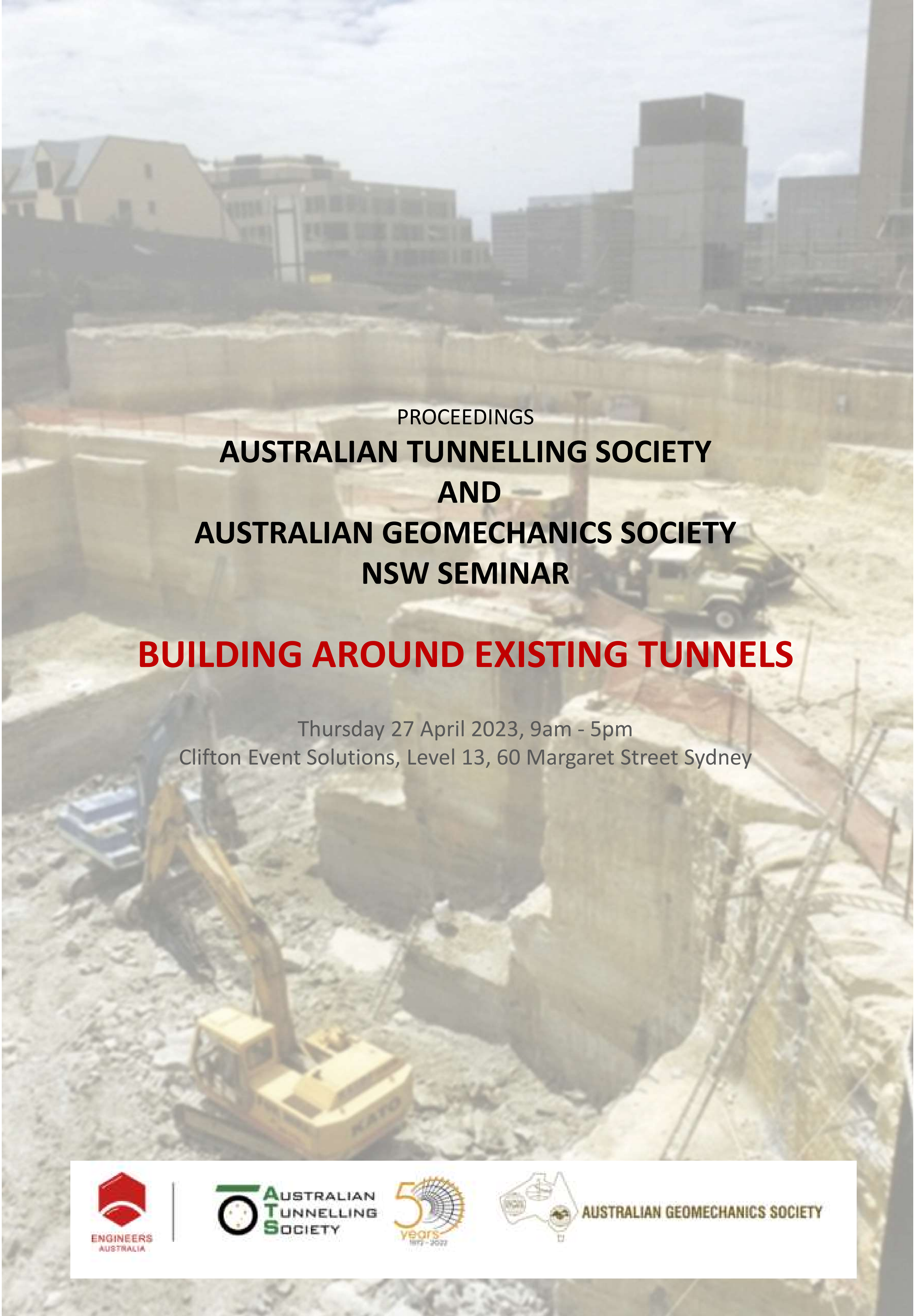

PROCEEDINGS

AUSTRALIAN TUNNELLING SOCIETY
AND
AUSTRALIAN GEOMECHANICS SOCIETY
NSW SEMINAR

BUILDING AROUND EXISTING TUNNELS

Thursday 27 April 2023, 9am - 5pm
Clifton Event Solutions, Level 13, 60 Margaret Street Sydney

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Front cover photo: Shangri-La Hotel, The Rocks, Sydney, 1988
City Circle tunnel – Circular Quay to Wynyard Station

CONTENTS

Design of the Cross River Rail station caverns for future over tunnel development

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ABSTRACT

Four new underground railway stations are being constructed in Brisbane by a joint venture between CPB Contractors, BAM International Australia, Ghella and UGL (CBGU JV) as part of the Cross River Rail Tunnel, Stations and Development (TSD) package. The project extends beneath the Brisbane River and CBD, and includes underground stations at Roma Street, Albert Street, Woolloongabba, and Boggo Road.

The design of the permanent lining required consideration of the influence of future over tunnel developments. The requirements were provided in the Project Scope and Technical Requirements (PSTR) and included a range of excavation geometry and building load scenarios to be considered on portions of the alignment which passed beneath or adjacent to developable land. The development scenarios included excavation exclusion zones either side of and above the tunnels.

Station-specific requirements also needed to be considered for development proposals where preliminary details were available.

Design of the permanent linings for the various excavation and loading scenarios were undertaken with the standard approaches to geotechnical ground-structure interaction modelling and structural analysis for the tunnel linings. In addition to the scenarios involving loads and excavations, the client also required the cast in situ permanent concrete linings to be designed for additional distortion (i.e. ovalisation).

The paper summarises the design approaches required to address the various future development requirements of the project and concludes that consideration of these impacts at the design stage is a simpler and more efficient means of facilitating development around metro stations compared to assessing redevelopment proposals as they arise.

PROJECT DESCRIPTION

Tunnel, Stations and Development works

Cross River Rail (CRR) is a new 10.2km long metro rail line in Brisbane between Dutton Park in the south and Bowen Hills to the north, which includes 5.9km long twin tunnels below the Brisbane River and CBD. The TSD component of the project includes construction of twin Tunnel Boring Machine (TBM) excavated tunnels and mined running tunnels; new underground stations at Boggo Road, Woolloongabba, Albert Street and Roma Street; and dive structures at each end of the alignment.

Each station includes a cavern of up to 290m length and 20.6m clear span, with associated connecting adits and station shafts. The cavern permanent lining typically comprises steel fibre-reinforced concrete (SFRC) in the crown, bar reinforced concrete in the sidewalls, and bar reinforced invert slabs.

The Pulse consortium (including the CBGU JV) was awarded the contract to design and construct the TSD works in 2019.

Figure 1. The Cross River Rail project extends beneath the Brisbane River and CBD, and includes underground stations at Roma Street, Albert Street, Woolloongabba, and Boggo Road.

Ground conditions

The four underground stations intersected a range of geological conditions, including the Neranleigh-Fernvale Group (NFG), Brisbane Tuff and Aspley Formation rock masses. Detailed descriptions of these rock mass units are presented in Cammack *et al.* (2022).

The Boggo Road cavern was excavated predominantly within the Brisbane Tuff and Aspley Formation, with a typical ground cover over the crown of only 4m.

The Woolloongabba cavern is mostly located within the Brisbane Tuff, but also has zones of the underlying Aspley Formation and NFG, with ground cover ranging from 10m to 15m.

The Albert Street and Roma Street caverns are located within the NFG rock mass, with the Roma Street cavern also intersecting the Normanby Fault Zone, with ground cover ranging from 18m to 21m.

Existing and proposed over tunnel development

The Albert Street station is located in the Brisbane CBD and is proximal to many substantial buildings and deep basements. Roma Street station is located within and beneath the existing Roma Street railway station precinct and adjacent to the Inner Northern Busway (INB). The Boggo Road station cavern was excavated beneath the existing Park Road railway station and Boggo Road busway station.

The PSTR required consideration of a range of future over tunnel development conditions in the design of the permanent linings to expedite future development above and adjacent to the station caverns, as described below.

PROJECT SCOPE AND TECHNICAL REQUIREMENTS

Overview

The PSTR required that "*the design shall allow for future development of the land above and adjacent to the Tunnel and Underground Structures by designing and constructing for loading and unloading in addition to the applicable design loads*".

As part of achieving this, the PSTR required that the permanent tunnel linings be designed to consider a range of future over tunnel development (FOTD) scenarios:

- Notional development configurations/allowances, defined in terms of excavation geometries, surface surcharge loads, and building loads. These were applied to the station caverns.
- Specific development proposals where details were available.

- Additional ‘ovalisation’ distortion to be applied to the station cavern permanent linings.

Notional requirements

Notional development configurations and allowances to be considered in the design included:

- Additional dead and live loads applied at the ground surface above the caverns to simulate future fill and/or traffic and other live loads.
- Loads applied 1m above the crown of the cavern excavations to simulate future pile and/or footing loads.
- Excavation zones above and adjacent to the tunnels to simulate future building basements.

The most adverse combination of the following conditions was to be allowed in the design of the station caverns (Figure 2):

- Uniformly Distributed Load (UDL) of 20kPa applied at the ground surface.
- Vertical load of 50kPa applied 1m above the tunnel crown.
- Excavation allowances of:
  - Up to 7m below the ground surface.
  - Residual ground cover of at least 10m above the cavern crowns.
  - Pillar width of at least 10m between the side wall of the cavern and adjacent building basement excavations.

The requirements for the running tunnels differed a little from those applicable to the caverns, though are not considered further here.

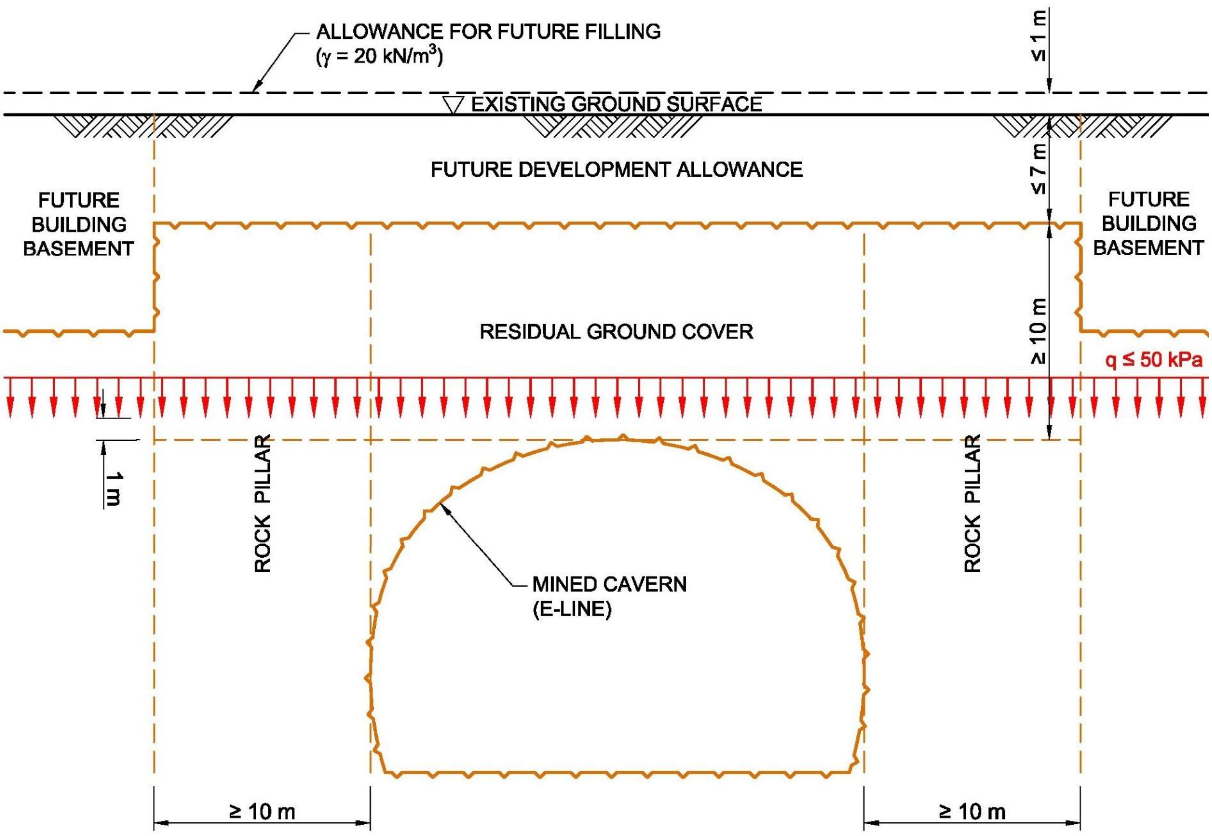

Figure 2. Generic FOTD requirements applicable to the station caverns.

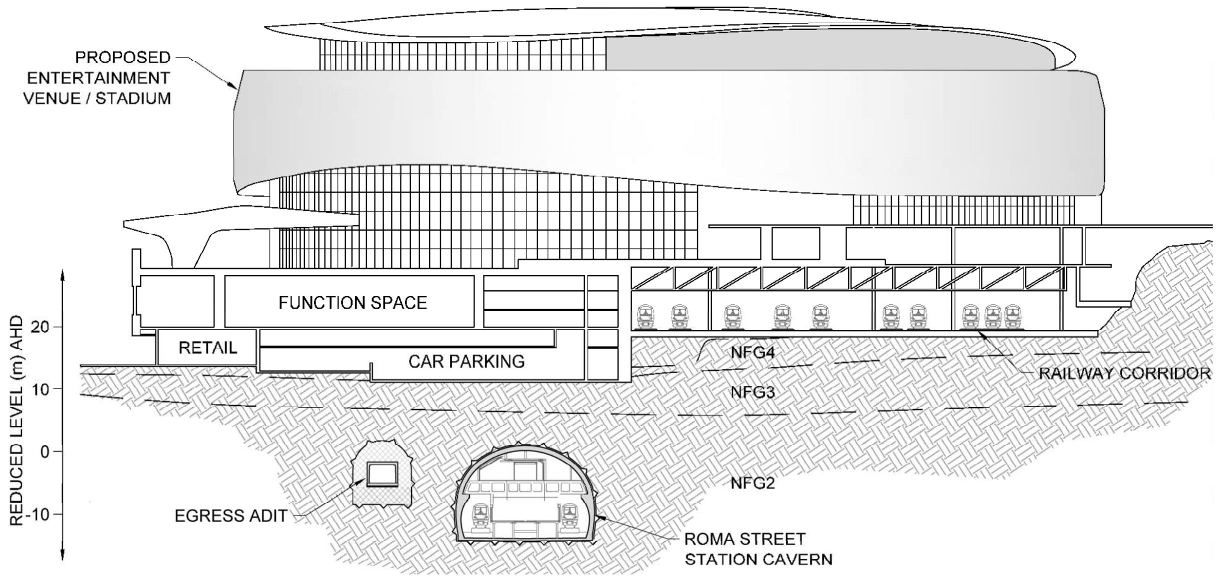

Figure 3. Concept arrangement of the proposed entertainment venue and stadium overlying Roma Street cavern.

Station-specific developments

The station-specific developments included additional allowances for basement excavations to the notional FOTD allowances as well as developments that had been approved by Brisbane City Council or the Queensland State Government.

Additional basement allowances for the Roma Street and Woolloongabba stations required allowance for future excavation to within 3m horizontally from the extrados of the cavern lining to a maximum depth of either 10m from the finished surface level or up to 10m difference in excavation level either side of the cavern.

The Lot 2 site is located immediately adjacent to the Albert Street station cavern. The site was used to provide an access shaft during construction of the underground works, with the intention that it would subsequently be redeveloped. Additional excavation and surcharge allowances were prescribed by the PSTR for this site.

Other approved future developments which needed to be considered in the tunnel design included a proposed entertainment venue and stadium to be constructed over the Roma Street station cavern. This venue is proposed to host sporting events for the 2032 Brisbane Olympic and Paralympic Games (Figure 3).

Additional ovalisation / distortion

The PSTR also required an additional ovalisation / distortion to be applied to the permanent lining (Figure 4), with the FOTD requirements comprising:

- Additional 20kPa UDL to be applied at the ground surface.
- A 75kPa live working load surcharge to be applied at the ground surface, and
- "*In addition to the deflection caused by the ground load and surcharge as appropriate, the tunnel permanent support must be designed to accommodate an additional distortion of ± 15mm on diameter to allow for future development. This shall be analysed by reducing the horizontal / vertical ground load to produce the additional distortion*".

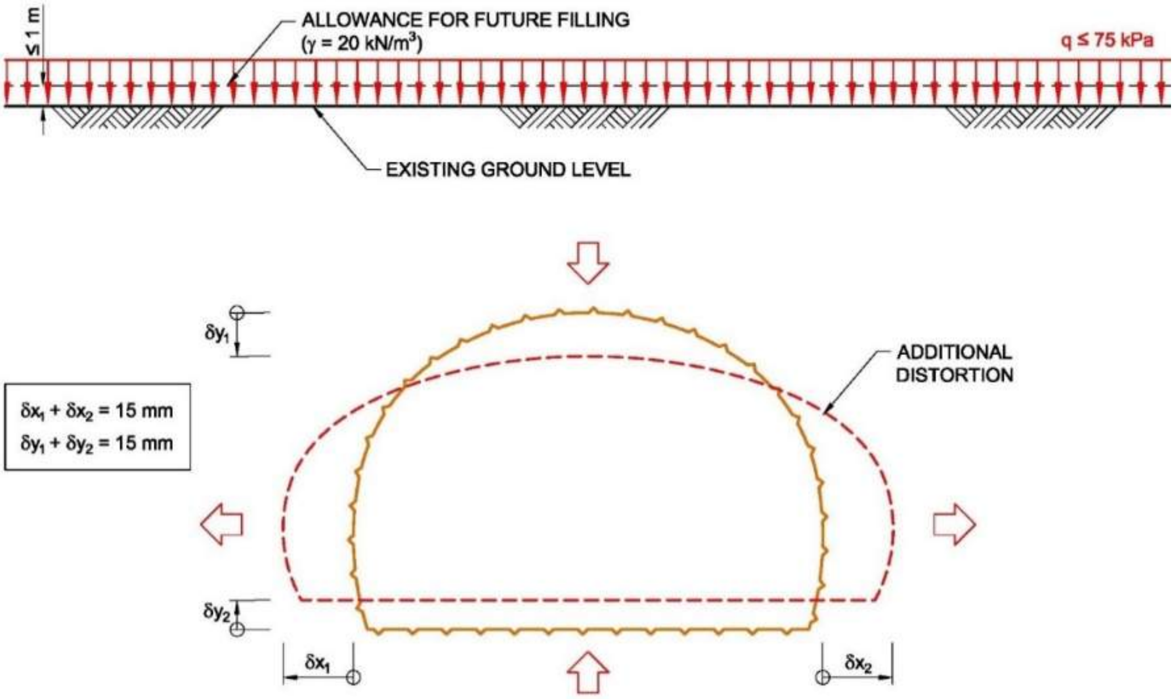

Figure 4. Initial 'distortion' FOTD scenario as described in the PSTR.

As discussed in the following sections, the ovalisation scenario has precedent for segmentally lined tunnels in soft ground and weak rock, with the additional distortion intended to simulate the loss of lateral support due to adjacent excavation. The requirement to impose these displacements by reducing the lateral support was key to ensuring that the displacement criteria matched the stiffness of the proposed structure.

OVALISATION CRITERIA IN INTERNATIONAL TUNNELLING GUIDELINES

Various international design codes and guidelines include provisions for considering ovalisation of tunnel linings as a method of accounting for potential future developments. The PSTR FOTD requirements were found to be very similar to the ovalisation (or diametrical distortion) requirements contained in the Singapore Land Transport Authority (LTA) tunnel design documents.

The following tunnel design documents include provisions for considering ovalisation of tunnel linings:

- LTA Guidelines for tunnel lining design.
- LTA Design criteria for road and rail transit systems.
- ACI Report on design and construction of fiber reinforced precast concrete tunnel segments.
- ITA Guidelines for segmental tunnel linings.
- LACMTA Metro rail design criteria.
- BSI Code of practice for design of concrete segmental tunnel linings.
- ATS Tunnel design guideline.

The ovalisation requirements in the above guidelines are limited to segmentally lined tunnels, with both the ACI Report and ITA Guidelines specifically referring to tunnels excavated in soft ground.

The design methodologies in the LTA Guidelines are applicable to circular segmental linings constructed in soft ground, earth or soft rock. Loads include future development with a 75kPa overburden pressure (catering for a development load equivalent to a five-storey building) combined with an additional diametrical distortion of ±15mm. These requirements, plus some of the load combinations, are essentially the same as the relevant CRR PSTR clauses.

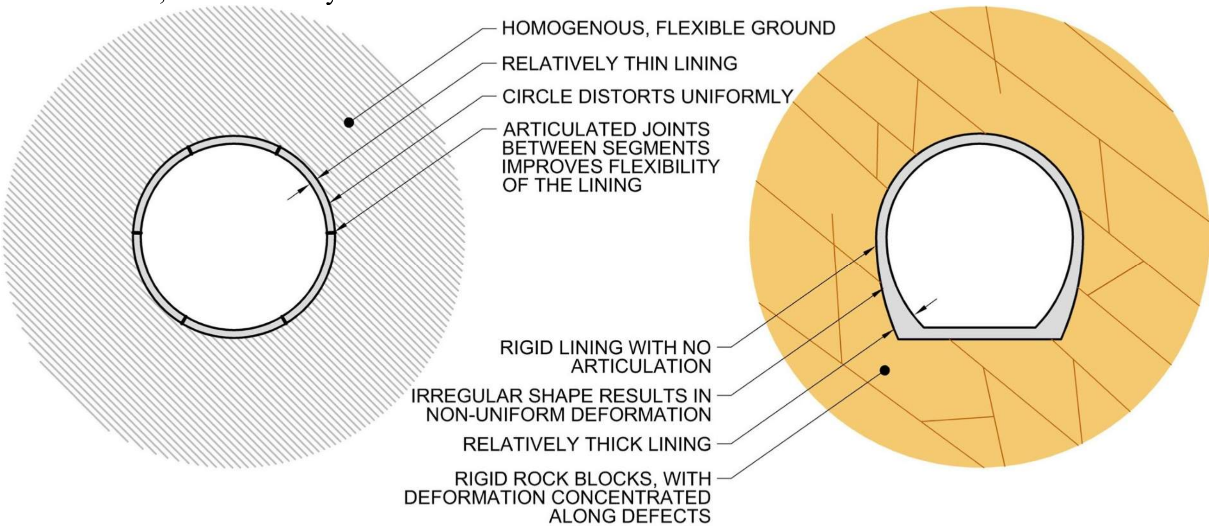

Figure 5. Comparison of stiffness characteristics between a segmentally lined circular tunnel constructed in soft ground (left) with a non-circular cast in situ concrete lined tunnel excavated in hard rock (right).

