002):

$$\sigma_1 = \sigma_{(N\text{-}S)} = 1.5 + 1.2\sigma_v \text{ to } 2.0\sigma_v \text{ MPa} \quad (1)$$

$$\sigma_2 = \sigma_{(E\text{-}W)} = 0.5\,\sigma_1 \text{ to } 0.7\sigma_1 \text{ MPa} \quad (2)$$

$$\sigma_3 = \sigma_v = 0.024H \text{ MPa} \quad (3)$$

where, σ_v is the vertical stress, $\sigma_{(N\text{-}S)}$ the horizontal stress oriented approximately 20° East of the true north, $\sigma_{(E\text{-}W)}$ the orthogonal to $\sigma_{(N\text{-}S)}$.
The adopted construction sequencing in the 3D FEM model is presented in Table 3.

Table 3: Adopted numerical simulation construction phases in 3D FEM model

Phase	Description	Construction stage
1	Initialisation of stresses	Existing condition
2	Application of in-situ stresses for rock layers (Units 3a and 3b)	
3	To reset displacement to zero and active basements for Section 1 (GFH) and Section 2(FH)	
4	To reset displacement to zero and backfill the existing basement within Section 2 and remove ground floor slab	Stage 1
5	Install and prestress upper level of anchors within Section 1	
6	• Install diagonal bracings within LL-CQT site • Excavate to bulk excavation level within LL-CQT site	
7	• Install new reinforced concrete wall within Section 1 • Install shoring system (secant piled wall) within Section 2	
8	Excavate to 0.5 m below the diagonal bracings level within Section 2. The backfill materials located in between the existing wall and new wall was left in place	Stage 2
9	Install diagonal bracings within Section 2 and walers	
10	• Excavate to the lower level of anchors within Sections 1 and 2 • Remove basement slab within Section 2	
11	Install and prestress lower level and anchors	
12	• Demolishing remaining slabs • Demolish remaining separation wall between Section 1 and Section 2 • Excavate to -5.6 m AHD	
13	• Install and prestress rock bolts • Install raker prop	
14	Excavate to -11.5 m AHD	
15	Excavate to -17.6 m AHD (bulk excavation level)	

At any given phase where excavation was carried out below the groundwater level, the dewatering is carried out within the excavation up to 0.5 m deep below the achieved base of excavation.

3.5 ADOPTED ASSESSMENT CRITERIA FOR VARIOUS TANK STREAM COMPONENTS

There was no specific assessment criterion available for the Tank Stream stone arch, twin concrete pipes and transfer chambers. Based on indicative criteria provided by Sydney Water for the brick oviform and literature resources (Burland and Wroth 1974; Boscardin and Cording, 1989), the following impact assessment criterion for various components of Tank Stream at construction Stages 1 and 2 were adopted (Table 4).

Table 4: Summary of adopted criteria for impact assessment

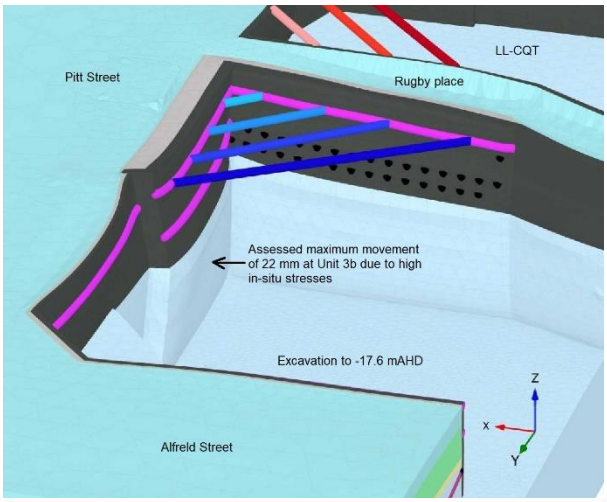
Components	Criterion
Brick oviform	• Net tensile strain of zero within oviform structure at the end of construction • Max. incremental tensile strain of 0.01 % due to transverse displacement along the longitudinal alignment • Max. incremental tensile strain of 0.01 % due to vertical displacement along the longitudinal alignment
Stone arch and twin concrete pipes	• Ratio of horizontal strain to angular distortion of less than 0.44 • Longitudinal tensile strain less than 0.05% • Transverse tensile strain less than 0.0125% • Deflection ratio (sagging ratio) less than approximately 1 in 600
Transfer chambers	• Tensile strain less than 0.05%

4 ANALYSIS RESULTS AND DISCUSSION

4.1 EXCAVATION-INDUCED GROUND MOVEMENTS

Figure 4(a) presents the deformation of the ground for Section 2 where the proposed excavation level is deeper at -17.6 m AHD and the critical portions of the Tank Stream (i.e. brick oviform and upstream stone arch) are located in a close proximity. The deformed shape has been magnified to visually signify the deferential movement and identify the point of maximum lateral movement. The assessed maximum movement occurs within Unit3b (Class III or better Sandstone). These results were likely influenced by the relaxation of high magnitude of in-situ stresses which are typically present in Sydney basin. Figure 4(b) presents the assessed profiles of horizontal movements at various excavation stages within Section 2. The assessed horizontal movements during the excavation are predominantly influenced by the relief of rock in-situ stresses within Units 3a and 3b, the horizontal movements increase as the excavation depth increase. The assessed ground movements generally reduce towards the ground surface where the retention system (i.e. secant piled wall, new reinforced concrete wall and existing basement walls) along with the support features (anchors, rock bolts, diagonal bracing waler and prop) are installed. Trigger levels as 80% of the assessed horizontal movements were proposed for Section 2, which will allow a progressive monitoring and earlier identification and mitigation of any exceedance of assessed movement during the excavation.

(a)



(b)

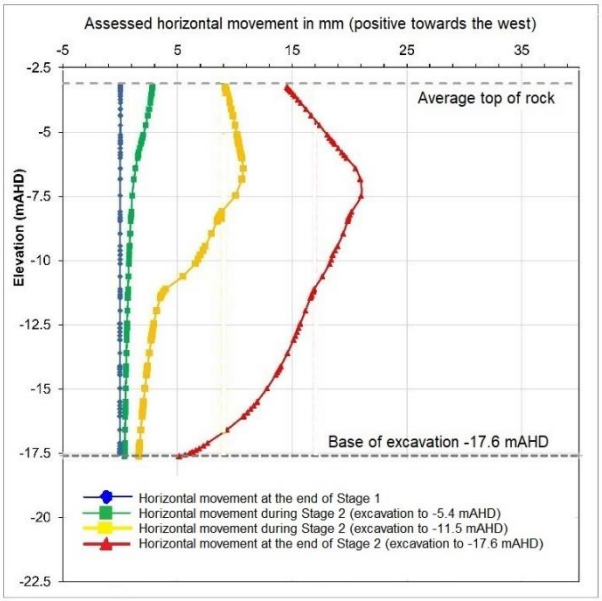


Figure 4: (a) Deformed shape of 3D FEM model (150 times exaggeration) induced by full excavation up to -17.6 m AHD (end of Stage 2) and (b) Assessed profiles of horizontal movements at various excavation stage within Section 2

4.2 IMPACT TO TANK STREAM DURING CONSTRUCTION

Analyses results for the construction stages (Stages 1 and 2) of the project are presented in Table 5 for the critical cross sections of the Tank Stream components. Results for the longitudinal sections are given in Table 6. Maximum net strain values were assessed by deducting the assessed strains for the construction stages from the strains assessed for the existing conditions. The existing condition considers the existing GFH and FH basements, Tank Stream and applied 20 kPa surcharge starting from 0.5 m offset from the outer edge of the existing wall, which was assumed to be in place for a considerable amount of time in the past.

Table 5: Analyses results for Tank Stream cross sections from the start of Stage 1 to the end of Stage 2

Component	Maximum differential deformation/shortest cord length	Maximum tensile strain (%)	Maximum net strain (%)
Brick oviform	<2 mm / 1 m	0.005	< -0.01
Transfer chamber no.1	<2 mm / 1 m	0.02	-0.03
Upstream stone arch	<2 mm / 1 m	0.005	<-0.01
Transfer chamber no.2	<2 mm / 1 m	0.04	-0.01
Twin concrete pipes	<2 mm / 1 m	0.02	0.003

Note: compression is negative and tension is positive

Table 6: Analyses results for Tank Stream longitudinal sections from the start of Stage 1 to the end of Stage 2

Component	Maximum differential horizontal movement/cord length	Maximum tensile strain (%)	Maximum net strain (%)	Horizontal strain/angular distortion
Brick oviform	1 mm/1.5 m	0.004	<-0.01	<0.44
Upstream stone arch	1.5 mm/2 m	0	<-0.025	<0.44
Twin concrete pipes	2.5mm/2.5 m	0.008	<-0.005	<0.44

The analyses show that the strain levels within various components of Tank Stream at the end of the construction works related to Stage 1 and Stage 2 would still likely be within compression state except for the twin concrete pipes. The response of all Tank Stream components were assessed to be within the proposed criteria outlined in Table 4 above.

5 MONITORING RESULTS

The ground movement monitoring was undertaken within the existing basement structures located near the Tank Stream during Stage 1 and Stage 2 utilising Inclinometers (INC 1 and INC 2). The locations of the inclinometers are shown in Figure 1.

The inclinometer readings undertaken up to 7 August 2020 when excavation had reached -17.4 m AHD at Section 1 and -3.5 m AHD at Section 2 indicate that the horizontal movement of the basement wall has occurred away from the tank stream by up to 9 mm which is within the proposed trigger levels as shown in Figure 5. The modelled results are larger than the indicated in the measurement this would likely be due to some relaxation in horizontal stress arising from earlier development at the site, which was not considered in the model.

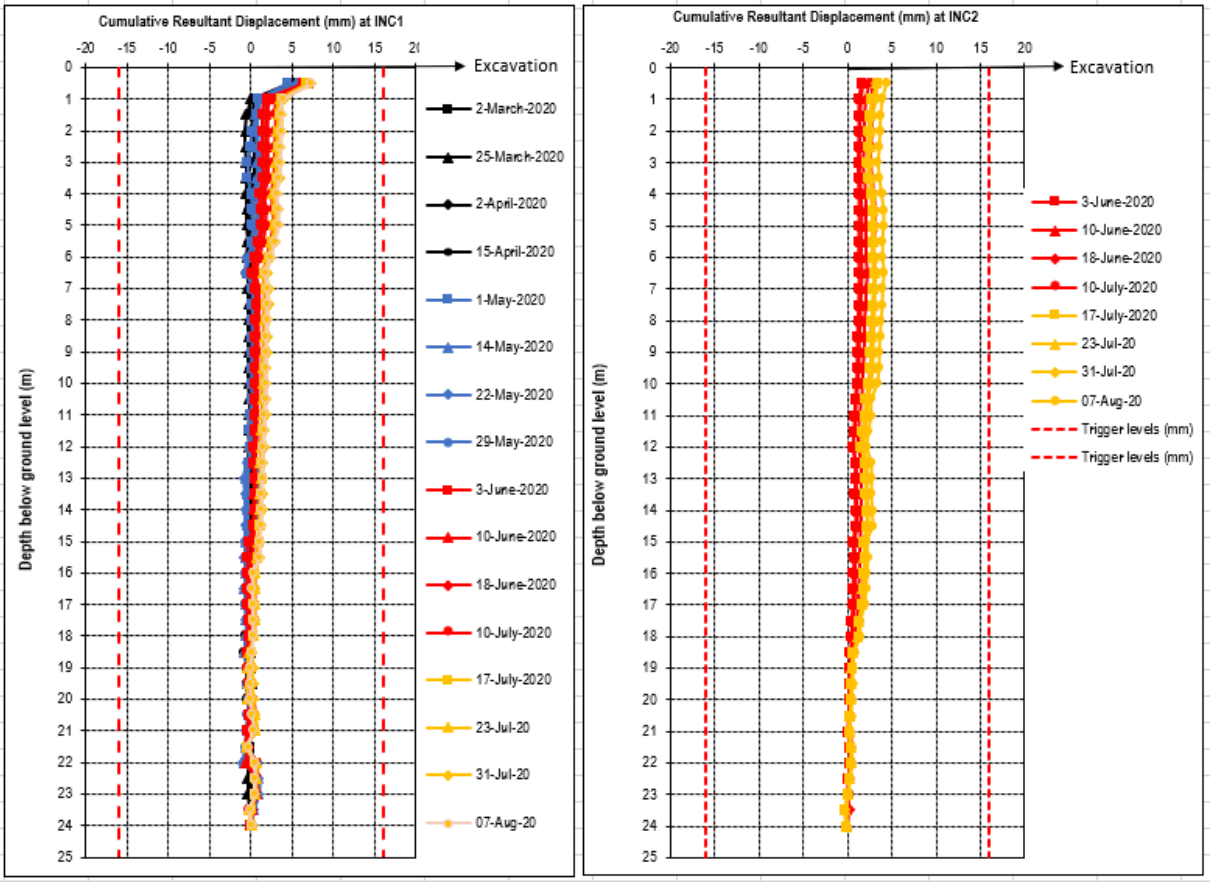


Figure 5: Cumulative resultant displacement (mm) from INC 1 and INC 2.

6 CONCLUSIONS

A 3D FEM model was established to predict the ground movement and investigate the impact of construction on the Tank Stream.

- Based on the indictive criteria by Sydney Water, assessment criteria including no net tensile strain in brick oviform was proposed. The numerical analysis indicated all Tank Stream components were assessed to be within the proposed criteria.
- The assessed maximum movement during excavation was 22 mm at sandstone layer, which is predominantly influenced by the relief of rock in-situ stresses. The horizontal movements increase as the excavation depth increase. In addition, movement trigger levels as 80% of the assessed horizontal movements were proposed for Section 2 (Fairfax House) allowing a progressive monitoring and early identification and mitigation in the event that observed movement exceeded expectations during the excavation.
- Inclinometer monitoring undertaken up to 17 July 2020 indicates favorable outcome where the basement walls moved towards the excavation by up to 6.5 mm which is within the proposed trigger levels.

7 ACKNOWLEDGEMENTS

The authors are grateful for the permission from AW Holding group for publishing the project data. The authors would like to express their gratitude to Ross Best and James Searle from Coffey Services Australia, Pty Ltd for their assistance during the preparation of this paper. The assistance received from Terry Tarboton and Jayakanthan Velautham during the construction of the numerical model is greatly appreciated. It is also acknowledged the Inclinometer monitoring was undertaken by Timothy Masudi and Farid Sabir from Coffey Services Australia, Pty Ltd.

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SHORT AND CUMULATIVE LONG TERM IMPACTS OF SUBTERRANEAN CUT-OFF WALLS AND EXCAVATION DEWATERING ON ADJACENT STRUCTURES IN URBANISED AREAS

Bosco Poon[1], Kim Chan[2], Rikito Gresswell[3] and Firman Siahaan[4]
[1]Technical Director, [2]Senior Technical Director, [3]Senior Hydrogeologist, [4]Senior Geotechnical Engineer. GHD Pty Ltd

ABSTRACT

This paper describes the hydrogeological and geotechnical assessment in relation to the short and cumulative long-term impacts associated with excavation, subterranean construction and dewatering in the Double Bay commercial area on the structural integrity of adjacent residential and commercial buildings. The paper comprises three main components, namely, (i) definition of groundwater study area and its geological and groundwater settings; (ii) assessment of potential cumulative impact of future developments on long-term groundwater change; and (iii) assessment of potential impact of short-term construction dewatering on risks of damage of adjacent buildings.

1 INTRODUCTION

Urban development is increasingly aiming to maximise the value of land in the Double Bay region. Double Bay is a harbour side eastern suburb of Sydney, in the state of New South Wales. Many developments are considering the construction of basements, underground car parking and other associated below ground structures. Where the water table is intersected, temporary dewatering is required to ensure safe and stable construction conditions. Partial or full depth penetration cut-off walls are sometimes adopted for the purpose of dewatering the sites. The potential for these cut-off walls to cause disruption to the groundwater flow, either by allowing the flow of water beneath the wall during dewatering within the sites or by altering the long-term groundwater seepage patterns requires careful consideration. The magnitude of these groundwater level impacts can be significant when the developments are considered from a cumulative perspective. In terms of the built environment, the depressurisation of compressible sediments can lead to differential settlement, which have significant impacts on existing buildings. This paper outlines the geotechnical and hydrogeological assessment in relation to: (i) the impact of long term regional groundwater level change due to future developments and (ii) the impact of surface settlement as a result of groundwater drawdown caused by short term construction dewatering.

2 REGIONAL SETTING

Double Bay sits in the valley between the ridgelines of Edgecliff/Darling Point and Bellevue Hill/Point Piper, occupying the low elevation harbour front area. The valley follows the former Cooper Creek alignment, which emanates from Cooper Park, running from Bellevue Hill, north to the harbour (Figure 1). This watercourse, and its entry into the harbour, has resulted in variably deep alluvial sediments within the valley base, with the greatest depth of soils close to the bay, where boreholes have encountered greater than 40 m of mainly coarse grained sediments, occasionally peaty sands with stiff clay basal layers. Climate data from the Bureau of Meteorology indicates an average annual rainfall of around 1230 mm.

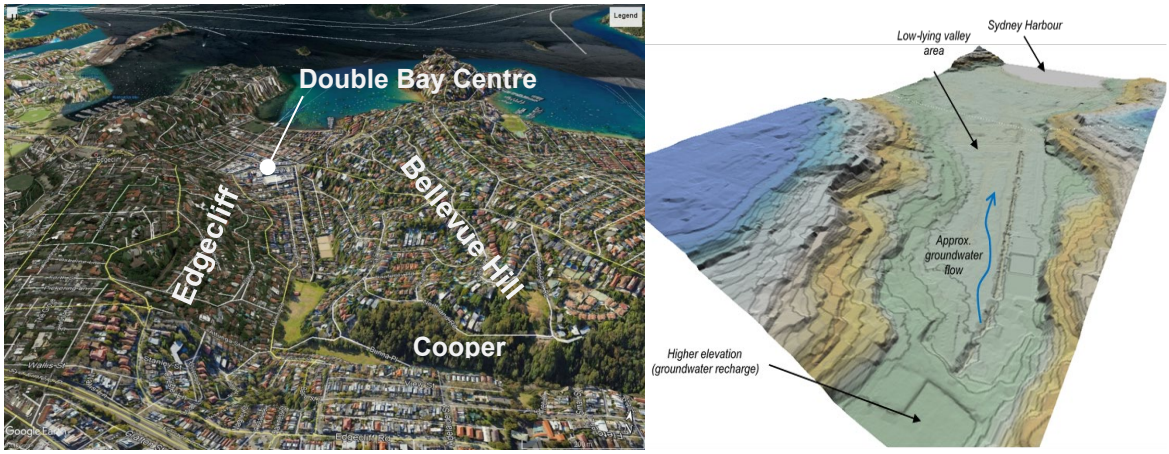


Figure 1: Double Bay regional setting

3 GEOLOGICAL SETTING

Over two hundred available boreholes in the Double Bay area dating back to the 70's have been compiled to create a 3D geological model using the commercially available software package Leapfrog. Figure 3 shows the spatial distribution of the available geotechnical data points. Figure 2 shows the 3D visualisation of the ground conditions. It can be seen that Double Bay was originally a sandstone valley that has now been buried with sand dominated alluvium.

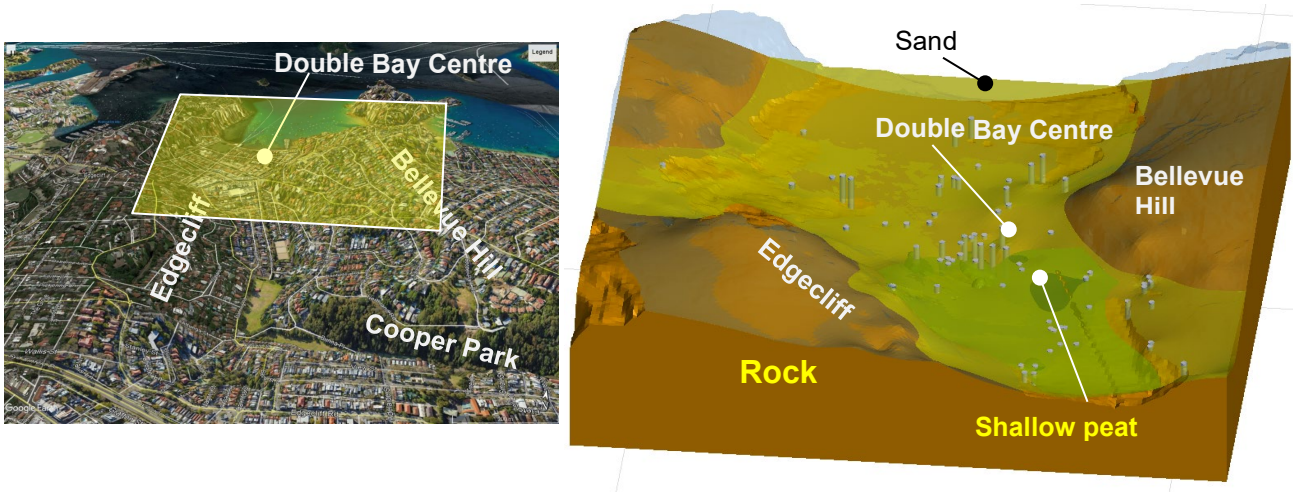


Figure 2: 3D geological model generated using Leapfrog software program

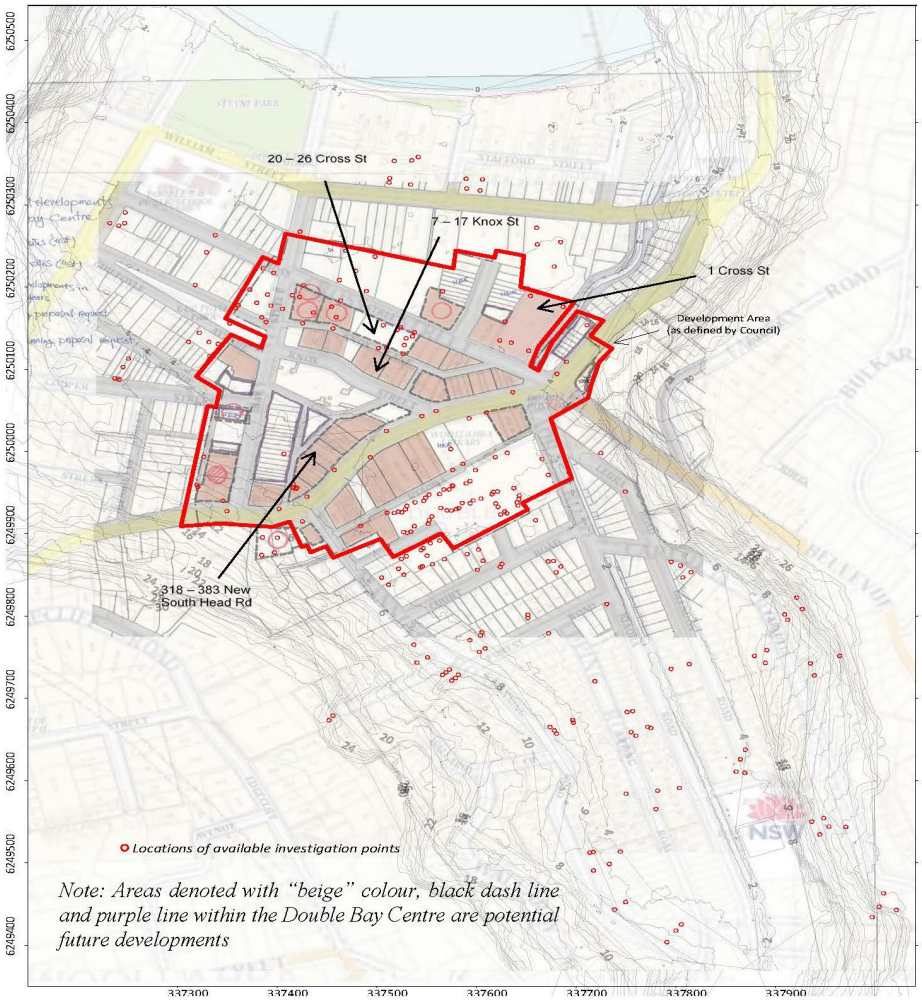


Figure 3: Locations of all available geotechnical investigations in Double Bay

The subsurface profile encountered in the Double Bay area and delineated in our geological model can be broadly categorised into three main strata as follows:

- *Bedrock* – Hawkesbury Sandstone underlies the Quaternary deposits. Hawkesbury Sandstone generally comprises medium to coarse grained quartz sandstone with minor shale and laminate lenses. It is typically extremely to highly weathered and fractured at the top and becomes moderately to slightly weathered and only slightly fractured with depth. Collation of available data suggests that the weathered sandstone bedrock surface follows the general shape of ground surface. An assessed contour of bedrock level is presented in Figure 4a.
- *Alluvium* – The alluvial region generally follows the shape of the valley. The yellow highlight shown in Figure 4a outlines the approximate extent of the alluvium over the sandstone bedrock. A geological section drawn along section A-A was developed in the north-south direction roughly parallel to the direction of groundwater flow. This geological section, shown in Figure 5, indicates that the sand dominated alluvium generally fill the incised valley and extend to a maximum depth of about 35 m as the bed rock dips north towards Sydney Harbour. The alluvial sand overlying the bedrock is generally clean and medium to fine grained. It varies in consistency from loose at shallow depth to very dense at depth. Interlayered sandy clays, clays and lower peats of typically stiff to very stiff consistency are also encountered. It appears that these bands are found at lower depths and encountered mainly at the southern Double Bay study area to the south of Kiaora Lane.
- *Upper Peat* – The upper peat layers were considered to be the most compressible deposits and are generally encountered at shallow depth of 0.5 – 2.5 m. Previous investigations indicated that the dark grey peat lenses are of high plasticity with high moisture content organic clay materials. The presence of peat has been observed intermittently although it was consistently noted in the area located to the south of Forest Rd (see Figures 4a and 4b). The upper peat layer have thin thicknesses of generally less than 1.5 m, but they are considered to have significant influence on dewatering induced settlements and further discussion of this material is given in Section 6.

