Lecture Notes in Civil Engineering

Hadi Khabbaz
Cholachat Rujikiatkamjorn
Ali Parsa *Editors*

Geotechnical Lessons Learnt—Building and Transport Infrastructure Projects

Proceedings of the 2021 AGS Sydney Annual Symposium

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Preface

This document contains the accepted papers submitted and peer reviewed for the 25th annual symposium organised by the Sydney Chapter of the Australian Geomechanics Society (AGS). During the last five decades the AGS has stood as a beacon of the wider geotechnical community. Due to the continuation COVID-19 pandemic, it is the second time that the virtual presentations were delivered 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 12 November 2021, is to showcase state-of-the-art practices, new research findings and case histories that demonstrate geotechnical advances and challenges in Building and Transport Infrastructure. The organising committee invites papers on geotechnical aspects of smart solutions and improvements in geotechnical approaches for transport infrastructure projects, advances in tunnel design and construction and geotechnical challenges in design and construction—case histories and lessons learnt.

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, Kourosh Kianfar, Mehdi Tamadon, Saman Zargarbashi.

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

Ultimo, Australia — Hadi Khabbaz
Sydney, Australia — Cholachat Rujikiatkamjorn
Sydney, Australia — Ali Parsa

Australian Geomechanics Society

Geomechanics is the application of engineering principles to the earth sciences to improve continually the accuracy, efficiency, cost-effectiveness and safety of construction projects both above and below ground, including the recovery of the earth's mineral resources. It remains an imprecise discipline due to the infinite variety of conditions in the earth's crust but correspondingly offers a fascinating and rewarding field of research and practice.

The Australian Geomechanics Society was founded in 1970. Its origins lie in the National Committee of Soil Mechanics of the Institution of Engineers, Australia, established in 1953 and the call for a corresponding society in rock mechanics. In 1973 the society was expanded to include the third discipline of engineering geology and has remained substantially unchanged since that date.

The society is affiliated to the International society of Soil Mechanics and Geotechnical Engineering (ISSMGE), the International Society for Rock mechanics (ISRM) and the International Association for Engineering Geology and the Environment (IAEG).

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About the Editors

Prof. Hadi Khabbaz has conducted original research in ground modification, unsaturated soil mechanics, tunnelling and transport geotechnics. He has been involved in design of civil infrastructure and soil behaviour modelling research for over 20 years. His research has been focused on the theoretical and numerical analysis of problematic soils, underground spaces, granular particles and unsaturated porous media with strong applications to real life engineering problems. He has made several significant contributions in the field of unsaturated soil mechanics and ground improvement techniques with fundamental and pioneering nature. His early work on unsaturated soil mechanics has been increasingly cited in literature and considered as a significant contribution to the field. He has sound numerical, experimental and programming skills. His research output and publications on geotechnical aspects of rail track foundations overlying soft soil is also extensive. To date, 17 research students completed their theses under his supervision and he has 16 completions as a cosupervisor. At present, he is the principal supervisor of 5 research students. He is an active assessor of the ARC and the Office for Learning and Teaching (OLT) grants. He is a technical reviewer of many international journals and several research organisations.

Dr. Cholachat Rujikiatkamjorn is a Professor at the Transport Research Centre, School of Civil, and Environmental Engineering, Faculty of Engineeering and Information Sciences, University of Technology Sydney, Australia. His key areas of expertise include ground improvement for transport infrastructure and soft soil engineering. He has published over 270 articles in international journals and conferences. While maintaining a strong focus on quality teaching, to date, he has secured over $2 million in research funding, mostly from external sources. He is currently a CI of one ARC-DP project, 3 ARC-LP projects, an ITTC Rail and a CRC-Rail project. He is currently the supervisor/co-supervisor of 10 HDR students and 4 Research Associates. His research achievements over the years and involvement in Port of Brisbane reclamation were the main driving force in the introduction of new Australian Standards (AS8700) on Vertical Drains in Soft Soils.

Dr. Ali Parsa is an Associate Geotechnical Engineer with more than 18 years of experience. After 6 years of industry experience, he joined University of Technology Sydney (UTS) in 2009 & worked on an industry (i.e. Menard) sponsored project incorporated different aspects of soft soil improvement. After receiving his Ph.D. in 2014, Dr. Parsa re-joined industry and since then have worked in infrastructure and construction sectors and had involvement in delivery of more than 150 projects including major infrastructure projects. In last several years, he has collaborated with leading multidisciplinary companies, many engineers from other design disciplines, variety of clients/authorities and major joint ventures in Australia. He has also been involved with several R&D programs and development of new approaches to deliver efficient & robust designs for challenging geotechnical problems. Dr. Parsa is the elected Chair of AGS Sydney Chapter. He is coordinator of AGS Sydney Chapter annual Symposium and editor of proceedings since 2017. Ali also represents Australia in two technical committees of International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE), TC103 (Numerical Methods) and TC207 (Soil-Structure Interaction).

Smart Solutions in a Circular Economy for Advancing Railroad Design and Construction Using Recycled Materials

Buddhima Indraratna, Yujie Qi, Cholachat Rujikiatkamjorn, Miriam Tawk, Fatima Mehmood, Sinniah K. Navaratnarajah, Tim Neville, and Jim Grant

Abstract Ballasted rail tracks are the most common type of transportation infrastructure. However, ballast progressively degrades under dynamic and impact loads. The degree of degradation will be accelerated due to the increasing demand for elevated speeds of passenger trains and heavier axle loads for freight trains. It is, therefore, necessary to develop novel and cost-effective technologies to enhance the

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longevity and performance of tracks through amended design and construction. Over the past two decades, a number of studies have been conducted by the researchers of Transport Research Centre (TRC) at the University of Technology Sydney (UTS) to investigate the ability of recycled rubber mats/pads, as well as waste tyre cells and granulated rubber to improve the stability of track substructure materials including ballast and sub-ballast layers. This paper presents an overview of these novel methods and materials based on comprehensive laboratory tests using iconic testing facilities. Test results from comprehensive laboratory tests and field studies have indicated that the use of energy-absorbing rubber inclusions can substantially improve overall track stability. The findings reflect the following: (i) the inclusion of recycled-rubber based synthetic energy absorbing layers (SEAL: SFS-CW-RC mixture) significantly attenuates the magnitude of the dynamic load with depth and ballast breakage, (ii) an alternative solution by using CW-RC mixtures as capping layer is also introduced in this study, and the compressibility of the rubber is captured by cyclic compression triaxial tests, (iii) the installation of under ballast mats (UBM) significantly reduces permanent vertical and lateral deformation of ballast as well as ballast degradation, (iv) waste tyre cells infilled with granular aggregates effectively increase the stiffness and bearing capacity of the capping layer and help mitigate track displacement, and (v) field tests indicate geogrids and shockmats are efficient methods to reduce the track displacement and ballast degradation. These research outcomes provide promising approaches to transform traditional track design practices to cater for future high-axle rolling stock carrying heavier loads.

Keywords Railway · Recycled rubber · Large-scale laboratory tests · Field tests

1 Introduction

With the fast-growing population and productivity of the agriculture and mining sectors, Australia is facing a soaring demand for transportation. Especially after the successful bid of the 2032 Olympics, more reliable and sustainable transport infrastructure is under urgent need and will be a way to pave Australia future prosperity. The ballasted rail network (over 36,000 km) as one of the main transport infrastructures in Australia needs to improve significantly to cater to faster and heavier trains [1]. However, ballast deterioration due to ballast breakage, extensive settlement and lateral dilation is the main issue confronted by railway industries, which causes frequent and costly track maintenance. In NSW alone, a total of $34 billion was spent last year on key maintenance activities such as ballast replacement and cleaning etc. [2]. Therefore, researchers and practising engineers are seeking smart and cost-effective solutions such as incorporating resilient geo-inclusions to improve track performance. Among them, the use of recycled rubber in railways has become increasingly popular in recent years.

Recycled rubber, mainly derived from waste tyres, can be easily obtained due to the large stockpiling worldwide. In Australia alone, over 56 million EPU (equivalent passenger unit) waste tyres are produced every year, while around 50% is dumped in useful lands, which presents fire hazards and harbours disease vectors like mosquitos and vermin [3–5]. Since the agreement of the Council of Australian Government (COAG) with the implementation of the export ban on the whole waste tyre overseas in 2019, the Australian local government is under severe pressure to find innovative solutions to recycle the waste tyres [6, 7]. Except for the application in playgrounds, car parks, horse racing tracks, sports grounds, recycling waste tyres in large-scale transport infrastructure like railways is another promising alternative method [8–10]. Moreover, recycled rubber materials (e.g. granulated rubber, rubber mats/pads, waste tyre cells) have high damping properties and energy absorbing properties, which are advantages over traditional materials to be used in dynamic loading projects (e.g. railways, highways, airport runways). It has been found that mixing rubber crumbs with other materials such as sand or other waste aggregates can significantly increase the ductility and energy absorbing capacity of the mixtures, albeit a bit reduction in the overall shear strength [11–17]. Indraratna et al. [12] introduced rubber crumbs into the waste blends of steel furnace slag and coal wash and developed a synthetic energy absorbing layer (SEAL) for railway subballast, and comprehensive laboratory studies were conducted on the SEAL mixtures and found that the addition of rubber could efficiently reduce the particle breakage of coal wash and the swelling potential of steel furnace slag, hence forming a balanced energy-absorbing reservoir for rail tracks [18]. Granulated rubber from waste tyres has also been mixed with ballast materials to help increase the efficiency of energy dissipation and reduce the ballast breakage [19–21]. Jayasuriya et al. [22] investigated the performance of the track by installing the under sleeper pads made from waste tyres for stiff subgrade conditions (i.e. situations to simulate bridges or concrete deck) and found that the ballast degradation could be reduced by more than 50%. Other studies by Johansson et al. [23], Navaratnarajah et al. [24], and Ngamkhanong and Kaewunruen [25] reported that the inclusion of under sleeper pads could efficiently reduce the track vibration (at the frequency below 250 Hz) and ballast settlement. Large-scale laboratory tests have been conducted by Navaratnarajah and Indraratna [26] and confirmed that, by installing under ballast mats, an overall ballast breakage can be reduced by approximately 35–45% and the permanent lateral strain can be reduced by around 10%. Moreover, the installation of recycled rubber shock mats under ballast could efficiently reduce the plastic strains induced by impact loads around 50% reported by Indraratna et al. [27] through field investigation. Another novel application of recycled rubber is using waste tyre cells to reinforce the capping layer proposed by Indraratna et al. [28], and both laboratory testing and numerical studies have shown that by using tyre cells with infilled recycled ballast materials as capping layer could help to increase the stiffness of the capping layer about 50% and reduce the ballast deformation and degradation, and also reduce the stress transmitted to the subgrade [29].

This paper aims to review the innovative applications of using recycled rubber materials in railways conducted by the geotechnical team of Transport Research Centre at the University of Technology Sydney in recent years including (i) replacing traditional subballast materials with a synthetic energy absorbing layer by mixing recycled rubber crumb (RC) with mining by-products like steel furnace slag (SFS) and coal wash (CW); (ii) installing under ballast mats (UBM) to mitigate ballast degradation and deformation; (iii) reinforcing capping layer by recycled tyre cell to increase the track stiffness and reduce track displacement. Theoretical concepts of these applications are introduced and comprehensive laboratory tests of the track specimen incorporating these recycled rubber products using iconic testing facilities are described and discussed, and in the end, field tests and ongoing track design are also elaborated.

2 Granular Mixtures for Railway Subballast/Capping Layer

2.1 Application Concept

2.1.1 A Synthetic Energy Absorbing Layer (SEAL) by Mixing SFS, CW and RC

The SEAL composite contains three waste materials, namely, steel furnace slag (SFS), coal wash (CW) and rubber crumbs (RC). SFS and CW are the common mining by-products from steel manufactory and coal mining in Australia. Because of the expansive feature of SFS and the particle breakage of CW, SFS and CW need special treatments or mix with other materials to be applied in civil engineering. A successful example was using the CW-SFS blends for Port Kambla (NSW, Australia) reclamation [30]. A proper amount of SFS (30–45% by weight) in the CW-SFS blends could satisfy the reclamation specifications with negligible swelling (<3%) and particle breakage of CW (<10%) [30, 31], and it has been reported that the land reclaimed using the blends is functioning well [13]. To extend the application of CW-SFS blends to dynamic loading projects (i.e. railways), rubber crumbs were introduced in the mixture to further reduce the swelling potential of SFS and the breakage of CW, and at the same time increase the energy absorbing capacity of the mixture to help dissipate the continuous energy input from the moving train. Due to the low shear strength of pure rubber, the percentage of SFS in the mixture was increased as SFS was the stiffest material among the three. Therefore, a blending ratio of 7:3 (by weight) was selected for SFS:CW in the mixture and has been proved to maintain sufficient strength for the SEAL mixture (i.e. SFS-CW-RC mixture) using standard compaction method (Fig. 1) to be used for subballast [12]. The purpose of the SEAL is to provide an energy absorbing reservoir for the rail track and reduce the energy go to the ballast and subgrade layer (Fig. 1), hence reduce stress distributed to other layers and the ballast degradation. However, as the addition of RC will influence the geotechnical property significantly, it is imperative to investigate the performance of the track incorporating SEAL having different rubber contents under dynamic loading.

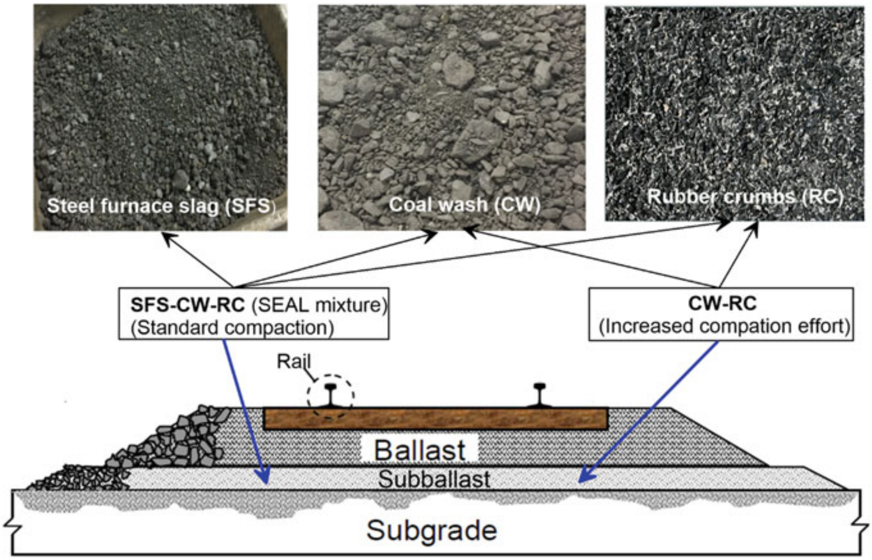


Fig. 1 Using the mixtures of SFS, CW and RC to replace traditional subballast materials

2.1.2 Recycled Rubber Crumbs-Coal Wash Mixture for Capping Layer

In light of the research on SFS-CW-RC (SEAL) mixture by Indraratna et al. [12], another method was proposed to use CW and RC only for subballast/capping layer to avoid the risk of the swelling potential caused by SFS. To compensate for the absence of the stiffest SFS in the mixture, the CW-RC mixture will be compacted using increased compaction effort (Fig. 1). Unlike traditional rigid materials, rubber crumbs are compressible and highly deformable. In previous studies considering rubber-aggregate mixtures, rubber was considered incompressible [32–34]. Ideally, when there is no confinement, rubber particles have a Poisson's ratio of 0.5 [35, 36]. This means that, when unconfined, rubber does not contribute to the volumetric strain for any applied load. However, using hydrostatic compression tests, Plachy et al. [37] showed that the compressibility of rubber depends on pressure. In this context, the study stressed the importance of developing more realistic mechanical models to capture the compressibility of rubber under loads.

For traditional materials, the volumetric strain is calculated as the ratio of the change in the total volume (ΔV) and the initial total volume (V_0). Given that rigid aggregates are relatively incompressible, any change in the total volume is due to a change in the volume of voids inside the sample only (ΔV_v). In traditional triaxial testing, the change in the volume of voids is measured by monitoring the volume of water inside the saturated sample (ΔV_w) and the volumetric strain (ε_v^*) is calculated as:

$$\varepsilon_v^* = \frac{\Delta V}{V_0} = \frac{\Delta V_v}{V_0} = \frac{\Delta V_w}{V_0} \tag{1}$$

When compressible aggregates like RC are added to the mixture, the volumetric strain calculated from the above equation represents the change in the volume of voids but does not capture any change in the volume of rubber particles (ΔV_{RC}). To measure the total volumetric strain expressed as:

$$\varepsilon_v = \frac{\Delta V}{V_0} = \frac{\Delta V_v + \Delta V_{RC}}{V_0} \tag{2}$$

the Hall effect sensors were used to measure the axial strain (ε_1) and the radial strain (ε_3) and calculate the total volumetric strain from the equation:

$$\varepsilon_v = \varepsilon_1 + 2\varepsilon_3 \tag{3}$$

The traditional void ratio expressed as the ratio of the volume of voids and the volume of solids is usually calculated from the void volumetric strain, ε_v^*, and the initial void ratio, e_0, as:

$$e = e_0 + \varepsilon_v^*(1 + e_0) \tag{4}$$

The above equation assumes that the volume of solids is a constant. Having measured the total volumetric strain, ε_v, the actual void ratio e capturing the change in the volume of voids and the change in the volume of solids (i.e., rubber particles) can then be calculated from the equation [38]:

$$e = \frac{\left(\frac{e_0}{1+e_0}\right) + \varepsilon_v^*}{(1 + \varepsilon_v) - \left[\left(\frac{e_0}{1+e_0}\right) + \varepsilon_v^*\right]} \tag{5}$$

2.2 *Testing Program*

2.2.1 Large-Scale Physical Model for Tracks with SEAL

To verify this concept, large-scale cubical triaxial tests using the track process simulation apparatus (TPSA) were carried out to examine the deformation and ballast degradation of a track specimen incorporating SEAL with different rubber contents (0, 10, 20, 30 and 40% by mass). The size of the triaxial chamber was 800 mm long, 600 mm wide and 600 mm in depth. The PSPTA triaxial chamber correctly simulates realistic stress and boundary conditions. In a real rail track, the lateral movement of the ballast is not fully restrained, particularly in the direction parallel to sleepers.

Unlike traditional test rigs with a fixed rigid boundary, the TPSA with movable vertical walls can allow the lateral strains to occur against the lateral confining pressures applied at the boundaries of the unit cell. Here, a plane strain condition is simulated by ensuring a near-zero lateral strain in the longitudinal direction (parallel to sleeper), while the sidewalls of the other direction (perpendicular to sleeper) were allowable to move.

Figure 2a is a schematic representation of the large-scale test specimen with a 150 mm thick SEAL between a 200 mm thick layer of ballast and a 100 mm thick layer of structural fill (subgrade), and a concrete sleeper with a rail on top of the test specimen. The area surrounding the concrete sleeper was filled with shoulder ballast (in red). Here, ballast (field dry unit weight 15.3 kN/m^3) and subgrade materials (field dry unit weight 21.4 kN/m^3) were obtained from a local quarry, and they were all compacted to filed conditions. The SEAL mixtures were compacted to 95% of their maximum dry density which is in the range of 20.0–11.6 kN/m^3 by increasing the rubber content. Figures 2b–d show the procedure for preparing the test specimen. To check ballast breakage, the ballast directly under the sleeper was painted white and was sieved again after each test. The cyclic loading test was carried out at a frequency f = 15 Hz, and the vertical stress under the sleeper is 230 kPa to simulate a train with a 25-tonne axle load running at 115 km/h. The effective confining pressure was15 kPa to simulate the field conditions. Each test was completed after reaching 500,000 cycles.

2.2.2 Cyclic Triaxial Tests for CW-RC Mixtures

Four mixtures of coal wash (CW) and rubber crumbs (RC) with 0, 5, 10 and 15% RC:CW ratio were considered in this study. Rubber contents were selected based on the optimization study performed by Indraratna et al. [12] on a mixture of CW, RC, and SFS. Cyclic triaxial compression tests were performed using the Dynamic Triaxial Testing System. As part of a broader study, four cyclic triaxial tests were performed with a rest period under a confining pressure of 25 kPa and a cyclic deviator stress of 100 kPa to study the effect of rubber crumbs on the deformation behaviour of the mixture. These loading conditions were selected based on the recommendations for a capping/subballast material in railways where the material is likely to be used. The tests included a rest period every 40,000 cycles. The duration of the rest period was selected to represent typical waiting times between the passage of two consecutive trains. Also, the Hall effect sensors were used to measure the radial strain, as shown in Fig. 3.

The testing samples were all compacted to the same initial void ratio and had a diameter of 100 mm and a height of 200 mm. Cyclic triaxial tests were performed in three stages, saturation, consolidation, and cyclic loading. During saturation, de-aired water was injected from the bottom to expel any trapped air inside the sample while the back pressure was gradually increased until a Skempton value of 0.97 was reached. Then, the sample was consolidated isotropically to the desired effective

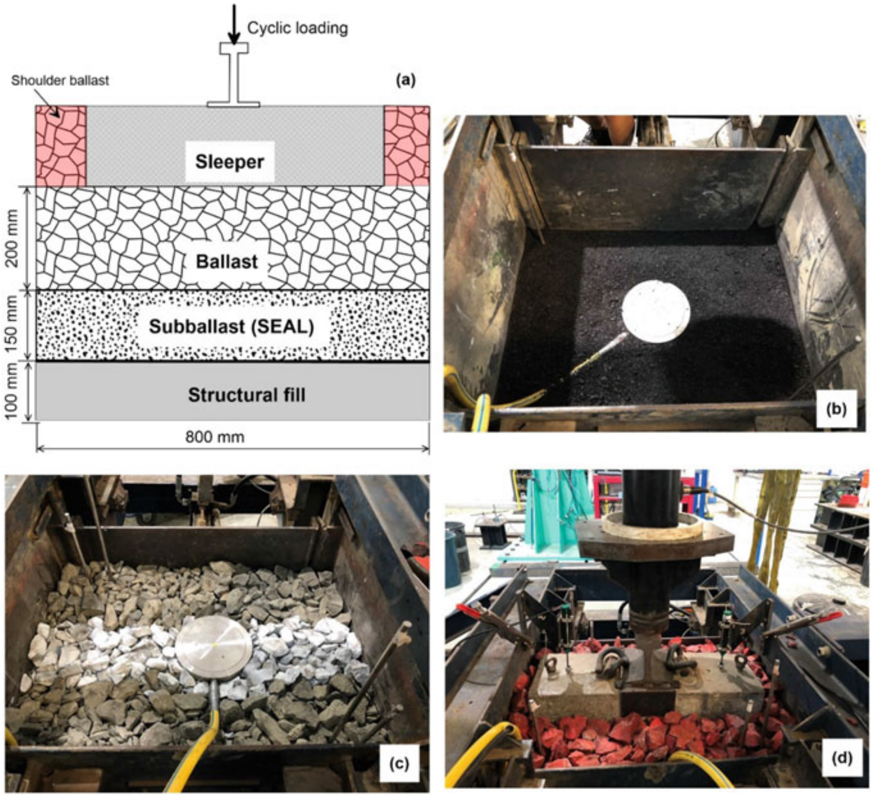


Fig. 2 **a** Schematic view of the physical model for Track specimen with SEAL; **b**, **c** installing SEAL, ballast layer and pressure cell on top of them; **d** final view of the track specimen before testing

confining pressure. Finally, cyclic loading was applied by a one-way stress-controlled actuator. A frequency of 10 Hz was used to represent high speed trains.

2.3 Results and Discussion

2.3.1 Performance of Track with SEAL

Figure 4 shows the settlement and the lateral displacement of the track specimen with traditional materials and SEAL having different rubber contents. It is observed that the settlement increases with loading cycles, and except for the one with 40% rubber the settlement of most of the test specimens stabilises with minimal strain rate ($<0.5 \times 10^{-7}$ mm/mm/cycle). As expected, more rubber is included in SEAL, more settlement is observed for the track specimen (Fig. 4a). When 40% rubber is

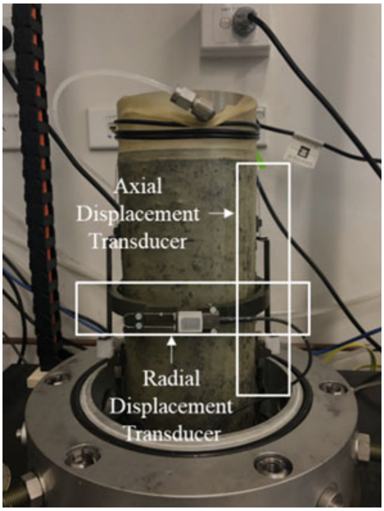


Fig. 3 Cyclic triaxial sample with Hall effect sensors (modified after Tawk et al. [39])

added, the track specimen collapses with extensive settlement (more than 40 mm) within 2000 cycles. In terms of lateral displacement, adding 10% rubber in SEAL reduces the lateral dilation, but when more rubber is added, the lateral movement is increasing with fluctuation (unstable) (Fig. 4b). This may be because that when more rubber is included, the skeleton of SEAL tends to be taken by rubber materials, hence the SEAL mixture behaves like rubber which shows a bounce effect under dynamic loading [40, 41]. When compared with the track specimen with traditional materials, the track specimen with SEAL having 10% rubber shows a comparable settlement and less lateral movement, suggesting 10% rubber is a suitable amount to be included in SEAL for superior track deformation behaviour.

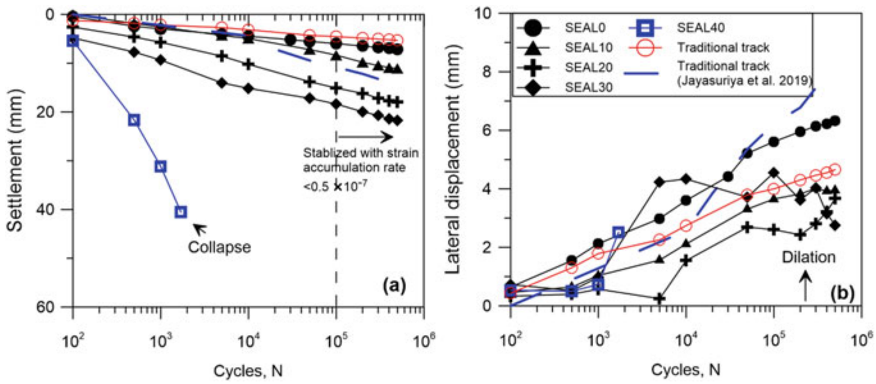


Fig. 4 Deformation behaviour of the track specimen with SEAL mixture having different RC contents: **a** settlement and **b** lateral displacement (modified after Qi and Indraratna [40])

Figure 5a shows the maximum stress measured on top of the ballast, subballast and subgrade layer. Noted that as the load distributes along with the depth of the tack specimen, the measured stress decreases as the depth increases under the sleeper. Under dynamic loading, the actual stress subjected by the track substructure is usually higher than the applied load due to the dynamic amplification effect. Suppose the rail and sleeper are rigid, the applied maximum deviator stress on top of ballast should be 230 kPa, but due to the dynamic amplification effect, the measured actual stress on top of ballast for the track having 40% rubber is more than double. The specimen with 10% rubber shows the minimum dynamic amplification effect as the measured stress is around 240 kPa. The same trend can also be observed for other layers (i.e. subbllast and subgrade). This indicates with a proper rubber content (10%) in SEAL, the dynamic amplification effect can be reduced, while too much rubber will adversely intensify the effect. Further investigation on the effect of rubber contents in SEAL on the track performance is related to the ballast breakage, as shown in Fig. 5b. Ballast breakage index (BBI) is used to evaluate the ballast degradation. It is proposed by Indraratna et al. [42] based on the particle size distribution curves of ballast before and after the test (Fig. 5b). BBI of the track specimen with traditional materials is similar to the track specimen with SEAL having 0% rubber. The addition of rubber will significantly reduce the ballast breakage index (46–66%). Having 10% rubber inside the SEAL seems sufficient to reduce ballast breakage as only a minimal change in BBI is observed when more rubber is added.

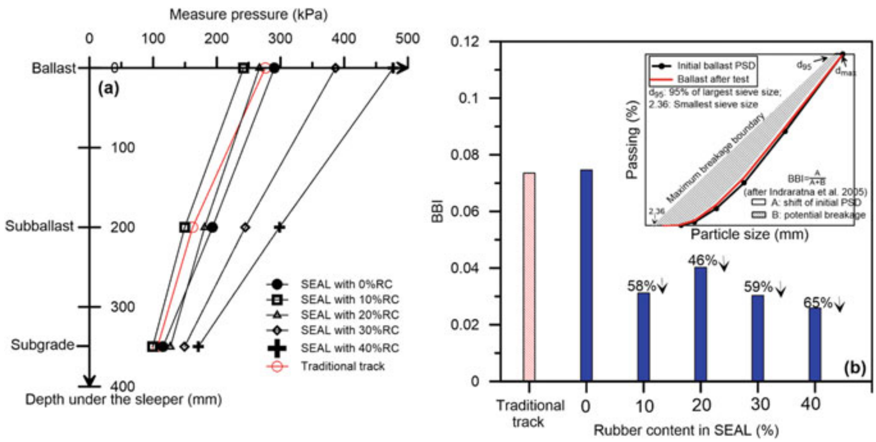


Fig. 5 Test result of the track specimen with SEAL: **a** measured pressure on top of ballast, subballast and subgrade, and **b** BBI (modified after Qi and Indraratna [43])

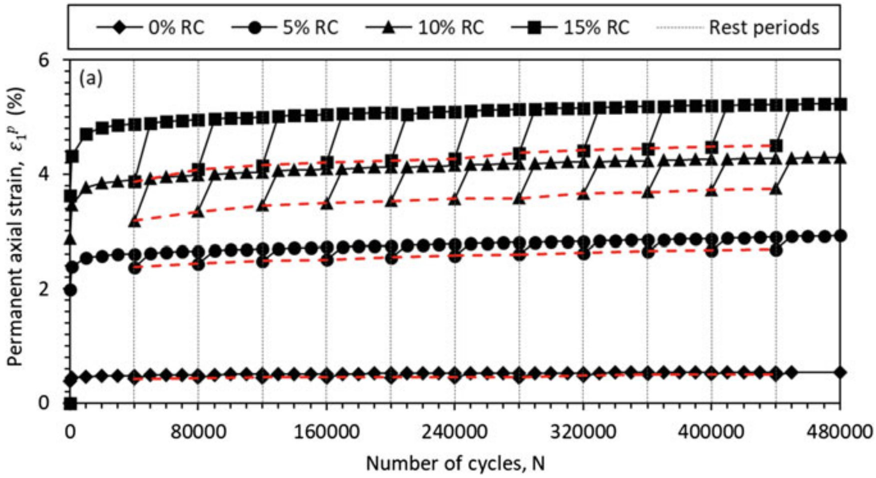


Fig. 6 Permanent axial strain of CW-RC mixtures under cyclic loading with a rest period (modified after Tawk et al. [39])

2.3.2 Cyclic Compressibility of CW-RC Mixtures

Permanent Axial Strain

The permanent axial strain under cyclic loading is used to predict the actual settlements of any material under live loads. Figure 6 shows the permanent axial strain of CW-RC mixtures under a confining pressure of 25 kPa and deviator stress of 100 kPa for 480,0000 cycles with a rest period every 40,000 cycles. In general, the permanent axial strain increases with increasing rubber content. This is attributed to the deformation of rubber particles, which facilitates the rearrangement of particles under loading and hence results in a higher axial strain. For the mixture with no RC, the axial strain before and after the rest period is the same. When rubber is added to the mixture, the axial strain decreases when a rest period is applied. This means that, during cyclic loading, part of the energy delivered to the system is being stored elastically through the compression of rubber particles and is recovered once the cyclic loading is stopped.

Volumetric Strain

The void volumetric strain (ε_v^*) and the total volumetric strain (ε_v) are shown in Fig. 7. In general, a rebound behaviour is observed during the rest period when rubber is added to the mixture for the void volumetric strain and the total volumetric strain. When no rubber is added to the mixture, the difference between the void volumetric strain and the total volumetric strain is minimal. This indicates that the CW particles experience negligible deformation. However, when rubber is added to the mixture,

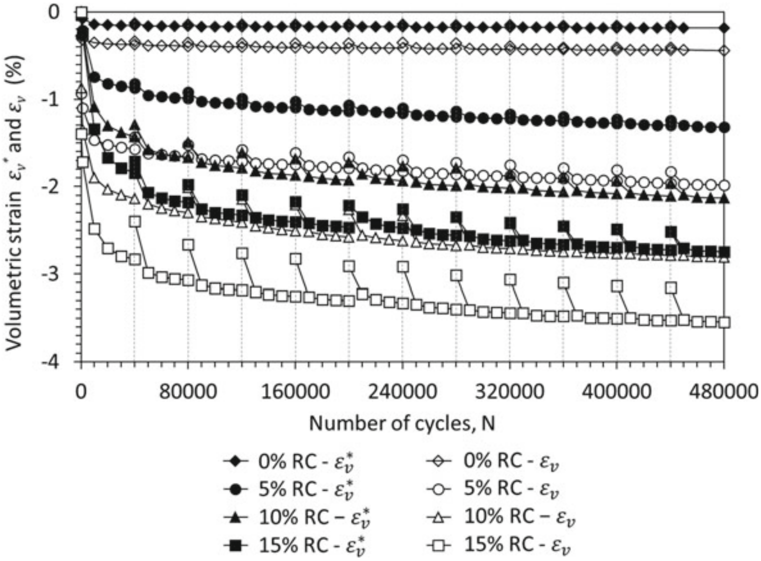


Fig. 7 Void volumetric strain and total volumetric strain of CW-RC mixtures (modified after Tawk et al. [39])

a significant difference between the void volumetric strain and the total volumetric strain is observed. This proves that rubber particles experience a change in their volume, thus contributing to the total volumetric strain as expressed in Eq. (2).

Void Ratio

The void ratio calculated using Eqs. 4 and 5 is plotted in Fig. 8. For the mixture with no rubber, the same void ratio is calculated using both equations. This is an expected outcome knowing that the volume of CW particles is constant. When rubber is added to the mixture, the void ratio considering the change in the volume of solids is greater than the void ratio calculated using the traditional equation (i.e., Eq. 4). In addition, during the first few cycles, the mixtures with RC experience an increase in the void ratio. This is explained by the change in the volume of the solid phase (i.e., rubber particles) along with the decrease in the volume of voids. This indicates that some classical relationships that were developed for rigid materials which are relatively incompressible must be modified before they can be applied to any material with a compressible component.

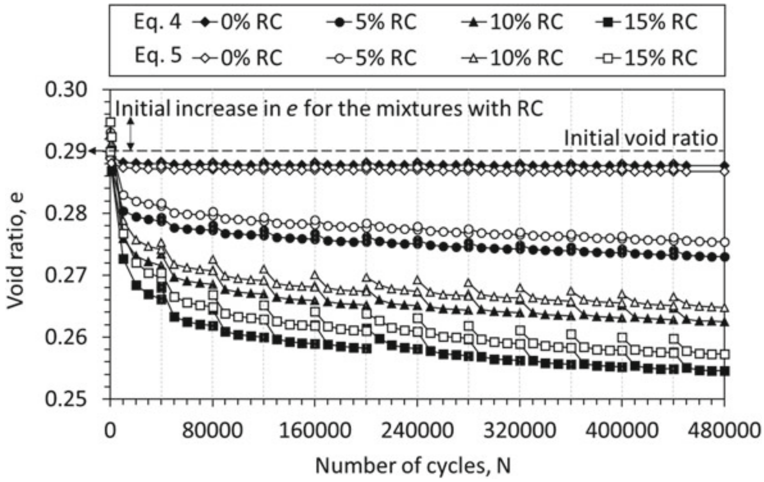


Fig. 8 The void ratio of CW-RC mixtures (modified after Tawk et al. [39])

3 Using Under Ballast Mats (UBM)

3.1 Application Concept

The resiliency of the track diminishes where the subgrade is very stiff (in the case of concrete base) hence, additional artificial substructure inclusions become imperative to restore the track resiliency. Ballasted tracks are subjected to even greater stresses and faster deterioration in sections where a reduced ballast thickness is used and places where heavier concrete sleepers are used instead of lightweight timber sleepers. Compared to the ballast, the high ductile nature of resilient rubber mats can absorb a significant amount of energy during dynamic loading at a relatively high train speed [44]. Depending on the thickness and stiffness of the UBMs, the dynamic forces decreased considerably that leads to a reduction in ballast degradation [26, 45]. UBMs actively decrease the amount of strain energy transferred to the ballast layer, thus reducing particle deformation and degradation. UBM placed at the bottom of the ballast layer eliminates the hard interface between the ballast and underlying stiff layers, which enables the ballast to bed into a relatively softer rubber mat and increases its contact areas of the ballast at the interfaces, which helps to distribute the cyclic load more uniformly over a larger contact area. Therefore, the excessive inter-particle and interface contact stresses of ballast are eliminated, which in turn reduces ballast deformation and breakage [46–48]. However, studies conducted to analyse the effectiveness of rubber elements such as UBMs in minimising ballast deformation and degradation under cyclic loading are limited. The UBMs used in this study were manufactured locally from recycled shredded tyres after removing the non-rubber components (fibre and steel cords) from the waste tyres. These rubber mats have high compressive and impact strength and are typically used as a protective layer on top of the deck of a concrete bridge built for rail tracks [44].

3.2 Testing Program

To assess the role of UBMs in more detail, a series of large-scale cyclic loading tests were conducted using the track process simulation apparatus (TPSA). Photographs of the TPSA cubicle triaxial chamber, mimicking a very stiff subgrade using a concrete base, placement of UBM on top of the concrete base, placement of ballast inside the triaxial chamber using vibratory compaction, placing instrumentation devices (pressure cell and settlement plates) and final cyclic load testing setup are shown in Fig. 9. A plane strain condition was simulated by restraining the sidewalls parallel to the sleeper (i.e. $\varepsilon_2 = 0$), and the other two sidewalls were allowable to move and ε_2 was measured by the system. Lateral confinement (σ_3) was applied by a hydraulic jack connected to movable vertical walls that simulated a low confining stress of 15 kPa. As shown in Fig. 9, a single layer of UBM was used on top of a concrete base to stiffer subgrade conditions (i.e. concrete in this study) found at the rail track over concrete bridges and tunnels. The UBMs made of recycled tyre waste used for laboratory testing were 10-mm thick and were made of recycled rubber granulates encapsulated within a polyurethane elastomer compound.

The tests were conducted by varying axle loads and loading frequency, simulating fast and heavy haul trains, and the influence of UBM was tested both with and without placing the rubber mat on top of the concrete base. The cyclic loading was the equivalent of a 25t and 35t axle load to simulate a passenger and coal train,

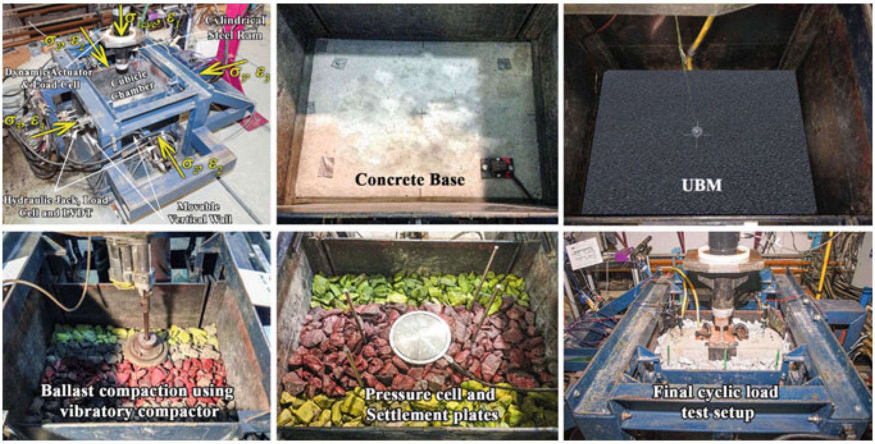
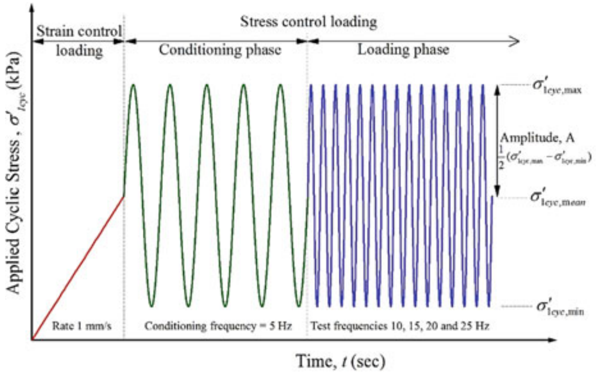


Fig. 9 Photographs of the testing device and laboratory test procedures (modified after Navaratnarajah and Indraratna [26])



Applied Cyclic Stress Parameters (kPa)	Axle Load	
	25t	35t
Maximum Cyclic Stress, $\sigma'_{1cyc,max}$	230	320
Mean Cyclic Stress, $\sigma'_{1cyc,mean}$	130	175
Minimum Cyclic Stress, $\sigma'_{1cyc,min}$	30	30
Amplitude, $A = \frac{1}{2}(\sigma_{1cyc,max} - \sigma_{1cyc,min})$	100	145

Fig. 10 Applied cyclic stress for 25t and 35t Axle Loads (modified after Navaratnarajah and Indraratna [26])

respectively. The 25t axle load was tested with loading frequencies of 15, 20 and 25 Hz, and the 35t with loading frequencies of 10, 15 and 20 Hz. The range of train speeds based on these loading frequencies are 70–180 km/h [44]. The corresponding cyclic loading in a sinusoidal waveform is shown in Fig. 10.

3.3 Performance of Track with UBM

The vertical and radial strain of the ballast with and without UBM was recorded at the selected number of cycles N = 1–500,000 as shown in Fig. 11.

They show that the ballast settles quickly within 9,000 cycles because of its initial densification and compaction when the angular corners of the particles start to break. Still, once the ballast becomes stable after 110,000 cycles, the strain rate reduces with the number of load cycles. High vertical and lateral plastic strains can be observed due to the concrete foundation. The presence of UBM considerably decreases the plastic strains by 15–20%, and the lateral one is decreased up to 10% for the frequencies of 10–25 Hz and axle loads of 25t-35t.

The layer of balllast was divided into three sublayers (0.1 m) with different colours, to assess particle degradation separately. Pressure cells were installed at three depths i.e. (1) ballast-sleeper; (2) ballast–ballast; and (3) ballast–concrete base, to measure the pressure distribution with the depth. Particle size analysis of each layer was conducted before and after the test to determine particle breakage (BBI).

Figure 12 presents the profles of pressure and BBI. The results confirm the increase in stresses in the layer due to the increase in axle load and frequency. A UBM layer at the top of a concrete base contributes to a notable decrease in pressure, especially at the concrete base-ballast and middle portion of the layer. Stress distribution due to a rigid base is now within about 1–1.5, 5–10 and 10–20% at the ballast-sleeper, ballast–ballast and ballast–concrete base, respectively. As expected, higher BBI is

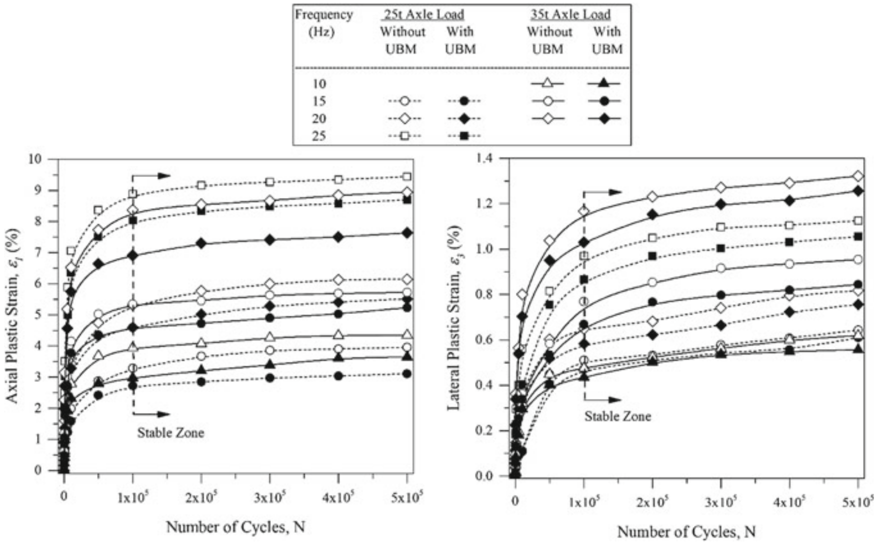


Fig. 11 Axial and lateral plastic strain with the number of cycles for ballast with and without UBM (modified after Navaratnarajah and Indraratna [26])

observed at the top portion due to greater inter-particle contact stress and this reduces at the greater depths. Using UBM underneath the ballast layer can considerably reduce particle breakage. The UBM decreases the degradation by more than 40% on average. For each sub-layer, the reductions are 22% at the top, 40% at the middle and 55% at the bottom layers.

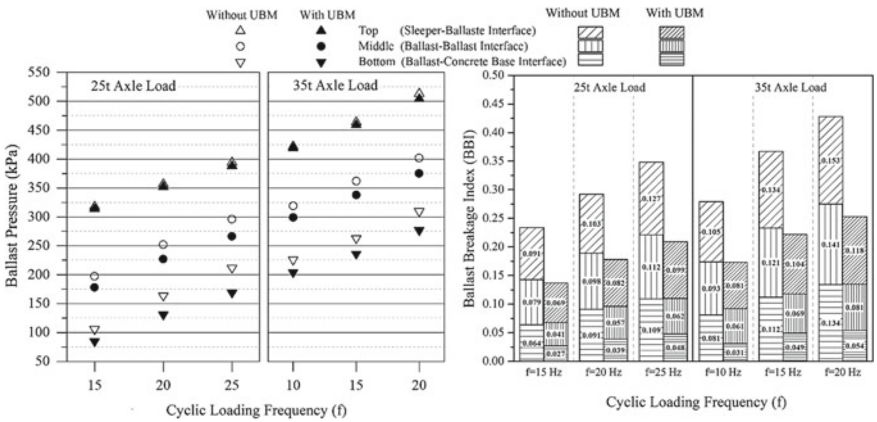


Fig. 12 Ballast pressure variation and ballast breakage index (BBI) with and without UBM (modified after Navaratnarajah and Indraratna [26])

4 Tyre-Cell Reinforced Railway Capping Layer

4.1 Application Concept

In this technique, the standard subballast layer of the railway track is replaced with waste rubber tyres having compacted infill materials. The waste rubber tyre is used after removing one of its sidewalls (the resultant herein referred to as "tyre cell"). The infill material is added to the tyre cell, and this unit cell is repeated throughout the width and length of the required track segment (Fig. 13). This method has the additional benefit of utilizing the 3D cylindrical geometry of the tyre cells along with the energy absorption characteristics of the tyre rubber. As a result, the infill material is more tightly contained, hence increasing the track's bearing capacity.

This concept of tyre-confined granular material in the subballast layer has three main engineering benefits to offer: (1) the confinement provided by the tyre cells increases the stiffness of the capping layer and thus reduces the vertical strains in the track; (2) the deformation in the ballast layer decreases due to the prevention of the lateral spreading of the underlying granular material; (3) the load bearing capacity of the subballast layer increases and this results in lesser loads transferred to the top of the subgrade. This application is considered for use in the subballast layer only. Owing to the maintenance needs as well as the necessity to conform to rigorous standards of ballast placement and compaction in some countries, such as Australia, putting tyres in the ballast layer will provide a practical challenge. The applications presented in this study are intended to guarantee that track building and maintenance activities are not hindered.

This technology of tyre cell reinforcement in the subballast layer is pending an international patent (International (PCT) Patent Application No. PCT/AU2018/050074).

Fig. 13 Placement of tyre cells in the field application

4.2 Analytical Studies

This section presents the analytical expressions to determine the increased bearing capacity due to the tyre cell-reinforcement in the granular material. The values calculated through these expressions are compared with the experimental studies carried out with plate bearing tests in the laboratory. Different studies have been conducted to assess the effect of planar (geogrid or geotextile) and cellular (geocell) reinforcements on the soil foundations. The tyre cell reinforcement is a new concept and a lot needs to be explored about its performance and design methodologies. Due to the close similarity with geocells, this study will evaluate the effect of tyre cell-reinforcement on granular material based on the concepts of geocell reinforcement presented in the literature.

Zhao et al. [49] suggested that the cellular reinforcement functions in three ways: (i) lateral resistance effect, (ii) vertical stress dispersion effect, and (iii) membrane effect. This study will mainly focus on the lateral resistance and vertical stress dispersion effect of the cellular reinforcement.

The bearing capacity equation for the reinforced foundation can be calculated by adding the increment due to the reinforcement to the unreinforced bearing capacity:

$$q_r = q_{un} + \Delta q \tag{6}$$

where q_r = bearing capacity of the reinforced layer; q_{un} = bearing capacity of the unreinforced layer; $\Delta q = \Delta_{q_1} + \Delta_{q_2}$ = increase in the bearing capacity due to reinforcement.

As mentioned above, the increase in bearing capacity (Δq) can be attributed to the following factors (i.e. lateral resistance effect and vertical stress dispersion effect).

4.2.1 Lateral Resistance Effect

This effect produced by the cellular confinement was first suggested by Koerner [50]. The tyre cells have a three-dimensional geometry that helps confine the infill granular materials. Due to this confinement, the lateral spreading of the granular materials is completely restricted. This results in the increased shear strength of the infill materials. Also, the interfacial resistance due to the interaction between the tyre cell wall and the infill aggregate enhances the load bearing capacity of the subballast layer (Fig. 14). The increase in bearing capacity (Δ_{q_1}) due to this lateral confinement can be calculated as below. The interface friction angle (δ) for the cellular confinement and the infill has been used in the range of between one half and two-thirds of the friction angle of the infill material. In this study, the friction angle of the infill material measured through direct shear tests in the laboratory was 39° and the selected value of δ is 23°.

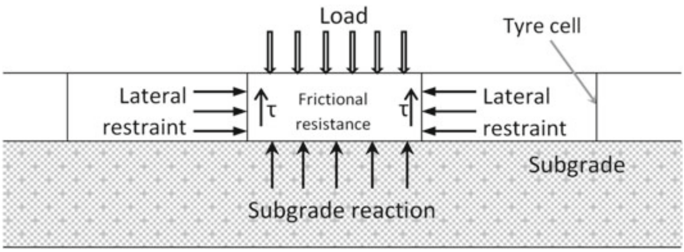


Fig. 14 Lateral resistance effect of the tyre cell reinforcement

$$\Delta_{q_1} = 2\tau \tag{7}$$

$$\tau = P \tan^2(45 - \frac{\varnothing}{2})\tan \delta \tag{8}$$

where P = applied pressure on top of the tyre cell;

$\varnothing$ = angle of friction of infill;

δ = friction angle between the infill and the tyre wall.

4.2.2 Vertical Stress Dispersion Effect

Due to the tyre cell reinforcement, the subballast layer behaves as a geo mattress with a higher modulus that distributes the load over a wider area. The load distributed over a wider area in the subballast layer causes lesser stress transferred onto the top of the subgrade (Fig. 15). This phenomenon is called the "stress dispersion effect" and was first presented by Binquet and Lee [51]. Its contribution towards the bearing capacity (Δq_2) can be calculated as per the expression given below. The angle of load dispersion varies from 30° to 45° [52], and the value used for calculation in this study is 30°.

$$\Delta q_2 = P\left(1 - \frac{b}{b + 2h_r \tan \theta}\right) \tag{9}$$

where b = width of footing transferring load to the tyre cell;

h_r = height of reinforcement;

θ = angle of load dispersion [49].

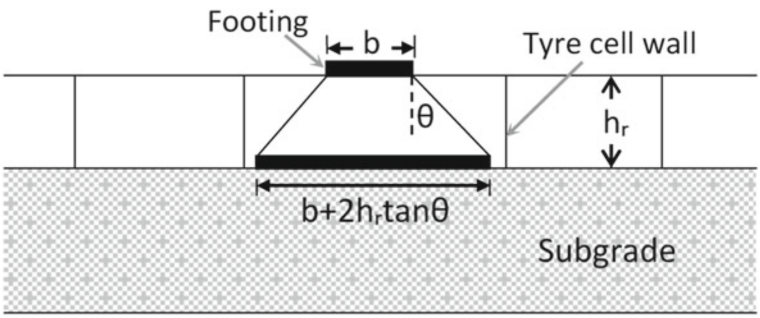


Fig. 15 Vertical stress dispersion effect of the tyre cell reinforcement

4.3 Experimental Studies

Two plate load tests were performed on the unreinforced and tyre-cell reinforced samples in a test box (800 mm long, 600 mm wide, and 400 mm high) [29]. A 50 mm thick layer of compacted clayey sand was placed as the bottom layer in the box to act as subgrade, the top 350 mm was filled with the subballast material (size ranging from 75 µm to 19 mm). For the reinforced sample, the subballast material was filled up to a height of 200 mm over the subgrade, then a 2 mm thick geotextile was placed. The tyre cell was placed on the geotextile and was later filled with the subballast material inside and outside the tyre cell. The subgrade and subballast materials were compacted with a vibratory hammer to a density of 17 and 21 kN/m^3, respectively. The interior tyre cell wall was cleaned and degreased before attaching the strain gauges to it with an industrial adhesive. The schematic of the test setup is shown in Fig. 16. The properties of the test materials are given in Table 1.

A circular loading plate of 200 mm diameter was used in the tests. The centre of the plate coincided with the centre of the tyre cell. Displacement controlled loading was applied at a rate of 0.2 mm/min. Throughout the tests, the vertical load, displacements and the strains inside the tyre cell were recorded. The strain gauge readings showed that the strains in the tyre cell were extremely small. The circumferential tensile strain and the axial compressive strain were 0.061 and 0.0085%, respectively, under the vertical loads of 1610 kPa (Fig. 17). This confirms the ability of the tyre cell to provide confinement to the infill and maintain its shape under high loads which is an advantage over the geocells that tend to deform significantly after the loading application.

The load–displacement curves of the reinforced and unreinforced samples (labelled as “measured”) are shown in Fig. 18. For the test without reinforcement, the stiffness was approximately 51.9 kPa/mm but for the test with tyre reinforcement, the stiffness of the sample was 78.4 kPa/mm. This indicated a 51% gain in stiffness due to the tyre cell reinforcement.