The LTA Design Criteria has similar loading requirements to those in the LTA Guideline.

The ovalisation requirements given in international design codes and guidelines, in particular those contained in the LTA Guidelines and Design Criteria, indicate that the ovalisation approach is intended for application to circular, segmentally lined tunnels constructed in soft ground. This scenario represents a much more flexible structure compared to the cast in situ concrete linings of mined tunnels excavated in hard rock (Figure 5).

The three conditions mentioned above (circular profile, segmentally lined and excavated in soft ground) do not apply to the mined tunnels of the CRR project, as none of the ~30 mined tunnel profiles adopted in the project are circular and none are segmentally lined (they employ cast in situ concrete or permanent shotcrete linings), and are excavated in hard rock, not soft soil.

Given the above, it was concluded that the ovalisation criteria adopted by the PSTR was not originally intended to be applied to the conditions of the CRR project. Details in the application of the criteria were further developed in consultation with the Client, as described below.

PERMANENT LINING DESIGN APPROACH

Ground-structure interaction analyses

The structural actions experienced by the tunnel permanent lining are a function of the lining properties, applied loads and the support conditions offered by the surrounding ground.

Figure 6 presents the results of a sensitivity analysis exploring the change in structural actions in a cavern concrete lining due to imposed horizontal displacements associated with an adjacent deep excavation. The graph shows that both the maximum axial force and maximum bending moment in the lining increase in response to displacement associated with the neighbouring excavation.

Where the support from the surrounding ground changes (for example because of nearby FOTD excavation) and/or the loads change (for example due to application of additional FOTD loads) then the structural actions experienced by the permanent lining will be affected. Therefore, it is necessary to consider the interaction between the ground and the tunnel lining for FOTD loading scenarios.

The above approach is complicated by the following factors:

- Uncertainty in the geological and geotechnical conditions of the rock mass.
- Deformation and stress changes in the ground experienced during construction of the tunnel.

- Structural design within the AS 5100 Bridge Code (as commonly mandated for Australian rail tunnels) uses a limit state design approach. For the Ultimate Limit State (ULS), if different load factors were applied to different loads, then unrealistic ground responses could occur, generating unlikely structural actions. This view is supported by Clause 2.3.3(c) of AS 5100.3 which advocates an unfactored analysis approach for ground-structure interaction problems, with the corresponding commentary (AS 5100.3 Supp1 - 2008) stating:

  "This unfactored approach to the geotechnical modelling is taken because, in the case of soil-supporting structures, both the applied loads and the resistance of the wall system are a function of the soil parameters. To take a factored approach and modify the soil parameters by a capacity reduction factor to establish a geotechnical model to determine the resistance of the system, changes the entire geotechnical model such that it no longer represents a realistic model of the actual structure. When such an approach is adopted, the geotechnical model is not realistic and the geotechnical engineer can be misled regarding the geotechnical behaviour of the structure (for example, maximum bending moments in the wall may change in position as well as quantum). Further, when the analysis is conducted using computer modelling, the design engineer is even further removed from the reality of the actual geotechnical conditions."

  The above commentary is even more relevant to tunnel design where the ground both applies loads and supports the tunnel lining.

The following approach was used when considering FOTD load cases:

- Assess the ground loads applied to the tunnel permanent lining based on a finite element model to simulate the ground-structure interaction. The model includes working / unfactored loads, a realistic geological model, conservative properties for the permanent lining, and includes the initial tunnel excavation stages and installation of ground support.
- Check the proposed permanent lining using the ground load derived from the previous step, adoption of appropriate ground support conditions, and application of limit-state design methods (e.g. factoring of the ground load).

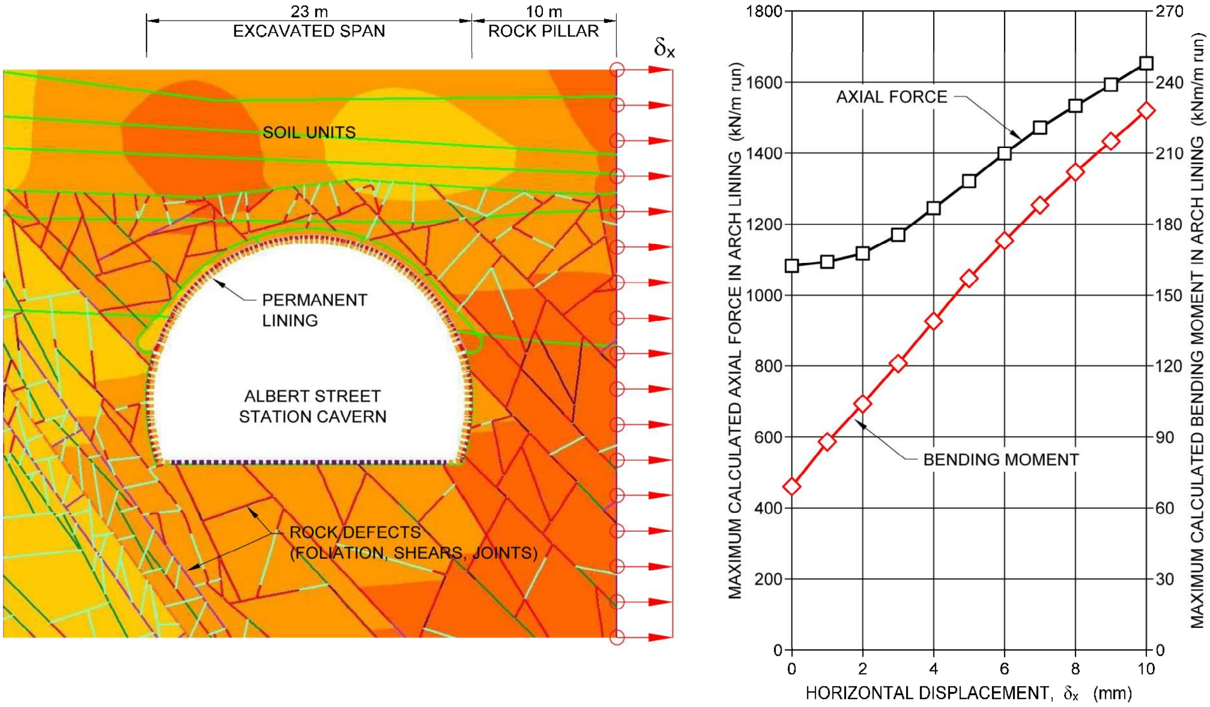

Figure 6. Example analysis showing sensitivity of structural actions in a cavern concrete lining to imposed horizontal displacements in an adjacent deep excavation.

Assessment of FOTD load cases comprises multiple ground-structure interaction analyses, focussed on assessing the governing ground load magnitudes and distributions. A key aspect is the adopted properties of the permanent lining in the ground-structure analyses, including:

- Compressibility of the waterproofing layers which form an interface between the permanent lining and the rock excavation.
- Stiffness of the permanent lining.

The crown and sidewalls of the CRR mined tunnels were encapsulated by a continuous sheet waterproofing membrane, which was protected by a thick geotextile fleece. The compressibility of the fleece can influence the deformation of the lining and degree of load transfer from the ground to the permanent lining. The following properties were adopted for the waterproofing layers when assessing ground loads:

- Elastic-perfectly plastic yield model (with Mohr-Coulomb parameters; zero cohesion and tensile strength, with a friction angle of 20°).
- Normal stiffness of 2,000MPa/m and shear stiffness of 200MPa/m.

The adopted normal stiffness is significantly higher than indicated by test results for geotextile fleece products. This approach is conservative when assessing ground loads.

Load transfer to the permanent lining is also dependent on the stiffness of the concrete lining, with higher stiffnesses attracting greater loads. For CRR, the primary support had a design life of 10 years, meaning that the ground loads would begin to be redistributed to the permanent lining after this period. During this time, the permanent lining would also experience creep (i.e. resulting in a lower effective modulus). However, for the purposes for assessing the FOTD ground loads, the benefits of creep were ignored in the analysis, and a modulus at the upper end of the plausible range was adopted.

The resulting load distributions were rationalised, with design ground loads assessed for each tunnel profile and location.

Two-dimensional (2D) finite element analyses were used for the ground-structure analyses. The model was based on that undertaken for the primary support design, with each stage of excavation and installation of the primary support simulated. The permanent lining was introduced, with the primary support then degraded to simulate the end of its design life, resulting in the transfer of the ground load to the permanent lining. Finally, the FOTD loads and excavations were applied.

The models included a joint interface, representing the waterproofing layers, between the rock mass and/or primary lining and the permanent lining elements, allowing for the estimation of ground loads via the following two approaches:

1. Averaging the normal stresses calculated along the joint interface.
2. Calculating the average normal stress applied to the lining based on the calculated axial force in the permanent lining via the hoop stress analogy.

The adopted ground loads were incorporated in the routine structural analysis of the permanent lining using established limit state design principles.

Applicability of distortion approach

The application of the distortion approach originally required by the PSTR highlighted issues associated with a purely deformation-based criteria, as such criteria are independent of the stiffness of the structure and do not necessarily represent the expected ground behaviour. These issues were more pronounced for the serviceability limit state and calculated cracks widths, with ultimate limit state (strength) considerations being less problematic.

Due to the tunnel geometry and hard rock ground conditions, it was found that in many cases the lateral support around the tunnel could be fully removed and the resulting distortion was less than the mandated 15mm on diameter.

Consequentially adjustment of the PSTR ovalisation requirement was negotiated with the Client (Figure 7), with the agreed modified criteria as follows:

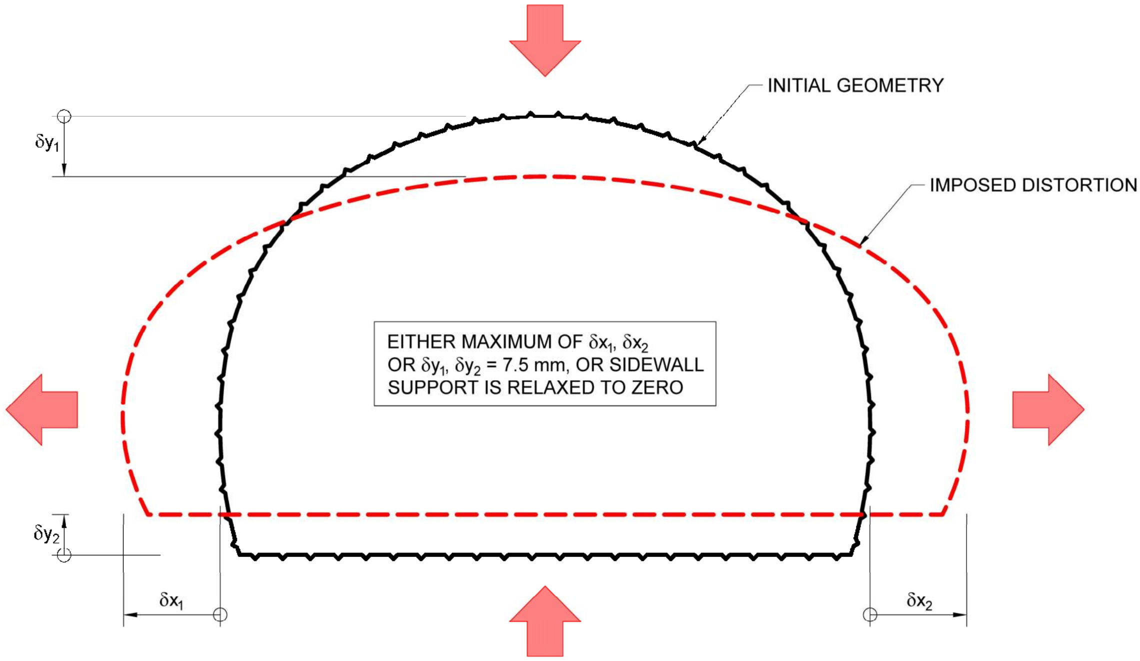

Figure 7. Modified PSTR 'distortion' FOTD scenario.

- A relaxation of the permanent lining sidewall support stiffness until an additional deformation of 7.5mm occurred in either the crown, invert or the sidewalls, or until the sidewall support stiffness had been completely relaxed.
- Adoption of a creep-adjusted concrete modulus of 12.2GPa (i.e. similar to the LTA Guideline's suggestion to adopt long-term parameters).
- Adoption of load factors of unity (i.e. consistent with the LTA Guidelines).

Notwithstanding the relaxation of the requirement, this scenario for hard rock and non-segmentally lined tunnels was without precedence, with the outcomes from this load case highly dependent on the tunnel profile and in some cases requiring thicker permanent linings.

CONCLUSIONS

The CRR project required that the design allow for future development of the land above and adjacent to the tunnels. The consideration of the impacts of future development at the design stage is a simpler and more efficient means of facilitating development around metro stations compared to assessing redevelopment proposals as they arise.

Ideally in the consideration of the impact of future development on existing tunnels, the criterion to be achieved by the development should not be excessively onerous to achieve, though this aim needs to be balanced against being overly generous to future developments resulting in the tunnel design being excessively burdened by future development considerations. The generic development requirements in the PSTR achieved this balance.

To avoid unnecessarily increasing the cost of the tunnel design and construction, it is important when considering potential future over tunnel development that this be limited to realistic scenarios and impacts. Appropriate ground structure interaction methods of assessment are described in the paper.

The ovalisation clause in the PSTR is based on overseas practice for circular, segmentally lined tunnels constructed in soft soils. These conditions are not relevant to the CRR mined tunnels which are non-circular, with cast in situ concrete linings, and excavated in hard rock conditions. The authors suggest that deformation-based criterion be specified with caution as they are blind to the stiffness of structures and may not represent the expected ground behaviour.

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Preliminary Impact Assessments for New Building Development Applications over existing Tunnels

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ABSTRACT

New building developments require development applications and approvals. Where new developments are proposed in the vicinity of existing operational tunnels then the existing tunnel asset owner may also review the new building development application for approval. New developments planned to be constructed over existing tunnels require investigation and analysis to assess the impact they may have on the existing adjacent tunnels. Potential impacts can occur during building excavation, where the ground typically moves towards the basement excavation, then as the building is constructed and occupied, it loads the ground. Both cases impact the tunnel, for example changing the compression in the gaskets in segmentally lined tunnels. Due to these potetial impacts, Future Development "Deemed to Comply Conditions" are established to limit how much buildings are able to affect the existing nearby operational tunnels.

When undertaking preliminary assessments of the potential impacts for a development application, a good understanding of the model to be used and the parametric variables and interpretation of model outcomes are required. This enables appropriate modelling of the scenario with reasonable accuracy without compromising time and cost for the development application. Investigations of various different types of finite element modelling, including loading impact calculations are underaken. Additional focus on progressive model complexity from simplicity, appropriate use of 2D and 3D modelling, and sensitivities including continuum and discontinuum modelling, and loading stress distribution are assessed. Further discussion is also provided on those allowable future development conditions which typically provide developers with guidance on what their new proposed developments can and cannot do with respect to the nearby operational tunnels.

INTRODUCTION

Urbanised cities are forging ahead with significant new underground infrastructure, including large road and rail tunnels. These new tunnel infrastructure pass beneath or beside existing properties. Any new developments proposed on these existing properties near existing underground tunnel infrastructure can have an impact on the existing underground tunnel infrastructure, both during construction and final operation of the new development.

New building developments require development applications and approvals. Where new developments are proposed in the vicinity of existing operational tunnels then the existing tunnel asset owner also reviews the new building development application and approval. New developments planned to be constructed over existing tunnels require analysis to investigate the impact they will have on the existing adjacent tunnels. Impacts occur during building excavations, where the ground moves towards the new building excavation and causes tunnel movement. Building loading during construction and operation results in additional stresses in the ground around the tunnel. Due to this, typically "Future Development Deemed to Comply Requirements" have been established to limit how much future buildings are able to affect the existing nearby operational tunnels.

During the preliminary assessments typically undertaken at the development application stage, a good understanding of the parametric variables and interpretation of model outcomes and model limitations are required to appropriately model the scenario with reasonable accuracy without compromising time and cost for the development application. This paper presents an investigation using a typical case study on what is involved in the preliminary impact assessment of building developments over existing tunnels such as deemed to comply requirements, finite element modelling, and new building loading impact calculations; with additional focus on the progressive increase of modelling complexity from simplicity, emphasising the appropriate use of 2D and 3D modelling, and sensitivities including continuum and discontinuum modelling, and loading stress distribution. The outcomes from this paper demonstrate that simple conservative assessments can be undertaken at the development application stage to provide guidance to both developers and the infrastructure asset owner/operator/maintainer for development application assessments.

FUTURE DEVELOPMENT ALLOWABLE CONDITIONS

Future developments typically consist of multi-storey high-rise buildings containing commercial/retail and/or residential premises. During the planning of major tunnels, tunnel delivery authorities typically acknowledge that future developments could occur above and adjacent to their tunnels. Accomodation for future development conditions to be imposed upon the tunnels may be allowed for by requiring the tunnel designers to allow for certain future conditions. The future building development conditions that tunnels are typically designed and constructed to withstand are as follows:

- Loading due to the self weight, live loading, wind load and seismic loading from the building distributed onto the adjacent tunnel
- Unloading due to any bulk excavation, including basements etc
- Ground movements associated with the above, in order to limit cracking of the tunnel permanent concrete lining and ensure any tunnel waterproofing measures (such as TBM segment gaskets) remain operational

These conditions that tunnels are typically designed and constructed to withstand are typically written into the design and construction performance specifications for the procurement of the tunnel infrastructure as shown in Table 1.

Table 1: Typical Future Development Allowances for Tunnel Design and Construction

Additional Loading	Continuous Excavation	Distortion
i) Building vertical loading up to "*YY*" kPa (working load) acting on the ground at a level of 1m above the tunnel crown and in uniform and patterned (including symmetric and unsymmetric) arrangements which give the most unfavourable loading condition on the tunnel; and ii) Allow for a build-up of surface level with a minimum of one metre of fill equivalent to "*ZZ*' kPa	i) Up to "*X*"m below natural surface to allow or future development; ii) With a minimum of "*X*"m residual ground cover above the tunnel crown; and iii) With a minimum "*X*"m pillar with between the side wall of the tunnel and any adjacent building basement excavation	i) The tunnel permanent support must be designed to accommodate an additional distortion of +/- "*AA*"mm on tunnel diameter to allow for future development

Note: The design must allow for the additional loadings and continuous excavation to be applied separately and together, including asymmetrical arrangements, and in any order to give the most unfavourable loading condition on the tunnel

YY, ZZ, X and *AA* are all numbers that are explicitly defined by each large delivery authority and may vary from project to project. Indicative allowable conditions are shown in Figure 1 and Figure 2.

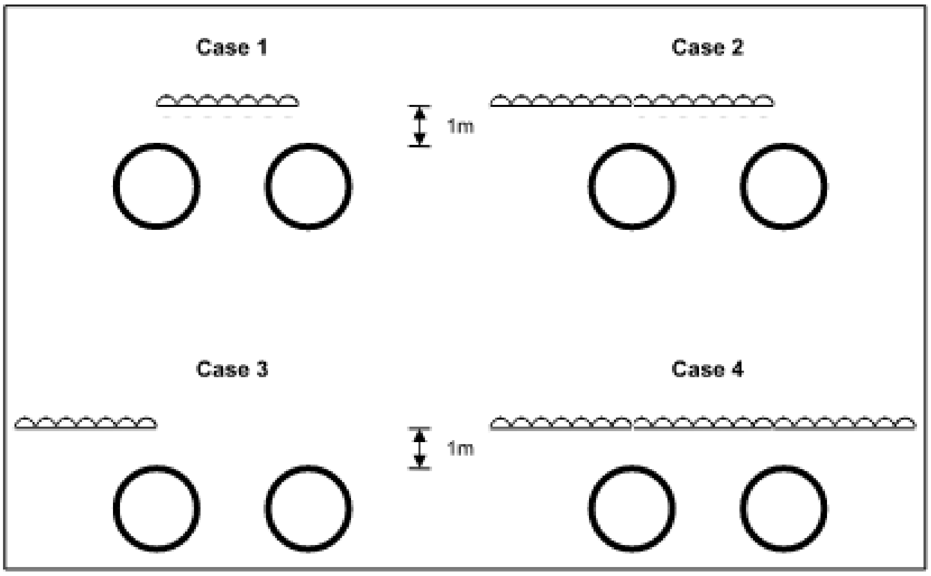

Figure 1: Typical Allowable Additional Loading from Future Developments

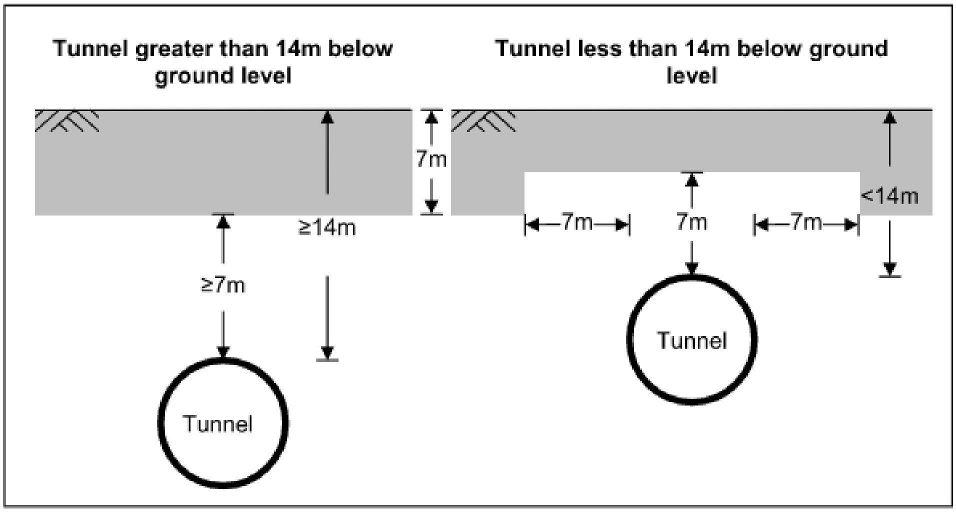

Figure 2: Typical Allowable Continuous Excavations from Future Developments

In addition any construction vibration caused by the future development would need to be limited and monitored so as to not adversely impact on the existing tunnel infrastructure. The vibration criteria would typically be defined with trigger levels (green, amber, red) and monitored by devices installed in or adjacent to the existing tunnel infrastructure. The vibration limits may be set by the current codes/standards or by the existing tunnel infrastructure owner based on the type of assets. Typical actions would be agreed between the asset owner and building developer/constructor for each of the trigger levels prior to construction of the future development commencing.