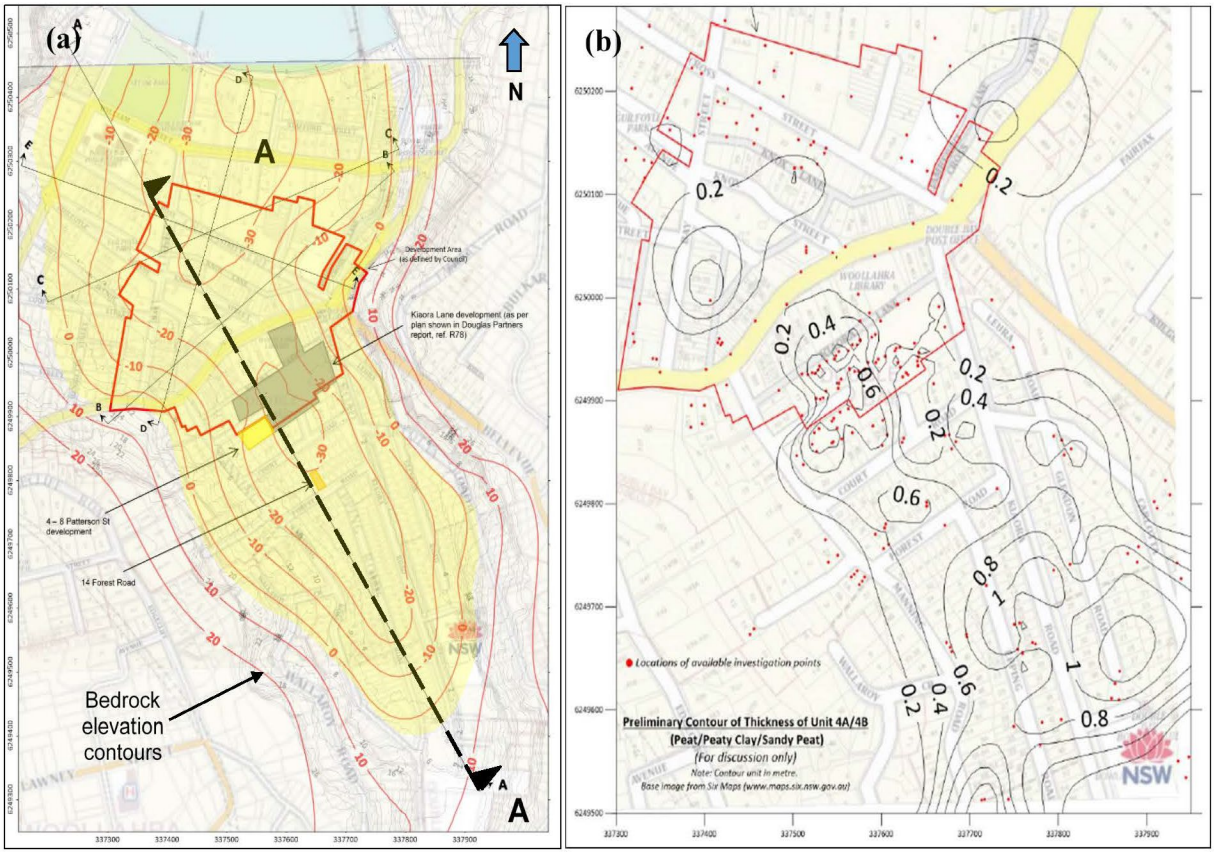


Figure 4: (a) Contour of top elevation of bedrock; (b) Contour of upper peat thickness

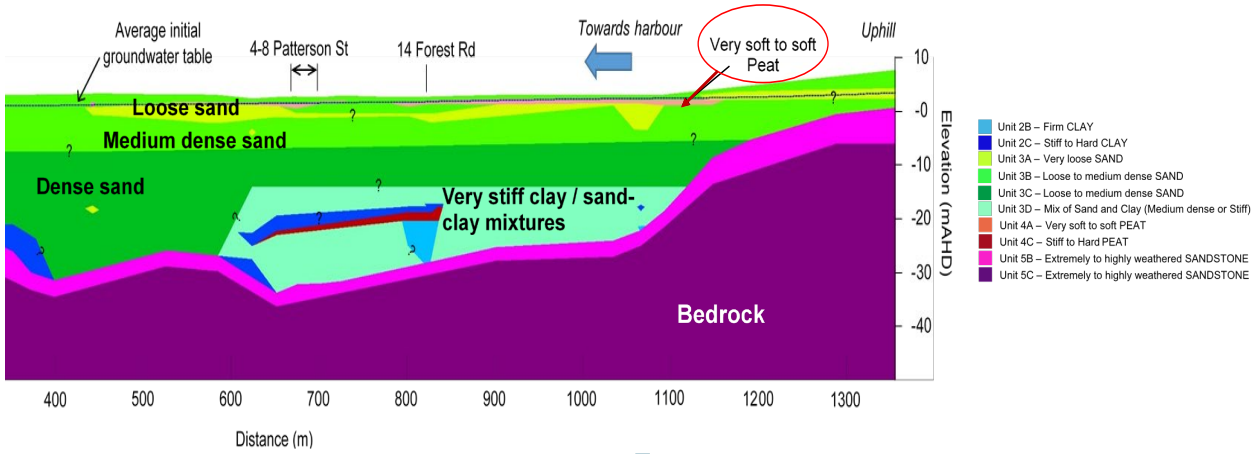


Figure 5: Geotechnical profile along Section A-A in Figure 4a

4 GROUNDWATER SETTING

4.1 OVERVIEW

Hydro-stratigraphic units (HSUs) are zones within the groundwater system that have similar hydrogeological properties and behave in a similar manner from the point of view of groundwater flow. For the Double Bay area, the hydro-stratigraphy is broadly divided into the Alluvium, comprising unconsolidated sediments, and the Bedrock, which underlies the Alluvium and forms a fractured rock aquifer. The Alluvium forms an unconfined aquifer, within which the water table (upper surface of the shallow groundwater system) is located. The Bedrock is confined beneath the Alluvium within the Double Bay area, and becomes unconfined where it outcrops outside of the valley and forms a regional aquifer.

Groundwater use within the study area is based on the data extracted from the Bureau of Meteorology(BoM)'s Australian Groundwater Explorer, which identified 40 bores in the approximate Double Bay area. The uses of these bores were identified as following (although the operational status of these groundwater bores is not known): (i) 28 bores for water supply; (ii) 6 bores for monitoring; (iii) 1 bore for irrigation; (iv) 1 bore for dewatering; and (v) 4 other bores with unknown purposes. The depths of these bores range from 2.75 m to 52 m, with an average depth of 9.6 m.

A search of Water-NSW also identified 48 bores in the approximate Double Bay area, however, this dataset did not identify the use of each bore. The data obtained from the review of existing geotechnical and hydrogeological investigation reports indicate several bores constructed within the Double Bay area specifically for monitoring purposes. The data available from these investigation reports as well as those from the Water-NSW database and BoM database provide the basis for interpreting the groundwater flow directions and trends in the following section.

4.2 GROUNDWATER FLOW DIRECTION

To assess groundwater flow directions in the Double Bay area, contours of water table have been prepared using groundwater level data extracted from the existing geotechnical and hydrogeological investigation reports (Figure 6). The contours are interpreted from groundwater levels taken at different points in time, many of which are opportunistic measurements collected from open-holes at the time of field investigations. As such, there are some local variability and the contours should be considered indicative only. Despite these limitations, the interpreted contours provide useful indications of groundwater flow directions, confirming the northerly groundwater flow towards the coastal boundary along the centreline of the valley and flow from topographically elevated areas along the valley edges towards the valley centre. The hydraulic gradient is around 0.003 along the valley centreline, indicating a gentle hydraulic gradient across the Alluvium comprising permeable sediments. The data currently available is insufficient to ascertain local variability in the water table due to anthropogenic influences such as groundwater pumping and existing basement structures.

4.3 GROUNDWATER TREND

The available groundwater monitoring data within the Double Bay area show seasonal variations ranging from around 0.5 to 1 m over the long term with a clear correlation with rainfall. This indicates that the water table within the Alluvium is sensitive to rainfall-derived recharge (the Alluvium is readily replenished by recharge). The range of seasonal fluctuation is smaller at bores closer to the coastal boundary where the groundwater level is constrained at mean sea level. This can be seen in Figure 7, where BH6 is approximately 200 m from the coastal boundary and shows much smaller seasonal variations than BH1 located approximately 400 m farther inland.

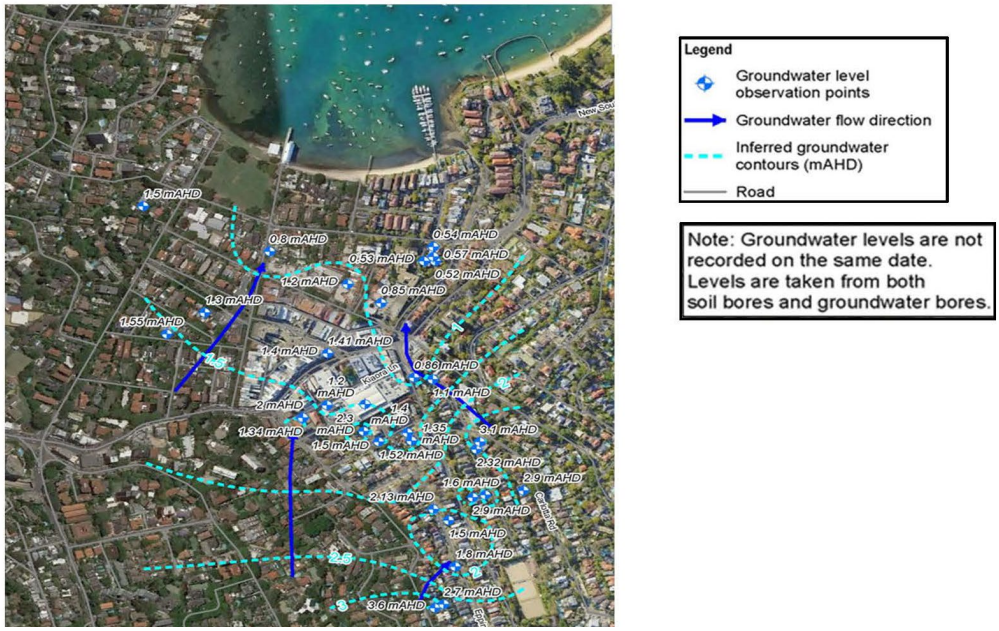


Figure 6: Interpreted groundwater contours and flow directions

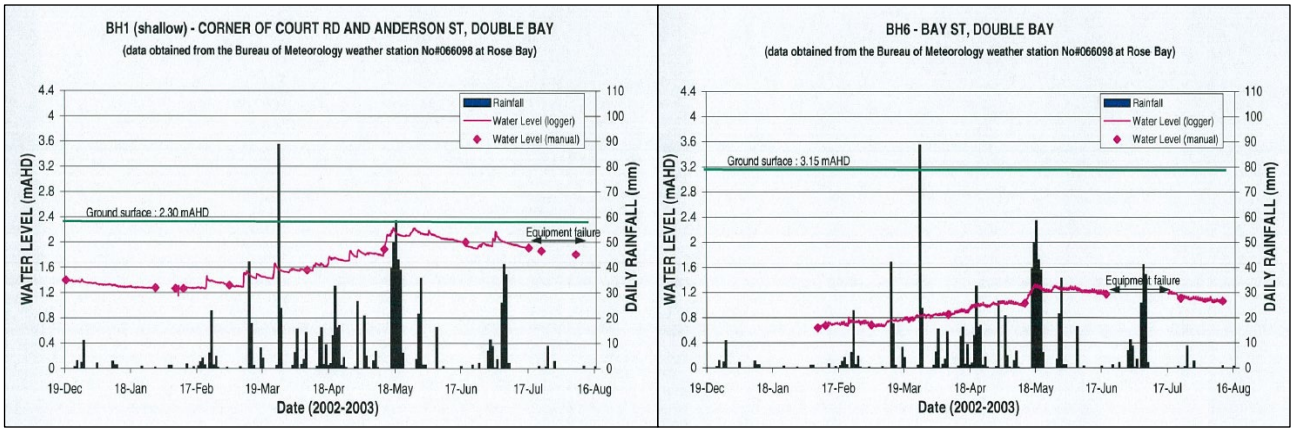


Figure 7: Seasonal trend (after Coffey, 2003)

4.4 HYDROGEOLOGICAL PARAMETERS

From the point of view of groundwater flow, the critical in-situ material properties are the hydraulic conductivity and storage coefficients (specific yield and specific storage). These properties control the resistance of the subsurface material to flow and the rate in which it is drained and/or re-saturated in response to stresses (and the rate in which aquifer pressure is propagated in a fully confined system at depth). Components of inflow and outflow, such as recharge and evapotranspiration, are also important although these are rarely measured in the field and more commonly inferred through other means (such as model calibration), using field-derived estimates of in-situ properties as constraints.

This section provides a summary of prior estimates of hydrogeological properties derived from field testing and modelling undertaken in the Double Bay area. These estimates provide the basis for parameterising and calibrating the regional numerical groundwater model described in Section 5.2.

Aquifer testing completed as part of geotechnical and hydrogeological investigations in the Double Bay area include CPTU dissipation testing and in-situ permeability testing (such as falling and rising head tests). Table 1 summarises the horizontal hydraulic conductivity values collected during field investigations. The majority of these tests have targeted discrete horizons within the Alluvium, as flow into shallow excavations are controlled by the properties of this shallow aquifer. While low hydraulic conductivity values have been derived from discrete clay lenses, the abundance of sand within the Alluvium and high hydraulic conductivities associated with the sand intervals indicate that the aquifer as a whole behaves as a high transmissivity system.

Limited information is available from the bedrock. The hydraulic conductivity of the weathered sandstone bedrock could be variable depending on the weathering profile and presence of jointing in the rock. Information available from other parts of the Sydney area indicate that the mean horizontal hydraulic conductivity in the upper 100 m of the Hawkesbury Sandstone ranges from around 0.01 to 0.1 m/day (around 10^{-5} to 10^{-4} cm/sec) (Tammetta and Hawkes, 2009). There are no estimates of vertical hydraulic conductivity although a horizontal to vertical permeability ratio of 10:1 has been reported (Longmac Associates, 1990), which is common in layered sedimentary aquifer systems.

There are no site specific estimates of storage coefficients. Specific yield of 0.1 to 0.3 is commonly assumed for the alluvium comprising fine sands and specific storage of 10^{-6} to 10^{-4} /m is reported in the literature for confined Hawkesbury Sandstone (GHD, 2015). For most lithologies, specific storage of 10^{-6} to 10^{-5} /m is considered realistic, with recent work by Rau *et al.* (2018) suggesting a plausible upper threshold of around 1.3×10^{-5} /m for specific storage in confined aquifers.

Table 1: Aquifer test data

Lithology	Method	Reference	Number of tests	Hori. permeability (cm/sec)
Sand with silt	In-situ permeability	Longmac Associates (1998)	3	$4.9x10^{-4}$ to $2.3x10^{-3}$
Sand with silt	In-situ permeability	Coffey (2003)	1	$<1x10^{-3}$
Sand	In-situ permeability	Longmac Associates (1990)	-	$6x10^{-4}$ to 2 $x10^{-2}$
Sand	In-situ permeability	Coffey (2003)	7	$1x10^{-3}$ to $1x10^{-2}$
Sand	In-situ permeability	Douglas Partners (2016)	1	$1.2x10^{-2}$ to $2.3x10^{-2}$
Clay	CPTU	Longmac Associates (1998)	10	$2.5x10^{-5}$ to $2x10^{-4}$
Clay bands	Laboratory testing	Coffey (1989)	2	$7.1x10^{-9}$ to $5.8x10^{-8}$
Clay/peat	In-situ permeability	Longmac Associates (1990)	–	$1x10^{-7}$ to $6x10^{-4}$
Sandstone	In-situ permeability	Longmac Associates (1998)	1	$9.4x10^{-6}$
Sandstone	In-situ permeability	Longmac Associates (1990)	–	Negligible small to $9x10^{-4}$

5 REGIONAL GROUNDWATER MODELLING

5.1 OVERVIEW

Due to the shallow water table in the Double Bay area, there is high potential for future developments to interact with groundwater. Regional groundwater modelling was carried out to provide:

- Spatial distribution of piezometric heads, depth to groundwater and associated seasonal range across the study area, such that the likely level of groundwater interference at future development sites could be understood.
- Potential cumulative long-term impacts of multiple subterranean structures (basements), including the magnitude and spatial extent of changes to the water table.

The regional groundwater modelling was carried out using the commercially available software program MODFLOW-USG (Panday *et al.*, 2013). The model domain covers the local groundwater catchment. The domain is large enough to fully enclose the extent of the Alluvium and capture the influence of key hydrological stresses. Along the coastal boundary, a constant head boundary condition is assigned with a head value of 0.1 mAHD. Elsewhere, a no-flow boundary condition is assumed along the model boundary that follows inferred regional flow divides (topographic ridge). Recharge and evapotranspiration are prescribed to the uppermost nodes near ground surface. Two recharge zones have been defined for the Alluvium and the outcropping Bedrock, to account for different recharge rates expected in these units of different properties. Both recharge and evapotranspiration rates have been adjusted during model calibration and are described further in Section 5.2.

The model layers are based on the Leapfrog geological model and includes the Alluvium, Peat and Bedrock. Although the Peat lenses are generally thin or localised, they have been incorporated into the model for consistency with the geological modelling. Figure 8 presents a cross-section that outlines the model layers. With the exception of the Bedrock layer (layer 7), each model layer is discontinuous and pinched out against the adjacent unit. This means there are areas where some model layers are absent e.g. layer 1 locally overlies layer 4 directly.

5.2 MODEL CALIBRATION

The regional groundwater model was calibrated transiently by comparing the simulated water levels against a combination of single groundwater level measurements collected from 25 bores at different times and time series of groundwater level measurements obtained from 8 monitoring bores. The model calibration period starts in January 2002 and finishes at the end of 2019, capturing 18-years of climate data. The model parameters have been adjusted during calibration to derive representative hydraulic conductivity (horizontal and vertical), specific yield and specific storage for each HSU. Recharge is calculated as a percentage of average daily rainfall for each stress period. Rainfall is first converted to

recharge using a factor and applied over the Alluvium. This Alluvium recharge is then converted to Bedrock recharge using another factor. This two-stage approach maintains a sensible ratio between the two recharge rates throughout the calibration process, ensuring that recharge applied over less permeable Bedrock is no greater than recharge over more permeable Alluvium. Evapotranspiration rate and extinction depth are adjusted as single model-wide values.

Table 2 summarises the calibrated model parameters. These parameter values are generally consistent with the published range of values. Figure 9 presents examples of calibration results for the available data of several monitoring bores within the study area. The hydrographs show that the modelled heads match the observed heads reasonably well, with seasonal fluctuations appropriately replicated. In particular, smaller seasonal fluctuations observed closer to the coastal boundary are also simulated by the model consistent with the expected groundwater behaviour.

5.3 ASSESSED DEPTH TO GROUNDWATER TABLE

Figure 10 presents maps of depth to groundwater for the wet and dry periods. These maps have been generated by subtracting the modelled surface of the water table from the regional ground model. The maps provide indications of areas where the water table is shallow and the expected seasonal range. For example, Figure 10b indicates areas of shallow water table (< 1m below ground) in the area to the south of Kioara Lane during wet period, consistent with high groundwater levels measured in a monitoring bore located in this area (refer to BH1 in Figure 7). Within the context of potential future developments, the maps provide useful indications of the risk of groundwater interference to be outlined in Section 5.4.