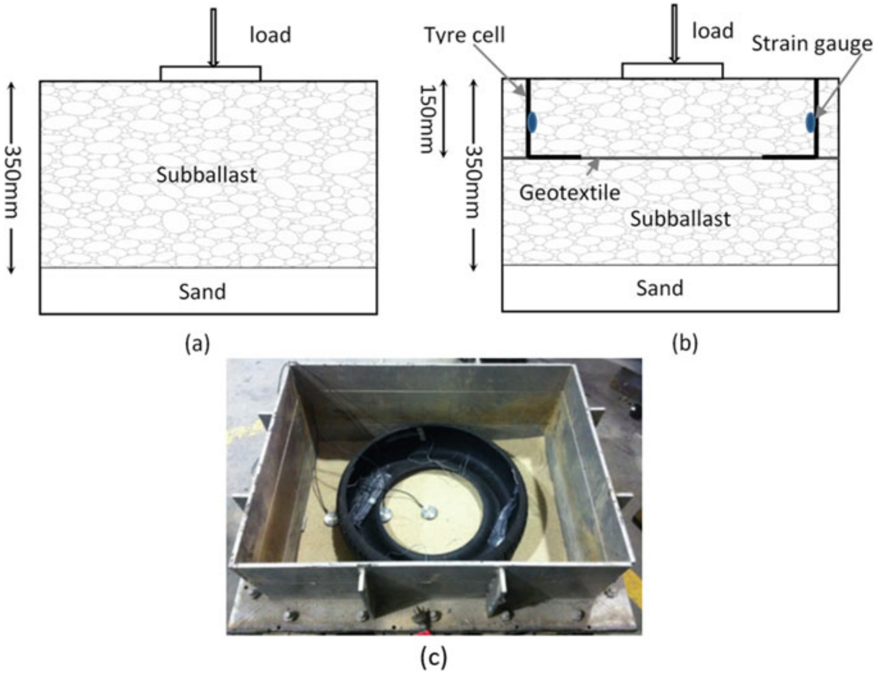


Fig. 16 Schematic showing the test setup for the **a** unreinforced, **b** reinforced sample, and **c** photo showing the tyre cell in the test box (modified after Indraratna et al. [29])

Table 1 Properties of the test materials

Test materials	Properties
Tyre infill (subballast)	$d_{max} = 19$ mm, $d_{min} = 0.075$ mm, $C_u = 16.3$, $C_c = 1.3$, $\phi = 39°$, $\gamma_d = 21$ kN/m^3
Subgrade	$d_{max} = 4.75$ mm, $d_{min} = 0.01$ mm, $C_u = 2.4$, $C_c = 1.1$, $\gamma_d = 17$ kN/m^3
Tyre cell	Diameter = 560 mm, width = 150 mm, ultimate tensile strength = 19.45 MPa
Geotextile	Tensile strength = 16.8 kN/m, tensile elongation <24%

The increase in the bearing capacity was also calculated considering the lateral resistance effect and the vertical stress dispersion effect explained above. The increment was added to the unreinforced bearing capacity recorded in the lab at respective loading points. The resultant reinforced bearing capacity (labelled as "calculated") is shown in Fig. 18. There is a reasonable agreement in the measured and calculated bearing capacity of the reinforced sample up to a stress level of 700 kPa. The stress transferred to the top of the subballast layer is unlikely to exceed this high value in the field, and even if it does, the calculated bearing capacity value will be underestimated and the design would still be safe. In conclusion, this method of determining

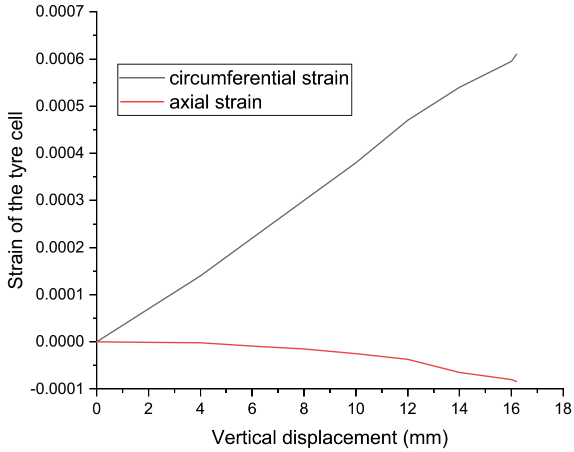


Fig. 17 The variation in the circumferential and axial strain of the tyre wall with the vertical displacement of the sample

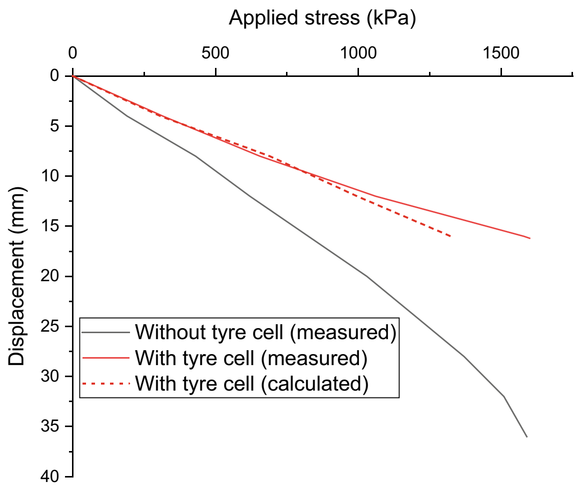


Fig. 18 The load–displacement curves for the unreinforced and reinforced sample

the increment to bearing capacity calculation can be used with confidence to predict the bearing capacity of the reinforced subballast layer in conjunction with conventional bearing capacity calculations for the traditional tracks. Due to the increased bearing capacity, the vertical displacement of the specimen with tyre cell reduces, further proving the advantage of reinforcing the capping layer with tyre cells.

5 Practical Applications

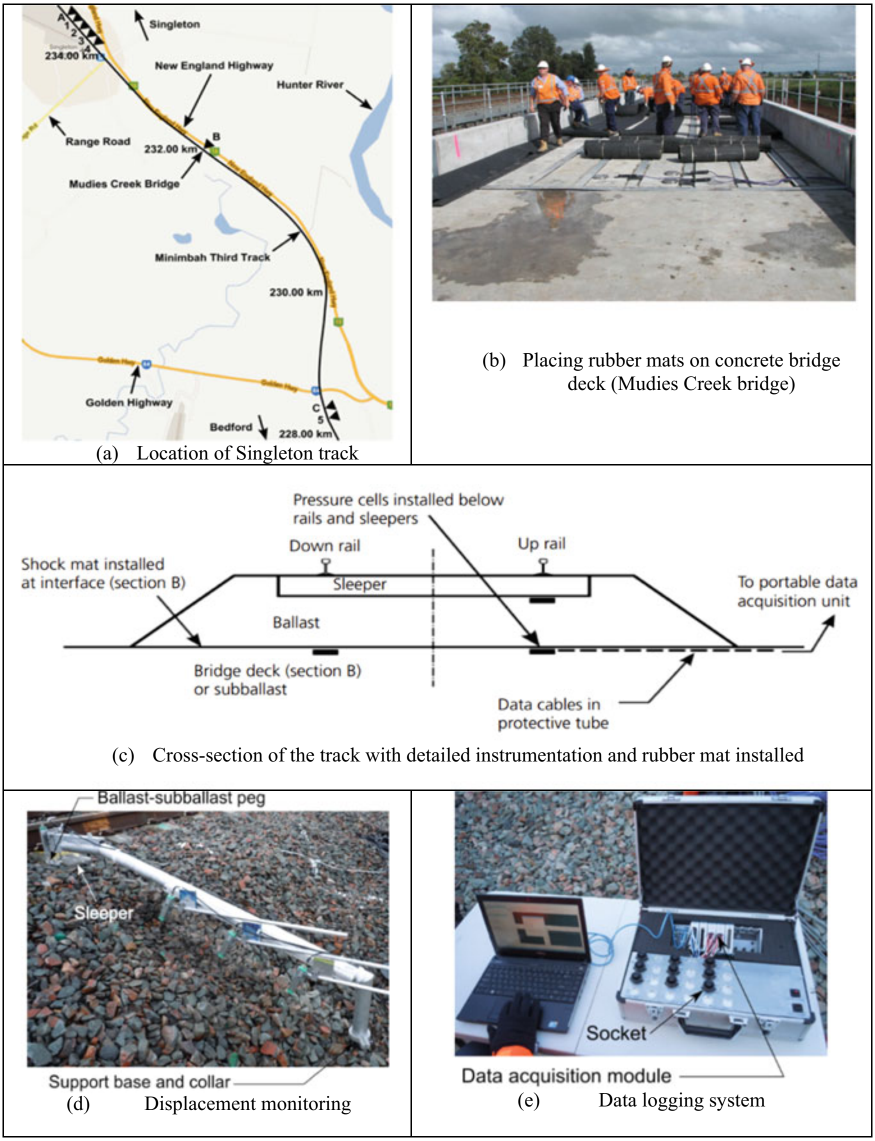
5.1 *Singleton Track with Shockmats (UBM)*

Fully instrumented tracks at Singleton, NSW Australia were built to study the effects of geo-inclusions and shock mats (Fig. 19a). A total of 8 sections were constructed on various types of subgrades, (i) soft soils, (ii) stiff concrete bridge decks (Mudies Creek), and (iii) siltstone. A typical track substructure was composed of a 0.3 m thick ballast followed by a 0.15 m thick capping layer. A structural fill (0.5 m) was placed on the subgrade. Geogrids and geo-composite (geogrid + non-woven geotextiles) were placed below the ballast layer. The geogrids consist of (1) geogrid-G1 (44 × 44 mm aperture); (2) geogrid-G2 (65 × 65 mm aperture); and (3) geogrid-G3 (40 × 40 mm aperture). A shock-absorbing mat was installed under the ballast at the bridge deck to minimise ballast degradation (Fig. 19b). Further details can be found and explained elsewhere by Indraratna et al. [27]. Different sensors and instrumentations (pressure plates, electronic potentiometers, strain gauges and settlement pegs) were installed, as shown in Fig. 19c, installed on the grid were used to measure mobilized strains. Signals from sensors were recorded using a mobile data logging system at a frequency of 2 kHz as described earlier by Indraratna et al. [27].

The vertical deformations (S_v) of ballast with load cycles (N) are shown in Fig. 20. The increasing settlement rate can be observed with the increase in the number of cyclic loads. Settlement of the reinforced tracks is 20% less than that of conventional tracks, confirming the effect of geogrid in reducing track settlement. Similar behaviour was also recorded in the laboratory [53], mainly attributed to the particle-geogrid interlock. Moreover, the geogrid can reduce settlement when tracks built on the soft subgrade. The BBIs with and without geogrids and rubber mats are shown in Fig. 21. BBI increases with number of cycles and intensifies on stiffer subgrade (hard rock). The geogrid and rubber mats reduce the ballast degradation. The rubber mats are a more effective in reducing the ballast breakage (70%) when compared with the conventional section.

5.2 *Chullora Track Design*

Transport Research Centre, the University of Technology Sydney in collaboration with Sydney Trains through funded ARC ITTC-Rail and ARC Linkage projects is committed to transfer theory to practice with innovative applications of recycled rubber tyre reinforcements in ballast tracks in the Chullora Technology Precinct (Fig. 22). Four key principles are to be examined in the field through rigorous design to include recycled rubber products. It will be most advantageous and scientifically sound for all the above methods to be tested along the same track on the same subgrade terrain through a fully instrumented monitoring system so that the relative



(a) Location of Singleton track

(b) Placing rubber mats on concrete bridge deck (Mudies Creek bridge)

(c) Cross-section of the track with detailed instrumentation and rubber mat installed

(d) Displacement monitoring

(e) Data logging system

Fig. 19 Details of instrumentation of experimental sections of track at Singleton (modified after Indraratna et al. [27])

merits of each of these approaches can be evaluated and compared to the standard conventional design of ballast tracks in a controlled straight track section.

Based on the standard penetration test (SPT), cone penetration test and laboratory testing, clayey silt with low plasticity was encountered in all boreholes from the surface to a relatively shallow depth. The thickness of this soil layer was found to

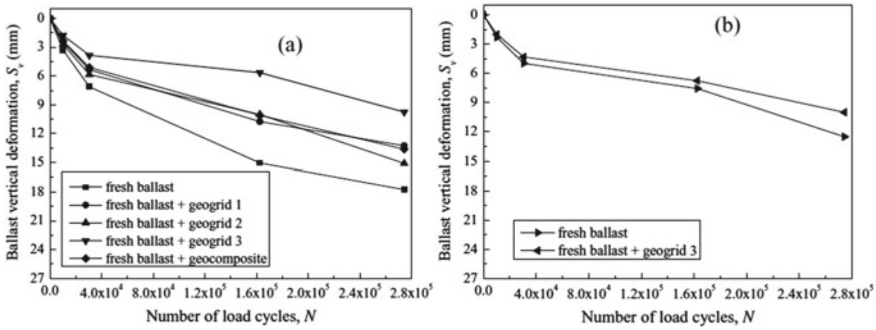


Fig. 20 Ballast vertical deformation with and without geogrids and geocomposite (modified after Indraratna et al. [27])

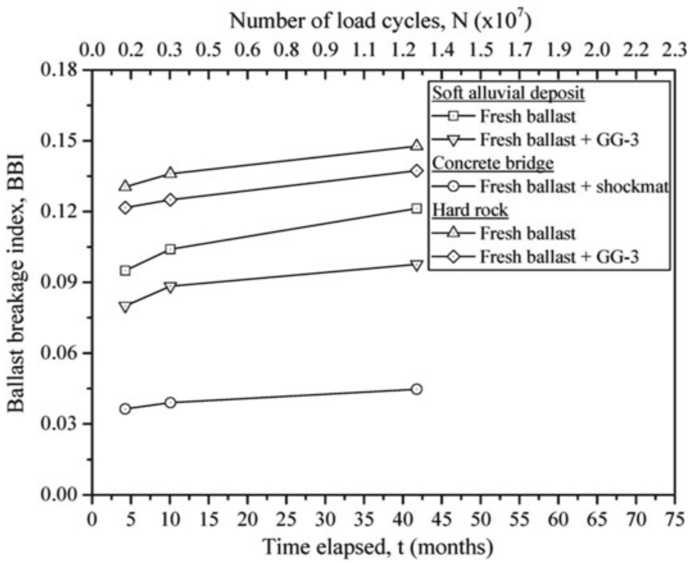


Fig. 21 Ballast breakage index of the track section with and without geogrid and shockmat (modified after Indraratna et al. [27])

range from 1.2 to 4.8 m. Based on the SPT and CPT test results, the natural clay was generally assessed to be medium to stiff at a depth of 4 m from the surface. The natural clayey soil was found to have moisture content (18%) significantly less than the plastic limit of 24%. Groundwater was not encountered in any of the boreholes during the investigation hence the observed drained and stiff conditions of the dry subgrade. The track is currently under construction to incorporate the above novel applications.

Taking the track section incorporating tyre cells, the waste tyres are placed in the capping layer after removing one of their sidewalls, the resultant called as tyre

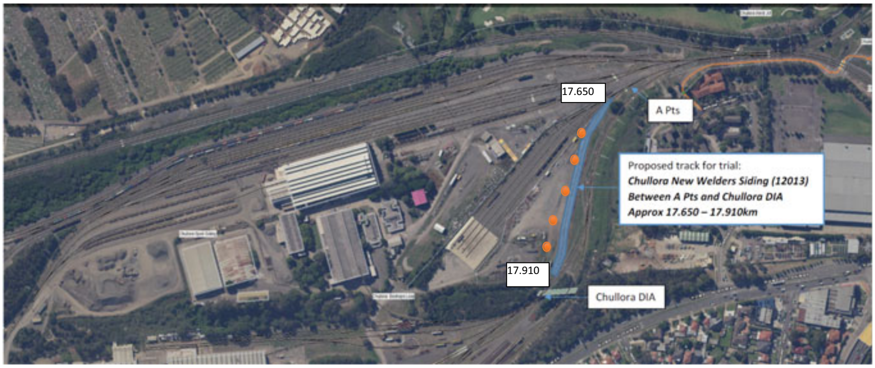


Fig. 22 Locations of trial site within Chullora technology precinct

cell (Ecoflex). The tyre cells are then filled with the compacted infill materials. The placement of the tyre cells to prepare the capping layer is shown in Fig. 23. A typical cross-section of the track is being designed using the method proposed by Indraratna and Ngo [54] to obtain approval (Fig. 24). It is expected that the track performance can be enhanced and track maintenance costs can be minimised by reducing the deterioration of ballast aggregates and decreasing excessive settlements of the formation layers (i.e. capping and structural fill layers).

Fig. 23 Tyre cell assembly placed for the preparation of capping layer

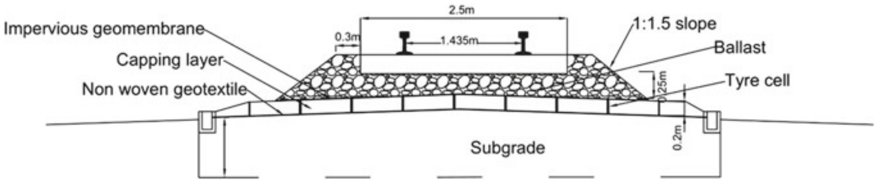


Fig. 24 Typical cross-section rubber tyre cells with recycled ballast infill

5.3 Port Kembla Outer Harbour Reclamation

This section presents the application of Coalwash (CW) and Blast Oxygen Steel slag (BOS) at the Port Kembla Outer Harbor reclamation, NSW, Australia, as shown in Fig. 25 [55]. The geotechnical properties of CW and BOS blends based on laboratory testing indicate that two potential mixtures can be chosen for the field trial [30]. Coalwash (CW) is a granular waste product from coal mining. The steel furnace slag (SFS) is a by-product of steelmaking plants. A Dendrobium coalwash was produced by Illawarra Coal and Australia Steel Milling Services supplied an SFS produced via the basic oxygen method (BOS).

The field trial was performed at the site with an area of 55 m by 14 m where the depth was 1.4 m. The area was divided into 2 zones for two blends adopted including the combination of equal parts of CW and SFS (CW50-SFS50) and 20% CW and 80% SFS (CW20-SFS80) by volume percentage. The materials were mixed using an excavator where they were compacted using a 13-tonnes smooth steel roller with a 30-Hz vibration. For a lift thickness of 0.3 m, at least 90% maximum dry density was obtained after four and eight roller passes. Based on Dynamic Cone Penetration Tests (DCPTs) and plate load tests (PLT), the compacted fills show their suitability for structural fill in terms of shear strength as their California Bearing Ratio (CBR) values were in the range of 25–50. The swell potential was assessed due to the presence of CaO and MgO. After 5 months, the swelling percentages for CW50-SFS50 and CW20-SFS80 were 5 and 6% respectively due to hydration. This

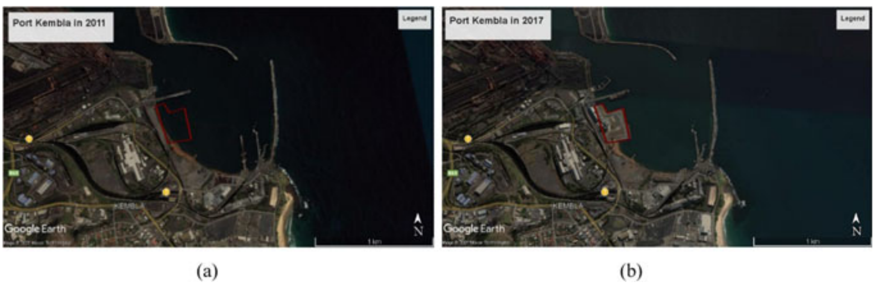


Fig. 25 Location of Port Kembla reclamation project **a** before the reclamation in 2011 and **b** after the reclamation in 2017

implies that surcharge fill (30–60 kPa) would be required to suppress its swelling potential.

5.4 Moss Vale Road, Kangaroo Valley

In this section, Fly Ash (FA) is combined with Coal Wash (CW) to act as a void filler for a base/subbase material. The purpose of FA is to provide a fabric with reduced porosity and potential breakage of coal wash particles. A laboratory investigation was conducted by Wang et al. [56] to characterise the mixture with FA contents (0–13%). The addition of 7% FA can give a minimum void ratio under modified Proctor compaction energy. Based on the unconfined compressive test, this mixture yields the highest shear strength with a CBR greater than 100. The collapse potential is well below the allowable range with a permeability of less than 5×10^{-7} m/s. Based on the above data, a field trial was conducted at Moss Vale Road, Kangaroo Valley, NSW. The area was divided into 2 sections (3.5 m width and 7 m long), (a) standard section using densely graded base (DGB20) and (b) Coalwash Fly Ash Mixture (7% FA with 6% water content).

The construction started with milling the existing road base (260 mm) until the existing subgrade layer. Either CWFA or DGB20 was compacted using a roller followed by the placement of a seal on the surface (Figs. 26 and 27). Based on the suction reading, the results show that the CWFA can prevent the infiltration of moisture during rainfall. The rutting profile of CWFA indicates better performance in comparison to the conventional section (DGB20). The long term performance is being monitored at the site.

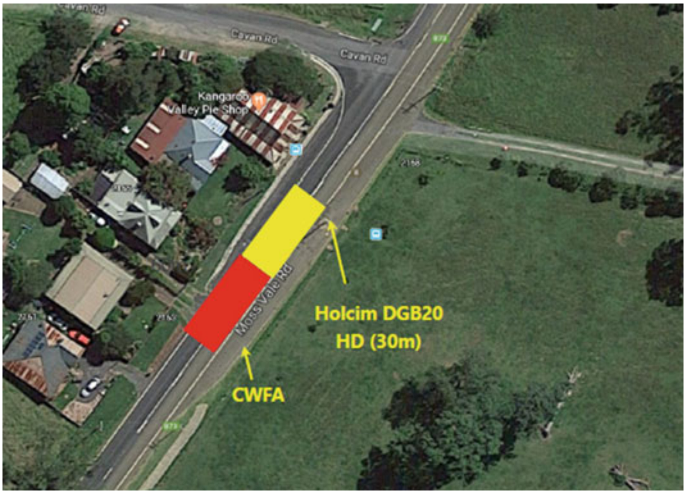


Fig. 26 Location of the trial site at Moss Vale Road, Kangaroo Valley

Fig. 27 Construction sequences **a** Removal of existing layer, **b** compaction using Roller, **c** complete surface

6 Conclusions

This paper reviews four novel applications using recycled rubber products in railway foundations to enhance the track performance and reduce ballast degradation, which includes under ballast mats (UBM), tyre-cell reinforced capping layer, and rubber crumbs (RC) mixed with coal wash (CW) and/or steel furnace slag (SFS) (i.e. SFS-CW-RC and CW-RC mixtures) for subballast. Research concepts, laboratory investigations using iconic facilities (i.e. TPSA, ELDYN, plate load testing facility), and field investigation have been elaborated and discussed. The following remarkable findings can be concluded from this paper:

- By increasing the RC content inside SEAL (SFS-CW-RC) mixtures, the track settlement increased and the ballast breakage decreased, while the lateral dilation decreased when 10% RC was added and fluctuation occurred when more RC was included. 10% RC (by mass) was recommended to be included in SEAL for a superior track performance with less ballast breakage, lateral dilation, pressure distributed in the substructure layer, and comparable settlement comparing to traditional track specimens.
- The rebound behaviour of CW-RC mixtures under cyclic loading was captured by applying a rest period after a certain cycle of loading. The compressibility of rubber was identified by comparing the total volumetric strain and the volume change of the voids. A novel equation was proposed to calculate the void ratio change due to the rubber compression.
- By reinforcing the capping layer with a tyre cell, the stiffness of the specimen was increased 51%, and the vertical displacement was reduced due to the increased bearing capacity based on the plate load tests. Moreover, the tyre cell was proved to have negligible deformation (i.e. circumferential tensile strain and the axial compressive strain) under higher vertical loads (1610 kPa).
- When increasing the axial load and loading frequency, the deformation (settlement and lateral dilation), the pressure distributed in ballast, and the ballast breakage of the track specimen increased. By installing the UBM, the axial plastic strain was reduced by 10–20%, the lateral plastic strain was reduced by 5–10%, the pressure distribution in the ballast layer (especially in the middle and bottom)

was curtailed, and the ballast breakage was reduced by 20–60% (from the top to bottom ballast layer) under the tested loading frequencies (10–25 Hz) and axle loads (25t and 35t).

- The field tests carried out on the sections of track with geogrids and shock mats showed enhanced deformation and reduced ballast degradation due to the geogrid-particle interlock and the energy-absorbing property of the rubber mats. The geogrid reinforcement was also found to be more efficient on softer subgrades and rubber mat is the most efficient method to reduce ballast breakage.

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Geotechnical Designs Gone Wrong—Lessons Learnt

Patrick K. Wong

Abstract Since the occurrence of several apartment building problems in recent years, government agencies have taken steps to promote a greater level of building design and construction compliance. For example, the Design and Building Practitioners Regulation 2021 (NSW) [1, 2] that came into force on 1 July 2021 requires engineering reports associated with Class 2 buildings (i.e. residential or part-residential) to be signed by a registered Design Practitioner with relevant qualifications and skills, and a completion compliance certificates to be issued by the various design discipline practitioners of the project and submitted by a registered Professional Engineer. Perhaps time will tell, but the author is sceptical that the introduction of additional government regulations would necessarily reduce the number of failures in geotechnical practice. The author believes that it may be helpful for the industry to publish more case studies on geotechnical failures so that lessons can be learnt from common mistakes. This paper outlines some common geotechnical design problems encountered based on 50 or so expert witness cases that the author has been involved in during the last decade and provides a generalized discussion on some of the most prevalent geotechnical design issues and the lessons learnt.

Keywords Geotechnical design · Failures · Lessons learnt · Groundwater · Retaining walls · Ground movement · Earthworks · Hydro-consolidation · Latent conditions · Jet grouting · Soft soils

1 Introduction

In addition to the introduction of additional building regulations in NSW, professional indemnity insurance for practicing engineers and building inspectors have

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increased dramatically over recent years due to a number of high-profile problems on high-rise residential buildings. The author is skeptical that the introduction of additional government regulations would necessarily reduce the number of failures in geotechnical practice and certainly the reliance on insurance is not the solution to this growing problem. For many decades, design reports on Queensland projects have required these reports to be signed by Registered Professional Engineers of Queensland (RPEQ) which is a separate organization to Engineers Australia. However, there is no indication that there are less geotechnical problems associated with Queensland projects than in NSW.

Most geotechnical designers would have a genuine self-belief that they have the "qualifications and relevant skills" as required by the Design and Building Practitioners Regulation 2021 (NSW) before they take on a design task, but yet, failures do happen. The problem is often caused by our inability to identify gaps in our knowledge; in other words, "*we don't know what we don't know*". This is the equivalent of the well-known Dunning Kruger effect [3] in phycology and is related to the cognitive bias of illusory superiority and comes from people's inability to recognize their lack of ability.

The author believes that a greater awareness of common geotechnical problems through publications and education could play a role in addressing this issue. However, geotechnical practitioners are more likely to publish success stories rather than failures and are often restricted by their client or legal circumstance to publish case studies of the latter. Valuable case studies on failures can be found in specialty conferences such as the International Conference on Forensic Civil Engineering and International Conference on Structural and Foundation Failures. Many papers are published on major disasters (e.g. [4] regarding the Nicoll Highway Collapse in Singapore and [5] regarding major dam failures), but there are very few published papers on smaller, common geotechnical design failures that still cost the industry 10s–100s of million dollars in each case as well as much heartaches for those involved. White [6] says that "*Failures provide one of the best learning tools to improve geotechnical practice*" and in his presentation, he provided examples including large and small problems resulting from failures in design & documentation, specifications and construction, soil variability and probability assessment.

2 Frequency of Different Types of Failures from Author's Experience

In the last decade or so, the author has been involved in 51 expert witness cases of geotechnical issues that have gone "wrong". Most of these can be considered "failures", with about 16% of these being latent condition claims in which one parties says one thing and the other says another. Depending on whose view one takes, these could also be considered as "failures" in investigation, interpretation and/or documentation. Figure 1 presents the categories of these cases:

It can be seen from Fig. 1 that the majority (20 out of 51 or about 39%) of these cases relate to shoring and retaining wall problems, and this is not surprising for the following reasons:

- Temporary shoring occurs frequently for excavations such as for basement construction and because of the temporary nature of these structures, their design often lack the rigorous process such as design verification that is required for permanent structures, and construction supervision by qualified geotechnical professionals is often omitted.
- The complex soil-structure interaction and relationship with groundwater in retaining wall design is either not or poorly understood.
- Ground movements associated with installation of shoring (including ground anchors where applicable) and excavations can cause damage to adjacent infrastructure, even if the shoring wall is stable.

The next most frequent type of failures from the author's experience is earthworks which totals 13 out of 51 cases or about 25% when latent condition claims involving earthworks are included.

Latent condition claims come next with 8 out of 51 cases or about 16%, and 6 of the 8 cases of latent condition claims are related to earthworks.

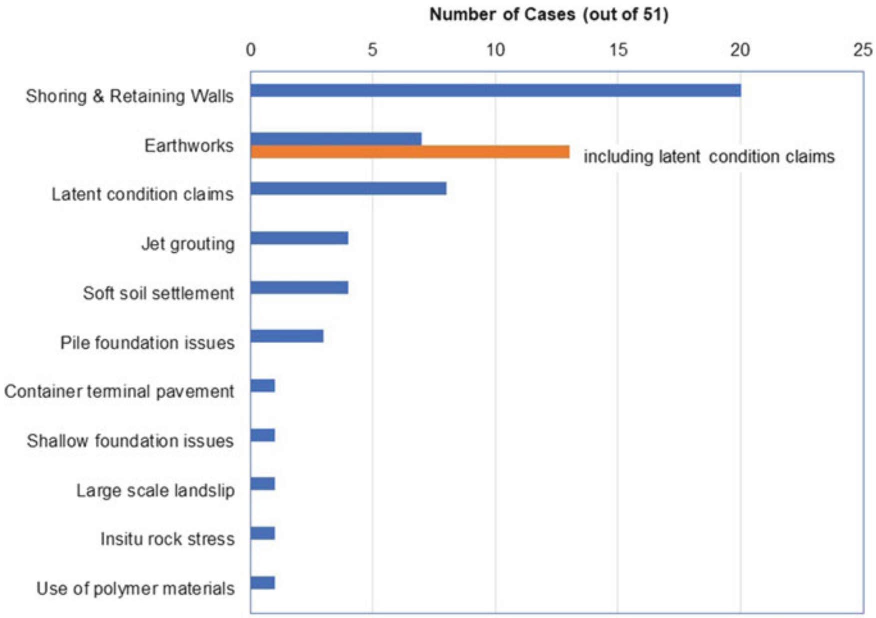

Fig. 1 Frequency of geotechnical problems (Author's Experience)

Jet grouting was the runner up to be equal fourth with soft soil problem. The former is associated with ground deformation causing damage to adjacent structures during jet grouting and the latter is associated with excessive post-construction settlement following removal of surcharge ground improvement.

Based on the frequency of occurrence shown in Fig. 1, examples of these more frequent problems are provided in the following sections, together with limited commentary on the less frequent problems.

The last 5 geotechnical problems, with 1 case each out of the 51 cases, are not covered in this paper.

3 Shoring and Retaining Wall Problems

Shoring and retaining wall problems from the author's experience are broadly divided into the following two categories:

- Design related (17 out of 20 cases)
- Construction related (3 out of 20 cases)

Complete failure (or collapse) of retaining structures is rare, but did occur in 1 of the 20 cases. Most design issues for shoring and retaining walls are associated with excessive movement of the adjacent ground due to the lack of consideration on serviceability requirements and adequate assessment of potential damage to existing infrastructure (see [7]).

The majority of retaining wall issues are involved with the presence of groundwater as shown in Fig. 2.

Some are associated with inadequate allowance for potential rise of groundwater level in design from that observed during the geotechnical investigation. Others involving groundwater are associated with drawdown effects or poor construction tolerance such as deviation of secant piles causing gaps between piles. One of the cases was associated with undermining of an adjacent house due to internal erosion of the sand foundation from the dewatering process. All of these design and construction

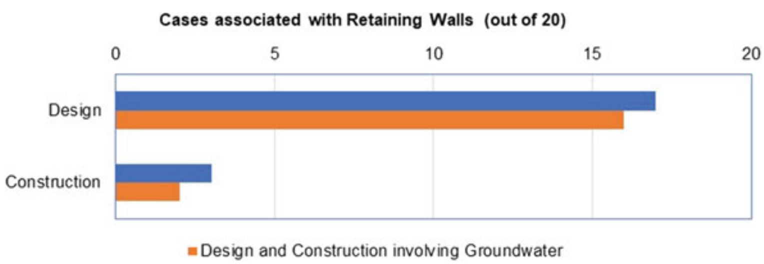

Fig. 2 Involvement of groundwater in retaining wall problems

deficiencies have the potential to cause excessive settlement of adjacent structures or excessive inflow to render it unsafe to work within the excavation.

An example of a design failure to recognise the potential bottom drainage of a sand layer which caused consolidation of the overlying soft clay layer from basal depressurisation is illustrated in Fig. 3.

The sand layer was only exposed on one corner of the site, but it was present from the geotechnical investigation and should have been accounted for in design to ensure that the shoring wall was deep enough at this corner to seal the groundwater from entering the excavation.

Lessons Learnt (Groundwater Considerations for Retaining Wall Design):

- Pay attention to the geology and hydrogeology of the site.
- Consider the variability of the subsurface profile, conduct more geotechnical investigations as required, and take these into account in the design.
- Groundwater problems can come in many different forms, ranging from excessive inflow, basal heave, excessive uplift pressure, internal erosion, settlement from depressurisation of overlying soft and compressible materials as shown in Fig. 3 above. Designers should recognise each of the potential risk and take measures in the design and construction process.
- Make allowance for groundwater rise from that observed during the investigation. In the Botany Sands for example, groundwater level could readily rise 1 m or more above the normal level during wet weather conditions.

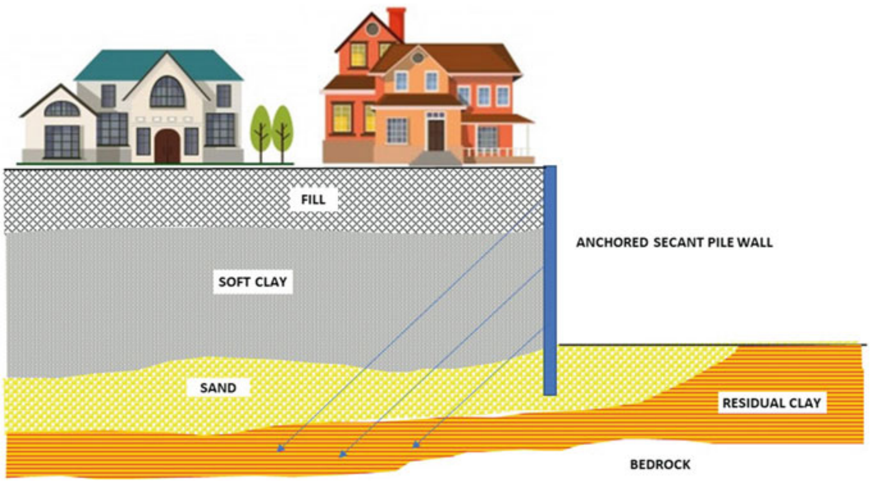

Fig. 3 Example of basal depressurisation of soft clay causing settlement issues

- For secant pile walls, adopt appropriate overlap in the design that takes into consideration of the equipment (e.g. cased or uncased), skill and track record of the contractor, use of guide-walls and construction supervision to limit out of verticality.
- Ground anchors constructed in saturated sand could cause anchor hole collapse and/or withdrawal of excessive material during the drilling process and result in settlement of adjacent infrastructure. Appropriate control of groundwater inflow and methods to stabilise anchor holes during installation must be considered.

Surprisingly, a number of design issues relate to the simple fact that the bond zone of tie-back anchors was inappropriately located within the active zone of the soil being retained as shown in Fig. 4.

From their basic soil mechanics training, designers with appropriate geotechnical professional qualifications should know the following active wedge angle θ (angle from vertical) in relation to the effective friction angle, ϕ of the soil to be retained:

$$\theta = 45^{o} - \frac{\varnothing}{2} \tag{1}$$

Hence, why do we still encounter such problems, including 1 wall out of the 20 cases actually collapsed and causing major structural damage to adjacent buildings?

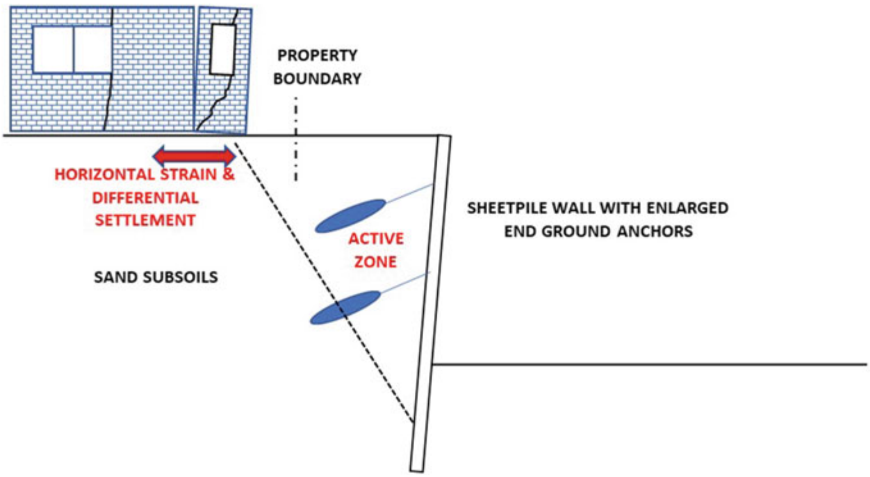

Fig. 4 Inappropriate location of anchor bond zone

Lessons Learnt (Anchorage Bond Zone):

- Serviceability limits must be considered in design.
- The bond zone of retaining wall tie-back anchors should be located beyond the active zone.
- The lateral wall deformation reduces with increasing distance of the bond zone from the active zone, and with increasing bond length.
- It is advisable to draw the line of the active zone from a perpendicular distance of at least 1 m from the excavation level.

4 Earthworks Problems

In relation to geotechnical problems associated with earthworks, the types of problems experienced by the author fall broadly into 3 categories of creep & hydro-consolidation, shrink-swell and compaction issues as illustrated in Fig. 5.

4.1 Creep and Hydro-Consolidation

Often, it is difficult to separate creep and hydro-consolidation and as such, these two mechanisms are discussed as a combined settlement below.

Long-term creep of compacted fill is usually not significant. However, when the depth of fill exceeds about 10 m, long-term creep effects become more significant due to the stress level exceeding the "preconsolidation pressure" from compaction.

Hydro-consolidation is commonly encountered in natural "collapsible soils" which are defined as any unsaturated soil that goes through a radical rearrangement of particles and resulting decrease in volume upon wetting, additional loading, or both. In compacted fill, hydro-consolidation is less commonly encountered due

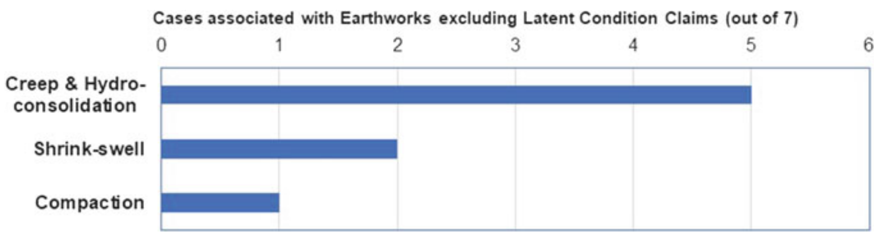

Fig. 5 Types of common earthworks problems. *Note* the one with compaction issue is also amongst the 5 with creep & hydro-consolidation issues)

to the reasonable level of compaction general required for engineered fills but could become more problematic in deep fills.

In recent times, creep and hydro-consolidation problems in compacted fill have increased due to a number of reasons which include but not limited by the following:

- Infrastructure projects are getting larger, with multiple level interchanges driving greater embankment heights.
- The desire for urban renewal projects such as residential development in disused quarries by backfilling has also driven greater fill thickness. Some of the backfilled quarries are up to 20 m or more deep.
- Developments in the south-west and western area of Sydney, including the Western Sydney Airport, have required the use of locally sourced materials derived from Bringelly Shale. Such fill typically comprises a mixture of residual clay and weathered rock which are more prone to deterioration and breakdown in particle contacts following upon increased moisture content following initial placement and compaction.

The backfilling of disused quarries presents an ideal setting for post-construction creep and hydro-consolidation to occur because the depth of filling and groundwater table could take many years to recover in a low permeability clay backfill. Shown in Fig. 6 is an example of a house located over 18 m of compacted backfill in an old quarry. Despite density test results showing compliance with the compaction specification for the quarry backfill operation, alarming settlement is still occurring at a rate which is close to linear with normal time scale, indicating that the large settlement is likely to be associated with recovery of the groundwater level and hydro-consolidation with time as well as a potential component of creep.

The following mitigation measures should be considered to reduce the risk of creep and hydro-consolidation of compacted fill:

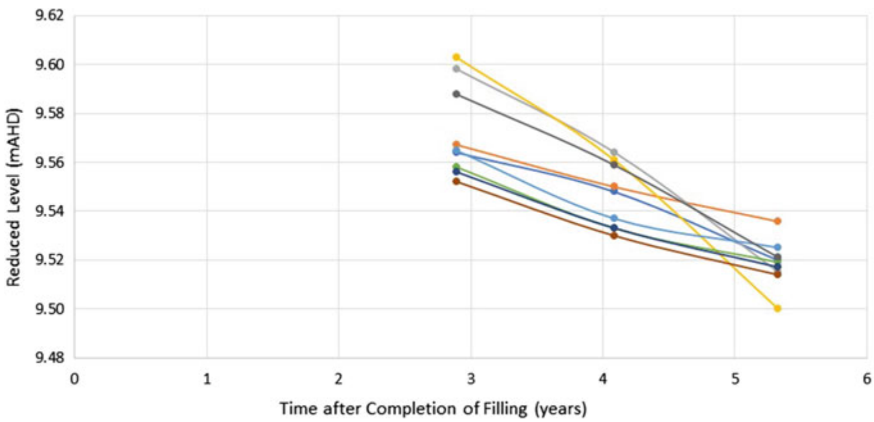

Fig. 6 Example of post-construction settlement of a house built on a backfill quarry. Lessons Learnt (Hydro-consolidation and creep settlement)

- Use material that are less prone to degradation under high stress and/or in the presence of water to reduce volumetric strain associated with hydro-consolidation.
- Avoid placing material with high volumetric strain associated with hydro-consolidation in the zone subjected to groundwater rise following compaction.
- Adopt higher compaction requirements to induce particle breakdown and to reduce void content (e.g. 98% or higher of Standard Dry Density Ratio, and if necessary, use Modified Dry Density Ratio depending on material types).
- If the structure to be supported by the compacted fill is sensitive settlement and differential settlement, adopt a compaction moisture range between Standard Optimum Moisture Content (SOMC) and a few percent above SOMC.
- Place stricter geotechnical supervision during construction to reduce the risk of differential materials placed beneath future structures to reduce the potential for differential settlement.
- Provide a buffer zone on embankment batters constructed using more inert materials to protect the central core from environmental influence of moisture.

With the construction of the Western Sydney Airport currently in progress, the region is likely to undergo significant transformation with future infrastructure development such as rail and road, hospital and schools as well as commercial and residential developments. These are likely to involve significant cut and fill earthworks in the current rural land and fill derived from Bringelly Shale in the region is known to be more prone to deterioration with moisture increase and therefore creep and hydro-consolidation following construction. Muttuvel et al. [8] provide an excellent discussion on the subject of long-term settlement associated with fill derived from Bringelly Shale.

4.2 Shrink-Swell Issues

In relation to shrink-swell issues, the problems are usually associated with incorrect site classification, and in particular, inadequate consideration of the influence of trees in close proximity of the proposed building. AS2870-2011 provides clear warning that the removal of trees could also cause adverse impact to shrink-swell.

Fill compaction problems are rare for fill placed under Level 1 Testing as defined by [9]. However, it is worth stressing that moisture range for compaction as well as dry density ratio should not be overlooked, particularly in material prone to shrink-swell or degradation with moisture changes following placement.

One important point to also stress is that differential settlement associated with deep fills should not be confused with site classifications and deemed to comply standard stiffened slabs given in [10]. This is because the beam dimensions and spacing given in the code for different reactive site classifications are designed to

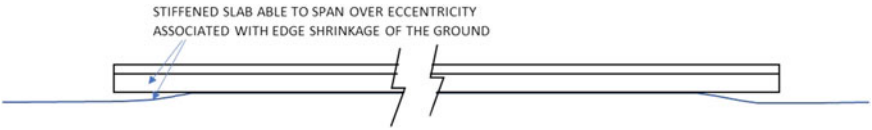

Fig. 7 Stiffened slab designed to span over relatively short eccentricity from shrink-swell movements

span over relatively short eccentricity of mound shapes associated with shrink-swell movements of the ground as shown in Fig. 7. However, if the same ground settlement occurs as a differential settlement over a bowled shaped profile over a large span, the stiffened slab is likely to follow the shape of the settlement profile.

One of the case studies I have come across had a total post-construction settlement of 50 mm which is within the free ground movement, 40 mm $\leq$ ys, $\leq$60 mm, and for which a Class H1 slab which was designed and constructed would have been able to handle such movement if it was caused by shrink-swell. However, the house was built over a deep backfilled quarry, and the settlement profile was such that a differential settlement of 30 mm occurred over a cord length of 10 m. The author suspects that the settlement was caused by hydro-consolidation, and the differential settlement was caused either by a zone of material having greater hydro-consolidation strain potential, and in combination with the fact that settlement in the centre of a uniformly loaded raft is always greater than the edge due to interaction of a continuum, even if the material has uniform stiffness.

As such, the house cracked badly, and consistent with a damage category of 4–5 observed and which could be assessed using the method described by [7] for the case of $\Delta/L = 0.03/10 \times 100\% = 0.3\%$ for a L/H ratio of 1.5 as shown in Fig. 8b.

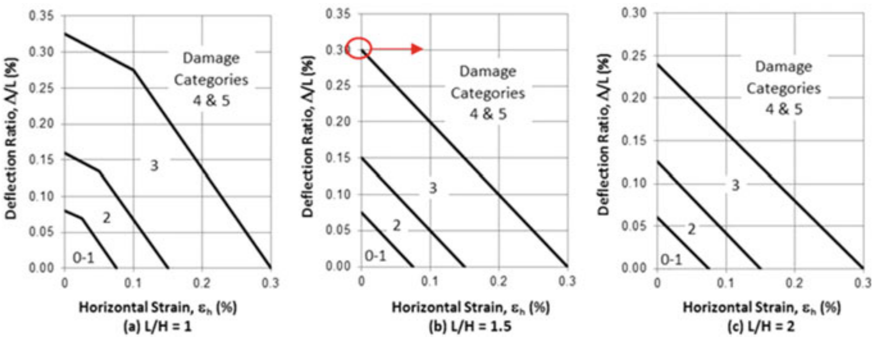

Fig. 8 Damage assessment charts based on [11] extended by [7]

Lessons Learnt (Settlement of Earthworks):

- Post-construction settlement due to creep and hydro-consolidation, particularly in deep fill, should be considered in design of earthworks, and if necessary, adopt mitigation strategies such as those discussed above.
- Site classification of shrink-swell movements must take into account the proximity of trees and timing of tree removal relative to the building construction.
- More attentions need to be paid to the importance of adopting appropriate moisture range for compaction in addition to the level of compaction specified.
- For settlement mechanisms that are not related to shrink-swell movements (e.g. creep and hydro-consolidation, mining subsidence, or load related settlement), equivalent site classes in relation to those given in [10] should not be used unless the potential differential settlement has been assessed to be the same or less than those applicable for shrink-swell movements. An assessment using sound engineering principle should be adopted.

5 Latent Condition Claims

Only limited discussion will be given on latent condition claims in this paper because of the nature of such problems involving different parties who often have opposing opinions on whether a "latent condition" actually existed. Expert witnesses have an overriding duty to assist the Court impartially on matters relevant to the expert witness's area of expertise and are bound by the [12] and adhere to the fact that their duty is to the Court of Law or Arbitration. Unfortunately, this is not the case in the real world, and in the author's opinion, some expert witnesses often taking an "advocate" position on behalf of the party they have been engaged. In the 51 cases the author has been involved in, only one was the author appointed by the Court to assist the judge in his determination, and in another case the opposing parties decided to jointly seek an independent expert opinion from the author.

Needless to say, these matters are usually not "black" or "white". Latent conditions do not usually involve "failures" as such, except perhaps the failure of the parties to recognise the alleged "latent conditions" from the available geotechnical information at the time of tender. Not surprising is the fact that many of these claims relate to earthworks, such as the following:

- More "rock" being encountered than could be anticipated from the available geotechnical data at tender.
- Less "rock" being encountered than could be anticipated from the available geotechnical data at tender. In this case, the Contractor may have assumed that the

"rock" to be excavated could be rebated by the quarry receiving off-site disposed rock.

- Rock was more difficult to excavate than could be anticipated from the available geotechnical data at tender.
- More unsuitable materials having to be excavated and disposed of to spoil than could be anticipated from the available geotechnical data at tender.
- Geological structures such as faults, sheared zones or joint swarms which could not be reasonably identified from the available information at the time of tender.

Therefore, it is important that the Principal should engage competent geotechnical consultants to carry out sufficient geotechnical investigations and clearly articulate the uncertainties and risks in the geotechnical reports. Having said this, however, the author is almost certain from his experience that the occurrence of latent condition claims and the rate of which will continue regardless of how well geotechnical investigations and reports are performed when contractors underbid projects and will try to find any way possible to recover the cost from the principal.

Lessons Learnt (Latent Condition Relating to Earthworks and Other):

- The Principal should carry out sufficient geotechnical investigations and geotechnical reports should clearly articulate the uncertainties and risks.
- Expert witness should take an unbiased view and should not act as an advocate.
- On complex matters, it is the author's opinion that judges or arbitrators should adopt the use of independently appointed experts to assist with their deliberation because such experts would be truly "independent".

6 JET Grouting Induced Ground Movement

The experience of the author in all four cases in this category concerns excessive ground movements and damage to adjacent structures associated with the process of jet grouting. Although jet grouting is supposed to be a non-displacement grout improvement technique, ground displacement can occur when excessive ground pressure build-up in the ground as a result of blockage or restrictions occur in the annulus surrounding the jet grouting stem. This occurs most frequently in layered soil conditions in soft clay with interbedded stiff clay and/or dense sand layers. The risk of large movements increases during the installation of closely spaced jet grout columns. Numerous case studies and investigations on ground movements associated with jet grouting can be found in the literature, and some of these include [13–18].

The problems encountered due to jet grouting has prompted researchers to develop techniques to quantify the potential movement induced by jet grouting, such as those reported by [19, 20].

Specialist contractors experienced in jet grouting are also aware of this problem and can take measures to mitigate the risk of damaging adjacent infrastructure such as that reported by [21]. However, contractors may be engaged as subcontractors by the main contractor of the project and they may not be aware of the staging of other components of the project and/or limits on tolerable ground movement, and thus not in the position to offer appropriate advice. Nevertheless, as a contractor performing groundworks, it is the author's opinion that they have a duty to inform and take appropriate measures to mitigate the risk as directed by the main contractor.

Lessons Learnt (Jet Grouting Induced Ground Movements):

- Recognise that jet grouting could cause ground movements and take appropriate mitigation measures to protect adjacent infrastructure.
- Involve the jet grouting specialist contractor to discuss this issue before commencement of works.
- Adopt a "hit and miss" construction sequence for closely spaced jet grout columns.
- Employ predrilling techniques to increase the size of the annulus surrounding the jet grout stem.
- Adopt appropriate jet grouting sequence relative to adjacent infrastructure to reduce movement at the infrastructure.
- Install pressure relief holes to reduce ground pressure build-up.
- Monitor adjacent ground movement carefully during all stages of jet grouting.

7 Soft Soil Post-construction Settlement Issues

The majority of soft soil problems is associated with post-construction settlement being significantly greater than expected following the removal of surcharge. In this context, "significant" refers to post-construction settlement that is generally greater than 50% of the design estimate and occurring at a rate faster than a log-time scale expected of secondary consolidation or creep settlement, and considered to be excessive due to the potential differential settlement near more rigidly supported structures such as bridges.

In almost all cases, the excessive settlement was a result of the surcharge being removed too soon, and primary consolidation was in fact incomplete. This occurs most frequently in deep soft soils for which the lower layers have lower permeability and are at or near normally consolidated after surcharging, coupled with the failure of downhole settlement extensometers following kinking due to large settlement, or in some cases absent in the monitoring program. In other cases, there is a lack of pore pressure measurements or ignored in the engineer's believe that the observed excess pore pressure is either not valid or due to some other phenomenon unrelated to residual primary consolidation.

Even with the best of soil sampling and laboratory testing, [22] demonstrated through an industry soft soil settlement prediction exercise that prediction of settlement and time-rate of settlement in a deep soft soil profile is extremely difficult and requires a great deal of care. Even when settlement monitoring data is available, [23] indicates that the use of graphical observational methods such as the Asaoka or hyperbolic methods require at least 70% consolidation before a reasonable estimate of primary consolidation can be made from the settlement data. The author's observation on the use of the graphical observation methods in deep soft soils requires more than settlement data from settlement plates because the overall settlement curve may mask the behaviour of the deeper soft soil layers due to the upper layer have reached primary consolidation but the deeper layers have not. Where downhole extensometers are available, the settlement curve between each pair of gauges with depth should be used in the graphical method rather than simply relying on the surface settlement plate. The use of downhole piezometers will further improve the accuracy of prediction. Research on the use of Bayesian Updating to improve the accuracy of future prediction from monitoring data is currently being undertaken at the University of Newcastle via an ARC linkage grant. The Bayesian Updating Method is described in [24].

Lessons Learnt (Soft Soil Post Construction Settlement):

- Adopt appropriate sampling and testing techniques to adequately characterise the compressibility parameters of the soft soil.
- Instrumentation should include downhole extensometers and piezometers. If downhole extensometers fail due to kinking associated with large settlements, consider replacing them if post-settlement is critical for the project.
- Do not ignore excess pore pressures. The presence of excess pore pressure means ongoing primary consolidation is still occurring.
- Consider the use of probabilistic assessment techniques to quantify probability of exceeding design criteria and if mitigation measures would be warranted.

8 Conclusions

This paper has outlined some common geotechnical problems from 51 cases of expert witness cases that the author has been involved in during the last decade. Due to the confidential and privileged nature of these cases, only generalized discussions are given. Many of the problems would appear to be obvious to many of the readers, yet the fact that they materialized indicates the lack of knowledge and/or poor judgement adopted in design. The explanations of the failures and lessons learnt represent the author opinions only and different opinions are likely to exist. It is the author's hope that by publishing more "geotechnical failures" in a non-project specific and generalized fashion would increase the knowledge of the geotechnical community and reduce the occurrences of such failures in the long run.

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The Challenges of Field Measurement of Suction Within Free-Standing Mainline Railway Embankments

Andrew Leventhal, Tim Hull, and Nasser Khalili

Abstract For over 100 years, adequate performance has been observed for free-standing mainline railway embankments despite the typically rudimentary earthworks techniques used. Notwithstanding this performance, use of effective stress limit equilibrium analyses often do not satisfy the design criteria adopted today. Embankments on the Main Southern Railway are frequently up to 20 m high and have been subjected to environmental events; including drought, intense rainfall, flooding on their upslope side and substantial earthquake loading over the last century. In addition, these same embankments can be subjected to the influence of subsidence from underground mining. One feasible explanation of their adequate performance is the presence of suction within the body of the embankments. The phenomenon of suction has attracted much study and is thought to be understood by the geotechnical profession. Nevertheless, recognition of suctions within engineering analysis are seldom attempted. What is known as suction can be measured under laboratory conditions and its presence is accepted in the field. However, its measurement in the field, especially at depth, is technically challenging. There is a dearth of research and reported installations of this nature, particularly for embankments and the authors are endeavouring to correct this. The hope is that this paper will stimulate discussion within the geotechnical profession by providing an update on the authors attempts to measure suction at depth in the field with commercially available instruments, and whilst illustrating the challenges faced.

Keywords Soil suction · Field readings at depth · Mainline railway · Free-standing embankment

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1 Introduction

Whilst perhaps not familiar with its exact nature, the geotechnical community has long recognised "suction" in clay soil as a concept. However, the means of incorporating the concept in practical analyses has not proved to be straightforward. It is fair to say, at least in the authors' experience, that geotechnical engineers lack experience, and hence confidence, in the use of suction in analyses. Accordingly, practitioners will need to gain experience in including suction in analysis to calibrate such use. The authors' endeavour is, in some small way, to further address that situation with this paper, illustrating the steps taken recently to measure suction in the field.

This paper presents:

- some concepts associated with suction;
- a brief discussion on the challenges for traditional soil mechanics instrumentation to measure suction;
- a description of trials of the chosen instrumentation in the field;
- results of measurements taken over a period of a year;
- comments on the uncertainty of suction measurements due to the variability of environmental influences; and
- assess measurement shortcomings to inform the way forward in the measurement of suction.