DEVELOPMENT APPLICATIONS

The provision of documented allowable conditions means property developers have some certainty around what can or cannot be undertaken on the existing property with respect to building height/form (additional loading), basement excavation (continuous excavations) and the allowable movement (distortion) from these loading and unloading conditions on the existing operational tunnels.

If a proposed development achieves the above future development conditions, and this is adequately demonstrated via engineering assessment included as part of the development application, then the developer should have a degree of confidence that the existing tunnel infrastructure owner will have no objection to the proposed development. The engineering assessments of the future development undertaken to support the development application should be simple, cost-effective methods. Such methods could include:

- Loading (additional loading) assessed via simple footing loading theory, such as that originally proposed by Boussinesq 1885
- Unloading (continuous excavations) assessed via simple 2D continuum finite element modelling (FEM)
- Tunnel distortion due to the development's loading and unloading via simple 2D continuum finite element modelling (FEM)
- Construction vibration assessed via simple vibration theory, using known site constants if available as well as documenting the proposed construction method/equipment

Future developments may step outside of the allowable future development conditions (eg deeper basement excavations), but developers should expect that much more rigorous and detailed engineering assessments would need to be undertaken, along with extensive consultation with the existing tunnel infrastructure asset owner. Rigorous detailed engineering assessments should include loading and unloading impacts on the existing tunnel waterproofing systems, including TBM segmentally lined tunnel gaskets where relevant, and any groundwater drawdown impacts from the proposed future development.

The risk of non-approval of the future development by the existing tunnel infrastructure asset owner is likely to increase where proposed development's additional loading (building height/form etc) and basement excavation (continuous excavations) exceeds the future development allowable conditions.

FUTURE DEVELOPMENT ASSESSMENT METHODS

The following is an assessment of the impacts for a simple proposed future development over the top of an existing operational tunnel. A comparison between the simple analysis versus more complex rigorous models and engineering assessments is undertaken.

FEM MODELLING

The following is a comparison between the use of a 3D versus 2D model with regards to necessity of use and accuracy of results. This section will discuss the inputs for the models and discuss and compare the 3D versus 2D results they each output.

The models assume a simplified square building with a basement over twin tunnels, where one tunnel (Tunnel 1) lies directly beneath the centre of the building and the other tunnel (Tunnel 2) is offset by a tunnel diameter.

Two types of geology are used in the model, with the change occuring at the location of the tunnel axis.

Modelling inputs

Figure 3 shows the basis of the model used across all Plaxis3D, RS2 and Settle3 models.

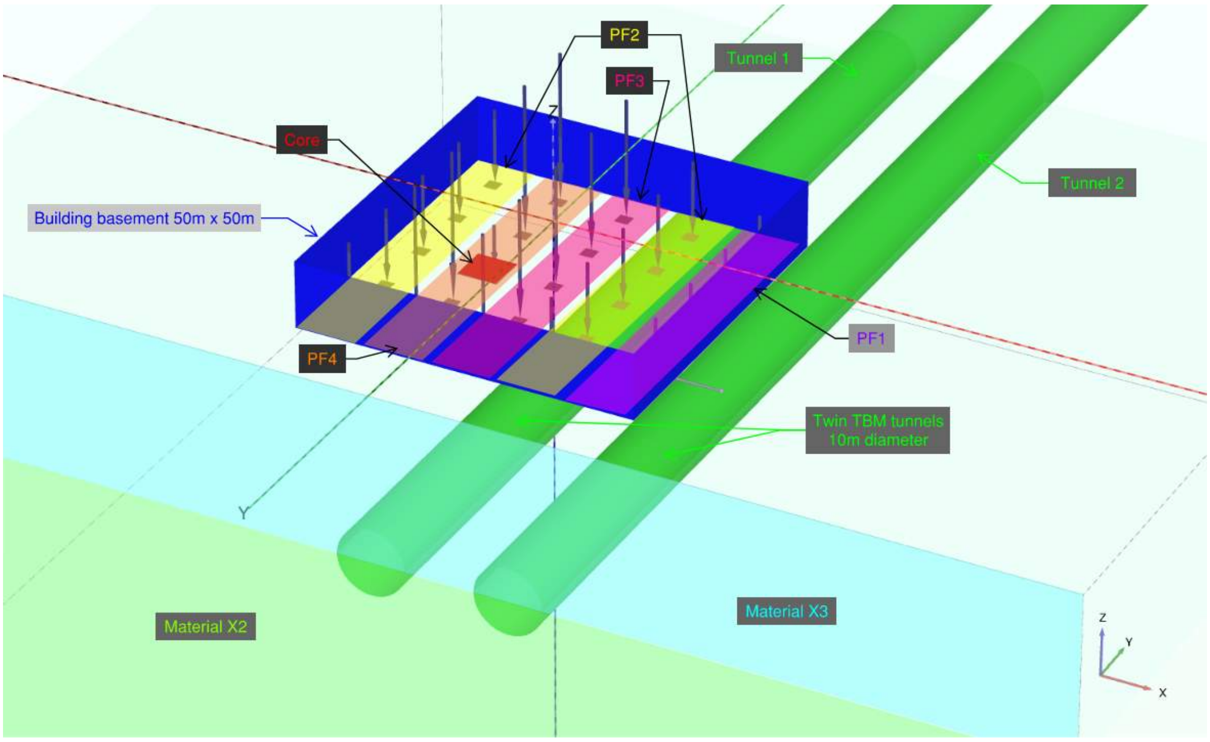

Figure 3: Model Basis (Plaxis 3D shown)

Inputs and Assumptions

The tunnel, building and geological information assumed for model purposes have taken into consideration existing conditions and previous projects to reflect realistic values. The loading scheme also reflects experience from previous projects with regards to column arrangement, core location, load per column/core, total overall loading of the building, and pad footing size.

- The surface level is at RL 0m.
- The building basement is 50m long by 50m wide by 10m deep. The underside of the basement is at RL -10m
- The twin tunnels have an outer diameter of 10m, with axis level at RL -25m
- Lining is applied to both the basement wall and tunnel lining for stability and model convergence
- The column layout is a 5 by 5 grid of 10m column spacing where a single core is located to one side.
- Core and columns are assumed to be located on the same level as the underside of the basement (RL -10m)
- The geology consists of 2 different material layers changing at the tunnel axis. Material X3 lies from RL 0 to RL -25m and material X2 lies from RL-25m to the bottom of the model.
- In the discontinuum model, a 30m by70m region around the twin tunnels is designated to discontinuum properties and joint network.
- The assumed construction sequence is outlined in Table 5.
- The loading arrangement is shown in Figure 3 with the load values and pad footing size corresponding to Table 2.

Table 2: Load acting on pad footings and pad footing size

Load type	Vertical load (kN)	Length (m)	Width (m)
PF1	950	2	2
PF2	2380	2	2
PF3	3810	2	2
PF4	4760	2	2
Core	8570	6	6

Model properties

Lining was used for the basement wall and tunnel in the model. The lining properties used are shown in Table 3:

Table 3: Lining properties

Lining	Thickness (m)	Unit Weight (MN/m3)	E (MPa)	Poisson's Ratio
Basement wall	1	0.025	30000	0.2
Tunnel Lining	0.25	0.025	30000	0.2

The ground material properties for ground type X3 and X2 are shown in Table 4.

Table 4: Model Geotechnical Properties

	Parameter	X3	X2
Continuum	Unit Weight (MN/m3)	0.024	0.024
	c' (MPa)	0.225	0.375
	phi' (º)	42	42
	E (MPa)	1000	2000
	Tensile strength (MPa)	0.04	0.1
	Poisson's Ratio	0.25	0.25
	K0 E-W	1.5	1.5
	K0 N-S	1	1
Discontinuum	c' (MPa)	0.3	0.9
	phi' (º)	42	47
	E (MPa)	2500	4000
	Tensile strength (MPa)	0.1	0.25
	JCS	12	15
	JRC	6	8
	Joint phi residual (º)	43	32
	Joint Normal Stiffness (MPa/m)	4000	8000
	Joint Shear Stiffness (MPa/m)	400	800
	Bedding Inclination (º)	1	1
	Cross Joint Inclination (º)	85	80
	Bedding spacing (m)	0.5	2
	Cross-Joint spacing (m)	1	3

Construction sequence

The construction sequence of the models remains the same to capture the reasonable order of events. More complex models involve gradual relaxation of the tunnel in 2D or gradual tunnel advancement in 3D to capture a more realistic tunnel behaviour. Since the interest of these models is the effect of a building development over an existing tunnel, the tunnel construction has been simplified.

Table 5 shows the construction sequence used in the models. Plaxis 3D uses all 6 stages as it can capture the loading effect, however RS2 is unable to achieve this due to 2D limitations, hence RS2 uses only stages 1-5. The loading assessment from Settle3 is used to supplement this.

Table 5: Construction Sequence for Plaxis 3D model

Stage	Description
Stage 1: Initialise	Allows the model to initialize to arrive at in-situ conditions
Stage 2: Tunnel 1	Excavation of Tunnel 1 and tunnel lining wished-in place
Stage 3: Tunnel 2	Excavation of Tunnel 2 and tunnel lining wished-in place
Stage 4: Building basement wall	Basement wall lining is installed. Plaxis 3D model displacement values reset to zero in order to see displacement results only after tunnel construction is completed.
Stage 5: Building basement excavation	Basement is excavated causing unloading effect on tunnel
Stage 6: Building loading	Building is loaded as per column layout and load values

Plaxis 3D inputs for unloading and loading modelling

The 3D model was conducted with FEM using Plaxis 3D (only in continuum) to investigate both the unloading effect of basement excavation and the loading effect of the building loads.

The advantage of the 3D model is it allows the 3D effect of the building excavation which is finite in both horizontal axis directions to be modelled. It also allows the loading to be accurately represented in the 3D space. In more complicated models, 3D models can also capture the effects of cross passages, non-symmetrical models and non-linear alignment geometry.

The disadvantage lies in the time required for model setup, meshing, running and results interpretation, which becomes magnified with higher model complexity. More complexity also results in an increased likelihood of encountering errors and additional time for debugging.

RS2 inputs for unloading modelling only

The 2D model was conducted with FEM using RS2 in continuum with an additional sensitivity model to see discontinuum effects, to investigate the unloading effects of basement excavation.

While 2D models cannot provide the benefit of capturing 3D effects, they have the advantage of a much easier and faster model building, running and results interpretation. A 2D model assumes a single cross section behaves the same infinitely into the page, which provides a conservative result with regards to a basement excavation – a basement can be constructed in 3D with its actual dimensions in and out of plane while in 2D it is simplified to an infinite trench of the same in plane width dimension.

In addition, 2D models are much faster and easier to run as a discontinuum model than in 3D, due to the number of elements and element interaction introduced in three dimensions.

A discontinuum model assesses whether the distinct jointing effect is more critical than assuming an equivalent overall ground mass acting as one (only applies to rock, soil acts as a continuum).

Settle3 for loading modelling only

The 2D modelling for load stress compliance was conducted using Settle3. Settle3 analyses vertical loads acting on the ground to calculate settlement and stress using the Boussinesq theory. This assessment was done to supplement RS2 for efficient model building and loading iteration to assess the stress at 1m above the tunnel.

The benefit of this analysis is that it is based on closed form solutions in a 3D space, allowing for quick calculations and clear results. However, the effect of stress redistribution due to the presence of tunnels needs to be considered.

Modelling results

The following models were assessed:

Table 6: Models assessed

Model ID	Description
Model 1	Plaxis3D continuum model with unloading basement excavation and building loading
Model 1a	Plaxis3D continuum model with building loading without the presence of the basement
Model 1b	Plaxis3D continuum model with building loading without the presence of the tunnels
Model 1c	Plaxis3D continuum model with building loading without the presence of both the basement and the tunnels
Model 2	RS2 continuum model with unloading basement excavation
Model 2a	RS2 continuum model with unloading basement excavation
Model 3	Settle3 model with building loading

The following comparisons were made between the models:

Table 7: Model comparison

Comparison	Models being compared	Description
Unloading 3D and 2D	Model 1 vs Model 2	Plaxis3D basement unloading (continuum) vs RS2 basement unloading (continuum)
Continuum and discontinuum modelling	Model 2 vs Model 2a	RS2 basement unloading (discontinuum) vs RS2 basement unloading (discontinuum)
Loading 3D and 2D	Model 1 vs Model 3	Plaxis3D loading stress vs Settle3 loading stress
Loading with presence and absence of basements and tunnels	Model 1 vs Models 1a,1b,1c	Plaxis3D loading stress vs Plaxis3D loading stress with and without basement and tunnels

Unloading impact on tunnel movement

The unloading effect due to the basement excavation causes the existing tunnel to naturally move towards the excavation. This causes a displacement on the tunnel, and the future development conditions require that the total displacement acting on the tunnel diameter is to be within a given limit.

Plaxis 3D continuum results vs RS2 continuum results

The displacement results in Table 8 show the comparison between the 3D and 2D continuum models. For simplicity, the diameter displacement can be calculated by taking the difference between the movement at the crown and at the invert due to the tunnels being located close to directly under the location of maximum loading.

Table 8: Diameter displacement results between Plaxis3D and RS2 continuum

Model	Tunnel	Vertical displacement at crown (mm)	Crown % difference from RS2 continuum	Vertical displacement at invert (mm)	Invert % difference from RS2 continuum	Diameter displacement (crown invert difference) (mm)	Diameter displacement % difference from RS2 continuum
Model 2: RS2 continuum	Tunnel 1	5.9	-	4.0	-	1.9	-
	Tunnel 2	4.3	-	3.2	-	1.1	-
Model 1: Plaxis3D	Tunnel 1	5.2	-12%	2.2	-44%	3.1	+54%
	Tunnel 2	3.6	-18%	1.8	-44%	1.7	+56%

The results show that overall, the Plaxis3D model produces lower displacement results compare to RS2. This is expected as the 3D model can capture the basement as a box, limiting the amount of excavation in the out of plane direction, whereas the 2D model assumes the basement is an infinite trench in the out of plane direction, thereby calculating a larger result.

In addition, the overall diameter displacement for the 3D model is 2-3mm, whereas the 2D model is 1-2mm. This difference is due to the RS2 results calculating larger displacements for each of the crown and invert, the difference can therefore be smaller than what is seen in the 3D model.

Sensitivity - RS2 continuum results vs RS2 discontinuum results

A sensitivity case was conducted to show the difference between continuum and discontinuum results (both in 2D using RS2). Table 9 show the diameter displacement results.

Table 9: Diameter displacement results between RS2 discontinuum and RS2 continuum

Model	Tunnel	Vertical displacement at crown (mm)	Crown % difference from RS2 continuum	Vertical displacement at invert (mm)	Invert % difference from RS2 continuum	Diameter displacement (crown invert difference) (mm)	Diameter displacement % difference from RS2 continuum
Model 2: RS2 continuum	Tunnel 1	5.9	-	4.0	-	1.9	-
	Tunnel 2	4.3	-	3.2	-	1.1	-
Model 2a: RS2 discontinuum	Tunnel 1	5.6	-5%	3.9	-1%	1.6	-14%
	Tunnel 2	4.2	-4%	3.1	-5%	1.1	-1%

The raw results (crown and invert displacements) show the difference between the model types is not very significant, but shows that for the crown and invert displacements, the continuum model has slightly higher movement results. This is expected as the continuum parameters are a rock mass equivalent properties of the discontinuum parameters when accounting for the weakness of joints. Should the joint network be blocky enough to behave almost like a continuum such as this model, then it is expected both continuum and discontinuum models yield similar results. In this case, the continuum case is slightly more conservative, which shows that it is appropriate to conduct a continuum 2D model rather than a discontinuum one. Should results be approaching critical limits, such as the Future Development conditions, the more complex discontinuum model could then be conducted for further refinement of the results and verification.

Loading stress criteria at 1m above tunnel – Plaxis 3D vs Settle3

Loads from the building acting on footings travel through the ground and impact the tunnel. The closer the tunnel is both transversely and vertically create more loading stress on and around the tunnel. A limit is given for stress that is allowed to act from a future development building at 1m above the tunnel to ensure the tunnel behaves to its design.

Plaxis 3D model loading results vs Settle3 loading results

Figure 4 shows the results along the largest stress plan perpendicular the tunnel, at 1m above the tunnel crown.

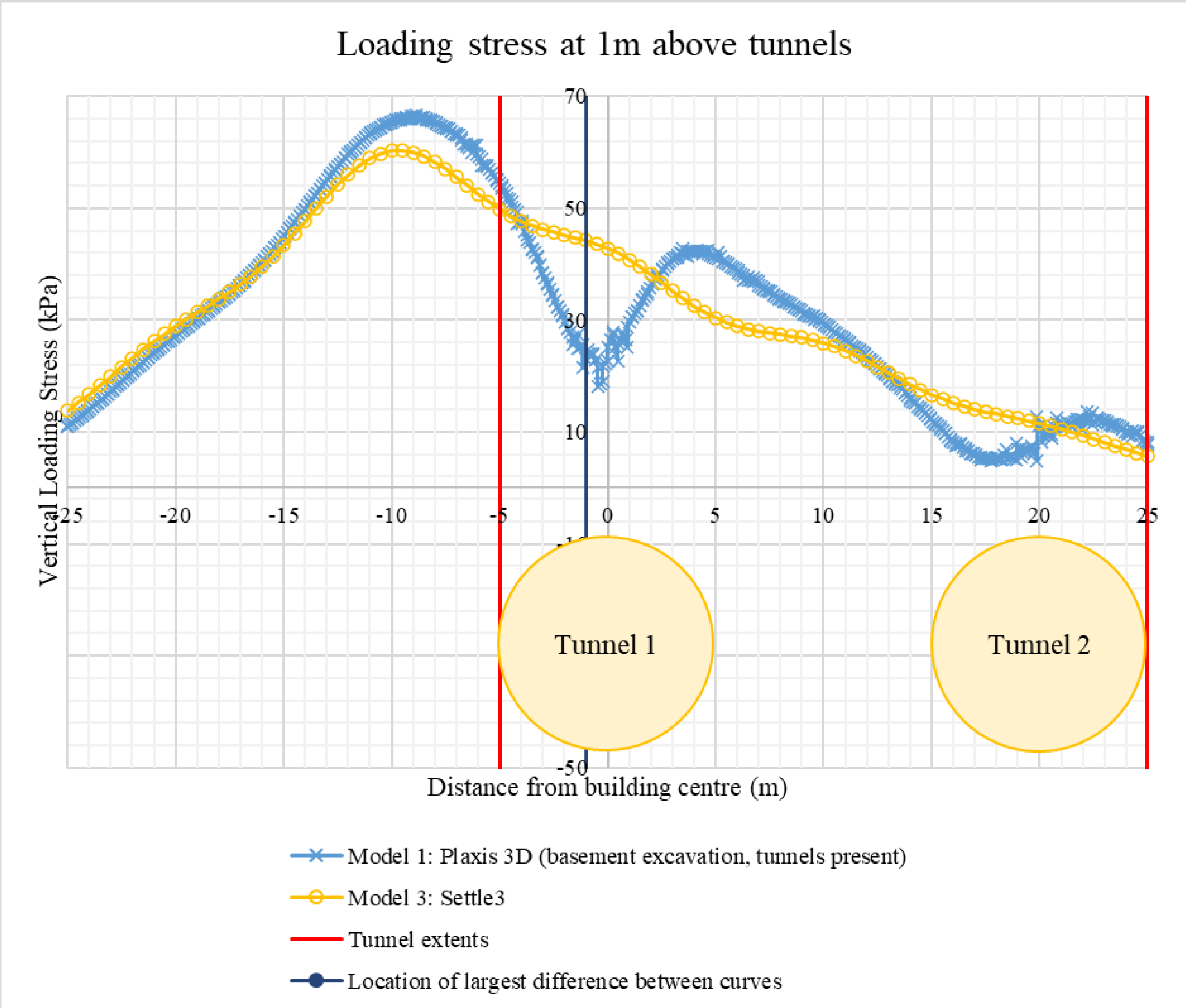

Figure 4: Comparison of vertical loading stress between Plaxis3D and Settle3

Only the location directly above the tunnels are of interest as indicated by the red lines in Figure 4. Table 10 are the results comparing the loading stresses between the two models, the maximum loading stress occurs on the west boundary of Tunnel 1 (X=-5m), the average loading stress is compared for only within the tunnel region, and the largest stress difference occurs over Tunnel 1 (X=-1m).

Table 10: Comparison of vertical loading stress between Plaxis3D and Settle3

Model	Vertical loading stress at maximum location (X=-5m) (kPa)	% difference from Settle3	Average loading stress across tunnel location (kPa)	% difference from Settle3	Vertical loading stress at largest stress difference (X=-1m) (kPa)	% difference from Settle3
Model 3: Settle3	49.8	-	25.8	-	44.2	-
Model 1: Plaxis3D	54.5	+10%	24.3	-5%	23.1	-48%

The results show the Plaxis3D model is larger in loading stress to Settle3 by about 10%, but is 5% smaller in average stress and there is a 48% difference over Tunnel 1. This would be due to the stress redistribution around the tunnel, which redistribute the vertical stress at the crown and increase it closer towards the tunnel walls. This shows that Settle3 is conservative with respect to the average stress and does not account for the stress redistribution which results in conservatively large stresses over the tunnel crowns. However, at the point of interest (location of maximum stress), the Plaxis3D accounts for the stress redistribution at this location, resulting in larger stress result than Settle3. The increase in stress for both models at this location is due to the large core load acting at that location. Therefore, this stress increase will change depending on the location of the core. These results show that despite the building core being close to the tunnel, there is only a 10% increase in vertical loading stress.