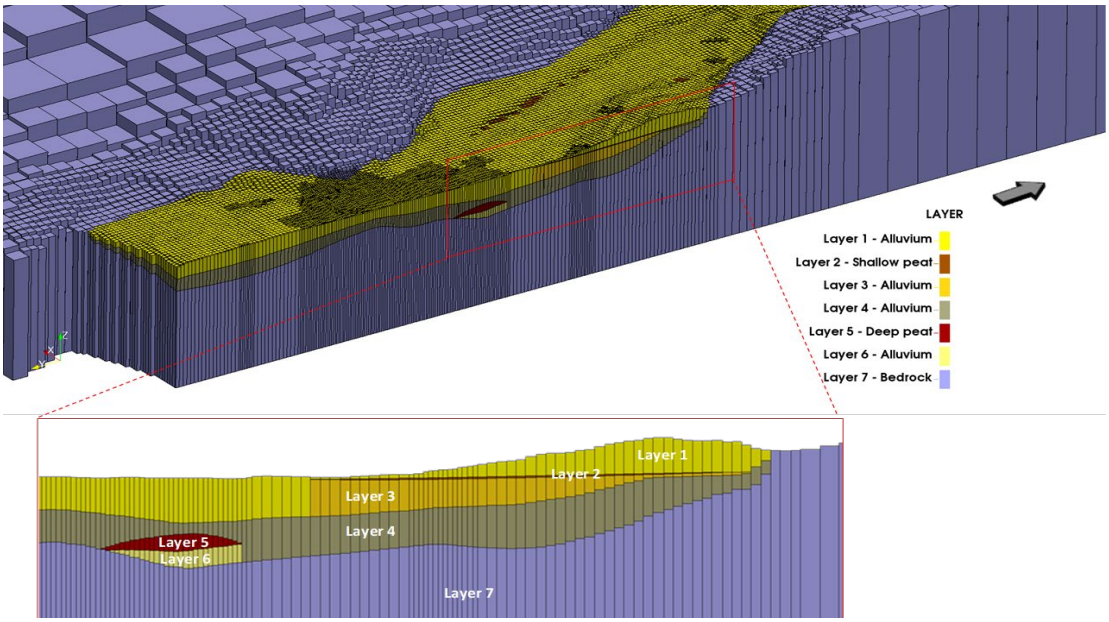


Figure 8: Regional groundwater model

Table 2: Calibrated model parameters

Parameter	Calibration values
Alluvium horizontal hydraulic conductivity (K_H)	0.5 to 10 m/d (average 3 m/d)
Alluvium hydraulic conductivity anisotropy (K_H/K_V)	10.47
Peat horizontal hydraulic conductivity (K_H)	0.035 m/d
Peat hydraulic conductivity anisotropy (K_H/K_V)	6.36
Bedrock horizontal hydraulic conductivity (K_H)	0.044 m/d
Bedrock hydraulic conductivity anisotropy (K_H/K_V)	11.05
Alluvium specific yield (Sy)	0.08
Alluvium specific storage (Ss)	1.2×10^{-5} /m
Peat specific yield (Sy)	0.085
Peat specific storage (Ss)	2.7×10^{-6} /m
Bedrock specific yield (Sy)	0.022
Bedrock specific storage (Ss)	5×10^{-6} /m
Alluvium recharge	20% rainfall (average 237 mm/yr)
Bedrock recharge	4.4% rainfall (average 52 mm/yr)
Evapotranspiration	1200 mm/yr
Evapotranspiration extinction depth	2.5 m

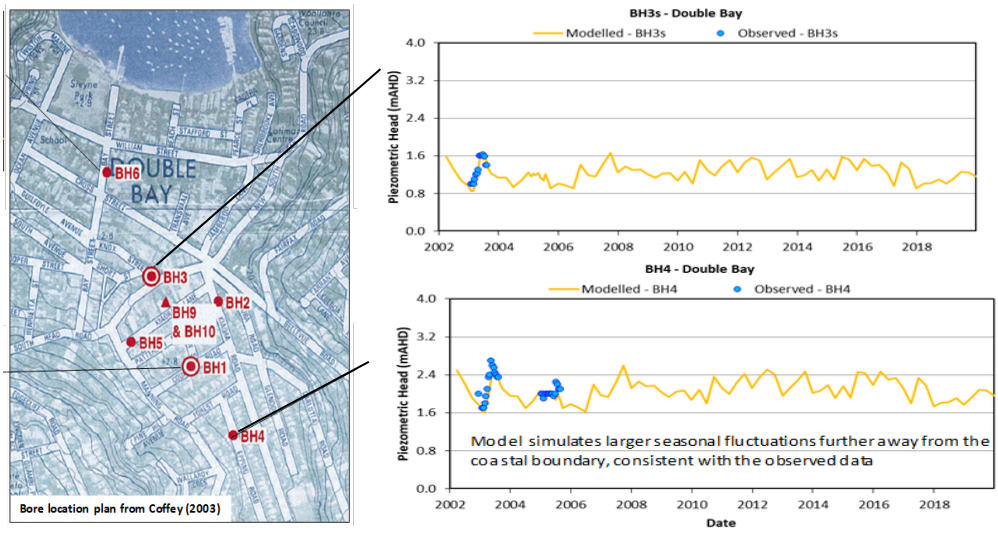


Figure 9: Modelled hydrographs (Courtesy of Coffee for the borehole location plan)

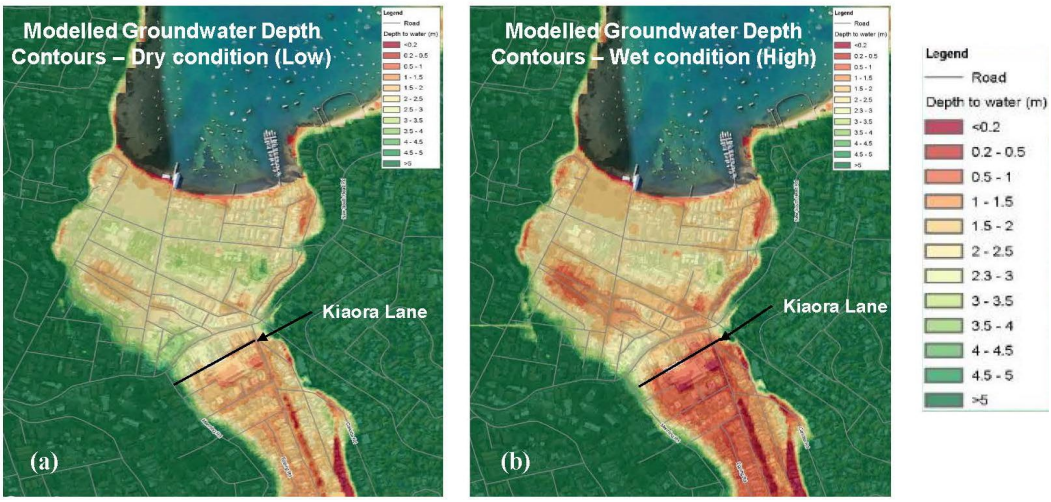


Figure 10: Modelled depth to groundwater for (a) dry season and (b) wet season

5.4 POTENTIAL LONG TERM CHANGES TO WATER TABLE WITH FUTURE DEVELOPMENTS

The potential long term cumulative impacts of future basements have been assessed by incorporating these features into the calibrated model as zones of low permeability material. The future developments being considered in the present assessment are indicated in Figure 3. Two scenarios with regards to these future developments have been modelled:

- A full cut-off scenario where the basements/low permeability zones are assumed to fully extend to the base of the Alluvium/ top of Bedrock.
- A partial cut-off scenario where the basements are assumed to terminate 12 m below ground surface, allowing groundwater to flow below them.

Figure 11 presents the modelled maximum change in the water table for the partial cut-off and full cut-off scenarios. The positive change indicates drawdown (lowering) of the water table and negative change indicates mounding (raising) of the water table. The figures indicate the following:

- The partial cut-off results in localised and minor lowering of water table (less than 200 mm) at the northern side of Double Bay Centre.
- The full cut-off results in mounding of the water table on the up gradient side and drawdown on the down gradient side due to impedance of groundwater flow. The maximum drawdown and mounding simulated by the model are generally less than 0.3 m. Up to around 0.4 m of mounding is simulated in the southwest, where groundwater flows from the valley edge; however, this occurs in an area of low risk of groundwater interference where the depth to groundwater is greater than 2 m.

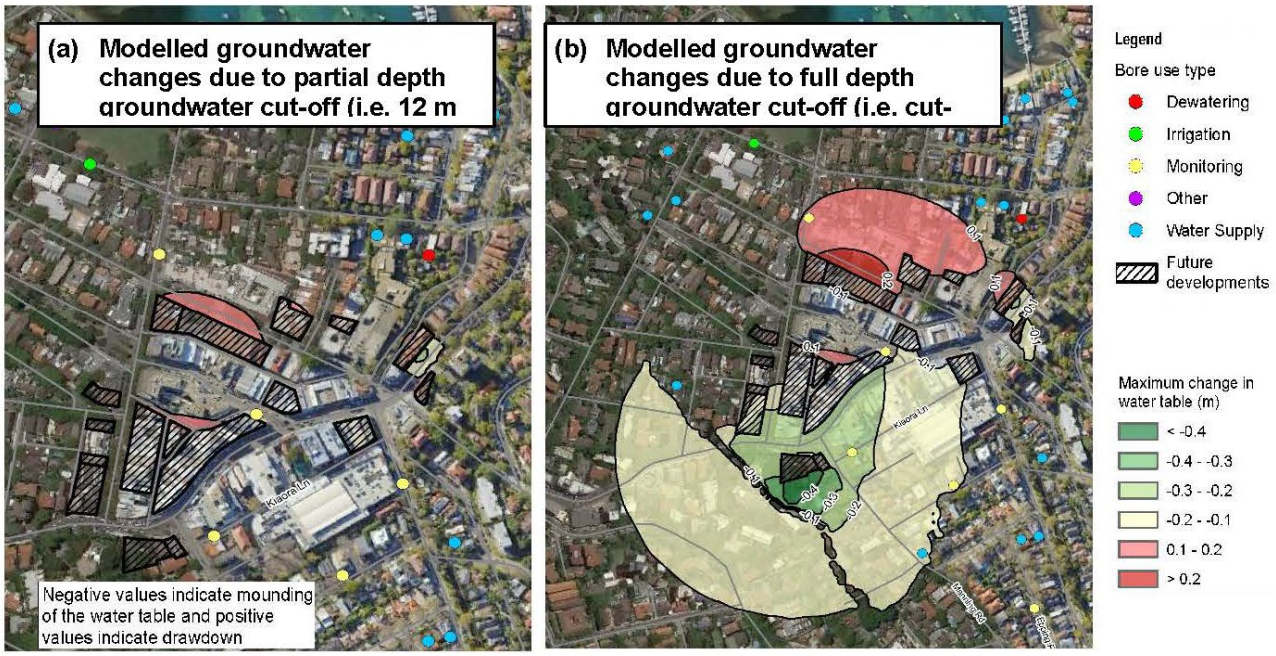


Figure 11: Contours of maximum change in long term water table for (a) partial cut-off and (b) full cut-off cases

6 GEOTECHNICAL ASSESSMENT OF DEWATERING INDUCED SETTLEMENT

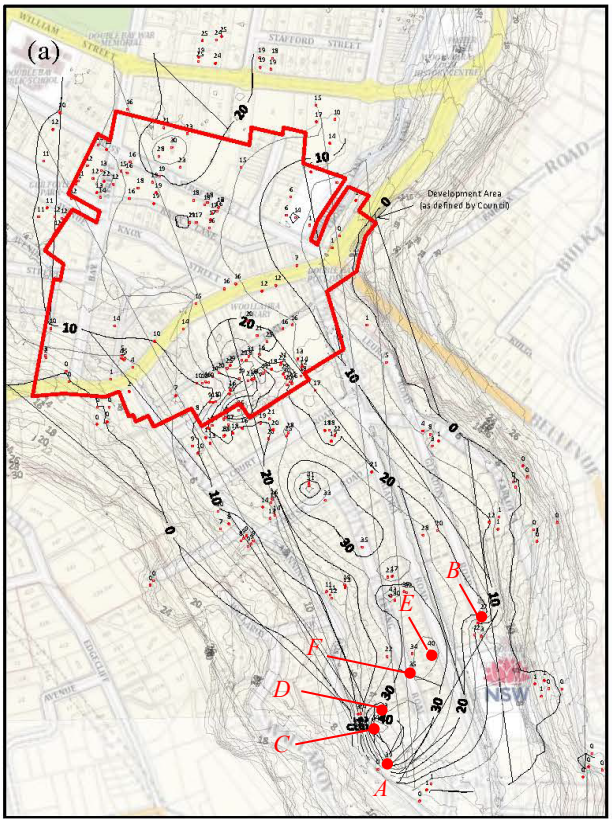
For the sandy alluvium generally encountered within the Double Bay valley, the impact of construction dewatering is expected to extend far beyond the excavation footprint. The lateral impact can extend up to some 800 m away from the excavation near the recharge point at the sandstone hillside. Further, the severity of the dewatering-induced settlement is strongly related to the ground conditions of each site. The lowering of groundwater in areas with presence of compressible upper peat soils would cause a much greater settlement than other areas without the peat layers. Consequently, a "Settlement Index Plot" in response to a fixed groundwater drawdown depth was developed based on 270 analysed settlement points, each was assessed based on available site specific geotechnical investigation data. The locations of the settlement points are shown by the red dots on Figure 3. The contours of assessed settlement index in response to an assumed 1m depth of groundwater drawdown are presented as Figure 12a. This assessed Settlement Index Plot shows similarity to the isopach map of upper peat layer thickness depicted in Figure 4b in terms of the locations of peat and the assessed settlement concentrations. The assessed ground surface settlement in response to groundwater drawdown depths for various settlement points within the area with upper peaty soil are shown in Figure 12b. The following points are highlighted with regards to the settlement plots in Figure 12:

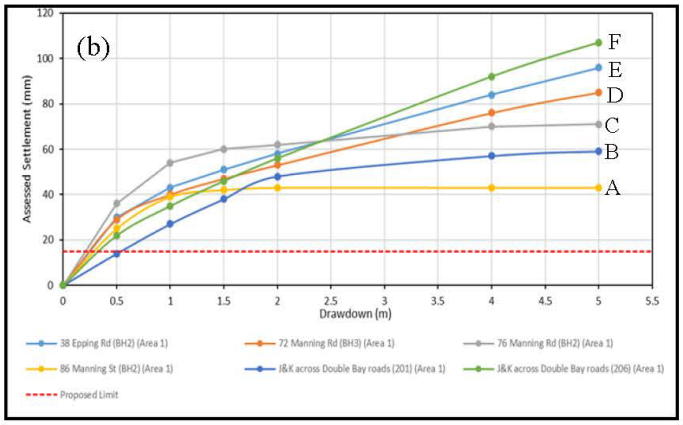
- The dewatering induced settlement for each data points was analysed based on one-dimensional (1D) method where soil layers were represented by elastic soil model with characteristic Young's moduli for granular materials; and consolidation model with compression and recompression coefficients for fine grained soils. The compressibility properties adopted for the different materials are listed in Figure 12c. These engineering parameters were derived on the basis of:
  - Available information from past studies within the area
  - Review of in-situ testing results from available geotechnical investigation data
  - Review of geotechnical laboratory testing results where available
  - Calibration against known surface settlements induced by dewatering at selected locations
- The initial water table prior to water drawdown was assumed to be at the middle of the seasonal fluctuating range, which is typically near the top of the peat layer at about 1 m below ground surface. The lowest level of the fluctuating range, on the other hand, is typically below the top of the upper peat layer.
- The portion of the upper peat layer within the seasonal water fluctuating range should have been consolidated due to past water table fluctuation. The settlement induced by the groundwater drawdown could be reasonably predicted by the re-compression ratio of the soil. In reality, this portion of the peat layer has been subjected to cyclic loading with numerous unloads and reloads in the past. This would have ratcheted down the re-compression ratio from a higher initial value during the first few unload/reload cycles. The portion of the upper

peat layer below the seasonal water fluctuating range is generally very soft to soft. The lowering of the groundwater could potentially load up the soil to its normally consolidated state.

- It is interesting to highlight that the assessed total settlement experienced at certain areas with shallow bedrock (e.g. curves A, B, and C in Figure 12b) appears to plateau beyond 2 m depth of groundwater drawdown. The main reason for this assessed behaviour is that the upper peat layers within this area are generally occurred at shallow depth and are within or above the existing groundwater fluctuation range. The further drop of water table due to dewatering will not incur additional loading to these shallow peat layers.
- For other data points located in the areas where bedrock is relatively deep, the settlement plot indicates continuing increase in settlement with the increase in the groundwater drawdown (Curves D, E and F in Figure 12b). This increase is considered to be primarily caused by compression of the lower peat/clay layer within the sand dominated alluvium.

Owning to the sandy nature of the alluvium present in Double Bay, the lowering of groundwater caused by dewatering could extend to a considerable distance away from the dewatering location. In a 3-dimensional context, the impact of dewatering can be felt over a substantial area within the valley. This is especially prominent at the southern Double Bay area where compressible upper peat soils exist. This implies that the impact of dewatering is not restricted to localised areas close to the dewatering site, but could affect the entire Double Bay valley.





(c) Material	Compression ratio CR = cc/(1+e$_0$)	Re-compression ratio RR = cr/(1+e$_0$)	Young's modulus (E') MPa
Fill	–	–	10
Very soft to soft upper peat	0.35	0.058	–
Very stiff lower peat	0.3	0.05	–
Firm to hard clay layers within the sandy alluvium	0.1	0.014	–
Very loose to loose sand	–	–	5
Loose to medium dense sand	–	–	15
Dense to very dense sand	–	–	35
Residual soil	–	–	50

Figure 12: (a) Contours of assessed settlement index for 1m drawdown depth, (b) Examples of ground surface settlement response to drawdown depth; (c) Adopted compressibility parameters

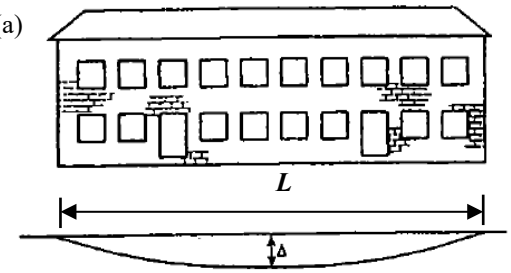
7 REMARKS ON TOLERABLE SETTLEMENT OF RESIDENTIAL BUILDINGS

To effectively control the potential damage caused by dewatering, it is essential to assess the maximum acceptable settlement for the buildings in the Double Bay area. The settlement tolerance applicable to the existing buildings, typically one to two storeys constructed on shallow footings, can be appraised based on Australian Standards AS2870-2011 and relevant published literature, e.g. Burland *et al.* (1997, 2002) on building settlements and associated damages. Other considerations including possible past damages of the buildings, flexibility of the structures and pipe drain tolerances are also given as part of the appraisal.

Cracking in masonry walls is usually, but not always, caused by differential settlement. With reference to the schematic representation shown in Figure 13a regarding the deflection ratio Δ/L at which cracking is initiated, Burland (1997,2002) provided the limiting Δ/L values in percentage for the five categories of damage for masonry wall with zero horizontal

strain (see Figure 13b). These five categories of damage with reference to walls have been adopted in the current AS2870-2011. In essence, the categories are numbered 0 to 4 in increasing severity. Normally categories 0, 1 and 2 relate to 'aesthetic' damage, 3 relates to 'serviceability' damage and 4 represents damage affecting 'stability'. Burland *et al*. (2002) have indicated that the dividing line between categories 2 and 3 damage is particularly important. If the damage exceeds Category 2 the cause is usually much easier to identify and is frequently associated with ground movement.

(a)



(b)

Category of damage	Normal degree of severity	Limiting Δ/L (%)
0	Negligible	0.05
1	Very slight	0.075
2	Slight	0.15
3	Moderate	0.3
4	Severe to very severe	>0.3

Figure 13: (a) Schematic representation of wall deflection, (b) Relationship between category of damage and limiting Δ/L for zero horizontal strain (after Burland *et al*. 2002)

By way of example, using Δ/L of 0.075% (maximum value for category 1) and for a building comprising full masonry construction with a typical wall length of 15 - 20 m, a differential wall settlement of 15 mm could be calculated as the maximum tolerable value before cracking become visible and is classified as being at risk of Category 2 damage.

In relation to pipe drain tolerances, the acceptance criteria of 0.1 degree for joint rotation of relatively rigid pipes such as cast iron pipe can be adopted based on CIRIA (1996). The aforementioned threshold deflection ratio of 0.075% corresponds to a rotation of about 0.043 degrees, which is deemed to be satisfactory for the allowable joint rotation of rigid pipes.

Theoretically correct and simple as it may seem, the evaluation of differential wall settlement is not always straight forward. Alternatively, total ground (surface) settlement limits could be used as an ultimate measure to control damage of buildings caused by dewatering. If the building is conservatively assumed to have no stiffness so that it conforms to the 'greenfield site' subsidence trough, then it is possible to consider y_s to be conservatively the same as the differential wall settlement. The adoption of this conservative assumption is reasonable because the surface settlement limit that is applicable to existing buildings will have to be assessed in light of possible past damage and flexibility of the buildings. Relatively rigid and damaged structures now are likely to be more sensitive to increased surface movement due to loss of stiffness, and therefore some reduction in the settlement limit might be appropriate.

8 CONCLUSIONS

Double Bay is situated in the valley between the ridgelines of Edgecliff/Darling Point and Bellevue Hill/Point Piper, occupying the low elevation harbour front area. The alluvium within the valley, comprising sand with minor silts, clay and peat, forms a highly productive water table aquifer, which is underlain by the less permeable fractured bedrock aquifer. The water table fluctuates in response to seasonal variations in rainfall, with up to 1 m of variation observed in monitoring bores constructed within the alluvium. A calibrated regional groundwater model has shown that cumulative impacts associated with multiple subterranean structures (basements) could lead to mounding and lowering of the water table over the long term, albeit this is generally estimated to be controllable.

For the sandy alluvium generally encountered within the Double Bay valley, the impact of construction dewatering is expected to extend far beyond the excavation footprint. Further, the severity of the dewatering-induced settlement is strongly related to the subsurface conditions of the site. The lowering of groundwater in areas with the presence of compressible upper peaty soils would cause a much greater settlement than other areas without the peat layers. From the compilation of 270 boreholes and CPT investigations conducted in the past, a "Settlement Index Plot" was developed that indicates the different degrees of susceptibility to dewatering-induced ground surface settlements for the different areas within the Double Bay valley.