The work builds on [1, 2] wherein the analyses for assessment of instability of long-standing unsaturated mainline railway embankments were presented for discussion.

This paper is in the form of "*where we are now, warts and all*". A thorough treatment of suction related literature has not been attempted, and the reader is invited to carry out their own review and to challenge some, or any, of the views expressed herein. If that, indeed, does occur this paper will have achieved one of its aims.

2 Suction

Suction in clayey soils has been considered by several disciplines including agricultural science, soil science and soil physics. Soil suction was recognised by Edgar Buckingham circa 1860 and, given the arid nature of Australia, research of soil suction and soil physics progressed well in Australia, e.g., [3], both from CSIRO. However, instances of the adoption of the effects of soil suction in Geotechnical Engineering is relatively recent (see [4]).

A short discussion of suction mechanics is presented below, followed by a short discussion of instrumentation and summarised in some interim notes.

2.1 Concepts—Simplified Picture

Some principles are provided in Hull and Leventhal [1] to which reference should be made. Saturated ground below the vadose zone is well accepted as being associated with a positive and isotropic pore water pressure.

In soil above the phreatic surface, "negative" pore water pressure can arise, i.e., the absolute liquid pressure is less than atmospheric pressure. This "negative" pore pressure is referred to as "soil suction" and is not well understood in a practical sense.

This picture of pore water pressure seems to match observations, e.g., the short life of sandcastles and the much longer existence of steep slopes in clay.

Importantly within an unsaturated zone, although air is present, water remains within the pore spaces, and unless the soil is very dry, the water is continuous—existing as an interconnected film (web or membrane) of water. Hence the remaining water contributes capillary tension forces between particles at contact regions, which becomes apparent as soil suction. It is noteworthy that an assumption that the suction remains an isotropic phenomenon is now unlikely. This is because the pore spaces will likely have a preferred arrangement because of their original deposition and subsequent environmental effects, or due to arrangement because of compaction. Surface tension forces are then unlikely to combine to produce isotropic resultant "pressures". This aspect of suction is beyond the scope of this paper.

The existence of soil suction as a continuum within the unsaturated zone is evidenced by many phenomena, including evaporation and transpiration. In both cases, these phenomena decrease saturation due to removal of moisture; either near surface in the case of evaporation or below ground around roots in the case of transpiration. This creates a localised decrease in pore water pressure, i.e., an increase in soil suction. This leads to an energy differential between areas of high and low pressure, and results in a flow of moisture from wetter to dryer areas and "bypassing" of the air voids. Both phenomena are known to endure over prolonged periods. This supports the concept that pore water in the unsaturated zone is continuous, otherwise these processes would be expected to cease almost immediately once local water sources had been depleted.

Importantly, in some cases, observed behaviour cannot be adequately explained when ignoring suction and techniques were developed to incorporate the effects of soil suction in analysis as discussed by [5–8].

2.2 Reliance on Suction and the Challenges

It might be argued that there is a case to always include the beneficial effects of soil suction in stability analyses. However, to do so relies upon confidence that no, or limited, increase in saturation occurs over time since increase in saturation tends to remove suction effects.

The most obvious mechanisms to reduce soil suction are either rainfall infiltration or raising of the phreatic surface. The observed effect of rainfall on instability was a significant factor in the development of techniques to include soil suction effects in analysis of steep shallow depth slopes in Hong Kong [9, 10].

Experience with cuttings in the dry climate of South Australia has shown observed behaviour can be explained [11] by incorporating unsaturated soil mechanics in assessment of stability of cuttings in South Australian clays.

Long standing cuttings in London Clays eventually suffered instability which can be attributed to extended exposure to the natural environment (e.g., moisture content change due to many cycles of wetting and drying) leading to a loss of strength and behaviour that could also be attributed to creep (e.g., [12]). Cuttings may also be prone to ongoing seepage while free-standing embankments are less prone to seepage saturation.

Soil suction has been relied upon in temporary excavations where negative pore pressures can be generated either by unloading or natural (drying) development of partial saturation. Both effects are recognised to be not always reliable in the long term due to redistribution of excess pore pressure (positive or negative deviation from steady state conditions) and/or infiltration of water. The inclusion of the effects of soil suction in temporary slopes is more commonly based on experience rather than engineering calculation, and frequently includes a nominal (though perhaps often poorly justified) contribution from cohesion.

The reliance on unsaturated conditions continuing and the complications associated with its inclusion in analyses, in the past have been reasons why the effects of soil suction are not commonly incorporated into slope stability analyses. The most common and valid reason that soil suction should be included in analyses is where the observed behaviour of slopes cannot be adequately explained otherwise. This is the case for free-standing mainline railway. Hence the current attempt to measure suction in the field.

2.3 Measurement of Suction

A range of methods exist to measure suction in soils which are generally termed as being direct measurement of the suction in a liquid, or indirect, this latter measuring a quantity other than pressure in the water, and which is calibrated by some means to provide the suction value. The direct techniques involving tensiometers (including [13]) suffer the likelihood of loss of water from the measurement chamber, and hence are challenged.

As explained by [14]:

> If the water contained in the voids of a soil was subjected to no other force than that due to gravity, the soil above the water table would be completely dry. However, powerful forces cause the water to be drawn into the otherwise empty void space in a similar way to how water is drawn upwards into a tube with a small bore by capillary attraction. These forces include molecular, physical-chemical forces acting at the boundary between the soil particles and

> the water, evaporation and transpiration acting at and close to the surface. The combination of these forces gives soil lying above the water table an attraction (or potential) for water that is termed 'soil suction' and is usually represented as negative pore water pressure (although there is no proof that the pressure in the water is actually negative relative to atmospheric pressure). Graphically it is customary to use the water table as a reference, e.g., zero, and to represent the pore water pressures in ground lying above the water table as negative values and an extrapolation of the positive pore water pressures that exist below the water table.

Ridley went on to note that:

> There are a wide range of devices for measuring soil suction, many of which make what are known as indirect measurements, which means that rather than measuring the soil suction directly they measure a related property (e.g. humidity) that is related to the soil suction and convert the measurement to a soil suction using an appropriate calibration. These indirect methods of measurement are comprehensively covered by Ridley and Wray [13].

Ridley [14] presented data on direct measurements of negative pore water pressure using an instrument developed by [13]. That suction probe is a high-quality machined instrument that requires a reservoir of water to maintain its action as a tensiometer. It is assembled dry, then water is introduced into the instrument under laboratory conditions. This is an expensive proposition.

Accordingly, the authors herein have chosen commercially available instruments that measure suction indirectly. The Watermark instruments record resistance recorded across corrosion-resistant electrodes embedded within a granular matrix and contained within the instrument sonde. The resistance is reported by the manufacturer to have been calibrated to suction. These were developed for the agricultural industry and principally used for the management of crop irrigation.

Indirect methods have evolved, primarily associated with agricultural use, and rely often on measurement of moisture content in a soil. Hence moisture in the soil and suction are assumed to be related. An example is where the presence of moisture alters the electrical resistance of a soil, and electrodes in a medium in equilibrium with the soil, which measures this resistance. Calibration is thus required and will obviously depend *inter alia* upon the nature of the soil type.

In the various methods, the junction between soil and instrument is a porous filter. This is the weak link in most instruments, as the system relies upon the moisture changes in the soil to be reflected in the measurement made by the instrument. The interaction of the filter with the total system is likely to influence the reading. This is assumed to be catered for by calibration, but this is not a trivial task—and is vital to the success of the indirect method.

2.4 Interim Notes

Direct and indirect measurement both involve a medium between the soil and the means of measurement in the instrument. Properties such as the air-entry pressure of these mediums influence the measurement either directly by altering the system or by introducing a delay in the time required to obtain equilibrium.

Direct measurement requires extreme care in (read costly) design of the instrument. Traditional water column based tensiometers are limited in the depth of burial of the tip of the probe to the order of 1–2 m because the pressure measuring instrument is located at the top of the water column. Further, the water column requires maintenance, as it can suffer loss of water from its storage reservoir, taking it off-line. Suction probes suitable for application at depth [15], are not readily available as far as the authors are aware, would be costly to obtain and be beyond the budget of most projects.

Indirect measurement instruments have been available for agricultural purposes for some time, do not have the issues associated with cavitation and may provide a low maintenance solution to the measurement of soil suction, particularly at depth. Agricultural applications are normally associated with measurement of suction in terms of the ability of plants to extract water via rootlets and are not required to detect high suctions. In that regard, they are also expected to be useful for the purpose at-hand; which is not to detect the high suctions associated with clays drying. The interest herein is in the detection of suction at much lower values to justify a strength (or stiffness) gain above that measured under saturated conditions.

3 Installation of Suction Sensors at Depth Within Free-Standing Embankment

The site of the free-standing embankment is at chainage 69 km (from Sydney) on the Main Southern Railway (MSR). That railway chainage is near the township of Douglas Park, and the landform is gently undulating across residual Ashfield Shale (Rwa)—see [2] for further description of the geomorphology, whilst noting that the embankment described therein is situated at a railway chainage further south that this test site.

The authors selected Watermark soil moisture sensors (Model 200SS) [16] to trial the efficacy of their performance. Note that the authors declare that they do not have any relationship with the suppliers of Watermark instruments other than by way of standard commercial purchase of the instruments.

The six instruments were installed at depths within the embankment using vertical open hole auger drilling, at a location near the maximum height of the approximately 12 m high embankment. To keep the installation simple, single instruments were installed in each test hole, separated by 1.5 m along track. The depths of installation were from 2 to 8 m below the embankment crest. Three of the sensors were accompanied with temperature sensors. See Figs. 1, 2 and 3 which illustrate the installation.

Five of the sondes (WM-1 to WM-5) were installed in accordance with the manufacturer's advice—being encapsulated within a slurry of borehole cuttings that were placed back into the test hole. WM-6 was installed at the same 4 m depth as its neighbouring sonde WM-5, but by way of a trial of an alternative matrix to surround

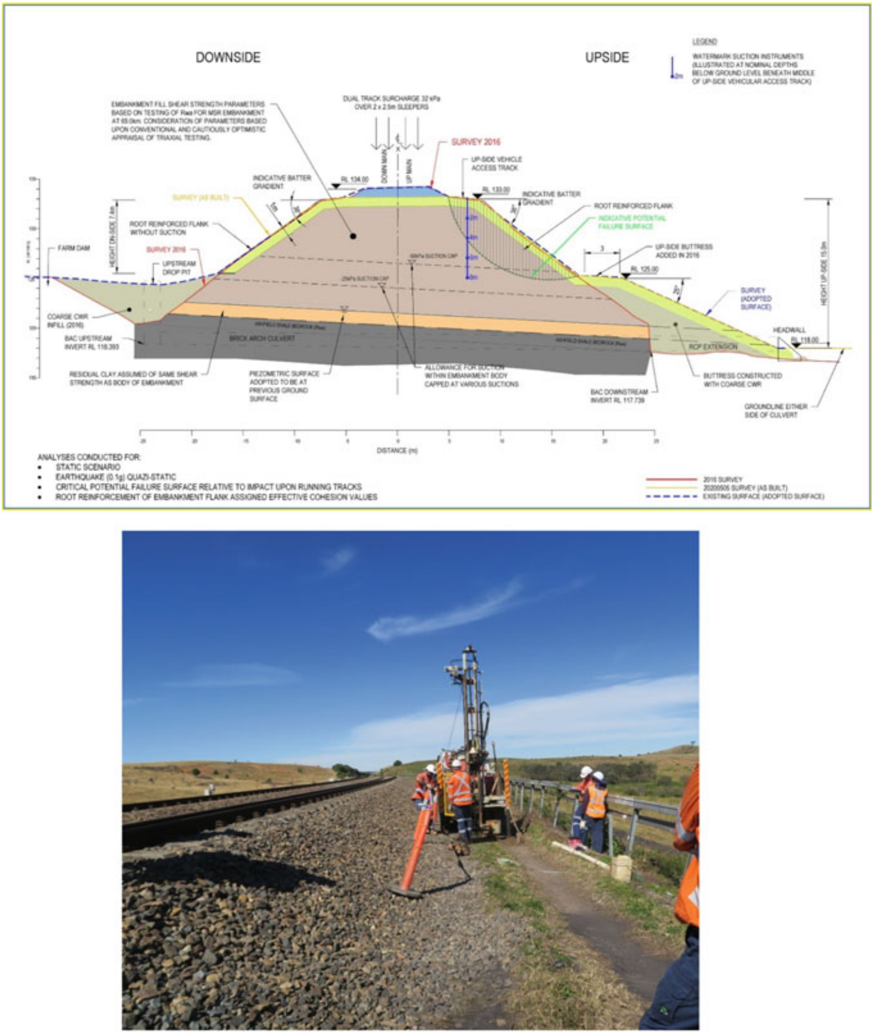

Fig. 1 Type-section at MSR 69.0 km looking DN (top) and view of the site during installation (bot)

the sonde was placed within a pack of dry silica dust. The instruments were installed between the wheel tracks of the access road beside the dual railway tracks. The instruments were attached to a data logger established immediately to the side of the instrument site, which in turn was linked to the existing embankment monitoring instrumentation backbone to permit remote download of monitoring results.

The instruments were installed on 27 and 28 August 2020. Monitoring was live within a week. Modification to improve the protection of the cabling under the access road was conducted on 16 Nov 2020.

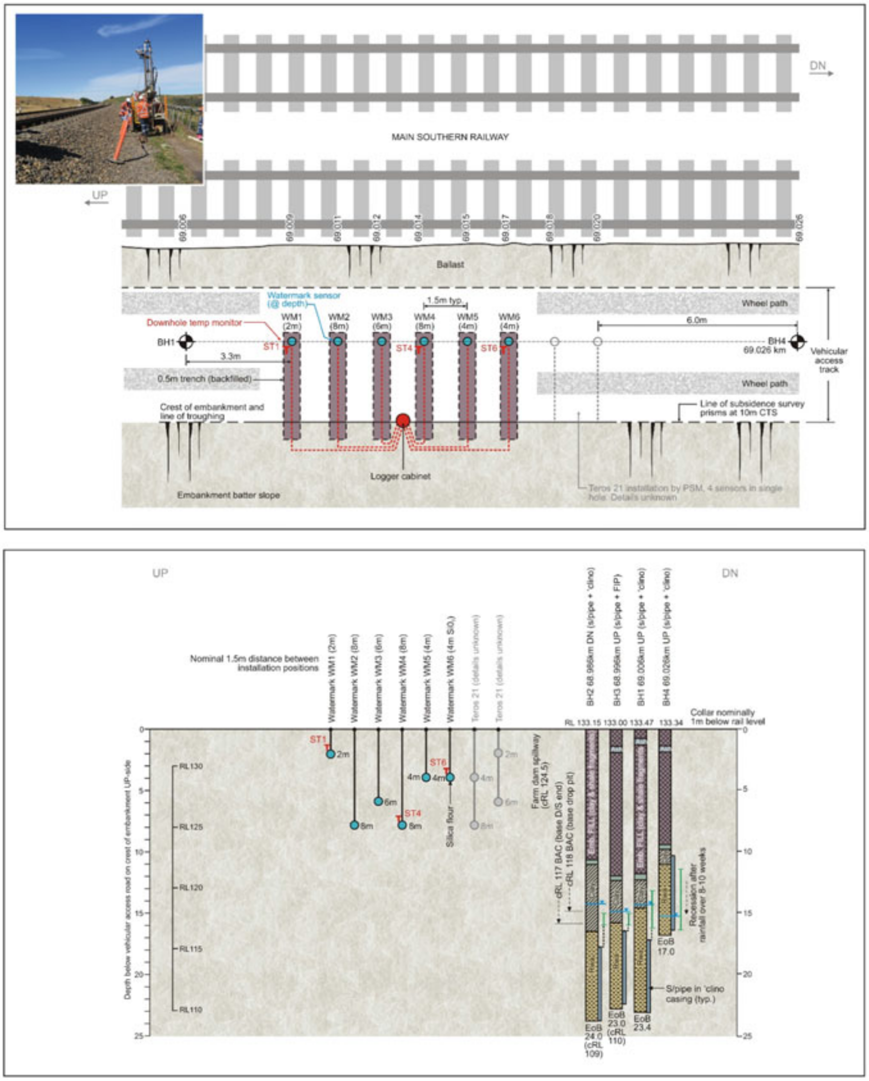

Fig. 2 Plan (top) and elevation (bot) of the Watermark instruments installation at MSR 69.0 km

In addition, a series of Teros21 instruments were installed by others immediately adjacent to the string of Watermark instruments, as shown on Fig. 2. Unfortunately, the installation details of the Terros21 units are not known, though potentially the Teros21 instruments were not saturated prior to installation and backfill into the Teros21 test holes was not conditioned for moisture. In addition, deep standpipes (using the casing of downhole inclinometers—as illustrated in Fig. 2), and shallow tensiometers about the flanks were installed within the embankment as part of the

Fig. 3 Watermark 200SS sonde prepared for installation, (top) with temperature monitor attached to outside of PVC conduit in proximity to the sonde and (bottom) the recommended hole in casing just above instrument

monitoring routine. A weather station fitted with a pluviometer for recording rainfall operated on the other side of the tracks to the monitoring site and recorded rainfall depths (in millimetres) at 5 min intervals.

The installation was funded by South32, Illawarra Metallurgical Coal.

The small hole in the 20 mm diameter plastic conduit supporting the Watermark instrument (as seen in Fig. 3) is recommended by Irrometer [16] in their installation instructions. It is presumably to drain the conduit of condensation or other inflow of water in a shallow depth agricultural irrigation environment where free water may be encountered. Its value in the deep installations at this site is being considered.

4 Results to Date

The Watermark instruments have been monitored continuously from their installation to the present (time of writing being August 2021).

4.1 'Early Days' Results

The '*early days*' results up to the end of October 2020 are presented in the top of Fig. 4 along with a record of the 5-min rainfall less than 30 m away from the instrumented site. At that time, the suction values inferred from the Watermark responses were believable. In Fig. 4, the open circle symbols represent the Watermark readings taken at the time of installation, and the data logged readings commenced some two weeks afterwards. Whilst WM5 shows some lag, the Watermark measurements show a trend towards reaching stable readings after approximately two months. This is consistent with the fact that, except for WM6, the installed Watermark instruments were saturated prior to placement and surrounded by a slurry of the parent material from the borehole. That notwithstanding, the system would still be expected to require some time to come into energy equilibrium between the surrounding soil and the material between the electrodes in the Watermark probe. As noted, the exception is pre-saturated instrument WM6 which was installed in dry silica flour. The increase of suction up to late September 2020 is interpreted to indicate the instrument is "chasing" high suction associated with the dry silica flour, before responding to lower suction levels being achieved through the silica flour at the sensor equilibrating with the soil.

As well as the adopted six indirect measurement Watermark instruments installed by the authors, there were a series of four direct measurement tensiometers T5–T6 at shallow depth on the flanks of the embankment, and four indirect measurement Teros21 instruments that were installed by others immediately adjacent, as shown on Fig. 2. Unfortunately, the installation details of the Terros21 units are not known. It is inferred from their readings that the Teros21 instruments were not saturated prior to installation, and that backfill into the Teros21 testholes was not conditioned for moisture.

The distribution of Watermark suction values plotted with depth in the bottom section of Fig. 4 is believable; with the dashed line showing the water pressure if the soil were saturated and the phreatic surface was at 14 m deep and the dotted line is reflecting that the suction may cease to be present somewhere between 1.5 and 2 m depth.

Two of the Tensiometers were found to not be recording while the other two, T7 and T8, reported a reading between 20–40 kPa, see Fig. 4. Both showed a daily variation of suction which was thought to be a result of environmental effects on the recording system rather than a true measurement of suction in the soil varying with temperature.

During this time however, the Teros21 results were considered not reliable, and likely were well short of reaching equilibrium with results recording less credible values. This can be observed in Fig. 5.

The Teros21 instruments do not track on Fig. 4 because the instruments are recording suction values that are considered to be unrealistically high (being several MPa) for this type of fill.

By way of example of the challenges observed with the Teros21 recordings: (i) The instrument T21-1 @ 2 m starts recording a suction beyond 1 MPa and rapidly

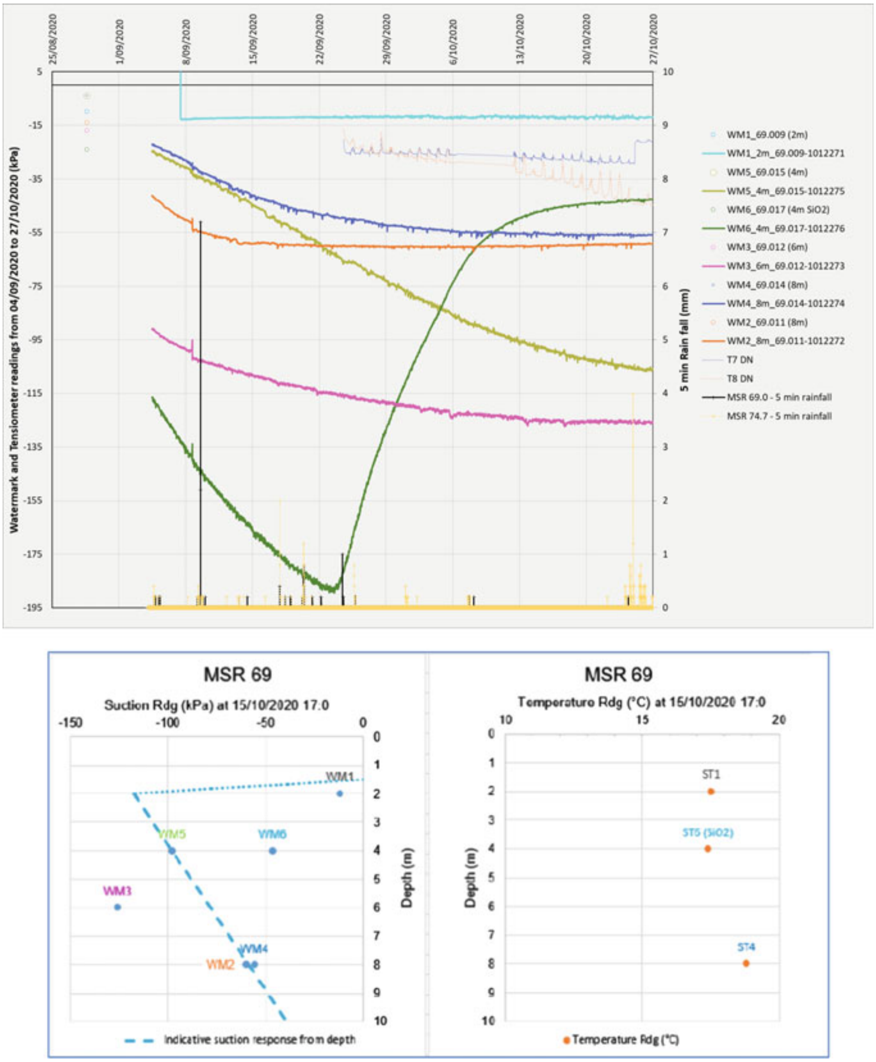

Fig. 4 Results: (top) History of MSR 69.0 km suction instrument readings and rainfall—6× Watermark (WMx) and 2× Tensiometer (TX DN). [Note that, at the reasonable scale adopted, the Teros21 instruments do not plot given the MPa values they recorded]; (bottom) The suction values as of 15 October 2020 plotted with respect to depth within the embankment

reduces to record about 10 kPa suction. (ii) T21-2 @ 4 m and T21-3 @ 6 m start recording suction readings beyond 1 MPa. (iii) T21-2 @ 4 m readings reduce to be about 10 kPa while T21-3 @ 6 m is still approaching 1 MPa. (iv) T21-4 @ 8 m starts at about 460 kPa and reduces to about 240 kPa. At the end of the logged data, two instruments appear to be recording a stable 10 kPa suction at 2 m and 4 m depth, one is recording 240 kPa at 8 m depth and the other 1 MPa at 6 m depth. Note

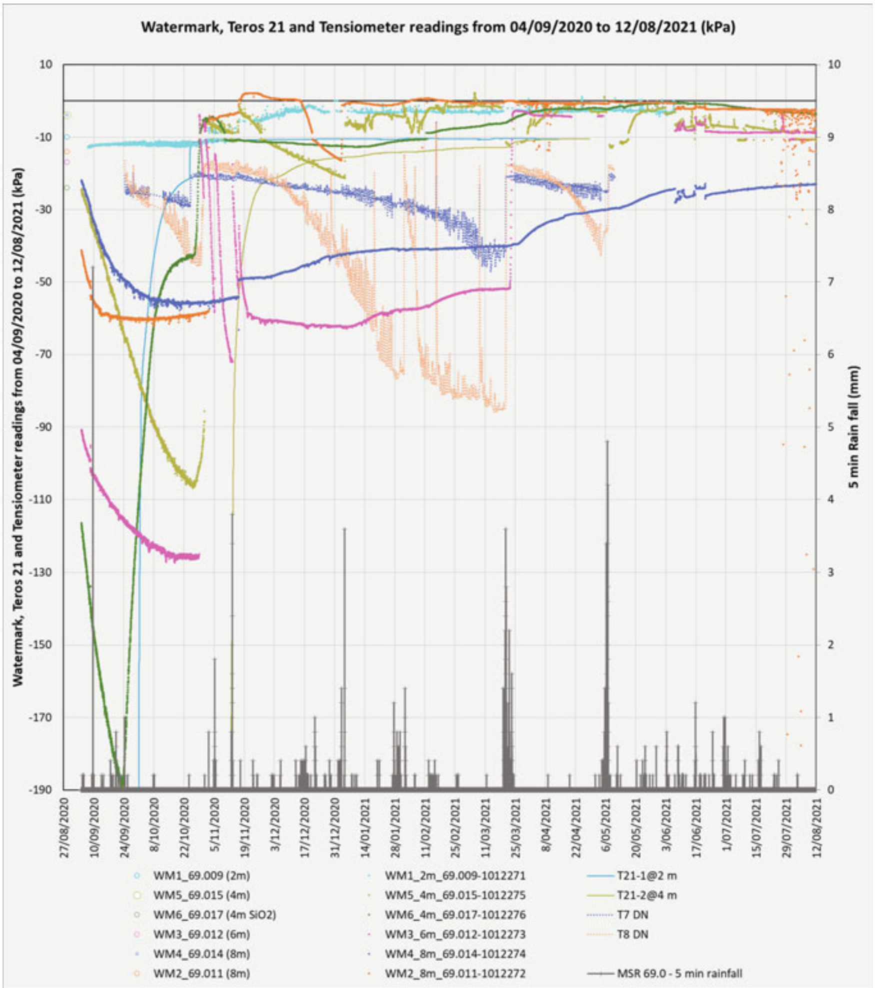

Fig. 5 History of MSR 69.0 km suction instrument readings and rainfall—6× Watermark (WMx), 2× Teros 21 (T21-x) and 2× Tensiometer (Tx DN)—at a scale which does not plot two Teros21 instrument results, none of which have been read since April 2021

that the logging was managed by others, and routinely not available for independent assessment. On this basis, it was considered that the Teros21 instruments did not provide reliable results during the reporting period.

Furthermore, pressure plate testing conducted by the authors upon Teros21 units indicated wide variation between units, and suction readings that were inconsistent with supplied calibration data. On this basis, it was considered that the Teros21 units appear to operate by way of recording differential results rather than recording a measure of suction values per se.

There had been several minor rainfall events at the monitoring site during this period with no major response in the instruments.

4.2 Results Post Rainfall of Late October 2020

The results following the rainfall of late October 2020 are "challenging". It is interpreted that the rainfall (which was assessed to be an event with an Annual Exceedance Probability, AEP = 63%, being approximately equivalent to an Average Recurrence Interval, ARI, of 1 yr) permitted surface water to penetrate and modify the moisture conditions around the tip of the sonde. The options considered include: (i) water penetration from the surface by way of a localised high permeability zone as a consequence of the end-tipping construction technique. Therein, local inclined high permeability zones are proposed to provide permeation of water through the body of the embankment however this is not consistent with the response time identified; (ii) penetration to the sonde by way of short-circuiting through the protective conduit plumbing for the electrical connection from the sonde to the logger. Groundwater has been measured within the underlying shale—its rise into the body of the embankment to the location of the sondes is barely conceivable though of low likelihood and not fully discounted.

The first option is considered the least likely, principally because of the necessity for such a layer to be uniformly inclined across the footprint of the instruments, though cannot be discounted out-of-hand. To address the second option, the protection of the conduit was upgraded in mid-November 2020. Unfortunately, should moisture have penetrated into the body of the embankment to the locations of the sondes, it was anticipated that the return to previous conditions would take an extended time for recovery. During this exercise, evidence of moisture was not confirmed within the conduits.

The possibility of logger issues contributing to the adverse rate of recovery of the readings was also considered, but this has not been resolved. This is in the context that the Watermark instruments employ a resistance measurement system and can be subject to stray earth loops and currents at the logger.

Figure 5 shows the challenges. These include:

- Watermark instruments WM1 and WM2 have always presented erratic readings which are believed to be linked with the method of logging employed.
- The remaining Watermark instruments are perhaps showing that they were installed at a time of relative drought and are now recording lower suctions as rainfall events continue to occur.
- Rainfall impacts are observed and are broadly consistent with adverse responses (suction reading reductions) in the Watermark and Teros21 instruments, certainly with the four main rainfall events.
- The loss and recovery of suction recorded in the Tensiometers confirms the caution to be shown at shallow depths on the flanks of such embankments, which is

potentially a consequence of interaction with the vegetation and the environment external to the embankment. Having said that, the results for DN-side Tensiometers T7 and T8 are higher than all but WM4 at 8 m depth.
- The use of the "drainage" hole in the conduit just above the Watermark instrument was included in all installations, in accordance with the manufacturer's recommendations. In hindsight it would appear that it may have worked in reverse to its intended function in this instance by and adding water to the backfill around and in proximity to the sonde. However, the presence of water in the conduits was not confirmed during the retrofit. This is to be investigated.
- Neither the Watermark nor the Teros21 instruments responded with a daily variation in reading in the same manner as the Tensiometers. It is not clear if the Tensiometer variation is a true suction response that might be linked to rootlet induced suction or the impact of temperature change on the measurement system employed.
- A large eucalypt exists beside the embankment about 25 m from the monitoring site. The influence of rootlets around this large tree upon the suction regime has yet to be appraised.

5 Conclusion

The measurement of soil suction for geotechnical purposes is difficult enough under laboratory conditions. The awkward truth is that the measurement of suction within the field, at depths meaningful for instability assessments, remains challenging.

The presence of suction within the embankment is inferred through the calibrated Watermark sensors. The challenge is reliability of the installation methodology to exclude the potential for introduction of unwanted water from the surface to the sensors. The authors continue their endeavours to obtain reliable measurements within the bodies of geotechnical 'structures'.

The authors note that **t**echnical **a**dvances **r**arely **d**evelop **i**n **s**eclusion from real-world situations and as such monitoring provides support for adoption of available unsaturated soil mechanics principles in the assessment of the behaviour of free-standing railway embankments and elsewhere.

Acknowledgements The authors gratefully acknowledge South32, Illawarra Metallurgical Coal for their encouragement in the technical pursuits presented herein, and for the use of geotechnical results generated on their behalf.

Request of the Reader

Research continues to improve understanding of the underlying nature of soil suction and develop modern methods of describing the important measurements from testing that are needed for assessing its beneficial and reliable impact. However, they will remain untested if the profession does not embrace the fundamental concept of relying on the existence of soil suction in certain natural situations and with additional management conditions imposed relying on suction in man-made scenarios.

What is asked of you, the reader, is to "join in this discussion, with the intention to advance the science". By no means do the authors pretend to have all (or perhaps any) of the answers, and we look forward to your contributions.

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Excessive Post-construction Settlement of Improved Ground—Case Histories

Kim Chan

Abstract This paper presents a few case histories where excessive settlements were reported following ground improvement of a number of soft soil sites. The case histories involved different ground improvement techniques such as preloading and surcharging with and without prefabricated vertical drains (PVD), deep soil mixing, concrete injected columns and vacuum consolidation. The ground conditions and the adopted ground improvement designs are discussed. The observed post-construction settlements for the various cases are also presented. Further, the back-analysis works are detailed in order to provide some insight into the possible contributing factors to the measured excessive settlements. It is clear from these case histories that observational approach by monitoring the ground behaviour during and after ground treatment and construction should be adopted to ensure that the post-construction performance is consistent with design expectation. In addition, the paper demonstrates that selection and design of the ground improvement techniques should be conducted with clear understanding of the theoretical background and limitation of the improvement techniques, regardless of the system adopted. Consideration of construction activities and staging is also important in order to capture the impact of various construction loading on soft soil consolidation and settlement.

Keywords Ground improvement · Soft soils · Case histories · Settlement · Back-analysis

1 Introduction

As suitable construction sites with favourable ground conditions become relatively scarce, less desirable sites sometimes with problematic soils are required to be utilised for construction. The options to deal with sites with problematic soils could include designing the structures to cope with the challenges associated with problematic soils,

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removing and replacing the problematic soils with suitable materials and improving the soils using an appropriate ground improvement method. Depending on considerations of various constraints, it is sometimes preferable to adopt ground improvement techniques to treat the problematic soils prior to construction.

There are many documented case histories where ground improvement techniques have been implemented successful to treat problematic soils such that various infrastructures have been constructed with satisfactory performance. The ground improvement techniques used successfully included various soft treatment methods (e.g. preloading/surcharging, vacuum consolidation, electro-osmosis, light-weight fill, etc.) and many hard treatment methods (e.g. piled embankment, concrete injected columns, shallow and deep soil mixing, stone columns, jet grouting, etc.). Nevertheless, there are far fewer reported case histories where the performance of the infrastructures did not meet the design criteria following ground improvement.

The author has come across a few projects over the years where the post-construction settlements of the treated soils did not meet the specified design criteria. This paper documents a few examples where post-construction settlements exceeded the nominated settlement limits. These examples covered a number of different ground improvement systems. While it is more often than not that the exact causes of the underperformance cannot be pinpointed, this paper serves to demonstrate that many ground improvement systems can sometimes underperform for various reasons. It is therefore up to the designers and constructors to be cognizant of the potential issues associated with various ground improvement techniques such that allowance and contingency planning can be made during the early stages of the project, regardless of the ground improvement techniques adopted.

Following are a few case histories where the post-construction settlements exceeded the prescribed settlement limits.

2 Case Histories

2.1 Case History 1

2.1.1 Brief Description of Project

The expansion of a coal terminal required that the coal stockyard be constructed on reclaimed swamplands and soft soil sediments. The general geotechnical model of the stockyard areas consisted of dredged sand fill underlain by soft soil deposits over dense sands and stiff clays at depth. The underlying bedrock was taken as a nominal 50 m depth. The geotechnical section along the stockyard is shown in Fig. 1.

The natural soft soil deposit was sub-divided into 2 main units, namely an upper sandy clay and a lower high plasticity clay. The thickness of the soft soil strata was generally up to about 4 m, except the northwestern region where the thickness was up to about 15 m. Details of the properties of the soft soil units are given in [1].

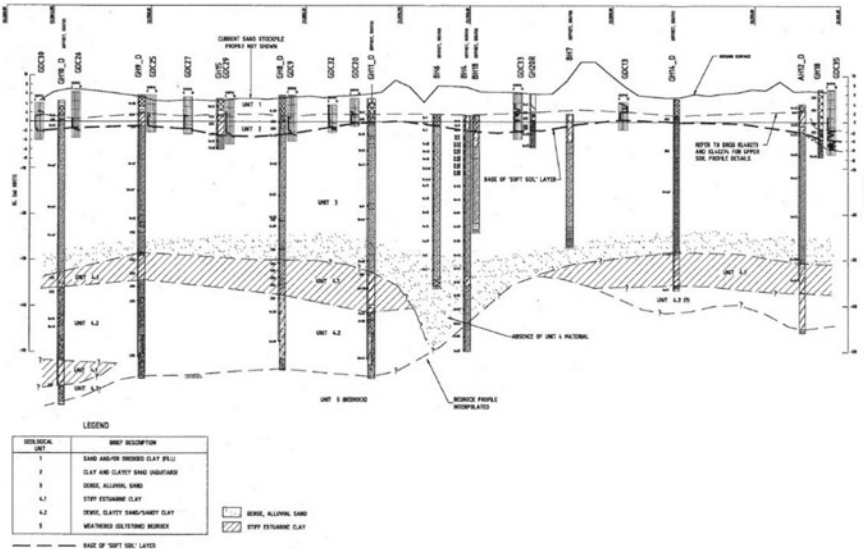

Fig. 1 Geotechnical section along the stockyard

2.1.2 Ground Improvement Design

The chosen ground improvement technique was to preload the stockyard areas with dredged sand fill. The adopted preloading strategy was to initially fill the entire new stockyard area to approximately the cut/fill balance of the final stockyard profile. A moving dune technique and static preloads were used above this profile to apply the necessary surcharging impact in different areas. The designed fill thickness varied depending on the stockyard chainage but was typically about 3.5 m. The moving dune consisted of a 5 m high sand mound across the full width and was shifted (unloaded) across the designated area, after a waiting period of about 7 days to achieve a degree of consolidation of about 90%. However, in the northwestern region where the underlying soft soil was relatively thick, a static preload was applied up to a height of about 15.5 m total thickness. This static preload was placed in stages for stability consideration, and the rate of consolidation was accelerated by using prefabricated vertical drains (PVD) installed to the base of the soft soil. The overall static preload period was up to about 10 months depending on the degree of consolidation achieved.

During design, the total primary settlement of the site under the preload was estimated assuming 1-dimensional loading to specified fill levels. In addition, an oedometer approach (in terms of compression ratios) was used for the prediction of settlement in the upper soft clays whilst an elastic theory (in terms of Young's modulus, E') was adopted for settlement estimation for the underlying strata. The settlement contribution of the lower dense sand and stiff clay strata was estimated generally from SPT data. The validity of this assumed elastic lower model was to be confirmed during construction by downhole extensometer settlement monitoring.

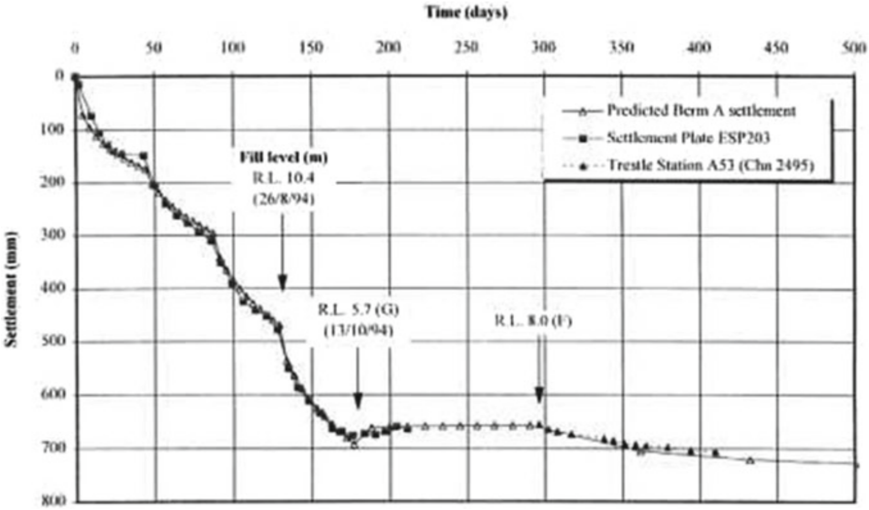

Fig. 2 Typical preload and trestle settlement monitoring results

2.1.3 Preload Monitoring Results

The performance of the preload was monitored using settlement plates, downhole settlement extensometers, borehole inclinometers and pneumatic piezometers. In addition, CPTu testing was carried out at various stages of preloading in order to assess the gain in undrained shear strength of the soft soil. In summary, the preload monitoring results were generally consistent with the predicted behaviour. In particular, the total settlement measured was in good agreement with the estimated settlement values. A typical settlement versus time curve for a settlement plate in the static preload area is shown in Fig. 2, for illustration purposes.

2.1.4 Post-construction Trestle Settlement

Subsequent to the construction of the stockyard, certain trestle footings on the machinery berms were monitored for movement. In less than a couple of years, the contractor's geotechnical designers reported that the settlements occurring at the trestle survey marks were in excess of prediction. In addition, they noted that the results exhibited an "unusual" quiescent period for the initial readings before the settlement started to increase. Unfortunately, the available monitoring data was limited. In particular, "gaps" in monitoring data existed between the reclamation/preloading stage and the commencement of trestle monitoring (Refer Fig. 2). In addition, the downhole settlement information was limited in extent and period of operation. A definitive assessment of the stockyard behaviour was thus not possible.

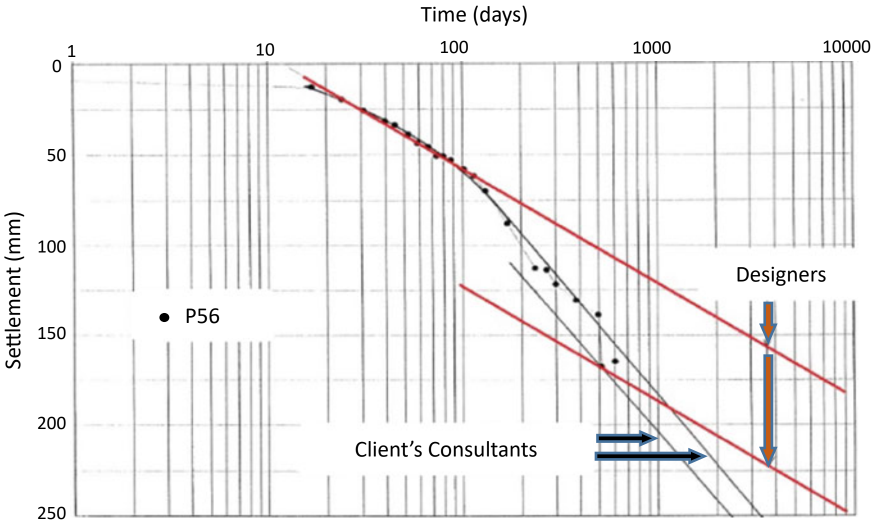

Fig. 3 Typical plot of trestle settlement together with two ranges of future predictions

Both the contractor's geotechnical designers and the client's geotechnical consultants performed independent predictions of future trestle settlements for the design life of 25 years. The contractor's designers predicted 13 out of 21 locations would exceed the specified post-construction settlement limit of 150 mm. Conversely, the client's consultants estimated that 15 locations would exceed the long-term residual settlement criteria. A typical plot of trestle settlement together with the 2 ranges of future predictions is shown in Fig. 3.

2.1.5 Deep Clay Assessment

The predicted settlements were separated into 2 components (upper soft soil and deep clay) during back-analysis by the client's consultants. The upper soft soil creep rate was estimated as the product of the upper soft soil thickness at each location and a likely composite soft soil creep rate of 1.0% per log time cycle. The difference between the monitored trestle settlement rate and the upper soft soil creep rate was then taken as the net settlement rate from the deep clay contribution. This net deep clay settlement rate was used together with the deep clay thickness to calculate the unit creep rate of the deep clay.

The deep clay stratum was of (Pleistocene) estuarine origin formed during a marine transgression some 140,000 years ago. It was located below the lower sand unit at about RL-30. The deep clay thickness varied from over 15 m at the eastern end of the site to non-existence near the middle section before increasing to over 25 m thick at/beyond the western boundary. This deep clay unit can be sub-divided into 2 sub-units. A stiff, high plasticity clay underlied the lower dense sand at about

RL-30. This sub-unit was up to about 7.5 m thick. Further, a layer of sandier material lied below the deep high plasticity clay sub-unit. It varied in composition from sandy clay to clayey sand using the USC system. It was generally stiff/dense and was interlayered with sand and gravel layers.

Based on the oedometer results, the over-consolidation ratio (OCR) of the deep clay at the time of initial site investigations varied from about 1.0–1.9 with an average of 1.2. The deposit thus appeared to be only lightly over-consolidated. Further, this OCR value related to the prevailing ground surface level of about RL 5 m to RL 6 m at the borehole locations. The inferred pre-development OCR (with ground surface at about RL 1.5) was therefore about 1.5.

From the shear vane test results, the corresponding in-situ undrained shear strength, s_u ranged from about 90–145 kPa. The corresponding in-situ strength ratio, s_u/σ_v' was between 0.26 and 0.40. For the adopted normally consolidated strength ratio, So of 0.23, the inferred OCR value was between 1.12 and 1.73 with an average value of 1.30. This value of OCR was consistent with the oedometer results discussed above.

The average creep to virgin compression ratios, $c_{\alpha\varepsilon}/c_c$ of the two deep clay sub-units (4.1 and 4.2) were 4.5 and 2.8% respectively. These values are consistent with the published values for clayey and sandy soils respectively [2, 3]. It was therefore expected that the combined creep rate of the deep clay could vary between about 0.2% for a heavily over-consolidated condition to 1.3% when the deep clay is normally consolidated. The trestle monitoring results showed that the inferred deep clay unit creep rates generally were within the above expected range. In addition, apart from a few low values, the unit creep rates were typically between 0.5 and 1.0%. The high unit creep rates inferred that the deep clay was lightly over-consolidated which is consistent with the laboratory (oedometer) results. The final low OCR implies that the on-going creep rate of the deep clay was higher than that prior to the stockyard construction when the OCR was higher, thus resulted in higher settlements than expectation.

2.2 *Case History 2*

Case History 2 involves the twin bridges over a major creek in the Northern Rivers region of New South Wales. The area in the vicinity of the bridge site is underlain by soft soils. The ground improvement system adopted for the approach embankments of the bridges was preloading and surcharging with PVD to accelerate the soft soil consolidation process [4].

2.2.1 Ground Improvement Design

The ground conditions at the southern abutment of the twin bridges included a sequence of Holocene sediments of 8 m thickness, comprising a thin desiccated clay layer over soft to very soft alluvial clay of 7 m thickness underlain by up to 30 m thick Pleistocene soils dominated by stiff to very stiff clay over a layer of medium dense to dense sand.

The ground improvement design involved a "soft" treatment solution utilising preloading and surcharging with the incorporation of PVD. The designed embankment fill thickness and surcharge fill thickness were 4.8 and 7.2 m respectively. The PVD were installed to approximately 10 m depth with 1.5 m spacing in a triangular arrangement. The preload waiting period was approximately 1.5 years.

Prior to the placement of embankment and surcharge fill, the bridge abutment site was installed with a range of geotechnical monitoring instrumentation including settlement plates, vibrating wire piezometers (VWP), extensometers, hydrostatic profile gauges (HPG) and inclinometers.

2.2.2 Hold Point Release for Preload/Surcharge Removal

The hold points for removal of the preload/surcharge were released following the preload waiting period. Prior to releasing the hold points, the available monitoring data were reviewed. The following points were noted:

- The HPG and VWP data at the bridge abutment site were deemed to be unreliable. These data were not included in the assessment for hold point release.
- The extensometer data were limited and available for a 4-month period only, and the settlement data from the start of filling was also unavailable. As such, it was not possible to derive the relevant compression parameters for the different subsoil layers using the extensometer data. The extensometer data were used only for the assessment of the degree of consolidation using the Asaoka [5] and hyperbolic graphical methods (e.g. [6–8], etc.) for an intermediate loading stage only.

The removal of the preload/surcharge fill occurred in March 2018 after the hold points were released. Following the removal of the surcharge, the abutment fill was further excavated to approximately 3.6 m below the final surface level (FSL) for the construction of abutment piles and headstock. This was followed by the construction of embankment backfill, bridge approach slab and the adjoining plain concrete pavement (PCP). The excavation to RL 1 m (i.e. 3.6 m below FSL) was extended south with an extensive length of over 25 m for ease of construction activities.

2.2.3 Post-construction Settlements

In mid-October 2019, it was noticed that the approach slab sealant had failed due to movement in the slab. Cracking on the eastern F-type barrier was noticed shortly after. Survey monitoring between June 2019 and November 2019 indicated that the bridge approach slab had settled 50 mm where it tied into the PCP. There was virtually no movement at the expansion joint of the approach slab where it is hinged to the bridge abutment. The bridge abutment itself had been monitored continuously with negligible settlement since construction as it is supported by piled foundation.

Subsequent survey monitoring (carried out in November 2019) and comparison of surface levels with design or "work as executed" levels indicated that the settlement was widespread within the general embankment. The general embankment settled an average of about 20–35 mm between June and November 2019. For the same period, the settlement along the 100 m zone located south of the southern abutment of the twin bridges gradually increased up to about 60 mm.

Further survey up to June 2021 indicated settlements up to about 132 mm at the end of the approach slab.

2.2.4 Post-construction Investigation

A number of potential causation factors of the recorded excessive post-construction settlement were considered, including more adverse properties of the soft soil than those adopted in design, such as high compressibility, low over-consolidation ratio, low permeability and high creep rate, as well as significant smearing of the PVDs and changes in groundwater level.

A geotechnical investigation campaign comprising 10 CPTus was conducted in November 2019 following the excessive approach slab settlement was identified in order to assess the prevailing conditions of the soft soil and to gain better understanding of the ground conditions. The following points can be drawn from the CPTu results:

- Inspection of the borehole logs for the pre-drilled holes at the embankment CPTu locations indicated that the as-placed fill thickness was up to about 5.8 m. This is consistent with the sum of the embankment fill height above original ground level and the recorded settlement plate readings. Groundwater was typically at about 0.5 m below the original ground level. Therefore, part of the fill has settled below the groundwater table.
- After the application of fill embankment load, the embankment CPTus consistently showed an upper soft clay layer (about 6 m thick) with higher corresponding cone tip resistance values (q_t) compared to those of the CPTus away from the embankment. This is considered to be due to soft soil compression and strength gain under the applied preload and fill load. For the middle and lower clay layers, however, the embankment CPTus showed results similar to those of the CPTus away from the embankment in terms of layer thicknesses and measured cone resistance values.
- The undrained shear strength (S_u) of the clayey soils was inferred from the q_t values by adopting a cone factor (N_{kt}) of 16. Subsequently, the pre-consolidation pressure (σ'_p) of the soils was assessed from the inferred S_u values based on the SHANSEP approach.

- The assessed σ'_p values versus depth for a typical CPTu plot is presented on Fig. 4. Also shown in this plot are the final effective vertical stress ($\sigma'_{v\ final}$) with soil depth under the as-placed 5.8 m thick embankment fill.
- All the embankment CPTu results showed that the σ'_p value within the middle region (about 2.5–2.8 m in thickness) of the upper soft clay was lower than the assessed $\sigma'_{v\ final}$ value. This indicates that as of November 2019, the middle section of the upper soft clay layer underneath the embankment (treated by PVD) was still undergoing primary consolidation. The difference between the $\sigma'_{v\ final}$ and σ'_p values in this section gives an estimate of the excess pore water pressure remaining that needed to be dissipated.

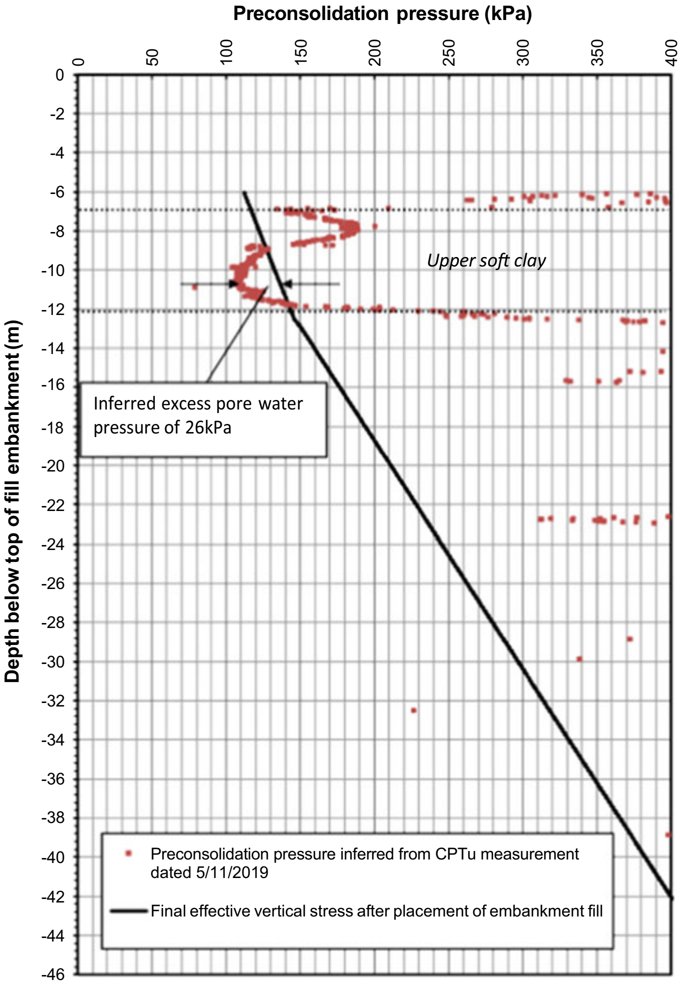

Fig. 4 Typical plot of assessed σ'_p and $\sigma'_{v\ final}$ with depth

2.2.5 Back-Analysis

A 2D finite element analysis (FEA) using the commercially available software program PLAXIS 2D was carried out for the back-analysis of the foundation settlements. The salient features of the FEA are as follows:

- Large strain computational method with updating finite element mesh and pore water pressure was adopted. This computation method accounts for the increase in unit weight of soil elements by considering change in void ratio as well as the buoyancy effect as the embankment fill settled below the groundwater table.
- Following the hold point releases, bulk excavation took place to about 3.6 m below FSL over a 25 m length. This bulk excavation differed from the assumption made in the design of removing surcharge to FSL only.
- Various literature (e.g. [9]) indicated that the ratios of creep strain rate for soil in the over-consolidated state to the creep strain rate for soil in the normally consolidated state calculated by means of soft soil creep model implemented in PLAXIS 2D are reasonably consistent to that proposed by Mesri et al. [2].

From the post-construction CPTu information, it is evident that the excess pore water pressure within the upper soft clay was still relatively high at the time of CPTu testing. The elevated excess pore water pressure could be due to either the dissipation of excess pore water pressure being much slower than anticipated prior to the hold point releases, or excess pore water pressure being regenerated as a result of reloading at the time of embankment backfilling. It is noted that in the absence of reliable pore water pressure monitoring data at the time of hold point releases, likelihood of slow dissipation of excess pore water pressure could not be identified prior to preload removal.

The slow excess pore water pressure dissipation rate could be attributed to the following two main reasons:

- Much lower permeability/coefficient of consolidation values for the upper soft clay compared to those assumed in the design and hold point release back-analysis.
- More severe remoulding of the upper soft clay caused by the PVD (which were closely spaced) installation than previously anticipated.

It is noted that the slow dissipation of excess pore water pressure is unusual given the fact that this clay layer had been treated with PVD to accelerate the dissipation rate. Further, the other PVD treated areas for the rest of this section did not experience the same magnitude of settlement despite the fact that the surcharge effort was relatively low, i.e. surcharge fill thickness was less than that at this abutment.

The regeneration of excess pore water pressure was considered likely to have resulted from removal of an excessive amount of the embankment fill (excessive in terms of the length/area excavated and the depth of excavation removed from the immediate construction area of the piles and the headstock), exacerbated by the extensive delay in backfilling behind the headstock.

Figure 5a presents a plot of the assessed excess pore water pressure for the time immediately before surcharge removal from the FEA carried out on the cross section.

The predicted maximum excess pore water pressure within the upper soft soil treated with PVD was 95 kPa under the applied total fill thickness of about 8.2 m, i.e. equivalent pressure of about 164 kPa.

It is noted that the surcharge level at the time of hold point release was RL 7.65 m while the design level was RL 4.80 m. The removal of the surcharge of 2.85 m thickness would have caused the maximum excess pore pressure to drop instantaneously by around 57 kPa to about 38 kPa.

However, the subsequent removal of the abutment fill to the headstock underside level of RL 1 m (about 6 m of fill excavated) could have caused the generation of negative excess pore pressure (or suction) of −25 kPa within the upper soft clay pressure (Fig. 5b). This assessment is considered to be reasonable given removal of the abutment fill is equivalent to a reduction of about 130 kPa pressure.