Sensitivity on basement and tunnel influence on stress redistribution at 1m above tunnel

Sensitivity models were conducted in Plaxis3D to investigate how the redistribution of stress due to the basement and tunnel affected the loading stress at 1m above the tunnel. The sensitivity models looked at the absence of the basement in presence of the tunnel, vice versa, and when neither the basement nor tunnel were present (essentially the same type of model as Settle3).

Figure 5 shows a graph comparing all results.

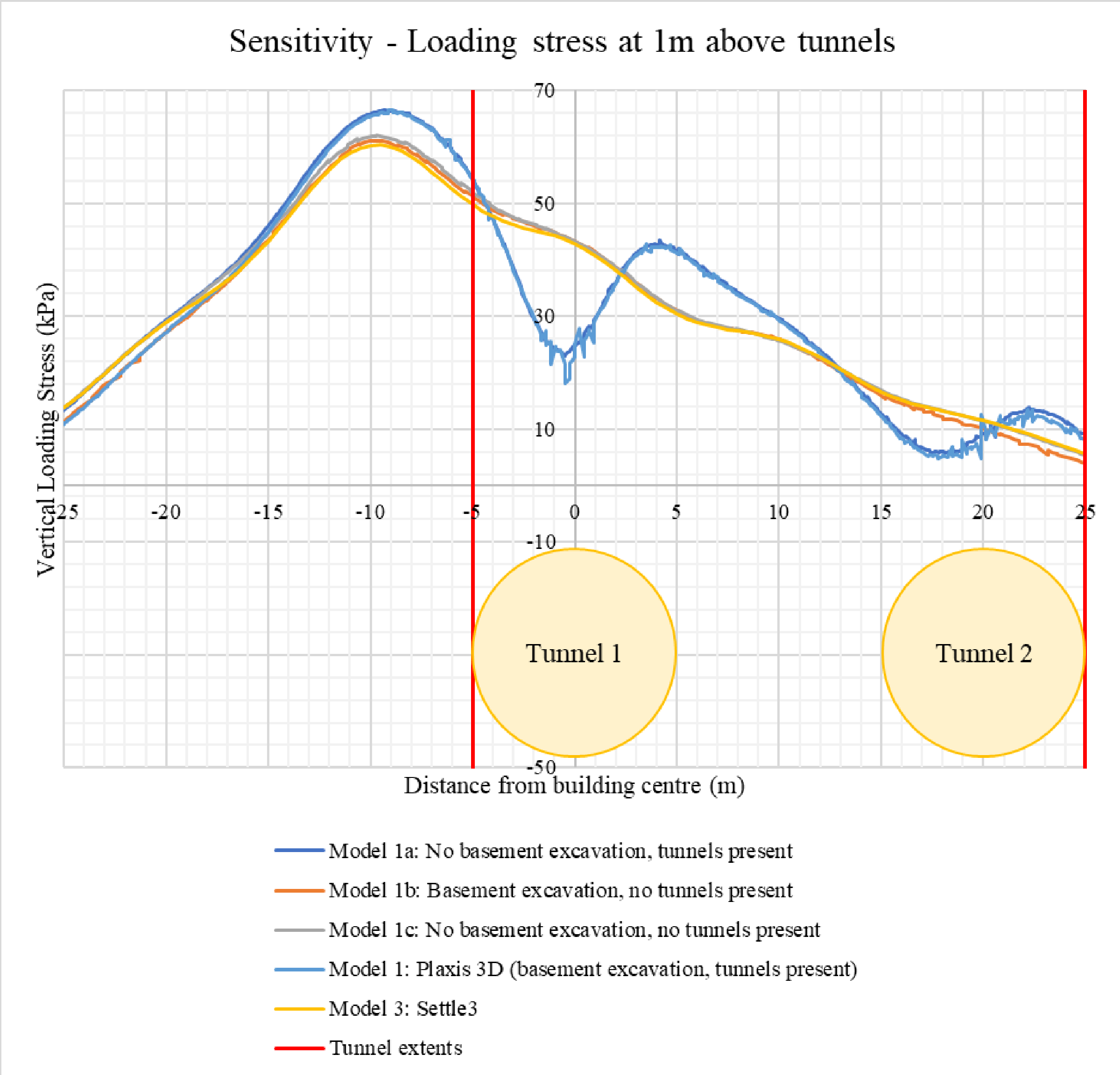

Figure 5: Comparison of vertical loading stress sensitivity cases

It can be seen from Figure 5 that the loadings stress between Model 3 and Model 1c yields almost the same behaviour, which is expected since this sensitivity case has no basement and no tunnel, which is what Settle3 models. Model 1b also shows this behaviour despite the basement present. In addition, where the tunnel is present in both the Model 1 and Model 1a, very similar behaviour is also observed, with any discrepancies due to the meshing of the 3D model.

This indicates that the basement does not impact the loading stress much at all, but the tunnel presence does cause substantial stress change. This is caused by the tunnel redistributing the stress around itself, so the stress at the crown and invert is predominantly horizontal (hence vertical stress is very small). Therefore, there are significant decreases in stress near the tunnel centres.

The large increase in stress west outside of Tunnel 1 boundary is due to the core load modelled. Typically, this increase would not be seen directly over the tunnels as large building loads are ideally designed away from the tunnel or carried by piles to below the tunnel invert.

The comparison of the loading stress at again Tunnel 1 west boundary (largest stress location) is shown in Table 11.

Table 11:Comparison of vertical loading stress results between sensitivity cases

Model	Vertical loading stress (kPa)	% difference from Model 1 Plaxis3D final results
Model 1: Plaxis3D final results – Basement excavation, tunnels present	54.5	-
Model 1a: Plaxis3D – No basement excavation, tunnels present	54.8	+1%
Model 1b: Plaxis3D – Basement excavation, no tunnels present	51.6	-5%
Model 1c: Plaxis3D – No basement excavation, no tunnels present	52.1	-4%

The results show that differences between the sensitivity results is not very significant, up to a 5% difference. As discussed earlier, the most significant difference is the behaviour of vertical stress as it is redistributed around the tunnel.

Outcome Summary

The outcomes of the modelling comparisons have been summarized in Table 12:

Table 12: Modelling comparison outcomes

Comparison	Models being compared	Description	Outcome
Unloading 3D and 2D	Model 1 vs Model 2	Plaxis3D basement unloading (continuum) vs RS2 basement unloading (continuum)	Plaxis3D have lower results than RS2, up to 20% difference for crown displacements up to 5mm and up to 45% difference for invert displacements up to 4mm
Continuum and discontinuum modelling	Model 2 vs Model 2a	RS2 basement unloading (discontinuum) vs RS2 basement unloading (discontinuum)	Discontinuum up to 5% higher than continuum
Loading 3D and 2D	Model 1 vs Model 3	Plaxis3D loading stress vs Settle3 loading stress	Plaxis3D results 10% higher than Settle3
Loading with presence and absence of basements and tunnels	Model 1 vs Models 1a,1b,1c	Plaxis3D loading stress vs Plaxis3D loading stress with and without basement and tunnels	Up to 5% difference in loading stress results

CONCLUSIONS

Based on the above findings, the comparison between the 3D and 2D modelling of the future development effect on the tunnel displacement is reasonable, noting the 2D model provides conservative (greater) predictions. The differences in loading between Settle3 and Plaxis3D was 10% which is sufficient especially when considering the low level of detail required for this type of modelling.

The discontinuum model sensitivity case showed barely any difference in movement (up to 5%) to the continuum model, therefore continuum is sufficient to use (if the rockmass assumptions used are appropriate).

The loading sensitivity cases of Plaxis 3D with respect to the presence and absence of tunnels and basement, show that at the location of largest stress (closest to maximum building load), there is also barely any difference in stress (up to 5%) between the 4 models.

The Settle3 assessment cannot consider the tunnels below and the effect these tunnels have on the stress re-distribution from the future development loading. Therefore, where Settle 3D results predict loading from the future development close to the allowable future development condition loading, more rigorous modelling and assessment of the loading condition on the existing tunnel infrastructure should be undertaken.

The model setup, run and debugging time must also be considered. To prepare and run a 3D model takes much longer than a 2D model. A 2D model creates much less elements during meshing and can increase its density much easier than 3D. In addition, due to the run time being much quicker, several sensitivity cases can be conducted, as well as easily incorporating design changes if required.

The advantage of a 2D model is that reliable results can be outputted in a short amount of time. This scenario is ideal for the purpose of development applications for future development buildings on existing tunnels, as they do not require detailed design level analysis. 3D models can still be used, however only after interpreting 2D model results, and determining that the additional accuracy and time spent are worthwhile.

The outcome of this assessment has shown it is sufficient to conduct the 2D models in place of a 3D model for initial preliminary assessment of development applications for future developments over existing tunnels.

In conclusion for development applications for future developments, the following is recommended:

- Future Development loading (additional loading) assessed via simple footing loading theory, such as that originally proposed by Boussinesq

  Based on the loading comparison between 2D and 3D, it was found that the difference between Plaxis 3D and Settle3 loading results were small (10% difference) and could vary depending on load location. This shows that as an initial assessment it is appropriate to simply use the Boussinesq theory in Settle3 to determine Future Development loading on the tunnel. Should results approach the allowable stress conditions limit, then more detailed 3D modelling can be conducted.

- Future Development unloading (continuous excavations) assessed via simple 2D continuum finite element modelling (FEM)

  When investigating the results between 2D and 3D unloading displacements, the RS2 model shows more conservative results (20-40% larger) than Plaxis3D. In addition, the sensitivity case between 2D continuum and discontinuum models show negligible difference (5%), showing that it is appropriate to analyse the unloading movement effects of the tunnel using 2D continuum modelling as opposed to 3D.

- Tunnel distortion due to the development's loading and unloading via simple 2D continuum finite element modelling (FEM)

  The 3D modelling shows more conservative results for the diameter displacement due to unloading effects. This is due to the 2D model having larger conservative movements at both the crown and invert, hence the difference between to calculate the diameter displacement is smaller than for 3D. Despite this, the significant difference is only large due to the results being small (2-3mm). It is still worth conducting the investigation first in 2D to provide a valuable estimate of the range in which the diameter displacement lies, and should further modelling be necessary, 3D can then be conducted.

- Future Development construction vibration assessed via simple vibration theory, using known site constants if available as well as documenting the proposed construction method/equipment

Reference List

Boussinesq, J., *Applications des potentiels à l'étude de l'équilibre et mouvement des solides elastiques*, Gauthier–Villars, Paris, 1885.

Ground response due to deep excavations adjacent to underground infrastructure in Sydney

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ABSTRACT

ABSTRACT: *A deep basement was recently constructed in Sydney's Hawkesbury Sandstone for a property development adjacent to critical transport infrastructure. At over 40 metres deep, the excavation is among the world's deepest basement excavations. This paper presents the geotechnical design challenges, construction outcomes, use of the Observational Method (OM) on the excavation, and discusses the impact assessment which enabled adjacent asset owners to establish realistic risk baselines during early stages of the project which ultimately reduced the project cost and mitigated construction risk.*

When lateral displacements approached limits agreed with adjacent asset owners, the OM and impact assessment process offered a flexible framework for reassessing ongoing movements and effects on adjacent infrastructure so that construction could safely proceed. The case history demonstrates the benefit of adopting the OM for excavations to address safety and program requirements, react to unexpected movements during construction, and mitigate unsatisfactory performance.

Keywords; Deep excavations, impact assessment, Observational Method, risk management, site retention, instrumentation & monitoring.

INTRODUCTION

Ground movements during deep basement excavations can seriously impact adjacent infrastructure and utilities. We need to consider the excavation-induced displacements to assess the impact on those assets, and the necessary measures to mitigate unsatisfactory performance.

This paper describes ground response issues associated with an excavation for a property development on Sydney's North Shore. The project involved excavating over 43 m deep, and is among the world's deepest (known) building basement excavations. The excavation took place next to critical road and rail transport infrastructure, and the excavation's influence on these assets was an important design consideration. An added complexity was the potential impact of building excavation and imposed building loads on nearby utilities, bridges and underground structures.

PROJECT BACKGROUND

The project development (known as "Eighty Eight") is located at 88 Christie St, St Leonards, NSW, about 7km north of the Sydney CBD. Figure 1 shows an aerial view of the development under construction.

The new development included two residential towers (47 and 26 storeys high) and a commercial tower (15 storeys) over a large retail precinct with 10 levels of below-ground basement up to 43 m deep. The structure for the building is founded on pad footings.

The 6,500 m^2 basement excavation extends to within 2.5 m of the Sydney Trains boundary to the west with the Pacific Highway to the north.

The site boundary is surrounded by sensitive buried utilities, road and rail infrastructure, and nearby buildings for which ground movement was a key consideration (Figures 2 & 3). As a result, part of the development approval involved establishing an excavation protection strategy for neighbouring infrastructure. This required assessing the potential impacts that the proposed deep excavation could induce, designing ground control measures, and a monitoring strategy to confirm the predictions and reduce geotechnical risks.

Figure 1. Aerial view of the excavation showing adjacent transport infrastructure

Adjacent transport infrastructure

The initial scheme had proposed a basement about 16 m deep. Or this scheme, the footprint of the proposed basement excavation was about 20 m from the existing rail corridor, and about 14 m from the earlier-planned CBD Rail Link tunnels (Figure 2). During design development, the basement was increased to 10 levels to a maximum of more than 43 m deep adjacent to Pacific Highway, and the CBD Rail Link tunnels were moved east as part of the Sydney Metro project.

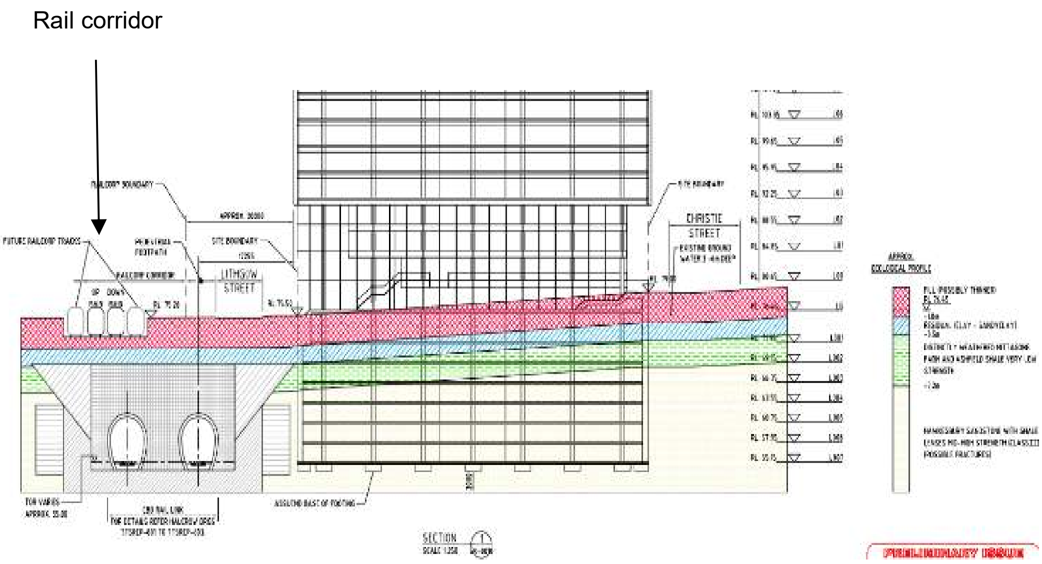

Figure 2. Initial scheme showing ground conditions and proposed rail tunnels on western boundary

EXCAVATION DESIGN

Geology

The site geology comprises the upper sedimentary formations of the Sydney Basin stratigraphic sequence – sub-horizontal beds of Triassic-aged rock comprising (youngest to oldest) Mittagong Formation and Hawkesbury (Sydney) Sandstone.

Design considerations

Generally, the ground conditions in Sydney are favourable for basement construction where good quality sandstone occurs at shallow depths. However, lateral displacements typically occur for deep excavations even in good quality sandstone due to the relief of the relatively high horizontal stresses in the Hawkesbury Sandstone. These step displacements occur due to changes in stiffness and shear strength within the bedrock mass particularly at the sub-horizontal bedding defects.

Key geotechnical inputs need to be quantified to enable a reasonable, safe, cost-effective and prudent design of a site retention system in rock. These inputs include ground conditions both within and outside the site, characteristics of rock mass discontinuities, rock mass strength and stiffness, groundwater conditions and the in situ stresses.

Figure 3. Location plan showing adjacent transport infrastructure and aerial view of excavation progress.

Understanding the geological and geomorphological history of the site provides important insights into likely ground conditions, including: potential variability, presence of significant features like palaeovalleys, drainage features, dykes, faults and other rock mass properties including discontinuity patterns, weathering profile, magnitude and direction of insitu stresses, contamination and groundwater conditions.

Subsurface conditions

We assessed ground conditions from geotechnical information from investigations carried out by WSP and others within the proposed development and surrounding areas. Rock strength index tests included unconfined compressive strength (UCS) and point load on rock core samples. To understand and quantify the rock mass characteristics, we also used downhole geophysical surveys and borehole imaging to learn details of orientation, spacing, aperture, and infill characteristics of various rock mass defects including joints and bedding partings.

The development site is underlain by uncontrolled fill and residual soil, followed by Mittagong Formation and Hawkesbury Sandstone. The Mittagong Formation is characterised by interlaminated sandstone and siltstone comprising fine-to-medium grained, light grey sandstone with dark grey siltstone bands that are (generally) extremely weathered and very low strength. The Hawkesbury Sandstone is characterised as medium-to-coarse grained, grey, with cross bedding and medium-to-high strength.

The rock classification adopted the Sydney classification system (Pells et al, 2019). This system was developed for foundations and is based on UCS of saturated substance (i.e. intact sandstone or shale), defect spacing and percentage of seams within a defined vertical interval of the near-horizontal bedded rock. Both strength testing and borehole imaging identified a weaker shale/laminite band at about 40m deep – close to final excavation depth. Figure 4 shows a summary of ground conditions and rock strength/ imaging data.

Excavation restrictions

Restrictions for the development by the adjacent asset owners, TfNSW (ASA, 2015) and RMS (RMS, 2012) included:

- Anchor systems cannot be used in the rail easement.
- Construction and operation of external developments shall not affect the stability and integrity of railway infrastructure through loading from the development and ground deformation.
- Maximum 30 mm displacement on the Pacific Highway.
- Requires constant monitoring of ground movements due to bulk excavation and monitoring of track structures.

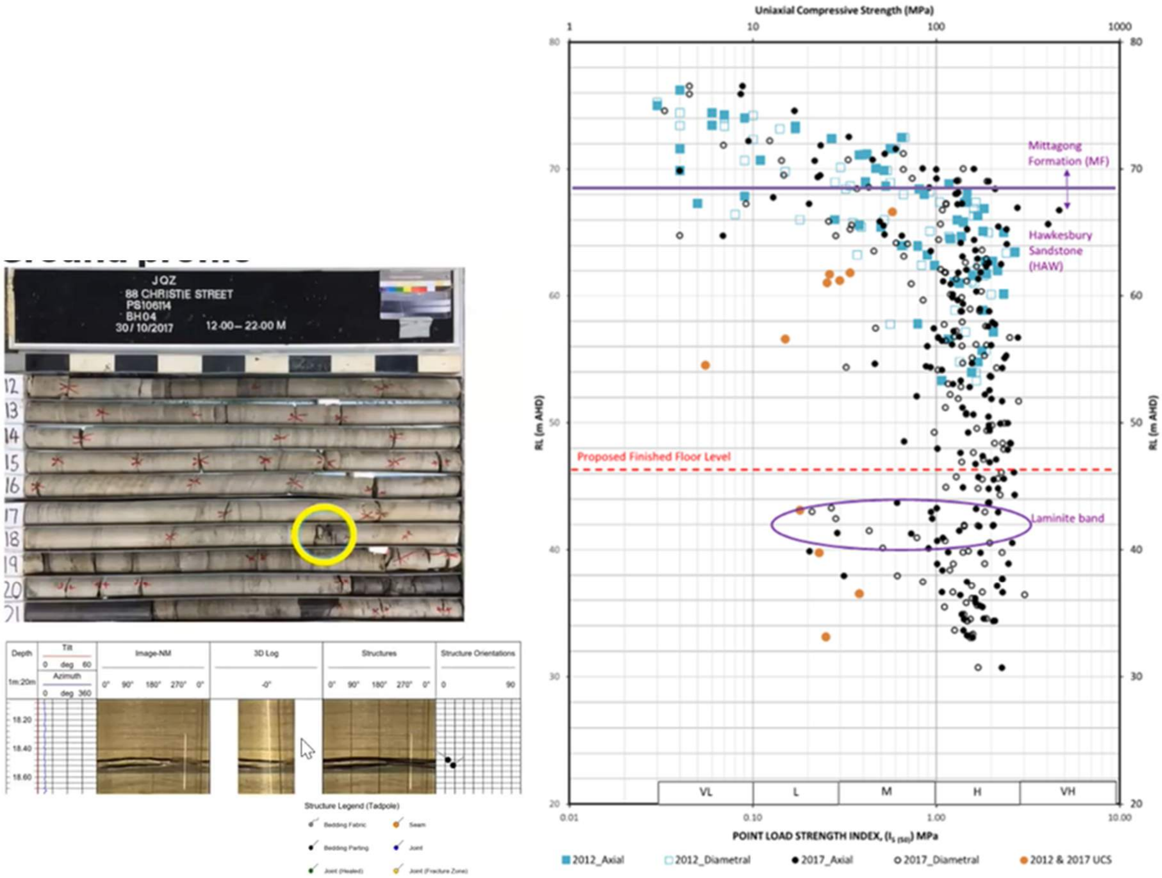

Figure 4. Ground conditions summary and rock strength data

Selecting retention systems for the excavation to satisfy these criteria, and avoid temporary and permanent anchors within rail, road and utilities corridors along the project boundaries had a major influence on basement geometry and the choice of retention systems.

Excavation support

Most of the basement excavation face comprises a sequence of weathered shale and sandstone. The temporary shoring system supporting the 10 levels of basement along the Sydney Trains boundary (Figure 5) consisted of:

- A stepped double contiguous concrete pile wall with short dowel anchors on the front wall, and shotcrete located along the middle of the west (railway) wall.
- Ground anchors within a 20 m wide square buttress of rock on the south corner that was not excavated.
- Ground anchors for 20 m length in the north corner adjacent to the Pacific Highway railway bridge. The north corner was not fully excavated, which allowed angled ground anchors to be installed within the remaining triangle of rock.