To effectively control the potential damage of existing buildings caused by dewatering (typically one to two storey structures supported on shallow footings), a ground surface settlement limit can be considered based on AS2870-2011 and relevant published works by Burland et al. (2002). Other considerations including possible past damages of the buildings, pipe drain tolerances and historic groundwater level fluctuation have also been given as part of the appraisal process.

9 ACKNOWLEDGEMENTS

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WHY AESTHETICS NEEDS EARLY CONSIDERATION IN TUNNEL DESIGN AND EXCAVATION – A CASE STUDY FROM THE M8

Helen Baxter-Crawford[1] and Jonathon Sutton[2]

[1]Principal Engineering Geologist,
SMEC, Sydney Australia (formerly Golder)
[2] Senior Geotechnical Engineer,
Golder Associates, Sydney, Australia

ABSTRACT

Road tunnels are more complex than the driving eye would surmise – hidden behind the dark ceilings and colourful wall panels are rock, support bolts, multiple layers of shotcrete, possible water drainage, access passages, fire control systems, electrical systems, and speed cameras. During the design phase of a tunnel project all these components are given significant consideration as they govern the ultimate size of the excavation that is needed. At construction, focus shifts from what will be added to what and how can rock be extracted quickly and safely. The crown and face condition come to the forefront as inputs into the ground support requirements and as short-term prediction of excavation conditions in the following days. Classification of the rock mass is skewed to encapsulate the face, shoulder and crown condition as well as the bolt zone above the crown. Side wall and floor conditions are noted where possible, but visibility is often limited to the current cutting (about 1.5m, with previous cuts already shotcreted) or not possible due to mud and rock waste. Once excavation has progressed, focus shifts to the functionality, endurance and aesthetics the general public will see – the smooth road surface, the painted ceiling with clear road signs, and the architectural panelling on the walls. For the latter two, consideration is required as to how these heavy items will be anchored to the roof and walls. For the M8, the broader face/shoulder/crown rock mass classification was found to be inappropriate to the narrow anchor zone for the architectural panels, which required anchoring specifically at a height 3m above the floor excavation level. This was particularly the case where narrow siltstone beds occurred with the sandstone rock mass and where dykes and shears intersected the tunnels. Fortuitously, compilation mapping including the side walls had been completed for another purpose for the project and was able to be repurposed to assist with panel construction design. Different bolt lengths were required for each rock class in sandstone, shale and dolerite (dykes) with the conditions specific to the zone 3m above the floor level reassessed by experience senior geological staff as the input for the design. A package of works was provided for each wall, indicating the rock class at the anchor zone, both as a map and tabulated. This improved construction sequencing, costing and allowed pre-purchasing of all materials before work commencement. Significant costs would have occurred had the daily mapping not captured the side wall conditions and had compilations been required solely for this purpose.

1 INTRODUCTION

The M8 is a newly opened twin road tunnel spanning some 9 km connecting the M5 at Kingsgrove to the on-off ramps at St Peters Interchange in Sydney's south and forms a crucial segment in Sydney's WestConnex Project. The mainline tunnels will ultimately connect the M5 to the M4-M5 link under construction beneath St. Peters and to the north-west, while at the surface the on/off ramps lead into the Interchange and will eventually link to the Sydney Gateway, Figure 1. The project was completed in three stages:

1. Design phase – completed by a joint venture team with scope including borehole investigation program, geotechnical model development, assessment of ground types, and design of support requirements suitable to each ground type.
2. Construction phase – this included all ground excavation mapping and installation of tunnel support.
3. Aesthetics phase – this included road surfacing, installation of lighting, safety systems, signage and architectural panelling on the tunnel side walls.

One aspect of the build was pertinent to all three phases – the geotechnical properties of the rock in which the tunnel was constructed.

This paper addresses the geological/geotechnical framework systems implemented for the project and how they focussed on tunnel support criteria. It presents the limitations of the framework in addressing the aesthetic works component and how this was overcome. It provides suggestions for future projects to include consideration of all end-users of mapping data collection to better plan how data is collected. This would improve efficiencies and could result in significant cost savings.

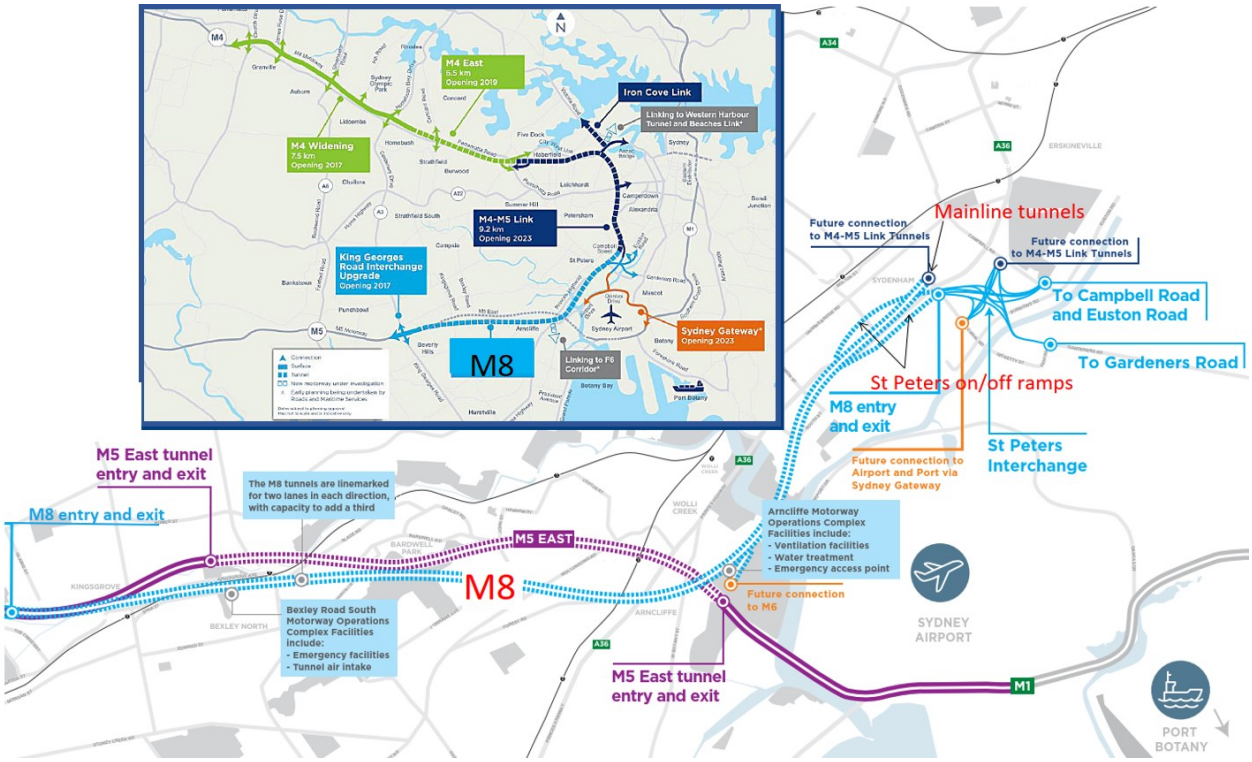

Figure 1: Map of the M8 within the WestConnex project (modified from https://www.westconnex.com.au/)

2 PROJECT GEOLOGY

The M8 mainline tunnels were predominately excavated in Hawkesbury Sandstone with the connection to the M5 in the overlying Mittagong Formation and Ashfield Shale. The on/off ramps at St Peters progressed through Hawkesbury, Mittagong and all four sub-units of the Ashfield Shale before breaking through to the surface. In addition, the tunnels intersected several dolerite dykes and steeply angled fault zones.

Within the tunnels, the Hawkesbury Sandstone sequence varied:

- Fine to medium grained cross-bedded quartzose sandstone with major bed partings typically spaced 1 to 3 m.
- Massive, medium to coarse grained quartzose sandstone that formed channels cut into the cross-bedded sequences.
- Shale breccia layers and lenses of siltstone were common along the bed boundaries. The thickness of the lenses ranged from a few centimetres to over 2 m.

The Mittagong Formation was a mix of interbedded to interlaminated sandstone and siltstone.

All four sub-units of the Ashfield Shale were present beneath St Peters, displaced by regional faulting. They comprised (in age oldest to youngest):

- Rouse Hill Siltstone – finely laminated dark grey to black siltstone with <5% sandstone lamellae;
- Kellyville Laminite – distinctly interlaminated siltstone and sandstone in approximately equal proportions;
- Regentville Siltstone – interlaminated siltstone with up to 25% sandstone;
- Mulgoa Laminite – interlaminated to finely interbedded siltstone and sandstone.

Due to the depth of tunnelling weathering was mostly restricted to the M5 connection and in the Mulgoa Laminite at the St Peters on/off ramps.

3 TUNNEL CONSIDERATONS

The tunnel design phase is complicated and many facets control the ultimate excavation footprint – not just whether the road has as two or three lane capacity. Amongst others, designers need to consider height allowances for trucks, inclusion of breakdown bays, emergency egress passages, space for electrical and fire suppression systems in the crown, space for speed cameras, height allowance for road signage and drainage requirements. Secondary to this comes achieving a design that will allow for such a footprint and will support the encountered geology. To facilitate support installation and improve cost effectiveness, the geology is classified into groups of similar rock-mass conditions and support requirements to suit each class are developed. This allows geological/geotechnical modellers to predict quantities of each ground type, thereby assisting budgeting and in some instances purchasing of specific bolt sizes/lengths, shotcrete and other support infrastructure, including plant machinery.

At construction, the focus shifts to what is being excavated and how quickly and safely it can be completed. Site geologists/geotechnical engineers are used to map the excavation as it proceeds and advise the current ground conditions. The information feeds into support type selection and continued excavation is permitted. From a practical standpoint, the classification of ground conditions is scaled to define a single ground type for the excavation except in rare circumstances, like where the face crosses a lithological contact. It is also skewed towards assessing the crown and shoulders, more so than the lower face and walls, as the crown conditions dictate the support requirements at that location. The face conditions may provide a preview to assist in predicting any future changes to conditions.

At the time of mapping, only a couple of metres of unsupported rock are exposed. Side walls are mapped in the newly exposed faces, but observations may be limited due to dust and plant obscuring the view. The previous cuts are covered in shotcrete which may include complete coverage of the side walls, depending on the lithology intersected and durability limitations of that lithology. As such, the ability to back-capture all the side wall information is limited.

At the completion of excavation, focus turns to the functionality, endurance and aesthetics of the tunnel. It was at this stage of the project that it was recognised that the classification of ground conditions was of limited suitability for anchor design for the architectural panels.

4 TUNNEL SUPPORT ROCK MASS CLASSING

At design phase, the Hawkesbury sequences and Mittagong-Ashfield lithologies were segregated into rock mass classes based on the site investigation data collection (Golder, 2017). The classing system was similar in principal to the Sydney Rock Mass Classification Scheme of Pells et al (1998) though the boundary conditions were modified to suit the project specifics and tunnelling criteria. Five classes were established for the Hawkesbury and five for the Mittagong/Ashfield and incorporated:

- UCS strength
- Spacing of discontinuities (overall and specifically related to bedding and joints) and
- Percentage of seams within the zone.

It was a wholistic approach that worked well in construction practise. The boundary conditions recognised and incorporated tunnel scale such that an entire face and crown would typically satisfy all criteria for one particular class. An excavation cut that may warrant two or more classes being assigned would be where dykes or other contacts were intersected. The mapped classifications then became a key component in the selection of the ground type and subsequent tunnel support to be installed and approval process for the permit to tunnel.

Of critical interest to the subsequent aesthetics phase the Hawkesbury classes were inclusive of any siltstone lenses or shale breccia present within the stratigraphic unit. The presence of the siltstone resulted in a downgrading of the sandstone class. However, the overall strength and discontinuity spacing conditions in the Hawkesbury class were still better than the equivalent category for the Ashfield Shale. The scale of the classing system was therefore not suitable for installation of architectural panels. Dykes were also incorporated into the class of the surrounding rock rather than be classified uniquely.

5 ARCHITECTURAL PANEL SPECIFICATIONS

Following paving of the roadway and installation of the electrical and drainage systems, architectural vitreous enamel wall panels were installed along the tunnel side walls to hide the infrastructure from view, Figure 2. In the case of M8 the panels also provide location information and assist drivers in maintaining focus for the seven-minute drive, Figure 3. The typical panel support system used on the project consisted of a bracket system with threaded rods, “C-channels” with shoulder bolts, connection bolts and chemical-grouted anchors. Stainless steel (grade 316) were used for the embedment into the tunnel walls with the chemical grout. The panels were attached through pre-fabricated keyhole slots and comprised a vitreous enamel front face and a backing board of calcium silicate in order to achieve flatness and for anti-

drumming purposes (Ceratec, 2019), Figure 4 (Ceratec, 2018). The vitreous enamel panels had a specified reflectance value of 60% in order for the tunnel lighting design to achieve a minimum sustainable outcome.

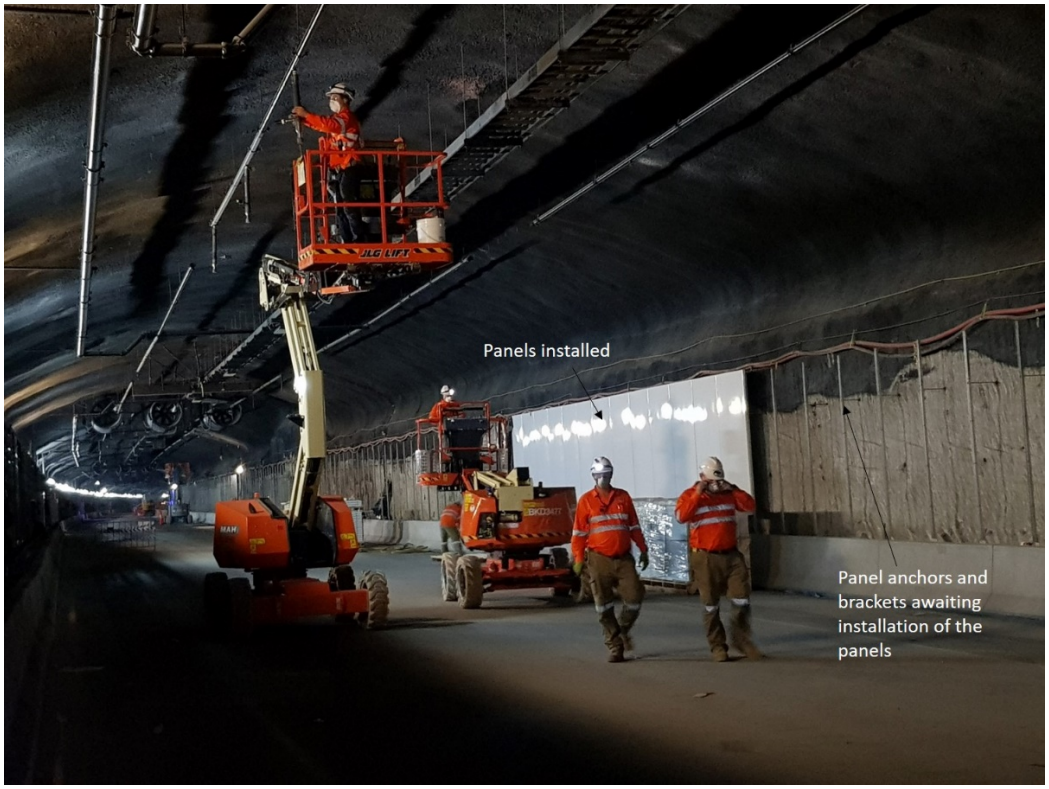

Figure 2: Architectural panel installation in progress (courtesy of @WestConnex Facebook page)

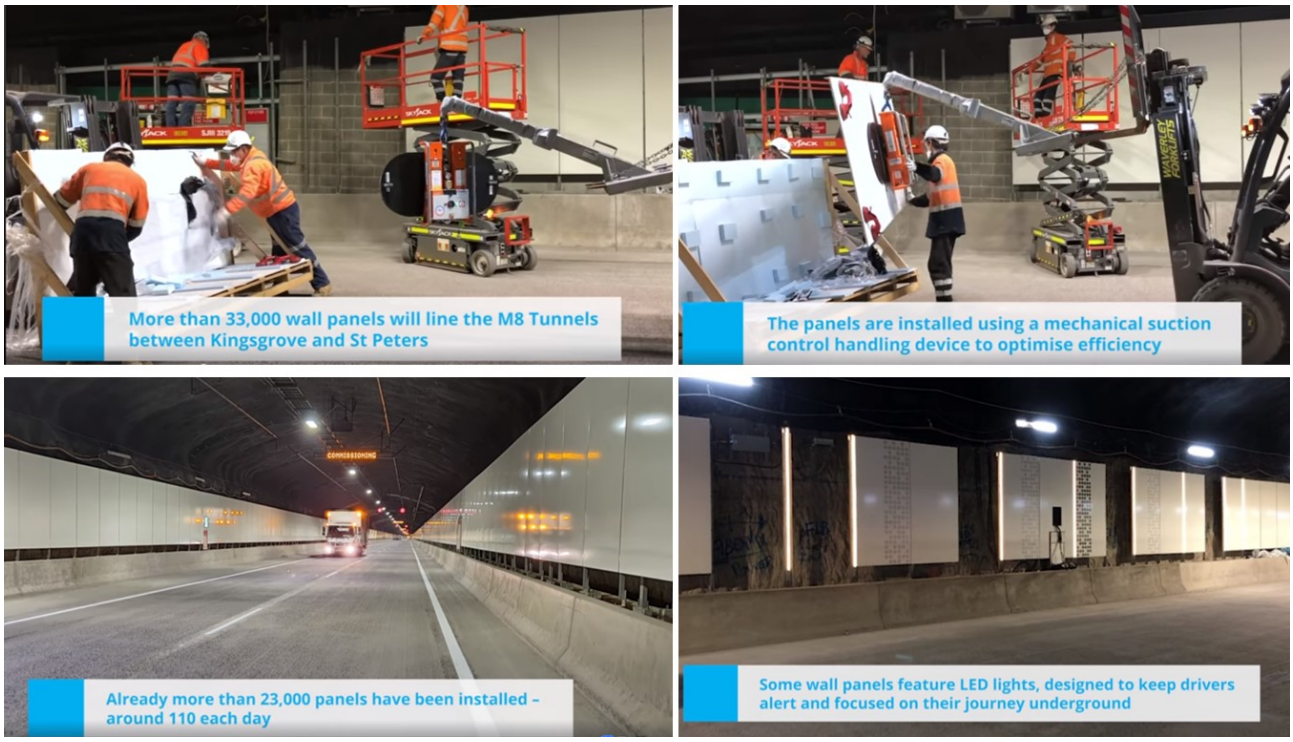

Figure 3: Installation of architectural panels in the M8 (courtesy @WestConnex Facebook page)

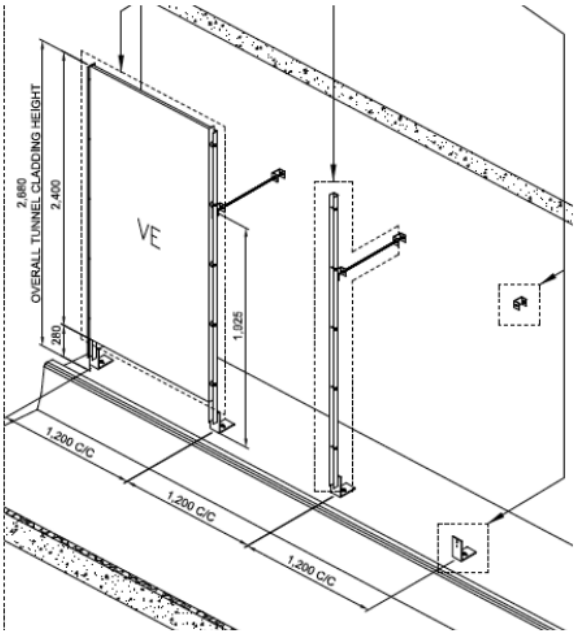

Figure 4: Typical Ceratec Hook-on Vitreous Enamel (VE) panel system

The panels were 2.4 m high by 1.2 m wide and weighed 78 kg each. Brackets were anchored to the side walls at a standard height 3 m above the tunnel invert. The length of the anchors was directly related to the rock class attributes assigned at that particular height above invert and designed to account for both dead weight and the live load attributed to wind or vehicular gusts. Once bracketing was in place, the panels were lifted into place with a mechanical suction control handling system to maximise efficiency of installation as over 33 000 panels were required for the M8 project. The total length of tunnels covered by panels is in excess of 40 km (each wall of each tunnel).

Installation required the drilling of anchors into the sidewalls. The anchor bond lengths varied depending on the lithology and quality of the material present in the side walls. Due to the narrow zone where anchoring would be undertaken, each anchor needed to be specific to the material encountered at that location to ensure longevity of the panel adhesion. This was particularly critical where the lithology encountered at the level 3 m above invert differed to the overall lithology of the wall – like where siltstone beds were present in Hawkesbury Sandstone or dykes were intersected. Dykes were of particular interest to the panel installers as they were subvertical structures, with often comparable width to a panel. No classing framework had been established for dykes as part of the design phase – they were incorporated into the ground type classification of the intruded stratigraphy.

The location of geological structures was also important to the panel team as these could cause very localised degradation to the rock mass that may intersect the panel installation zone. Both the Hawkesbury and Ashfield Shales exhibited subvertical fault zones where there were concentrations of very closely spaced joints over distances of less than 2 m separated by improved, less fractured rock mass. Panel installation across the fault zone would require long anchors and bond length to accommodate the reduction in strength and rock quality. The fault zones were typically treated with shotcrete at the time of excavation to assist in maintenance of side-wall integrity so physical review of the walls at the time of anchor installation was not an option.

Sub-horizontal or gently dipping bedding shears were narrow features common in the Hawkesbury. The shears were typically around 100 mm wide but continuous for many tens of metres. For anchor integrity it was important to recognise where these features would cross the anchoring level.

6 MAPPING (UN)SUITABILITY

Upon request from the panels team to provide rock mass classifications for the side walls to assist their installations, it was quickly recognised the routine daily mapping and rock classing would be insufficient to suit their needs. The scale of the classifications was too large. Fortuitously the daily maps had already been compiled into a single detailed map for each tunnel and these were repurposed to guide the panel installation. There would have been several weeks of work involved in compiling and cross-checking daily maps to define the rock mass class of the anchor zone had this not been available. This would also have delayed purchasing of anchors and potentially delayed the start of installations.

The compilation maps were assessed for specific class along the anchoring level 3 m above the invert and provided to the installation team as a site guide, Figure 5.