The dissipation of this negative excess pore pressure occurred rather rapidly such that at the time just before reloading following pile and headstock construction, the excess pore water pressure was predicted to have reduced to −7 kPa (Fig. 5c).

This means that the benefit of having soil suction after fill excavation has eroded and upon refilling back to the design level (i.e. placement of about 3.6 m thick fill), a maximum positive excess pore water pressure of +65 kPa (= −7 kPa + 3.6 M × 20 kN/m^3) would have been generated.

Note that if there was no loss of suction or if the preloaded fill was stripped to the design level only without excavating down further to the headstock underside level, then the maximum excess pore water pressure upon reaching the design level would have been reduced to +38 kPa (= 95 kPa − 2.85 m × 20 kN/m^3). In that instance, the elevated excess pore water pressure would have been lower than that currently experienced, leading to significantly less settlement than that recorded.

Figure 5d presents a plot of excess pore water pressure at the time of CPTu predicted by FEA. The predicted maximum excess pore water pressure within the upper soft soil treated with PVD was 23 kPa, and the predicted σ'_{p} value of this un-consolidated clay beneath the middle of the fill embankment was about 95–100 kPa. The assessed σ'_{p} values at the time of CPTu were in general agreement with the values inferred from the CPTu results.

As detailed above, an under-consolidated clay layer experienced the extensive excavation of embankment fill and backfilling after about 9 months. Hence, primary consolidation has been re-initiated within the under-consolidated layer during the backfilling resulting in unusual settlement due to primary consolidation and the associated high creep rate. This impact would have been aggravated due to the slow rate of pore water pressure dissipation that could not be observed or detected without the reliable pore water pressure monitoring data during the original preloading period.

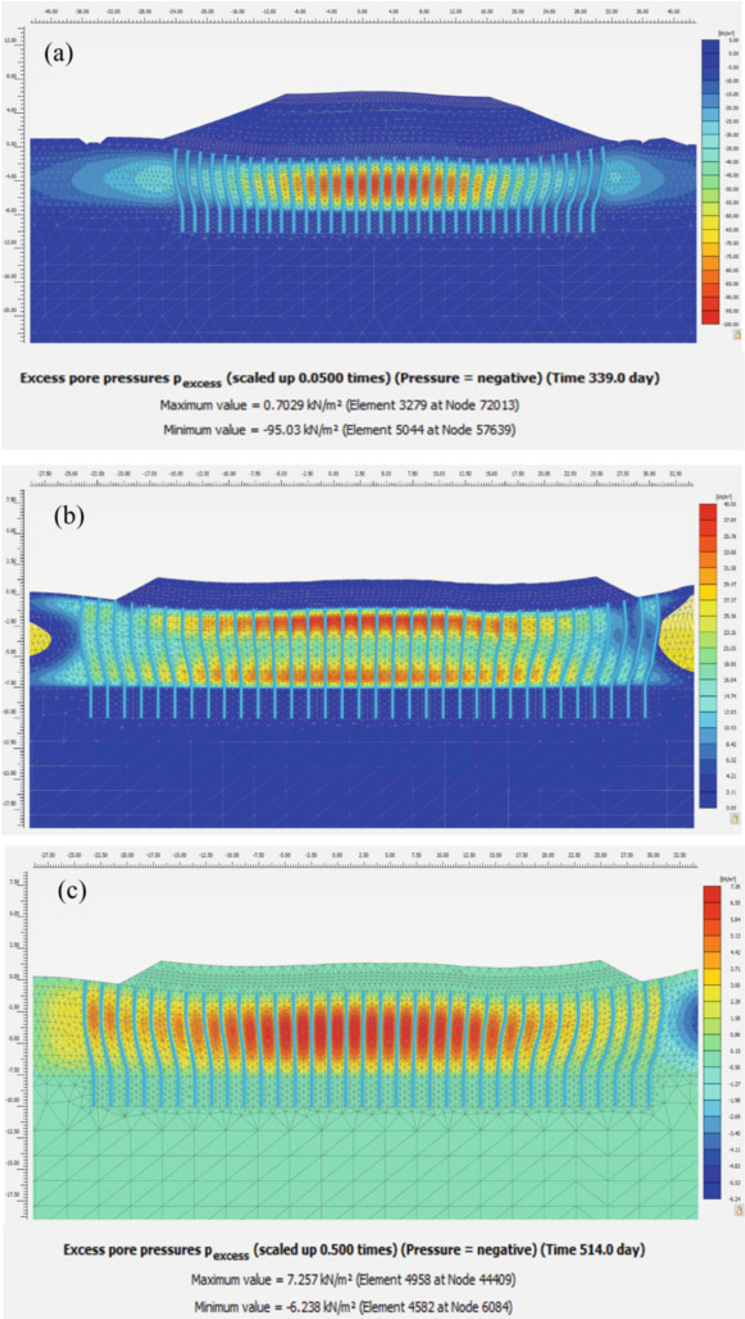

Fig. 5 Predicted excess pore water pressure **a** before surcharge removal **b** immediately after fill excavation to RL1 m **c** just before reloading **d** at the time of CPTu

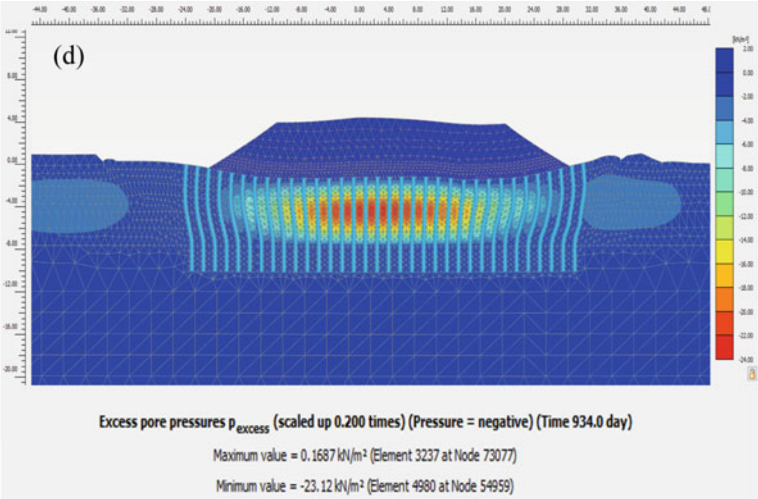

Fig. 5 (continued)

2.3 Case History 3

2.3.1 Ground Improvement Design

Case Histories 3 and 4 involve a section of a 6-km highway that crosses a floodplain along the Richmond River, where the depth of soft soil was from 2 to 27 m. Several ground improvement techniques were adopted, including preloading and surcharging, vacuum consolidation, stone columns, rigid inclusions, and dry deep soil mixing (DSM). Mainly, DSM was employed in multiple sections to reinforce the transport embankments and associated structures owing to their cost-effectiveness. Regarding sustainability, the soils at the sites were re-mixed with the binder to create a column and produced minimal spoil. Further details of the DSM treatment used for the project and its performance can be found in a number of published technical papers, e.g., [10–14], etc.

One of the diversion roads adopted DSM columns to support a box culvert for one of the diversion roads. The section was 500 m long to divert the existing traffic during the construction of the primary section for 2 years, which became a local road. The design criteria were to limit the post-construction settlement (PCS) between 50 and 100 mm in 40 years. The soil profile contained a thin desiccated clay over soft clay (3.5–7.5 m thick) where Stiff clay was below the soft (1–2 m thick). The stiff clay was underlain by residual clay located 5.6–8.9 m below the ground level. DSM ground treatment aimed to provide stiffer soil strata to curtail long-term vertical settlements and increase embankment stability. Based on the adopted design parameters (a 0.8m column diameter with a shear strength of 150 kPa where the assumed constrained modulus was 150 times the column shear strength), the spacing of individual columns was 1.4 m in a triangular pattern. Design calculations expected approximately less than 60 mm of primary consolidation settlement and 10 mm of long-term deformation during the design life.

2.3.2 QA Testing/Monitoring and Observations

Prior to construction and as part of detailed design, a limited program of laboratory trial mix and prototype field testing was carried out to enable design properties of the DSM columns and rate of consolidation of the treated ground to be determined. The design and pre-construction testing was based on SGF Report 4:95E (1997). The ratio of laboratory-mixed sample strength and the field strength of the DSM was assumed to be 2.5 based on EuroSoilStab [15] and EN 14679:2005 [16].

Further, the project specific specification required a three-stage process to demonstrate that DSM columns were successfully constructed. Firstly, laboratory mix trial testing was performed to obtain a first pass estimate of the column strength for a given cement content. Secondly, field trials were performed in each area treated using DSM prior to production commencing. After a hold point was released, production commenced and QA/QC testing was performed to demonstrate successful construction.

The following QA/QC tests were specified for assessing the quality of the DSM columns:

- Column exposures to confirm the diameter;
- Pull out resistance tests (PORT) in conjunction with unconfined compressive strength (UCS) tests to assess ultimate shear strength of columns; and
- The UCS tests were also used to assess the modulus of the DSM samples.

Comparisons between PORT, CPT, hand vane, pocket penetrometer and UU triaxial tests were also performed for calibration purposes.

Field tests were conducted at various stages of DSM column installation, initially as field trials to determine the mixing parameters required to achieve the specified strength, followed by QA/QC testing on selected production columns. Some of the test results indicated that the strength of the DSM columns could be variable with depth. In fact, some PORT and CPT results showed that the strength of certain sections of the columns could be lower than the specified shear strength of 150 kPa as shown in Fig. 6.

The exact reasons of the lower column strength than expected are not known. Various explanations were proposed such as inadequate binder dosage, high moisture content, excessive air voids in the columns, high organic content, the geotextile separator entangled with the mixing tool causing ineffective mixing, etc. It is possible that, while the above laboratory and QA/QC testing was undertaken in accordance with the project specific specification, the low strength was caused by a combination of some of the above reasons. However, the likely causes were not able to be confirmed or refuted conclusively. In hindsight, extensive laboratory and field testing could have

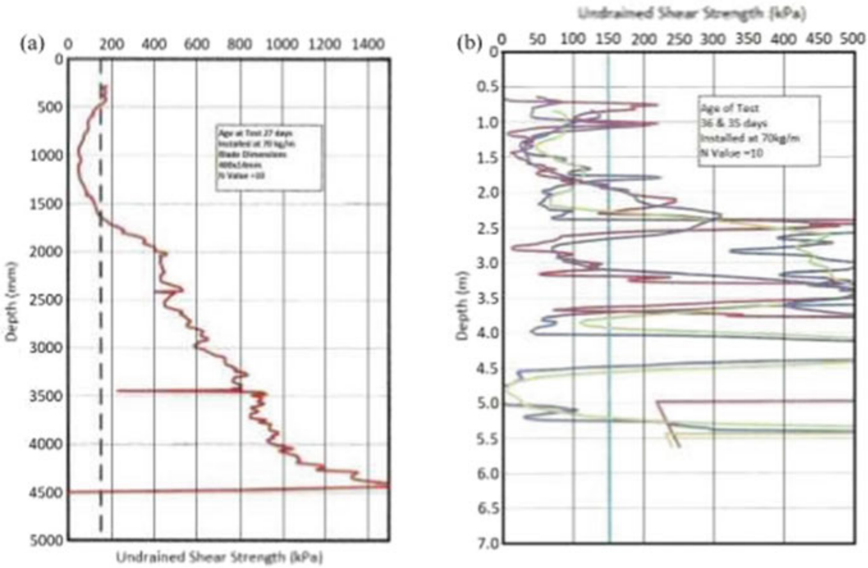

Fig. 6 QA test results showing sections of low column shear strength **a** PORT **b** CPT

been conducted to gain better understanding of the behaviour of the DSM treated ground.

Some of the top sections of the DSM columns were exposed in this area as part of the QA/QC testing. The exposure revealed that, while the remoulded soil was about 800 mm in diameter as specified, the cemented section was typically limited to the inner 600–700 mm of the columns only (Refer Fig. 7a). Furthermore, tactile assessment and shear vane testing at the top of the exposed columns showed that the shear strength of the columns was variable and generally less than the designed value of 150 kPa.

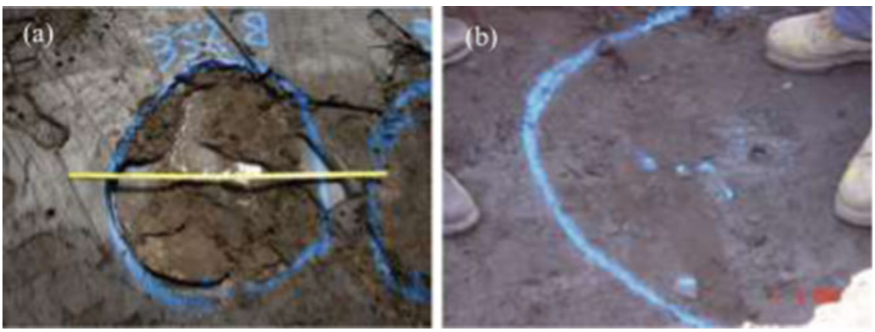

Fig. 7 **a** Undersized column **b** Water produced from central area of column

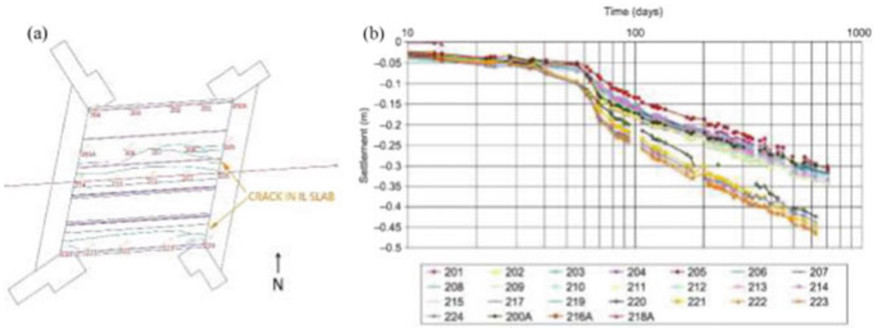

Fig. 8 **a** Location of survey points on culvert base slab **b** Box culvert settlement monitoring results

During the column excavation, a clear trend was also observed where the centre of the DSM columns near the surface was either hollow with a diameter of 40–60 mm or very soft with virtually no strength. Some of these columns produced water from the central untreated part of the column (Refer Fig. 7b).

The mixing tools were subsequently redesigned and provided with blades to cut through vegetation and avoid the risk of clogging by adjusting the blade configuration. Following adjustment of the mixing tools, other DSM treated sites did not experience the same issues.

Settlement of the box culvert was monitored during construction and fill placement using 25 survey points located on top of the culvert base slab as shown in Fig. 8a. The settlement monitoring results are shown in Fig. 8b. It can be seen that the magnitude of settlement across the base slab was not uniform, ranging from about 300 mm at the western side of the culvert to over 450 mm at the southern side after about 600 days since the construction of the culvert base slab. Differential settlement was about 0.7%, which was larger than the 0.2% expected. The culvert settlements exceeded both total and differential settlements estimated during detailed design.

In general, the lowest settlements occurred in the middle of the culvert and at the western side. The southern end of the culvert settled faster and settlements were approximately 35% higher compared to all other monitoring results. It is noted that fill material was placed firstly to the top of the culvert on the southern side (where the highest settlements were measured) followed by filling on the northern side and finally backfilling over the culvert to the design level. One effect of filling one side of the culvert prior to filling the other side was that it applied a horizontal load onto the culvert units that could generate higher stresses into the crown units, link slabs and dowel connections.

The design predicted that the slab would dish (i.e. sagging moments in both longitudinal and transverse directions). However, in the transverse direction it appeared that the slab was hogging and therefore the deflected shape was saddling (not dishing). Due to the increased differential settlement, cracking had occurred on the base slab.

2.3.3 Back-Analysis

A back-analysis of the monitoring results was conducted by the design team. It was assumed that the DSM columns had an achieved shear strength of 100 kPa uniformly distributed along the length of the column. Two net DSM column diameters, namely 600 and 700 mm, were considered.

Two-dimensional finite element consolidation analysis was undertaken using PLAXIS 2D. In summary, the smaller diameter and lower shear strength of the DSM columns than those assumed in the design had a major impact on the predicted settlements for the culvert, as the DSM treated zone was assumed to act as a homogeneous soil block with equivalent properties (i.e. shear strength and Young's modulus). Additionally, the area replacement ratio was reduced due to the smaller diameter.

Based on the results of the analysis for the smaller, lower strength DSM columns, it is likely that the as-installed columns became "overstressed", resulting in yielding of the columns. The effects of DSM column yielding had not been captured in the PLAXIS 2D analysis using an equivalent soil block for the treated DSM column areas, therefore actual settlements could have been higher than those computed in the PLAXIS 2D analysis due to column yielding.

Furthermore, the PLAXIS 2D analysis results showed that due to the difference in relative stiffness between the backfill and the culvert, the soil adjacent to the culvert side walls was arching onto the culvert structure, resulting in additional soil loads being applied at the two sidewalls (Refer Fig. 9). It can be seen from the plot of relative shear stress contours

that the arching effect resulted in loading from a soil wedge approximately 30°–45° from the vertical, extending from the base of the culvert, being applied to the two edges of the culvert. The additional loading resulted in larger settlement at the two sidewalls than the centre of the culvert, leading to differential settlement across the culvert and hogging mode of base slab deformation.

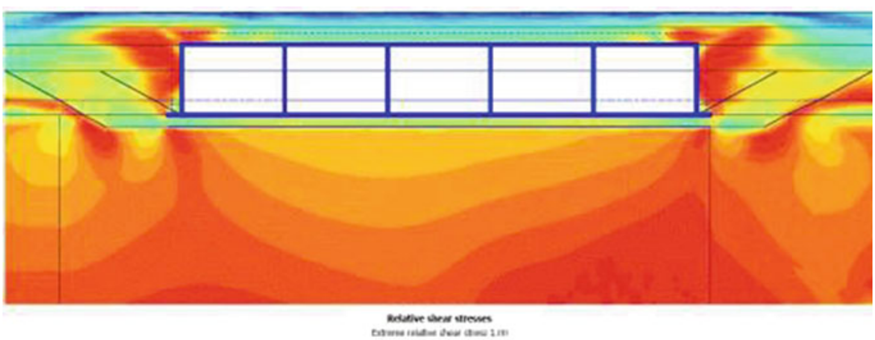

Fig. 9 Relative shear stress contours from PLAXIS modelling

2.4 Case History 4

Case History 4 is located approximately 3 km south of the box culvert discussed in Case History 3. It involves the southern approach embankment to the twin bridges across a creek identified during the early works investigation to be underlain by significant soft soils. Various soft soil treatments were performed prior to the detailed design and construction of the main highway. These earlier treatments included surcharge with PVD and vacuum consolidation performed at the southern approach embankment of the bridges. In particular, vacuum consolidation was adopted as an innovative trial to expedite embankment filling and to accelerate the consolidation of the soft soils. Some of the details and performance of the vacuum consolidation at this site have been documented previously (e.g. [17–20], etc.).

2.4.1 Ground Improvement Design

The twin bridges were underlain by Holocene alluvial deposits over Pleistocene stiff clay, residual soils and weathered argillite bedrock. The Holocene alluvial deposits comprised clays of very soft consistency near the surface, increasing in strength with depth. The approximate extent of the Holocene alluvial deposits at the abutment location was about 20–25 m thick. The thickness of the Pleistocene stiff clay increased typically from about 3 m at the southern end to about 15 m at the northern end. Residual soils comprised generally very stiff to hard clays and were limited to a maximum thickness of about 4 m. The site plan and the geotechnical section at the southern abutment of the twin bridges are shown in Fig. 10.

Vacuum consolidation (VC) was carried out at the southern abutment over a distance of 120 m. At the beginning of March 2007, the vacuum system comprising 2 m thick working platform/drainage blanket was constructed and the vacuum pump

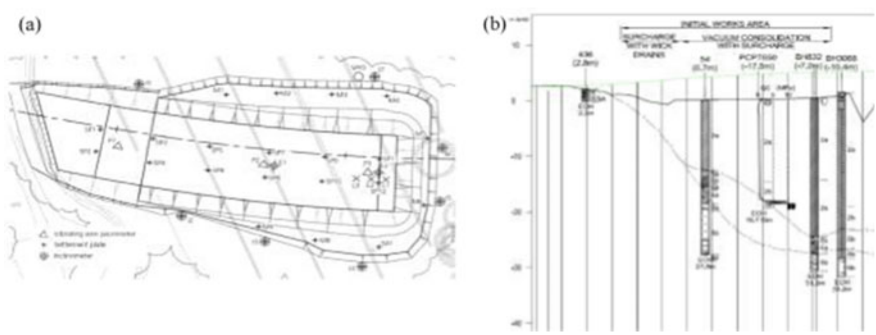

Fig. 10 Southern abutment of the twin bridges **a** Site plan **b** Geotechnical section

was switched on. Embankment filling commenced subsequently over the VC platform to a maximum fill thickness of 8.5 m. The filling was completed in mid-July 2007 and the vacuum system was switched off at the end of November 2007.

2.4.2 Initial Monitoring Results

The monitoring data taken on 18 March 2008 indicated that the VC had achieved up to about 4.7 m of settlement in one year. The fill level at that time was about 1.1 m below the design level (RL +4.9 m) at the abutment. The plots of settlement and excess pore pressure versus time are shown in Fig. 11a, b respectively.

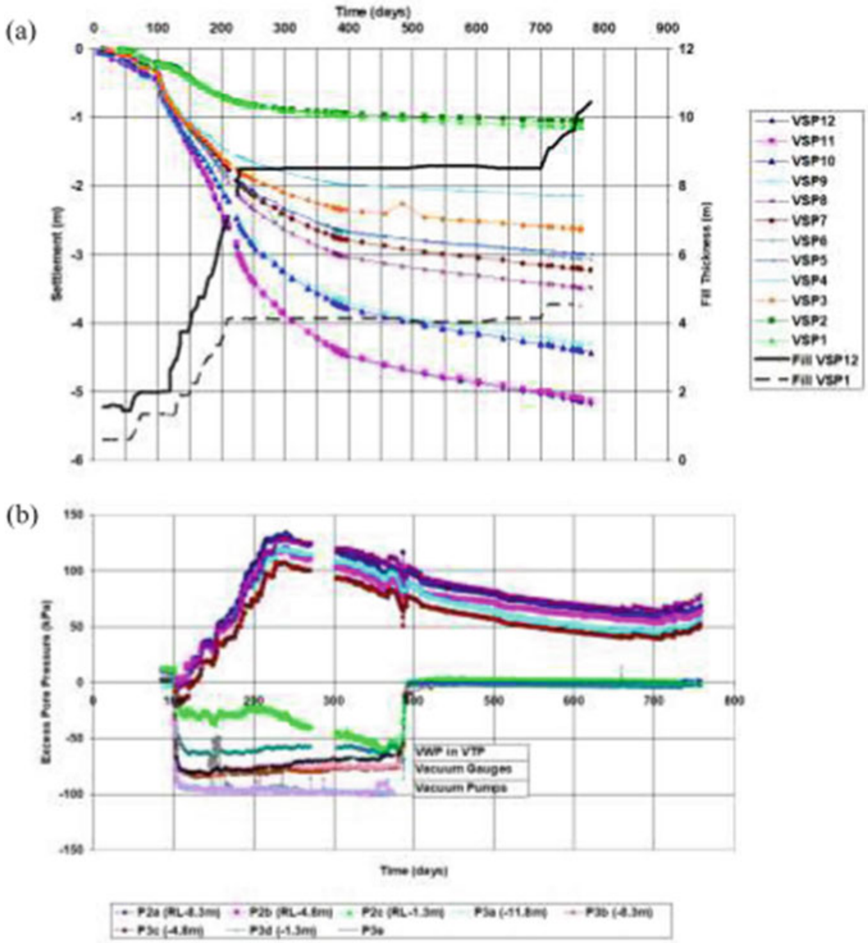

Fig. 11 Vacuum consolidation monitoring results **a** Settlement plate **b** Excess pore pressure

The monitoring data indicated that these embankments had achieved an average overall consolidation of nearly 90%. However, the design finished level was above the settled fill level at the southern abutment. Therefore, the majority of the soft soil layers were in a normally consolidated state and significant creep settlement would follow further primary consolidation. It was assessed that if no further ground treatment was carried out in the abutment areas, the post-construction settlement (PCS) could well reach 500 mm or more in 100 years. This assessed PCS and associated lateral displacement at the abutment were considered to impose unacceptable loading on the bridge abutment piles. As a result, further soft ground treatment was required.

2.4.3 Remedial Treatment

Following numerous analyses and discussions among various parties on different remedial options, a decision was made to place an additional surcharge of up to 2 m thickness for an additional 12 months waiting period prior to the piling commencing at the beginning of February 2010. This was then followed by replacing up to 3.5 m thickness of conventional earth fill with lightweight bottom ash fill. The subsequent post-construction settlement was predicted to be about 300 mm in 40 years and 375 mm in 100 years. It was considered that the lightweight fill option would likely provide better "value for money" than other treatment options for whole of life considerations. In this instance, the main design consideration was to ensure that the bridge structure (particularly the bridge abutment piles) could function satisfactory during its 100-year service life.

2.5 Case History 5

A new rail provisioning and maintenance facility to support a coal transportation network was constructed. The 3.2 km long site was located on a floodplain which was underlain by soft alluvial soils of 15–25 m depth. This posed significant geotechnical design challenges to the site development due to significant ongoing settlement potential (up to 3 m) under loading.

2.5.1 Foundation Treatment Design

The site had a number of development constraints including ecological wetlands, existing rail infrastructure, natural and man-made watercourses and creeks and underground utilities including a high-pressure gas main. A number of possible foundation treatment options were initially considered for the buildings. Based on the ground conditions at the site and anticipated relative loading of the fill and structures (including live loads), concrete injected columns (CICs) were considered the most appropriate foundation solution. This has a distinct advantage over a piled solution in terms of the structural design, as the building slabs can be designed as ground bearing rather than as suspended slabs. As a consequence, the thickness of the slab and quantity of reinforcement could be reduced. The toe depth of the CICs was also significantly less than what would be required for an appropriate pile design requiring socketing into competent bedrock.

The detailed design adopted 450 mm diameter columns from the outset of selection of the technique as this was considered to be an industry standard size relating to the type of rig used. The CICs were not designed in accordance with AS2159 as they were not considered structural elements in the manner that piles are. Nevertheless, a geotechnical strength reduction factor (ϕ_g) of 0.75 was applied to all materials.

All CICs had a centre to centre spacing of 1.6–1.8 m and a design toe depth level of 1–2 m into the underlying stiff to very stiff clay or medium dense sand. A minimum 28 compressive strength of 10 MPa was adopted for the CICs following consideration of past project experience, published case studies (e.g. [21–23], etc.) and advice from the ground improvement contractors. Based on the back-analysed data from the above-referenced case studies, a column stiffness of 10 GPa (for concrete of at least 10 MPa) was assumed. The long-term settlement was estimated as less than 50 mm thus meeting the design criteria.

2.5.2 Ground Slab Monitoring Results

During construction, based on the recommendation from the ground improvement sub-contractors, the contractors decided to use CICs with the diameter reduced to 350 mm at slightly closer centres than specified. The compressive strength of the concrete was increased from 10 MPa to typically 15–20 MPa. Following the construction of the ground slab and track installation, the level on the top of the slab was taken as the baseline reading for settlement monitoring. Six months after the commencement of monitoring, settlements of 40–54 mm had been recorded along the centreline of the new building with 49–54 mm at the two ends and 40 mm in the middle. A typical settlement versus time plot is shown in Fig. 12.

In addition, the recorded settlement across of the ground slab was relatively uniform instead of the typical "dished" shape settlement profile generally expected from a typical uniformly loaded slab on ground. This recorded settlement shape across the slab could possibly imply that the ground slab was relatively rigid compared to the underlying CICs and soft soils.

2.5.3 Back-Analysis and Predicted Future Settlement

The contractor's geotechnical advisors undertook the back-analysis of the settlement monitoring data and the prediction of future performance. Based on this work, it was predicted that the post-construction settlement could exceed the specified settlement design criteria for the project. Whilst the predicted exceedance was purportedly relatively minor (up to 12 mm over the 50 mm limit), the designers considered that

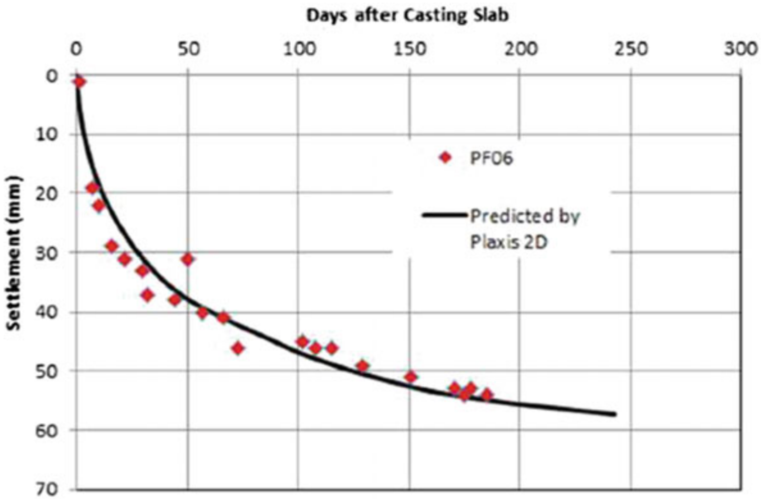

Fig. 12 Typical settlement of the ground slab with time

the resulting structural performance of the ground slab and the building could be worse than what was allowed for in the design since the settlement measured by the contractor was taken from a survey of the track installation rather than from the ground slab construction, i.e. the settlement occurred between slab construction and track installation has not been accounted for. Further, it is possible that the predicted post-construction settlement could be optimistic because of the issues discussed below:

- The stiffness, i.e. elastic modulus, and configuration of the CICs were selected in the detailed design to meet the settlement criteria. However, the back-analysed elastic modulus of the CICs nominated by the contractor's geotechnical advisors was about 5% of the design value, i.e. back-analysed elastic modulus of 0.56 GPa against 10 GPa assumed in detailed design. This back-analysed modulus value was not consistent with other published case studies with similar column systems (e.g. [22, 24]). If the back-analysed elastic modulus was correct, it implied that the CICs had not met the design intent and were significantly softer than those assumed in the design. The geotechnical advisors assumed in their back-analysis that the CICs were elastic and had infinite strength. However, elastic modulus of concrete is directly related to the concrete strength. For such low back-analysed stiffness value, it is expected the concrete strength was similarly affected. It is likely that under this scenario, the CICs could deteriorate and fail with time leading to significantly greater ground settlement than that predicted by the geotechnical advisors.
- The geotechnical advisors also reported that the strength parameters of the various soil units have been back-analysed to reduce by approximately 60% due to remoulding of the soil during CIC installation. The remoulding process would also

make the soil more compressible than when it was in its natural state. While the geotechnical advisors predicted the post-construction settlement to be up to 62 mm only based on the original soft soil parameters, the remoulding potentially could lead to greater long term creep settlement and hence greater post-construction settlement of the building.

Apart from the possibility of extremely low column stiffness and low soil strength, one other possible explanation was that the CICs were defective and were not providing the stiffness that the design assumed and relied upon to limit settlement.

Alternatively, it is possible that the Young's Modulus of the CICs was in fact higher than the 0.56 GPa back-analysed, but the soft soil in-between the CICs had been remoulded to lower stiffness and strength during installation. In that case, the predicted future settlement could be greater than the currently predicted values when some of the building loads were supported by the remoulded soft soil. However, without further testing of the CICs and the surrounding soft soil, it is difficult to confirm the actual reasons for the excessive settlement measured during construction.

3 Lessons Learnt and Conclusions

The main objective of this paper is to demonstrate that post-construction settlements can exceed the design criteria regardless of the ground improvement system used. The case histories illustrate that there are advantages and disadvantages with every ground improvement system. No system should be discounted for future consideration simply because of one example with undesirable performance. It is therefore not the intention of the author to give examples of underperformance of every ground improvement system.

In many of the case histories discussed above, the exact causes of the excessive settlement following ground improvement cannot be readily identifiable or are at least not conclusive. While it is desirable to understand the main causation factors such that any potential mistakes do not repeat in future projects, sometimes it is not possible to be definitive.

The delivery of a successful ground improvement project is therefore dependent more upon the design and the implementation rather than the adopted ground improvement system itself. Based on the experience of the author working on various underperformed ground improvement projects, it is proposed that the designers and constructors should pay attention to a number of salient factors as follows:

- Observational approach based on detailed monitoring should form an integral part of the ground improvement design. The design does not end at the issuing of drawings for construction. The designers and constructors should allow for some flexibilities in the design and construction programme to make adjustments in the ground improvement design in response to the monitored actual ground behaviour.

- Review of the monitoring data should not stop at comparing the monitoring results with design predictions. There can be many reasons why the measured results are similar or even better than prediction during construction. However, acceptable monitoring results during construction do not necessarily guarantee that post-construction settlement will not exceed design criteria. It is important that back-analysis is carried out during construction to back-figure the design parameters before appropriate future prediction can be made.
- In this regard, the back-analysis should utilise all available monitoring data rather than simply focussing on ground surface settlement alone. Ideally, the back-analysed model should be able to “fit” all monitoring data, such as ground surface settlement, pore water pressure response, substrata compression, etc. Monitoring data different to expectation should not be discounted without understanding the reasons for the unusual behaviour.
- The back-figured design parameters and properties should fall within admissible ranges. Engineering judgement should be applied in the selection of the parameters and properties rather than blindly performing a “curve fitting” exercise.
- Adequate field trials and laboratory testing should be conducted to understand the behaviour of the ground and the adopted ground improvement techniques. Further, appropriate QA/QC testing should be undertaken during construction to ensure that the quality of the adopted ground improvement systems complies with the specified criteria and is consistent with design assumptions. Depending on the ground improvement system adopted, a number of typical technical specifications and standards (e.g. [15, 16, 25–28], etc.) provide advice and recommendations on QA/QC testing. Another source of QA/QC testing recommendations for various ground improvement systems can be found at the GeotechTools website [29]. For public infrastructure projects, the relevant authority specifications (e.g. TfNSW QA specifications R223 [30] and R225 [31], etc.) should be used.
- The ground improvement design should made cognisant of the construction activities and sequencing. Such construction activities should be captured in the ground improvement design where appropriate.
- In the selection of the ground improvement system, the designers should understand the theoretical background and limitations of the ground improvement system adopted. While most of the available ground improvement systems can treat soft soil effectively, different systems are more appropriate for different ground conditions and/or project requirements.

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A Smart Geotechnical and Geological Approach for Future Building and Transport Infrastructure Projects

David J. Och

Abstract There is a rapid and unprecedented scale of infrastructure planning and development across the Sydney region. A SMART approach that captures historical ground investigation and regional geological data is required to support early transport planning by Government. This will allow the refinement of geological and geotechnical knowledge gaps that will be augmented with additional investigation once these corridors are further assessed as the design develops. To allow a SMART approach in infrastructure planning and development, Government Departments and potentially the private sector could integrate their internal geological and geotechnical data as part of a centralised state-wide data collection centre. This will require Government to legislate a registry system for factual geotechnical data for all Departments and Authorities. Consideration would also need to be given to how to release this information from the private sector many of whom would claim this was their intellectual property despite typically being derived (and paid services for) from Government projects. Consideration should be given to a two-stage process so as not to derail the implementation due to potential delays with the private sector:

(i) Combine and integrate geological and geotechnical data from historical Government projects including those delivered under corporatised government entities.
(ii) Integration of factual data obtained from the private sector.

Any data compiled under both (i) and (ii) will need to be relied upon without any impact or recourse to the originators. This has been key to the success of similar data sharing mechanisms in the United Kingdom (British Geological Survey) and the Netherlands (Dutch Geological Survey). A way of making this work successfully in New South Wales, following successful international models such the UK and Netherlands, is to have government allow contracts or documentation to have

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historic data relied upon. The State will achieve better value for money by way of having significantly more geological and geotechnical data as part of Environmental Impacts Statements to inform approvals and stakeholders as well as for its Request for Proposals (RFP). In all cases with more reliable information a better outcome will be achieved by way of increased certainty and avoiding approval delays, possible injunctions, as well as more informed Request for Tenders (RFTs).

Keywords Geotechnical · Database · Boreholes · Circular economy · Sustainability · Data · Infrastructure · Open-source · Standards

1 Introduction

Across the Greater Sydney Region in the next several decades there will be a rapid and unprecedented growth in our population. This will see a greater requirement for future planning and development of key infrastructure to allow our populations to grow from the suburbs to the regions.

The geological setting of the Sydney Basin is well understood and documented, but the geotechnical conditions are the key to progressing project designs. However, much of the historic associated project data collected during the development of existing infrastructure projects are compiled as tabulated data or plans in PDF form. In recent major NSW infrastructure projects, there is a requirement that geotechnical factual datasets are provided in a digital format (i.e. *.ags or *.csv) [1]. As NSW has no central geotechnical database/repository (with exception for Public Works—DIGS and MinView) that captures new and historical (factual and interpreted) geophysical, geological, and geotechnical investigations data by the public sector in NSW, these key data are mainly found to reside in consultancy databases [2, 3]. Therefore, during early planning, and scoping phases much of the data is sourced from in-house databases as commercial sensitivity restricts stakeholder engagement along proposed corridors. Government DA online databases provide a valuable source of information (structural, geological and geotechnical) allowing the planners and designers to submit a request for information (RFI) that the client (government agency) submits to the local councils or other government agencies once dialogue has been established for any data that can assist the project desktop study. Much of the data is limited to borehole logs without any or minimal corresponding geotechnical data to assist design. Therefore, in many cases the lack of clarity of data captured during these desktop studies can only confirm geological rather than geotechnical conditions requiring the client to initiate a new ground investigation. The data captured in early studies can be subject to loss or are poorly archived due to a change or merger of government agencies, therefore, if the next iteration or another project (different government agency) is being developed data will be the key issue again. Hence a circle of discovery is embarked on again!

A SMART approach that captures historical ground investigation and regional geological data should be mandated that follows the successful implementation of data capture by many international Geological Surveys and will be presented below. This will allow the refinement of geological and geotechnical knowledge gaps that will be augmented with additional investigation once favourable corridors are defined. These large infrastructure projects have many corridor options under consideration, and the reliance of historic data assists in these early design phases. Once preferred options are defined the geological and geotechnical gaps observed can be filled by an early phase of ground investigations to achieve a reliance to allow progression to the next phase. Therefore, ground investigations should be planned to fill in key gaps of knowledge where corridors focus in on an area (e.g. harbour crossings) being mindful of protecting key locations (i.e. stations etc.) from the ever-watching stakeholders with a vested interest in these future developments. As the progression of these designs into tender phases is very rapid, many locations (i.e. stations and shafts) in key residential, commercial and industrial areas, are commonly not under ownership of the transport planners (i.e. Government) and therefore, access to these locations to carry out further geotechnical investigations can only be carried out during tender phase or even after contract award.

2 Approach

A refined geotechnical model based on broad data sources is key to developing a successful infrastructure project. Thorin et al. [1] note the importance of The Australian Standard on Geotechnical Investigations (AS1726-2017) requiring "literature review" known as a desktop study, a key function in assessing gaps in historic data. Therefore, a methodology of assessment—A SMART approach (after [4]) is key to developing robust ground models to assist through the many phases of design. To achieve a successful model many sources of data are required, but in the time frame of these developing projects a single source database is needed to allow efficient design objectives. The following objectives are defined highlighting key challenges that the clients, designers and contractors need in delivery of these major projects:

- **Specific**—systematically targeting relevant construction data, spatial, environmental, geological, geophysical and geotechnical data [1] to assist in the scoping of the project corridors being evaluated (i.e. projects corridors above and below the ground, brownfield or greenfield).
- **Measurable**—the early data collected needs to provide attributes that can allow both qualitative and quantitative assessment during the early phase of design to allow characterisation of material relevant to the project (i.e. excavatability, discontinuities, foundations).
- **Attainable**—has the characterisation to provide geotechnical data been very limited in certain areas of the project corridor, if not, a 1st phase of investigations needs to be carried out at locations defined as gaps in knowledge. The investigation will be governed by stakeholder sensitivities for possible station, shaft, bridge or dive locations on these infrastructure projects. Further iterative

investigation (2nd and 3rd) phases will be required as the project is better defined, and locations become available to fill in the geotechnical gaps identified during design that can use the power of cloud-based servers to maintain the integrity over time.

- **Relevant**—in many cases, due to land acquisition issues, the realistic collection of new data from investigation will not be available so the specific parts historic data will need to be relied on more often, therefore, there will be an expectation of data reliance that the data processed will be relevant and support the future direction of the design. Also, current investigations need to be planned so data collected is relevant to the key drivers of the project (i.e. hydro, mixed face conditions, abrasivity, strength, contamination etc.).
- **Time-dependent**—these major infrastructure projects are time-dependent from the initial scoping to Request for Proposals (RFP) then Request for Tenders (RFT) which occurs over relative short times (2–3 years). Therefore, realistic investigations need to be planned post desktop study, once gaps in historic data are defined relative to defined corridors which can take up to 9 months from completion of desktop study to award and commencement of geophysical, geotechnical, contamination investigations. As the design is progressed from Definition to Reference the alignment should be well established with the 2nd phase of investigation nearing completion allowing for assessment of data during RFP and RFT phases. A 3rd phase of investigation should have commenced once property acquisitions have been completed. Tenderers should have also provided feedback during the RFT on the earlier investigations (a list of preferred boreholes) that can allow the tender to be priced with risk minimization.

To allow a SMART approach in infrastructure planning and development, Government Departments and potentially the private sector could integrate key attributes of their internal geological and geotechnical data as part of a centralised state-wide spatial data collection centre. Centralising information and leveraging the disparate datasets residing in various pockets across the industry will result in multiple efficiencies and substantial cost savings to both the NSW Government and professional services companies alike.

3 Discussion

Many historic geotechnical datasets from large infrastructure projects are stored by the many government departments in hard-file libraries or digital-files are archived away in storage, with only a few of these departments now preserving the hard-factual data in spatial retrieval systems (GIS database systems). Recently the department of Public Works released all their hard-copy reports to the Geological Survey of New South Wales which was incorporated as a layer (PDF) in the MinView online portal. At present there are several NSW publicly available spatial geo-enviro-hydro databases:

1. DIGS—serves as a repository for geological information such as geological reports and maps, mining exploration reports, and papers, with only a few historic geotechnical reports [5]. https://digsopen.minerals.nsw.gov.au/digsopen/
2. MinView—MinView is a web mapping application that provides free access to view, search and download a comprehensive range of geoscientific data for NSW. https://minview.geoscience.nsw.gov.au/#/?lon=148.5&lat=-32.50000&z=7&l=
3. WaterNSW—provides an online portal where access permitted to all WaterNSW's hydrological and hydrogeological data. https://realtimedata.waternsw.com.au/
4. NSWepa—NSW Environmental Protection Authority provides an online Environmental Science Portal. https://www.epa.nsw.gov.au/
5. GADDS—a Geophysical Archive Data Delivery System that provides magnetic, radiometric, gravity and digital elevation data from Australian National, State and Territory Government geophysical data archives https://portal.ga.gov.au/persona/gadds
6. SEED—Sharing and Enabling Environmental Data portal—has been developed with and for the community of NSW, as a central place for everyone to find data about the NSW environment. https://live.seed.nsw.gov.au

As a community the geotechnical profession within government agencies has yet to fully embrace the digital opportunities offered by recent advances in GIS technology incorporating standardised open digital data transfer formats such as AGS [1]. For a centralised geological and geotechnical database to be established with success there will need to be a broad baseline standard that allows the capture of SI data in a format as specified in project services briefs and general specifications (i.e. Sydney Metro, General Specifications—Geotechnical Investigations, SM-20-00,130,154, v.1.0, 26 November, 2020) following specified standards (i.e. AS1726-2017).

To achieve these objectives, the establishment of a standardised services brief for all site investigations is suggested to provide a framework to the reform of the process and allow data collection into a cloud-based database where immediate access can be provided to assist local or regional projects or assessments (i.e. British Geological Survey etc.). Centralising information and leveraging the disparate datasets residing in various pockets across the industry will result in multiple efficiencies and substantial cost savings to both the NSW Government and professional services companies alike. With the implementation of the NSW Spatial Collaboration Portal and the NSW Spatial Digital Twin these datasets collected and processed during projects can be added as layers into these portals. The factual data along with other online portals noted above could form part of the NSW Spatial Collaboration Portal with interpreted models (i.e. 3D geological-geotechnical models) can form part of the subsurface NSW Digital Twin in association with structure constructed.

This will require Government to legislate a registry system for factual geotechnical data for all Departments and Authorities. Consideration would also need to be given how to release this information from the private sector many of whom would claim this was their intellectual property despite typically being derived (and paid services for) from Government projects. A way of making this work successfully in New South Wales, following successful international models such the UK and Netherlands, is to have government allow contracts or documentation to have historic data relied upon without any impact or recourse to the originators.

3.1 Case Study 1—British Geological Survey

This envisaged geotechnical database could be modelled on the British Geological Survey (BGS)—National Geotechnical Properties Database (NGPD) deposited 'open' data (Data Deposit Portal) from commercial site investigations carried out for civil engineering along major UK infrastructure developments. The earlier database was found to be inefficient to use and manage and there was a decision to create and populate a relational database that comprises 54 data tables and 33 dictionary tables. The tables are defined by key attributes that describe project, national Grid coordinates, boreholes details, lithological description, and in situ test data; sample data; and a range of laboratory index, mechanical properties and chemical test data on soils, rocks and water [6]. The collected geotechnical records supplied to BGS by clients, consultants and contractors with a preference in AGS digital data over paper records due to the quicker intake of data rather than manual input. Geophysical surveys undertaken in association with intrusive geotechnical engineering projects also form part of deposited data source in the NGPD.

The current NGPD (December 2019) contained many thousand site investigation reports linked to associated holes. The in-situ field results linked to the investigation holes total over 3 million records including sampling with over 5 million associated laboratory tests. These datasets form the basis for the geotechnical attribution of 2D and 3D digital geological models and integrated into the BGS Groundhog Desktop Geological Software that is available as a Community version or Professional edition allowing users to interpret site data and develop conceptual 3D digital geological models (https://www.bgs.ac.uk/technologies/software/groundhog/). The great volume of laboratory data forms part of themed datasets which includes engineering geology, geophysics, hazards, hydrogeological etc. The dataset for engineering geology is further titled into aspects of civil engineering (i.e. strength, excavatability, bulking volume etc.). The data collected can allow another dimension of parameterisation, characterisation and assets housed within the subsurface such as Project Iceberg (Digital Twin), which was developed by the BGS, Ordnance Survey and Future Cities, will de-risk future investment through better knowledge of the subsurface.

The process of depositing data into the NGPD is a voluntary process that must comply with freedom of information legislation and in recent time the BGS has started to release confidential data that has been in the repository for more than 4 years, but can remain confidential under valid justification from the depositor, or with written permission this data can be requested to be released. The data from the NGPD is supplied 'as is' or for 'information only' and under the terms which material is deposited as open data at the BGS will exempt liability against the original donor/depositor where BGS releases the information.

3.2 Case Study 2—Geological Survey of the Netherlands

The Netherlands' Ministry of the Interior and Kingdom Relations introduced into legislation (The BRO Act), on the 1st January 2018, a key registry of the Subsurface that has formed a central database of public data across the Netherlands [7, 8]. This imposes a statutory obligation upon all municipalities, provinces, water boards and government agencies to submit standardised digital data of subsurface data that have been acquired with public funds. The law also applies to three subsurface models produced by TNO, Geological Survey of the Netherlands (GDN). This was possible as the GDN could allow their existing National Data Repository to be upgraded with the Ministry of the Interior and Kingdom Relations allowing for additional regulations that specify the data standard data capture. The database is progressing through a first phase and will be expanded over time to include multiple subsurface domains classified based on data types and made public accessible (i.e. DINO*loket* https://www.dinoloket.nl/) as part of the Ministry's open data policy, progressively moving towards the Key Register of the Subsurface (BRO).

DINO*loket* is a publicly open portal that allows access to a databank of geo data on the shallow and deep geology from century-old drilling data to recent geophysical research within the Netherlands with many thousands registered users [9]. The databank comprises borehole data, groundwater data, cone penetration test data, vertical electrical soundings, the results of geological, chemical and mechanical sample analyses, borehole logs, and seismic data [9].

Ministry of the Interior and Kingdom Relations of the Netherlands is combining this dataset into a National Framework of Key Registers (E-Government Generic Digital Infrastructure (GDI)) that includes 11 National Key Registers, 6 Administrative Key Registers and 6 Geospatial Key Registers—surface (5 dataset) and subsurface (BRO). The present scope of BRO includes data obtained from civil engineering, agriculture, water management and natural resources (BRO I) that may be extended to include soil contamination and archaeology (BRO II). The future step for BRO will be the integration of subsurface data and other geospatial key registers into a 3D virtual living environment (Digital Twin) to help accelerate developments by ensuring transparent and understandable data for engineers, geologists, planners and the general community. If any subsurface data provided is found to be inaccurate there is a requirement that this be reported to the minister stating a reason. The Minister will consider if the source data owner needs to further investigate any issues with the data.

4 Conclusions

To allow a SMART approach in infrastructure planning and development, Government Departments and potentially the private sector could integrate key attributes of their internal geological and geotechnical data as part of a centralised state-wide data collection centre. This will require Government to legislate a registry system for factual geotechnical data from all Departments and Authorities. Consideration would also need to be given as to how to release this information from the private sector, many of whom would claim this was their intellectual property despite typically being derived (and paid services for) from Government projects. Data provided as part of DA submissions (following key standards i.e. as per data deposited into the BGS and TNO-GDN registries) to local government should be captured in NSW government databases such as NSW Spatial Collaboration Portal. Reports along with geotechnical factual data and interpretations (i.e. parameters, characterisations and subsurface modelling) from major infrastructure projects should be captured in the NSW Spatial Digital Twin.

Consideration should be given to a two-stage process so as not to derail the implementation due to potential delays with the private sector:

i. Combine and integrate geological and geotechnical data from historical Government projects including those delivered under corporatized government entities.
ii. Integration of factual data obtained from the private sector.

Any data complied under both (i) and (ii) will need to be relied upon without any impact or recourse to the originators. This has been key to the success of similar data sharing mechanisms in the United Kingdom (BGS—British Geological Survey) and the Netherlands (TNO—Geological Survey of the Netherlands).

A way of making this work successfully in New South Wales, following successful international models such in the UK and Netherlands, is to have government allow contracts or documentation to have historic geotechnical factual and interpretation data relied upon (i.e. EP24-2006—A Code of Practice for Risk Management of Tunnel Works). The State will achieve better value for money by way of having significantly more geological and geotechnical data captured and processed during earlier design phases as part of Environmental Impacts Statements to inform approvals and stakeholders as well as for its Request for Proposals (RFP). In all cases with more reliable information a better outcome will be achieved by way of increased certainty and avoiding approval delays, possible injunctions, as well as more informed Request for Tenders (RFTs).

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Deep Wet Soil Mixing Columns Ground Treatment Technique-Lesson Learnt from Project

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Abstract Deep Wet Soil Mixing (WSM) ground treatment columns have been designed and constructed first time on TfNSW Lisarow to Ourimbah Stage 3B project to meet performance requirements for embankment and retaining wall foundation. However, during construction, the project faces a number of challenges such as reuse of several thousand cubic meter of soil–cement mixed waste material produced from WSM construction, various quality control testing issues and construction difficulties arising from interlayered stiff/dense soil. Hence, the project team developed extensive field trials and innovative solutions to mitigate all these challenges. Instrumentation monitoring results obtained from settlement plate, inclinometer, piezometer, survey plugs and wall tags suggested that primary consolidation settlement of the WSM treated ground is virtually completed within one month after the construction. Field monitoring results also reveal that settlements and lateral movements of the bridge approach embankments and hybrid retaining wall (Reinforced Soil Wall combined with L-shaped wall) foundations are much less than the predicted design values. In addition, the trend of the 12 months monitoring results provides confidence on the long-term performance of these structures, which is expected be smaller than the predicted values. Furthermore, it is revealed that about 5000 m^3 of soil–cement mixed waste material/spoil have been recycled, and used successfully as fill materials for various civil engineering applications at the project site. Following the several field testing regime, it is concluded that only Unconfined Compressive Strength (UCS) testing from the cored samples is the most reliable quality control measure for high strength WSM materials. In addition, strength and deformation parameters of the field core samples are proven to be increased with the increase of curing time.

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Hence, it is recommended to include curing effects on the design of deep soil mixed columns ground improvement technique for future projects.

Keywords Deep wet soil mixing · Settlements · Lateral movements · Unconfined compressive strength · Foundations

1 Introduction

Transport for NSW (TfNSW) is progressively upgrading the existing Pacific Highway to a four-lane road within the Central Coast region of NSW, Australia. The proposed upgrade called "Lisarow to Ourimbah Stage 3B" requires construction of 1.6 km of road widening including bridge approach embankments and transition zones, which are supported by a combination of reinforced soil wall and "L" shaped reinforced concrete wall (maximum 8.5 m high). These structures are required to be constructed on over up to about 20 m thick soft to firm clays with interlayered loose to medium dense clayey sand and stiff clay. In addition to this complex geology, the project faces several challenges such as meeting stringent post construction settlement criteria for the pavements and retaining walls, and construction within a constrained site including environmentally sensitive wetlands which limit the extent of the widening footprint. As such, after assessing wide range of ground improvement techniques (e.g. surcharging with or without vertical drains, deep wet soil mixing columns, deep dry soil mixing columns, concrete injected columns and stone columns), Deep Wet Soil Mixing (WSM) columns ground treatment technique was adopted for the first time on TfNSW project to meet project requirements. WSM column diameters of 1.2 m with a length up to 21 m, and square pattern spacing from 2.2 to 3.7 m was constructed. However, during construction, the project faces a number of challenges such as reuse of several thousand cubic meter of soil–cement mixed waste material produced from WSM construction and various quality control testing issues. The quality control testing issues also imposed uncertainties on the performance of the approach embankments and retaining walls. Thus, the project team developed extensive field trials and innovative solutions to mitigate all these challenges.

This paper presents performance of bridge approach embankment and retaining walls (combined RSW and L-shaped) from several instrumentation monitoring results (e.g. settlement, lateral movement, pore pressure) obtained from WSM treated ground, reuses of soil–cement waste material in different civil construction activities and quality control issues of WSM columns. Results from the quality control assessment such as strength and deformation characteristics are discussed in-line with instrumentation monitoring results, and long-term performance of the structures are predicted. Furthermore, following the successful construction of this project, several challenges and lesson learnt issues are discussed.

2 Site Description and Geology

The new construction approaching the Main Northern Railway Bridge consists of embankment widening where the exiting road rises from a relatively flat alluvial plain with the approach embankment rising up to about 11 m near the southern abutment of the existing bridge over a length of approximately 220 m (CH 7740 to CH 7960). The road widening commences as an integral L-shaped reinforced concrete (RC) wall for embankment heights up to approximately 2.5 m transitioning to a combined Reinforced Soil Wall (RSW) overlain by an L-shaped RC wall up to a maximum of 8.5 m at the bridge approaches.

Ground conditions south of the Main Northern Railway are characterised by vertically and laterally variable thick alluvial soils overlying rock at variable depths of up to 20–25 m. Fill generally overlies very soft to firm sandy clay/silty clay, interlayered with loose to medium dense clayey sand and stiff to very stiff sandy clay layers up to a depth of about 20 m. The low to high plasticity organic alluvium has a moisture content range of 10–65%. These clays have low strength, medium to high compressibility and low permeability. The coefficient of horizontal consolidation of these clays varies from 5 to 20 m^2/yr. Typical undrained shear strength and consolidation ratio of the soil profile at the bridge approach embankment with retaining wall is shown in Fig. 1.