The wall configuration was based on a similar cantilever post-tensioned pile wall that was successfully adopted for the Gore Hill Freeway widening at Artarmon where project boundary constraints ruled out installing ground anchors (Hewitt et al, 2008).

Figure 5. Excavation support along Sydney Trains (west) boundary

IMPACT ASSESSMENT

The impact assessment involved geotechnical analyses that considered the proposed development's effects including basement excavation and building loads, ground properties, and correction of the natural stress field based on rock mass quality.

For detailed design, and damage risk classification, we used both 2D and 3D finite-element numerical modelling programs to assess ground movement. These numerical analyses included continuum (using FLAC 2D and FLAC 3D) and discontinuum analyses (using Rocscience programs RS2 and RS3).

Design parameters

The geotechnical design parameters adopted for this impact assessment were selected based on the Sydney classification system (Pells P.J.N, et al, 2019), results of the geotechnical investigation, the intact parameters, estimated Geological Strength Index (GSI) for each rock mass class, case history data, published data on sandstone and shale strength and stiffness, and the excavation depth.

The adopted geotechnical design parameters are summarised in Table 1. To address sensitivity, we reduced the values of cohesion (c) and tensile strength (σ) for Sandstone Class IV and Mittagong Formation to less than 50% of design values.

Table 1. Adopted design parameters

Material type	UCS (MPa)	Mass modulus(GPa)	GSI	Mohr-coulomb criterion		
				σ (kPa)	ϕ (deg)	c (kPa)
Sandstone I	30	3	75	300	55	1000
Sandstone II	25	2	65	100	50	500
Sandstone IV	10	0.5	45	25	45	250
Shale II	15	1	50	60	40	250

In-situ stress

The field in-situ stresses have a significant impact on both deep excavation conditions and induced ground movements in the excavation works' immediate area, due to high in-situ lateral stresses, which can be 'locked in' within the bedrock stratum.

We incorporated adjacent deep excavations, including "The Forum" building, north of the development within the modelling as part of the impact assessment for a holistic approach to the major and minor stress distribution within the subsurface geological units next to the excavation. We adopted the following in-situ stress relationship based on WSP's reference design for the nearby Sydney Metro City & Southwest project:

- Upper bound:

  $\sigma_{H(NS)} = 1.0\text{MPa} + 3.5\ \sigma_v;\ \ \sigma_{H(NS)}/\sigma_{h(WE)} = 1.5$ (1)

- Lower bound:

  $\sigma_v = \sigma_H = \sigma_h = 1.0$ (2)

  (stress field assumed to be lithostatic)

We applied the upper-bound stresses to fresh, good quality sandstone and shale (Class I and II). In poorer quality rock masses, the horizontal stresses are expected to be less, and the lower bound stresses were applied. The minor horizontal stress was applied perpendicular to the excavation's eastern and western walls. The major horizontal stress is applied perpendicular to the excavation's northern and southern walls.

Predicted ground performance

The retention system design addressed the following displacement mechanisms, observed to have caused ground surface deformation next to the excavation that could affect the railway.

- Lateral earth (soil) pressure acting on the shoring system causing it to deflect.
- Relaxation of the rock mass resulting from reduced lateral stress (stress relief).
- Anchor hole drilling and installation.

The proposed shoring system with soldier piles and anchors was designed to control the ground surface deformation due to lateral soil pressure in the upper parts of the proposed excavation. We chose the shoring system's layout and stiffness to minimise the ground movements and the impact on the railway tracks, and rail overbridge.

The rock mass relaxation due to stress relief from the basement excavation's deeper parts will happen irrespective of the shoring system type. The numerical assessment was calibrated against monitoring results from various deep excavations around Sydney, including monitoring results of the Embassy developments' basement excavation on the rail corridor's western side.

Figure 6 shows the site's north east quadrant with the predicted 3D ground movements in the rail corridor, Pacific Highway and rail overbridge. The estimated ground performance (from the numerical assessment; a database of movements for walls using published case history data (Hewitt et al, 2008 & Wong, 2013); and monitoring data from other nearby projects) indicates that lateral wall movements are generally in the 0.5 mm to 2 mm range per metre depth of excavation in rock (see Figure 7).

Deformation within rail corridor

The maximum predicted total vertical and horizontal deformations below the existing railway tracks after excavation were about 2 mm and 6 mm respectively (Figure 6). The maximum differential vertical and horizontal settlements below the existing rail track in the rail corridor due to the excavation were calculated at less than 1 mm and 2 mm. Displacement trigger levels for 'Line Alarm Level 1' (corresponding to about 50% of allowable displacement) were 10 mm for the applicable 60 km/h track speed. Lateral movement affects the line value, which is determined by three track locations over 8 m.

The field performance of the Embassy development next to the "Eighty Eight" development was reviewed against the typical rates of movement observed in similar ground conditions (Figure 7). The lateral wall movement at the Embassy development was about 0.5 mm per metre depth of excavation in rock. Trigger levels addressing total serviceability deflection (lateral displacement) of the wall in any one direction were 30 mm next to the Pacific Highway.

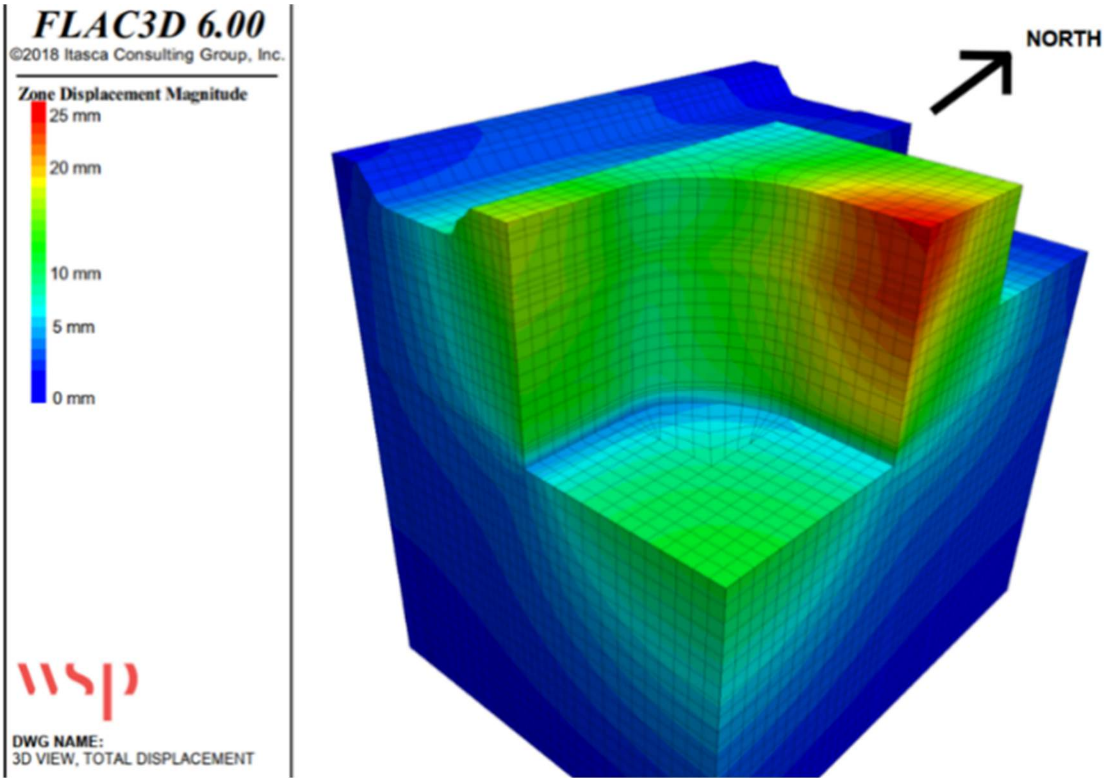

Figure 6. Predicted ground movements from 3D assessment

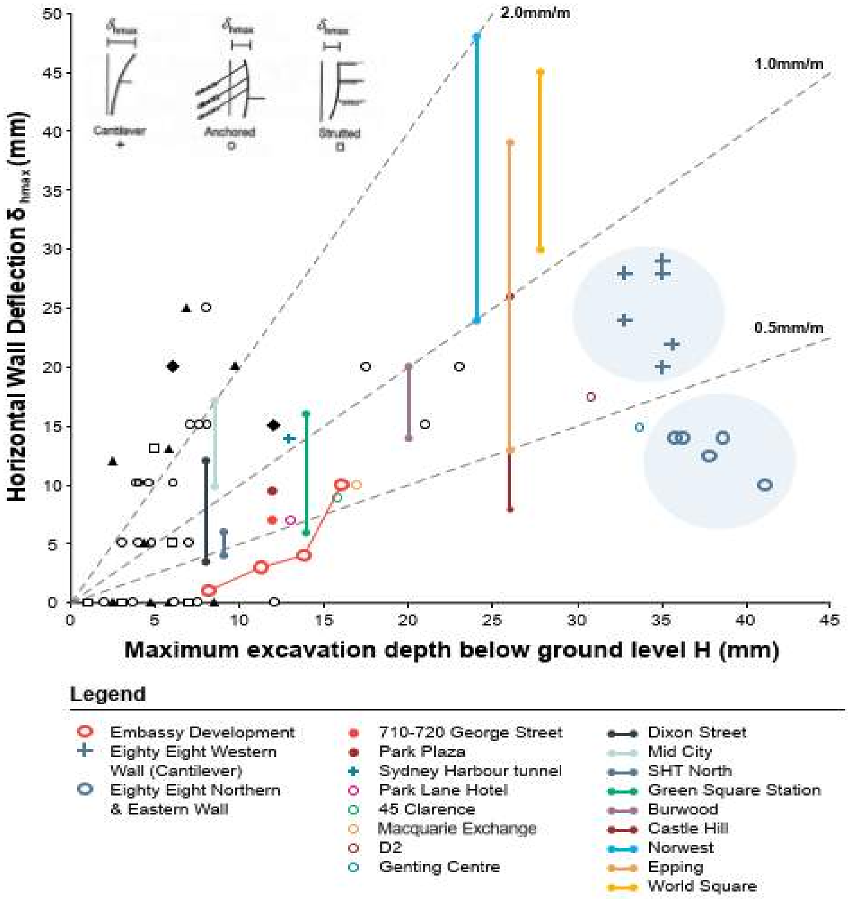

Figure 7. Wall movement database and project field performance

CONSTRUCTION PERFORMANCE

We adopted the observational method with some contingency measures to prevent a SLS or ULS from occurring, as described in CIRIA C760 (Gaba et al, 2017), that is:

- Installed temporary high stiffness anchors at a high level early in the excavation sequence to control ground movements from wall deflection, with waler and allowance for additional pre-stressing.
- Started excavating in the site's south-eastern corner to evaluate actual wall performance, recalibrate ground and analytical models, and identify recalibrated parameters.
- Along northern boundary, excavated 6 m horizontally in 6 m to 9 m wide vertical panels south of the Pacific Highway retaining wall to provide a stabilising "berm" effect.
- Allowed for additional anchoring/cable bolts along potential sub-horizontal shear/laminite planes identified from borehole imaging and installed instruments, ideally before they were exposed/displaced.
- Limited temporary excavation depths along northern boundary if SLS trigger level was approached.

Trigger limits were identified at key construction stages to enable appropriate and timely decisions and for the project team to intervene vis-à-vis how the site retention scheme is performing and how movements are developing compared to the recalibrated and SLS characteristic predictions.

Basement excavation

Examples of laser wall scanning, and the bulk excavation progress, rock condition and shoring adopted along the site boundaries are shown in Figure 8.

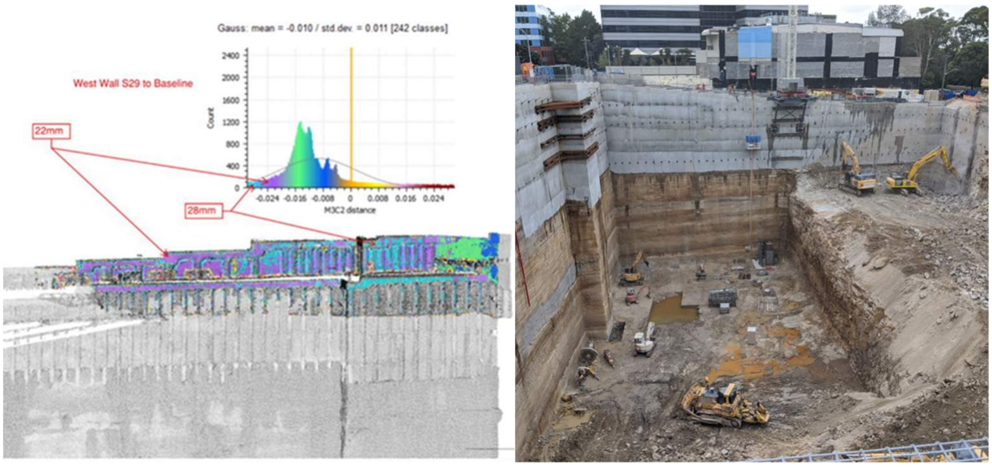

Figure 8. Monitoring and bulk excavation works - view to south

West (railway) wall

The investigation indicated weaker shale/laminite bands at about 40 m deep (see Figures 4 & 9). Hence, we planned to install a temporary high stiffness anchor to control ground movements from wall deflection, depending on the observed displacement.

During the bulk excavation, the automatic inclinometer measured more pronounced horizontal sliding movement on these two shale/ laminite bands. This sliding movement was caused by the release of in-situ stress in the sandstone that allowed the sandstone blocks above the shale bands to move slightly more than predicted. Review of these and other displacement data indicated movements approaching potential decision making triggers which triggered actions to mitigate further movement.

We decided to leave a rock buttress against the wall to be excavated last and install corner propping against the southern rock buttress. To reduce the potential for movement of the railway, we modified the length of the southern 20 m wide rock buttress anchors, so there was an adequate bond length below the shale bands, and installed hydraulic corner propping (Figure 5). Additional anchors were installed on the opposite corner to counteract this force.

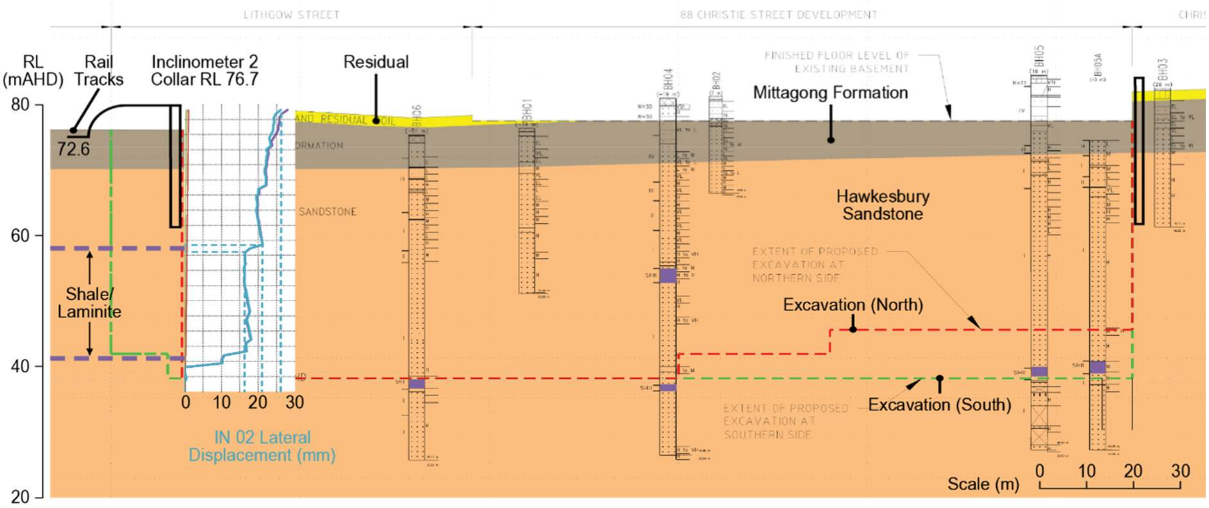

Figure 9. Bulk excavation profile from west to east showing inclinometer movement on shale bands

Instrumentation and monitoring

Comprehensive monitoring continued throughout the works, with strict green, amber and red conditions defined throughout excavation, anchoring/ propping and de-propping stages. A project team objective was to streamline data collection to maximise system and project integration, and shorten the review and decision-making process to improve construction safety. As a general trend, advances in construction monitoring are moving away from physical measurements at limited numbers of points, towards widely distributed, wirelessly connected sensor networks and digital scanning techniques. This data allowed the excavation contractor to optimise construction processes and improve project safety performance. The project's scale and proximity to existing infrastructure (rail, road bridge and pavement, and utilities) made monitoring for safety extra critical.

Methods included surveying deflections and rotation of the walls, laser wall scanning, ground settlement/heave, and rail track. Analyses using the 'traffic light' principle helped set trigger values to anticipate and control excessive ground movements.

As part of controlling the excavation process, we adopted instrumentation and monitoring points (Figure 10). Monitored frequency depended on the excavation pace and was supplemented by regular visual observations. Continuous wireless remote monitoring also helped assure and enable data-driven decisions and allowed action plans to be enacted to protect the public, environment, and workers. Monitoring satisfied the designer that the design's geotechnical models were representative, predictions of the ground and rock support behaviour were accurate, and helped verify compliance with the design requirements.

The rail track geometry was monitored in accordance with ASA Standard ESC 210 Track Geometry and Stability. In the early stages of the bulk excavation, we engaged a track certifier to inspect the track as a baseline, and then inspect the track later during bulk excavation. The maximum horizontal displacement on OHWS monitoring points was 17 mm towards the excavation and the maximum settlement was 6 mm. The maximum measured horizontal wall movement was 28 mm at the mid-point of the west (railway) wall, which was within limits agreed with adjacent asset owners and demonstrated excellent agreement with design predictions.

SUMMARY AND CONCLUSIONS

This paper presents the methodology adopted to predict and monitor the behaviour of the world's deepest (known) basement, constructed adjacent to critical infrastructure assets. Constructing the "Eighty Eight" project required excavating to over 43 m depth. The project demonstrates the many aspects associated with ground investigations, analysis, design and construction of deep basement retention systems that must mitigate against unsatisfactory performance. Impact assessment at the early stages of the project enabled adjacent asset owners to establish realistic risk baselines for the building during early stages of the project which ultimately reduced the project cost and mitigated construction risk.

Designing, excavating and constructing the site retention system incorporated several critical issues, including addressing stringent settlement and angular distortion criteria, construction safety, constructability, and the constraints of defined road and rail reserves.

The design and construction process was successful and effective in addressing all parties' concerns. Meticulous 3-D modelling of the basement excavation and construction process reassured adjoining owners that their assets would not be adversely affected and facilitated a basement design that controlled and contained impact on surrounding properties and infrastructure.

Monitoring data shows the pre-construction geotechnical models and design parameters were appropriate, and that an observation-based approach allows selection of adequate retention support design to manage the risks associated with elevated stress conditions, and changes in stiffness and shear strength within the rock mass particularly at the sub-horizontal bedding defects.

- The accuracy of this type of interaction assessment was greatly influenced by ground model parameters and rock mass properties. This emphasises the value of detailed ground investigations before modelling
- The real-time display of basement walls' movement allowed continuous management of deformation during excavation. The remote monitoring inclinometer system detected horizontal sliding along shale bands early, enabling data-driven decisions which triggered actions to mitigate further movement
- The real-time, automated remote monitoring and precise manual surveying of the railway track geometry and engaging a Track Certifier early in the bulk excavation works, helped address impacts on track geometry
- All track geometry measurements were below alarm levels for the applicable 60 km/h track speed
- Numerical wall prediction movements for the north and west walls (adjacent to critical transport and utilities assets) were in excellent agreement with monitored movements
- The maximum measured horizontal wall movement was 28 mm at the mid-point of the west (railway) wall.

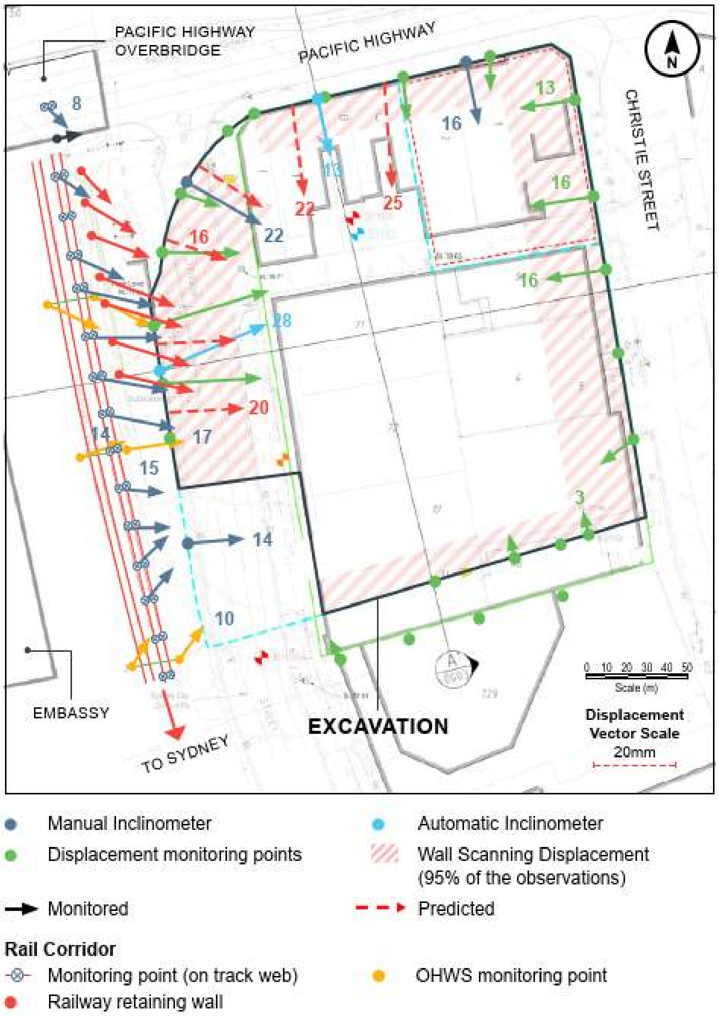

Figure 10. Instrumentation plan showing predicted and monitored displacement

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City Shaping Infrastructure Projects: The Sydney Metro Corridor Protection Development Review Process

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ABSTRACT

Sydney Metro is Australia's biggest public transport project. By 2030, Sydney will have a network of four metro lines, 46 stations and 113km of new metro rail. Sydney Metro is revolutionising how Australia's biggest city travels, connecting Sydney's north west, west, south west and greater west to fast, reliable turn-up-and-go metro services with fully accessible stations.