Anchor bond length requirements for each established rock class were calculated. Classification parameters were also established for dyke and the appropriate anchor bond lengths for those conditions calculated. Tables of quantities of tunnel metres in each class were given to the design team to enable purchasing.

7 RESULTS

The quantities assessment in the Mainline tunnels (99% excavated in Hawkesbury Sandstone) indicated:

- 12% of the material along the anchor level classed using the design daily mapping system as “sandstone class” was relegated to a siltstone class.
- This equates to 4 km worth of wall being reclassed as siltstone.
- Dyke intersections comprised just over 300 m length of tunnel walls. This equates to over 250 panels installed in dykes.

In the on/off ramps, mostly excavated in Ashfield Shale:

- 45 panels were anchored to dykes.
- 500 m worth of siltstone side walls were downgraded by at least one rock mass class along the anchor level.

These results indicate had the daily mapping design classing system been adopted more than 10% of the length of walls requiring panels would have had installations where the anchor bond length was less than optimal to achieve the design purpose. This would have posed a potential safety hazard and may have reduced the life expectancy of the panel anchoring system.

From a cost perspective, 10% of the panels required longer and therefore more costly anchors than would have been budgeted based on the daily mapping classification. However, the anchoring was now suited to the purpose and uniqueness of the excavation. The reduction in risk by appropriately anchoring the panels to the rock conditions far outweighed the cost of additional anchor length.

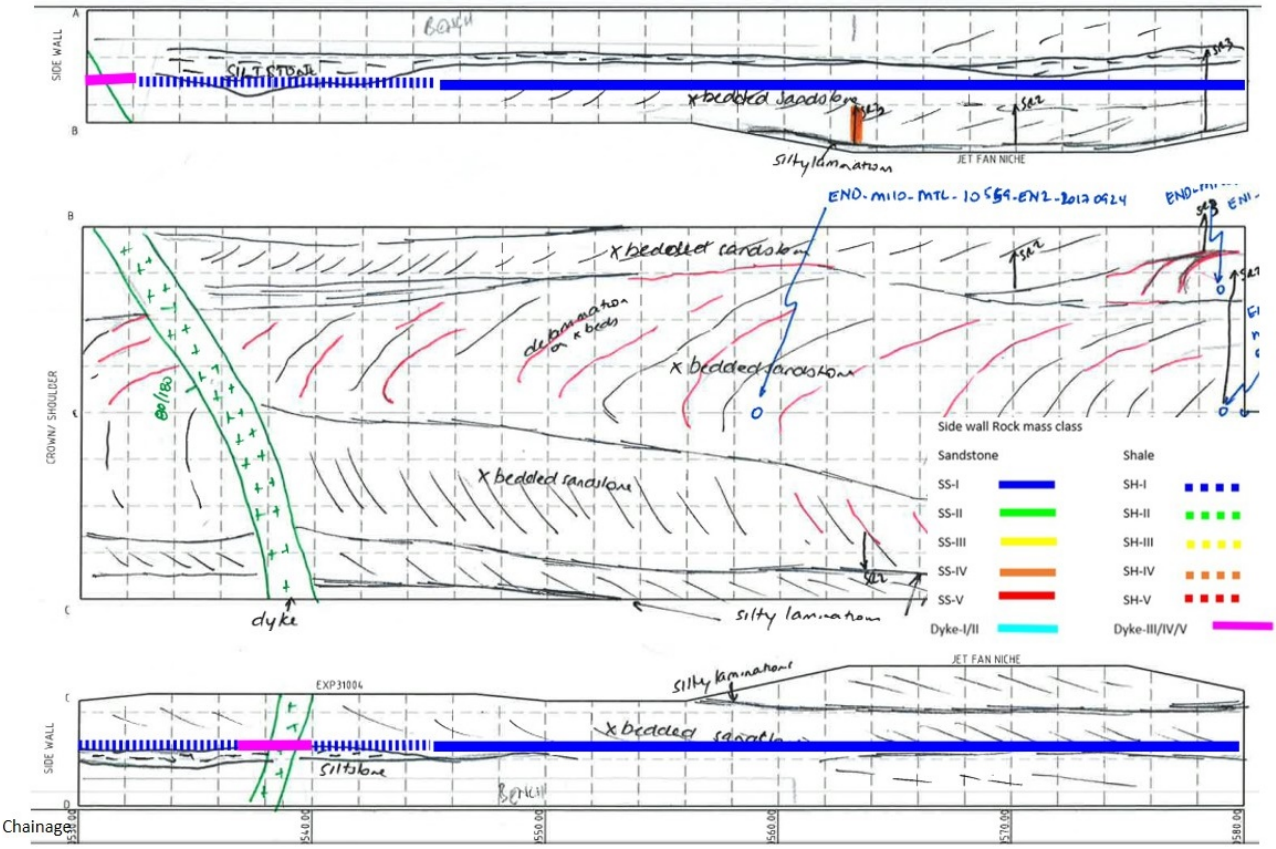

Figure 5: Example compilation map with indicated rock class at level 3 m above invert. (Not to scale. Data and legends not pertinent to this paper have been removed.)

8 WAY FORWARD AND CONCLUSIONS

For future projects consideration should be given to all end-users of the daily geotechnical maps and how the rock mass classing systems relate to later programs. Had mapping incorporated a specific rock classification for the level 3 m above the invert as part of daily routine, additional work could have been avoided.

For the M8, by undertaking a dedicated review of the rock classing along the anchor level, the likelihood of anchor failure has been significantly reduced. The purchasing of anchors was specifically suited to the projects unique geological profile.

9 ACKNOWLEDGEMENTS

The Authors would like to thank CPB Dragados Samsung Joint Venture who were responsible for the design and construction of the project and the asset owner/ operator, WestConnex | Transurban for permission to publish this paper.

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https://www.westconnex.com.au/

@WestConnex Facebook page – timeline photo dated June 2, 2020.

@WestConnex Facebook page – video post dated June 12, 2020. *With over 75% of wall panels now fully installed, the M8 is getting ready to open in the coming months.*

RE-ENGINEERING OLD FOUNDATIONS FOR A NEW STRUCTURE – GREENLAND CENTRE

Paul O'Brien[1] and David Airey[2]

[1]Senior Geotechnical Engineer
Golder Associates, NSW, Australia

[2]Professor at School of Civil Engineering
The University of Sydney, NSW, Australia

ABSTRACT

The concept of reusing existing building elements during site redevelopment within urban environments is gaining momentum. With ever-increasing pressure from modern society for engineers to focus on sustainable solutions, the industry is also beginning to recognise other benefits from maintaining, improving or re-engineering existing structures. These include fast tracking decommissioning and construction programs while at the same time saving on demolition and waste disposal costs. Along with the various challenges, such as testing, confirming the integrity of older structures and understanding foundation conditions, one of the major obstacles to effectively reusing existing building elements is the often complex re-engineering required to design and prove a solution is viable. In urban areas where space is at a premium, usually the 'bigger is better' approach prevails, and existing columns and footings will be required to support higher applied loads than they were originally designed to carry.

The Greenland Centre in Sydney's CBD is an example where the re-use of existing structural elements has been successfully adopted, by re-engineering of footing arrangements. The steel portal frame and piled footings of an old 27 storey building were retained, with the piles augmented with new pad footings to increase the footing bearing capacity to support a significantly higher 67 storey building. Finite element analysis was conducted to assess various footing arrangements to optimise the final design so that serviceability limits for the new structure could be achieved. Settlement monitoring and stress, strain measurements have been conducted through construction, with results so far shown to be within the limits predicted in the initial modelling process.

The paper discusses the general issues involved in the reuse of foundations and present results from the Greenland Centre to illustrate the advantages and challenges of reusing existing foundation elements.

1 INTRODUCTION

Many buildings around the world erected during the construction boom of the mid-1900s are nearing the end of their design life. When old buildings require demolition to make way for new, developers and engineers are required to make the decision to either demolish and rebuild an entirely new structure or to explore the opportunity to integrate the old structure or part thereof into the new design. Foundations are often considered the most feasible structural elements to reuse due to their functional value, the challenges associated with removing them (they are usually left in place) and the lack of available space in the ground below existing buildings for new footings, particularly in urban environments (Agrawal, *et.al.* 2018).

The decision to re-use foundations and develop a viable approach to achieve this requires consideration of many different factors such as existing foundation conditions, geological setting, spatial constraints and the design and loading distribution of the new structure. The first step in this process is to understand the arrangement and condition of existing footings, as this will inform whether foundation reuse is feasible. Once the condition and as-built information of the existing footings are understood, the load distribution from the design of the new building requires interrogation to explore the most efficient option for the given footing arrangement. For footing reuse scenarios, this typically consists of either supplementing existing footings with new footings, implementing ground improvement techniques to increase subgrade strength or altering the existing footings or foundation material to achieve the required support for the new structure.

2 WHY CONSIDER REUSING EXISTING FOOTINGS

In urban environments, there are several factors that can lead to advantages or requirements for footings to be reused. Inevitably, foremost in the mind of developers is money - if there is potential for cost savings by reducing material and demolition costs this can often be the biggest driver to pursuing the reuse of foundations (Valenta, *et.al.* 2007). In some cases, particularly in urban environments, the complexity around removal of old or installing new footings can be time consuming and a time saving can be made implementing reuse strategies which can ultimately also save the bottom line.

In the past few decades a key driver for reuse has been to enhance sustainability and reduce the environmental impacts associated with development. With a carefully developed foundation reuse plan, a substantial reduction in impact to the environment can be achieved through both reduction in waste and construction materials, as well as avoiding significant CO_2 emissions and the increased congestion that result from transportation of old and new materials in and out of urban spaces.

Developers, architects, engineers, consultants and construction firms have a moral imperative to design and construct environmentally and socially responsible developments, including incentives such as enhanced reputations and commercial rewards that may come with implementing this strategy.

Another important factor that can often apply when reusing or repurposing existing structures is the requirement to maintain and protect historical significance. Urban areas often contain a higher density of items of historical value, many of which are protected by State or Federal Law. As a result, removal of old building elements can be prohibited and developers are required to design and construct new structures either avoiding any protected items or integrating them into their future proposed design.

3 FOOTING REUSE METHODOLOGY

The concept of foundation reuse is not new, with numerous examples evident around the world dating back as far as 2000 years ago (Muench, 2015). Advancements in investigation and construction techniques have changed some of the methods in which foundations are tested and re-purposed, however the methodology largely remains the same. There are a number of different approaches available for design and construction of footings in previously developed urban environments. Chapman, *et.al.* (2007) suggested that footing options can be generalised into four categories listed below and illustrated in Figure 1:

- Option 1: Avoid old footings (leave in place) and build new footings
- Option 2: Remove old footings and replace with new footings
- Option 3: Direct re-use of old footings; and
- Option 4: Reuse old footings and supplement with additional footings

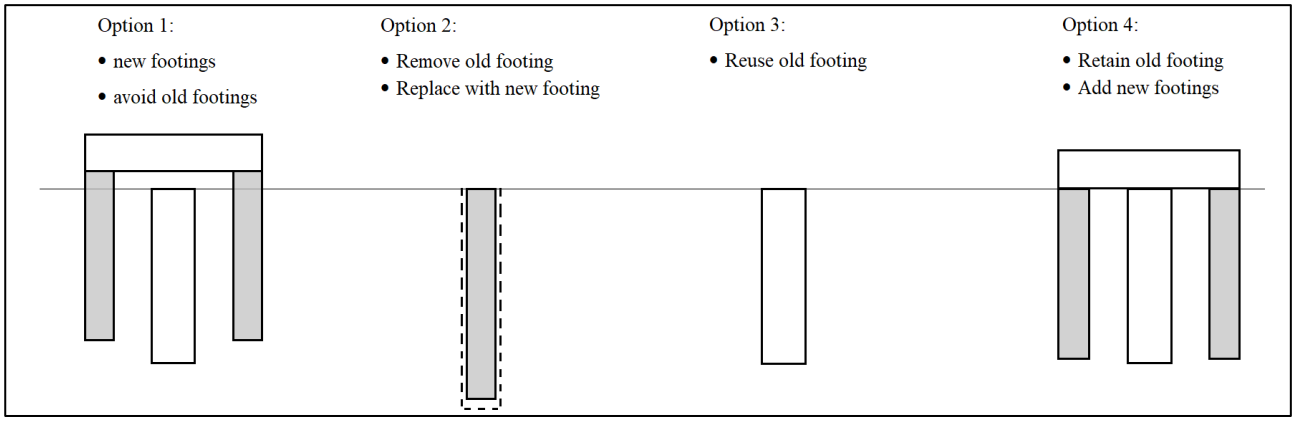


Figure 1: Foundation options for new structures in urban environments (Chapman, et. al. 2007)

Options 1 and 2 are not considered a form of footing re-use as the pre-existing footings are essentially made redundant due to avoidance or removal. Nowadays, due to urban areas being prime real-estate and developers typically opting for bigger and often taller structures, Option 3 is not considered a viable stand-alone option as pre-existing footings will rarely be fit for purpose for the new structure due to different loading distributions and typically higher loads.

A review of foundation reuse examples across the globe shows that Option 4 is the most common form of foundation re-use, particularly within urban settings, because of the flexibility it allows in integrating the old footings into the new load carrying arrangement. Other recent examples of foundation reuse suggest that there are two additional categories:

- Option 5: Direct re-use of old footings in conjunction with ground improvement; and
- Option 6: Direct re-use of old footings with structural alterations or augmentation

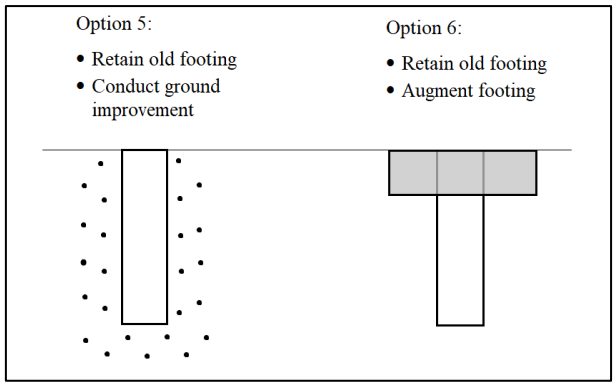


Figure 2: Alternate foundation options for new structures in urban environments

Option 5 can often be an economical approach in cases where existing footing arrangements can be effectively integrated into the new structure, however higher bearing capacity is required due to higher loads imposed by the proposed structure. Option 6 involves the altering or augmenting of existing footings to allow for either higher loads or alternate load distributions between the old and new structures.

3.1 ADDING NEW FOOTINGS TO EXISTING (OPTION 4)

Adding new footings to structures in conjunction with using existing footings is the most commonplace form of foundation reuse due to the flexibility it permits in design. It can be implemented through a variety of footing arrangements, which can be adjusted to suit the specific construction and installation limitations of developments in urban areas, namely spatial constraints associated with congestion of building elements and utilities (buried cables, pipes, sewers, stormwater drains).

Watanabe, *et.al.* (2015) provides an excellent example of where the developers successfully implemented a sophisticated design and construction integrating new bored piles between old, within one of the most congested metropolises in the world, Tokyo, Japan.

While additional new footings can be constructed in a similar manner to that of the old structure, with recent advances in footing design and construction practices there are often alternate ways to satisfy the bearing capacity of the new structure that are more optimal from both commercial and practicality of installation points of view. Often an inhibiting factor to the use of conventional footing installation or piling methods in footing reuse scenarios is the lack of available space for installation equipment, particularly low headroom restraints when structures or basements are retained. The Condor Tower redevelopment, summarised by Stewart (2012), provides a typical example of this, where although the original building was founded on a raft slab, the alternative method of micro-piling was adopted to enable installation within the restricted spatial constraints available.

3.2 REUSE OLD FOOTINGS WITH GROUND IMPROVEMENT (OPTION 5)

Ground improvement methods come in various shapes and forms, and under the right circumstances can provide a very economical solution to meet additional loading requirements for foundation reuse scenarios, particularly in the urban environment. The spatial constraints on developments in built-up areas often do not allow for mobilisation of large piling rigs, but may be suited to some ground improvement equipment that can have a smaller working footprint. Depending on the presence, arrangement and condition of existing footings and surrounding infrastructure, this can be the preferred option to enable protection and maintenance of historic or sensitive structures in close proximity to existing footings (Chepurnova, 2014).

Typically ground improvement is conducted to improve the strength and stiffness of poor soil subgrades, but it can also have some application in improvement of rock. The efficiency of improvement techniques is dependent on ground conditions, with methods such as vibro-compaction and dynamic compaction being better suited to granular soils and methods like soil stabilisation and column/displacement methods more applicable for cohesive soils. Ground improvement in rock conditions typically consists of either reinforcement methods (bolts/anchors/tendons), consolidation grouting for fractured rock or underpinning.

Many standard ground improvement methods are impractical in foundation re-use applications, due to old foundations and associated structures impeding installation. Jet grouting is the most common form of ground improvement in foundation re-use applications, where a chemical compound is injected into subgrade material to increase its strength, stiffness and bearing capacity. Historically cementitious material was used, however due to recent advances in chemical engineering other compounds such as resins and polymers are being more utilised (Sabri, *et al.* 2018).

The Kentucky International Convention Centre redevelopment is an example showing the success of ground improvement techniques in enabling footing re-use (Post, 2019). The foundation solution adopted was the injection of polyurethane grout to reinforce and strengthen sand subgrade beneath existing footings. This method was proven to provide the additional bearing capacity required for the additional loads, and enabled the existing structure to remain in place while foundation treatment was underway which would not have been possible with conventional piling rigs due to limited head-room.

Depending on subgrade conditions, ground improvement can effectively be undertaken in conjunction with installation of additional footings to enable footings re-use. This is illustrated in the example of the Condor Tower (Stewart, 2012).

3.3 AUMENTATION OF OLD FOOTINGS (OPTION 6)

Footing augmentation, although a similar concept to Option 4, is unique as the aim is to modify or enhance the existing footings, without adding any new footings, to meet the requirements of the new structure. Depending on the existing footing arrangements and the ground conditions, this would typically involve the widening of existing footings (to create a pad footing) and increase the surface area in contact with the subgrade. The outcome is to spread the load over a larger area and hence decrease the applied pressure on the subgrade.

The advantage to this solution is that there is limited additional footing construction required, generally negating the requirement for large piling equipment that is not suited to difficult access urban environments. A limitation of this option is that footing augmentation is only considered to be suited for footings founded in rock due to its higher stiffness and bearing properties, particularly for high-rise buildings or settlement sensitive structures (pad footings in sand or clay may need to be unpractically large to meet serviceability requirements of any new larger structure).

4 EXISTING FOOTING CONDITION ASSESSMENT

Prior to progressing with any option feasibility study or design for structures that involve footing reuse, a detailed assessment of the existing footings is essential. This typically involves understanding the as-built details, foundation material properties and the condition of the structural elements. It is critical to assess the reliability and capacity of existing footings in order to determine whether foundation reuse is even feasible (Laefer and Farrell, 2014), and if so what level of additional foundation support the new structure will require.

Gathering this information and interpreting it can be challenging due to foundations often being difficult to access (below utility rooms or basements, etc.) and that they are typically buried and embedded into the subgrade. Due to relatively recent advances in geotechnical investigation methods and footing inspection and testing techniques, successful collection of information and data from existing footings can be achieved in many cases, but this can require significant planning and careful execution (Colls, 2019).

4.1 AS-BUILT INFORMATION

Many buildings where developers are considering foundation re-use are likely ageing and nearing the end of their design life. In urban areas this often means they were constructed several decades ago and are likely to have gaps or inaccuracies in the as-built information due to less accurate measurement techniques and often inadequate record keeping (prior to cameras and computers being available). For these ageing structures it is generally the case that accurate records of the as-built construction are not available, possibly because the likelihood of re-use of foundation elements was not ever considered. Nowadays, there is greater emphasis on full life cycle considerations and for foundation construction a need for better record keeping which should facilitate reuse in the future. (Berglund, *et al.* 2006)

It is necessary to determine and prove the footing dimensions and embedment depths to understand the existing bearing capacity of the footings before progressing to choosing foundation reuse options. There are a variety of different ways to assess as-built footing information, and these can include:

- Borehole Drilling: Core through middle of footing to determine toe level (see Figure 3). Holes can also be drilled adjacent to footings to assess the width and extent of concrete (see Figure 4).
- PIT (Pile Integrity Testing): footing toe level can be calculated using wave technology.
- Geophysical methods: Can include downhole hydrophone testing as well as magnetometer testing (only for steel reinforced piles - to locate where steel reinforcement extends to and hence infer pile toe).

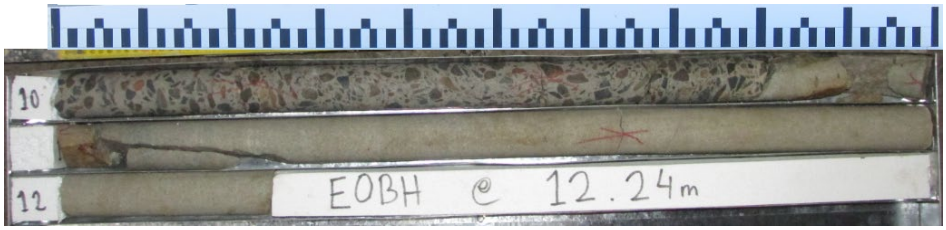


Figure 3: Borehole drilled through centre of pile – pile toe ~10.8m depth

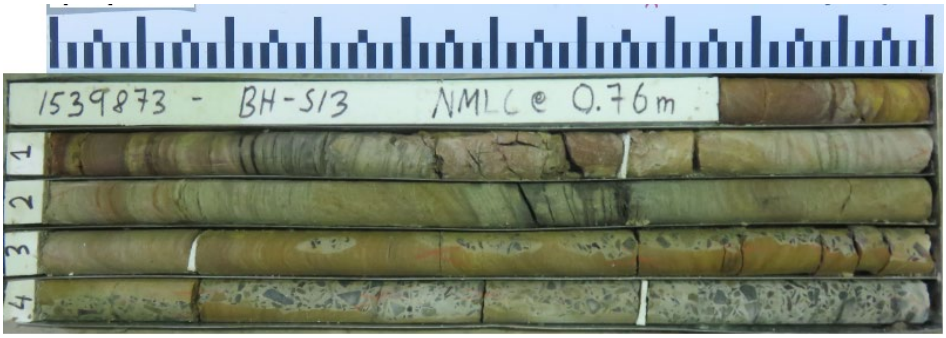


Figure 4: Borehole drilled through edge of pile/footing – evident between ~3.5m and ~4.5m depth

Although borehole drilling is typically the preferred method, often multiple methods are conducted to provide confidence in results.

4.2 CONDITION OF FOUNDATION MATERIAL

Foundation material is typically assessed using borehole information, often attained in conjunction with as-built investigations. This typically includes production of geotechnical logs to map the strata adjacent to and below existing footings. This enables formulation of site geological model, which can be utilised for footing capacity assessment and performance modelling.