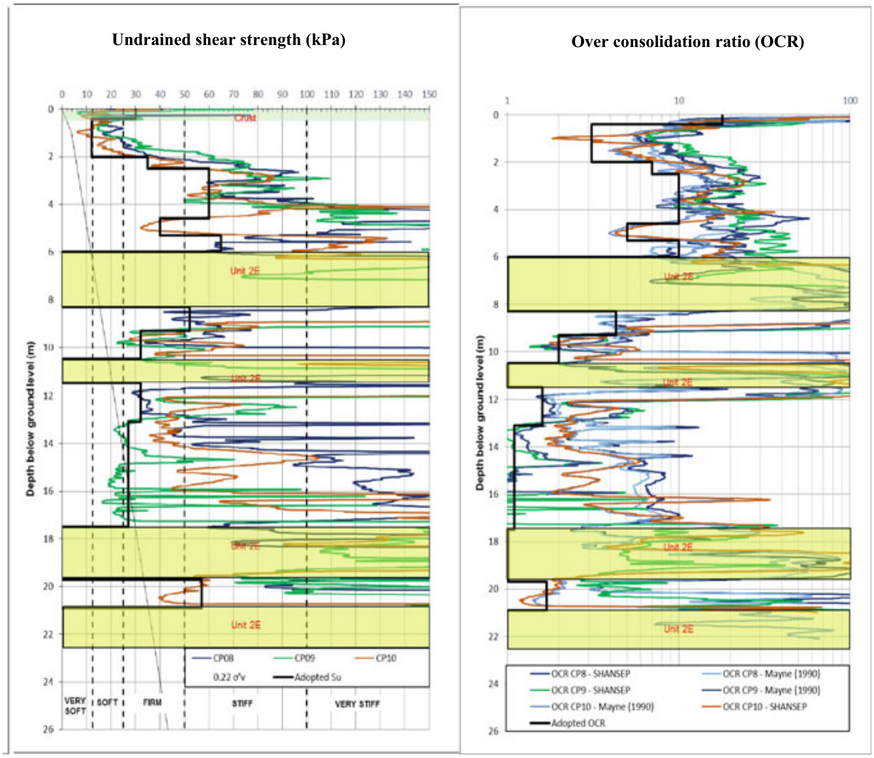
3 Project Design Criteria

Design requirements for the bridge approach embankments and associated retaining structures were as follows:

(a) Post construction settlement (PCS) limit of 50 mm in 40 years within 20 m of bridge abutment and 200 mm elsewhere for flexible pavements;
(b) Maximum retaining wall foundation settlement is limited to 100 mm in 100 years and
(c) Differential settlement is limited to 0.5% change of grade in any direction for pavement and retaining wall foundation.

4 Ground Treatment Design

The ground treatment design considered series of options such as Surcharge with/without prefabricated vertical drains (PVD), Dry Soil Mixing (DSM), Wet Soil Mixing (WSM), Concrete Injected Columns (CIC) and Driven Precast Concrete Columns (PCC). Following the review of site geology, costing, construction time, various uncertainties and environmental constraints; WSM was preferred for the detail design and construction. Geotechnical design considerations included: WSM



Note: Soil unit 2E= Medium dense to dense clayey sand/silty sand

Fig. 1 Typical undrained shear strength and over consolidation ratio of the soil profile at the bridge approach embankment with retaining wall

column design for settlement and stability control, the impact of the widened embankment on the existing embankment, lateral sliding, foundation extrusion and construction staging. WSM design was carried out following the Swedish Geotechnical Society [14] method and previous experience [6, 9–11]. Accordingly, WSM column shear strengths of 400 and 300 kPa for the embankment crest/retaining wall foundation and embankment batter was designed. The corresponding columns stiffness's were 200 and 150 MPa respectively. A detailed WSM design methodology of this project is described in elsewhere [7].

Following the various design iterations, WSM column diameters of 1.2 m and square spacing between 2.2 and 3.7 m for the embankment crest (Zone 1), and 3 m panel spacing for batter (Zone 2) were constructed. The corresponding area replacement ratios were between 0.23 and 0.08 (Zone 1), and 0.38 (Zone 2), respectively. The columns at the embankment batter were in a panel configuration with 100 mm overlap. The columns were constructed to a maximum length of 21 m (including 1 m into stiff to dense material). For lateral sliding stability and basal extrusion control, high strength tensile geotextile has been designed within a basal reinforced gravel

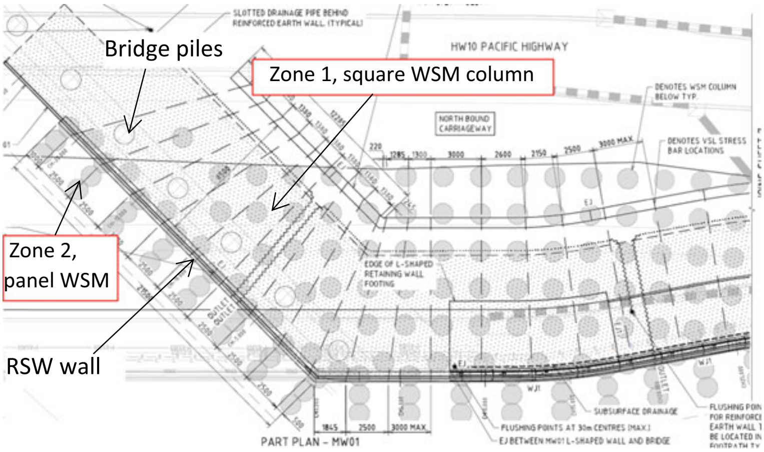


Fig. 2 Schematic as built WSM arrangement for bridge approach embankment and retaining wall foundations

mattress in accordance with BS8006—1995. A schematic as built WSM arrangement for bridge approach embankment and retaining wall foundations is shown in Fig. 2.

5 Instrumentation and Monitoring Results

Series of instruments such as settlement plates, inclinometers, vibration wire piezometers, survey plugs and wall tags were installed along the WSM columns improved embankment and retaining wall to monitor their performance during and after the construction. Figure 3a, c show instrumentation monitoring plan and elevation for embankment and retaining wall.

5.1 Settlement Results

Settlement of the bridge approach embankment using settlement plate BSP023 is measured, and the results are shown in Fig. 4. As can be seen, maximum 10 mm settlement is recorded with a fill thickness of 7 m. This primary consolidation settlement value is substantially lower than the predicted ones (predicted settlement, 67 mm). Furthermore, survey plugs attached on the anchor beam of the RSW wall is monitored, and the results are shown in Fig. 5. The results show that in one year, the wall moves between 1 and 4 mm only.

In order to understand the predicted behaviour with the measured responses, the predicted settlement of the embankment and combined RSW wall using Plaxis 2D

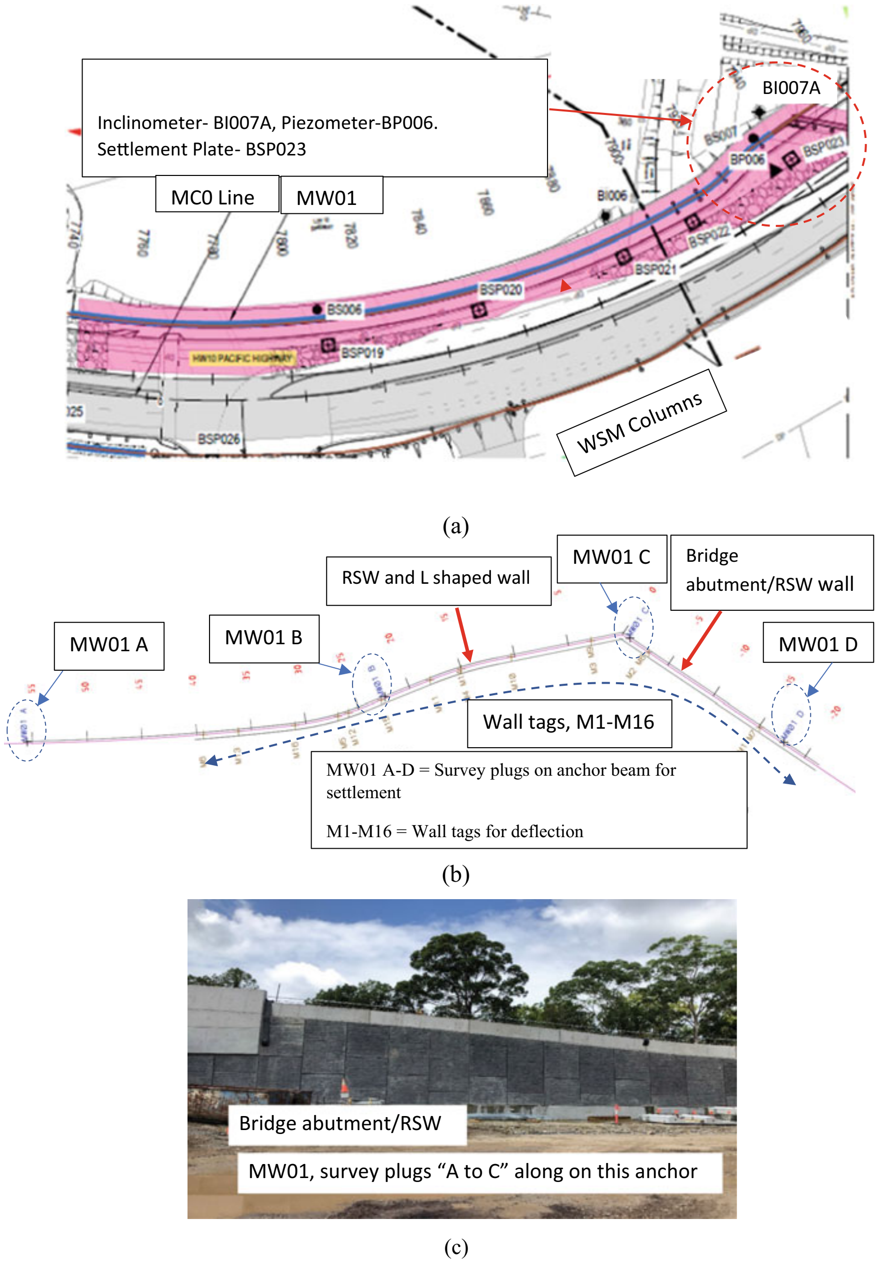


Fig. 3 Instrumentation monitoring plan and elevation of bridge approach embankment and retaining wall (**a**) Plan of settlement plate, inclinometer and vibrating wire piezometer (**b**) Plan of survey plugs and wall tags (**c**) Bridge abutment and wall elevation

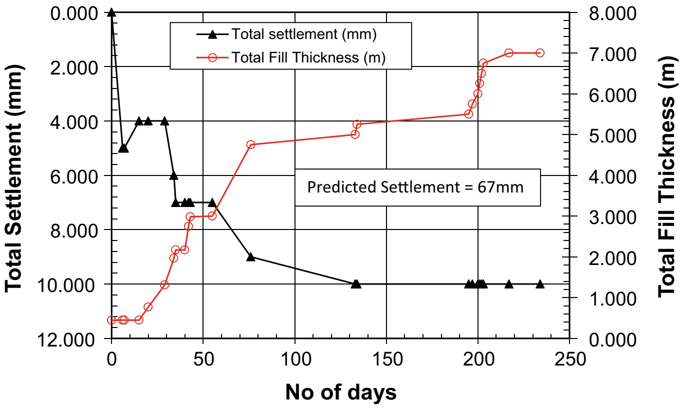
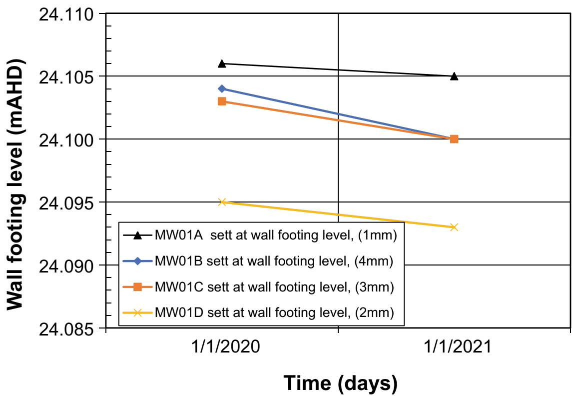


Fig. 4 Settlement monitoring results (BSP 023) for bridge approach embankment/RSW fill

Fig. 5 Settlement monitoring results of the RSW footing



finite element (FE) analyses is shown in Figs. 6 and 7 respectively. The results indicated that the predicted settlements meet the specified design criteria for pavement and retaining walls over the 40 years to 100 years design-life.

Following the comparison from settlement monitoring results (Figs. 4 and 5) and predicted analyses using Plaxis (Figs. 6 and 7), it can be postulated that post construction settlement (including creep) of this WSM treated embankment and RSW wall would be substantially smaller than the predicted values over the design life.

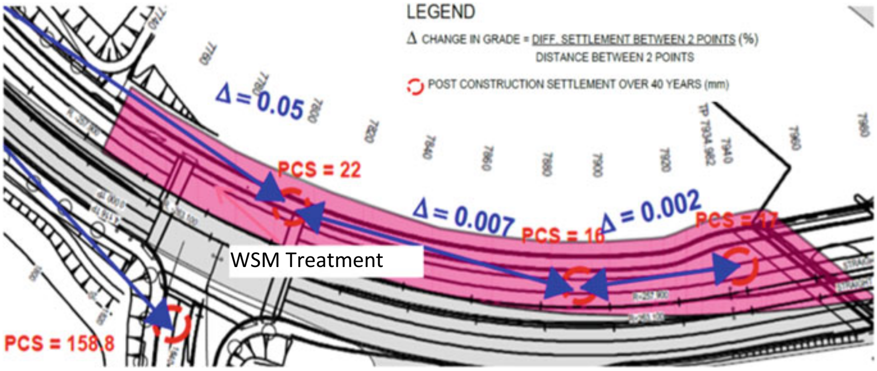


Fig. 6 Predicted 40 years PCS and differential settlement for bridge approach embankment (after Kamruzzaman et al. [7])

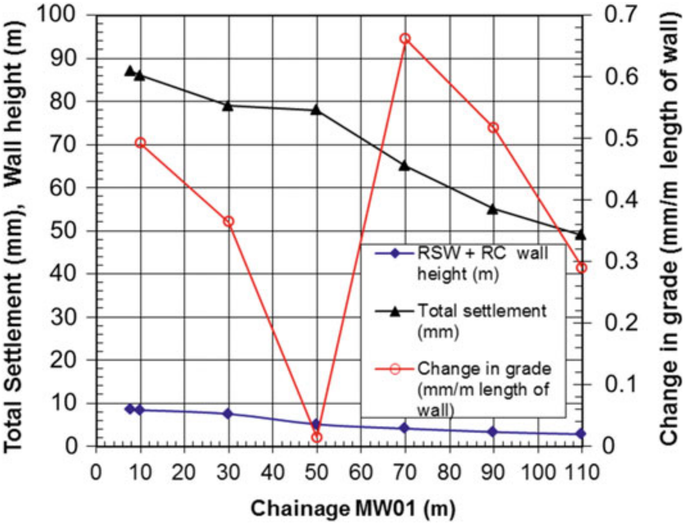


Fig. 7 Predicted 100 years settlement for the combined RSW and RC wall foundation (after Kamruzzaman et al. [7])

5.2 *Lateral Movement Results*

Inclinometers were installed at the base of the batter slopes of the RSW wall/close to the bridge abutment as can be seen in Fig. 3a. Figure 8 shows the lateral ground movement profile from Inclinometer BI 007A for the RSW combined with L-shaped wall at the bridge approach. Monitoring results suggest that the maximum ground movement within the improved ground zone is in the range of 5–1 mm only, and mostly concentrated within the 2–3 m depth from the ground level.

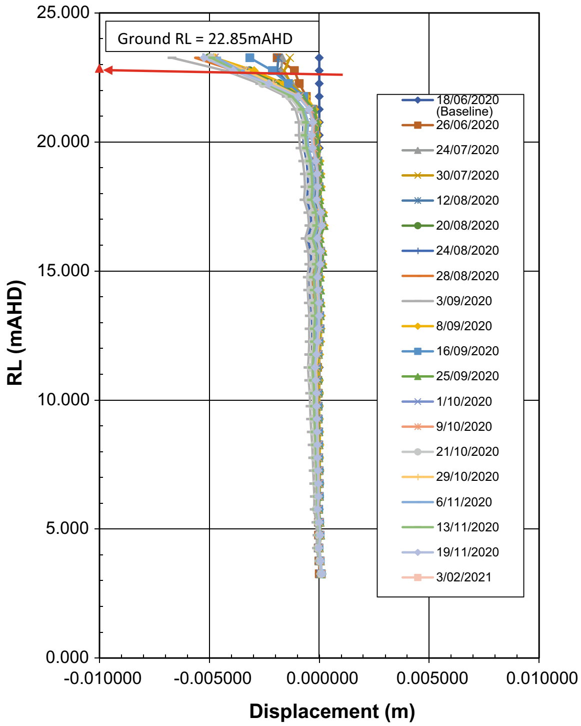


Fig. 8 Inclinometer monitoring results, X-axis is cumulative displacement (BI007A, next to the RSW wall)

Furthermore, series of survey wall tags are installed at various wall elevations, and the monitoring results are shown in Fig. 9. As can be seen, the wall movement (including post construction after January 2021) varies between 2 and 3 mm, but mostly lies within the 1–2 mm band. Both inclinometer and survey tag results also imply that the WSM treatment enables protection of RSW/bridge piles from the impact of lateral movement and down drag forces of soft soil over the monitoring periods, and likely for the whole design life of the structure.

5.3 Pore Pressure Responses

Vibrating wire piezometer BP006 was installed at the WSM treated bridge approach embankment/RSW fill as shown in Fig. 3a. Results of this piezometer reading for a period of about one year is shown in Fig. 10. As can be seen, virtually almost no change of pore pressure is noticed at various depths of the treated ground (RL 20.5 to RL 4.5 mAHD), even after reaching full thickness of the embankment fill. This result is consistent with observed smaller settlement and lateral movement monitoring results as explained above, where the treated ground becomes heavily over-consolidated.

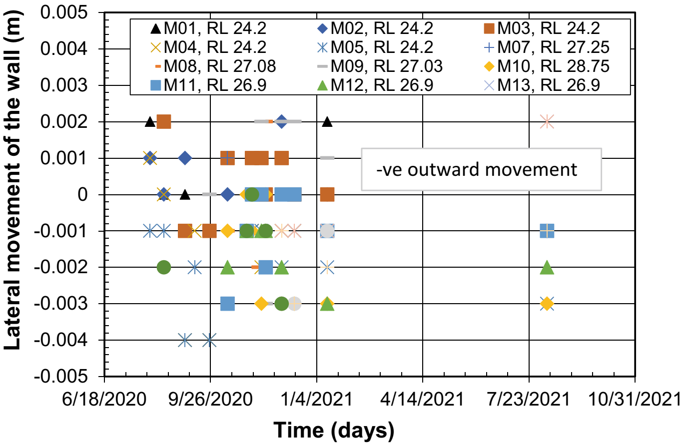


Fig. 9 Wall movement monitoring results using wall tags

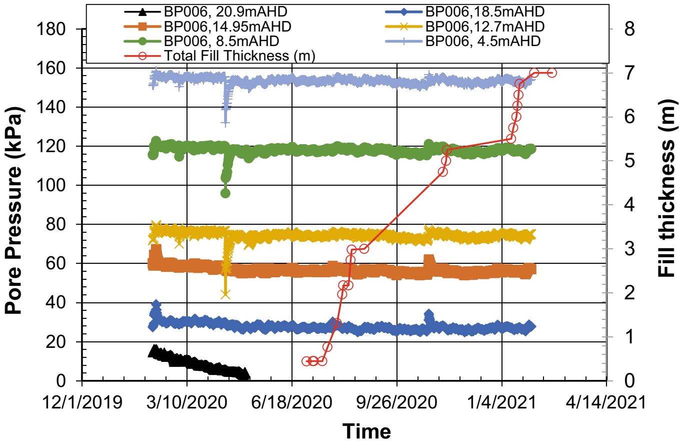


Fig. 10 Pore Pressure monitoring results (BP006) at the bridge approach embankment/RSW fill

6 Construction Quality Control

Extensive quality control testing including strength and deformation parameters of the WSM columns were in place to validate the functionality of the treated ground, and its effect on the embankment/RSW fill. A project specific WSM specification [12] comprising requirements for laboratory mixing trials, the installation of field trial columns, automatic computer monitoring during installation, field validation testing and construction records was developed based on TfNSW QA specification R223 [13]. Accordingly, all tests for this project were carried out from the three lots

Table 1 Summary of testing program

Lot description	No. of trial columns	No. of production columns	No. of full depth coring	No. of UCS testing with modulus	No. of CPT testing	No. of column head exposure
Lot 1 (Ch7740-Ch 7805)	11	103	28 (14 trial and 14 production)	179 (82 trial and 92 production)	28 (9 trial and 19 production)	12 (3 trial and 9 production)
Lot 1 (Ch7805-Ch 7890)	12	163				
Lot 1 (Ch7890-Ch 7960)	16	209				

as per the specification. A summary of the field and laboratory testing program for the three lots is shown in Table 1.

Following the testing regime as per Table 1 above, results from WSM column coring and Uniaxial Compressive Strength (UCS) testing with stress–strain measurement, Cone Penetration Tests (CPT) and column head exposure tests have been described in the following sections.

6.1 Coring and UCS Results

Coring (63 mm triple tube coring system) was performed to the full depth of 28 columns as per Table 1. A typical core log and results from all production columns coring are presented in the Fig. 11 and Table 2 respectively. As can be seen, the core recovery for all columns is over 90%, except one with 82%. This result satisfies the fit for purposes QA requirement from R224.

Furthermore, strength and deformation behaviour of the in-situ cored samples were evaluated through unconfined compressive strength testing, and the results are

Fig. 11 Typical core log of WSM sample

Table 2 Core recovery from production columns

WSM column number	Core age (days)	Core depth (m)	Core recovery (%)
190	25	21.6	99
224	25	25.0	95
374	27	21.0	99
462	22	21.5	95
240	26	22.0	95
444	24	19.4	100
473	27	19.4	100
150	23	21.4	95
45	20	18.1	95
116	27	21.6	100
66	22	18.3	82
38	21	18.2	90
426	24	21.1	100
339	25	21.8	100

shown in Figs. 12 and 13. As can be seen from Fig. 12, all cored samples cured between 28 and 56 days satisfies both strength and modulus requirements for the project, except 3 samples. These 3 samples were tested at 28 days curing period, but is expected to increase their strength and deformation modulus over the time due to pozzolanic reaction process between soil–cement matrix [2, 4, 5]. This notion is further explained through testing of few cored samples taken from the same depth, but cured at 28 and 56 days, as can be seen in Fig. 13. The result shows that strength of the WSM cored samples increases as the curing time increases. Literature review suggest that soil–cement pozzolanic reaction mechanisms continues even up to 20 years [3, 8], and thus the increase of long-term settlement and lateral movement response of the embankment/RSW fill is expected to be minimal from the measured responses explained in Sect. 5 above.

6.2 CPT Test Results

The project specific specification R224 required carrying out Pull Out Resistance Test (PORT) as per other deep soil mixing project experience and RMS QA R223 [13]. However, following the trial columns testing using PORT, it was releveled that WSM samples were too stiff, and thus the PORT tests are not suitable for WSM samples. Alternatively, 100 MPa capacity CPT tests were proposed and a total of 28 columns (19 production and 9 trial columns) were tested. Figures 14 and 15 show typical CPT plot and q_c results along with depths for 19 production columns. As can be seen in Fig. 15, the CPT test was also terminated between 2 and 14 m depths

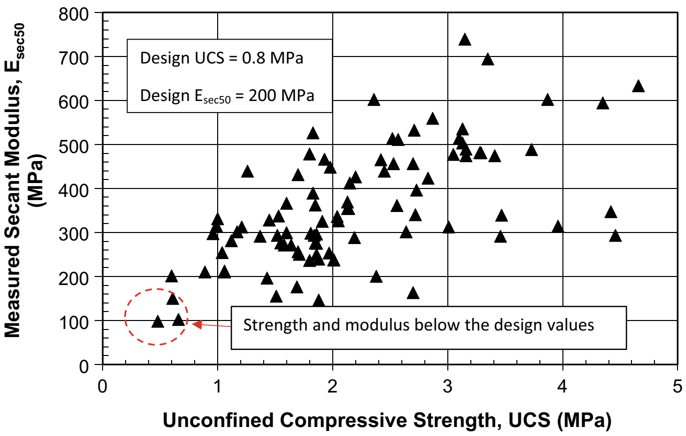


Fig. 12 UCS and E_{sec50} results of WSM cored samples

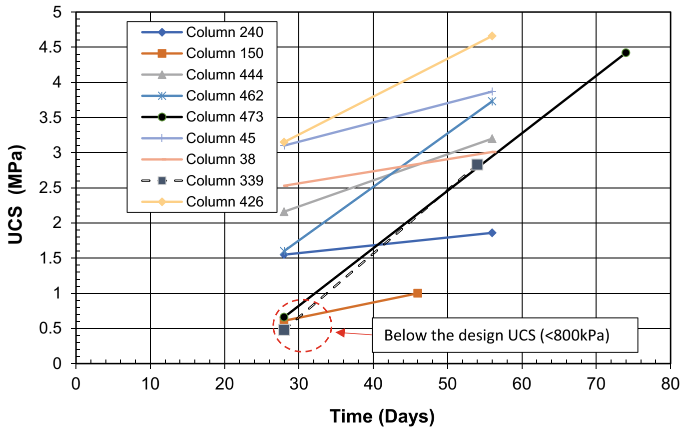


Fig. 13 Effect of curing on UCS results of WSM samples

due to high stiffness of the constructed columns for curing periods between 7 and 37 days. The results also suggest that the interpreted undrained shear strength from q_c values (using a conservative factor of 20) satisfied the design requirement.

6.3 *Column Head Exposure*

As per the specification, 12 column heads were exposed for visual examination and pocket penetrometer testing. Figure 16 shows typical 3 exposed columns at 1.1 m depth. Following the visual examination, the columns were found to be non-defective

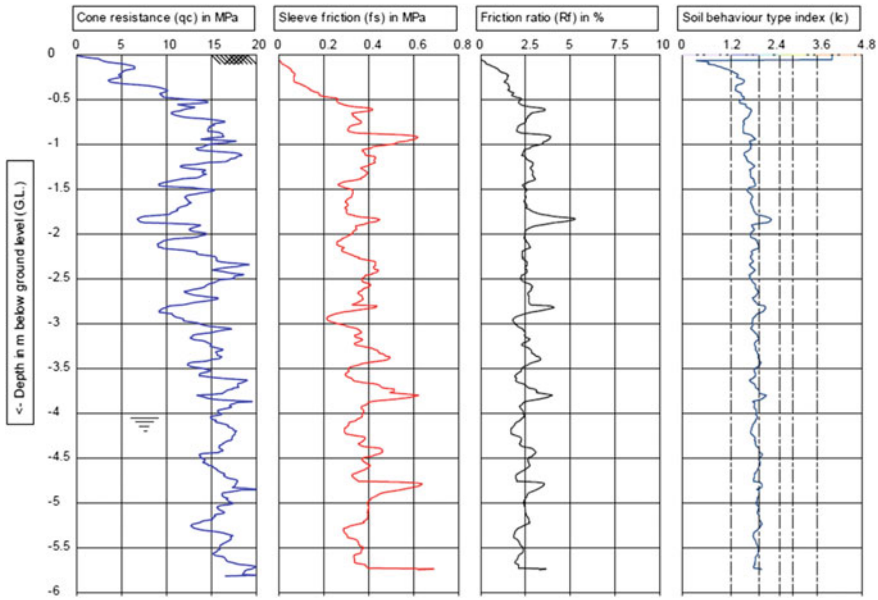


Fig. 14 Typical CPT results of WSM column

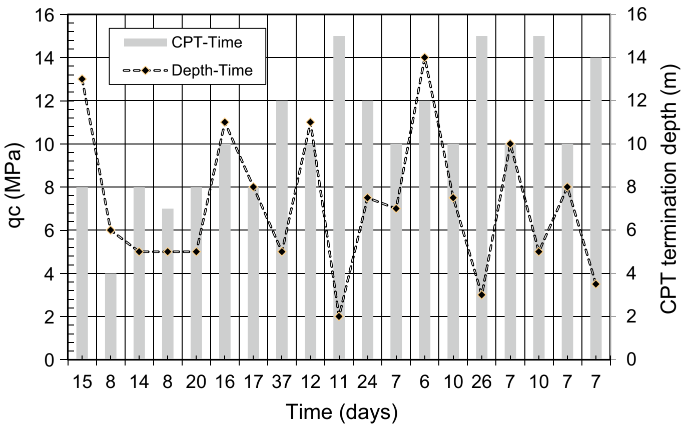


Fig. 15 CPT results of WSM samples along with termination depth

(e.g. no crack), and pocket penetrometer results also show much higher interpretative compressive strength than that of the cored UCS values as explained in Sect. 6.1 above.

Fig. 16 Exposed column head of WSM columns at 1.1 m depth

7 Lesson Learnt

7.1 Use of Recycled WSM Waste

During the design, several project discussions were held with the specialised subcontractor, and only a small quantity of spoil was anticipated from the Wet Soil Mixing process. However, during a meeting, prior to equipment arriving on site, it was advised that the quantity of spoil would be almost equal to the quantity of grout pumped into the ground, due to the new equipment (refer to Fig. 17), which is not usually used for WSM treatment. The contractor was then responsible for the disposal of 3,000 to 6,000 m^3 of soil–cement slurry/spoil that had not been envisaged or budgeted for. There were concerns about the cost and impacts on the project budget due to removing of this large quantities of spoil offsite as a General Solid Waste (GSW). However, after much discussion with various stakeholders including TfNSW environment and geotechnical subject matter experts, consultants, contractor, specialist subcontractors, and several successful trial, almost 5,000 m^3 of spoil was able to be incorporated within the site works.

Several treatment processes were adopted at site for the spoil. Initially soil was classified as liquid waste as a slurry (refer to Fig. 17), and then as it sets, classified as General Solid Waste (non-putrescible) material. It was also understood that once the material is processed, it cannot be re-used as Excavated Natural Material or Virgin Excavated Natural Material. However, when the material becomes solid the

Fig. 17 WSM construction rig and produced recycled soil–cement waste

resource recovery exemption for "public road materials" within the road corridor as "TfNSW excavated road material" can be applied (in a similar way as concrete wash out material). As a result, the end product became a cement stabilised sandy silt material that performed well during and after rainfall events. The material had a high pH, and thus runoff was monitored and contained within low bunds. *Finally, the spoil was used in temporary working platforms, temporary access tracks, a levee bank, and below the pavement in the carriageway and a carpark. The successful incorporation of the spoil within the project boundary reduced the projected staged material importation requirements for the project.*

7.2 WSM Mixing, Trial and Construction Issues

Following the soil–cement spoil issues raised in the previous section during the planning stage of the construction, a series of laboratory mixing comprising different percentages of cement content, water to cement ratio, curing time and construction of field trial columns were planned as per QA specification R224. During the trial column installation at three lots, it was revealed that pre-drilling is required using additional amount of water in the range of 150–180L/m^3 of soil, to minimise the production of soil–cement spoil and also to penetrate interbedded stiff/dense soil layers. Furthermore, the trial columns confirmed that design depth of some production columns could be terminated at a higher level from the drilling refusal using hydraulic pressure of >270 bar. As a result, 131 WSM production columns were constructed at least an average of 1.5–2.5 m shorter than the design depth. Finally, from three lots, the project constructed a total of 10,426 linear meter (including 678 linear meter during trial) WSM columns having 1.2 m diameter with a cement content of 300 kg/m^3 of soil and net w/c ratio of 1.

7.3 WSM Testing and Deviation from R224

The project specific specification R224 required carrying out Pull Out Resistance Test (PORT). Accordingly, few trial columns were tested using PORT, and it was releveled that WSM samples were too stiff, and PORT tests are not suitable to verify the strength of the full length of the production columns. Alternatively, 100 MPa capacity CPT test was then proposed and a total of 28 columns (19 production and 9 trial columns) were attempted to test. However, following the results from CPT tests as explained in Sect. 6.2 above, it proved that CPT test was also terminated at a shorter depth, as it deviates from the column centre due to very high stiffness of the constructed WSM columns. This means in-situ testing for the entire length of the columns are not possible using even CPT rig, when they are required to be tested for 28 days design strength requirement. Only full depth coring (using 63 mm triple tube core barrel) and UCS testing along with deformation modulus measurement will satisfy QA requirement.

8 Conclusions

Deep Wet Soil Mixed (WSM) ground treatment columns have been designed and constructed for the first time on TfNSW Lisarow to Ourimbah Stage 3B pacific highway upgrade project, and the following conclusions are drawn:

(a) The soft clay deposits (interbedded with loose to medium dense sand and stiff clay) at the Main Northern Railway Bridge approaches have a high compressibility, low shear strength, and low permeability. Constructing high embankments and retaining walls (up to 8.5 m) over these poor ground conditions would have resulted in large total and differential settlements if ground treatment using WSM were not adopted.
(b) Following the rigorous laboratory mixing and field trials, WSM column diameters of 1.2 m with a length up to 21 m, and square pattern spacing from 2.2 to 3.7 m were constructed to control total and differential settlements of the retaining wall foundations, and to provide a smooth transition for the bridge approach embankments. The adopted WSM ground treatment design satisfied various challenging requirements of the project such as construction programming, complex geology and ground treatment costs.
(c) The instrumentation monitoring results obtained from settlement plate, inclinometer, piezometer, survey plugs and wall tags suggested that primary consolidation settlement of the WSM treated ground is virtually completed within

1 month after the construction. Field monitoring results also reveal that settlements and lateral movements of the bridge approach embankments and RSW foundations are much less than the predicted design values. In addition, the trend of the 12 months monitoring results provides confidence on the long-term performance of these structures, which is expected be better than the predicted ones.

(d) Furthermore, it is revealed that after an extensive field trial, and close collaborative work between various stakeholders including TfNSW environmental and geotechnical subject matter experts, contractor and specialist subcontractor, about 5000 m^3 of soil–cement mixed waste material/spoil was treated at site. This spoil was then used in temporary working platforms, temporary access tracks, a levee bank, and below the pavement in the carriageway and a carpark. The successful incorporation of the spoil within the project boundary reduced the projected staged material importation requirements for the project.

(e) Following the several field testing regime, it is concluded that only Unconfined Compressive Strength (UCS) testing from the cored samples is the most reliable quality control measure for high strength WSM materials. Neither PORT nor CPT testing is recommended for high strength WSM materials (~UCS > 400 kPa). Results also suggest that direct measurement of modulus of WSM columns are important in addition to the strength parameter. Both strength and deformation parameters of the field core samples proven to be increased with the increase of curing time, which is consistent with findings from many literature review. Hence, it is recommended to include the curing effects on the design of deep soil mixed columns for future projects.

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Improvements to the Observational Method in New South Wales Road Tunnel Construction

Phil Clark

Abstract The past 5 years has seen an unprecedented boom in tunnel construction in Sydney. Road tunnels in particular, continue to push both design and construction to their limits, no less than when Sydney Harbour Tunnel was constructed 30 years ago. Integral to the safe and efficient construction of road tunnels has been the application of the Observational Method in design and construction. This paper describes some important "lessons learned" in implementing the Observational Method in New South Wales road tunnels since the construction of the Sydney Harbour Tunnel. Underground infrastructure construction in New South Wales in the early 1990's had very few precedents. Construction of the major underground excavations relied on application of the principles of the Observational Method as described by Peck in 1969. This concentrated on validating design assumptions against detailed monitoring data. As confidence in the ability to predict the behaviour of rock masses in the Sydney Region increased, it could be argued that appreciation of the fundamentals of the Observational Method diminished to a process of collecting data for the sake of collecting data, rather than being a live tool to identify and manage geotechnical hazards. The tunnel collapses in the Cross City Tunnel (2004) and Lane Cove Tunnel (2005) led to the industry reassessing tunnel construction risk management. The Permit to Tunnel (PTT) process was born, and at its heart, it provides a means to manage geotechnical risk via a formalised process that includes reviewing of observations by both designers and constructors and agreement to continue construction, within the main principles of the Observational Method. However, over the past 5 years, the author has observed waning deference to the Observational Method. Construction processes used on major projects, including the Observational Method, the PTT and associated processes have become opportunities for contractors and designers to modify certified designs in an ad hoc manner, often without design changes being adequately reviewed against key criteria such as safety, stability and durability. This paper aims to identify key concerns with the current implementation of the Observational Method through the PTT process. Recommendations are

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proposed to reset practice to ensure designs are constructed safely and owners are provided with assurances that construction delivers the intended design.

Keyword Observational method · Permit to tunnel · Risk management

1 Introduction

"The real purpose of the scientific method is to make sure nature has not misled you into thinking you know something you actually don't know" [18]. Although originally written in the context of maintaining motorcycles, this truth is applicable to both to scientific investigations as well as the construction industry. Pirsig [18] then states "if you get careless or go romanticising scientific information, give it a flourish here and there, nature will soon make a complete fool of you".

Tunnel construction is intrinsically linked with nature as humans interact with and ultimately attempt to control nature, where nature is the ground in which tunnels are constructed. Linked with attempting to control nature is the application of the Observational Method.

Nature always throws challenges, but often the challenges are self-imposed. For example, the recently constructed M8 tunnel, in Sydney, Australia, has underground twin caverns where one motorway merges into another that, at the time of construction were the widest constructed tunnels in New South Wales, and among the widest sections of road tunnels in the world. Further to this, they are also among the deepest road tunnels in the Sydney Region, where high horizontal in situ stress more often than not results in relative construction difficulties and they are located within the Woolloomooloo Fault Zone, a complex zone of faulted rock at least 200 m wide. The caverns are also located below an approximately 40 m deep palaeochannel infilled with Quaternary sediments with direct hydrogeological connectivity with the tidal Cooks River [17]. Normally one of these geological hazards would be enough to try and relocate the tunnel, but to have a number of major geological hazards in the same location and at the same time attempt to push design and construction precedents, suggests this part of the project was not business as usual. As road projects become more and more complicated as a response to consumer demands and expectations, projects will contain ground risks that are more challenging than those faced in the past. Consequently, there also needs to be more developed, mature and smarter methods to manage ground risk.

This paper discusses improvements to the Observational Method that is currently applied in New South Wales tunnel construction to ensure risks related to tunnel structures are adequately managed. The improvements can be readily adapted to other forms of geotechnical and civil construction.

2 Background

2.1 The Observational Method

Nicholson et al. [13] defines the Observational Method as:

> The observational method in ground engineering is a continuous, managed, integrated process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate. All these aspects have to be demonstrably robust. The objective is to achieve greater overall economy without compromising safety.

The method can be implemented at any stage of a project to realise benefits, but should only be used when there is sufficient time in construction to implement planned modifications or emergency plans and where reliable observations can be made. It can also be used as a 'best way out', when implemented after a construction incident occurs.

This can be contrasted against predefined design, which Nicholson [13] described as:

> A single robust design is fully developed before work starts on a particular phase. Monitoring is sometimes used, but in a passive way to confirm design predictions are not exceeded. There is no primary intention to vary the design during construction.

Associated with any definition of the Observational Method is a need to describe what it is not. [15] noted that Observational Method was becoming discredited by misuse, being invoked by name but not deed and:

> Among the essential but often overlooked elements are to make the most thorough subsurface investigations that are practicable, to establish the course of action on the basis of the most probable set of circumstances and to formulate, in advance, the actions to be taken if less favourable or even the most unfavourable conditions are actually encountered. These elements are often difficult to achieve but the omission of any one of them reduces the Observational Method to an excuse for shoddy exploration or design, to dependence on good luck instead of good design. There are far too many instances in which poor design is disguised as the state of the art merely by characterising it as an application of the Observational Method.

2.2 Sydney Tunnelling Boom

Thirty five years ago, tunnel designers in New South Wales (NSW) were uncertain how to excavate tunnels in a safe and efficient manner. The first major project was the Sydney Harbour Tunnel (opened in August 1992), which included a combination of major infrastructure, including cut and cover structures, an immersed tube and conventionally mined and shaped tunnels. At the same time as the Sydney Harbour Tunnel was being constructed, a number of other underground structures in Sydney that utilised the observational approach in combination with very detailed monitoring and back analysis, were also being designed and constructed. These structures included the infamous the Opera House Carpark, for which special analytical design tools were developed [16]. The tunnel construction boom in Sydney continued throughout the 1990s and early 2000s where tunnel construction was largely based on the Observational Method and where tunnel designs continued to push the limits of tunnel construction. These tunnels included the Eastern Distributor, a piggy back road tunnel that has the northbound carriageway sitting above the southbound carriage way in a single excavation. Unfortunately, in the mid-2000s, two major incidents occurred, both related in one way or another to the application of the Observational Method as it was applied on these projects at the time. A rock fall from the tunnel crown on the Cross City Tunnel, resulted in a fatality and a piping failure on Lane Cove Tunnel, where a vertical, dyke, altered to clay passed through a low cover tunnel intersection, resulting in significant damage to an apartment building [2]. An outcome of these incidents was that, despite a recently announced pipeline of tunnelling projects, the appetite to construct further tunnels in Sydney diminished.

Following on from these incidents, there was a contractor lead initiative to develop a more robust approach to tunnel excavation, also known as the Permit to Tunnel (PTT). The PTT formalises processes inherent in the Observational Method, but in doing so suffers from the same criticism many provide to quality systems whereby they can be seen as a tick box exercise (e.g. [3]).

Despite the lack of major tunnel construction in the late 2000s and early 2010s, since 2015 there has been an explosion in volume of tunnels constructed in Sydney, including one major metro line (delivered in two stages and much of it underground), more metro lines in procurement and a number of major road tunnels, including NorthConnex, WestConnex, M6 Stage 1 and the Western Harbour Tunnel and Beaches Link program of works. Each of these projects have unique features, and each have set a number of design and construction precedents in the Sydney region. For example, NorthConnex, is the longest and deepest road tunnel in Sydney, being around 10 km long and up to 95 m below the surface. WestConnex is being delivered in a number of stages, two of which are now operational (M4E and M8) and both have set a number of construction precedents in the local geological environment. The final two stages of WestConnex (M4M8 Link and Rozelle Interchange) are under currently construction and include the largest underground mined caverns, with 35 m spans, and more than 50 cross over locations, where one tunnel passes above another and where there are 4 levels of road tunnels and a future metro tunnel passing below. This WestConnex project connects a number of significant roads, including Australia's busiest road, Victoria Road, and the future Western Harbour Tunnel (WHT).

2.3 Development of the Observational Method

The observational approach to construction has been around since the dawn of civil engineering as a profession. Engineers have always used observations to support construction approaches, mainly due to a lack of design theory and where structures have essentially been designed using trial and error [13]. Nicholson et al. [13], describes applications of the Observational Method in the middle ages as well as mining in the nineteenth century.

Formal adaption of observational approaches in geotechnical engineering have occurred since the development of geotechnical engineering as its own discipline. Early practitioners such as [23, 24] identified that the ground often varied from that found during a site investigation stage and that the design would need to be modified if the as found conditions were less favourable than the designer assumptions. In their 1967 textbook, Terzaghi and Peck noted that the design on the basis of most unfavourable assumptions is inevitably uneconomical and that if modifications to the design during construction could be made, then savings could be provided.

The Observational Method as such was formally introduced by Peck in 1969 [14]. Peck considered that the application of the observational method included:

- Sufficient site investigation to establish the general nature and properties of the ground.
- Assessment of the most probable conditions and most unfavourable conditions.
- Design and ground behaviour based on the most probable conditions.
- Development of a new design in advance of any significant foreseeable deviation from the most probable conditions, based on observations made during construction (planned modifications).
- Application of the design modifications to suit the as found conditions.

Since its introduction, the observational method has been used in numerous engineering applications, including tunnels, embankments, retaining walls, piling works, groundwater control and hazardous waste remediation, and incorporated in Eurocode 7 [4].

In 1999, CIRIA published a technical guide on the Observational Method that included both technical and commercial guidance as well as examples from construction projects, including tunnelling [13]. Key updates from the Peck [14] description of the Observational Method include:

- Differentiation between planned modifications and progressive modifications and where a planned modification is only recommended where previous case histories in comparable ground conditions or precedents exist.
- Progressive modification is a design based on more probable condition and where designs are modified progressively towards the most probable condition based on back analysis and re-assessing previous design predictions.
- Identification that more probable designs are based on design parameters that are between moderately conservative and most probable. They are often referred to as cautious estimates.

More recently, the Observational Method has been adapted into a systems engineering approach to manage construction risk, particularly on complex engineering projects such as tunnel construction [10].

2.4 Use of the Observational Method in Sydney Tunnels

Construction of the major underground excavations in Sydney tunnels have relied on application of the principles of the Observational Method as described by Peck [14]. Initially, this concentrated on validating design assumptions against detailed monitoring data. The approach then changed to developing a suite of designs, with a particular design to be implemented depending on the as found ground characteristics. An example of the ground classification from Lane Cove Tunnel is provided as Table 1 (from [2]). Associated with each ground type was a support type, which in conjunction with an associated construction sequence, formed the design. It was generally not possible to know what design would be constructed until the ground was excavated, hence accurate geological observations were critical to the process. Generally, surface monitoring would not be undertaken in a manner that could influence construction and in tunnel monitoring was typically processed after the support type had been recommended. For higher risk areas such as low cover to the surface, or where third party impacts could eventuate, increased monitoring (including real time monitoring) and increased geological surveillance was undertaken, however, for the majority of tunnelling activities in Class I, II or III Sandstone or Shale, monitoring results did not influence design or construction methods. The general observational approach of utilising a set of design solutions in combination with geological observations was also noted in ICNSW [6].

> The roof support installed at any tunnel location on the Project was dependent on the IFC documentation and the ongoing assessment of the "as found" geological conditions. Both the design and construction of MVT-1 were to be carried out based on this documentation and assessment.

Hence, critical to the implementation of the design are observations by geologists. However, it is not often possible for geologists to provide accurate assessments of the as found conditions. Often observations would be made in cramped, dusty, poor light conditions, where a geologist is expected to interpret the rock mass characteristics 4–5 m above their head, in the roof of a tunnel. In the workplace safety prosecution of the Cross City Tunnel (CCT) contractor, the judgement noted that there appeared to be no technical review of the geological mapping that should have identified the risk to the excavation and would have led to a technical intervention of a change in support type [7, 8]. This would indicate that geological mapping was, in this instance, being done for the sake of record keeping only and not as a tool or method to identify and manage geotechnical hazards. Likewise, in the Lane Cove Tunnel (LCT) Supreme Court decision, that determined the cause of the piping incident, the absence of commentary by the geologist as to whether or not there were any non-compliances to the construction sequence, indicated to the judge that the geologist thought the construction approach was acceptable [11].

After the CCT and LCT incidents, this author understands that key contractors and designers worked together to develop a better process to implement the Observational Method. This process became more commonly known as the Permit to Tunnel (PTT).

Table 1 LCT ground classification system for shale ground types

Ground type	Strength	Saturated UCS (MPa)	Defects	Typical Sydney rock class	Q value
LCT G6	Medium to high	>7	2 Joint sets plus random (Bedding is one set) Dip of joints >45° Discontinuity spacing >0.6 m Minor shear zones, faults, dykes Minor clay seams or weak beds Dry or minor water inflows	Class I–II shale	>2.2
LCT G7	Low	2–15	2 Joint sets plus random (bedding is one set) Discontinuity spacing >0.6 m Minor shear zones, faults, dykes Minor clay or sandy beds, seams or joints Dry or minor water inflows	Class III shale	>0.2
LCT G8	Very low	<2	2 Joint sets plus random (bedding is one set) Discontinuity spacing >0.02 m Minor shear zones, faults, dykes Minor weak clayey or sandy beds, seams or joints Dry or high water inflows	Class IV shale	<1

(continued)

Table 1 (continued)

Ground type	Strength	Saturated UCS (MPa)	Defects	Typical Sydney rock class	Q value
LCT G9	Extremely low	<1	2 Joint sets plus random (bedding is one set) Discontinuity spacing <0.2 m Fault zones with crushed, weathered or broken rock, vertical or sub-vertical features such as weathered Dykes and associated clay infill Significant iron staining Dry or high water inflows	Class V shale	<0.27

This process was first implemented on Brisbane's Airport Link Project in 2008. The Permit to Tunnel (PTT) process is discussed in greater detail in Sect. 3.

2.5 Workplace Health and Safety

The design, construction and operation of tunnels in Australia are governed by the requirements of each state's Workplace Health and Safety Act and Workplace Health and Safety Regulations. These acts and regulations are harmonised across Australia. There are a number of Codes of Practice that have been developed to provide practical guidance on how the requirements of the WHS Acts and Regulations need to be implemented. The key Codes of Practice applicable to tunnels in New South Wales are the Safe Design of Structures [20], Construction Work [21] and Excavation Work [22]. These Codes of Practice define acceptable work practices and responsibilities for clients, contractors, designers and operators to ensure the health and safety of construction workers, operators and tunnel users. Other Codes of Practice that are still valid in New South Wales include 'Tunnels under Construction' [25]. Associated with the codes of practice, Safework Australia has also developed a guideline for tunnelling works [19].

Within the legislative framework, it is important to understand the responsibilities of designers, contractors and clients with respect to the application of the Observational Method in New South Wales tunnel construction, as aspects of the Observational Method are considered in different codes of practice. Further to the codes of practice requirements, the Safework Australia guideline identifies that ground support in particular, should follow an observational approach. This, therefore, acknowledges that the Observational Method is the preferred approach to be followed in tunnel construction.

2.5.1 Client Responsibilities

The Client commissioning a project needs to consult with the designer about how health and safety risks arising from the design are eliminated or minimised so far as is reasonably practicable. They also need to provide the designer and contractor with any information in relation to hazards and risks on the construction site. The Client needs to agree to the use of the Observational Method. This is normally inherent in the awarding of any contract to design and construct a tunnel.

2.5.2 Principal Contractor Responsibilities

The majority of New South Wales tunnels are design and construct contracts, where the client requires the contractor to manage and control the workplace. The principal contractor has a duty to ensure the construction work is planned and managed in a manner that eliminates or minimises health and safety risks so far as is reasonably practicable. The observational method is one such way where the contractor is able to demonstrate that safety risks are managed so far as is reasonably practicable as overly conservative design using the most unfavourable design applied everywhere can be seen to be neither reasonable nor feasible.

2.5.3 Designer Responsibilities

Designers have a number of responsibilities in the design of safe structures including:

- Assessing known hazards and risks during the design stage,
- Assessing what is reasonably practicable,
- Applying the relevant design standards.

Designers do not have management and control over construction, but they must consult, cooperate and coordinate design activities with their client. Their responsibilities are limited to their original design and not any modifications by others. This is where clearly defined processes relating to the application of the Observational Method are critical as it is often not clear who is the designer. The Supreme Court decision into the cause of the Lane Cove Tunnel collapse (refer to Burman et al. [2]

for further information on this incident) found that three parties were the designers of the location of the collapse [12]. At the location of the collapse, the main designer had mandated a design. However, another company that provided construction phase services added further design requirements by way of a ground support determination (GSD). As the design was mandated in this location, the purpose of the GSD at this location was to identify any additional ground support that may be required, and also to identify whether or not the contractor had been following the design. The design drawings and GSD instructions were then reinterpreted by the contractor as a Site Instruction (SI) that in turn further modified the design, hence all three parties were judged to be the designer.

2.5.4 Safe Design

Safe design means the integration of control measures early in the design process to eliminate, or, if this is not reasonably practicable, minimise risks to health and safety throughout the life of the structure being designed.

The safe design of a structure will always be part of a wider set of design objectives, including practicability, aesthetics, cost and functionality. These sometimes-competing objectives need to be balanced in a manner that does not compromise the health and safety of those who work on or use the structure over its life. As noted earlier, application of overly conservative design may not be considered to be reasonably practicable, but any alternative, including an Observational Method, must still be undertaken in a safe manner.

2.5.5 Safe Construction

Management of project risk is often implemented through identification of hazards, assessment of risk, then management of risk throughout design, construction and operation. Risk management includes processes to transfer and manage risk between various risk owners, including multiple designers, contractors and the client or operator and project stages.

It can also be difficult to clearly delineate who has responsibility for the management of hazards and risks associated with a structure as designer responsibilities for separate design elements may be concurrent and overlapping and construction of the elements may also overlap. Therefore, it is important to establish clear communication processes to ensure all parties are clear on who owns a particular risk at each development phase of a project.

3 Permit to Tunnel

3.1 Background

The Permit to Tunnel (PTT) provides a means to manage geotechnical risk associated with the application of the Observational Method, via a formalised process that includes reviewing of observations and monitoring data by both designers and constructors as well as an agreement to continue construction. It captures the main principles of the Observational Method as defined above by Peck [14] and Nicholson [13]. Generally, it provides a process that controls the selection of ground support and the excavation sequence to ensure the safety of the construction works and third party impacts, whether they are surface infrastructure or adjacent underground infrastructure, including deep basements. Typically a PTT procedure, plan and forms are developed as a suite of documents. Additionally, similar documentation may be provided for shafts and excavation within retaining structures, often referred to as a Permit to Excavate (PTE). Documentation should clearly identify the roles and responsibilities and the procedures to enable functions of the roles to be fulfilled.

As indicated above, the Permit to Tunnel (PPT) is a relatively recent development in New South Wales tunnelling with the first documented use on a transport tunnel being the M2 Widening. The PTT process on that project is briefly described in [5]. The purpose of the PPT system on the M2 tunnel widening project was to ensure tunnel stability was rigorously and consistently reviewed before allowing traffic back through the tunnel following excavation works. The system required a high degree of reliability to ensure safety to the public and involved acceptance and sign off by both contractor and design representatives on a daily basis. One of the key inputs to the PTT was the observations made as part of the geological mapping process. Another key input to the PTT process was monitoring data to check the adequacy of the ground support and confirm the validity of the design [5].

The PTT may be developed with a number of lenses, namely a mechanism to provide immediate advice to the construction team, a daily review of all available and relevant information and a weekly review or audit and calibration across project sites. There is normally a team involved in the PPT process and includes site engineers, geologists, monitoring and survey staff, designers and site managers. Good practice involves all in the PTT team agreeing the contents of the PTT including the decision to implement the next round of ground support. The next round of ground support cannot be installed until the PTT has been approved and signed by the PTT Team.

3.2 PTT Process

During excavation and the implementation of the design, the process typically requires the actual ground conditions and behaviour to be assessed, then the appropriate support type to be installed over the life of the PTT (typically 24 h). As noted for the M2 Widening, the purpose of this process is to confirm that the excavation and allocated ground support are stable to enable work to proceed in a safe manner. This process normally is undertaken in three key stages, with a fourth stage functioning as a process review or audit stage:

1. Inspection and collection of relevant information
2. Assessment and interpretation of the information
3. Determination of the support requirements
4. Process review/audit.

3.2.1 Inspection and Collection of Relevant Information

The inspection and collection of relevant information typically or should involve:

- geological mapping of exposed excavation from a safe working area
- monitoring, measuring or estimating groundwater inflows, depending on access to the inflows
- witnessing rock bolt hole drilling in the tunnel crown to assess the strength of rock in the tunnel crown
- collection of available monitoring results, including in-tunnel convergence, extensometer and shear displacement monitoring, and surface monitoring, including surface ground movement (both conventional terrestrial survey and InSar satellite data), surface inclinometer and extensometers, building survey and groundwater monitoring, as required by the instrumentation and monitoring plan
- collection of construction records, including rock bolt installation records, structural lining thickness records and tunnel excavation profile records

3.2.2 Assessment and Interpretation

Information and observations collected during the Inspection and collection stage, needs to be reviewed and assessed for adequacy, reliability, consistency and then compared with the design assumptions and any specific requirements on the design drawings. This assessment is typically undertaken daily, but could be more frequently if there is a significant change in ground conditions. This assessment may be undertaken by a number of means. Often there is a draft interpretation and assessment provided for members of the PTT team to review before a formal PTT meeting. This pre-meeting evaluation is typically undertaken to facilitate the determination of multiple PTTs in a timely manner.