Safety is paramount. The safe operation of Sydney Metro infrastructure needs to be ensured. Similarly, planned infrastructure must also be protected to ensure feasibility of future metro construction. As such Sydney Metro, under delegation from Transport for NSW (TfNSW), has an obligation to review and ensure potential impacts are managed for developments adjacent to both existing and planned infrastructure. Reviews for development proposals near Sydney Metro underground infrastructure are conducted in line with the publicly available Sydney Metro Underground Corridor Protection Technical Guidelines. These guidelines support the requirements of the rail authority under relevant planning instruments including the State *Environmental Planning Policy (Transport and Infrastructure) 2021—and State Environmental Planning Policy (Precincts—Western Parkland City) 2021. Sydney Metro endeavours to collaboratively engage and work with developers to find technically excellent solutions and opportunities. We achieve this with integrity, trust and transparency. At the forefront of this engagement is the Sydney Metro Corridor Protection Team. The team is committed to supporting opportunities for development, place making and integration with the local precincts without inhibiting the structural integrity, safety and operations of the Sydney Metro rail corridors. This paper summarises how the team reviews a development proposal and works with developers to achieve an optimal solution for all parties. Through engagement the team strives to collaboratively deliver technically excellent, sustainable city-shaping transformations to achieve the best result for Sydney Metro, our communities and Greater Sydney as a whole.*

INTRODUCTION

Sydney Metro has an obligation to review development applications (DA's) of projects adjacent to exisitng and planned infrastructure, on a case by case basis to ensure their consequential impacts are assessed and managed. This paper summarises the Sydney Metro assets to be protected, the legislation under which Sydney Metro operates and the processes by which Sydney Metro reviews development applications.

SYDNEY METRO ASSETS

Sydney Metro's assets comprise several operational and in development lines, over station developments and associated infrastructure. An overview of these assets has been included in the following sections.

Metro North West Line (including the converted Epping to Chatswood Rail Line (ECRL)

The Metro North West Line extends for 36 km from Chatswood through to the north west. The line incorporates 23km of new track (4 km of elevated track, 3 km at grade track 15 km of new tunnels) and 13 km of ECRL tunnels between Epping and Chatswood which have been modified and converted to form part of the new line.

The 15 km of new tunnels comprise twin bored running tunnels with a 7.0 m external diameter and are generally supported using pre-cast concrete segmental lining, except for mined tunnels between the Epping Service Facility and Epping Station where in-situ concrete has been used. There are 61 mined cross passages supported using permanent cast in-situ concrete lining. Other structures include mined nozzle enlargements at Castle Hill, Hills Showground and Norwest and a crossover cavern east of Castle Hill Station, all supported by cast in-situ concrete lining.

The ECRL tunnels are also bored tunnels but with a 7.2 m external diameter. The running tunnel support generally consists of temporary primary support using rock bolts and shotcrete, and final support using unreinforced cast-in-situ concrete lining, nominally 200 mm thick. A section of the running tunnels was lined with shotcrete for construction reasons.

There are 13 stations in total with eight new stations built and five stations upgraded for the ECRL. Of the eight new stations; Castle Hill, Showground and Norwest are contained within cut and cover concretre boxes; Cherrybrook and Bella Vista follow an open cut configuration; Kellyville and Rouse Hill are elevated and Tallawong is at grade.

The metro stations at North Ryde, Macquarie Park and Macquarie University are comprised of large span platform caverns and concourse caverns whilst Epping Station comprises two platform caverns connected by cross passages beneath the existing surface station.

Sydney Metro City & Southwest

The Sydney Metro City & Southwest comprises a new 30km metro line extending rail from the end of the Metro North West Line at Chatswood, under Sydney Harbour, through new CBD stations and southwest to Bankstown.

Sydney Metro City & Southwest will deliver new metro stations at Crows Nest, Victoria Cross, Barangaroo, Martin Place, Pitt Street, Waterloo and new underground metro platforms at Central Station. In addition it will upgrade and convert all 11 stations between Sydenham and Bankstown to metro standards.

The city section consists of a short section of surface track from Chatswood Station to the dive and portal structure then underground infrastructure that extends under St Leonards, Crows Nest, North Sydney and Sydney Harbour and then beneath the Sydney CBD to Central and Waterloo and through to Sydenham, where the metro comes to the surface at a portal and dive structure at Marrickville.

Twin running tunnels approximately 14 km in length were excavated using tunel boring machines (TBM's) and supported using a precast concrete segmental lining. to create a watertight environment. The tunnels predominantly align through siltstone and sandstone, except below the Sydney Harbour where TBM tunnelling was required through marine ground sediments for a length of around 170 m.

A total of 57 mined cross passages are located between running tunnels at regular intervals, with spacing of around 240 m. The cross passages were excavated using mechanical methods and are supported using a permenant tanked lining, formed using permenant cast in-situ concrete lining. A services shaft connects with a cross passage at Artarmon. The shaft is supported by permanent cast in-situ concrete lining.

Waterloo Station, Barangaroo Station, Crows Nest Station and the underground metro platforms at Central Station are cut and cover box structures which contain island platforms. Pitt Street Station and Martin Place Station have binocular platform caverns that connect with two entrance and services shaft structures, whilst Victoria Cross Station has a single span cavern with an island platform, which also connects with two entrance and services shaft structures.

A mined cross over cavern was constructed immediately north of Barangaroo Station. Mined tunnel enlargements are provided to house tunnel ventilation equipment at either end of the Victoria Cross Station caverns, the northern end of the rail crossover at Barangaroo, the southern end of Waterloo Station and at the northern end of Crows Nest Station. The nozzle enlargements were excavated using mechanical methods and supported using a tanked permanent lining, formed using cast in-situ concrete. Dive structures and portal structures are located at Marrickville and Chatswood. A stabling yard is located at the Marrickville portal site.

The Southwest section is all at grade or elevated and is currently part of the T3 Bankstown Line, and will be converted to metro standards from Sydenham to Bankstown.

Eleven existing stations at Sydenham, Marrickville, Dulwich Hill, Hurlstone Park, Canterbury, Campsie, Belmore, Lakemba, Wiley Park, Punchbowl and Bankstown will be upgraded to improve accessibility for customers and meet the standards required for metro operations.

Sydney Metro West

This 24 km underground railway will connect Greater Parramatta and the Sydney CBD with nine stations at Westmead, Parramatta, Sydney Olympic Park, North Strathfield, Burwood North, Five Dock, The Bays, Pyrmont and Hunter Street in the CBD, and a stabling and maintenance facility at Clyde. Tunnels will generally be bored using tunnel boring machines with short mined sections leading in stations and crossover caverns. Stations will include a mix of underground ‘cut and cover’ boxes and single span mined caverns. Turnback caverns will be located at Westmead and Hunter Street. All three tunnelling contract packages have been awarded and construction began in 2020.

Sydney Metro - Western Sydney Airport

Sydney Metro - Western Sydney Airport line (SM-WSA) is a 23km new rail linking St Marys through to the new airport and the Western Sydney Aerotropolis. The line extends south from St Marys and connects via twin running tunnels to the Claremont Meadows services facility, followed by Orchard Hills Station with a portal dive structure nearby. Moving further south is Luddenham Station.There are two stations within the airport site, at the airport terminal and the airport business park. There is a services facility at Bringelly. The line ends at Aerotropolis Station and excavation site. The tunnelling contract was awarded in December 2021. In March 2022, the contract to deliver 10.6 kilometres of elevated viaduct, earthworks for track formation, a rail bridge over the new M12 motorway, a rail bridge within the airport and associated works, was awarded. In December 2022 the largest Public Private Partnership (PPP) contract in New South Wales was awarded to deliver six new stations between St Marys and the new Aerotropolis, 12 new metro trains, core rail systems and the stabling and maintenance facility to be built at Orchard Hills.

Over Station Developments

It should be noted that Over Station Developments and precincts may also form part of Sydney Metro infrastructure assets but Sydney Metro developments are outside the scope of this paper and will not be discussed further.

Associated infrastructure

Sydney Metro also protects infrastructure defined in legislation, which includes approved State Significant Applications for railways and rail infrastructure facilities. State Significant Applications are available on the Department of Planning and Environment's website and define the Sydney Metro project area.

Some rail infrastructure facilities are located away from the railway e.g. construction sites, precast facilities or utilities. A Before You Dig Australia search will indicate if Sydney Metro has existing rail infrastructure facilities in proximity to a site.

DEVELOPMENT APPLICATION LEGISLATION

Depending on the location and scope of a proposed development, the *State Environmental Planning Policy (Transport and Infrastructure) 2021* (T&ISEPP) or *State Environmental Planning Policy (Precincts – Western Parkland City) 2021* (Western Parkland City SEPP) may trigger the requirement for referral for comment or concurrence from Sydney Metro for works in the vicinity of a rail corridor.

The terms 'referral' and 'concurrence' are defined in the NSW Department of Planning and Environment's 'Development referrals guide' as:

> *A **referral** is where the consent authority must refer certain DAs to a referral authority where required under the legislation. This requirement is usually in an EPI and is typically for consultation purposes to obtain advice from the referral authority.*

For example, under *State Environmental Planning Policy (Transport and Infrastructure) 2021*, Chapter 2 (Infrastructure) section 2.98, where certain criteria are met, the consent authority must consult with Sydney Metro before determining development in or adjacent to rail corridors.

> ***Concurrence** is when agreement from a referral authority must be obtained before the council can determine a DA. Concurrence requirements are typically identified in environmental planning instruments (EPIs), but also exist in other legislation such as the NSW Roads Act 1993.*

For example, under *State Environmental Planning Policy (Transport and Infrastructure) 2021*, Chapter 2 (Infrastructure) section 2.99, where certain works are proposed within a specified distance to a Sydney Metro rail corridor, the consent authority must not grant consent to development without the concurrence of Sydney Metro.

A development application is required to list any authorities from which concurrence must be obtained before the development may lawfully be carried out.

If concurrence is required by Sydney Metro, the rail authority is required to consider the potential effects of future development on the safety or structural integrity and the safe and effective operation of existing or proposed rail infrastructure facilities.

Transport for NSW (TfNSW) has delegated its rail authority functions in relation to the Sydney Metro corridors to Sydney Metro. TfNSW is generally the rail authority for all other existing and future heavy passenger rail. If the proposed development requires comment or concurrence from TfNSW, separate documentation related to TfNSW rail infrastructure must be provided and will generally be dealt with separately.

Depending on a proposal's planning approvals pathway, Sydney Metro may not be formally referred a proposal for comment or concurrence despite the potential for impacts on the Sydney Metro infrastructure. An example may be a proposed activity by a public authority under Part 5, Division 5.1 of the *Environmental Planning and Assessment Act 1979*. Notwithstanding this, in order to ensure the appropriate management and mitigation of the potential impacts of a proposal on the Sydney Metro infrastructure, Sydney Metro would encourage applicants to notify Sydney Metro if their proposal would otherwise trigger legislation to protect both their investment and Sydney Metro infrastructure.

In addition to legislation to protect Sydney Metro infrastructure, Sydney Metro acquires sub-surface stratum for the construction and operation of underground rail facilities. The *Transport*

Administration Act 1988 (TA Act) provides Sydney Metro with the power to acquire the land and the land is acquired pursuant to the *Land Acquisition (Just Terms Compensation) Act 1991* (JT Act). The JT Act sets out the statutory process that Sydney Metro must follow when it is necessary to acquire land using a compulsory process.

Engineering and design define protection requirements to determine typical land take offsets for bored running tunnels, mined cross passages, crossover caverns and station boxes. Based on the fixed tunnel centre alignment and Land Registry requirements, the land take is a volumetric rectangular prism.

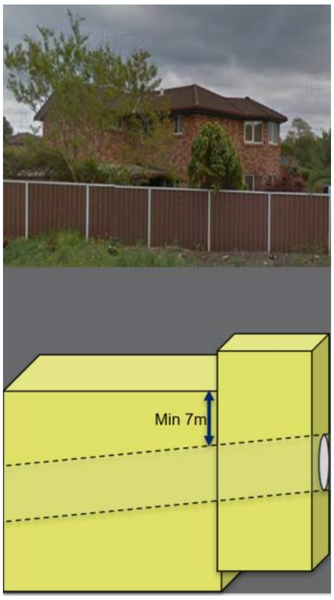

Figure 1. Schematic example of stepped substratum beneath a site

Sydney Metro typically acquires a 7m envelope around each running tunnel. The preferred vertical acquisition envelope (from extrados) is 10m for station caverns. These acquisitions are to make allowance for tunnel driving tolerances, construction access requirements and spatial provisions for the temporary ground anchors or earth rods. Patterned rock bolts are required to support the mined cross passages that span between running tunnels at intervals of around 240m.

Sub-stratum land acquired by Sydney Metro fits the definition of land in the first reserve protection zone. There are restrictions on what excavations and structures can be placed in the first reserve as outlined in the Guidelines. A concession may be endorsed by Sydney Metro for developers to locate discrete structures within the first reserve. However, Sydney Metro endorsement is dependent on developers demonstrating that they have considered multiple engineering options and there are no viable alternate options.

GUIDELINE DOCUMENTS

To assist external developers in the planning, design and construction near Sydney Metro infrastructure, two guideline documents have been produced, referred to within the remainder of this paper as the guidelines. Sydney Metro recommendations to the consent authority will generally be in line with these documents:

- Sydney Metro Underground Corridor Protection Technical Guidelines (Ver 2, 2021)
- Sydney Metro At Grade and Elevated Sections Corridor Protection Guidelines (Rev 0, 2018)

The Underground Guidelines generally apply to proposed developments near Sydney Metro running tunnels and other underground infrastructure whilst the At Grade and Elevated Guideline generally applies to viaducts, station precincts, operational services buildings, emergency evacuation points, at-grade sections and embankments and cuttings.

These guidelines support the requirements of the rail authority under relevant planning instruments to protect the safety, structural integrity and the safe and effective operation of existing or proposed rail infrastructure facilities from adjacent developments.

The guideline documents cover proposed developments near the following existing, under construction and future metro lines:

- Metro North West Line including the Sydney Metro converted Epping to Chatswood

  Rail Line (ECRL) section
- Sydney Metro City & Southwest
- Sydney Metro West
- Sydney Metro - Western Sydney Airport and
- Other future Sydney Metro corridors.

Information regarding existing and planned new metro infrastructure can be sourced from Sydney Metro.

CORRIDOR PROTECTION TEAM

Any DA referred to Sydney Metro for comment or concurrence will be reviewed by the Sydney Metro Corridor Protection Team. This team leads the provision of specialist advice and manages the identification, assessment and protection of corridors required for current and future metro infrastructure. The corridor protection team balances the need to manage the orderly development of land adjacent to and over metro stations and infrastructure with corridor protection requirements. The team comprises planners, analysts, engineers and subject matter experts (SME's) in the fields of:

- Geotechnics
- Tunnels
- Electrolysis
- Structures
- Acoustics
- Building sciences (wind assessments etc.)

The team manages and assesses the interfaces between external stakeholders and Sydney Metro. Where Sydney Metro may have a referral or concurrence role, the planners are usually the first point of contact within the corridor protection team. Generally the planners are allocated to specific Sydney Metro corridors. The planners will then notify the SME's of a development.

THE DEVELOPMENT APPLICATION

A development application (DA) is a formal application for a development that requires consent under the NSW Environmental Planning and Assessment Act 1979. The application is made to the consent authority, usually the local council. The planning approval stages for a development are usually:

- Pre-development application consultation / meeting (non-mandatory and as-required)
- Development application lodgement
    - Concept development application (where proposed)

    - Detailed development application
- Post development
    - Condition compliance

A summary of the typical submissions and review process at the DA stage are summarised below.

Pre-development Application Stage

A DA is required to list any authorities from which concurrence must be obtained before the development may lawfully be carried out. This puts the onus on the development team to identify whether there is the potential for Sydney Metro to have a concurrence role. An urban planner will be able to advise the development team when legislation may be triggered to require referral or concurrence to Sydney Metro. The NSW Department of Planning and Environment's 'Development referrals guide' may also assist the development team in this regard.

Prior to lodging a DA the development team should contact Sydney Metro to obtain information about the location of metro infrastructure near their site and download a copy of the Guideline documents from the Sydney Metro website.

Acceptance or rejection of a concurrence role for development in proximity to Sydney Metro railway infrastructure depends on whether the development meets specific criteria defined in the legislation. Typically it is triggered where proposed excavations or structures are likely to extend within the first or second protection reserves. This can be demonstrated by the development team by submitting survey drawings to Sydney Metro which clearly show:

- Sydney Metro substratum
- First reserve (if different to the substratum)
- Second reserve and;
- Proposed development including:
    - temporary excavations and/or structural elements
    - permenant excavations and/or structural elements

The following can be carried out or made reference to in order to determine corridor protection zones and the location of the Sydney Metro infrastructure and substratum (if relevant):

- Request the location of the Sydney Metro infrastructure for the proposed development site (refer to Section 11 for Sydney Metro contact details).
- Stratum information (where available) can be obtained through:
    - The owners who were notified of the location of the stratum as part of the acquisition process;
    - The survey plans of acquisition registered with Land Registry Services, NSW (a registered surveyor should be able to assist with this);
    - Before You Dig Australia Service.

It is recommended that experienced and qualified specialists be engaged early as part of the development team.

If the DA is lodged and referred to Sydney Metro for comment or concurrence, Sydney Metro can determine that a proposal does not trigger legislation, then Sydney Metro will reject the referral for comment or concurrence and generally no submission will be made by Sydney Metro. Clarity on whether the proposal triggers the legislation in the associated planning documentation will assist with this process.

If concurrence is triggered for example proposed excavations or structures greater than two metres will enter into the first or second protection reserves then Sydney Metro will accept the referral for a concurrence role through the NSW Planning Portal. The triggering of legislation requires the developer to submit a number of documents as part of their development application.

The information provided in the guidelines should enable developers to lodge the required documentation with their DA without the need for a meeting with Sydney Metro. However, it is understood that in some situations, where the development is located directly over Sydney Metro infrastructure, that developers may want to meet to discuss their preliminary design. In this situation a request should be sent to Sydney Metro.

Detailed Development Application Stage

Where Sydney Metro has a concurrence role, developers for proposed developments must submit a number of documents with their development application.

A comprehensive list of required documents and recommendations on what should be included within these documents can be found within the guidelines. A brief overview of what may be required is provided below:

- Survey plans
- Cross sections
- Geotechnical investigation report
- Impact assessment report
- Risk assessment report
- Instrumentation and monitoring plan (where deemed necessary by Sydney Metro)
- Noise, vibration and electrolysis studies and control measures if available (for low risk developments these may be submitted prior to the Construction Certificate stage for developments considered by the corridor protection SME’s to be at low risk).

For detailed DAs all the required documents must be submitted to an acceptable level of detail for Sydney Metro to be able to respond to the matters for consideration in the legislation. Documents should adequately reflect the design intent and preliminary documents will not generally provide the level of detail required for review. Insufficient detail will likely extend the DA review process.

Concept Development Application Stage

Concept development applications set out a high level concept proposal for the development of a site. Detailed proposals for the site or for separate parts of the site are to be the subject of a subsequent development application or applications. Sydney Metro will consider the likely impact of the concept proposals (and any first stage of development included in the application). Where legislation requires referral or concurrence in relation to Sydney Metro rail corridors for proposed developments, the developer must lodge the following documents as part of their concept development application package:

- Geotechnical desktop study and concept foundation design
- A detailed survey plan and cross sections (as per the pre-development application)

Subsequent detailed DA(s) will be referred to Sydney Metro for comment or concurrence in accordance with legislation.

Supporting Documentation

The guidelines should be consulted to determine what supporting documentation is required for each step of the planning process. This section provides an overview of the documentation required.

Survey plan and cross sections

Survey plans should be detailed and prepared by a NSW registered surveyor, which accurately defines the boundaries between the development, the rail corridor (including the first and second reserve), rail infrastructure and any Sydney Metro easements (including right of ways) or stratums, covenants and caveats.

Cross sections

Cross section drawings showing the rail corridor (including the first and second reserve), any proposed basement and/or foundation excavations and any temporary and permament structural elements. All measurements contained within the cross-section drawings must be verified by a registered surveyor.

The purpose of these plans and sections is to demonstrate the location of any proposal in relation to the Sydney Metro infrastructure. The Underground Guidelines outline what is permissible within the first and second reserves but generally, no excavations or structures (either temporary or permanent) are permitted within the first reserve (geotechnical investigations and instrumentation excluded). These drawings should therefore, as a minimum, demonstrate all proposals lie outside the first reserve.

It is expected that developers first consider all possible measures to keep any construction activity outside the first reserve. Should it be demonstrated to Sydney Metro that all possible measures to remain outside the first reserve are unviable and hence make the proposal development unviable, alternate measures that include limited intrusion into the first reserve may be considered by Sydney Metro. Such a process will require Sydney Metro to endorse a concession, land owner consent and a deed. The deed process requires legal negotiations between both parties. The preference is to enter into this process only for exceptional cases.

Geotechnical investigation report

Section 7.1 of the Underground Guidelines outlines what is required for a Geotechnical Investigation Report. The intent of the report is to provide information from which geological models can be developed for the interface between the proposed development and Sydney Metro infrastructure. This Geotechnical Investigation Report should provide the basis for the ground models and parameters used for any future impact assessment.

The Underground Guidelines do not explicitly state what level of investigation is expected. This is for the development team to decide. The investigation and report, however, should be sufficiently detailed to adequately describe the ground at the development / infrastructure interface with a level of detail comensurate with the potential risk the development poses to the Sydney Metro infrastructure. The onus will be on the development team to demonstrate to the Sydney Metro SME’s that adequate characterisation has been developed. Clarifications may be sought via Requests for Information or, for more nuanced technical matters, through meetings and discussions between Sydney Metro and the development team SME’s.

Impact assessment report

The requirements for an Impact Assesment Report are detailed within Section 7.2 of the Underground Guidelines. The purpose of this report is for the development team to demonstrate that the effects of the proposed development on tunnels and underground facilities will not cause unacceptable adverse impacts on future or existing Sydney Metro infrastructure.