Boreholes testing is one of the most common methods used to assess subgrade conditions, by assessing and collecting samples at various depths. Samples collected from investigations can be tested in the laboratory, with a wide range of testing possible, including consolidation and shear strength testing for soils and strength tests on rock such as Point Load Index and Unconfined Compressive Strength (UCS). In-situ tests can also be conducted, such as Cone Penetrometer Testing in soils to better understand the in-situ parameters of different soil strata, and pressuremeter testing in rock to enable assessment of rock mass modulus and estimate the expected settlement.

4.3 CONDITION OF STRUCTURAL ELEMENTS

The condition of the structural elements of footings such as concrete and steel is a critical step in deciding on whether the foundation is likely to perform as desired for the design life of the new structure. As the existing building being partially demolished or re-purposed is likely to be ageing, it is not reasonable to assume that the foundations are in the same condition as when they were first installed. Tsubakihara and Yamashita (2005) suggest that the three important factors requiring evaluation for existing footings are integrity, durability and bearing capacity.

Foundation integrity is a function of how structurally sound the footing element is, and whether there are any defects, cracks or deformities that are likely to impede on footing performance. A standard test to assess integrity is the Pile Integrity Test (PIT), which is a non-intrusive low strain test that utilises wave theory to determine the homogeneity of the footing material. There are other methods such as thermal and cross hole integrity testing which can supplement the standard PIT method.

There are a variety of different methods to assess footing durability. These can include carbonation testing techniques, tests to determine if concrete has been subject to chloride or sulphate attack, permeability testing, petrographic evaluation of aggregates and X-ray diffraction to assess mineralogy.

Concrete strength testing is also recommended to check against as-built information. In a similar manner to testing foundation material, UCS testing can be conducted on the concrete core samples attained from boreholes. This enables confirmation of strength for footing design purposes and performance analysis.

4.4 BEARING CAPACITY AND SERVICABILITY OF EXISTING FOOTINGS

Structure designers rely on sound foundations to ensure that the new structure will not degrade over time. Therefore checking the existing foundations will stand up to the new loads as well as assessing how much movement they may incur is critical for design.

Assessment of bearing capacity for foundations in soil is critical, to ensure amended and often increased load distributions will not lead to failure of the subgrade. In rock conditions, bearing capacity can be less important due to higher strengths and confinement. For assessment of bearing capacity, first principle footing capacity formulae are often used, in conjunction with footing design software using the information attained from the investigation of foundation and structural elements. Assessing footing serviceability and expected ground settlement beneath footings is also critical, so that structural designers can include provision within the structural elements to withstand the expected movement.

The capacity and serviceability of footings can be assessed using in-situ testing to provide information on real time footing performance. This can include high strain dynamic load testing using the Pile Driver Analyser (PDA) or undertaking a static load test to simulate expected loads for the new structure while monitoring pile head movements. Depending on the magnitude of the proposed loads, these in-situ testing techniques can be challenging to undertake in difficult access urban environments - dynamic load testing can require a high force to mobilise the pile (large hammer dropped from a height) and static load test can require a very high reaction force.

5 SUMMARY OF FOUNDATION REUSE LITERATURE

Foundation reuse has been successfully implemented in many scenarios and offers advantages in urban environments where space and access constraints apply, However, engineering innovation is required as it will generally require bespoke solutions. Modern societal expectations to reduce waste and reutilise scarce resources has compelled the industry to pursue reuse opportunities, with technological advances in testing and construction making this process more achievable. The key to determining the feasibility of adopting such methods, is to understand the existing condition of the foundation and subgrade, while interrogating the various footing integration and supplementation options available given the spatial and design constraints of integrating old with new. To illustrate some of the issues discussed above, the example of the Greenland Centre is presented where the original foundations have been reused in a development which is more than twicethe height of the building it replaced.

6 GREENLAND CENTRE – FOUNDATION REUSE PHILOSOHY

The Greenland Centre, located at the centre of Sydney's CBD, is a noteworthy example of where footings and additional structural elements were successfully reused as part of a new structure within a challenging urban environment. The reuse strategy was centred around retaining the steel skeletal frame and associated pier footings, while implementing an augmented footing approach to enable a significantly higher building to be constructed.

6.1 PROPOSED DEVELOPMENT

The project was conceived as part of the redevelopment of the former Sydney Water office tower into the proposed Greenland Centre Sydney, with the structure completed in August 2020 and the building currently undergoing fit-out and commissioning. The Greenland Centre Sydney is one of Australia's highest residential buildings. The project consisted of the partial demolition of the original 27 storey building, including the removal of the building interior, façade and above ground concrete elements. The steel frame and concrete pier footings have been retained from the old structure, and integrated into the new 66 storey, 235 m high super structure. The existing 3 basement levels were retained as part of the redevelopment, which involved the removal of all slab and column concrete material to the top of the existing basement excavation.

The main driver for the push to reuse the footing and steel structure was to maximise proposed floor space due to heritage site boundary restrictions. Other incentives included the obvious sustainability benefits attained from reusing materials, and that the developer had commercial incentive to conduct Research and Development as part of the unique reuse opportunity.

Figure 5: Pre-existing structure

Figure 6: Old steel frame retained

Figure 7: New building (at ~80%)

6.2 EXISTING FOOTING INVESTIGATIONS AND GEOLOGICAL SETTING

Prior to any significant advancement in feasibility or design of the proposed structure, a detailed understanding of the existing footings and geological conditions was required to assess subgrade and likely performance of the footings retained. Several stages of geotechnical investigations were conducted, which typically involved intrusive borehole drilling to enable core recovery for inspection and laboratory testing as well as some in-situ testing. Boreholes and core samples were used to both confirm the as-built information and condition of the existing footings, and to assess foundation conditions.

Strength testing was conducted on the collected core samples, with Point Load Strength Index (Is50) and Unconfined Compressive Strength (UCS) testing on the rock to determine rock classification and UCS testing on concrete samples to assess footing material strength. In-situ pressuremeter testing was also conducted to assess rock mass modulus and rock stiffness to inform design.

The original building as-built documentation (hand-drafted and dated early 1960s) indicates the retained steel frame is founded on an arrangement of concrete piers, embedding approximately 8m depth into the subgrade with a belled base designed to provide additional bearing capacity of the footings (see Figure 8). Boreholes were drilled through the concrete piers, targeting both the edge of the piers and the edge of the belled pile base to confirm as-built details such as pier toe levels, pier shaft width and belled toe dimensions. Generally the original as-built information was identified as slightly conservative with investigation results indicating pile toe levels were up to 2m lower and the belled section of piles extending up to 1m further from pier centre than documented.

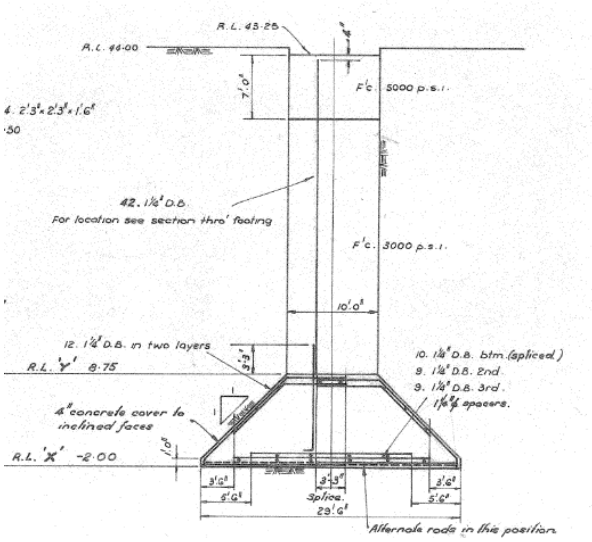
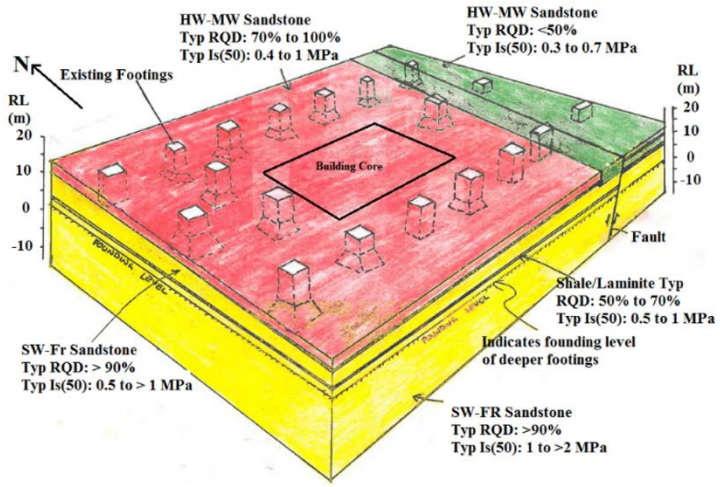


Figure 8: Typical belled pier arrangement (extracted from as-built drawings)

From information attained from the geotechnical investigations, a geological model was developed to inform footing design and aid footing performance modelling. Due to Sydney's CBD being found on relatively shallow sandstone and 3 levels of existing basements, the thin soil and weaker weathered rock profiles had been removed as part of the original building developments for the old structure. The geological model is illustrated in Figure 9 below, with adopted model parameters. The lower basement floor is predominantly found on medium to high strength sandstone (Upper Zone) and the piers are found in high to very high strength sandstone (Mid to Lower Zones). Assessment of investigation data inferred a lower strength 'Upper Zone' on the eastern edge of the site due to an inferred fault structure.



Zone	Upper	Mid	Lower
RL (m AHD)	15-10	5-10	<5
Weathering	HW-MW	MW	SW-FR
Strength (Is_{50}) MPa	0.4to1.0	0.5to>1.0	1.0to>2.0
Modulus (MPa)	1400	2400	5500
Typical RQD (%)	70-100	>90	>90

Figure 9: Geological Model and adopted model parameters

6.3 ADOPTED AUGMENTED FOOTING DESIGN

The significant increase in building height between the old and new structures resulted in an approximately 2-fold increase in applied force through the columns. With the requirement to retain footings and the steel structure, several foundation re-use options were originally considered. Installation of new footings to supplement the old was a problematic concept as spatial constraints due to existing basements and the steel structure would not allow for large piling equipment that would be required to penetrate the high strength rock. Ground improvement was not considered feasible due to the already high bearing capacity of the subgrade, limited footprint of existing footings and the high forces imposed by the new super structure.

The decision to adopt an augmented footing approach was largely influenced by two aspects. The first was spatial constraints dictating that only small to medium plant equipment with relatively low head-room had access to the lower basement. This resulted in rejection of a deep foundation solution (piles) and with shallow footings being considered the most practical approach for footing construction. The second factor that supported this approach was the relatively high subgrade strength at basement level and the efficiency at which shallow footings could transfer load to meet load bearing and serviceability requirements for the new structure.

As the developers opted for a reuse design strategy using the same column arrangement as the old, augmenting the existing pier footings with pad footings was a rational method so as to avoid any excessive load transfer systems which would ordinarily be required to integrate independent footings into the structure. The methodology was simple; encase the existing pier into a steel reinforced concrete pad footing, and integrate the steel reinforcement and concrete from the pad footing into the new structure. This included encasing the steel columns in steel reinforced concrete to help the structure support the higher loads from the additional building height and eccentricity (refer to Figure 10). Therefore, the bearing capacity borne by the original piers would be supplemented with additional capacity of the pad footings to meet the new applied loads.

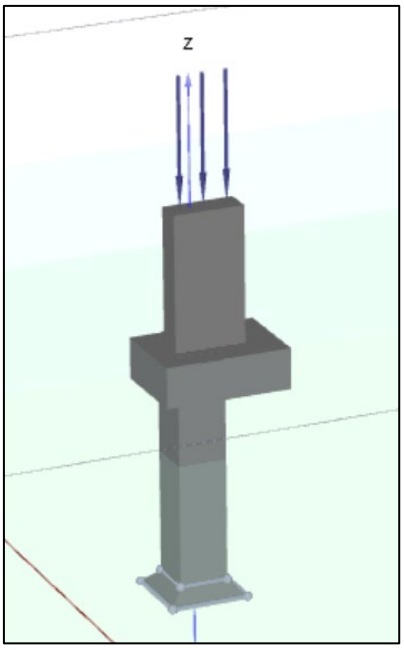


Figure 10: Example of existing belled pier augmented with pad footings

7 GREENLAND CENTRE – FOUNDATION REUSE DESIGN RATIONAL

With an understanding of the existing footing and subgrade conditions and a feasible footing reuse methodology selected in principle, the next step was to validate design. To accommodate the new loading arrangement of the proposed structure, many of the proposed columns were required to be slightly eccentric to the centre of the existing piers. Therefore consideration of the orientation, dimensions and position of augmented pad footing was necessary to withstand the variation in load distribution.

Three-dimensional finite element analysis was conducted on selected footings using the geotechnical software package PLAXIS3D. The purpose of the modelling was to:

- Assess the performance of selected existing belled piers without footing augmentation under the higher and eccentric loading from the proposed new structure
- Explore various pad footing augmentation geometry to assess performance and optimise pad constructability
- Confirm a pad foot design to augment existing belled piers that meets the settlement and serviceability requirements of the structure

7.1 MODELLING RESULTS

Two of the highest loaded columns were selected for analysis, with the maximum serviceability loads provided by the structural designer.

Parameters from the geological model developed and outlined in Section 6.2 were used as input for the model, including some assumptions of the new and old concrete using Young's Modulus values of 20 GPa and 30 GPa respectively (from as-built documentation). The new pad footing is assumed to have no structural connection to existing belled pier. The analyses were conducted considering isolated footings, as group settlement effects were considered to be minor due to the high strength and stiffness of the subgrade.

Settlement results modelled using the Ultimate Limit State (ULS) loads for the new building on the original footings without pad augmentation indicated expected settlements as high as 20 mm, which was typically more than 2 times higher than the serviceability limits stipulated by the structural designers. This confirmed that footing augmentation was required, with the next stage of the assessment involving modelling of various pad footing sizes and dimensions to optimise design.

With challenges associated with excavating high strength rock in low headroom basements, the objective was to adopt pad footings that required minimum excavation. Thin pads with a large surface area typically showed inefficiencies in limiting settlement due to large bending moments and stiffness limitations in the pad structure. Thicker pads with smaller footprints were also unsatisfactory as they only provided minor improvement to footing performance, although generally

speaking the ground conditions improve with depth. The modelling indicated the optimal pad footing dimensions had a similar width/length ratio to that of the new column, with the pad depth approximately half the length of the shorter side of the pad. Figure 11 indicates an example of the reduced settlements resulting from footing augmentation with the pad footing that meets the serviceability limits of the new structure.

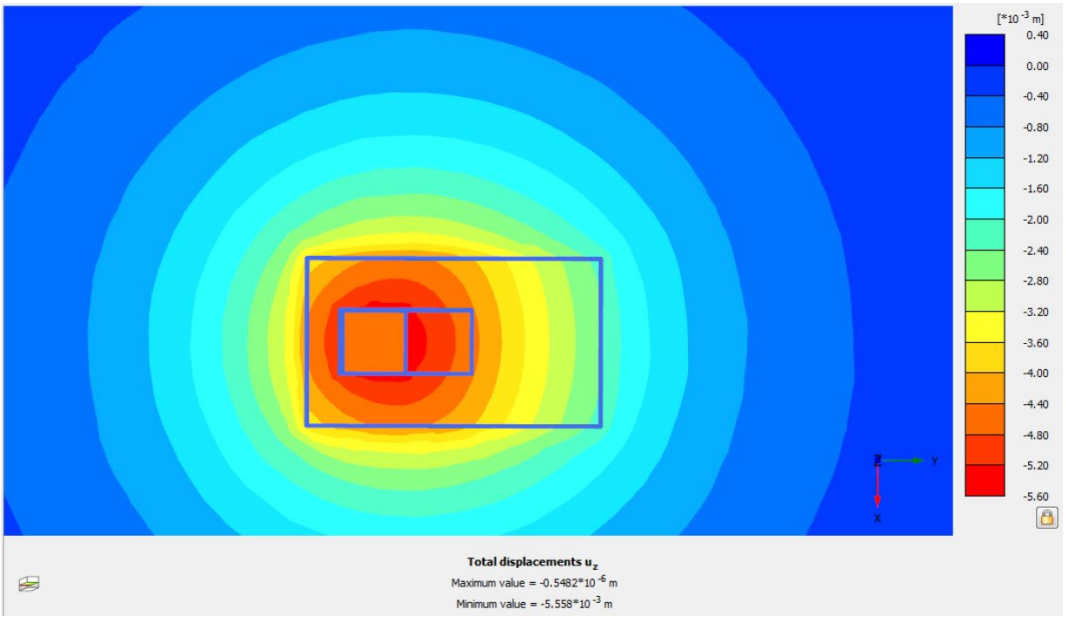


Figure 11: Example of new augmented footing under ULS loading

7.2 MONITORING DURING CONSTRUCTION AND OBSERVED PERFORMANCE

Survey targets were installed on several columns near the footing/column interface. A monthly level survey was commenced approximately halfway through the demolition phase of the project (removal of concrete elements from the structural frame). Survey results indicated that during the second half of the unloading phase footing levels 'rebounded' approximately 1 mm (extrapolated to a total of 2 mm rebound over full unloading phase) and that settlements measured during construction increased by between 5mm and 7 mm at the heavily loaded piers.

To enable comparisons between predicted settlement with observed settlement, two columns were modelled in Plaxis at 25%, 50%, 75% and 100% ULS, with maximum predicted settlement estimated. Figure 12 plots these predicted settlements for dates where completion milestones of 25%, 50%, 75% and 100% were observed, alongside actual settlement measured through level survey. Results indicate that the model over predicted settlement by about 25 percent. Further work to understand the cause of the discrepancy between prediction and reality is ongoing.

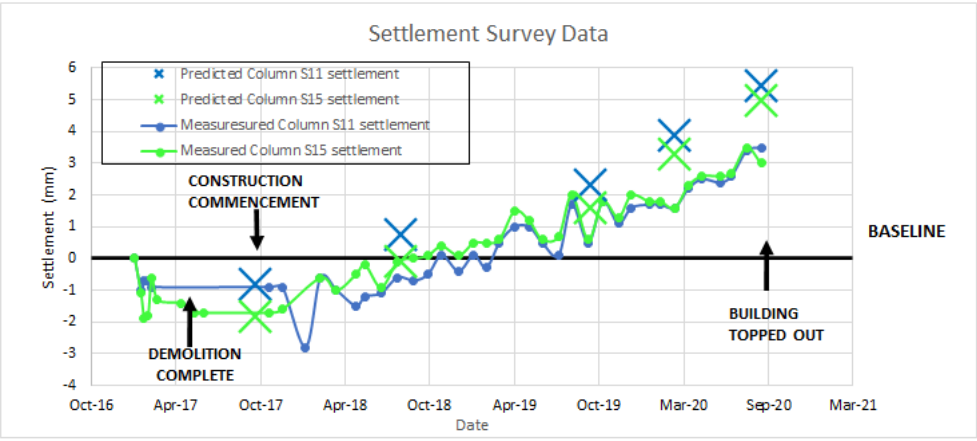


Figure 12: Predicted settlement from modelling compared with observed settlement during construction

8 CONCLUSIONS

Foundation reuse is becoming more commonplace, due to its potential for cost savings, ethical obligations on developers to design and construct sustainable solutions as well as other factors such as heritage constraints. Examples from around the world show that the philosophy and application of foundation reuse is not new, however, relatively recent technological improvements have helped implement this process. Advancement in investigation and testing techniques as well as numerical modelling can help significantly decrease the risks associated with reusing old foundations and constructing new in spatially challenging urban environments.

The Greenland Centre Sydney provides a benchmark example where the foundations and steel frame from an existing building were successfully reused and integrated into a significantly larger structure, within an urban environment. Spatial and heritage constraints were managed by implementation of an augmented footings methodology, supplementing existing pier footings with pad footings. Numerical modelling was utilised to optimise and validate the footing augmentation design process. Survey monitoring of footings through the construction process showed settlement during construction was slightly less than that predicted during modelling, indicating serviceability limits were within those stipulated by the building designers.

9 ACKNOWLEDGEMENTS

The authors would like to thank Greenland Australia to allow this paper to be published and for making it possible for the collection of data during the Greenland Centre project. The authors would also like to thank Golder Associates, Coffey and Arup for providing access to investigation data and the footing design and analysis information.

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GROUND RESPONSE DUE TO DEEP EXCAVATIONS IN SYDNEY SANDSTONE

Paul Hewitt[1] and Max Kitson[2]

[1]Technical Executive, Geotechnics
WSP Australia Pty Ltd, Sydney NSW 2000

[2]Principal Geotechnical Engineer
WSP Australia Pty Ltd, Sydney NSW 2000

ABSTRACT

A deep basement was recently constructed in Hawkesbury Sandstone for a property development on Sydney's North Shore located adjacent to critical transport infrastructure. With a depth of over 43m, the excavation is among the deepest basement excavations in the world. This paper presents the geotechnical design challenges, construction outcomes, use of the Observational Method (OM) on the excavation, discusses cost savings through innovative construction and design, and presents applications of smart technologies to instrumentation and monitoring.

When unusual displacement was observed adjacent to the transport corridor due to structural uncertainties and construction activities, the OM and verification process provided a flexible framework within which to reassess ongoing movements and effects on adjacent infrastructure to ensure construction could proceed safely. The case history demonstrates the benefit of adopting the OM for excavations to achieve savings in time and cost, and to react to unexpected movements during construction.

1 INTRODUCTION

Ground movements during deep basement excavations may potentially seriously impact adjacent infrastructure and utilities. We need to consider the displacements that deep excavations induce to assess the impact on those assets and the necessary measures to mitigate against unsatisfactory performance.

This paper describes the excavation for a property development on Sydney's North Shore. The project involved excavating over 43m deep, and is among the deepest (known) building basement excavations in the world. The excavation took place adjacent to critical road and rail transport infrastructure. The influence of excavation on existing transport infrastructure was an important design consideration. An added complexity was the potential impact that building excavation and imposed building loads could have on nearby utilities, bridges and underground structures.

2 PROJECT BACKGROUND

The project development (known as "Eighty Eight") is currently under construction at 88 Christie St, St Leonards, NSW. Figure 1 shows the development's location north of Sydney Harbour.

The new development includes two residential towers (47 storeys high) and a commercial tower (15 storeys) over a large retail precinct with 10 levels of below-ground basement up to 43 m deep. The structure for the building is founded on pad footings. Figure 2 shows an aerial perspective of the development. The 8,000m^2 basement excavation extends to the Sydney Trains boundary along the west of Lithgow Street with the Pacific Highway to the north.