3.2.3 Determination of the Support Requirements

The assessment by the PTT team is used to provide a determination of the required ground support, which is typically one of the pre-developed design solutions. The pre-developed design solution may require additional requirements, such as spot bolting and groundwater control. Additional instructions such as repairs to previously installed structural linings could also be included.

The PTT form is typically signed by the PTT team as a means to endorse the requirements. The process is very similar to a hold point release. The form may be 1–2 pages and composed of a series of checklists the approved support allocation, the time period that the PTT is valid for and space for additional instructions and requirements.

There is generally an ability to increase the ground support if adverse situations are encountered, but controls are normally added to ensure that the required support in never reduced. Situations where the support requirements may need to increase include:

- where excavated conditions do not match the requirements of the determined ground support
- changes in excavation equipment or construction sequence (typically site plant management issues)
- changes to groundwater inflows
- over excavation
- exceedances of monitoring trigger levels
- excessive in situ stress effects

These observations are typically made by the construction team and may need confirmation by a geologist or geotechnical engineer as part of any process to assess the suitability of the current PTT.

3.2.4 Process Review/Audit

At a predefined time period, often weekly, a process review and audit may be undertaken. If the project has an independent verifier or certifier, then they may also review or audit the adequacy of any PTT assessments and determinations. This stage is also an opportunity to review and calibrate the performance of geologists and surveyors collection relevant information as well as review the previous week's geological mapping and monitoring data in a more holistic manner.

3.3 Limitations

Although the PTT process is a documented method where a team decides on the next design solution to be implemented, it does has its limitations. One of the more important limitation is the fact that the implemented design is rarely certified as being fit for the intended purpose and as the selection of the design solution is typically approved by a team, accountability between the contractor and design for the selection of a design solution is never clear. As indicated above in reference to Lane Cove Tunnel, if an incident were to occur, it may require a court judgement and many years to understand which entity is responsible. Design solutions or options are typically certified by an independent competent party (often referred to the Independent Certifier or IC) and the PTT process is often certified, with occasional checks and audits, but the actual solution that has been implemented is rarely, if ever, certified. This does create an element of uncertainty as to the suitability of the constructed solution and also enables ad hoc design changes to be provided via instructions on the PTT Form. Ad hoc design changes are discussed further in the next section.

The Observational Method as currently applied in Sydney typically does not have any contingency plans or designs ready to be implemented, other than an extension to the PTT process that is more commonly known as the Monitoring Action Team (MAT) meeting. Where monitoring alarm triggers are exceeded, or the ground is not found to be suitable for the approved ground support, The PPT or MAT team normally reconvenes, maybe with additional support from the designer to develop a solution or identify how they will respond to the monitoring alarm being triggered. This is a reactive approach and not consistent with the Observational Method defined by Peck [14]. The consequences of this is that an unsafe incident could eventuate in the time taken to consider the next action. Alternatively there could be a slip in the construction programme. Both could be avoided with better foresight and design documentation.

Another significant issue is the time typically available to adequately consider the relevant documentation. With more than 20 active excavation headings now being a typical practice in Sydney, finding the time to adequately assess in excess of 20 PTT data packs is a challenge. This then results in inadequate decision making, or the opinion of one engineer carrying more weight than ordinarily would be the case in a team decision, which in turn creates a risk of inappropriate ground support being installed if the engineer has misinterpreted the ground conditions.

3.4 Current Trends

In recent major tunnelling projects in Sydney, a very similar process to the PPT has been used to formalise the selection of groundwater control solutions. This approach is consistent with the Observational Method, but as with the PTT process, there is a tendency to provide ad hoc design requirements via the PTT form. Assessing groundwater control solutions via a PTT process also includes decisions to undertake pre-grouting, post-grouting or installation of drainage and waterproofing elements. Grouting decisions in particular are often left to the contractor to decide the best course of action as whilst pre-grouting ahead of the excavation is generally seen as the most effective form of grouting, the construction program can slip. Consequently, contractors may choose to undertake post-excavation grouting, despite it not being as effective, to ensure the construction program remains on track. Although this may be the contractor's risk to manage, the reality is there can be long lasting impacts to both the tunnel durability and surface third party infrastructure if groundwater related risks are not adequately managed by the contractor.

Major tunnelling projects often have a complicated approach to the development of design documentation. Bertuzzi [1], in particular has provided an example of the design documentation and review process from a recent tunnelling project. Once a design is eventually certified, there is a general hesitancy to change the design as it means going through the design resubmission and review cycle again. The PTT process is being seen as a means to undertake ad hoc design changes or to defer aspects of detailed design to the PTT process, to enable the design to be certified faster. Although many of the design changes via PTT may be appropriate and reasonable, they are often not clearly documented, such that it is never clear what the as built design actually is. Any ad hoc design changes should follow a formal design change procedure, where they are scrutinised against key requirements such as safety, durability and maintainability. However, the nature of the PTT means that design changes made via a PTT process may not undergo the same level of scrutiny, such that there is a risk that the solution to be implemented does not reduce or eliminate risk so far as is reasonably practicable.

4 Improvements to the Observational Method in New South Wales Tunnel Construction

To enable the tunnelling industry to successfully continue to construct tunnels in Sydney in a cost effective manner that reduces or eliminates risk so far as is reasonably practicable, some modifications to the use of the Observational Method in Sydney tunnels as described above should be undertaken. Recommended improvements include:

1. **Certification of the solution to be implemented**. Any PTT decision needs to be certified by both the designer and contractor and ideally, by an additional independent organisation. Certification is intended to ensure that decisions made as part of any PTT process are fit for purpose and have been carefully considered by all parties providing the certification. This in turn enables all stakeholders, including the client and any insurers, to be confident that appropriate observations have been provided as the basis of any design solution to be implemented. Although standard design certification can take considerable time, a process is required to facilitate the contractor implementing the design solution and receiving retrospective certification. Certification of the solution to be implemented is a means to assist in the management of risks associated with inappropriate decision making and is expected to assist in the prevention of significant safety incidents.
2. **Developing as robust a geotechnical model as possible**. The geotechnical and hydrogeological models need to clearly identify any limitations in the model as well as geotechnical or hydrogeological risks that require construction observations to be undertaken as a means to control the impacts of the risk. This may include construction activities such as forward probing to provide advanced knowledge of the location and characteristics of adverse geological structures, or

a more detailed analysis of monitoring data and trends. This in turn allows time for designers and contractors to prepare to implement a predefined solution.

3. **Requiring that contingency or emergency response measures be predefined**. These predefined measures need to be incorporated into the design documentation and include actions and responses required to implement such a solution. This is a key requirement of the Observational Method defined by Peck [14] and a key requirement of the International Tunnelling Insurance Group Code of Practice [9].
4. **Developing a responsibility, accountability, consultation and informing (RACI) matrix**. The RACI Matrix needs to clearly identify each task in a tunnel's design, construction and operation workflow and assign accountabilities, responsibilities and who is consulted and informed at each stage in the workflow.
5. **Ensuring regular auditing, review and design validation of any Observational Method process**. This is to ensure the designs are appropriate, not overly conservative and economic to construct and that staff involved in PTT teams and decision making are competent. It also enables an opportunity to monitor the performance of the staff involved in the decision making and to ensure they are appropriately motivated and empowered.
6. **Apply a systems engineering approach to the implementation of the Observational Method.** Systems engineering approaches already underpin tunnel structure design processes whereby the design of tunnel structures considers the geological and hydrogeological setting, rock material and rock mass behaviour as well as the interaction between the ground and the structure. Consistent with the Observational Method and systems engineering, the design and construction of tunnel structures typically allow for design validation by monitoring how the structure responds to the ground. Systems engineering approaches can be further embedded into the Observational Method by refining the application of the Observational Method by incorporating the above improvements into the design and construction requirements of tunnels.

The above improvements are not considered to be onerous requirements and have already been implemented in some capacity in the more recent Sydney road tunnel projects.

5 Conclusions

This paper has identified the state of the practice in the application of the Observational Method in Sydney tunnel construction. As with many approaches to construction, the method has evolved with time. Aspects of the evolution are positive and beneficial to the industry but other aspects are not, and require resetting to align with the original principles of the Observational Method. This paper has presented improvements that can be made that are expected to result in a more reliable decision making process involved in the application of the Observational Method, that in turn are expected to reduce risk associated with safety, durability and achieving the design life such that nature does not make fools of us.

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Precision Tunnelling Under Heritage Building in Sydney CBD

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Abstract The design of a pedestrian tunnel has been completed as a part of the integrated station design of Metro Martin Place (MMP). This tunnel connects two deep station entrance shafts (up to 28 m deep) and is located immediately below a hundred-year-old heritage building in Sydney CBD area. The heritage building is ornately finished and therefore extremely sensitive to ground movement. During detailed design, it was identified that Mass Concrete Backfill (MCB) supporting the building foundations was found to extend within the tunnel profile. Numerical modelling was carried out to assess the design of tunnel support for ground and building loads. It has also been used to estimate the surrounding ground deformation and foundation settlement of the building. The tunnel is mainly formed within Hawkesbury Sandstone impacted by the Martin Place Joint Swarm. The ground model including overall stratigraphy, in-situ stress condition, and rock/joint parameters was developed according to available borehole information, surrounding tunnel and excavation mapping, and past project experience. 3DEC software package was utilised to develop a local model and a global model simulating the interaction between rock and joints due to the tunnelling using determinate Discrete Fracture Network (DFN). The local model assessed sensitivity of MCB in terms of settlement due to different rock-MCB interface parameters. The global model captured the overall ground deformation and considered the effects of staged construction for the entire project site. The numerical results formed the basis of final tunnel support design, the impact assessment of the heritage building and monitoring strategies to minimise the impact on the building above and provide safe design.

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1 Introduction

The Sydney Metro City and South West project comprises of new 15.5 km twin rail tunnels from Chatswood continuing under Sydney Harbour and through Sydney's CBD to Sydenham. The John Holland, CPB Contractors and Ghella Joint Venture was awarded the contract to design and construct the tunnel and station excavation works (TSEJV). The project includes twin TBM bored running tunnels, three open cut stations, three underground stations, 57 cross passages, dive structures and a crossover cavern.

Macquarie Group submitted an unsolicited proposal to deliver an Integrated Station Development (ISD) for Metro Martin Place (MMP) station development, which included significant changes to the original reference design scheme. The ISD scheme included a significantly larger mined tunnel complex, larger entrance shafts, a pedestrian tunnel, and two Over Station Developments (OSD) as shown in Fig. 1.

The TSEJV and their designers are responsible for the excavation and permanent support of mined caverns and tunnels for station and running tunnels, the excavation and temporary support of the southern entrance shaft include piled foundations for the southern OSD tower. Macquarie Group are responsible for the excavation and temporary support of the northern entrance shaft, construction of the station, northern and southern OSD towers. They are also responsible for the excavation and permanent support of the pedestrian tunnel (see Fig. 2) between the northern and southern

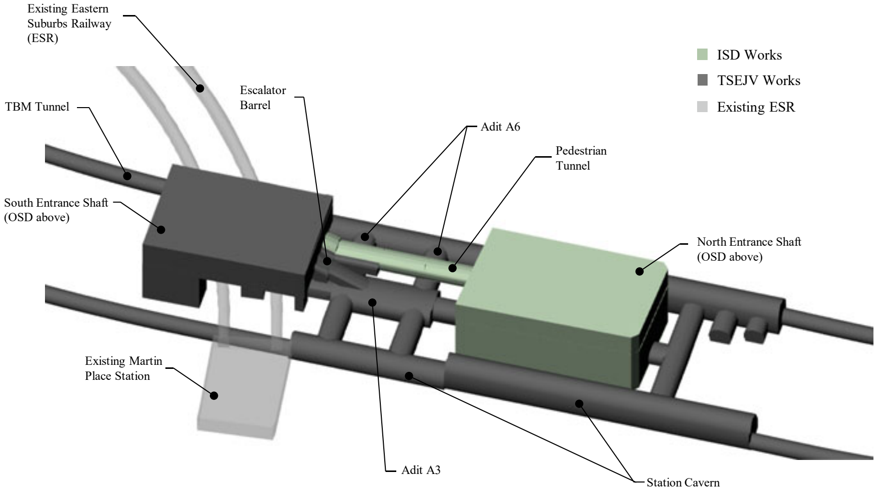

Fig. 1 Project location plan

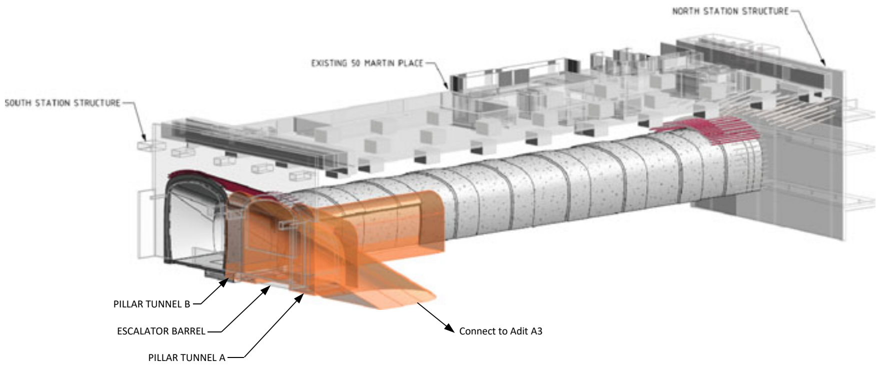

Fig. 2 3D CAD model of pedestrian tunnel between the northern and southern entrance shafts below the heritage listed 50 Martin Place building

entrance shafts below the heritage listed 50 Martin Place building. Arup have been retained by Macquarie Group and Lendlease for full multidisciplinary engineering design across the ISD.

1.1 Pedestrian Tunnel Overview

The pedestrian tunnel serves as an unpaid patronage access between the north and south station entrances and allows access to the retail areas in both shafts. The project included excavation beneath and underpinning of the heritage listed 50 Martin Place. Excavation was also carried out above and adjacent to the recently completed station caverns and adits as shown in Fig. 1. The tunnel excavation included a 5 m long inclined drive and a 61 m long horizontal drive with a final break through into the newly constructed permanent structure for the station northern entrance, as shown in Fig. 3. The ground cover to excavated tunnel span ratio varied along the alignment from 0.2 to 0.5. The tunnel alignment was primarily dictated by the station floor levels, location of the station concourse caverns and adits below the tunnel invert and the heritage building above the tunnel crown.

The completed tunnel will provide pedestrians with a clear span of 6.5 m and a height of 4 m. The finished floor level is RL6.5mAHD which is 11 m above the station platform finished floor level.

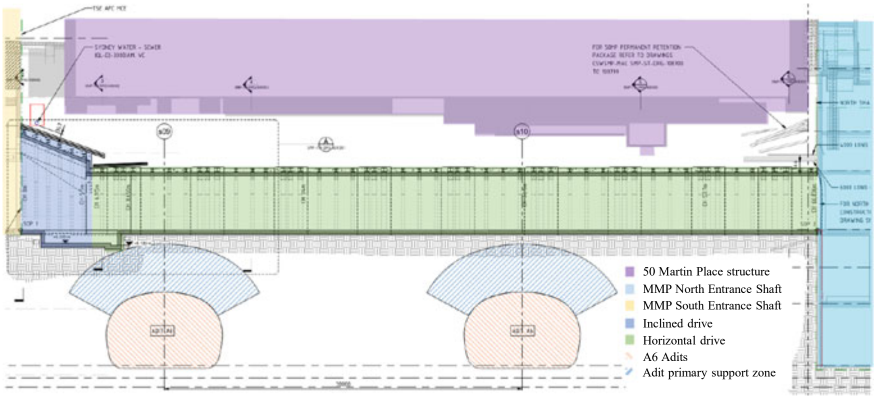

Fig. 3 Section view of pedestrian tunnel (along tunnel centre line)

2 Site Constraints

2.1 50 Martin Place

The pedestrian tunnel is situated immediately below 50 Martin Place (50 MP), which was originally constructed between 1925 and 1928 and is of exceptional heritage significance. The structure is predominantly reinforced concrete with concrete beams and columns on a 7 m grid with a 200 mm thick two-way spanning slab. Concrete encased steel was utilised in the columns through the Banking Chamber to achieve the required slender proportions. The exterior has terracotta and granite façades, and grand terracotta clad Ionic columns. The interior includes substantial use of marble and scagliola.

50MP is a 12-storey building with a lower ground and a sub-basement, with an entire column gridline located along the tunnel centre line. The original 1923 drawings indicated that the original structure to be supported on pad footings on average 2 m square by 1.2 m deep. The building had no significant modification prior to 1983. Between 1984 and 1990 the Commonwealth Bank commissioned extensive conservation work. This work included construction of a localised sub-basement for services and addition of a 7-vehicle basement carpark within the building.

The most recent refurbishments of the building were commissioned by Macquarie Bank Ltd. and were completed in 2014. The works included significant modification of the structure including:

- Existing roof top removed and replaced with new glass roof;
- New composite floor at Level 12;
- New post-tensioned concrete floor at Level 11;
- New major plant at Level 1, 2, 9 and roof top;
- Atrium enlargement and new stairs within atrium;
- New north-western lift core and modification to eastern lift core; and

- New lift pits and underground tanks.

In its current condition, maximum column working loads for the gridline above the tunnel centre line is 10.5 MN. The building deformation due to tunnelling was required to be limited to 10 mm with a maximum differential between two points of 1.3 mm/m. Tunnelling induced vibrations were limited to a maximum PPV of 7.5 mm/s for the heritage structure as a whole, while a more onerous 3 mm/s was also required for sensitive finishes within the Banking Chamber and other areas of the lower ground floor. Due to the heritage status and sensitivity of finishes of 50MP, investigations within the building were limited.

2.2 *Metro Martin Place Caverns*

The southern portal of the pedestrian tunnel is immediately adjacent to an escalator barrel, as shown in Fig. 2, which declines into a 20 m span flat arch cavern, Adit A3 orientated in same alignment of the pedestrian tunnel. Intersecting with adit A3 are four additional adits, Adit A6 orientated transversely to the pedestrian tunnel alignment. Two A6 adits are located immediately below the pedestrian tunnel invert (see Fig. 3) and a minimum clear distance between the extrados of both the A3 and A6 adits was 4.5 m.

Due to geometry constraints, the clear space between the escalator barrel and the pedestrian tunnel was approximately 1.6 m. Due to the reduced geometry of the rock pillar between both tunnel excavations and potential pillar instability, a pillar tunnel (Pillar Tunnel B) was excavated and mass concrete backfilled for the initial 24 m of the pedestrian tunnel prior to the tunnelling of both the escalator barrel and pedestrian tunnel.

2.3 *Sydney Water Sewer*

The southern tunnel portal was formed immediately below a 300 mm diameter vitrified clay sewer pipe as indicated in Fig. 3. The clear distance between portal canopy tubes was approximately 350 mm. From site investigations, the sewer was confirmed to be located within a trench excavated in sandstone then backfilled with mass concrete. A maximum tensile strain of 50 micro strain was imposed as the deformation acceptance criteria by way of Sydney Water ‘Building Over Asset’ process.

3 Geology and Ground Characterisation

MMP Station geological setting is the typical condition anticipated within the Sydney CBD, being mainly medium to high strength cross-bedded and massive subunits of Hawkesbury Sandstone with occasional interbedded siltstone units. Superficial Fill deposits (of variable composition and up to 3 m in thickness) overlie the sandstone.

The design ground conditions for the pedestrian tunnel were informed by investigation completed for the wider metro project, historic investigations within 50MP and surrounding properties, and supplemented significantly from historic mapping records of the original 1950s excavated Eastern Suburbs Rail tunnels, mapping records of the MMP tunnels, north and south entrance shaft excavations. Both the north and south entrance shafts primarily consist of Class I/II Sandstone (as categorised by [4]), with local reductions in class (to Class IV or Class III) due to the presence of the Martin Place Joint Swarm (MPJS). The MPJS typically consists of sub-vertical (70°–90°) NNE-SSW striking joints of regional scale persistence described as rough, undulating/curved, clean and occasionally clay infilled. A ‘swarm’ is represented as a very closely to closely spaced set (60-600 mm) of individual planes forming clusters at 5–10 m lateral spacings. Increase iron staining generally coincides with the localised ‘swarms’ with negligible influence on joint plane strength.

During both tunnelling of the main tunnel caverns and the north shaft excavation, several substantially persistent sub-horizontal shear seams were observed and generally comprised of clay and/or brecciated infill (ranging between 10 to 1000 mm in thickness). The geological structure sets were identified as listed in Table 1.

3.1 Identified conditions

During excavation of the northern entrance shaft located immediately adjacent to the northern elevation of 50 MP, (MCB) was encountered and coincided with the 50 MP footings. Test pit investigations, as indicated in Fig. 4, along the northern elevation of 50 MP confirmed that the MCB termination depth varied, with the MCB

Table 1 Summary of identified geological structures

Joint set	Dip angle	Dip direction	Comments
Bedding partings	0°–20°	Variable	Dip direction typically towards NE or SW
Cross bedding	0°–35°	Variable	–
Clay seams	0°–15°	300°–055°	–
Joint (Set 1)	75°–90°	010°–040°	The MPJS primary joint set
Joint (Set 2)	75°–90°	090°–120°	–
Geological structures (Sheared seams and faults)	5°–20°	290°–360°	–

located at the tunnel centre line extending below the termination level of the test pit at RL11.4mAHD.

Horizontal boreholes as presented in Fig. 4 were carried out at RL13.8mAHD (approximately 14.5 m long) and then at RL12.0mAHD during north portal canopy tube installation (approximately 20 m long). These investigations confirmed the presence of MCB with in the tunnel profile at 50 MP grid locations C7–C10. Downhole CCTV footage was obtained for both investigations and indicated that the MCB was generally intact and formed a reasonable and positive bonded interface with the surrounding rock mass, examples of the interface are shown in Fig. 5.

Based on the investigations and available site information MCB termination level for grid locations C1–C4 was assumed to be RL12.9mAHD, while grid locations C5–C10 were RL11.8mAHD, in other words penetrating the tunnel profile. Further,

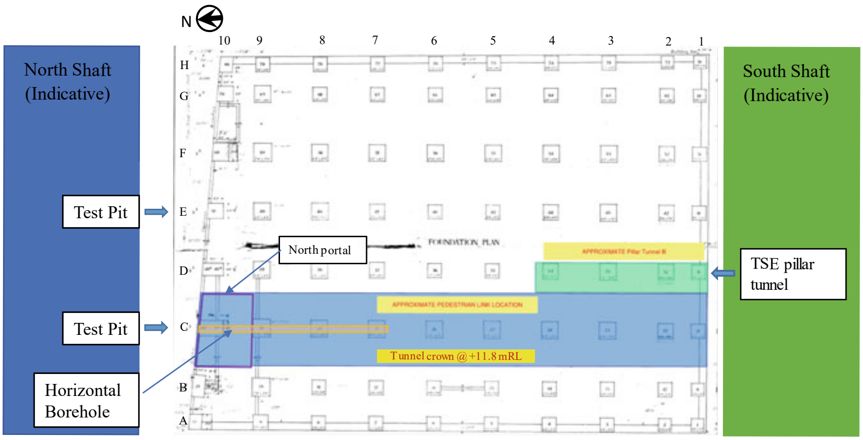

Fig. 4 50 MP footing plan and approximate locations of test pits, pillar tunnel and proposed pedestrian tunnel (modified after H.E. Ross and Rowe—Architectural Drawings, 1923)

Fig. 5 Downhole CCTV images indicating MCB-rock positive interface **a** MCB C1, **b** MCB C2

from all the investigations, reinforcement was not encountered in any of the MCB's indicating that the MCB did not form part of the structural portion of the pad footing.

It is noted that prior to commencement of construction, all available information had indicated that 50MP foundations were originally founded at RL16.5mAHD. This had been assumed at the initially feasibility stage and adopted right through to full detailed design stage. Identifying the presence of the MCB's triggered a full re-design of the tunnel support five months from the programmed start date.

3.2 Groundwater

Limited groundwater inflow was anticipated during excavations. The station entrance shafts are designed as drained in the permanent condition, therefore as the pedestrian tunnel extends between them both the temporary and permanent support are designed as drained structures.

3.3 Geotechnical Design Parameters

Rock mass parameters listed in Table 2 were determined using site specific in-situ test or laboratory tests. Where insufficient site-specific data exists to properly characterise a material parameter, reference was made to available published data including [1–3].

The magnitude of horizontal stresses over depth is summarized in Table 3. For Class V and IV sandstone, the correlation between major horizontal stress (σ_H) and minor horizontal stress (σ_h) is $\sigma_H = \sigma_h$; whereas for Class III or better sandstone, the corresponding correlation is $\sigma_H = 1.6\sigma_h$. Orientation of major horizontal stress has been adopted at 020° True North.

4 Considerations in Design and Construction Due to MCB

In addition to the constraints outlined in Sect. 2, the presence of the MCB required specific consideration for the design of both temporary and permanent tunnel support. Primarily, assessment of whether the building loads would be directly transmitted to the tunnel lining or be distributed through the rock mass needed to be determined. This assessment was heavily dependent of the MCB to rock interface, quality of the MCB itself and the influence of the chosen support type and construction sequence. Due to time constraints and site accessibility, investigations to confirm the state of this interface were limited to inspection of CCTV footage of horizontal bores only. Therefore, a series of sensitivity analyses using a local 3D model (Fig. 6) was setup to assess the performance of this interface, including the following scenarios:

Table 2 Adopted material properties

Material			Fill/residual Soil	Sandstone Class IV	Sandstone Class III	Sandstone Class I/II	Disturbed Zone
Substance	Uniaxial compressive strength, UCS (MPa)		–	10	15	25–30	10
	Young's modulus (MPa)		–	2000	3000	5000	2000
	Unit weight, γ (kN/m^3)		19	24			
	Poisson's Ratio, v		0.3	0.25		0.2	0.25
Tunnel scale	Mass Young's Modulus E (MPa)		40	500	1200	2000	–
	Mohr–Coulomb	c'(kPa)	0	120	250	500	–
		ϕ'(°)	34	43	53	58	–
		Tensile strength (kPa)	0	10	30	125	–
Block scale	Mass Young's Modulus E (MPa)		–	1500	2500	–	750
	Mohr–Coulomb	c'(kPa)	–	310	620	–	310
		ϕ'(°)	–	51	56	–	51
		Tensile strength (kPa)	–	250	500	–	250
Discontinuity Parameters			Bedding—Firm Clay Infill		Sub-Vertical Joints—Clean		
Infill Young's Modulus E (MPa)			20		N/A		
Mohr–Coulomb		c'(kPa)	10		0		
		ϕ'(°)	30		40		
Stiffness		Normal (GPa/m)	6		10		
		Shear (GPa/m)	0.6		1		

Table 3 Design in-situ stress with depth

Depth range	
0–10 m (with Class V and IV sandstone)	$\sigma_H = 0.5\sigma_v$
0–10 m (with Class III or better sandstone)	$\sigma_H = 2.5\sigma_v$
10–20 m	$\sigma_H = 2.5\sigma_v$
20 m and beyond	$\sigma_H = 1.5 + 2\sigma_v$
Where σ_v is the vertical stress in MPa	

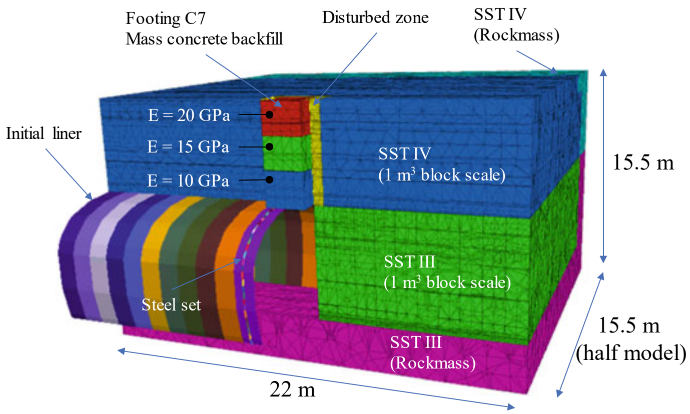

Fig. 6 Setout of local 3D model

- No tunnel support provided with MCB completely undermined and interface strength reduced.
- No tunnel support provided with MCB completely undermined and interface strength reduced and inclusion of a 0.5 m wide disturbance zone around the MCB.
- Staged construction with application of 250 mm thick steel fibre reinforcement with age dependent stiffness.
- Staged construction with application of 250 mm thick steel fibre reinforcement with age dependent stiffness and installation of steel sets.

A three-dimensional commercial numerical software package, 3DEC V5.2 was adopted to carry out the assessment. The adopted software package can capture the discontinuum behaviour of the rock surrounding the tunnel.

The results indicated, as expected, that reduction in interface strength increased deflections. At approximately 60% of the interface strength (equivalent to the shear strength of surrounding rock), the shear strength had been exceeded resulting in a sliding failure of the MCB due to the considered building load. Introduction of the disturbed zone resulting in a sliding failure at 67% interface strength. However, on introduction of support (in the form of shotcrete liner and steel sets) MCB sliding failure was not achieved. The adopted support sequence allowed for half of the MCB to be undermined and supported, then the second half undermined and supported. The analysis indicated that 50–60% of settlement occurred during the initial advance undermining the MCB, and 100% occurring by the second advance beyond the MCB provided that the full support has been installed. Each modelled advance was approximately 1 m.

Based on the above results an appropriate support method would be required to limit deflections on initial undermining of the MCB and throughout the remaining excavation of the tunnel. Consideration was given to pre-support in the form of canopy tubes however, the number and length required to satisfy the load conditions and support the MCB would result in increased excavation and space proofing issues with the existing structure. Full length canopy tubes, as an array of large diameter canopies (or horizontal piles) in the order of 300–600 mm were also considered. However due to the premature refusal of the investigation probe hole and other potential obstructions below 50 MP the risk of potential refusal of a horizontal pile somewhere along the length of the tunnel was considered too high. Further, due to the difference in construction programs between the north shaft and the south shaft, it was considered the northern entrance shaft program would be significantly advanced precluding adequate measures to be provided at the north shaft. Temporary then permanent steel sets were considered to be the best option as the sets would be installed as the MCB is undermined and designed for the permanent condition such that there are minimal movements beyond the initial undermining of the MCB further.

5 Numerical Modelling

Due to the complexity of underground structures and geological features and to maintain reasonable consistency between different sets of models, a 3DEC model was developed to carried out the assessment.

A simplified geological model with respect to a 'layer cake' stratigraphy has been used for the numerical model developed. Figure 7 shows the geological model adopted in the global 3D model which reflects the ground conditions as discussed in Sect. 3. A 0.5 m disturbed zone around each MCB was introduced to simulate the reduction of surrounding rock quality due to the construction of MCB as shown in Fig. 7. Since the behaviour of discontinuum modelling is strongly dependant on the orientation of defects, a realistic fracture network is one of the most essential inputs for modelling. A discrete facture network (DFN) was created using an Arup developed workflow [5] based on available borehole logs, face mapping and stereonets. A DFN model was imported into 3DEC to carry out the analysis.

As mentioned in Sect. 4, the majority of the deflection of the MCBs occurred immediately after initial undermining of the MCBs. Therefore, the adopted design strategy assumed that in the temporary case only 50 MP SLS column loads would be considered and fully supported by steel sets installed during the tunnel excavation (i.e., column load supported by shaft adhesion and steel sets). In the permanent case

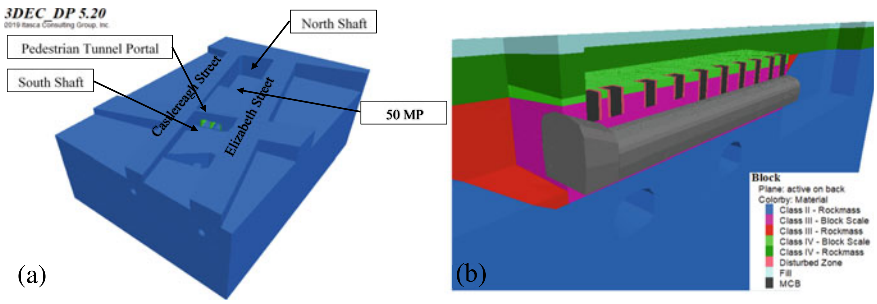

Fig. 7 **a** Full 3DEC model, **b** Geotechnical model local to pedestrian tunnel

50 MP ULS column loads were considered and supported by the combination of steel sets and permanent liner. It should be noted that pre-support at the south and north portals and at the interface between the portals and the main tunnel were not modelled along with any 'as-required' rock bolts.

Three different modelling approaches were conducted as follows.

I. *Unsupported 'wished-in-place'(WIP) excavation*

Following obtaining the relevant initial stress state for the pedestrian tunnel, the entire pedestrian tunnel was excavated in a single stage and the model was solved until it has reached equilibrium. Results of unsupported analysed provide an indication of unstable blocks and an upper bound of anticipated displacements.

II. *Supported 'wished-in-place' excavation*

The WIP excavation with the support of steel sets was used to obtain the axial forces, shear forces and bending moment of steel sets which was used as the basis for steel set structural design as the WIP modelling method generally provided the upper bound of stresses could be developed in the support structural members under 50 MP SLS load case. WIP model was considered to be conservative as it excluded the immediate rock relaxation and the corresponding stress redistribution in the surrounding rock, and therefore the ground load was directly transmitted to the steel sets and they were designed to resist this action. Considering the construction method of steel sets which would be encapsulated by grout with a minimum strength of 10MPa before next excavation, rigid connection between steel sets and surrounding rock was assumed.

III. *Supported staged-construction*

In the model analysing each individual construction stage, the excavation was advanced in maximum 1.25 m advance length stages. The steel set was applied one excavation stage behind. The results of this analysis case should provide an indication of anticipated displacement of the tunnel crown and settlement of 50 MP footings.

For model cases with tunnel supports, two types of steel sets, 250UC89.5 or 310UC118 were considered.

5.1 Model Results

The vertical displacement at tunnel crown without and with the support of steel sets under 50 MP SLS building load case is shown in Fig. 8a. As expected, the unsupported model resulted in the most vertical displacement at approximately 9 mm. Also, the convergence of the unsupported model implied that no unstable blocks were encountered. The staged construction model showed more vertical displacement than the WIP model, as the steel set support was modelled one advance behind. The maximum vertical displacement at the tunnel crown was predicted to be approximately 6 mm with the support of 250UC89.5 or 310UC118 steel sets. The vertical displacements at the footing level (+16.5mRL) along the pedestrian tunnel are presented in Fig. 8b. The unsupported model experienced the most vertical displacement approaching 10 mm. In supported modelling cases, the staged construction model returned more vertical displacement than the WIP model with the maximum vertical displacement of approximately 6 mm.

6 Tunnel Support

The results obtained from 3DEC analysis formed the basis of temporary and permanent tunnel support structural check for shotcrete lining, steel set and permanent cast-in-situ reinforced concrete lining following the workflow as shown in Fig. 9.

During finalisation of the design, as a risk mitigation an additional worst credible case was considered. Due to the reliance of the tunnel support on the interface strength between the rock and the MCB, a 'survivability case' was assessed in the unlikely (but plausible) event that shaft friction could not be relied upon and the tunnel support

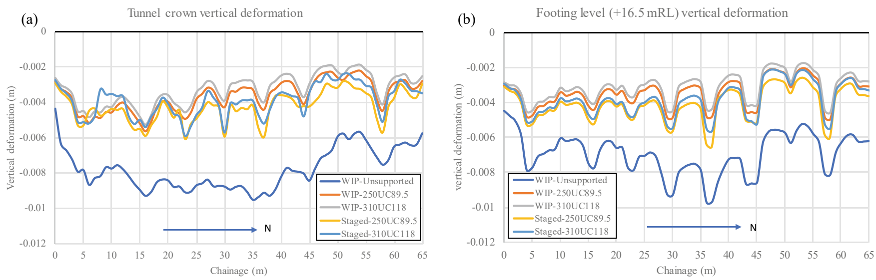

Fig. 8 Vertical displacement **a** at tunnel crown under 50 MP SLS load case and **b** along pedestrian tunnel at 50MP footing level

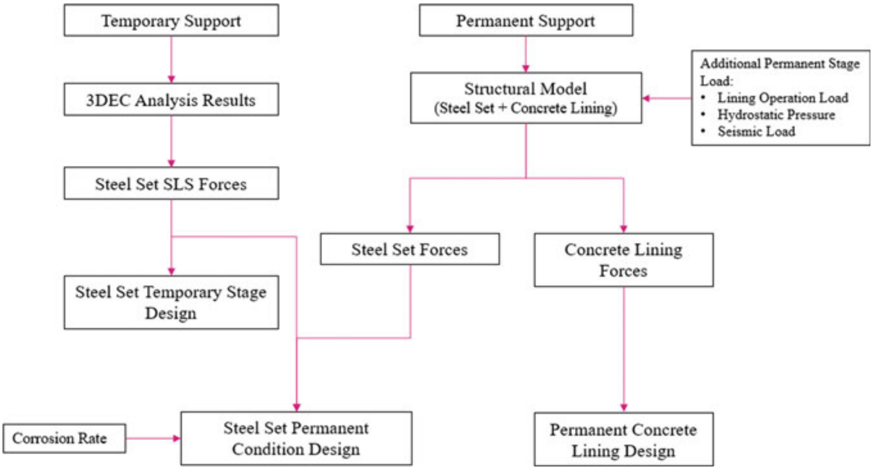

Fig. 9 Work flowchart of structural design check

was required to carry the full building column load. This resulted in three 400WC steel sets being required to support each MCB in the permanent condition. These sets required full welded fabrication including rolling of the crown and legs.

Therefore, the primary support for the pedestrian tunnel comprised of an initial 50 mm thick layer of steel fibre reinforced shotcrete (SFRS) applied to exposed excavation surfaces. Steel sets, 250UC, were provided at 1.4 m centres between MCB locations and encased in a minimum of 250 mm thick SFRS. At MCB locations three 400 WC steel sets were provided to support the MCB in both the temporary and permanent case, as shown in Fig. 10. The 400WC were fully encapsulated in SFRC and high strength grout.

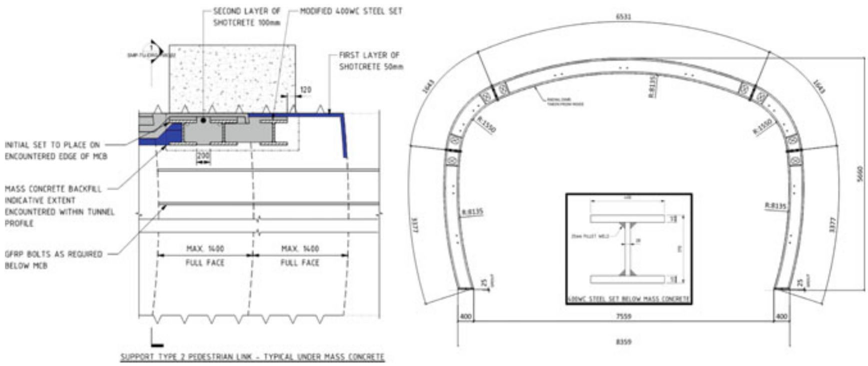

Fig. 10 Tunnel support under MCB

The permanent support system consisted of three 400WC at MCB locations and a 250 mm thick reinforced cast-in-situ concrete liner to form the internal profile. Sheet waterproofing membrane was provided between the shotcrete and concrete liner.

7 Construction

7.1 Permit to Tunnel

During construction a Permit to Tunnel (PTT) process was utilised to monitor and assist the excavation and construction works. This process was generally carried out daily involving a meeting between the principal contractor, tunnelling contractor, and tunnel designer. Typically, during a meeting the most current monitoring data and geological mapping of the excavated face, was reviewed and discussed and should there be any issues, they were typically resolved during the meeting or taken on notice by the tunnel designer to resolve as soon as possible. It was considered that this process provided sufficient flexibility in order to adequately deal with the conditions exposed at the current stage of excavations. Tunnelling could not progress till site personnel received a signed copy of the PTT form, signed by the three attendees of the PTT meeting.

7.2 Sequence

Both the northern and southern portals were provided with a pre-support pipe umbrella consisting of two rows of canopy tubes. Additional support, in the way of a full array of rock bolts around the portal was provided to reinforce the rock mass around the northern portal to avoid overbreak and potential loading onto the permanent north entrance shaft structure during breakthrough.

The southern portal inclined drive was formed using a Liebherr 900 fitted with a rock grinder. The advances over this portion of the tunnel were limited to 1 m as the full face was excavated in one operation and to limit movements. This length of tunnel was particularly sensitive to movements because of the presence of the Sydney Water sewer and the perimeter foundation of 50 MP supporting the grand façade.

The horizontal drive of the tunnel was formed using a Mitsui S200 road header. The advances over this portion of tunnel were limited maximum of 1.4 m advances. This was adopted so that the initial advance below the MCB would not extend beyond the MCB centre line, while also providing three full advances between the MCBs. Two 400WC steel sets were provided in the initial advance under MCBs. Positive connection of the steel set and MCB was achieved using a high strength grout. The following advance commenced once the grout had achieved 10 MPa and would fully

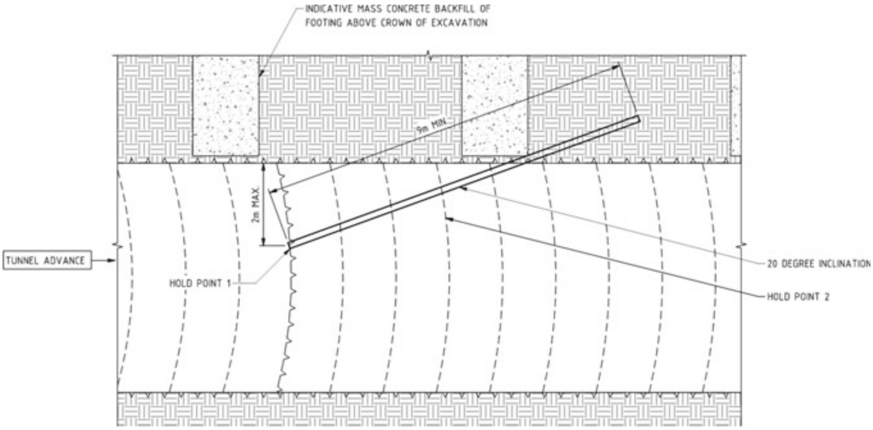

Fig. 11 Probe hole to confirm rock and concrete quality

undermine the MCB. The third and final 400WC was then installed and grouted in place. Prior to the advance after a MCB, probe holes were drilled to identify the presence of the next MCB and the quality of the rock-concrete interface, the rockmass, and the MCB itself as presented in Fig. 11.

7.3 Encountered MCB's

Out of ten MCB location eight were encountered with the tunnel profile. Only MCB at grid locations C2 and C3 were not exposed. Termination depths of the exposed MCBs were variable and founded within varying quality of sandstone. The quality and workmanship of the MCB was observed to be of a very high standard considering the period it was constructed (circa 1925). Concrete segregation was not identified, and positive rock-concrete connection was observed for all exposed MCBs. An example of exposed MCB during tunnelling is shown in Fig. 12.

7.4 Observations

Survey monitoring was carried out daily, both within the tunnel and the 50 MP structure. The tunnel monitoring comprised of convergence arrays typically at 5 m intervals along the tunnel alignment, installed in the previously supported advance following initial advance under an MCB. Monitored deformation of the tunnel crown is provided in Fig. 13 and shows generally similar or better performance compared to predicted deformation. The monitoring of the 50MP façade and internal columns also performed generally similarly or better than the prediction. Only a single occurrence

Fig. 12 Initial advance below C10 showing MCB extending below tunnel invert

of minor cosmetic damage to slab on grade tiling was observed during tunnelling and no structural damage was observed in 50MP.

8 Conclusions

This paper presents the challenges of excavating a pedestrian tunnel within the heart of the Sydney CBD and immediately below a building of exceptional heritage significance 50 Martin Place (50 MP) with a ground cover to tunnel span ratio between 0.2 and 0.5.

Mass Concrete Backfill (MCB) was identified to be present below the footings of 50 MP, which dictated a non-traditional support strategy leading to large steel sets forming both primary and permanent support to the MCB.

Numerical models were developed to assess overall ground movements and the potential failure mechanism of the rock-concrete interface at the MCBs, forming the basis of tunnel support design. The numerical results indicated that the main failure mechanism was the shear stress due to MCB displacement exceeding the shear strength of the MCB-rock interface.

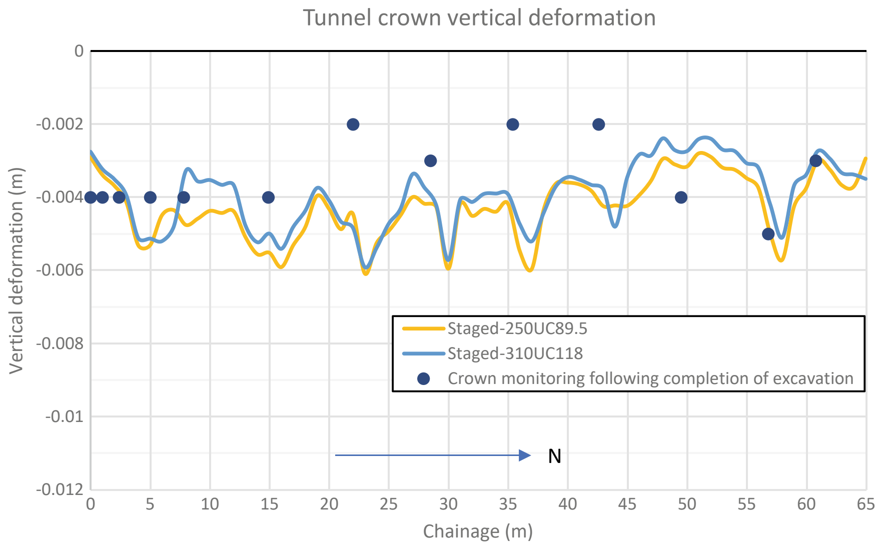

Fig. 13 Total crown vertical deformation

The use of large steel sets resulted in deformations within tunnel and 50 MP in good agreement with the predicted movements, resulting in overall negligible damage to 50MP and a successful project delivery with significant risk and site constraints.

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Design of Complex Permanent Tunnel Linings at Sydney Metro's Victoria Cross Station

Yun Bai and Strath J. Clarke

Abstract The permanent tunnel linings of Victoria Cross Station include complex intersections associated with their asymmetric geometry, significant groundwater and rock loads, and the large span of the station cavern. Victoria Cross is one of six underground railway stations recently completed by the John Holland CPB Ghella JV (JHCPBG JV) as part of the Sydney Metro City and Southwest project. Sydney Metro is Australia's biggest public transport project, which will deliver 31 metro stations and more than 66 km of new metro rail line. It runs from Sydney's northwest region, beneath Sydney Harbour, through new CBD stations and then southwest to Bankstown. Victoria Cross Station comprises a 265 m long cavern with several pedestrian and service adits connecting the cavern to two adjacent shafts. Located beneath Miller Street in North Sydney, it includes the largest cavern on the project with a clear span of 24 m and internal height of almost 16 m. The design of the permanent lining at the intersection between the lift access adit, lift shaft and the cavern was particularly challenging due to the complex geometry. The lift shaft intersects the crown of the cavern and connects to the access adit located above the cavern, with these two structures separated by only a 2 m thick sandstone slab. A range of numerical modelling techniques were developed to address the various design considerations which applied to this complex intersection. The design requirement for the station permanent lining to be tanked led to the lining having to accommodate significant groundwater loads in addition to the large rock loads associated with a cavern of this span. Three-dimensional finite element modelling was undertaken to assess the interaction between the cavern and overlying lift adit structures as well as to inform the articulation and waterproofing details at different structural interfaces.

Keywords Tunnel linings · Cavern · Numerical modeling · Design consideration

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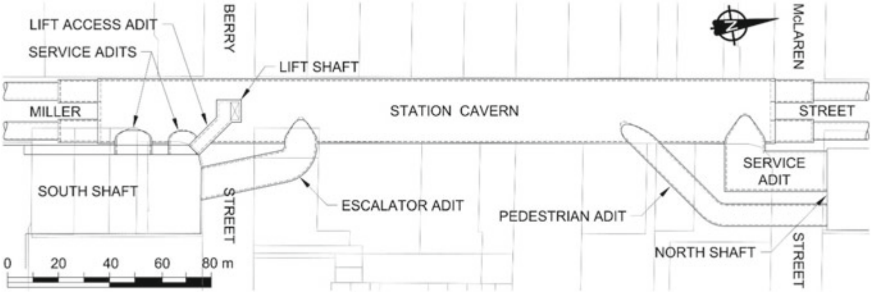


Fig. 1 Plan of Victoria cross station with the cavern aligned beneath Miller street in North Sydney, showing the location of the intersection between the lift access adit, lift shaft, and station cavern

1 Introduction

The permanent lining design of the Victoria Cross Station cavern intersection with the lift shaft and lift access adit proved particularly challenging due to the difficult geometry, with the overlying horizontal adit located within 2 m of the cavern crown.

Victoria Cross Station, located in North Sydney, is one of the five new stations constructed as part of the Sydney Metro City and Southwest (SMCSW) Tunnels and Station Excavation Works (TSE) constructed by the JHCPBG JV.

The TSE works included construction of three new underground stations, of which Victoria Cross Station has the largest cavern, with a length of 265 m, a clear span of 24 m, and an internal height of 16 m. The station cavern runs beneath Miller Street, and is connected to two adjacent shafts by three service adits, two pedestrian adits, and a lift shaft and access adit intersecting the cavern crown (Fig. 1).

The proximity of the lift access adit above the station cavern resulted in a range of potential load paths and interactions between these structures and the surrounding rock. The permanent linings also had to accommodate substantial groundwater loads imposed by the water pressures associated with fully tanked design requirement.

2 Ground Conditions

The ground surface is approximately 20 m above the cavern crown at the south end, with cover increasing to about 33 m at the north end. The shallow lift access adit has a ground cover of approximately 8 m, including 3 m of medium to high strength, predominantly massive, Class I/II Hawkesbury Sandstone above the crown, overlain by poorer quality rock with a thin superficial soil unit at surface (Fig. 2). The geotechnical design model adopted for design of the intersection is summarised in Table 1.

Seven vertical and inclined geotechnical investigation boreholes were drilled prior to the start of construction, supplemented by downhole imaging. Prior to the

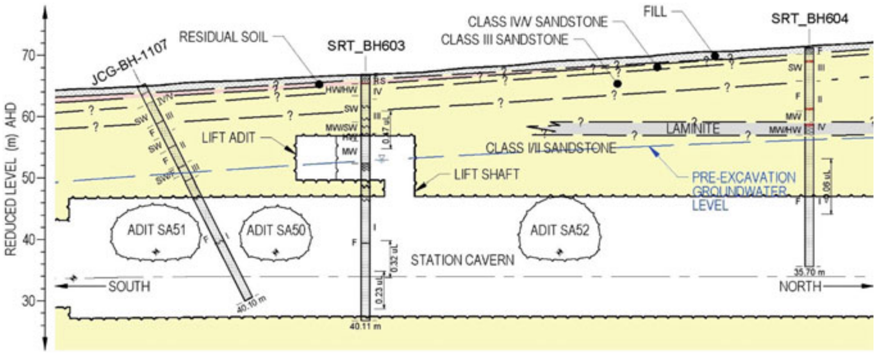


Fig. 2 Geotechnical long section through the southern end of the cavern, showing good quality Hawkesbury sandstone bedrock at the intersection between the cavern and lift shaft, with a persistent laminate layer present at the crown level of the lift adit

Table 1 Summary of ground conditions at Victoria cross station

Unit	Reduced level of top of unit	Thickness	Description
	(m) AHD	(m)	
Fill	63–83	0–1.6	Clay and silt with brick fragments. Variable
Residual soil	62–70	0–2	Sandy clay to clayey sand and sand
Hawkesbury sandstone bedrock	61–81	>50	Class IV and V sandstone (up to 3 m thick) overlying Class III sandstone (1.5–4 m thick), grading to Class I and II sandstone with depth A laminite layer (Class IV sandstone), typically 1–2 m thick, is present at approximately RL 57 m (Fig. 2)

Notes Rock classification as per [5]

commencement of tunnel excavation, geological mapping was undertaken of rock exposures in adjacent building basements. These investigations indicated the presence of orthogonal joint sets, with the dominant subvertical set striking roughly parallel to the cavern alignment. Sub-horizontal primary bedding planes had a typical spacing of one to three metres in Class I/II sandstone, reducing to one to two metres in Class III sandstone. These conditions are typical of the Hawkesbury Sandstone in inner Sydney.

Some low angle localised faulting was inferred from boreholes based on concentrated jointing, crushed seams, shear seams and core loss (indicated in the geotechnical long section in Fig. 2).

In-situ stress measurements undertaken at the site indicated that the stress regime was consistent with the expected range in Sydney, noting that stresses in Hawkesbury Sandstone are relatively high and result in a range of stress-relief behaviours during excavation.

Ground conditions encountered during tunnel construction were largely confirmatory of those inferred from the site investigations, except that the minor joint set was all but absent.

3 Groundwater

Groundwater levels were measured in four standpipe piezometers located in the vicinity of the station, which indicated standing water levels of between RL 44 m and RL 60 m prior to the commencement of excavation. This corresponds to between 2 and 12 m above the cavern crown level (Fig. 2).

The project required that the station cavern be designed as an undrained, or tanked, structure. This means the permanent structure will be subject to full groundwater loading once it recovers to pre-excavation levels, after being temporarily drained during construction. Additional groundwater loads to accommodate leaking or failed utility services were also catered for in the design.

A three dimensional (3D) numerical groundwater flow model was developed using the hydrogeological software package FEFLOW. The model parameters were calibrated against measured groundwater levels and historical rainfall data.

Typical post-construction groundwater conditions were calculated by including the Victoria Cross Station north and south shafts to the calibrated hydrogeological model, with both shafts modelled as drained structures. The sensitivity of the groundwater levels was investigated relative to a range of hydrogeological variables, including climate, anthropogenic factors (i.e. leaky services) and the potential occurrence of major geological structures (e.g. dykes or faults).

Groundwater loading scenarios considered for the design of the permanent lining included:

- Credible worst groundwater level (RL 50 m). This load case is associated with extreme climatic conditions, undetected long-term leakage of water mains, major geological structures, or a combination of these factors. The lift adit floor level is at RL 50.2 m and thus the design groundwater levels do not load the walls or crown of the lift adit, however these levels do impart groundwater pressures of up to 230 kPa to the cavern invert slab.
- Service groundwater level (RL 45 m). The service groundwater level corresponds to typical conditions expected over the design life of the station.

- Low groundwater level. Zero water pressure, corresponding to conditions immediately following lining construction.
- Recharge groundwater levels. The groundwater level gradually recovers once the tunnel waterproofing is installed, until the service groundwater level is reached.

It is noted that for the groundwater levels above the service level that the cavern effectively 'floats', as the upwards buoyant force exceeds the self-weight of the cavern lining.

4 Intersection Geometry

The lift adit and lift shaft are located to permit mobility-impaired station patrons direct access from the main entry building to the southern end of the station platform.

The lift adit leads from the southern shaft to the lift waiting area and is aligned 45° to the centreline of the cavern (Figs. 1 and 3). The lift adit and waiting area have a clear height of six metres. The excavated span of the lift adit is 5.5 m, with this dimension governed by clearance of the Mitsui S300 roadheader that the Contractor proposed to excavate the adit.