As per the investigation report, the Underground Guidelines do not explicitly state the level of detail expected. The onus is on the development team to demonstrate to the Sydney Metro SME’s that risks have been sufficiently mitigated. The level of detail and analysis techniques are expected to be appropriate and comensurate with the potential risks posed to Sydney Metro Infrastructure. The mechanism by which the infrastructure may be impacted should be stated and the adopted analysis technique should be designed to adequately quantify the expected impact. The guidelines do not require specific technique(s) (ie numerical modelling) to be employed, however, any employed technique(s) are expected to:

- Capture the identified impact mechanism
- Conservatively quantify the expected impact
- Be comensurate with the level of detail of the input parameters (ie the investigation report).

Where more advanced analytical techniques are adopted, it may be expected that elements of the analysis be calibrated against simpler models and/or instrumentation (site specific and/or similar instrumentation in similar ground conditions published within the literature).

Dependent upon the level of risk posed by a development proposal Sydney Metro may require a full structural assessment of its assets to ensure the design life of the asset will not be compromised by the development.

The report should be carried out by specialists with the right level of competency which should be informed by the complexity and level of risk. The Underground Guidelines outline what Sydney Metro consider to be appropriate minimum competency levels.

Where neccesary, dependent upon the complexity and potential risks, Sydney Metro may request the development team arrange independent verification of the engineering analysis and impact assessment. Requirements for an independent verification are outlined within Section 7.9 of the Underground Guidelines.

Risk assessment report

Developers have a legal duty to eliminate risks to ensure safe rail operations So Far As Is Reasonably Practicable (SFAIRP). As such developers must identify all reasonably foreseeable safety risks and hazards to the metro or its operations and eliminate these risks where reasonably practicable and where it does not minimise each risk.

Instrumentation and monitoring plan

The purpose of instrumentation and monitoring is to validate design assumptions and to demonstrate impacts are within acceptable pre-agreed limits. The requirement for instrumentation and monitoring will be determined by Sydney Metro based on the expected impacts of any development.

Noise and vibration assessment

The purpose of the noise and vibration assessment report is to demonstrate compliance with noise and vibration requirements outlined in *Developments Near Rail Corridor and Busy Roads – Interim Guideline, Department of Planning, NSW Government 2008 and T HR CI 12051 ST Developments Near Rail Tunnels.* The assessment must address ground or structure borne noise emissions from rail activity and should be completed by an appropriately qualified acoustic consultant. The assessment report must also document all assumptions and inputs used to demonstrate compliance.

Consideration should be given to whether section 2.100 of the T&ISEPP is triggered for impacts of rail noise or vibration on non-rail development. If triggered, measures should be outlined to ensure consistency with the requirements.

Electrolysis assessment

Corrosion of buried metallic structures located near electrified railways due to electrolysis is a significant concern and cost to infrastructure owners. Electrolysis assessment and introduction of mitigation measures during the design process as outlined in T HR EL 12002 GU - Electrolysis from stray DC current can reduce the need for expensive cathodic protection systems. Cathodic protection systems are regulated by the Electricity Supply (Corrosion Protection) Regulation 2020 and require coordination and approval by the NSW Electrolysis Committee.

Direct current (DC) in the presence of an electrolyte (i.e. soil) will result in corrosion of metal in the vicinity, in a process known as electrolysis. DC railways have leakage of stray DC current through the rail insulation system which will enter any metallic path in order to return to the source of supply. AC railways do not produce stray DC current in normal operation as the power is generated from alternating current (AC) rather than direct current , therefore any leakage current is alternating and does not meet the criteria for electrolysis. AC railways and power networks, as well as metallic utilities, can pick up and be carriers of direct current and therefore may still present an electrolysis risk to buried metallic structures.

For infrastructure built in close proximity to electrified railways, developers are required to engage an electrolysis expert to prepare a report on the electrolysis risk to buried metallic structures due to stray DC current. In cases where the electrolysis risk is assessed to be significant (i.e. where buried metallic assets will be located in close proximity to a DC railway or existing cathodic protection system) the electrolysis assessment and subsequent inclusion of mitigation measures in design should be undertaken at the Development Application stage. Where the proposed development is assessed to be at low risk of electrolysis no further analysis is required and the report can be provided to the Certifier with the application for a Construction Certificate.

Dilapidation surveys

Dependent upon the risk posed by a development proposal Sydney Metro may request a pre and post dipadidation survey be submitted. The purpose of these reports is to establish the condition of the Sydney Metro asset pre construction and record any deterioration during the development construction phase.

Minimum requirements for such surveys are outlined within Section 8.2 of the Underground Guidelines. It is recommended that both pre and post construction surveys document the entirety (as far as is reasonably pacticable) of the asset within the extent of the survey with high quality images. Should any defects noted within a post construction survey not have been explicitly identified during the pre-construction survey, a full photographic record will possibly enable Sydney Metro to establish whether such a defect had been missed or is in fact a new defect.

The onus will be on the developer to demonstrate that any new defects were not a result of their construction activities. This may be demonstrated through location, reference to submitted analyses and/or monitoring data. Any defects deemed to be caused by the developers construction activities will be required to be remediated by the developer. Where such defects were remediated by Sydney Metro, these costs will be recovered from the developer.

Drainage report

Where relevant Sydney Metro may request that a drainage report is prepared that details the proposed means of drainage that will be adopted to manage the collection of water, including groundwater, within basement levels of the proposed development.

Design Changes and Modifications

Design changes may be required at various stages of the planning approval process. The planning approval pathway will be different depending on what changes are required and at what stage the DA is at and may include:

- **Amendment** of a DA that has been lodged but not yet determined
- **Modification** of an approved DA (also known as section 4.55 applications) or
- **New DA**.

Modifications can be made to an approved DA if the modifications result in the development being substantially the same as the originally approved development, otherwise a new DA is required.

Modifications to an approved DA and a new DA will both require applications to be lodged with the consent authority through the NSW Planning Portal. Typically the DA number for a modification will retain the original number with an additional letter at the end. A new DA will be given a new number.

The consent authority will consider if legislation is triggered for both Modifications and new DAs and will refer them to Sydney Metro for comment or concurrence if required. Developers should always seek the advice of an urban planner in deciding what planning approval path is required for a design change.

REFERRAL OR CONCURRENCE RESPONSE

Where the information lodged with a DA does not enable Sydney Metro to be able to respond to the matters for consideration under the legislation, Sydney Metro will request further information via the consent authority. Under certain conditions a request for information will “stop the clock” on the assessment of the DA. This can increase the time taken for a consent authority to approve a DA.

Alternatively if adequate information has been provided Sydney Metro may provide comments or will recommend concurrence with conditions to be imposed on the consent which may include different requirements at different stages of the development. Sydney Metro will respond to the consent authority through the NSW Planning Portal.

The onus is on the developer’s team to assess the level of detail and analysis required. The Guidelines have been designed to help the development team and reduce the need for RFI’s and meetings. The guidance within these guidelines should first be exhausted prior to any communications or requests for meetings. Any communication and discussion can then deal with specific issues and not general advice.

POST DA APPROVAL

Once a DA is approved there may be conditions on the consent that were requested by Sydney Metro and imposed by the consent authority in the Notice of Determination. Sydney Metro may be required to confirm that the applicant has complied with the conditions imposed. Evidence should be provided to show compliance with the conditions. Generally the applicant should consolidate responses to address conditions of the post approval stage. The stages and general submissions are outlined below.

Construction Certificate (CC) Stage

A Construction Certificate (CC) can only be issued when all of the conditions required prior to construction have been complied with. Sydney Metro will request further information to resolve design issues prior to determination by the consent authority. Conditions recommended to the consent authority from Sydney Metro for compliance prior to CC may require:

- The certifier to provide Sydney Metro with written confirmation the works have been undertaken in accordance with the documents supplied to Sydney Metro under the DA process
- A detailed monitoring and action plan
- Safe work method statements
- Pre-construction dilapidation survey
- Noise and vibration study and incorporation of control measures
- Electrolysis study and incorporation of control measures
- Risk assessment and management plan
- Evidence of public liability insurance (level of insurance to be determined by the Sydney Metro Corridor Protection Team)

- Written evidence of lodgement of a bond or bank guarantee
- Hydrologic assessment demonstrating dewatering during construction will not have any adverse impacts on the rail corridor
- Assurance drainage is not discharged into the rail corridor.

Construction Stage

DA conditions may require sumbmission of documentation during the construction process. Submissions may include:

- Monitoring reports
- Construction progress
- Interim dilapidation reports

Occupation Certificate (OC) Stage

An Occupation Certificate (OC) can only be issued when the OC conditions have been complied with. OC conditions may include:

- As-built structural drawings (foundations and support details)
- Monitoring summary reporty
- Post-construction dilapidation survey (detailing any dilapidation between the pre and post construction surveys)
- Acoustic assessment report

SUMMARY

This paper provides a brief overview of the processes by which Sydney Metro protects its infrastructure from impacts from external developments. Sydney Metro recognises the need to develop land adjacent to infrastructure, and, through the review process Sydney Metro strives to safely and collaboratively deliver technically excellent, sustainable city-shaping transformations to achieve the best result for Sydney Metro, our communities and Greater Sydney as a whole.

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Development near underground rail corridors – engineering assessment with case studies

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ABSTRACT

Development near underground rail corridors can have adverse impacts on the structural integrity of rail infrastructure. Therefore, it is important to demonstrate that developments will not compromise the structural integrity of existing rail infrastructure and operation of rail lines. Deep excavations and foundations, if poorly designed and implemented, can potentially induce significant ground deformation, alter loading profiles on underground structures and other engineered features, and as such, can adversely impact existing rail infrastructure. The protection of underground rail corridor is secured through the application of processes, whereby building development that physically falls within these corridors, require additional consent or concurrence from the affected rail operator to gain development approval.

Over the last 10 years, the authors have been engaged by rail authorities in New South Wales to review numerous impact assessments of proposed developments, submitted by developers as part of the process of underground rail corridor assessment and protection. This paper presents a general discussion of the methodologies and approaches that have been adopted to assess the impacts of new development on the existing or planned rail corridors, in accordance with relevant guidelines and standards. Some typical case studies are discussed regarding the approaches that have been applied are part of the engineering assessment to quantify the influence of building foundations/deep basement excavation on underground infrastructure.

INTRODUCTION

The construction of buildings around underground rail infrastructure is an increasingly important issue due to the rising density of cities. Development near rail corridors can impact on the structural integrity of the transport infrastructure. Existing underground rail infrastructure is typically protected through the establishment of reserved spatial corridors, which function to secure safe rail operation from the future expansion of city infrastructure. This is achieved through the establishment of mechanisms or processes in compliance with standards and guidelines; whereby building development that physically falls within these corridors requires additional consent or concurrence from the affected rail operator to gain development approval.

This consent is needed as building development near protected corridors, if unchecked, could have undesirable impacts on the structural integrity of existing underground rail infrastructure (such as tunnels, caverns and surface excavations). For instance, poorly designed and implemented earthworks or deep excavations can cause subsidence, alter loading profiles on tunnels and other engineered features and, in a worst scenario, cause structural failure or even collapse.

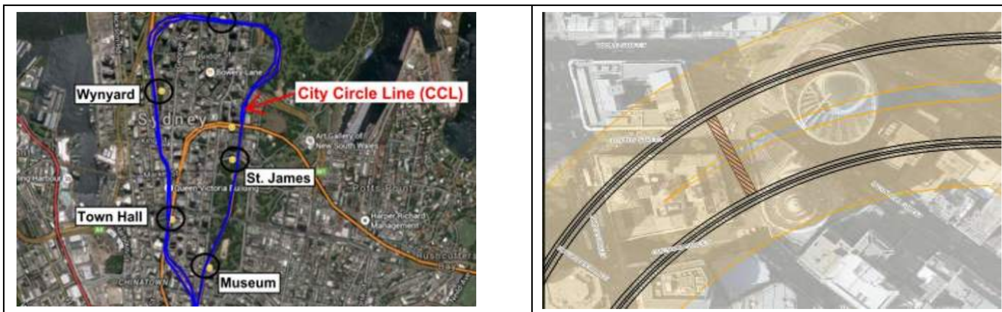
To protect a city’s current transportation corridors, as the built environment continues to expand, requires rail authorities and developers to have access to trusted technical advisers. These advisors must have relevant experience and background to provide practical advice that is underpinned by sound engineering principles.

The authors have been engaged by rail authorities in Sydney, Australia, over a number of years to provide technical advice on the assessment of developments adjacent to rail corridors. This paper discussed the process and engineering principles that have applies to conduct engineering impact assessments, as is illustrated with case studies, to ensure that the construction of the proposed developments did not result is unacceptable impacts to existing rail assets.

DEVELOPMENT SITE AND UNDERGROUND RAIL CORRIDORS

Rail Corridors

Within major cities there may be multiple rail authorities, each of whom are responsible for protecting their respective rail corridor (Figure 1). Invariably each operator imposes their own set of protection guidelines or mandated requirements.



Sydney Trains' City Circle Line | Sydney Metro City & SW

Figure 1. Typical preliminary information of rail corridors, NSW Australia

The scale of development, particularly the proposed basement levels, potential foundation types due to site ground conditions could have impacts on underground rail infrastructure. Local experience from previous projects indicate that the design requirements associated with uplift forces to foundations due to the presence of groundwater level or the application of wind loading to buildings require special attention. Notwithstanding, the most important step for any developer is to appreciate the physical position of their development in relation to the infrastructure within the rail corridor. This understanding can be problematic where the rail infrastructure is underground, and the infrastructure is aging with limited available survey information and as-built records.

Zones of Protection

Once the physical relationship is established and the consent requirements are identified, the developer will then need to reference the relevant rail guidelines or requirements to understand the extent and profile of the protected areas or reserves that surround the infrastructure. As shown by Figure 2 below, in the case of a rail tunnel or cavern, this zone may be represented in section as a rectangle that can encompass the tunnels and other underground openings, such as large span caverns. In the case of station boxes this zone or reserve can be represented as an underground offset perimeter that surrounds the sides and base of the station excavation. The purpose of deriving these reserved zones is to protect the existing and planned rail infrastructure from the adjacent development. Table 1 lists some construction restrictions that are typically applied to these protected areas, the details of which are defined in the relevant underground rail corridor protection guidelines (TfNSW 2018, Sydney Metro 2021).

Fundamentally, the aim of an engineering impact assessment, which normally includes the results of geotechnical investigations and detailed engineering analysis, is to prove compliance with the relevant standards or guidelines through the following:

- Prove that no adverse impacts arising from the proposed development are expected on existing underground rail infrastructure.
- Potential impacts from the operation of existing rail lines on the development have been considered and these have been determined by the developer to be acceptable.

The engineering assessments submitted to rail authorities for review generally focus on addressing the impacts of the proposed development on rail corridors. However, the impacts of existing rail operation can be overlooked, sometimes this can delay the issuing of consent from rail authorities.

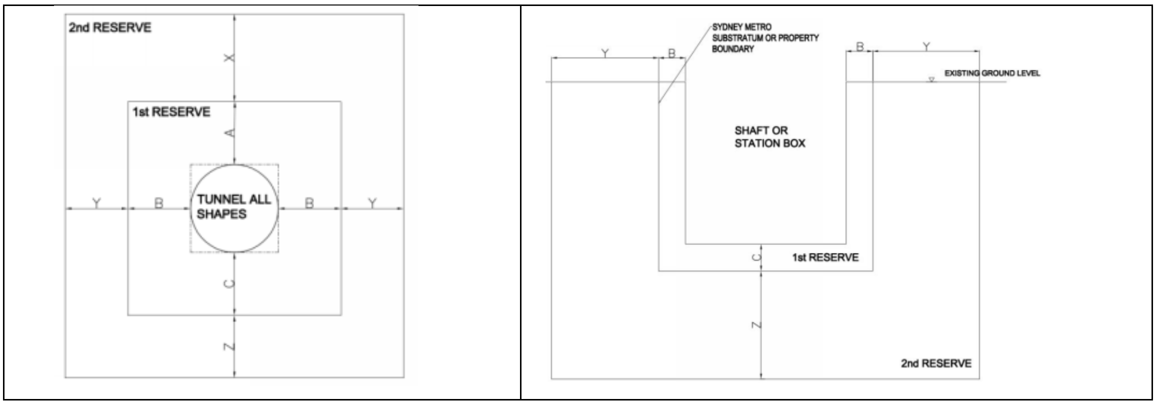


Figure 2. Typical protection reserves for rail corridors, NSW Australia

Table 1. Typical construction restrictions from rail authority

Types of construction	Fist reserve	Second reserve
Excavation for basements,	Not allowed	Excavations < 2.0 m depth from surface footings level, assessment not required. Excavation > 2.0 m depth, assessment required.
Shallow footings or pile foundations	Not allowed	Allowed, subject to load restrictions. Assessment required.
Tunnels and underground excavations	Not allowed	Allowed, subject to assessment.
Ground anchors	Not allowed	Allowed, subject to assessment.
Demolition of existing subsurface structures	Not allowed	Allowed, subject to assessment.
Penetrative subsurface investigations e.g. boreholes, instrumentation	Allowed away from support zone, Assessment required.	Allowed, subject to assessment.

Geotechnical Site Investigation

In the case of either planned or built underground infrastructure, it is important to gain a detailed understanding of the expected ground conditions below and around the development within the rail corridor. In part, this is because the interpreted conditions will inform the subsequent engineering assessments and as such the interpretation needs to be as accurate as possible.

Further, geotechnical inputs need to be quantified to enable a reasonable, safe, cost-effective, and prudent building design. These inputs include developing a ground model both within and outside the site, identifying and characterising the properties of the geotechnical units, understanding the range of potential groundwater conditions, and determining the magnitude and orientation of in-situ stresses.

The developer should also be aware that if the planned boreholes are located within the rail protection reserves, approval from the relevant rail authority is required prior to drilling works. The rail authority might require the location of proposed bore holes within the rail protection zone to be verified by project surveyor. The investigation may, under certain circumstances, proceed only once approval from relevant rail authorities is given.

Engineering Analysis

Where consent is required from rail authorities in Sydney, as triggered by state legislation, the developer may be required to carry out an engineering assessment to demonstrate that the effects of the proposed development on the existing tunnels and underground facilities will not cause adverse physical effects.

Through the consent process, rail operators have the authority to request that developers verify acceptable impacts through engineering analysis and impact assessment. The level of assessment required in invariably governed by the complexity of the project and its potential to physically effect the underground rail structures of concern.

The key technical obligations and accompanying documentation that the developer may need to undertake and submit to gain consent from the rail authorities can include the following:

- Numerical modelling: These assessments will require the preparation of numerical models that represents critical areas of rail/building interface and encompass the specific elements of rail infrastructure of concern. The modelling should incorporate the sequencing of construction and include phased building excavation and the eventual application of foundation loading. Models should include such features as relevant existing or planned structures, proposed building foundations and shoring systems for basement excavation. Further, the models need to accurately reflect the ground profiles, stratification, geological features, and worst credible geotechnical parameters, as derived from the interpretation of the results from site investigation.
- Ground movement and building impact assessment: These types of assessment deal with the severity of building induced potential ground movements and the associated actions on all modelled structural elements. The results from this modelling must be viewed against the acceptance criteria, as detailed in the rail authority guidelines or standards. These place restrictions on key parameters such as ground movements, changes in groundwater levels, stresses within the rock mass surrounding the infrastructure and structural actions as induced on underground support and internal structures.
- Instrumentation and monitoring plan: Depending on the severity of the potential impact, the developer may need to produce and implement an instrumentation and monitoring plan. This plan is needed to measure, review and verify the realised physical impacts are consistent with predictions. Plans typically establish a regime by which ground deformation, tunnel convergence, stresses in the structural support and surrounding rock mass stresses, cracking of the support structure, ground-borne vibration can be monitored. The plans also establish the protocols for reporting and action taking in the event that nominated trigger levels are reached or exceeded.
- Safe work method statements: The preparation of safe work method statements (SWMS) is a fundamental necessity of construction. The developer needs to provide confidence that their building can be constructed safely, without exposing the nearby underground rail assets to unacceptable risk. In some cases, where there is the potential impact of construction is severe, the rail authority and the developer may need to enter into a deed agreement to cover the building construction phase. In

which case, it is a typical requirement that a risk assessment report and safe work method statements are submitted prior to construction, to permit commencement of the works.

– Noise/vibration assessments: These are important assessments that need to be carried out to ascertain the potential impacts of noise and vibration during construction on rail operations and conversely the impact of noise and vibration from rail operation on the completed building. Noise and vibration monitoring form part of the assessment documentation that is submitted to rail authorities.

– Stray current analyses: Trains that are powered by electricity use DC current. The current is delivered by the overhead catenary cables and the return path to the substation is directed through the track. The main concern is that stray currents from the rail can detrimentally impact the design life of the buildings through accelerated degradation of structural elements. As such, suitable DC current mitigation strategies need to be considered and applied to the design of the development.

CASE STUDIES

Each new development and their potential rail impact are unique in terms of its form, features of the rail asset of concern and the ground conditions separating the building from the underground structure. The second case study illustrates the types of assessment that are undertaken on behalf of the developer to assist them with securing consent from the rail authority. These case studies deal with situations where the design of the building structure needed to consider the presence of rail assets that were either comparatively new or aging.

Case Study 1 - Development Adjacent to the Station Cavern and Shaft

The site

This particular development is adjacent to the Epping to Chatswood Line along the north-western site boundary on Waterloo Road, as shown in Figure 3 below, and involved the construction of a multi-storey buildings with three levels of basement excavation immediately adjacent to an existing rail station.

This existing station was completed in 2009 and is configured to include a 30m deep mined large span island platform type station cavern with twin cross adits connecting the cavern from a mezzanine level to the main concourse cavern. At either end of the concourse are escalator shafts that allow passenger access to the station from street level.

As shown in Figure 4, the station cavern is approximately 200m in length with a clear maximum height of 14m and span of 20m; the twin cross adits are 39m in length, each with a span of 12m; the main concourse cavern is 70m in length with an overall span of 16m; and each escalator shaft is 39m in length, 13m in width, with a depth of 36m (Rozek 2004).