2.1 ADJACENT INFRASTRUCTURE

The site boundary is surrounded by sensitive buried utilities, road and rail infrastructure, and nearby buildings for which ground movement was a key consideration (Figure 4).

An initial scheme of the development was about 14m from the earlier-planned CBD Rail Link. The earlier-planned rail transport project comprised twin tunnels connecting south of the city in the Redfern/Airport region to the north at St Leonards beneath the western edge of site and track quadruplication that added two more tracks, one on each side of the existing lines and within the existing protection corridor, as shown in the schematic diagram in Figure 3. The earlier-planned CBD rail tunnel project was abandoned, and a different route selected for CBD rail link which is now the Sydney Metro City & Southwest project (under construction).

Figure 1: Development location (JQZ, https://88byjqz.com.au/site.php)

Figure 2: Aerial view perspective of the excavation

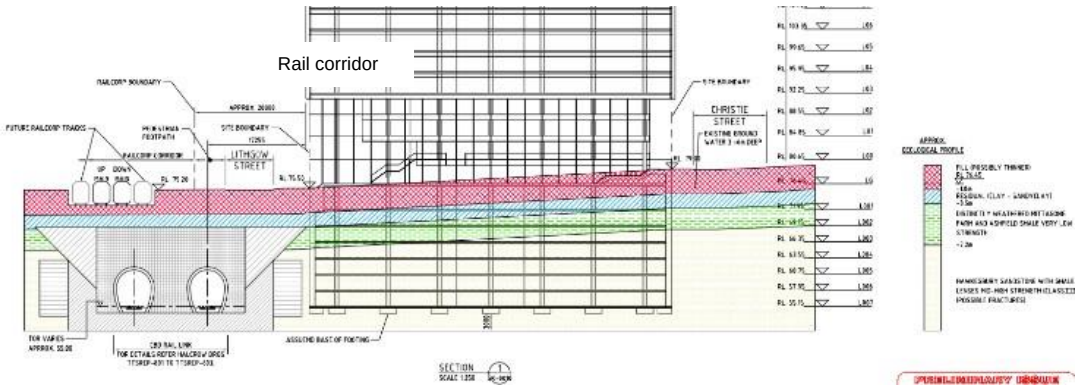

Figure 3: Schematic diagram with initial development scheme and earlier-planned CBD rail link

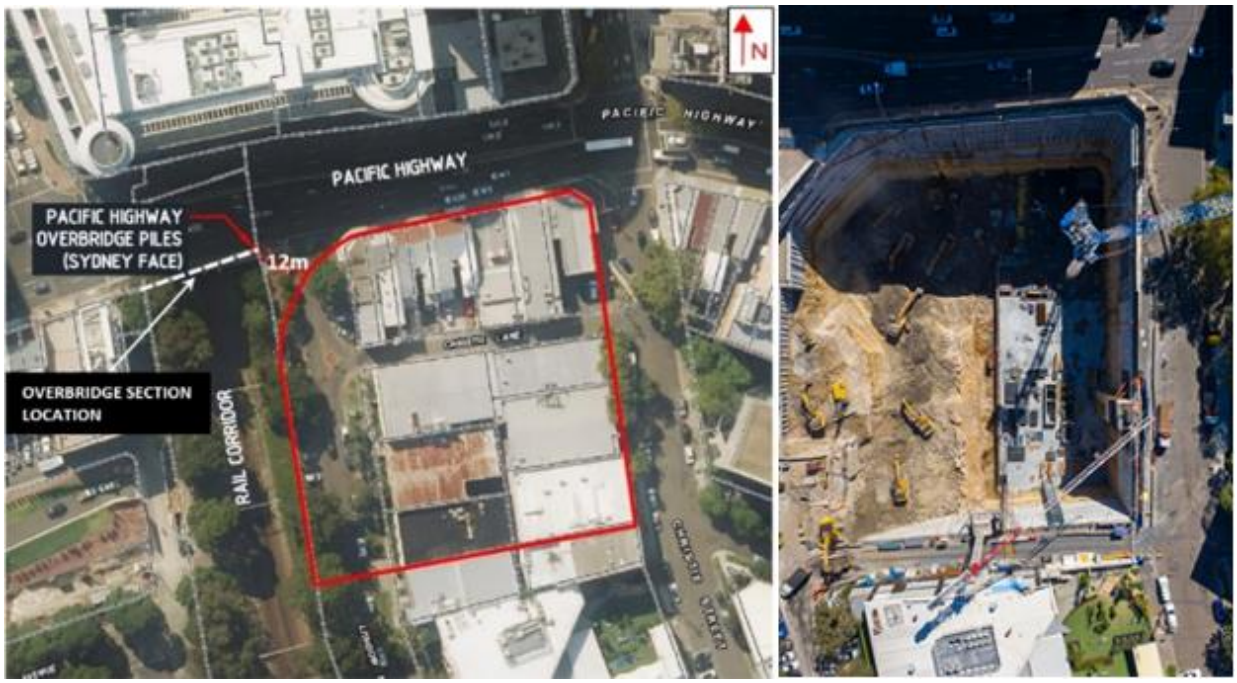

Figure 4: Location plan showing adjacent transport infrastructure and aerial view of excavation progress

3 EXCAVATION DESIGN

3.1 GEOLOGY

The site geology comprises the upper sedimentary formations of the Sydney Basin stratigraphic sequence, which consists of sub-horizontal beds of Triassic-aged rock comprising (youngest to oldest) Mittagong Formation and Hawkesbury (Sydney) Sandstone.

3.2 SUBSURFACE CONDITIONS

We assessed ground conditions from geotechnical information, including 52 borehole locations, from investigations carried out by WSP and others within the proposed development and surrounding areas. Rock strength index tests included unconfined compressive strength (UCS) and point load on recovered rock core samples. To understand and quantify the characteristics of the rock mass, we also used downhole geophysical surveys/borehole imaging to learn details of orientation, spacing, aperture, and infill characteristics of various rock mass defects including joints and bedding partings.

The development site is underlain by uncontrolled fill and residual soil, followed by Mittagong Formation and Hawkesbury Sandstone. The Mittagong Formation is characterised by interlaminated sandstone and siltstone comprising fine-to-medium grained, light grey sandstone with dark grey siltstone bands that are (generally) extremely weathered and extremely low strength. The Hawkesbury Sandstone is characterised as medium-to-coarse grained, grey, with cross bedding and medium-to-high strength. The subsurface conditions encountered are summarised in Table 1.

The rock classification adopted the Sydney classification system (Pells et al, 2019). The Sydney classification system, which was developed for foundations, is based on UCS of saturated substance (i.e. intact sandstone or shale), defect spacing and percentage of seams within a defined vertical interval of the near-horizontal bedded rock. Both strength testing and borehole imaging identified a weaker shale/ laminite band at about 40m depth close to final excavation depth. The summary of rock strength data is shown in Figure 5.

3.3 EXCAVATION RESTRICTIONS

Restrictions by the adjacent asset owners, TfNSW (ASA, 2015) and RMS (RMS, 2012) for the development included:

- Anchor systems cannot be used in the rail easement.
- Construction and operation of external developments shall not affect the stability and integrity of railway infrastructure through loading from the development and ground deformation.
- Maximum displacement of 30mm on the Pacific Highway.
- Monitoring of ground movements due to bulk excavation and monitoring of track structures required.

The selection of retention systems for the excavation to satisfy these criteria and avoid both temporary and permanent anchors within the rail corridor and road along the project boundaries had the major influence on basement geometry and the choice of retention systems.

Table 1: Subsurface conditions

Material unit	Material description	Thickness (m)	Top of unit, RL (m AHD)
Fill and residual soil	Clay and sand	1.5 - 2.0	82 - 75
Mittagong Formation	Class IV Sandstone and Shale	4.5 - 7.5	78.5 - 73.0
Hawkesbury Sandstone	Class IV Sandstone	2	74.0 - 68.5
	Class II Sandstone	3.5	72.0 - 66.5
	Class I Sandstone	19.6 - 24.0	68.5 - 43.0
	Class II interbedded Sandstone/ Shale	1.5	44.5 - 42.8
	Class I Sandstone	Not penetrated	43.0 - 41.3

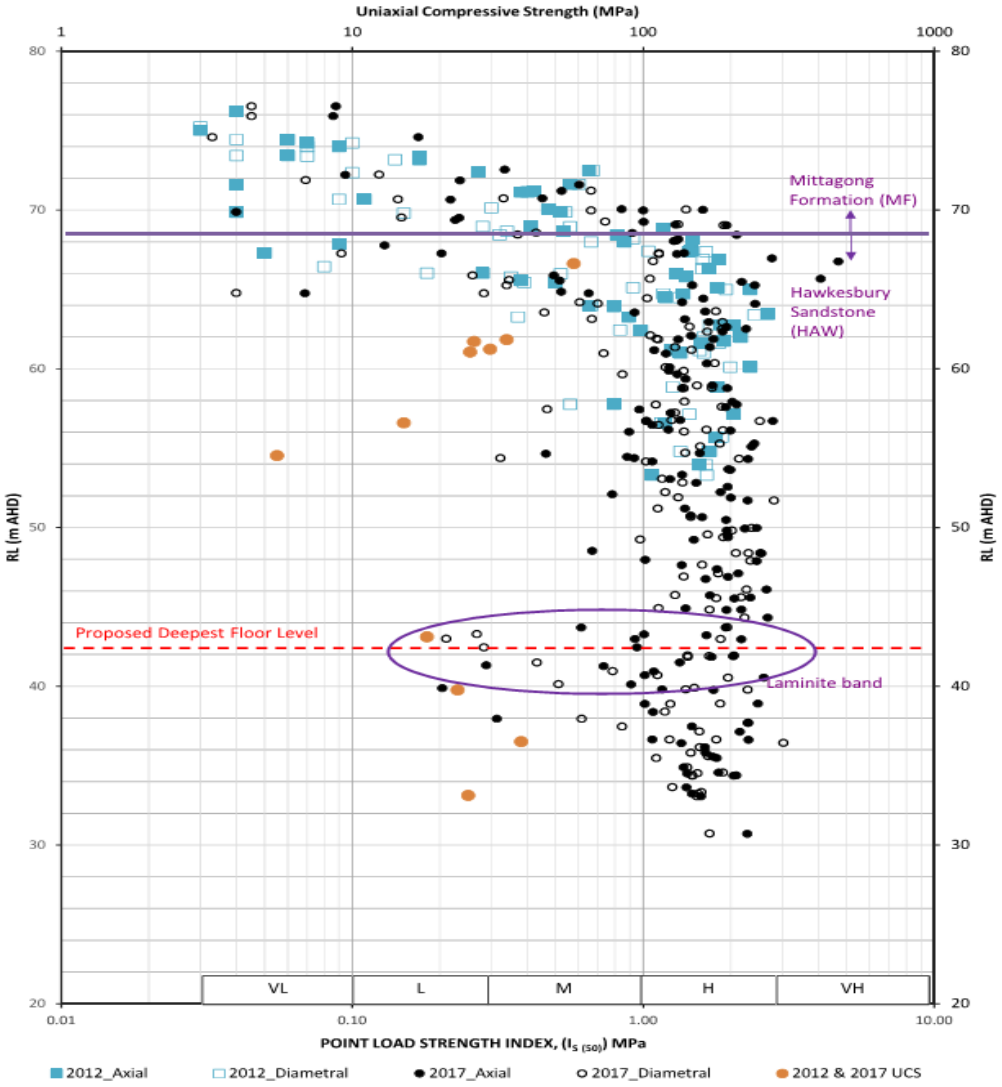

Figure 5: Rock strength data (WSP, 2018)

3.4 EXCAVATION SUPPORT

Most of the basement excavation face retains a sequence of weathered shale and sandstone. The temporary shoring system supporting the ten levels of basement along the Sydney Trains boundary (Figure 6) consisted of:

- A stepped double contiguous concrete pile wall where the front wall had short dowel anchors and shotcrete located along the middle of the west (railway) wall.
- Ground anchors within a 20m wide square buttress of rock on the south corner that was not to be excavated.
- Ground anchors for 20m length in the north corner adjacent to the Pacific Highway railway bridge. The north corner was not fully excavated which allowed angled ground anchors to be installed within the remaining triangle of rock.

Figure 6: Excavation support along Sydney Trains (west) boundary

The middle section of wall along the railway side was about 40m long, that could not be anchored. We used two piled walls to avoid installing long anchors under the railway line, where the upper soldier piled wall was a permanent wall installed on the railway boundary, comprising 750mm diameter piles spaced at 2.5m founded in min. Class II/III Sandstone up to 4.5m height. The lower contiguous piled wall was temporary, comprising 600mm diameter piles spaced at 1.5m also founded in min. Class II/III Sandstone anchored with short rock dowels 4.5m long in every pile. This lower wall provided the passive toe support to the upper wall. We adopted the following proposed excavation sequence along the Sydney Trains boundary:

- Install upper permanent piled wall on railway boundary to 4.5m depth.
- Install lower temporary piled wall 3m from Sydney Trains boundary.
- Excavation to commence. Install rock dowels in every pile and then shotcrete within piled wall.
- Geotechnical engineer/ geologist to inspect exposed rock face every 2.0m intervals of excavation depth and advise requirements for localised rock bolting and shotcrete.
- Excavate to a maximum of 500mm below the next level of rock dowels and install, then install strip drains and shotcrete. Repeat for each level of anchors until reaching high strength rock.
- Continue excavation in this manner until basement excavation level.
- Construct basement structure including all pad footings, column, walls and slabs up to the loading dock slab LB01.
- Once loading dock slab has reached design strength, continue construction.

The shoring system to the three remaining sides of the excavation comprised shotcrete concrete walls spanning to 600mm diameter soldier piles. The piles are spaced 2.5m apart with ground anchors providing temporary support. The ground control solution with a pattern of rock bolts and ground anchors was selected to support the excavation 10m to 12m below surface level.

4 IMPACT ASSESSMENT

The impact assessment involved geotechnical analyses that considered the effects of the proposed development including basement excavation and building loads, ground properties, and correction of the natural stress field based on rock mass quality.

Initially, we used two-dimensional (2D) finite element numerical modelling program RS2 for the Development Application (DA) submission that provided initial estimates of the potential impacts on the existing infrastructure.

The assumptions and limitations of a 2D model were too restrictive for the model to provide detailed estimates of ground movements and their impacts on the existing infrastructure. For detailed design, we used both 2D and 3D finite element numerical modelling programs to assess ground movement which considered the proposed excavation's geometry and provided more realistic estimates of the impact of construction on the existing infrastructure. These numerical analyses included continuum (using FLAC 2D and FLAC 3D) and discontinuum analyses (using Rocscience programs RS2 and RS3).

4.1 DESIGN PARAMETERS

The geotechnical design parameters adopted for this impact assessment were selected based on the Sydney classification system, results of the geotechnical investigation, the intact parameters, estimated GSI for each rock mass class, case history data (Figure 9), published data on sandstone and shale strength and stiffness, and the excavation depth.

The adopted geotechnical design parameters are summarised in Table 2. To address sensitivity, we reduced the values of cohesion and tensile strength for Sandstone IV and Mittagong Formation to less than 50% of design values (see values in brackets).

Table 2: Adopted design parameters

Material type	Unit weight (kN/m^3)	UCS (MPa)	Mass modulus (MPa)	GSI	Mohr-coulomb criterion		
					Tensile Strength (kPa)	Friction Angle (deg)	Cohesion (kPa)
Sandstone I	24	30	3000	75	300	55	1000
Sandstone II	24	25	2000	65	100	50	500
Sandstone IV	24	10	500 (200)	45	25 (0)	45 (35)	250 (30)
Shale II	24	15	1000	50	60	40	250

4.2 IN-SITU STRESS

The field in-situ stresses have a significant impact on both deep excavation conditions and induced ground movements in the immediate area of the excavation works, due to high in-situ lateral stresses, which can be 'locked in' within the bedrock stratum.

We incorporated adjacent deep excavations, including "The Forum" building, located north of the development (Figure 10) within the modelling as part of the impact assessment to provide a holistic approach to the major and minor stress distribution within the subsurface geological units adjacent to the excavation. We adopted the following in-situ stress relationship based on WSP's reference design for the nearby Sydney Metro City & Southwest project:

- Upper bound:
  $\sigma_{H(NS)} = 1.0MPa + 3.5\sigma_v$; $\sigma_{H(NS)} / \sigma_{h(WE)} = 1.5$ (1)
- Lower bound:
  $\sigma_v = \sigma_H = \sigma_h = 1.0$ (2)
  (stress field assumed to be lithostatic)

We used the upper-bound stresses applied to fresh, good quality sandstone and shale (Class I and II). In poorer quality rock masses, the horizontal stresses are expected to be less, and the lower bound stresses were applied. The minor horizontal stress was applied perpendicular to the excavation's eastern and western walls; the major horizontal stress is applied perpendicular to the excavation's northern and southern walls.

4.3 PREDICTED GROUND PERFORMANCE

The proposed retention system design addressed the following displacement mechanisms which have been observed to cause ground surface deformation adjacent to the excavation and could affect the railway.

- Lateral earth (soil) pressure acting on the shoring system causing it to deflect.
- Relaxation of the rock mass resulting from a reduction in lateral stress (stress relief).
- Anchor hole drilling and installation.

The proposed shoring system with soldier piles and anchors was designed to control the ground surface deformation due to lateral soil pressure in the upper parts of the proposed excavation (mechanism 1). We chose the layout and stiffness of the shoring system to minimise the ground movements and the impact on the railway tracks, and rail overbridge.

The relaxation of the rock mass due to stress relief (mechanism 2) from the deeper parts of the basement excavation will happen irrespectively of the shoring system type. The numerical assessment was calibrated against monitoring results from various deep excavations around Sydney, including monitoring results of the Embassy Residences (see Figure 2) basement excavation located on the western side of the rail corridor. Moving the boundary of the proposed development footprint and installing anchors just outside of the rail corridor will not significantly decrease ground movements of the rail corridor caused by rock stress relief, which are a result of an adjacent deep excavation.

Figure 7 shows the east-west cross sections of the development with the predicted ground movements from 2D numerical assessment due to the proposed excavation. As shown in Figure 7, the basement excavation below LB01 at RL69.5m AHD was extended to 3.1m inside the boundary to temporarily support the cantilever pile wall along the western boundary. This revised basement plan was adopted to address Sydney Trains' preference, such that no (temporary or permanent) anchors are constructed inside the rail property. The wall configuration was based on a similar cantilever post-tensioned pile wall which was successfully adopted for the Gore Hill Freeway widening at Artarmon at a location where project boundary constraints dictated that no ground anchors could be installed (Hewitt et al, 2008).

The adopted loadings for the 2D numerical assessment are more conservative than actual loading environments and the realistic loading reduces the impact of 3D effects. The predicted deformation from 3D numerical assessment indicates that the predicted deformations and stress concentrations obtained from the 2D assessment are likely to be conservative and reduces the displacement by up to 40% due to the 3D "buttressing effects".

Figure 8 shows the site's north east quadrant with the predicted 3D ground movements within the rail corridor, Pacific Highway and rail overbridge.

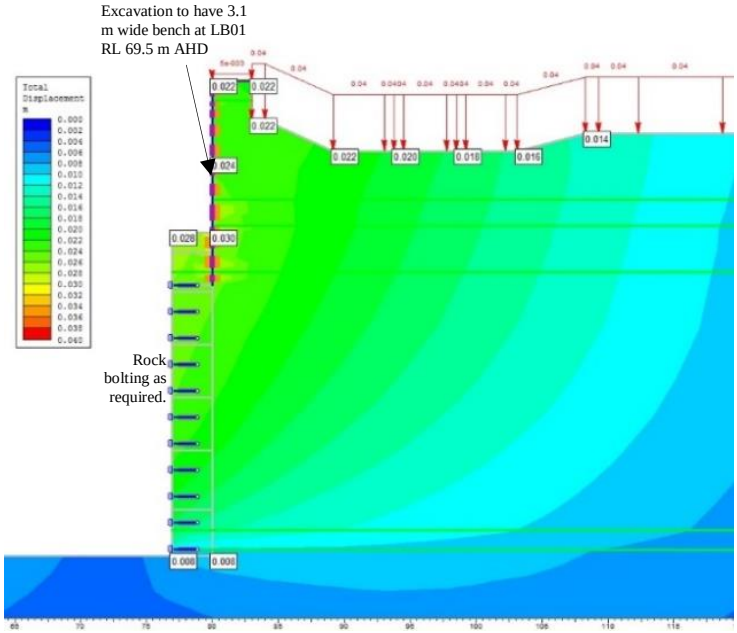

Figure 7: Predicted total ground movements of east-west section from 2D assessment

4.3.1 Deformation within Rail Corridor

The maximum predicted total vertical and horizontal deformations below the existing railway tracks after excavation completion are approximately 2mm and 6mm respectively (Figure 7). The maximum differential vertical and horizontal settlements below the existing rail track within the rail corridor due to the excavation were calculated to be less than

1mm and 2mm. Trigger levels for "Line Alarm Level 1" were 10mm for 60km/h track and 14mm for 40km/h track. Lateral movement affects the line value, with the line value determined by three track locations over 8m.

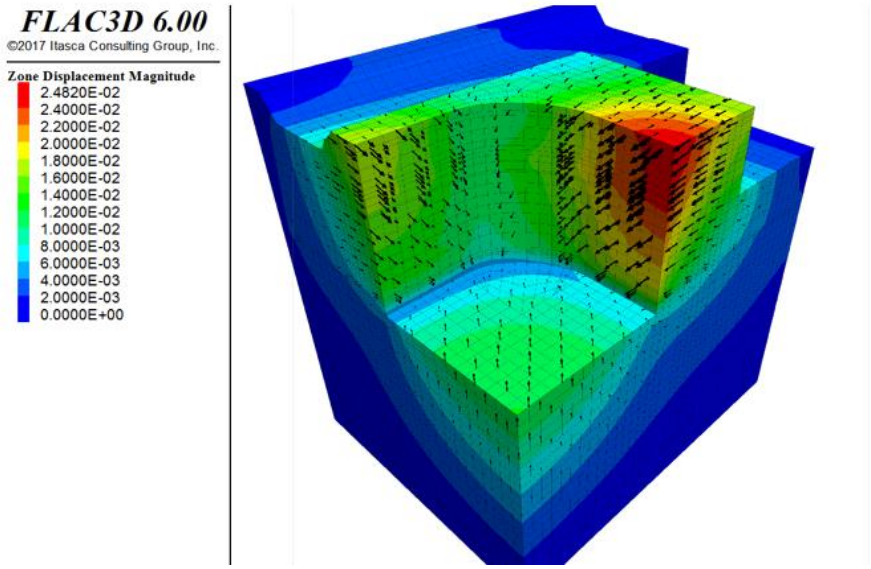

Figure 8: Predicted ground movements from 3D assessment

The estimated ground performance from the numerical assessment, a database of movements for walls using published case history data (Hewitt et al, 2008 & Wong, 2013), and monitoring data from other nearby projects indicates that lateral wall movements are generally in the 0.5mm to 2mm range per metre depth of excavation in rock. Figure 9 indicates the field performance of the Embassy development adjacent to the "Eighty Eight" development and the typical rates of movement observed in northern Sydney, in similar ground conditions. The lateral wall movement at the adjacent Embassy development was approximately 0.5mm or less per metre depth of excavation in rock. Trigger levels addressing total serviceability deflection (lateral displacement) of the wall in any one direction were 30mm adjacent to Pacific Highway.