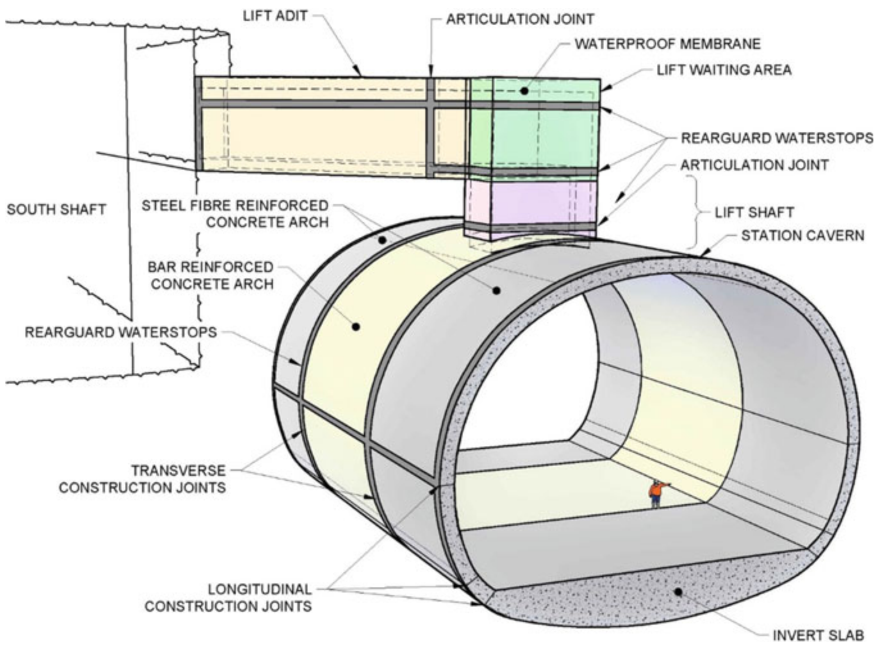


Fig. 3 Intersection geometry showing the relationship between the south shaft excavation, lift adit, lift waiting area, lift shaft and station cavern

The waiting area extends to the cavern crown below via a vertical lift shaft, which is almost 4 m deep. This geometry results in a thin rock beam of approximately two metres thickness between the excavation for the lift adit and the excavated opening for the cavern crown.

Two articulation joints are provided in the intersection to facilitate potential long-term ground movement. One joint is located between the cavern crown and the lift shaft; and the other is located between the lift adit and the lift waiting area (Fig. 3). These details were included in the design based on the results of the numerical analysis described in Sect. 7.

5 Design Criteria

The concept of limit states were applied to the design of the station adit and cavern permanent linings. The fundamental principle is to ensure that functional performance is maintained. This performance is achieved by a combination of quality control of materials and workmanship during construction (as defined in the relevant specifications), and design for appropriately adverse loading conditions, thereby imparting a suitable level of redundancy and robustness to the structure.

Ultimate Limit State (ULS) and Serviceability Limit State (SLS) load combinations were considered. The ULS addresses the structure's overall stability as well as the internal stability of the individual members. The internal structural reactions for reinforced concrete members were checked for adequacy using interaction diagrams and the capacity envelopes developed in accordance with AS5100 "Bridge design".

The SLS design of the permanent lining is largely governed by the control of structural deformation and crack widths, with appropriate detailing required to prevent groundwater ingress. The adopted deflection criteria required that the tunnel crown move no more than the span of the structure divided by 300, in accordance with AS1170 "Structural design actions". The maximum permissible vertical upwards deflection of the cavern and adit invert slabs was equivalent to the span of the structure divided by 500. The maximum allowable crack width for bar reinforced concrete was 0.3 mm, in accordance with the project specification.

Groundwater seepage through the lining was not permitted, meaning the complete absence of leakage, seepage, or damp patches of concrete.

6 Ground Loads

A range of load cases and load combinations were considered in the design of the permanent reinforced concrete linings of the cavern and intersection. These loads included self-weight, creep, temperature, shrinkage, contact grouting, rail loads (for the cavern), service loads, and seismic and fire loads [3].

The ground and groundwater loads are the most significant to the lining design and ground loads are briefly summarised below, as well as load cases specific to the geometry of the lift intersection. The assessment of ground load is often subjective and thus warrants the consideration of multiple approaches [1]. Geological structure checks, empirical methods such as Terzaghi's [7], Q system [2], and Cording et al. [4], and finite element analyses have been employed to assess appropriate design ground loads for the cavern and intersection.

6.1 Geological Structure Checks

The essence of this simple graphical method is that the proposed tunnel profile is placed at multiple random locations within a two dimensional (2D) geological design model. For the Hawkesbury Sandstone bedrock at Victoria Cross Station the geological model is dominated by rock defects comprising sub-horizontal bedding planes and sub-vertical joints striking north-northeast–south-southwest.

Potentially unstable blocks are defined as zones of rock which are intersected by the tunnel excavation and are separated from the surrounding rock mass by discrete defects. Their dimensions are dependent upon the defect orientation, spacing, persistence, and the excavation geometry. Blocks assessed as potentially unstable are identified and their size and extent assessed. The equivalent rock pressure is calculated as the weight of each block per metre run along the tunnel axis divided by the length of block exposed around the tunnel perimeter.

6.2 Empirical Methods

The Terzaghi classification [7] estimates rock loads as a function of tunnel dimensions and a qualitative description of rock mass quality. Application of this method indicated a load of 30 kPa for a 5 m span tunnel and 150 kPa for a 25 m span (Fig. 4).

The Q system [2] can be used to estimate the support pressure or rock load as a function of the quantified rock mass quality. The Q system implies that the rock load is independent of tunnel span.

Cording et al. [4] provided guidelines for estimating rock load when the dominant loading mechanism is gravity, i.e. loosening pressures. Cording suggested $p_i = nB\gamma$ where B is the span, γ is the unit weight of the rock mass and n is a constant. Little guidance is given as to the relationship between ground conditions and the parameter n, other than concluding that "*the value of n in the crown of large bolted caverns in fair to excellent quality rock typically ranges from 0.1 to 0.25*".

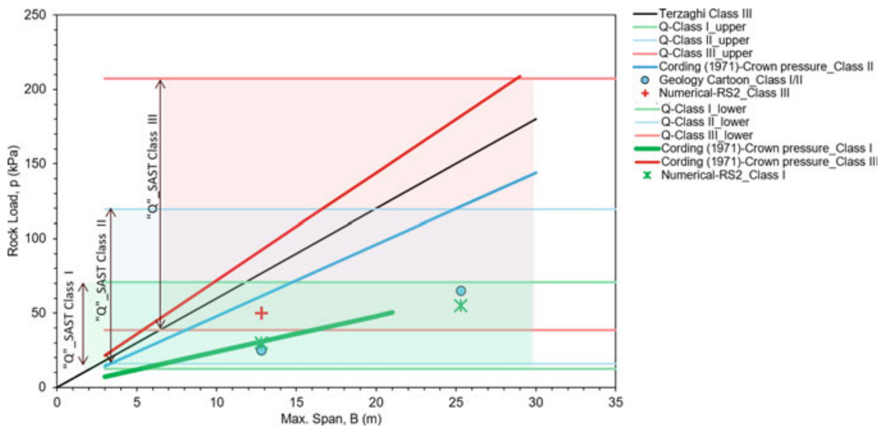


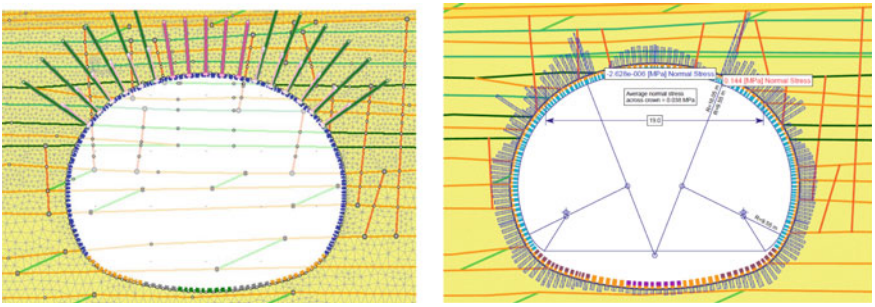
Fig. 4 Comparison of rock loads assessed from different methods

6.3 Numerical Analysis

Two-dimensional analyses were performed to assess ground loads using the finite element software package RS2, produced by Rocscience. The analyses included a detailed representation of geological conditions and employed the staged approach described below to simulate ground load transfer mechanisms between the temporary support and permanent lining:

1. Stress initialisation prior to the commencement of excavation.
2. Sequential excavation to simulate heading advance and ground relaxation, and installation of rock bolt support to simulate the proposed construction sequence and timing.
3. Installation of the waterproofing lining and the permanent lining.
4. Removal of rock bolt support from the model, to allow the rock mass to relax and deform and thus load the permanent lining.

The magnitude of ground load redistributed to the permanent lining depends on factors including tunnel geometry; ground conditions; in-situ stresses; rock mass strength and stiffness; as well as the stiffness of the waterproofing membrane and permanent lining. Some of these variables are difficult to reliably assess and were addressed by adopting conservative assumptions and testing a range of feasible values. Figure 5a presents an example of an RS2 finite element model of the cavern profile in ground typical of Class II sandstone. The resulting normal stresses distribution (i.e. ground load) can be extracted around the perimeter of the lining (Fig. 5b). The calculated normal stress ranges were up to 144 kPa, with an average normal stress across the central portion of the crown of 38 kPa. The normal stress profile in Fig. 5b exhibits localised peaks and troughs due to the modelled discrete rock defects intersecting the cavern excavation.



(a) Model of ground conditions and support (b) Normal stress distribution around lining

Fig. 5 RS2 model simulating staged construction of the cavern up the time that the temporary support reaches the end of its design life and the ground load is transferred to the permanent lining

The normal stress acting on the lining can also be calculated from the axial force, lining radius and lining thickness using the hoop stress approach. The distribution of the calculated axial force is often noticeably 'smoother' than the calculated normal stress distribution as the axial force is less susceptible to the effect of localised defects. The equivalent normal stress in the crown calculated using the hoop stress approach was 34 kPa for the example presented in Fig. 5.

6.4 Adopted Loads

Figure 4 compares the ground loads derived from the various methods discussed above. It can be observed that rock load increases with tunnel span and a reduction in rock quality, and that the empirical methods tend to predict larger rock loads compared to the graphical and numerical methods. For the cavern a design rock load of 55 kPa was adopted for design, with 21 kPa adopted for the smaller lift adit.

A horizontal rock load of 20 kPa was applied in the design of the vertical walls of the lift adit, waiting area and lift shaft.

Additional ULS load combinations were devised specifically for the intersection to explore potential adverse interaction(s) between the cavern, lift waiting area and lift adit (Fig. 6). By way of example, Fig. 6a shows a loading scenario intended to produce high compressive stresses between the crown of the cavern and the lift shaft. This scenario is associated with groundwater uplift pressures acting on the cavern and rock loads acting on the crown of the lift adit and lift waiting area.

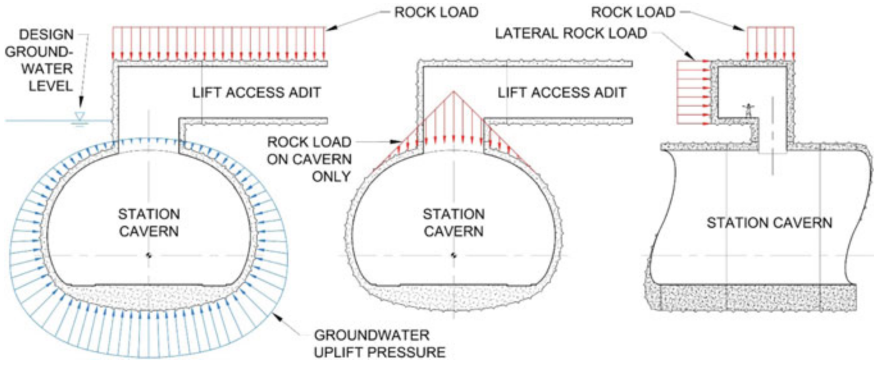


Fig. 6 Specific ULS load cases were developed for the lift intersection to investigate interactions between the lift shaft, lift adit, cavern, and variations in groundwater and ground loads

7 3D Numerical Analysis

Detailed three dimensional (3D) finite element analysis was required to simulate the structural behaviour of the permanent lining at the complex intersection geometry, in particular its interaction with the surrounding rock mass including the thin rock beam separating the lift access adit and station cavern.

Such analyses require inputs of geometry, material properties (e.g. strength and stiffness), and applied loads and load combinations. The results then require interpretation to assess the implications to the overall design, and address key issues such as the type of structural lining required (e.g. plain concrete, fibre-reinforced concrete, bar reinforced, or fibre *and* bar reinforced concrete); whether the analysed thickness is appropriate; and whether key serviceability requirements are met (e.g. deflection). Once these key issues are resolved, detailed design can proceed to determine reinforcement requirements, suitable locations for construction or articulation joints, and any other special details (e.g. inclusion of re-injectable grout hoses).

7.1 Material Properties

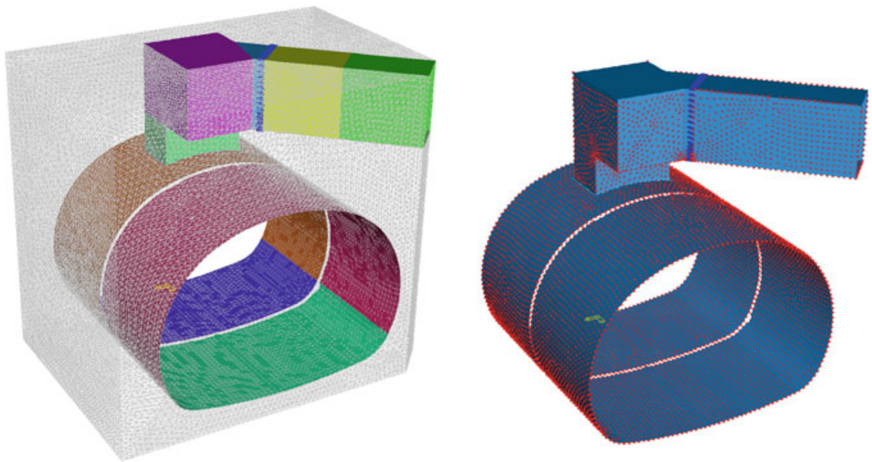
Material properties of the structure were chosen based on the limit state being assessed, with relatively low concrete moduli adopted when considering ULS behaviour and long-term deflection (SLS), 16 and 12 GPa, respectively. A higher concrete modulus of 32.8 GPa was adopted to assess crack widths under service conditions.

Waterproofing layers are installed around the outer surface of the permanent lining and include a 2 mm thick PVC waterproof membrane and a 5 mm thick protective geotextile 'fleece'. The normal stiffness and shear strength of these layers are included in the numerical model as they adversely impact the calculated structural actions, mainly due to the compressibility of the fleece. A normal stiffness of 300 MPa/m was adopted for the waterproofing layers.

7.2 Finite Element Modelling

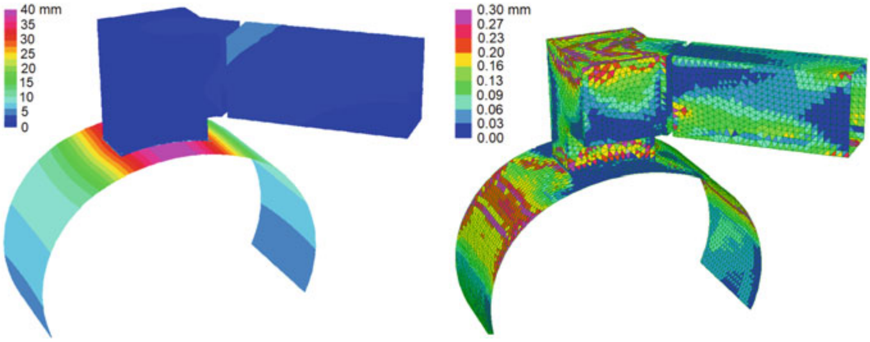
A 3D model of the intersection and surrounding ground was constructed using the finite element package Strand7 (Fig. 7). Plate elements were employed to represent the permanent concrete lining, with brick elements used to simulate the restraint provided by the rock mass surrounding the excavation. Short beam elements were required to model the interface between the rock mass and the concrete lining. Beam elements were also used to model the connection detail between the lift shaft and the crown of the cavern.

The ground and groundwater loads described previously were applied in the model in various combinations to ensure that the design solution was sufficiently robust and structurally adequate and satisfied the various design criteria (Figs. 8 and 9).



(a) Brick elements used to simulate rock mass　　(b) Permanent lining modelled with plate elements

Fig. 7 Strand7 finite element model used to investigate the interaction between the permanent lining of the lift shaft intersection and the rock mass which was present both between the lift adit and cavern crown, as well as surrounding the whole structure



(a) Calculated displacement (b) Calculated crack widths

Fig. 8 Strand7 model outputs showing calculated lining deformation and crack widths under serviceability load conditions (SLS)

Fig. 9 Completed permanent lining with lift shaft opening evident in the crown (9 November 2020)

7.3 Post-processing

The proportioning of the reinforcement in the lining was carried out using the "Sandwich Model" (FIB Bulletin 45, 2008) under various ULS load combinations. The reinforcement quantities obtained from the ULS assessment were validated against the crack width criteria for the SLS load combinations and adjusted as necessary such that relevant criteria were satisfied. Long-term deflections under SLS load combinations were also checked.

Fire design was checked in accordance with the approach in EN1992-1-2 employing the RABT-ZTV (railway) fire curve. During fire and post fire check analyses were undertaken, with reinforcement increased where required.

Additional analyses incorporating the variation in stiffness of the concrete lining that occurs with the onset of cracking was also undertaken. This method involves an iterative approach, commencing with the assumption that the reinforced concrete lining is uncracked (i.e. maximum stiffness). The results from the first iteration are used in conjunction with the proposed reinforcement details to assess the degree of cracking experienced, and thus to adjust the bending stiffness to be applied in the next iteration. This is performed for each plate element in the model, such that the stiffness varies across the model. The process was repeated until the results converged to a stable solution. This type of analysis is considered more reliable and realistic than the initial (single modulus) analysis [6].

8 Design Outcomes

The cavern crown lining predominantly comprises steel fibre reinforced concrete, however, detailed analysis of the lift intersection demonstrated that bar reinforced concrete was required to achieve the strength and serviceability requirements.

A lining thickness of 650 mm was adopted for the cavern crown and 600 mm for the lift shaft walls, waiting area and adit walls, and adit crown and invert slab. Movement joints were provided at critical locations, including between the cavern and lift shaft, and between the lift waiting area and lift adit (Fig. 3). Additional waterproofing details, including reinjectable grout hoses, were installed between the cavern crown and lift shaft to provide a more robust waterproofing solution.

9 Conclusions

The lift shaft to cavern intersection discussed herein comprises an unusual geometry, with the access adit located just two metres above the cavern crown. This proximity results in complex interactions between the adit and cavern structure, the rock which separates them, and the groundwater pressures which can cause the cavern to float. The discrete thin rock beam between the adit and cavern simultaneously supports the adit and loads the cavern.

Complex 3D finite element modelling was performed to explore various interactions between the adit and cavern and ensure that the structure fulfilled the various design requirements. The modelling was also used to inform aspects of the final design such as articulation joints and waterproofing details.

The permanent lining was completed in late 2020 with construction of the internal structures and rail fit-out works currently underway. The station is due to open to the public in 2024.

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Differentiating Fill and Natural Soft Clays—The Value of Desktop Studies in Building a Geological Model

Luke Johnstone and Sergei Terzaghi

Abstract This paper presents the methodology and rationale used when differentiating fill and natural soft clays in a site on waterfront reclaimed land. The area of reclaimed land has a history of settlement impacting existing developments, which has motivated careful consideration of the ground conditions for future works. As the reclaimed land was dredged locally then loaded by fill and warehouses, it closely resembles the underlying estuarine and marine sediment lithology and consistency. Two historical ground investigations and one Arup ground investigation have been conducted, including clusters boreholes, cone penetrometer tests, seismic dilatometer tests and lab testing. The four types of subsurface information were compared to build and verify a ground model with an emphasis on the extent of reclaimed land. Peripheral desktop study information including historical maps, imagery, sea level records, and the historical settlements observed at site were considered to assist in differentiating the fill and natural soft clays. The importance of understanding this site's history reinforces the value of a thorough desktop study when developing geological models.

Keywords Soft clays · Settlement · Desktop study · Ground model · Site investigation · Reclaimed land

1 Introduction

Soft sediments are highly compressible, prone to settlement and their classification underpins engineering on soft soils. This study focuses on a site in a waterside suburb in Western Sydney underlain by reclaimed land and estuarine/marine sediments. The

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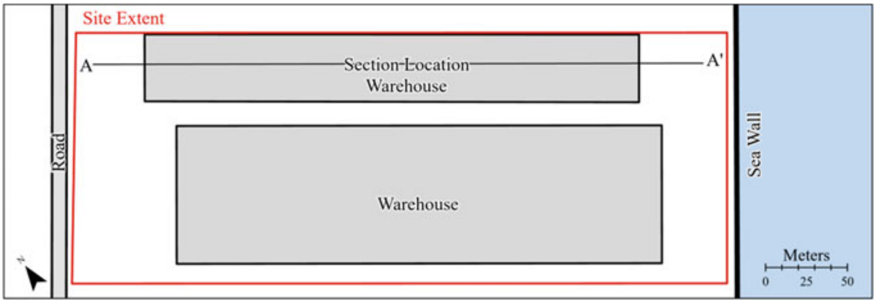


Fig. 1 Site setting

reclaimed areas of the suburb have a history of settlement impacting existing buildings and infrastructure. This history of settlement has driven careful consideration of sediment characteristics in future developments.

The site is approximately 400 m long and 150 m wide and currently hosts warehouses separated by concrete slabs, as presented in Fig. 1. It is bounded on housing developments on either side, a road on the west and waterfront on the east. The site elevation ranges from 2 to 5 mAHD and flat to undulating at up to 5°.

During engagement, Arup reviewed publicly available information on the site, historical ground investigations, conducted a ground investigation and built a ground model to inform design. The focus of the investigation was to classify the soft sediments encountered on site.

2 Ground Investigation

Three ground investigations (GI) have been completed including boreholes (BH), auger holes (AH), cone penetrometer test (CPT) and seismic standard flat dilatometer (sDMT). During the ground investigation surface level surveys were conducted, piezometers (PZ) and vibrating wire piezometers (VWP) were installed. A summary of ground investigations is presented in Table 1.

Two historical GI have been completed, in 1998 and 2017. The first was completed to assess the likely cause of observed settlement in a warehouse floor. A second was completed to support further development of the site. This GI and testing focused

Table 1 Ground investigation summary

GI	AH	BH	CPT	sDMT	PZ	VWP
Historical 1983			8		3	1
Historical 2017	42	18	20		6	
Arup 2021		3	9	4		

on contamination, rock level and soil condition. The 2017 GI clustered 18 deep geotechnical holes with CPT, and in these boreholes soft sediments lithology was inferred from the adjacent CPT.

Arup completed a subsequent ground investigation with a focus on soft sediments identified in previous ground investigations. To enhance recovery, identification, and classification of soils a number of techniques were used:

- Sonic coring boring was used to maximise recovery and allow for identification of sediment structures.
- Undisturbed samples were taken with U75 and push tubes with pocket penetrometer and shear vane tests performed on their base.
- Standard penetrometer tests (SPT) were taken at regular intervals.
- The CPT testing included pore pressure dissipation tests.
- sDMT testing included seismic testing.

The ground investigation included three grouped clusters of BH, CPT and sDMT for comparison between recovered soils and in situ testing, the location of Arup ground investigation presented in Appendix A.

3 Geological and Anthropogenic History

3.1 Tertiary

Reference to geological maps [2] indicate that the bedrock of the area is Ashfield Shale with Hawkesbury sandstone 500 m north of the site. The Ashfield Shale was deposited in a marine basin subject to cyclical infilling associated with the west to east migration of a large delta. The Hawkesbury Sandstone was deposited by large, braided rivers during the Middle Triassic [4].

3.2 Pleistocene

The project area lies within the Port Jackson drowned river valley system of the Lower Paramatta River. These are typically narrow steep-sided bedrock valleys associated with the vertical jointing of the Hawkesbury Sandstone. Herbert and Helby [14] describes this area as having basial deposits comprised of stiff clay. The surface of these clays is commonly deeply weathered with leaching, iron staining and mottling of white, red and light grey. A pre-Holocene age has been established by radiocarbon dating of peat and shell beds near Garden Island.

3.3 Holocene

The Pleistocene sediments were eroded during the last glacial minimum when sea levels were lowered, and fluvial processes were reactivated incising into the sediments. The next major phase of sedimentation commenced in the early to middle Holocene towards the end of the postglacial marine transgression. During this phase the valley systems were re-occupied by the sea as tidal delta, mud basin, and channel deposits were formed in an estuarine environment [15].

3.4 Anthropocene

The site area is the traditional home to the Wangal Clan of the Eora or Dharawal people (Lee et. al., 1998). Prior to reclamation the site was an estuary comprised saltmarsh, mangroves and mudflats. The pre-reclamation environment in 1893 is mapped by [9] and presented in Fig. 2a.

Post colonisation the existing land was cleared for grazing [1]. To facilitate reclamation of the estuarine areas a fascine dyke (seawall) was constructed in the early 1880s. The seawall was partially enclosed with an opening to the south to allow for tidal inflow and outflow.

Initial dredging and reclamation were completed between 1905 and 1917. This initial reclamation raised the ground to intertidal levels, allowing widespread growth of mangroves shown in imagery from the 1930s [11]. The increased ground level and widespread mangroves restricted water flow, channelising it into a distinct north-east trending channel. During the 1930s the channel intersected the site bounded by CPT06 and CPT08, by 1943 increasing ground level and mangrove growth constricted the channel, with the eastern edge moving from CPT08 to Cluster Three. The ground level was raised to current levels during dredging from 1948 to 1961. The evolving environment during reclamation from 1930 to 1983 is mapped by [1] and presented in Fig. 2b–c.

Post reclamation, the project area was primarily industrial including state abattoirs and brickworks, this industrial use resulted in contamination. The area was redeveloped with waterside apartment blocks in the 1990s. On the site, the first warehouse (the southern building) was constructed in 1965 and by 1980 all warehouses had

Fig. 2 Site historical environment, **a** 1893, **b** 1930, **c** 1951

been completed. The warehouses are portal frames with external walls supported on piles. The concrete slabs in the site have been observed to settle. The client indicated that a high pressure watermain burst in the vicinity of Cluster Three recently.

4 Ground Model

A geological ground model of the site was built using available ground investigation. The sub surface strata were generalised to geological units by origin, characterisation and behaviour. The thickness and distribution of these units varies though the site, however, they are represented by an interpretive geological section running the length of the site. The geological units are presented in Table 2 and section in Appendix A, and the location of the section is presented in Fig. 1. In building the ground model, all available ground investigation was considered, however, for clarity overlapping BH, CPT and sDMT have been removed.

Geological models inherently contain a measure of uncertainty as abstractions. However, typically these uncertainties are confined at the points of ground investigation. This is true for all geological units except the soft clays (DF and HC) as they have similar lithology and consistency. While they have similar lithology and consistency their age and history results in different engineering behaviour, making their differentiation essential. The comparison and differentiation of the soft clays is discussed in subsequent sections.

Table 2 Geological units

Origin	Unit	Name	Description
Anthropogenic Fill	IF	Imported Fill	Gravel and gravelly clay, shale, igneous and brick gravel
	DF	Dredged Fill	Soft to firm silty clay with trace shells and organics, sand lenses
Holocene Sediments	HC	Estuarine/marine clays	Soft to firm silty clay with trace shells and organics, sand lenses
	HS	Estuarine/marine sands	Very loose to loose silty sand
Pleistocene Sediments	PC	Estuarine/marine clays	Firm to very stiff mottled medium to high plasticity clay

5 Differentiating Fill and Natural Soft Clays

5.1 *Comparison of Units*

As the Dredged Fill has been sourced locally from the underlying Holocene Clays, the lithology identical. Both units include sand lenses, however, these may be present due to changing depositional environment or variations in fill source and type. In the south-east of the site Dredged Fill was placed on mudflats, mangroves in the middle and saltmarsh on the north-west (pre-reclamation environment is presented in Fig. 2a). During reclamation mangrove and saltmarsh flora and fauna grew though the reclamation area (Fig. 2b, c). Due to the changing environments within the site, the shell and organic component of the sediment cannot be used to differentiate between Dredged Fill and Holocene Clay. During reclamation and development of the site significant loads have been placed on the soft clays. As a result, all units have been loaded beyond the historical maximum of the natural material. The resulting materials are identical by lithology and consistency.

5.2 *Considering Peripheral Information*

During the desktop study publicly available information on the site history was gathered. This peripheral information can be used to guide interpretation of the geological model. The peripheral information may be used to infer pre-reclamation ground level, an assumed upper bound for the natural sediments (Holocene Clay). During the subsequent loading the natural sediments underwent settlement, the pre-reclamation ground surface lowered by subsequent settlement may be assumed as a lower bound for the fill materials (Dredged Fill).

5.2.1 Historical Ground Level

The historical environments observed in the site area (river, mudflat, mangrove, saltmarsh and forest, presented in Fig. 2) are all found within specific tidal rages. The extent of environments and historical tidal levels are used to construct the historical ground level. The typical tidal levels associated with each environment are adopted from MESA [8]. Historical tidal levels calculated as 2021 tidal levels [3] are adjusted to historical level by sea level rise at 0.65 mm/yr [10]. Environmental boundaries, associated tidal levels and assumed elevations are presented in Table 3.

Table 3 Environmental habitat in relation to tide level and elevation (m AHD)

Environmental boundary	Tide level	1893	1930	1951	2021
River to mudflat	Mean low tide	−0.58	−0.55	−0.54	−0.49
Mudflats to mangroves	Mean sea level	−0.09	−0.06	−0.05	0.00
Mangroves to saltmarsh	Mean spring high tide	0.41	0.44	0.57	0.62
Saltmarsh to forest	Highest recorded tide	1.40 recorded in May 1974 at Fort Denison			

5.2.2 Settlement

Post reclamation of the area, the natural sediments on the site have undergone loadings causing settlement, it is assumed that all of these settlements have occurred within the Holocene Clay unit. The historic loadings are presented in Table 4.

Three settlement levels are calculated using the parameters presented in Table 5, Eqs. 1 and 2 [13]. Equation 1 uses the results of sDMT in situ testing, with a representative coefficient of compression of $m_v = 0.4$. Equation 2 uses the results from odometer lab testing, two settlement levels are calculated, first with conservative

Table 4 Loadings applied to Holocene Clay

Loading stage	Loading	Unit weight (kN/m^3)	Quantity
Pre reclamation	HC self-weight	16	Thickness varies though the site and is presented in Fig. 3
Post reclamation	Weight of DF	15	
	Weight of IF	20	Assumed at 1.5 m
	Concrete, warehouses and freight	Na	30 kPa (historical study)

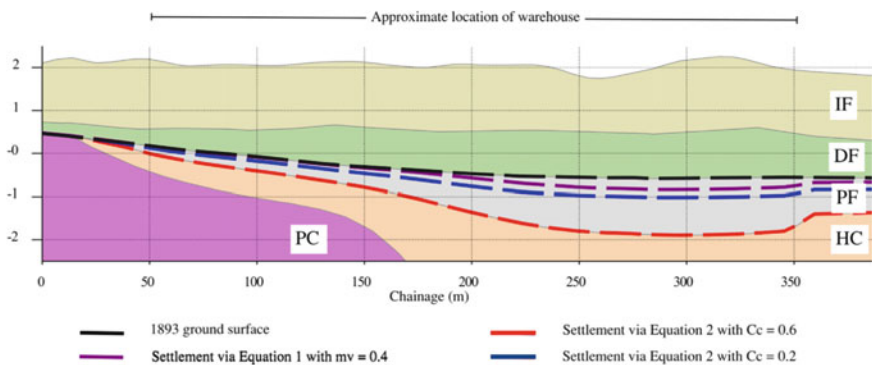


Fig. 3 Section with settlement levels and inferred units (20 × vertical exaggeration)

Table 5 Parameters values and commentary

Parameter	Symbol	Value	Notes
Coefficient of compression	m_v	0.4 MPa^{-1}	Representative value assumed from sDMT testing
Compression index	C_c	0.2	Conservative value assumed from odometer testing
		0.6	Non-conservative value assumed from odometer testing
Initial void ratio	e_i	1.4	Representative value assumed from odometer testing
Change in height of HC	δh	Variable	HC thickness and settlement varies though the site and is presented in Fig. 3
Thickness of HC	H_i		
Final stress	σ'$_f$	Variable	Initial and final stress varies though the site and is a product of loadings presented in Table 4
Initial stress	σ'$_i$		

compression index value of $C_c = 0.2$ and non-conservative value of $C_c = 0.6$. Calculated settlements are variable though the site and presented in Fig. 3. For calculation of effective stress, water level was assumed at 0 mAHD. There are limitations to the approach taken. It is noted this does not consider time or drainage and the parameters are associated with the existing clays as opposed to the pre-settlement clays. Secondary consolidation is not considered in these calculations, however, it is acknowledged as significant.

$$\delta h = H_i m_v \left(\sigma'_f - \sigma'_i \right) \tag{1}$$

$$-\delta h = \frac{1}{1 + e_i} H_i C_c \log_{10} \left(\frac{\sigma'_f}{\sigma'_i} \right) \tag{2}$$

The pre-reclamation (1893) ground surface is assumed as the maximum potential level for Holocene Clay, above this level all sediments are fill with a high certainty (DF). The pre-reclamation ground level reduced by the non-conservative settlement level is assumed as the lower bound for fill units and sediment below this level is Holocene Clay with a high certainty. The envelope between these two bounds is assumed to be fill with a low certainty, and this area will subsequently be named Potential Fill (PF). A section with settlement levels is presented in Fig. 3.

5.3 *Unit Validation*

5.3.1 Soil Behaviour

To validate the fill origin of the Potential Fill unit, its behaviour is compared to the overlying Dredged Fill and underlying Holocene Clays. The results of CPT and sDMT in situ testing is used to calculate soil parameters assessing the behaviour of each unit.

CPTs were carried out in accordance with ASTM D3441-16 using a truck-mounted rig. The test was conducted by pushing a 35 mm diameter instrumented cone tipped probe into the soil using a hydraulic ram system. Measurements were made of the end-bearing pressure on the cone tip and the friction on a 135 mm long sleeve located immediately behind the cone at 10 mm intervals. Soil behaviour type and parameters have been calculated by CPeT-IT v3.5.4.9 as described in [12]. The soil behaviour type is an empirical behaviour-based classification that must be verified with lab testing.

sDMTs were carried out in accordance with ASTM D6635-15 using a truck-mounted rig. The test is conducted with a stainless-steel blade having a flat, circular steel membrane. The blade is advanced into the ground and the steel membrane is inflated at 200 mm intervals and the pressure required to inflate the membrane recorded. Seismic shear wave velocities were also recorded at 0.5 m intervals. The number of measurements in each unit is presented in Table 6.

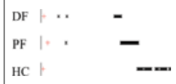
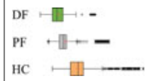
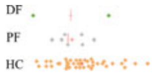
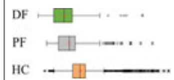
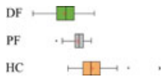
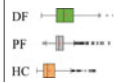
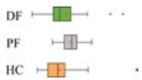
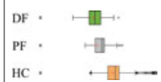
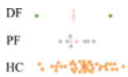
The relationship between unit and soil behaviour is presented in Table 7. This is presented as the distribution of parameter values for each unit plotted as a boxplot. Where values are split into clusters or there is a low number of observations a boxplot may be misleading. In these cases, the distribution of values is visualised as a scatterplot. Due to probity requirements the plots are dimensionless.

The results of in situ testing generally indicate that Potential Fill and Dredged Fill have similar behaviour verifying the fill origin of Potential Fill. To further verify this Dredged Fill and Potential Fill behaviour is often distinct from Holocene Clays. The soil parameters OCR and shear strength show the clearest relationship to geological unit. The parameters calculated from CPT and sDMT results typically agree. It is noted that these results are observations of means and value distributions. The relationships should be further tested for statistical significance.

Table 6 Number of in situ measurements

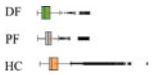
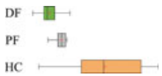
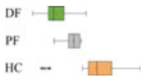
Unit	Fine grained			Coarse grained		
	CPT	DMT	Seismic testing	CPT	DMT	Seismic testing
Dredged Fill	507	19	2	99	2	1
Potential Fill	683	19	7	23	1	1
Holocene Clay	3076	104	40	102	5	2

Table 7 CPT and sDMT soil characteristics

Parameter	CPT distribution	sDMT distribution	Observations
Fine grained material			
Youngs modulus (Es)		Parameter not calculated from sDMT results	All units have similar mean HC outliers higher than DF and PF
Shear modulus (Go)			CPT mean and value range increase from DF to PF and HC sDMT distributions are similar
Shear strength (Su/Cu)			Mean increases slightly from DF to PF HC mean higher than PF and DF
Over-consolidation ratio (OCR)			HC mean is lower than PF and DF DF and PF value range and mean similar but variable between CPT and sDMT
Shear wave velocity (Vs)			Distribution of PF and DF units are similar, HC higher sDMT distributions are similar

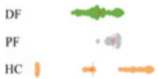
(continued)

Table 7 (continued)

Parameter	CPT distribution	sDMT distribution	Observations
Soil sensitivity (St)		Parameter not calculated from sDMT results	Mean increasing from DF to PF and HC HC has higher outliers than fill units
Note [1] No relationship observed in Constrained modulus (M) and friction angle (φ) Note [2] Shear strength (Su) adopted from CPT testing, undrained shear strength (Cu) adopted from sDMT testing			
Coarse grained material			
Friction angle (φ)			CPT range of values is similar, however HC has outliers significantly lower than DF and PF sDMT HC mean lower than DF and PF
Youngs modulus (Es)		Parameter not calculated from sDMT results	DF and PF distribution similar HC mean clearly higher than DF and PF
Shear modulus (Go)			Increase in value distribution and mean from DF to PF to HC HC mean higher than DF and PF

(continued)

Table 7 (continued)

Parameter	CPT distribution	sDMT distribution	Observations
Shear strength (Su/Cu)			DF and PF distribution similar HC mean clearly higher than DF and PF
Over-consolidation ratio			DF and PF exclusively 0 HC includes OCR of 0 and 5 and higher mean
Shear wave velocity (Vs)			DF and PF distribution similar HC has distinct clusters and higher range
Soil sensitivity (St)		Parameter not calculated from sDMT results	DF and PF values are 0 HC vlaues and average higher

Note [1] No relationship observed in Constrained modulus (M)
Note [2] Shear strength (Su) adopted from CPT testing, undrained shear strength (Cu) adopted from sDMT testing

In fine grained soils Holocene Clay exhibits a higher shear modulus, shear strength, undrained shear strength, shear wave velocity and lower OCR. Youngs modulus has similar mean values in all units, however, Holocene Clay has much higher outliers than Potential Fill and Dredged Fill. Soil sensitivity values have similar value distributions in all units, however, Holocene Clay has much higher outliers than Potential Fill and Dredged Fill.

In coarse grained soils Holocene Clay exhibits higher Youngs modulus, shear modulus, shear strength and undrained shear strength. Holocene Clay friction angle, OCR and shear wave velocity have varied distribution of values while Potential Fill and Dredged Fill are tightly clustered. Soil sensitivity has similar mean values in all units, however, Holocene Clay has much higher outliers than Potential Fill and Dredged Fill.

5.3.2 Site History

Analysis of the site history though the reclamation process provides a basis for understanding and interpreting observations within the soft clays. Within fill units the CPT cone resistance (Qc) is variable (associated with sand and shell lenses) while Holocene Clay is more consistent with gradational transitions in values. This trend is not seen at Cluster 3, potentially due to a water main bursting and disturbing the sediments in this area. The fill at Cluster 1 has a highly varied profile. Historical imagery shows that the outer bounds of the reclamation area adjacent to the seawall were filled first then progressively inwards. This early filling and construction of the sea wall may be the source of the varied profile in Cluster 03. A sand rich lens expressed as high Qc is often encountered at the base of fill units. During the reclamation of the project area a channel is formed running though the site, allowing tidal inflow and outflow. The channel and higher energy environment may be associated with an increase in grainsize at the base of the fill units.

5.4 Discussion

Arup conducted an extensive desktop study and specialist ground investigations with a focus on classifying soft sediments. The soft sediments in this site include a transition from locally sourced land reclamation fill to the underlying natural estuarine sediments, an essential boundary to consider when building a geological model. Differentiation of these units by composition and consistency carried a significant level of uncertainty so peripheral information such as sea levels, site history and historical settlement was considered. This information provided a refined model with volumes of high certainty Dredged Fill, low certainty Potential Fill and high certainty Holocene Clays. Examination of the behaviour of these soil volumes via CPT and DMT testing verifies the Potential Fill source as Dredged Fill. Trends observed in soil structure align with historical observations of channels forming within the site and reclamation history.

6 Conclusions

Soft sediments have a number implications for design, primarily settlement. To ensure that the risk of these sediments is mitigated it is important to clearly define their provenance and extent within a site. This paper highlights the importance of desktop studies and considering the peripheral information when performing a ground investigation and interpreting the results. The verification of a geological model in this manner drives increased confidence and allows for optimised design leading to safer, cheaper and more environmentally friendly construction.

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Geotechnical Challenges in Design and Construction of Bridge Foundations and Approaches in Hilly Granite Formation

Q. J. Yang and J. J. Dane

Abstract This paper presents a case study of geotechnical design and construction challenges of bridge foundations and approaches in a hilly granite formation in northern New South Wales, Australia. Firstly, the geological formation and existing cut slope conditions which have high risks of rock fall will be described. The original design was based on the available geotechnical information and assumed construction methodology. Reinforced concrete cantilever retaining walls founded on mass concrete were adopted for the bridge southern approach to resolve constructability issues over hilly terrain. The design considered retaining wall block sliding stability while overturning and internal stabilities were satisfied. Slope treatments using a rock fall fence together with individual boulder stabilisation or removal were also considered. It was found during construction that the actual ground conditions were different to that originally inferred and modifications to pad footing designs were deemed necessary. Additional investigations were undertaken, and the subsurface ground models updated to inform the revised design. For the northern bridge abutment foundation, a piled foundation was introduced to optimise the design with due consideration of temporary piling platform and access along a new geotextile reinforced approach embankment. The revised design was developed in close collaboration with the Contractor and the Principal. The foundation design of Pier 2 was revised using micro-piles to address the presence of a weak rock layer intrusion. In the end, key lessons learnt from this challenging project have been summarised for future project references.

Keyword Hilly granite terrain · Pad footing · Piled foundations · Retaining walls · Micropiles · Reinforced slope

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1 Introduction

The New England Highway Upgrade at Bolivia Hill is situated between Glen Innes and Tenterfield, New South Wales, Australia. The project is about 2.1 km in length and includes a bridge of approximately 320 m length crossing a steep valley. The bridge structure is an in-situ concrete box girder by balanced cantilever construction, with the main span being 150 m and the two end spans being 80 and 86 m. Arcadis was engaged by Transport for New South Wales (TfNSW, Principal) to undertake the concept design and Review of Environmental Factors (REF), and then the detailed design, in two stages.

This paper presents a review of the geological settings and the site history, the concept design developments of key elements, the geotechnical model and design parameters, the main bridge foundation design, the options considered for the approaches and the upslope rock fall risk management strategy. The geotechnical challenges encountered during the design development process and construction phase are further discussed. Key lessons learnt from this project such as the importance of constructability issue through hilly site formation are also summarised.

2 Project Background

Transport for New South Wales (formerly Roads and Maritime Services) proposed an upgrade of the New England Highway at Bolivia Hill, with primary project objectives to improve local traffic efficiency, road safety, road transport productivity, efficiency and reliability of travel. The New England Highway is commonly used by heavy vehicles, which make up approximately 25% of the total traffic volume in this area.

The Bolivia Range forms part of the Great Dividing Range in Australia and includes two main hills: Bolivia Hill at a Reduced level of 1225 m Australian Height Datum (AHD) and Little Bolivia Hill at a reduced level of 1100 m AHD. Surface elevations of the highway range from 935 m AHD at the south-eastern end to 817 m AHD at the north-eastern end of the project.

The existing road was constructed by cut and fill method in the 1950s, following the landform with curves requiring 75 km/h advisory speed. The road geometry, combined with the unforgiving steep terrain, resulted in a poor crash history compared with other sections of the highway network. Cuts were primarily formed into the upslope on the eastern side of the road with embankment on the western side of the road. The existing cuttings were typically 2–8 m high and battered at 30°–70° to the horizontal. The existing embankments ranged between 10 and 30 m in height at a batter gradient of 30°–55°. The exposed cut surface comprised granite bedrock of varying strength with little soil cover over bedrock. At the crest of the hill there were either isolated boulders or piles of boulders from construction of the adjacent disused railway some 100 years ago. The existing slopes were known for frequent rock fall and assessed to be medium to high risk.

3 Detailed Design Outcome of Key Elements

The design development process went through environmental assessment, concept design development and option engineering stages prior to detail design development. The approved detail design adopted a balanced cantilever bridge with a central span of 150 m to provide a straight alignment across a side valley.

3.1 Main Bridge and Abutments

An in-situ concrete box girder bridge by balanced cantilever construction was selected after due consideration of environmental constraints and constructability issues. The bridge is 316 m long and 12 m wide, as shown on Fig. 1, based on Arcadis Australia Pacific (AAP) reports [2].

The span arrangement and foundation solutions were heavily influenced by the steep terrain and granite landform, and the presence of Bolivia Wattle which created a construction exclusion zone. Access to the piers required significant temporary works, including large scale temporary retaining walls and rock bolt retention systems for upslope cuttings at the piers. Due to the access constraints for piling rigs and the very hard granite material expected, the originally proposed piled foundations were changed to pad foundations. This method of construction allowed the access tracks to be reduced in scale (although sections of the temporary walls were still up to 9 m high) as the working areas at the piers for cranes and concrete pumps could be set several metres below the underside of pad, whereas piling rigs would need access tracks to extend up to underside of pile cap level and be constructed at a lower gradient. Indicative access track designs including temporary retaining walls were

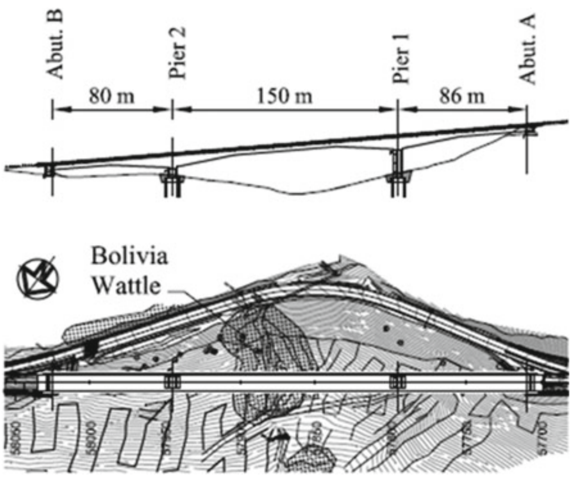


Fig. 1 Layout plan of proposed bridge and abutments

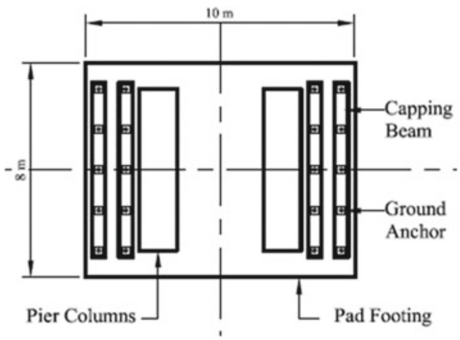
provided to the tenderers to ensure this significant element of the construction was adequately considered in their tender.

Figure 2 shows the 3D view of the proposed bridge close to completion of construction with respect to the existing road. The redundant section of highway is being retained for use as an access road for bridge maintenance.

The pad size for Pier 1 and Pier 2 presented in the final design, as shown in [19], is 10 m along the traffic direction by 8 m in the transverse direction, as shown on Fig. 3. Four rows of five temporary ground anchors were arranged at both ends of the pad to deal with the large overturning moments from out-of-balance load cases during the cantilever deck construction.

Fig. 2 Aerial view of the bridge near completion

Fig. 3 Layout plan of pad footing for Pier 1 and Pier 2



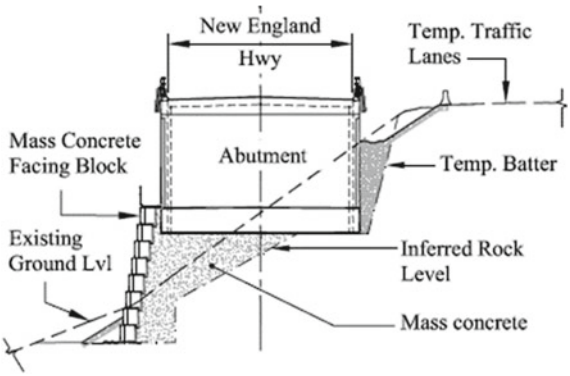


Fig. 4 Typical elevation of abutment retaining wall

The dimensions of Abutment A and Abutment B in plan in final design as presented by [19] are 7.26 m long and 11.8 m wide, with approach slab of 6 m length connecting to the abutments at both ends.

3.2 Northern and Southern Approaches

The permanent works design included two retaining structures, as shown on Figs. 4 and 5 [19], for the provision of the required road widening approaching Abutment A (southern end of bridge) and Abutment B (northern end of bridge). These were to be constructed adjacent to the existing road to allow for sufficient access to construct the bridge abutment structure, and to support the approach embankment fill (the road widening works) respectively. Part of the walls were to be formed in cuttings and the remaining are to be built over the existing ground by filling. A simple and flexible construction methodology was proposed for the wall and abutment foundations, using mass concrete with precast blocks as permanent formwork, to allow for some variance in foundation material level along the length of the structure.

3.3 Slope Stabilisation Works

The southern section of the alignment has been subject to “rock fall” risks for a long time; this site has been used for training for TfNSW (formerly RMS) slope risk assessment field works. TfNSW had previously undertaken slope assessment of the existing cuts and upslope, with some sections having an Assessed Risk Level (ARL) of 2 as reported by Roads and Maritime Services [16]. Subsequently TfNSW decided to carry out emergency remedial work prior to this upgrade project. The scope of works primarily included removal of unstable isolated boulders, stabilisation of rock

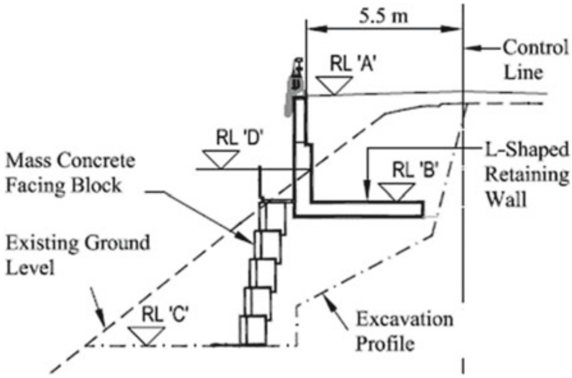


Fig. 5 Typical elevation of approach retaining wall

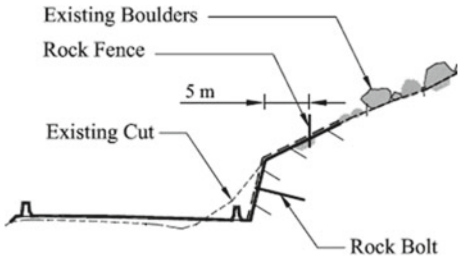


Fig. 6 Rock fence with new cut and upslope

boulders, and the "rock stockpile" formed during construction of the adjacent disused railway over 100 years ago.

A rock fence near the crest of the new cutting for the realigned southern approach was provided as part of the detailed design development. Rock fall modelling analysis showed that the proposed 35 kJ rock fall barrier located 5 m above the crest of the batter would be effective in preventing individual boulders less than 0.8 m diameter dislodged mid-backslope from reaching the carriageway. Furthermore, the design provided for field-mapped potentially unstable boulders to be stabilised using rock pins and grout. Figure 6 shows a typical cross-section of the rock fence in relation to the new cutting and the upslope profile and boulders.

3.4 New Cut Batter Design

There were several new cuttings proposed along the route of the scheme, primarily formed in rock materials of varying strength and defect spacing. Cuttings in soils were designed to have a batter gradient of 2 horizontal to 1 vertical for long-term stability and 1.5 horizontal to 1 vertical for short-term stability. The new cuttings in competent granite were to be trimmed to 1 horizontal to 4 vertical gradients.

The excavatability of the rock was assessed using the method by Pettifer and Fooker [15] and the results indicated that blasting would potentially be required for some portions of cutting through strong and massive granite at southern abutment and its approach retaining wall.

Assessment of the new cut batter slopes based on the defect information in the geotechnical investigation reports by Douglas [8, 9] indicated there were three possible failure mechanisms during the excavation phase:

- Localized slumping and shallow rotational movement: This would likely be induced by excavation and de-vegetation undermining within the highly weathered zones along the cutting.
- Boulder rolls: This would likely be caused by dislodgement of isolated rock block within the exposed cut face due to stress relief during and post excavation works.
- Block sliding and/or rotation of rock blocks: This would likely be due to the presence of unfavourable joints within the rock mass, with the block size dependent upon the joint set pattern where the cut face daylighted.

The contract documents provided for a qualified geotechnical engineer or engineering geologist to map the exposed rock cut face during excavation and confirm the need of any stabilisation measures on site. It was anticipated that minor scaling, rock bolting and shotcrete application may be required for highly weathered zones to stabilise the localised unstable rock mass where encountered.

As part of the design, the cuttings in soil were proposed to be grassed so that surface erosion can be controlled in the short and long term. The rock cuttings would be exposed and subject to long-term weathering, and therefore it was considered necessary to allow for clearing of the loose material from the slope and at the toe of the rock cutting. To ensure minimal maintenance in the long term a soft facing solution, such as mesh facing, was included in the suite of proposed treatments. This will minimize the potential risk of any rock falling onto the carriageway and affords the opportunity to carry out remedial works if deemed necessary.

4 Construction Stage Challenges

The project construction contract was awarded to Georgiou and SRG JV (GeorgiouSRG). Geoinventions was engaged by GeorgiouSRG to provide site investigation, geological assessment, and temporary work design services. Construction works started in May 2018 and completed in September 2021. Arcadis was retained to provide the construction stage services to TfNSW during the construction period.

4.1 Slope Stabilisation Works

The existing rock cut slope has experienced deterioration since construction in the 1950s and rock fall has occurred. It was considered necessary to carry out additional rock stabilisation works prior to major construction activities for the proposed new project works to ensure safety to the road user and construction crew. The design included a rock fall fence together with a schedule of potential unstable rock blocks that had to be stabilised. The contract documentation was set out in such a way that the contractor would be fully responsible for the safety of the road users and construction crew. Noting the general "precision" of the RMS Slope Risk Analysis procedure is such that in 80% of assessments two trained reviewers can independently generate ARL levels within plus/minus one ARL level, as reported by Baynes et al. [12], the design documentation allowed for varying quantities of a range of stabilization solutions.

After awarding the contract the contractor engaged an independent geologist to carry out an assessment of the rock fall risks to both road users and the construction crew down slope. It was apparent that the attitude to the uncertainties in risk rating to the road user and construction workers were different. The Contractor's responsibilities under WHS legislation drove a robust approach to worker safety (considering what is reasonably practicable to achieve) whereas the basis of the RMS slope risk assessment guidelines is safety of road users who are exposed to risk for merely seconds as they pass through the hazardous zone. Therefore, a greater emphasis was placed on safety for construction crew down slope, leading to increased upslope treatment measures including using rock fall mesh. Figure 7 shows the removal process of a potentially unstable rock block above the road as part of upslope stabilisation works.

4.2 Pier 1 Subgrade

The originally inferred founding material at Pier 1 was Class III granite based on the rock strength, defect spacing as proposed by Irfan and Powell [14] at the anticipated level of RL 849 m. There were two boreholes (BH04 and BH05) drilled close to Pier 1 footprint during the design phase, with borehole BH04 close to the northern end of the footing, as shown in Fig. 8, and the other (BH05) is approximately 10 m to the north. Therefore, as specified in the contract, two additional boreholes were required as a minimum but five were drilled by coring technique when access to Pier 1 was gained during construction. It was inferred from boreholes BH04, P1-B1, P1-B3, P1-B5, P1-B7 and P1-B9 that most of the rock mass beneath the pad footing satisfied the original design intent except for the northwest corner, as shown in Fig. 8.

The exposed founding material indicated that a localised shallow "remove and replace" by lean mix concrete up to 1.5 m deep was required to ensure uniformity of

Fig. 7 Removal process of a potentially unstable rock block (after TfNSW website)

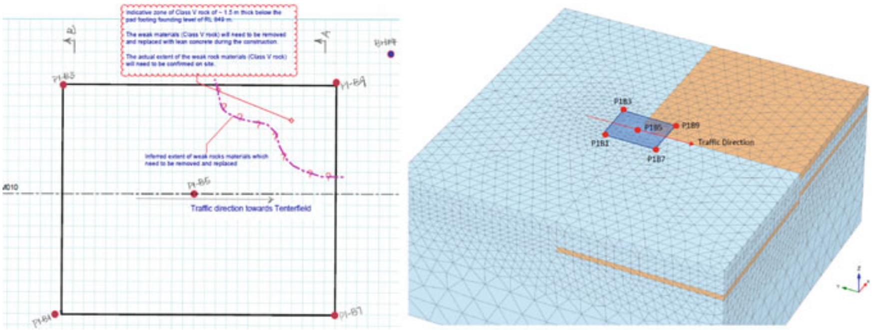


Fig. 8 Identified weak rock zone and additional borehole locations at Pier 1

the founding subgrade for the pad footing. Plaxis 3D modelling, as shown in Fig. 8, indicated differential settlement remained within acceptable limits.

4.3 Pier 2 Redesign

4.3.1 Ground Conditions

Two cored boreholes, BH06 and BH07 as shown in Fig. 9, were available for the detail design of the 10 m × 8 m pad footing. Three additional boreholes were drilled at Pier 2 by the contractor when access was available, as per the specification. The additional investigations indicated the presence of very low strength rhyodacite beneath part of the pad footing towards the upslope (south-east). Ten additional cored boreholes were subsequently undertaken by Geoinventions Consulting Services (GCS) [11, 12] to delineate the boundary of granite and rhyodacite, which is intruded in-between the granite formation as shown in Fig. 9. Figure 10 presents an inferred geological cross-section A-A of Fig. 9, showing the varying rock types and quality. Rhyodacite was interpreted to be very low strength with depths varying from 5 to 9 m. Figure 11, based on GCS report [11, 12], shows the cored log photographs of two distinctly different rock mass qualities (P2-B11 for granite and P2-B4 for rhyodacite) beneath the pad footing, with the redline mark showing the pad footing level of RL 849 m.

The concern for the performance of the bridge structure was serviceability, particularly during construction: the elastic response of the foundation would not be uniform across the pad potentially affecting the alignment of the 80 m cantilevers.

4.3.2 Option Development

Three initial options were investigated, including (1) Excavate and replace with concrete; (2) Trenching and replacement by concrete; and (3) Micropile ground improvement. Option 1 was not pursued due to the required depth of excavation in hard rock (as well as weaker material) and additional work/rework of temporary retaining system on the upslope cut. Option 2 had constructability difficulties due to limited working area for heavy excavators. Option 3 was preferred based on the site access and constructability. However, the micropile application was not typical for such a significant bridge structure in NSW. Therefore, there was a process to provide technical justification to TfNSW together with proof engineering of the alternative design foundation system before approval was given to proceed.

This option required excavating down to the design foundation level of RL 849 m and installing micropiles to form a partly piled raft footing with a more uniform foundation response. The proposed micropiles were to be approximately 200–300 mm in diameter, with the diameter adjusted to suit the equipment that the contractor has planned for the project. The micropiles would be grout piles and would typically

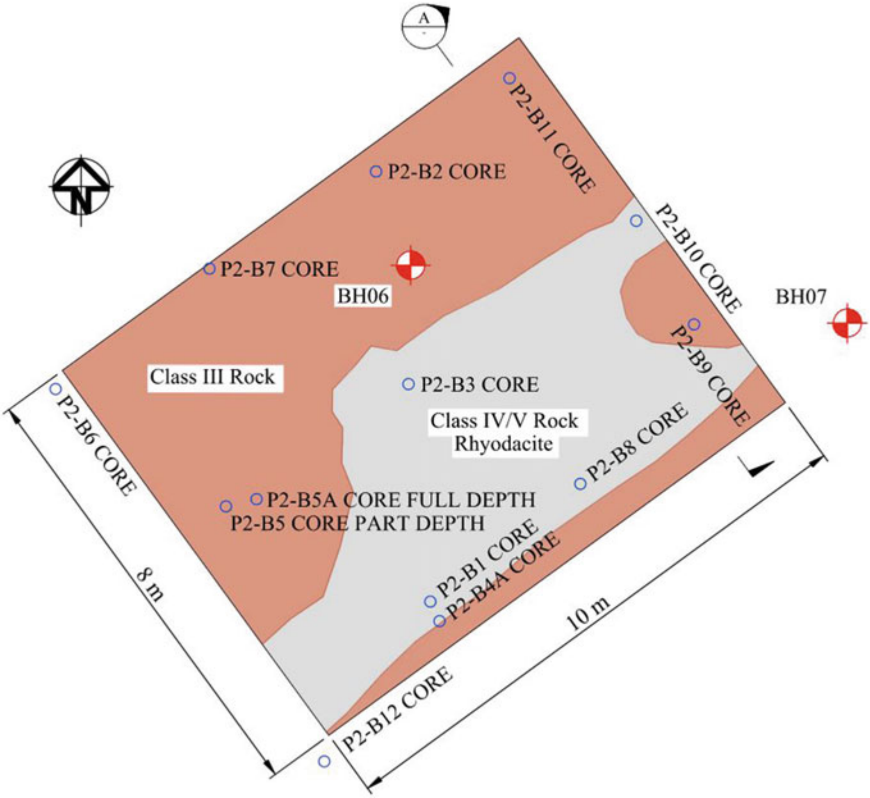


Fig. 9 The extent of rhyodacite and locations of drilled boreholes (after GCS [11, 12] information)

require at least one reinforcement bar through the middle of the pile. The adoption of micropiles required further analysis of the bridge structure to confirm its performance.

This option would not require any additional excavation compared to the other options. The micropiles were only required in the areas where very low strength Rhyodacite was found, and therefore the number required was confirmed by inspection when excavation reached the design founding level.

4.3.3 Rock Mass Classification and Geotechnical Design Parameters

The rock classification and the geotechnical design parameters were based on the original geotechnical interpretative Report. The rock mass classification was generally based on the method by Irfan and Powell [14] and the specific UCS values and defect spacing for each class of rock is presented in Table 1.

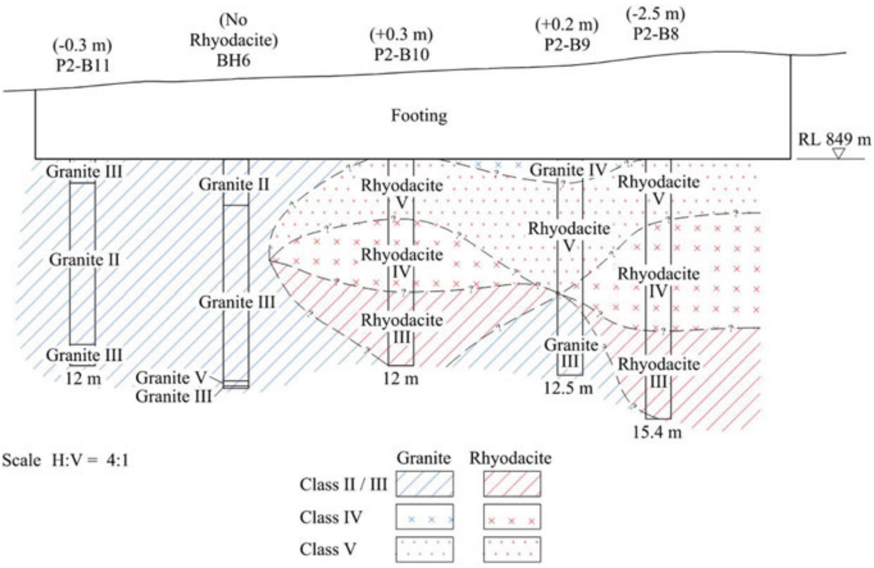
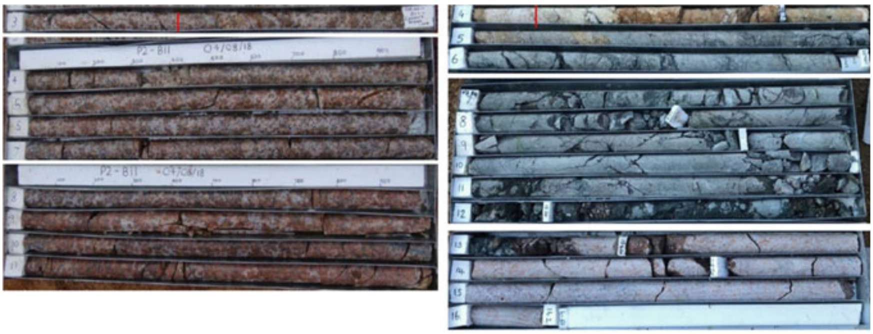


Fig. 10 Inferred geological cross-section A-A (after Geo-inventions Consulting Services [11, 12])



P2-B11 P2-B4

Fig. 11 Cored borehole photos of granite (P2-B11) and rhyodacite (P2-B4)

Table 1 Classification of granite and other rocks

Rock class	Point load strength, Is(50), (MPa)	Unconfined compression strength (MPa)*	Defect spacing (mm)
V	<0.3	<6	<60
IV	0.3–1	6–20	<60
III	1–3	20–60	>60
II	3–6	60–120	>200
I	>6	>120	>200

*UCS was taken to be 20 times $Is_{(50)}$

Table 2 presents the geotechnical design parameters adopted for the detail design development and the re-design for the pad footing and micropiles.

Table 2 Geotechnical design parameters

Unit	ϒ	c'	Φ'	k_0	f_{su}	f_{br}	E'	f_{all}	μ
1(a)- Fill	18	0	30	0.5	N/A	N/A	28	_	0.3
1(b)- Alluvium	20	0	33	0.45	0.03	N/A	40	_	0.3
Class V Rock	23	0	38	0.8	0.2	3	150	1	0.25
Class IV Rock	24	5	38	1	0.9	9	500	3	0.25
Class III Rock	25	140	38	1	2.0	24	900	6	0.2
Class II Rock	26	540	38	1	3.0	32	1600	12	0.2
Class I Rock	27	4000	38	1	4.0	50	2190	24	0.2

Definitions ϒ (kN/m^3) = bulk density; c' (kPa) = effective cohesion, Φ' (Deg) = effective friction angle, k_0 = in-situ stress ratio, f_{su} (MPa) = ultimate shaft adhesion for bored pile and passive ground anchors, f_{bu} (MPa) = ultimate end bearing for bored pile, E' (MPa) = Young's modulus, f_{all} (MPa) = allowable end bearing pressure, μ = poisons ratio

4.3.4 Micropile Detail Design Considerations

The Limit State structural capacities of the micropile for both the short term (Construction) and long term (Operational) were in accordance with Australian Standard AS5100 [5].

Construction and detailing of the micropiles was in accordance with the requirements of Project Specification B114 and applicable recommendations of the FHWA NHI-05-039 December 2005 Micropile Design and Construction—Reference Manual [10]. Static Acceptance Load testing of the micropiles was to be undertaken in accordance with FHWA NHI-05-039 with reference to AS2159 [4] where necessary.

The particular aspects considered in the micro-pile design included the following:

- Durability—use of a steel CHS 168 OD (outer diameter) casing placed over the length of the micro pile between the underside of the pad foundation and the Class III rock. An additional 0.02 mm/year corrosion allowance was accounted for in the assessment of structural capacity in the operational load cases;
- Settlements and rotations—The maximum rotation and settlement of the foundation was kept within the limit used in the conforming design of the bridge including substructure and superstructure;
- Slope stability—The global stability of the slope is not reduced by the improvement works;
- Structural section capacity—The axial and flexural capacities of the micropile in both the construction and operational phases have been provided;

- Casing availability—A 168 × 11 mm thick casing was procured to ASTM A53M [3] specification and this was adopted in the structural analysis;
- Construction tolerances—The micropiles were designed to resist induced bending moments resulting from verticality tolerances of 1% in accordance with AS2159. The addition of the steel casing has been considered in the structural capacity of the pile in both bending and axial compression. Verticality tolerance of up to 4% was also assessed as a sensitivity case; and
- Testing of piles for tension and compression.

A total of 29 micropiles, as shown in Fig. 12, were considered for the identified "weak" zone of Class V or Class IV rock inferred from the available cored borehole data.

The micro piles were taken to form an overall ground improvement and stiffening of the Class IV/V layer allowing the majority of loads to be transferred to the more competent Class III layers below. Structurally the micropiles were assumed to transfer these bearing loads in axial compression. An allowance for vertical tolerance of 1/100 has been included in the design in accordance with AS2159, however, up to 4/100 has been assessed to demonstrate the impact of the surrounding rock confinement on limiting induced moments.

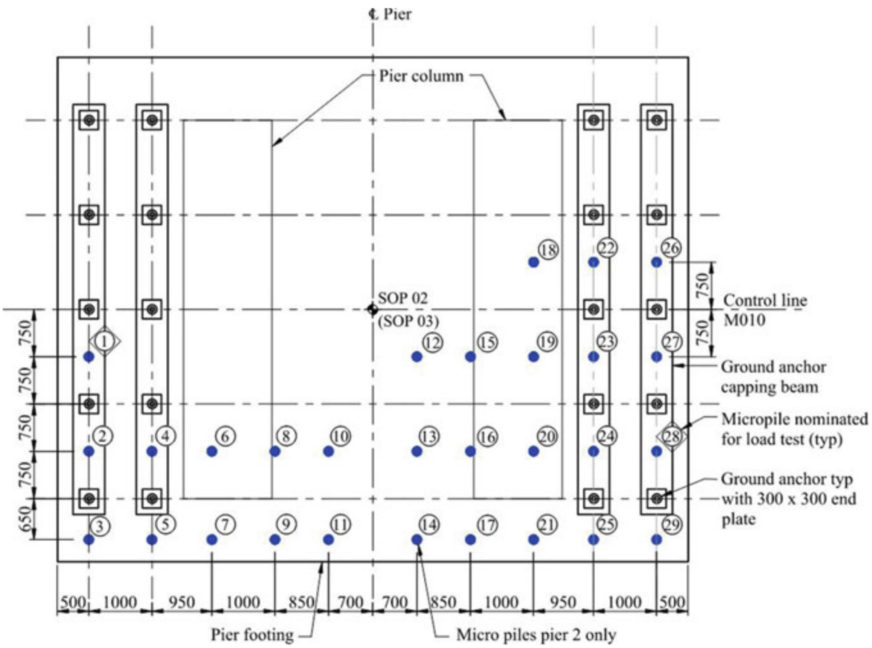


Fig. 12 Layout plan of 29 micropiles of varying lengths at Pier 2

4.3.5 Plaxis 3D Modelling and Results

Three-dimensional finite element analysis software, Plaxis 3D version 2017, was used to simulate the foundation behaviour to assess the performance of Pier 2 for both SLS and ULS loading conditions. Soil and rock strata were modelled as elastic–plastic materials obeying the Mohr Coulomb failure criterion. Micropile and footing were modelled as linear-elastic material, with the grout Young's modulus of 35 GPa and 30 GPa, respectively. The adopted material parameters for soil and rock strata were based on those shown in Table 2.

The model developed using Plaxis 3D is shown in Fig. 13. Three most critical cases were analysed using Plaxis 3D: (i) Case A—Critical construction-SLS, (ii) Case B—Critical construction-ULS, and (iii) Case C—Service earthquake.

The vertical, horizontal and equivalent bending moment in both longitudinal and transverse directions at the centre of the pad footing are presented in Table 3.

The results of the Plaxis 3D modelling for these three cases are summarised in Table 4, with the axial forces for Case B presented in Fig. 14. Note that the values shown at the bottom of each pile are loads at the tip of pile.

The calculated settlements and rotations induced on the foundation have been assessed and can be accommodated by the bridge structure both during the temporary construction and permanent operational states.

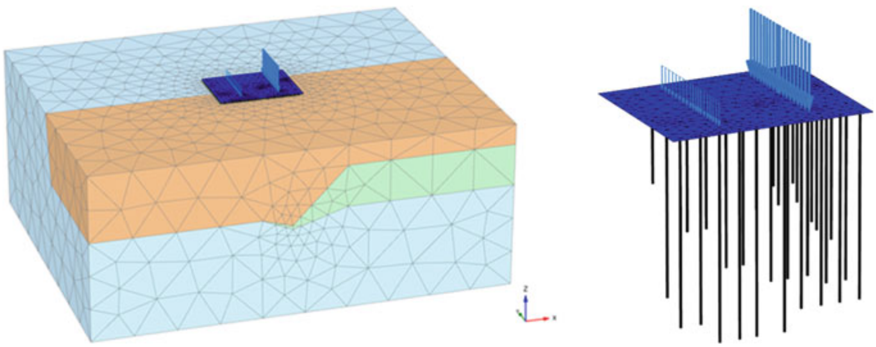


Fig. 13 Plaxis 3D model at Pier 2 with 29 micropiles

Table 3 Summary of loads acting on Pier 2

Cases	P (MN)	ML (MNm)	MT (MNm)	VT (MN)	VL (MN)
A: Critical construction-SLS	86.8	140.0	1.5	0.5	0.5
B: Critical construction-ULS	91.0	277.0	2.0	1.0	1.0
C: Service EQ	72.0	65.0	55.0	5.0	2.0

Note P = Vertical load at the centre of pad; ML = Bending moment in longitudinal direction; MT = Bending moment in transverse direction, VT = shear in transverse direction; VL = shear in longitudinal direction

Table 4 Summary of Plaxis 3D modelling results

Cases	N (kN)	Q (kN)	BM (kN)	S (mm)*	RT-L	RT-T
A	1650	166.1	19.4	2.5 to 16.5	1 in 715	1 in 2285
B	2103	215.1	28.3	–4.5–28	1 in 310	1 in 1330
C	1700	180.5	26.9	–1.5–16.5	1 in 650	1 in 840

Note *N = Maximum axial load of a single micropile; Q = Maximum resultant shear force of a single micropile; BM = Maximum resultant bending moment of a single micropile S = Calculated settlement range, *–ve means upwards; +ve downwards; RT-L = Rotation in traffic direction; and RT-T = Rotation in transvers direction

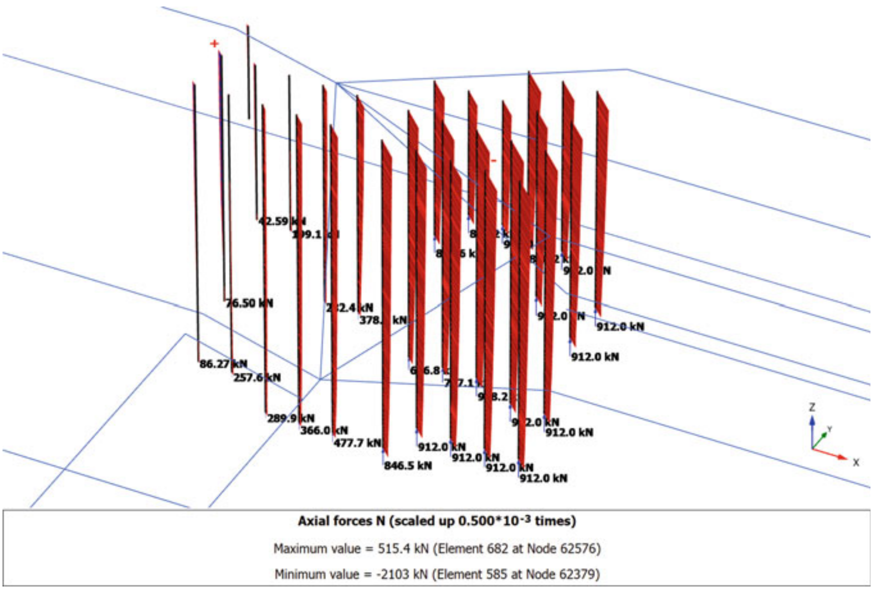


Fig. 14 Axial loads in micropiles from Plaxis 3D model—Case B

Pile geotechnical capacity assessment in accordance with [1] LRFD indicated that approximately 4 m of socket into Class III rock was required (when ignoring the casing to rock bond over the plunge length). For each location/area the pile length was adjusted to achieve the required geotechnical and structural capacity. In general, the length of the piles will be varying between approximately 5 and 11.5 m.

4.3.6 Structural Design of Micropiles

A typical micropile detail is presented in Fig. 15. An axial compression load of 2435 kN was designed for each micropile of 225 mm diameter, reinforced with a 168.3 ×

11 steel CHS section to ASTM A53M and centrally placed 75 mm diameter high-tensile steel bar. The micropile was infilled with grout of 65 MPa. As requested by RMS Bridge Branch each micropile was anchored within the footing and the CHS section was extended a minimum 1000 mm socket into Class III rock. The high-tensile bar was extended from the toe of the socket into the pad foundation and was terminated within the pad foundation with a cast-in anchor plate detail to provide full development of the bar.

The combination of CHS and high-tensile bar reinforcement contributed to the micropile capacity in axial compression. The high-tensile bar alone was considered in shear and tension capacity. The CHS section enhanced the capacity to cater for induced bending moments resulting from the applied loads and vertical tolerances in accordance with AS2159.

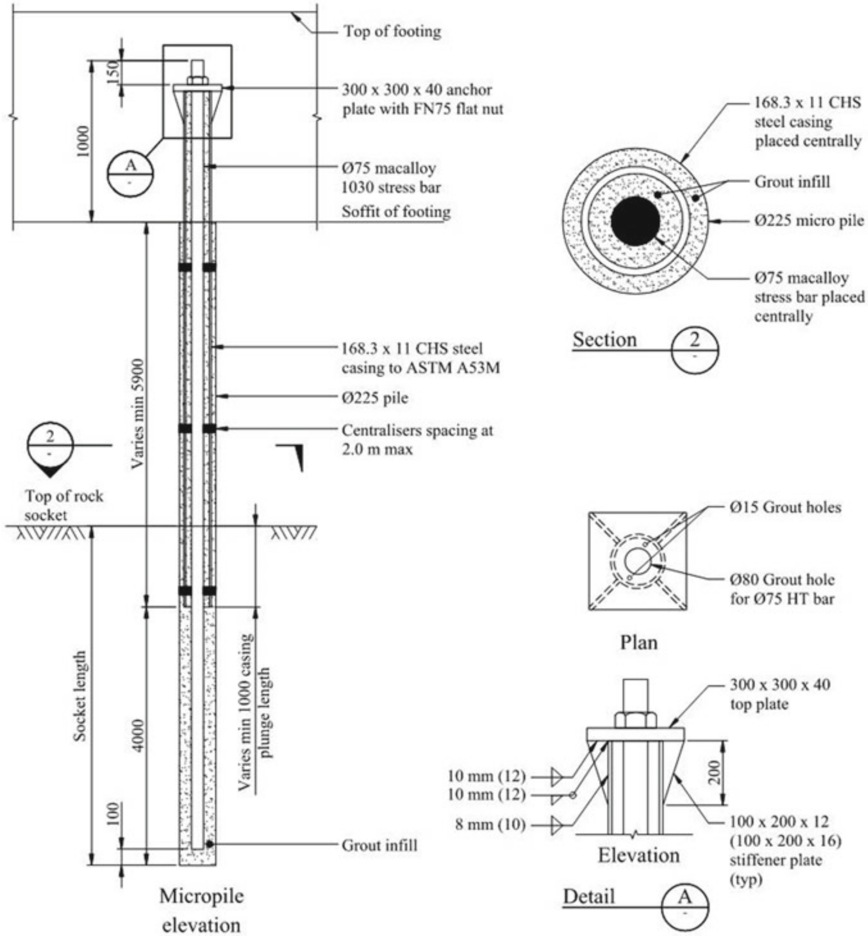


Fig. 15 Typical details of a micropile

The full CHS section was utilised for the critical short-term construction load cases. A corrosion allowance of 0.02 mm/year for the CHS as per AS2159 Table 6.5.3 (mild exposure) was considered in determining the capacity for longer term loading though limited information for the site indicating non-aggressive conditions. The high-tensile bar is adequately protected by the steel CHS and surrounding grout/concrete cover.

4.3.7 Construction Stage Verification

When the excavation of pad footing was carried out down to RL 849 m the exposed rock surface was assessed by an engineering geologist prior to pour of concrete. Figure 16 shows the approximate extent of mapped classes of rock mass at the founding level beneath the pad footing at Pier 2.

Fig. 16 Exposed rock at elevation of RL 849 m at Pier 2

4.4 Abutment B and Approach Retaining Wall

Abutment A and approach retaining wall were constructed as the original design, with some expected variation of the ground profile and support for the retaining wall.

Abutment B and approaches were originally designed using the same precast blocks and concrete infill for the lower part of retaining wall. However, the substrata profile for was found to vary more than was anticipated from the design investigations, and so after an optioneering and constructability exercise with the Principal and Contractor, the design was changed to a reinforced embankment and piled abutment foundation with the following features:

- Reduction in excavation of the existing road embankment and around abutment B; generally limited to removal of loose material and boulder removal on the existing face;
- Reduction in temporary works—soil nails and shotcrete—however some permanent soil nails required for slope stability;
- Reduction in concrete volume by removing the mass concrete foundation to the retaining walls and abutment, although somewhat offset by the mass concrete keyed toe for the new reinforced slope embankment;
- Reducing the overall height of the in-situ concrete retaining wall supporting the traffic barrier; and
- Reduction in retained height at Abutment B.

The alternative design introduced the requirements for an additional volume of fill material and geotextile reinforcement and relied upon the feasibility of constructing bored piles to support the abutment, which was confirmed by the Contractor. The piling rig was able to access the abutment location from the existing road using the partially completed reinforced embankment. Steel casings were required to retain the newly placed fill and existing road embankment (uncontrolled) fill and were pre-bored then driven to a stable rock layer.

4.4.1 Site Investigation

There were limited geotechnical investigation data available at the detailed design development stage due to access issues. As such additional boreholes were mandated as part of the construction stage requirement. Figure 17 shows borehole and test pits completed at concept design, detail design and construction stages, including boreholes along the edge of the existing highway, borehole near Abutment B, boreholes along the retaining wall and test pits bear the toe of the embankment. The available information confirmed the high level of variability of the inferred top of Class V and Class III rock levels. It is worth noting that the required boreholes for the piled abutment could not be acquired due to access issues until the reinforced earth embankment was constructed to around RL845 m.

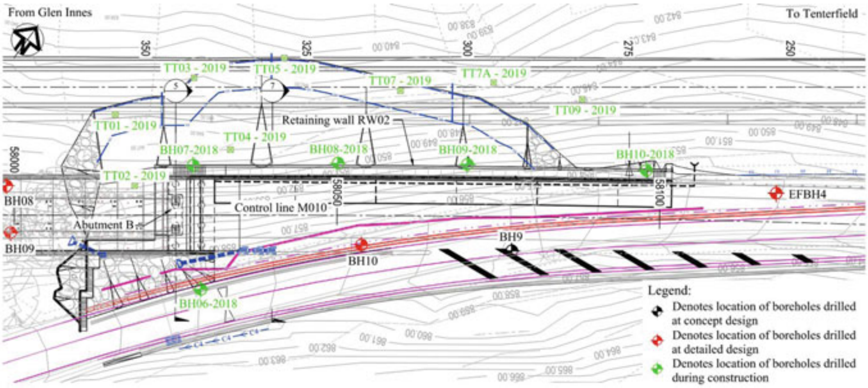


Fig. 17 Layout plan of boreholes drilled at various stages (without showing boreholes at abutment)

4.4.2 Abutment B and Slope Design

The modified design of Abutment B and approach retaining wall avoided significant additional temporary soil nail works and mass concrete foundations, due to deeper than expected rock levels.

It was proposed that the reinforced slope would be constructed bottom up to achieve a level at the abutment of RL 854.5 m. Access for piling works would then be from the north along the partially complete reinforced slope. The design of the reinforced slope needed to consider the temporary loads associated with the installation of the large diameter cast-in-place (CIP) piles.

4.4.3 Geotechnical Design and Analysis

The design of bored pile foundations followed the design assumptions and methodologies as described below:

- The serviceability assessment of the pile load capacity and displacement was carried out based on the SLS and the relevant parameters in Table 4.
- Geotechnical strength reduction factor of 0.4 was applied to the ultimate geotechnical capacity based on AS2159-2009.
- The ultimate geotechnical strength to resist the applied compression loading was calculated in accordance with AS5100.3 and AS2159 (2009) based on the design parameters derived at the original design.
- Shaft adhesion and end bearing parameters assumed a clean rock shaft socket and that the base of the pile will be cleaned using mechanical and/or air lift techniques prior to pour of concrete.
- For tension checking, shaft adhesion resistance together with rock cone/wedge was considered.

- Adequacy of piles was checked against lateral loads (i.e. ULS shear force and bending moment) at the top of rock socket based on using the methodology described in Hong Kong GEOGUIDE 1 [13].

4.4.4 Abut B Piled Foundation Design Summary

Eight (8) 1050 mm piles were designed for the Abutment B with each pile socket in Class III granite or better to achieve the pile SLS and ULS capacity. These piles varied in length from 8 to 13 m to accommodate the interpolated sloping rock profile. A pile socket of 3 m into Class III rock was required.

4.4.5 Reinforced Slope Design Outcome

The reinforced slope design was to provide piling plant access and platform to the top of pile level and subsequently to the final subgrade level after piling works. A typical cross-section of the slope is shown in Fig. 18, with a cross-section through the abutment B being presented in Fig. 19. A minimum width of 4 m was nominated for the high strength geotextile reinforcement to not only provide sufficient length for embankment stability but also for the trafficability of the proposed plant by the contractor.

The bearing capacity of existing fill was assessed inadequate for the new reinforced embankment of higher than 20 m. Therefore, removal of existing fill and replacement by rock fill keyed into Class IV/V or better rock near the toe of the reinforced embankment was proposed in this design to form a berm key. A layer of separation

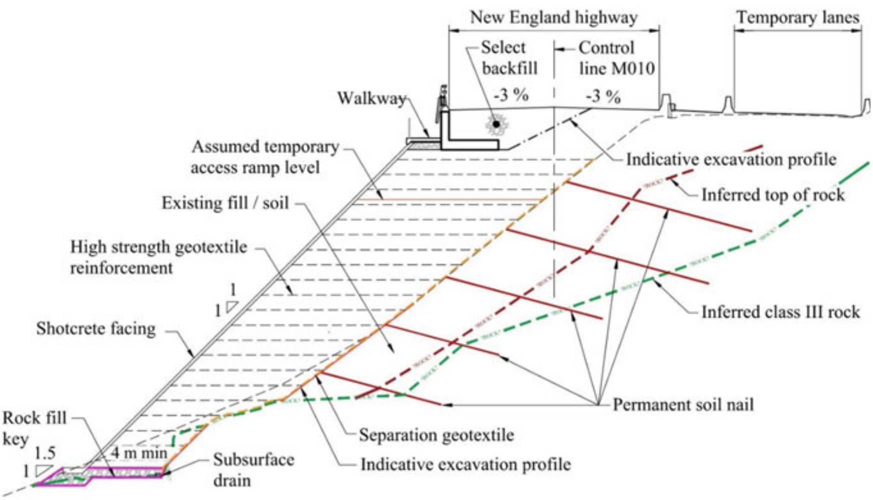


Fig. 18 Typical cross-section of reinforced embankment with permanent soil nails

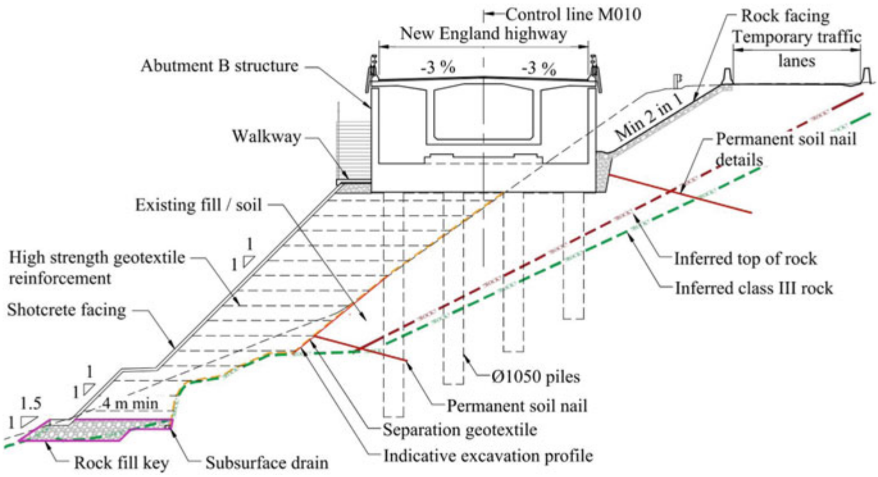


Fig. 19 Typical cross-section of reinforced embankment through piled Abutment B

geotextile was used at the interfaces with the existing soils and the new fills. This was to provide the stability of the embankment and avoidance of water pressure built-up.

A conventional slope of engineered fill at a gradient of 1 vertical to 1 horizontal would have been too steep to achieve the required factor of safety. Therefore, geosynthetic reinforcement straps were incorporated to enhance the internal stability of the engineered fill block based on BS8006-1995 [7]. The reinforcement straps were high strength geosynthetic reinforcement (minimum 400 kN/m at 5% long term strain) across the 1:1 slope fill block, which varied in length from 4 to 10 m.

Permanent soil nails were required to provide resistance against the potential slope instability planes through the existing uncontrolled fill. The slope stability assessment results indicated that 3–4 rows of permanent soil nails would be required to achieve the minimum factor of safety of 1.5 for the long-term condition. Some of these permanent soil nails doubled as temporary support for the excavation near the toe of the embankment. The proposed permanent soil nails were to be Ø40 Grade 500N reinforcement bar in accordance with AS 4671 in 150 mm diameter drill hole with a typical embedded length of approximately 4.5–6.5 m into Class IV/V rock or 3 m into Class III rock. The total soil nail length was expected to range from 8 to 15 m, subject to site validation testing.

As these soil nails were to be drilled through the uncontrolled fill, including boulders and cobbles, the Contractor was required to consider temporary/permanent casing to prevent drill hole collapse.

Instead of reinforced shotcrete face for the soil nail, it was proposed to have individual reinforced concrete (either Grade 40 concrete or sprayed concrete) soil nail head with the size of 400 mm × 400 mm and minimum 250 mm thick to be embedded in the slope.

To prevent potential weak slip plane between the new engineered fill and the existing uncontrolled fill, the interface treatment included removal of existing floating boulders and any loose material on the existing slope of the order of 0.6–1.2 m diameter. In addition, localized benching was formed as per RMS R44 [17] to avoid formation of a weak interface between the new and existing fills.

A subsurface drainage layer wrapped in geotextile layer as per R44 was provided between the existing and new batter interface to ensure minimal water pressure built up behind the wall. Furthermore, a plastic drainage pipe in accordance with RMS specification 3552 [18] was also included within the drainage layer to ensure adequate discharge of seepage water from the upper slope fill to the dedicated outlet point.

Comprehensive monitoring during construction was undertaken to validate design assumptions, determine additional drainage if seepage observed during the excavation of the existing slope, and implement contingency plan if movements beyond predicted values were encountered.

For a typical section of the RW02 wall, as shown in Fig. 17, there was enough space for the proposed retaining wall founding on the existing slope without the reinforced embankment. The global stability against the additional loading from the retaining wall and traffic surcharge was enhanced by inclusion of the permanent soil nails after removal of loose boulder on the existing slope face. The bearing capacity of the existing fill was improved by over-excavation of existing fill replaced by a reinforced compacted new fill. Erosion control measures was considered for the final slope surface, such as native grass mix plus compost blanket.

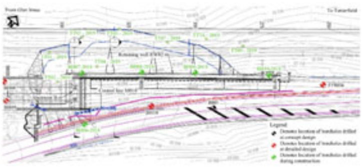
5 Construction Stage Photos and Lessons Learnt

Some of construction stage photos and lessons learnt from the process of design and construction of this project may be summarised in the Table 5.

6 Conclusions

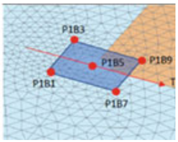
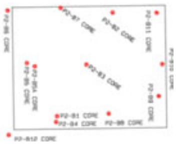
The paper has described a case study of the design of an in-situ concrete box girder bridge of approximately 320 m by balanced cantilever construction, with the middle span of 150 m, in hilly granite formation. Several technical challenges including re-design of the Pier 2 footing using micro-piles and Abutment B using bored piles together with a geotextile reinforced embankment as piling access track and platform. Lessons learnt from this project have also been presented in this paper, which could be useful for future projects. Out of these lessons learnt the wholistic design and construction approach was the key to project success, which requires input from all the parties involved. The performance monitoring to date indicated that the bridge performance is satisfactory, and it has been open to traffic in September 2021.

Table 5 Construction stage photos and lessons learnt

ID	Photo	Activity description	Lessons learnt
1		Steep terrain, the environmental constraints (Bolivia Wattle), access, and constructability drove the adopted bridge option and construction method	An integrated approach to design and construction is critical given the uncertainties of temporary works and method of construction
2	 	Rock fall risk assessment and stabilisation measures for assessed high ARL slopes using RMS slope risk assessment guidelines	Assessment guideline is subjective and can result in quite different recommendations depending upon the assessor. Safety of workers during construction requires a higher level of mitigation
3		Steep terrain and difficulty access constraints for site investigation data collection and variability of ground conditions	Appreciation of the potential high risks of variable ground condition may lead to change of design concept
4	 	Use of rock fall fence for the unstable rock blocks, the shotcrete for highly weathered rock and drape mesh for “blocky rock” in the new cuts	Variation of the rock quality in a 3D space is inevitable. Consideration of exposed rock cut face should be given in design
5	 	Use of precast block works as temporary form together with concrete infill to construct retaining wall in steep terrain	Adequate geotechnical site investigation data must be obtained to confirm the rock levels and depth of excavation in steep terrain

(continued)

Table 5 (continued)

ID	Photo	Activity description	Lessons learnt
6	 	Piling rig for 1050 mm diameter pile through fill and granite of highly variable strength and levels of Class III rock	Selection of appropriate type of piling rig, cutting heads and cleaning bucket, and provision of casing through fill to ensure production
7	 	Temporary support for excavation at pier locations for footings with temporary retaining wall works for the access road	Access difficulties -key to constructability and temporary works design for optimisation
8	 	Pier 1 interpretation and actual ground inspection and assessment	The weathering and rock quality change with depth was not highly variable within Pier 1 with only localized removal and replacement by concrete
9	 	2 boreholes available at detail design but the contact interface between granite and rhyodacite unable to be delineated until excavation to pad footing level	Granite and rhyodacite interface delineated with 10 additional boreholes and the very low strength of rhyodacite up to 9 m at Pier 2 unexpected, leading to ground improvement
10	 	Geosynthetics reinforced soil slope with shotcrete surface protection achieved 1 in 1 embankment batter without removal of existing fill and made piling platform feasible	Uncertainties of the existing fill quality and presence of weak rock required permanent soil nails and rock fill toe key and drainage to achieve the long-term stability of the embankment

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Shear Strength Testing for Geotechnical Structures

Rahsn Witt

Abstract As geotechnical engineers, we often poke our finger into soil or hit rocks with a hammer to provide information used to select material properties required for design. The observations made while visiting the site to use the finger or hammer provide valuable additional information about the site, including the setting, materials and geometry. In some cases the inspection and finger are sufficient in other cases, a combination of approaches, including laboratory testing should be undertaken to build confidence in the material properties adopted in our designs. As a designer the more information we have about our problem, including the material properties, the more efficiently and safely we can design our structures. Geotechnical engineers have a good practice of providing detailed specifications for placement of fill and although there is generally good control on fill placement there is often no specification for the shear strength limits or shear strength testing frequency. This paper provides some results of shear strength testing of fill material derived from sandstone tunnel spoil used for temporary retaining structures and working platforms on a recent infrastructure project. It has been asserted that the extent of testing should be linked to the sensitivity to material properties, the complexity, size and cost of the structure being designed.

Keywords Shear strength testing · Sandstone · Fill

1 Introduction

Geotechnical structures range in complexity from very simple to exceedingly complex. As engineers we have an important role in ensuring that our designs are fit for purpose. We must strike a balance between the cost of the design and the efficiency of the design.

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The fundamental concepts of shear strength of soils are well established by Coulomb in 1773 and Terzaghi in 1925, including the significance of water content by Hvorslev in 1937, and the pore water pressure by Terzaghi in 1938. In 1934 Jurgenson made one of the first descriptions of triaxial apparatus and the triaxial shear strength test. In a collaboration between, Harvard University, Massachusetts Institute of Technology, and Waterways Experiment Station, triaxial shear strength test apparatus was developed, together with a technique for testing and a method for interpretation of results by Bjerrum in 1954 [4]. The triaxial test provided a standardised procedure that does not suffer the fundamental shortcomings of the original direct shear test apparatus.

The fundamental concepts of shear strength of soils, including appropriate testing methods are well established. However, on many modern projects the benefit of this knowledge is ignored. The benefit of reducing uncertainty and associated risk, and increasing the efficiency of designs are sacrificed for perceived cost savings associated with forgoing appropriate material testing. There is a significant opportunity to optimise geotechnical designs and to reduce the risks associated with geotechnical designs if material properties are quantified through appropriate testing.

A significant volume of fill material is currently being generated from tunnelling activities in Sydney. However, the author is not aware of proper published test results for the shear strength of these materials. Given the particle sizes of the material being generated from road headers is generally 75 mm or smaller, the most appropriate test would be a large scale triaxial equipment with a diameter of 300 mm.

2 Material Properties

2.1 Generic Material Properties

A large proportion of geotechnical structures are constructed from site won fill. In most cases no laboratory testing is undertaken to confirm the shear strength of these materials. The material properties used in the design of geotechnical structures are often based on "experience". In practice, this means that the same properties are used for many designs in many settings with significantly different materials. The material properties sometimes come from textbooks or the memories of someone with many years of relevant experience. Table 1 below is similar to a table of material properties provided in a recent large infrastructure project conducted in Sydney in 2019.

The material properties for fill, provided in the report were not based on any laboratory test measurements of the strength or stiffness. The geotechnical parameters for the fill are informed by the in-situ tests (which are generally inappropriate for coarse granular fill) and experience with similar materials.

The properties were widely used for designing structures across the project. Designers using these properties often pick the lower bound, without any interest in

Table 1 Typical geotechnical parameters for design of retaining structures

Geotechnical unit	Total unit weight (kN/m^3)	Undrained cohesion (kPa)		Effective cohesion (kPa)	Effective friction angle (°)
Uncontrolled cohesive fill	16–20	Soft	12–25	0–5	20–30
		Firm	25–50		
		Stiff–very stiff	50–200		
Engineered cohesive fill	18–20	50–200		0–5	20–30
Uncontrolled granular fill medium dense	18–20	N/A		0–1	35–45
Engineered granular fill medium dense	18–20	N/A		0–1	35–45

justifying the adoption of higher values. In many cases, designers adopted a friction angle of 30°, irrespective of the material being retained.

2.2 Site Specific Material Properties

Material properties can be determined by various tests. Some methods commonly adopted to determine the material properties include:

- Pocket Penetrometer
- Dynamic Cone Penetrometer (DCP)
- Standard Penetrometer Test (SPT)
- Cone Penetrometer Test (CPT)
- Plate load test
- Vane shear test
- Observation and back analysis
- Laboratory testing

For a given case, one or a number of these methods in combination may be appropriate; however, in many cases basic field tests provide no useful information.

For example, many designers request Dynamic Cone Penetrometer (DCP) testing in rocky fill such as ripped sandstone. Unfortunately, penetrometer tests commonly refuse on individual particles within rocky fill and provide no useful information about the relative density or shear strength of the ground.

Table 2 Shear strength test apparatus

Test apparatus	Maximum particle size (mm)
Direct shear 100 × 100 × 50	10
Direct shear 300 × 300 × 100	20
Triaxial/UCT 50 mm diameter	10
Triaxial/UCT 100 mm diameter	20

3 Laboratory Testing

3.1 Common Laboratory Test Methods

In situations where there is a potential benefit of knowing the material strength, and it has been established that the available field tests are not appropriate, then laboratory testing may be appropriate. Testing for geotechnical applications can be time consuming and expensive. There are difficulties in sample collection and scale effects.

However for proposed new fill materials the sampling difficulties are generally not significant. The scale effects can be significant when the test apparatus prohibits the use of particle sizes that significantly characterise the material properties. For commonly available test apparatus this means that fill material must be screened to exclude all particles larger than 10 or 20 mm.

The direct shear and triaxial testing are the most common laboratory tests used to determine the shear strength of soils. Commonly available sizes are tabulated below, Table 2, together with the corresponding maximum accepted particle sizes of one fifth the minimum sample dimension. The unconfined compression strength test (UCT) is not undertaken regularly in geotechnical engineering, however, it can provide a fast method of obtaining undrained shear strength.

3.2 Unconfined Compression Strength Testing

The unconfined compressive strength test provides a method of obtaining undrained cohesion of soil. The benefit of this test is that sample preparation is fast and the apparatus is relatively simple. The test results include the effects of suction which may or may not be desired.

3.3 Direct Shear Test

Although the direct shear test is relatively fast and simple, the shortcomings are significant, the failure plane is defined, the stress on this plane are not uniform, and pore pressure cannot be measures. The direct shear test may be considered appropriate in isotropic homogenous sand soils.

3.4 Triaxial Test

The advantage of the triaxial test include, being able to control the drainage and the ability to measure the pore pressure. The ability to control and measure pore pressure allow the accurate determination of effective shear strength properties, essential in the assessment of strength with consideration of effective stress, [3].

3.5 Large Scale Triaxial Test

Scale effects can be significant in the testing of geotechnical materials and it is important that the test apparatus allows for testing of a representative range of particle sizes. Ideally the range of particle sizes would be close to the range of particle sizes being used on site.

A large proportion of the geotechnical structures being constructed in Sydney are being built from ripped sandstone generated from road header tunnelling activities. The particles generated from the road headers tend to be smaller than 75 mm.

A 300-mm diameter triaxial cell has been used by the author on a number of recent projects. This apparatus requires a small fraction of the road header spoil to be removed prior to testing to achieve samples containing material with particles smaller than 60 mm. Consequently, the test results provide a representative characterisation of the shear strength of these materials. Multi-stage, consolidated drained tests have been conducted on sandstone tunnel spoil over that past two years allowing more efficient geotechnical designs. The materials were tested following ASTM D7181-20 [2].

Some of the materials that were tested in the triaxial apparatus were also tested for unconfined compressive strength, following AS 5101.4:2008 [1].

The general arrangement of the triaxial test apparatus is shown in Fig. 1. A sample of the shear results is presented in Fig. 2, and the corresponding stress strain curve is shown in Fig. 3. The effective cohesion of 12 kPa and the effective friction angle of 36º have been determined. Figure 4 and Table 3 show a summary of the results from eleven tests conducted on ripped sandstone.

4 Know Your Materials

Results of testing on recent projects have allowed the author to utilise simple retaining structures where geometric constraints would have precluded such structures if the project generic properties had been adopted. Aside from an increase in efficiency of

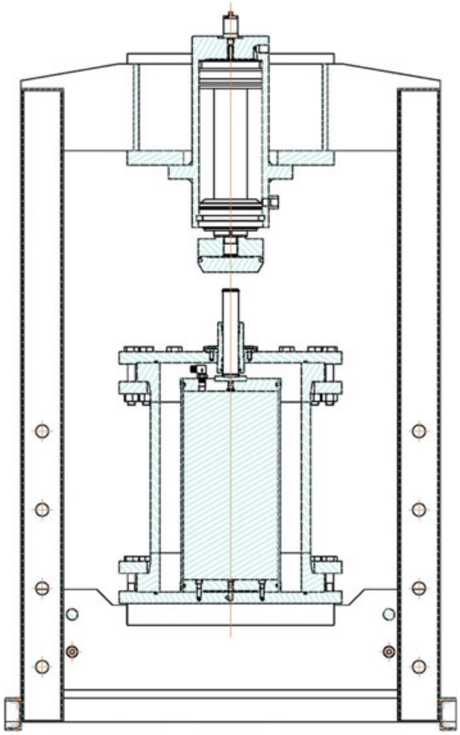

Fig. 1 A schematic large scale triaxial test arrangement

the design of a particular structure that comes a knowledge of material strength of more interest is when alternative types of structures become available, or in some cases, no structure may be required at all.

On one project that the author has been involved in, knowledge of the effective friction angle and the effective cohesion of a particular material would have allowed a two million dollar temporary piled retaining wall to be replaced with a cut batter. Establishing appropriate material properties could have been achieved by any one or a combination of plate loading, back analysis of temporary cuttings or laboratory testing. Acceptance of the material properties could only be achieved through an open mind of the client and the design/reviewer, in the knowledge that the generic material properties for the project could be refined in the light of new information.

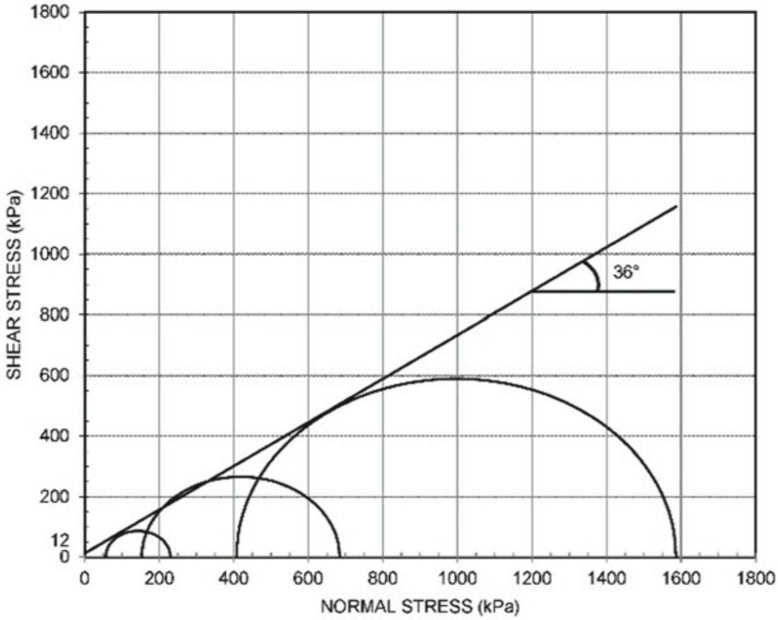

Fig. 2 Shear strength results from the large scale triaxial test using Mohr circles

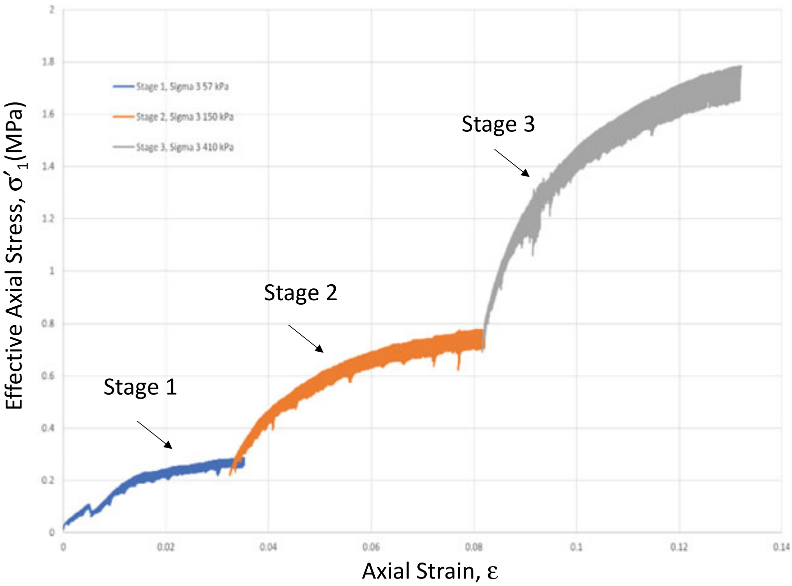

Fig. 3 Stress strain results from the large scale triaxial test

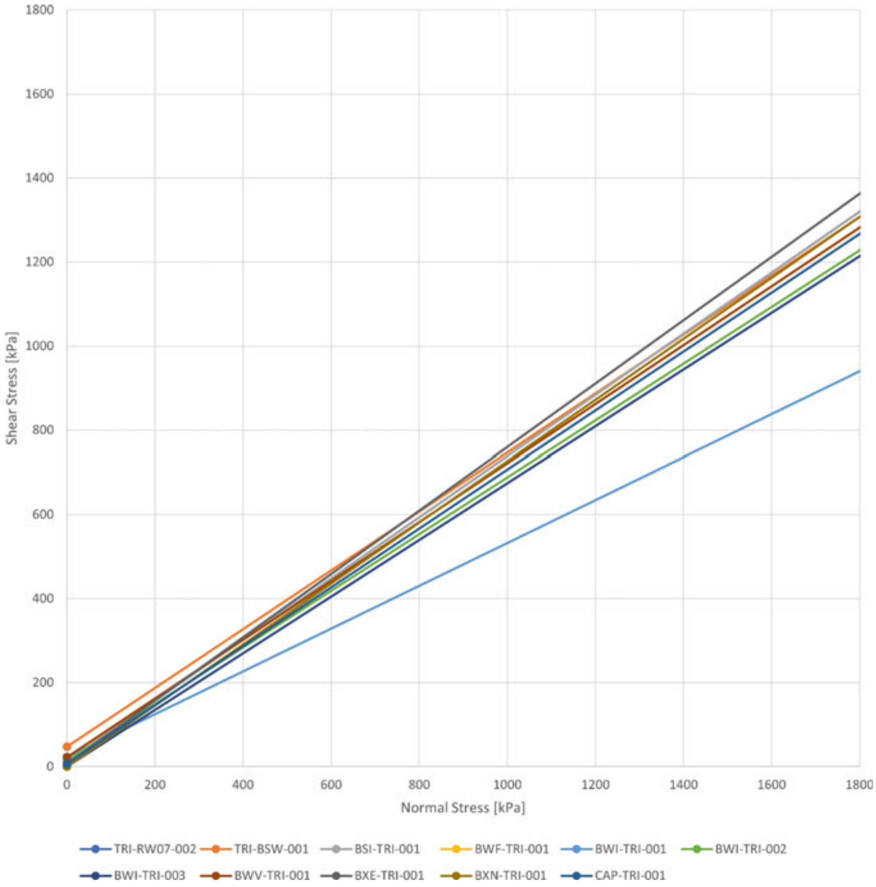

Fig. 4 Summary of shear strength test results of sandstone tunnel spoil

Table 3 Summary of eleven shear strength test results of sandstone tunnel spoil

Geotechnical unit	Total unit weight (kN/m^3)	Undrained cohesion (kPa)	Effective cohesion (kPa)	Effective friction angle (°)
Ripped Sandstone Fill	20–21	28–67*	0–47	34–37°

* Undrained cohesion was only tested on five samples

5 Conclusions

The main conclusions of this paper are as follows:

- The design of soil supporting structures forms a large portion of the works scope on many large infrastructure projects. These structures are often designed without verification of the soil strength that governs the design of the structure.
- More knowledge associated with the materials that surround the civil structures forming part of our infrastructures will facilitate to improve the efficiency of our designs.
- The level of testing should be appropriate with the sensitivity to material properties, the complexity, size and cost of the structure being designed.
- There is a potential for designers to expose themselves to unnecessary risks or their clients to preventable costs if the material properties used in the design have not been accurately characterised.

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