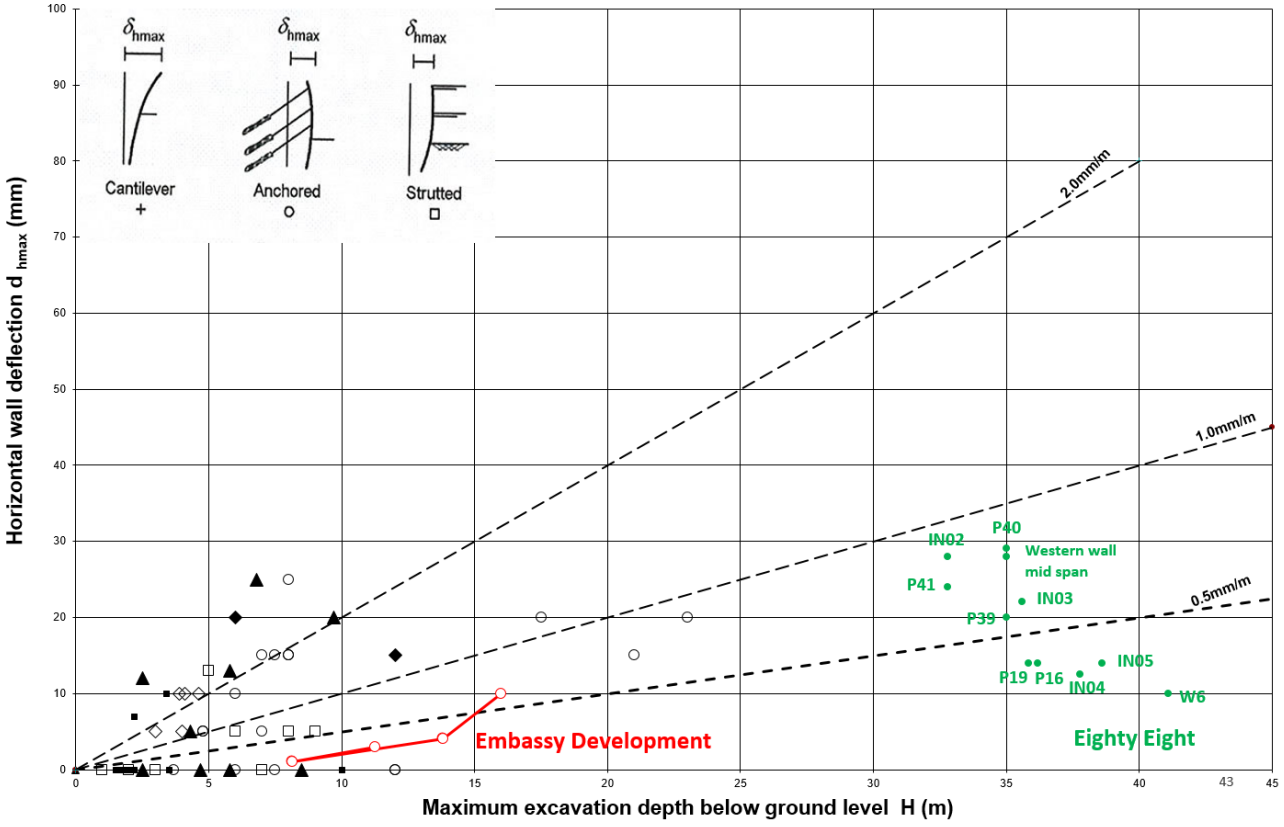

Figure 9: Field performance with different retention systems

5 CONSTRUCTION PERFORMANCE

We adopted the observational method with some contingency measures to prevent a SLS or ULS from occurring as described in CIRIA C760 (Gaba et al, 2017), including the following:

- The installation of a temporary high stiffness anchor at a high level early in the excavation sequence to control ground movements due to wall deflection, with waler and allowance for additional pre-stressing.
- Excavation start in the south-eastern corner of the site to obtain evaluation of actual wall performance, recalibration of ground and analytical models and identification of recalibrated parameters.
- Along northern boundary, excavation to proceed in 6m to 9m wide vertical panels for horizontal distance of 6m to south of Pacific Highway retaining wall to provide "berm" effect.
- Allowance for additional anchoring/cable bolts along potential sub-horizontal shear/ laminite planes identified from borehole imaging and instrumentation installation, ideally prior exposure/ displacement (adopt endoscope methods if necessary).
- Limiting temporary excavation depths along northern boundary if SLS trigger level approached.

Trigger limits were identified at key construction stages to enable appropriate and timely decisions and interventions by the project team in relation to how the site retention scheme is performing and how movements are developing compared to the recalibrated and SLS characteristic predictions.

5.1 BASEMENT EXCAVATION

The bulk excavation progress, rock condition and shoring adopted along the south and northern (Pacific Highway) boundaries are shown in Figure 10, as at July 2020.

Figure 10: Bulk excavation works - view to south and north

5.1.1 West (Railway) Wall

The investigation indicated there could be weaker shale/laminite bands at about 40m depth. (see Figure 5). We therefore planned the installation of a temporary high stiffness anchor to control ground movements due to wall deflection, depending on the observed displacement.

During the bulk excavation, the automatic inclinometer measured more pronounced horizontal sliding movement on these two shale bands (Figure 11). The sliding movement on the horizontal shale bands was caused by the release of in-situ stress within the sandstone that allowed the block of sandstone above the shale band to move more than had been predicted.

We used RS3 which is a Rocscience 3D finite element program for modelling support design and surface excavations. Initially we used a slice model to calibrate the model to match the movement in the inclinometer (Figure 12). The shale band was modelled as a ubiquitous joint model in conjunction with the Mohr-Coulomb parameters. The dip direction was set at 10 degrees into the excavation and the friction angle was reduced to 24 degrees to calibrate with the inclinometer results. Once the model was calibrated we assessed various options to reduce the movement and provide additional support to the wall, so that movements would not impact the railway.

We decided to leave a rock buttress against the wall to be excavated last, with corner propping to be installed against the southern rock buttress (Figure 13). To reduce the potential for movement on the railway we modified the anchor length of the southern 20m wide rock buttress anchors to have an adequate bond length below the shale band, and installed hydraulic corner propping (Figure 6). Additional anchors were installed on the opposite corner to counteract this force.

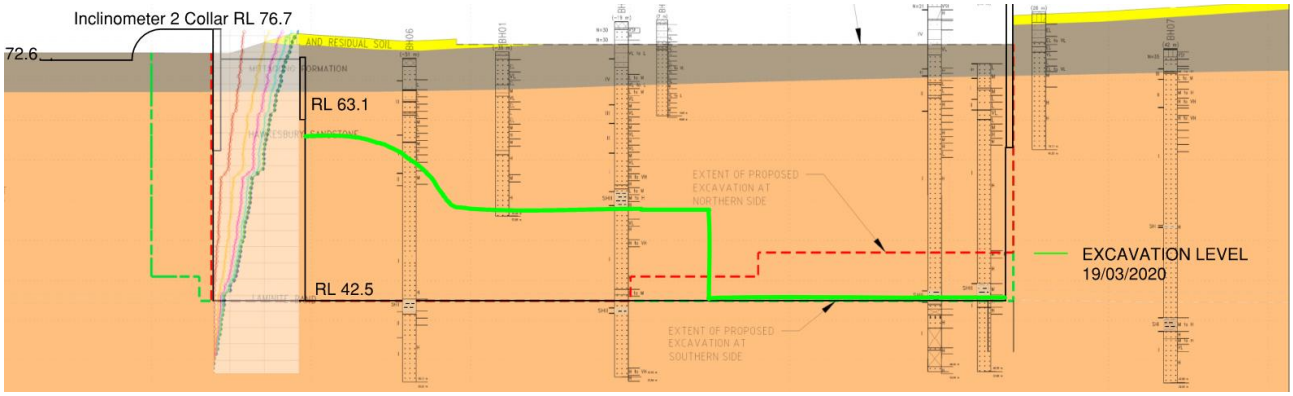

Figure 11: Bulk excavation progress from west to east showing inclinometer movement on shale bands – 14 November 2019 to 19 March 2020

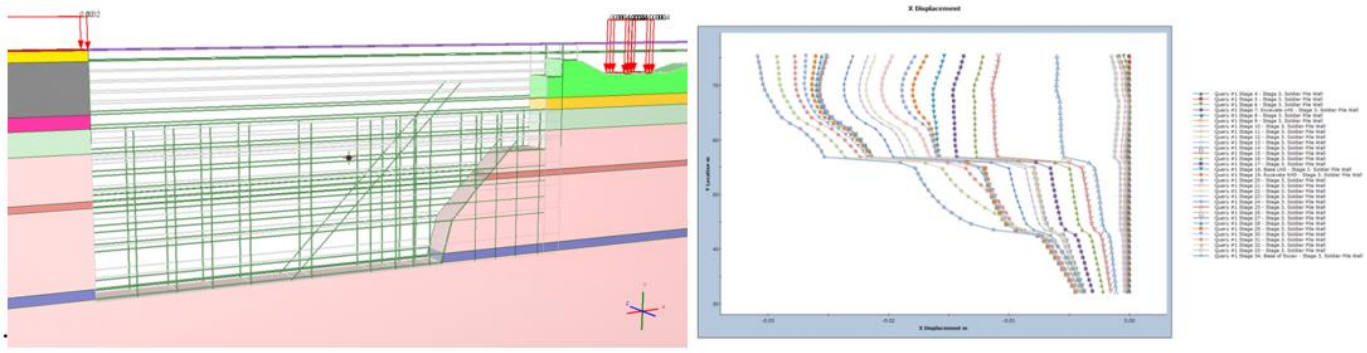

Figure 12: Calibrated 3D model with inclinometer measurements – 14 November 2019 to 19 March 2020

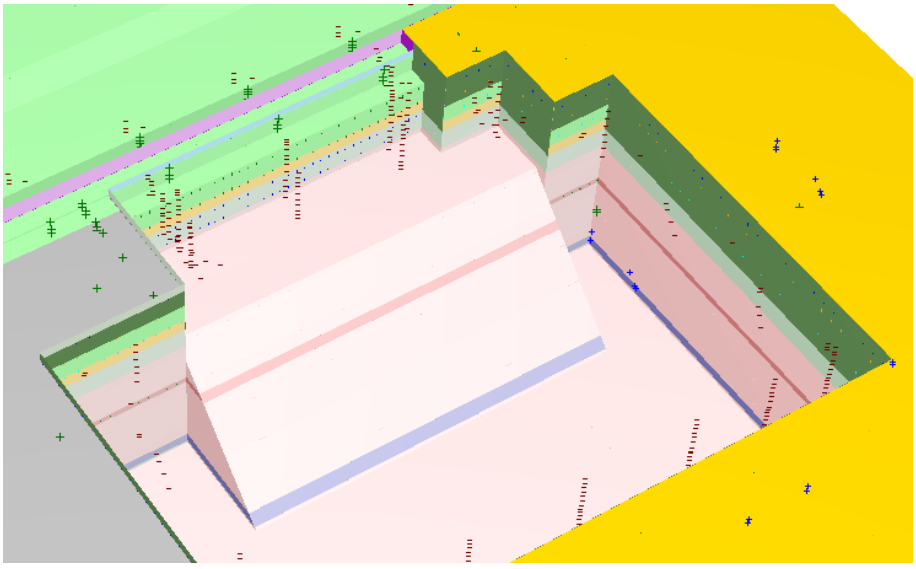

Figure 13: Staged excavation model leaving a rock buttress below the railway wall to be excavated last

5.2 INSTRUMENTATION AND MONITORING

A project team objective was to streamline data collection to maximise system and project integration, and shorten the review and decision-making process to improve construction safety. As a general trend, advances in construction monitoring are moving away from physical measurements at a limited number of points, towards widely distributed, wirelessly connected sensor networks, and to digital scanning techniques. This data allowed the excavation contractor to optimise construction processes and increase project safety performance. Monitoring for enhanced safety was also critical due to the project scale and proximity to existing infrastructure (e.g., rail bridge, underground utilities and road pavement).

Methods included surveys of deflections and rotation of the walls, laser wall scanning (Figure 14), ground settlement/heave and rail track. The analyses helped set trigger values based on the 'traffic light' principle to ensure we could anticipate and control excessive ground movements.

As part of controlling the excavation process, instrumentation and monitoring points were adopted as part of the excavation protection strategy as shown in Figure 15. The frequency of instrumentation monitoring was based on the excavation pace and was conducted in conjunction with regular visual observation. Wireless remote monitoring also provided assurance and enabled data-driven decision making. Table 3 shows the instrumentation schedule.

Walls were monitored to check the actual design performance during construction and to provide data for reviewing design and performance, and for risk management. Monitoring satisfied the designer that the geotechnical models employed in the design were representative, that predictions of the ground and rock support behaviour were accurate, and verified compliance with the design requirements. The maximum measured horizontal wall movement was 28mm at the mid-point of the west (railway) wall, which addressed restrictions by adjacent asset owners and demonstrated excellent agreement with design predictions.

Automated remote monitoring and precise manual surveying of the railway track and associated infrastructure on the North Shore Line was carried out as shown in Figure 16. Monitored assets include the Up Track North Shore Line, Pacific Highway bridge and piers, overhead wiring structures and upside crib wall. The Track Geometry was monitored in accordance with ASA Standard ESC 210 Track Geometry and Stability. In the early stages of the bulk excavation a track certifier was engaged to inspect the track as a baseline, and then subsequently inspect the track during bulk excavation.

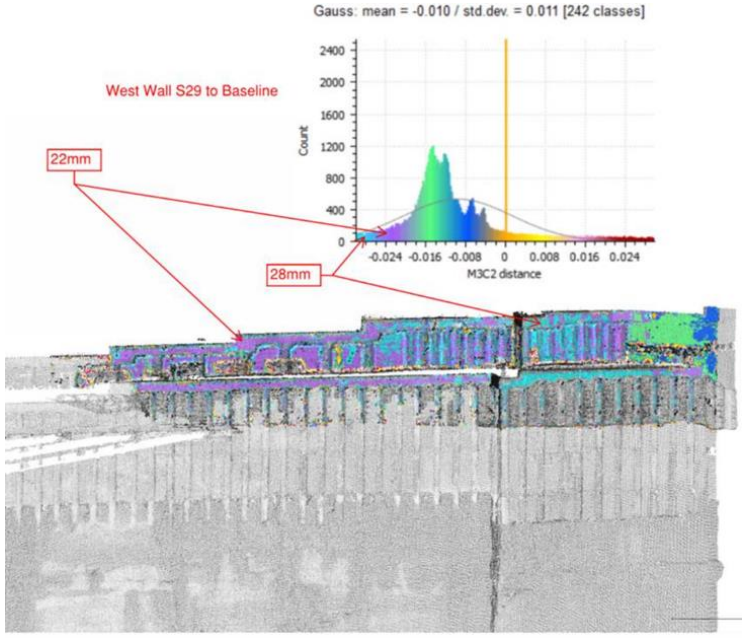

Figure 14: Laser wall scanning on west wall indicated mean displacement of 10mm and maximum 28mm

Table 3: Instrumentation schedule

Instrument	Number	Remarks
Inclinometers	5	3 manual, 2 real-time, remote GeoFlex type. See Figure 15
Displacement Points	112	Around site perimeter and buildings
Bridge displacement	3	Tiltmeters
Laser scan		Individual point accuracy of 2mm on site perimeter
Track monitoring	30	Automated remote monitoring and precise manual monitoring of rail corridor to west of site. See Figure 16
Vibration Monitoring	5	Site perimeter
Crack gauge	~30	Visual check

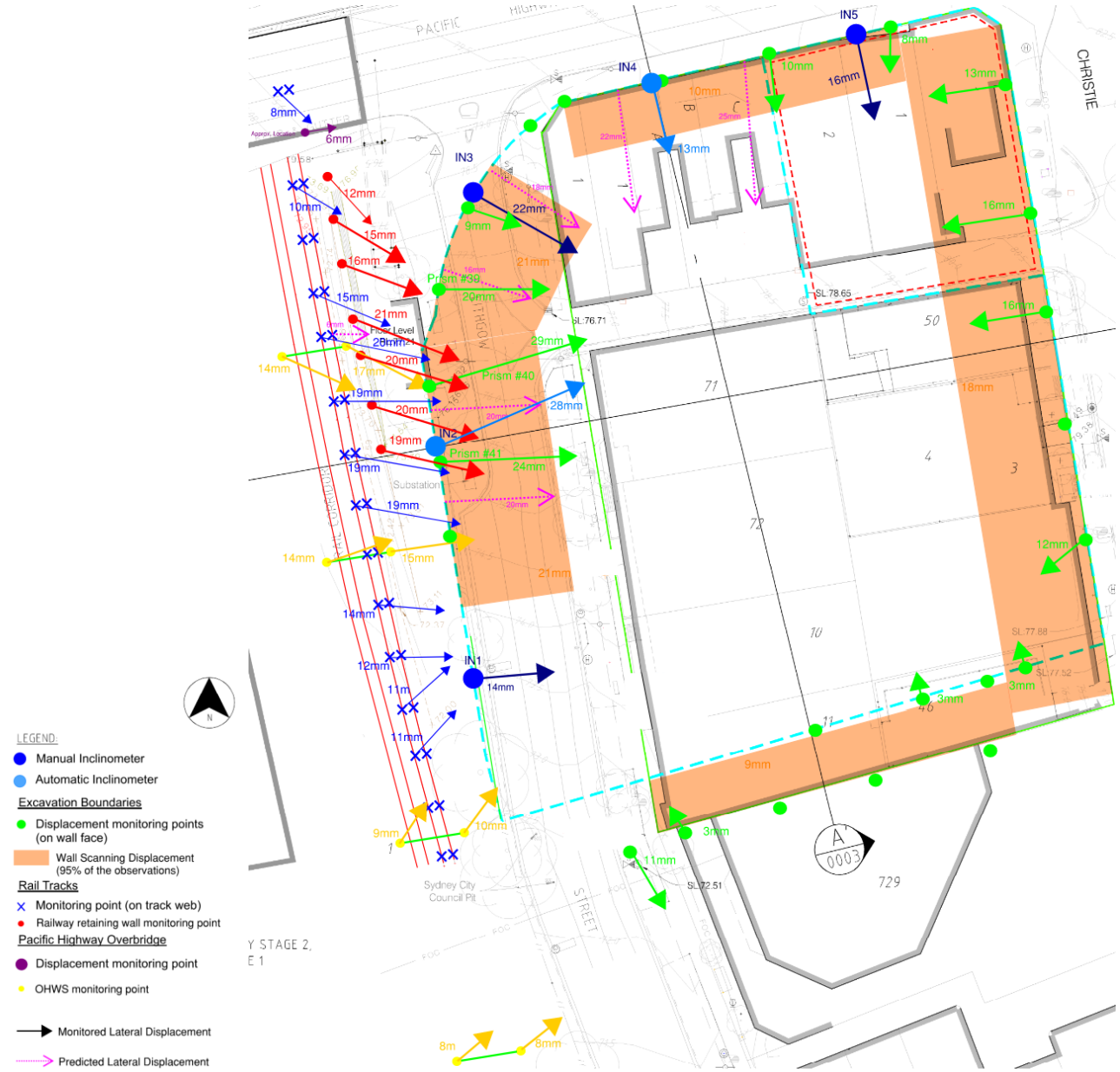

Figure 15: Instrumentation plan and measured displacement

Figure 16: View along railway showing automated remote monitoring detail

6 SUMMARY AND CONCLUSIONS

The construction of the project "Eighty Eight" required excavation to over 43 m below ground level adjacent to existing railway and highway assets. The project demonstrates the many aspects involved in respect to ground investigations, analysis, design and construction of deep basement retention systems in rock that are required to mitigate against unsatisfactory performance.

Design, excavation and construction of the site retention system incorporated several critical issues, including addressing stringent settlement and angular distortion criteria, construction safety, constructability, and the constraints of defined road and rail reserves. The design and construction process was successful and effective in addressing the concerns of all parties involved in the project.

Limited construction data available to-date indicates the displacement has been significantly reduced to achieve the performance criteria and validate the design implemented for the project. Monitoring data to date indicates the pre-construction geotechnical models and design parameters were appropriate, and that an observation-based approach allows selection of adequate retention support design to manage the risks associated with elevated stress conditions.

- Numerical analyses were carried out to assess the behaviour of the rock, site retention system, and adjacent infrastructure.
- The accuracy of this type of interaction assessment was significantly influenced by ground model parameters and rock mass properties. This emphasises the value of detailed ground investigations prior to modelling.
- The laser is a state-of-the-art development for monitoring wall movement. It offered precision and broad area coverage of wall movements through rain, dust and smoke (which occurred due to Sydney bushfires in January 2020).
- The real-time display of the movement of basement walls allowed continuous management of deformation during excavation, with the remote monitoring inclinometer system providing early detection of horizontal sliding along shale bands which enabled data-driven decision making.
- The real-time, automated remote monitoring and precise manual surveying of the railway track geometry and engaging a Track Certifier early in the bulk excavation works assisted in addressing impacts on track geometry.
- The maximum measured horizontal wall movement was 28mm at the mid-point of the west (railway) wall.

7 ACKNOWLEDGEMENTS

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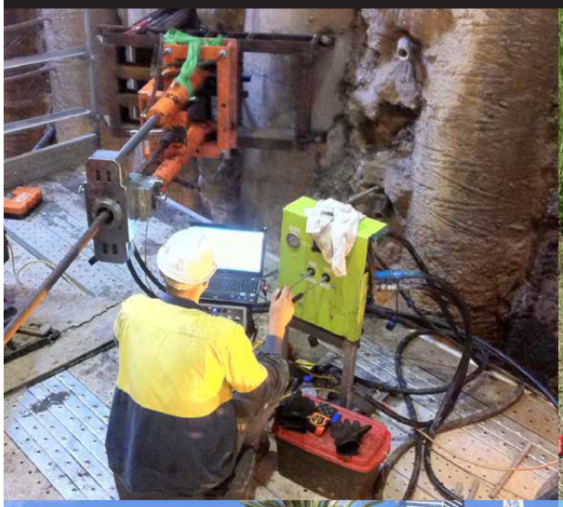

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