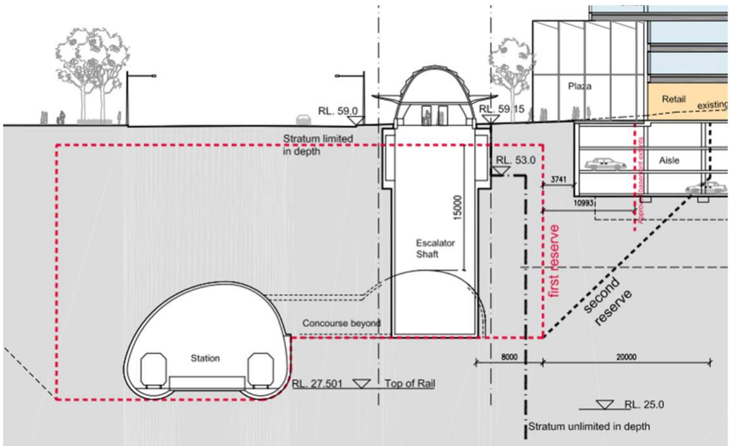


Figure 3. Development site and adjacent rail infrastructure (Protection reserves based on TIDC, 2008)

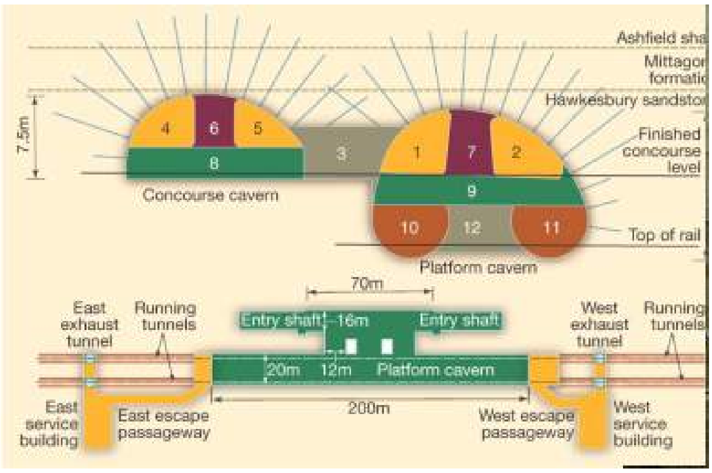


Figure 4. Macquarie Park Station outline details

Site geotechnical profiles

Based on the available geotechnical information and site-specific investigation, the basement excavation for the proposed building development intersected, in descending order, fill materials, residual clays (highly weathered shale) stiff to hard consistency, siltstone (known as Ashfield Shale) of varying strength from very low to medium strength with increasing depth, interbedded siltstone and sandstone (known as Mittagong Formation) of medium to high strength, grading into sandstone (known as Hawkesbury Sandstone) of increasing strength and quality.

Bedding in the Mittagong Formation and the Hawkesbury Sandstone is sub-horizontal, with bedding planes typically spaced at 100-300mm throughout the sandstone units within the Mittagong Formation, ranging up to 1.0m or greater in the siltstone units and in the Hawkesbury Sandstone. Hawkesbury Sandstone is also characterised by cross-beds dipping typically between 15 to 30 degrees, generally towards the north-east.

The station area is also characterized by two orthogonal east-west and north-south striking joint sets. Another geological feature present in the vicinity of the proposed development site is the North Ryde Fault Zone which is located approximately 150m away, from the eastern end of the development.

Engineering assessment

As part of a technical impact assessment the following issues were identified by WSP, who were supporting the developer with this application. The following critical issues were identified as needing consideration by the building design with analysis in some cases required to quantify their relative impact to the station structures:

- The basement retention system needed to ensure that changes in integrity of surrounding rocks and ground movement during basement excavation were minimised.
- Temporary ground anchors used to support the basement excavation were not permitted to encroach into the nominated protection zone.

– A feature of Sydney ground conditions is the presence of high horizontal stress conditions which can cause significant horizontal displacement along horizontal bedding when the stresses are relieved due to open excavation.

– Where this horizontal movement has the potential to occur across discontinuities within the rock mass this movement could damage the integrity of the existing permanent rock bolts supporting the station openings, which are double corrosion protected.

– The building foundation design would need to be such that the zones of load influence would not extend to within the area of the rail infrastructure.

WSP undertook numerical modelling to assist with predicting the impacts of the development and so prove acceptable effects. These models reflected critical cross sections that are shown in Figure 5 and Figure 6. These models incorporated the station concourse, cross adit and platform cavern, proposed basement excavation, geological profiles, and construction sequencing.

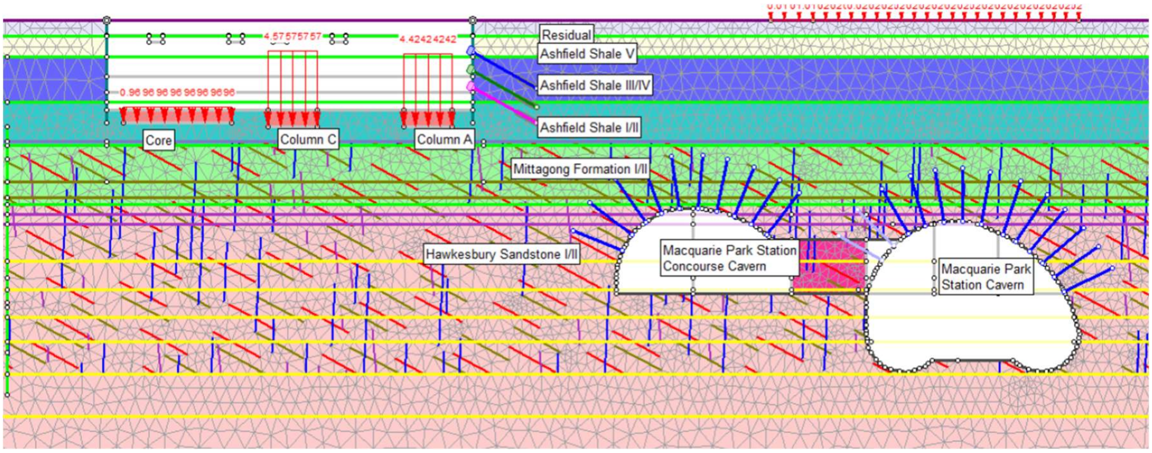


Figure 5. Geotechnical model for the section adjacent to station concourse and cavern

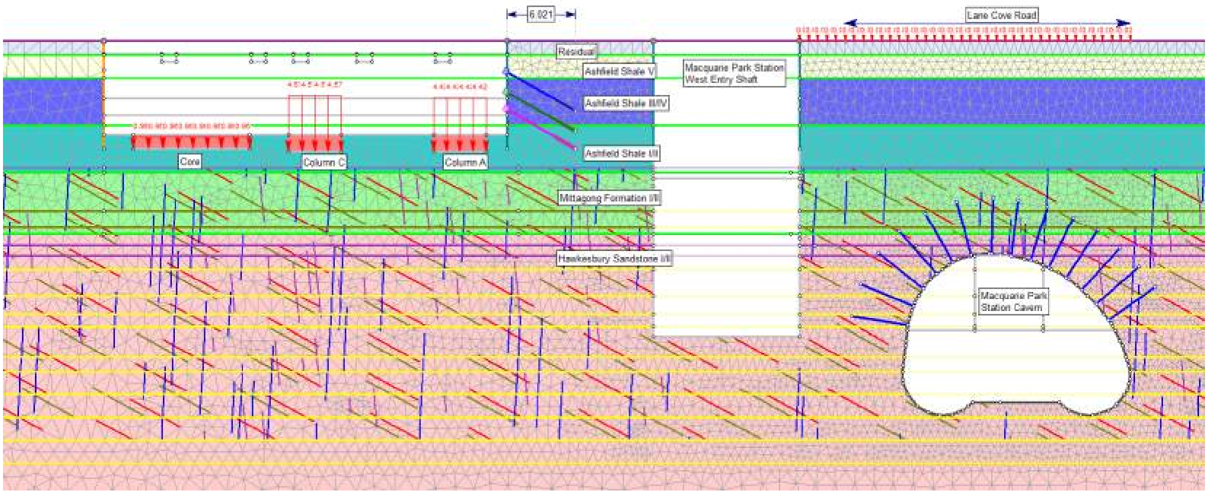


Figure 6. Geotechnical model for the section adjacent to station shaft and cavern

The site retention system adopted as part of the modelling for the proposed development comprised 600mm diameter soldier piles at a 1.5m spacing. The soldier piles would need a minimum embedment depth of 2.0m into the Ashfield Shale Class II, or better to be consistent with that adopted for the existing west entry shaft support. The shotcrete applied between the soldier piles was not modelled in the analysis.

Temporary tieback ground anchors were modelled for the site retention of the proposed development. The anchors were modelled to be 6.9 m in length, installed at 30˚ angles below horizontal to allow approximately 0.6 m clearance from the 'First Reserve' protected zone.

Geotechnical design parameters were developed based on the industry well accepted Sydney's Rock Classification System (Bertuzzi et al 2002, Bertuzzi 2014) and (K. Chan et al, June 2006).

The results of all the modelling indicated that the additional maximum vertical and horizontal deformations that would be caused by building construction would be limited to 3mm within the vicinity of the station caverns and less than 10mm within the vicinity of the station west entry shaft. Further, the change in stress distribution, prior to and after proposed basement excavation (including application of foundation loads), was found to be negligible around the areas where rock bolt support for the station cavern is present.

Similarly, the predicted maximum differential movement across modelled joints that could be caused by building construction in the vicinity of the rock bolt zones around the station and concourse caverns would be limited to around 1 to 2mm and 1mm respectively.

The results had proven that the station structures would only undergo nominal deformation and the existing permanent rock bolts would not be subjected to differential shear deformation that could cause damage to their protective sheaving. The developer was eventually given consent to proceed with their development on the strength of the analysis undertaken by WSP.

Case Study 2 - Development adjacent to existing station cavern and age rail tunnels

The site

This site of the second case study is located within Martin Place, near the centre of the Sydney CBD, within the Sydney Local Government Area. It is located on the corner of Martin Place and Macquarie Street. The proposed development required the demolition of the existing 28 storey commercial building, including basement levels, building footings and the construction of a 33-storey commercial office tower with lower-level retail use. The proposal required some minor excavation of the existing basement.

The site is adjacent to two key pieces of Sydney Trains rail infrastructure: the City Circle and Western Suburbs lines (constructed 1920's) to the east and the Eastern Suburbs Rail line (Martin Place Station Cavern and adjoining tunnels, constructed from 1967 to 1979) to the south (Figure 7 and Figure 8).

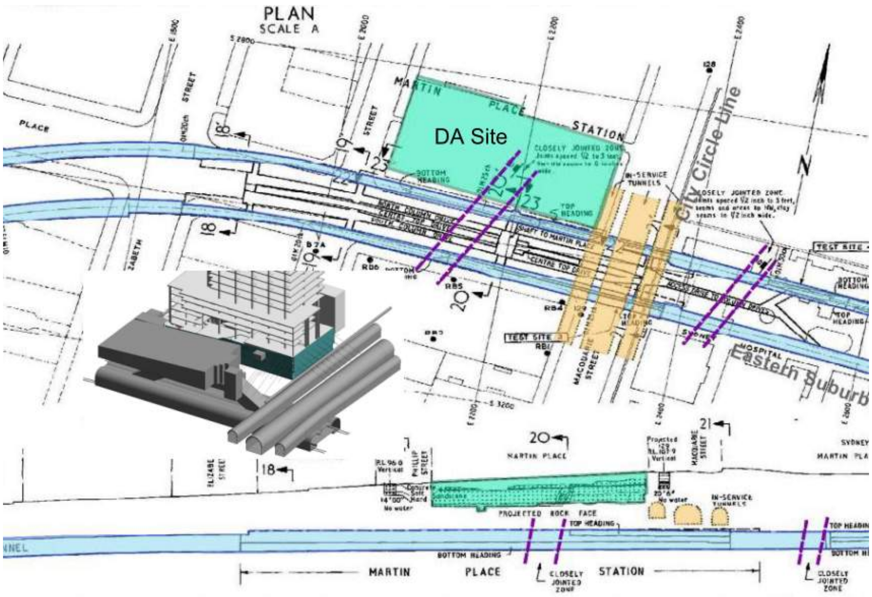


Figure 7. Plan showing DA site and adjacent rail infrastructure.

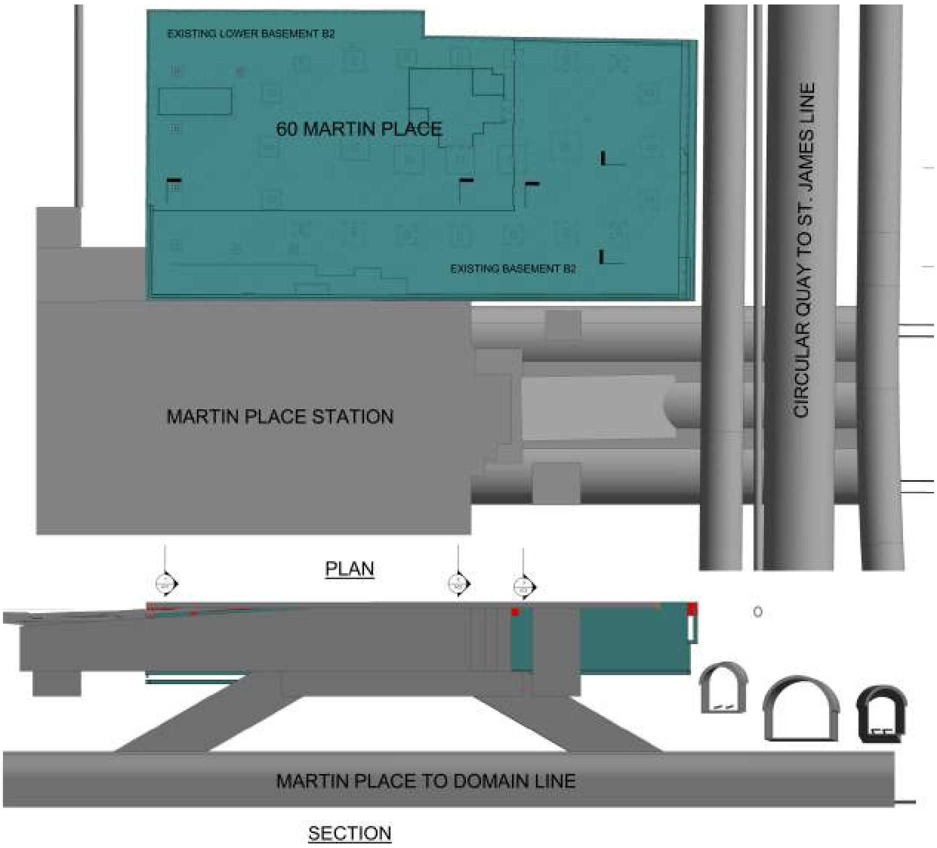


Figure 8. Plan and section view of DA site and rail infrastructure.

Site geotechnical profiles

Site specific geotechnical investigation and available geotechnical information indicated that the site contained a typical Sydney CBD geotechnical profile, with some 4 to 6m of residual material overlying weathered rock, which increases in strength with depth. Local faulting in the area (Martin Place Joint Swarm and GPO fault zone) was identified during construction of Martin Place Station (Figure 9). The rock mass, as is typical with Sydney sandstone in the CBD, contains significant horizontal compression stresses.

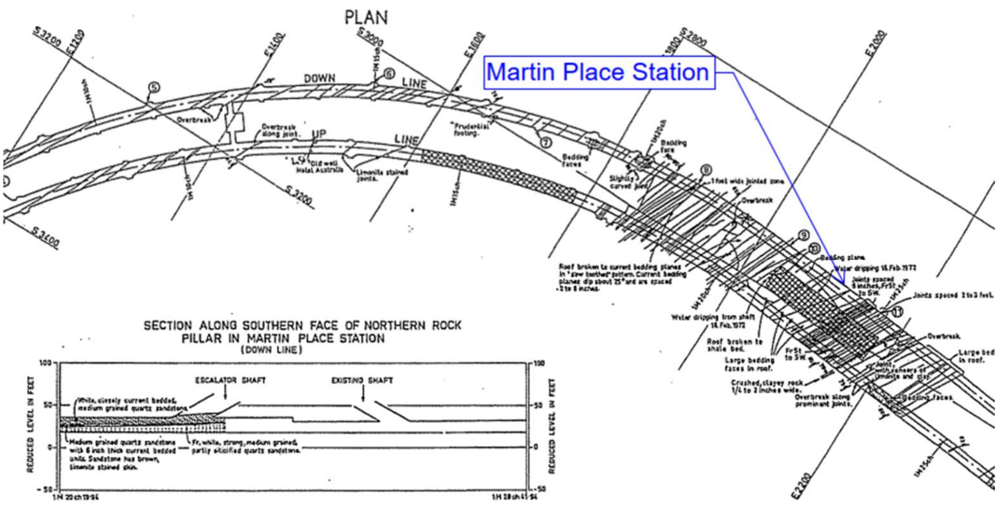


Figure 9. Faults identified during ESR line construction.

Engineering assessment

The main issue with this proposal centred on the fact that the site is close to both an existing station and aging rail tunnels, supported with low grade concrete lining. No specific requirements, advice on underground infrastructure protection/protection reserves, or concerns were provided at the time of development proposal by the rail authority. In consultation with the rail authority, it was established that the engineering assessment that would be undertaken by WSP needed to address the following:

- Assessment of the effects of the construction sequence using finite element and associated structural assessment (or similar) was required to investigate the potential issue of cracking of the tunnel structure and the uplift that would be induced by ground unloading by the demolition of the existing building.
- Additional localised basement excavation would cause some stress relief of the rock adjacent to the existing rail tunnels near their crown.
- The heightened sensitivity of impacts to the rail infrastructure due to their age from additional building loads within load influence zones.
- The requirement for temporary ground anchors to remain outside of the rail corridor.

A detailed finite element analysis, incorporating the history of tunnel construction and the proposed building/foundation construction sequences, was carried out to demonstrate the acceptable impacts on the adjacent rail infrastructure due to the demolition of the existing building, minor excavation and construction of the new building. Two sections were modelled to assess the impacts on the existing rail station cavern and the age rail tunnels (Figure 10 and Figure 11).

As part of a sensitivity analysis, two in-situ stress regimes were applied. These are as follows:

- Upper bound $\sigma_H = 1.0\text{MPa} + 4.5\ \sigma_v$ and $\sigma_H / \sigma_h = 1.5$.
- Lower bound $\sigma v = \sigma_H = \sigma_h = 1.0$.

Typical bedding plane and cross bedding within Sydney sandstone were also modelled for the two modelled sections.

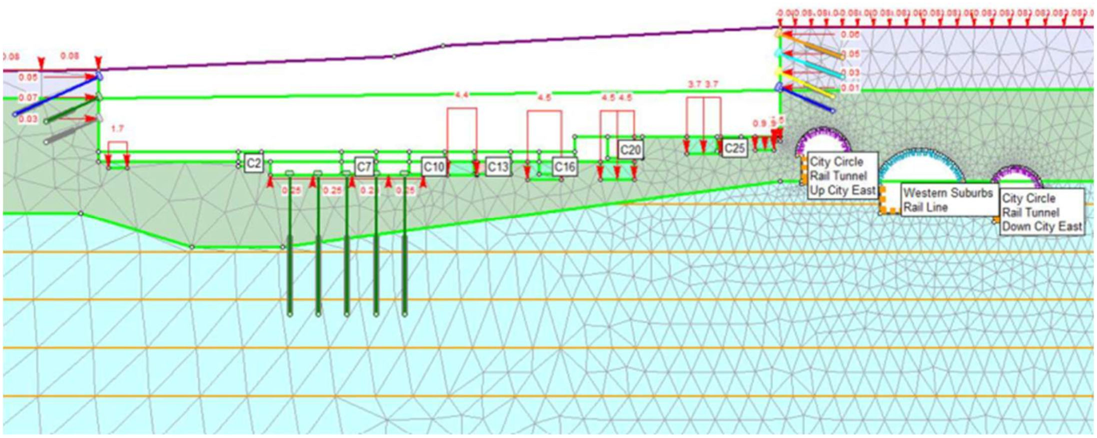


Figure 10. Geotechnical model for the section adjacent to Age Tunnels (Macquarie Street)

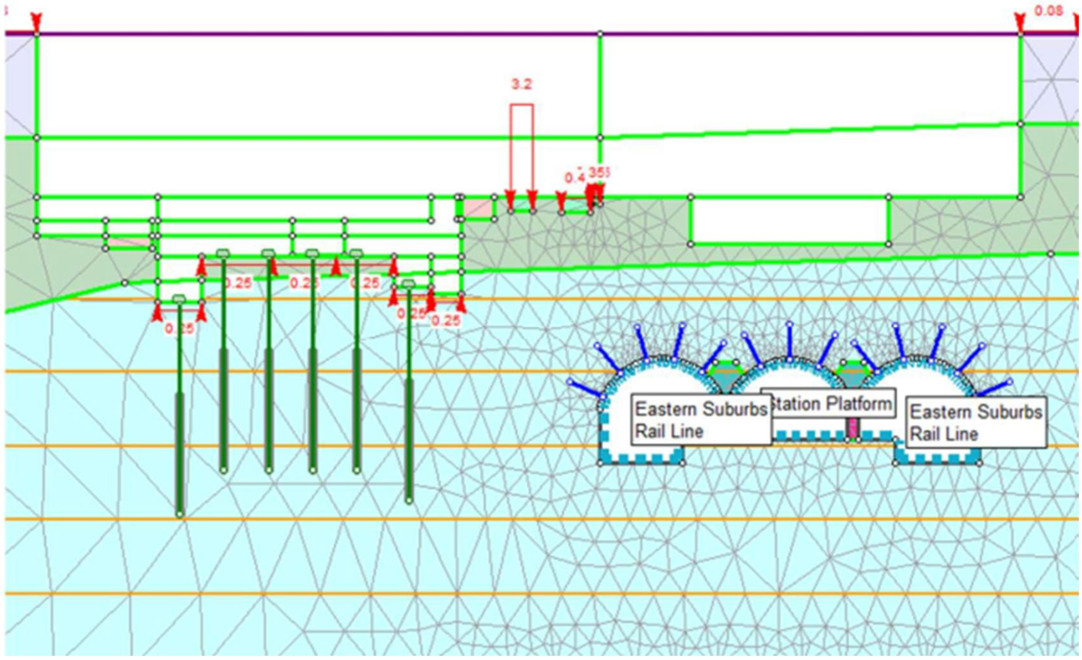


Figure 11. Geotechnical model for the section adjacent to Martin Place Station

The results from the engineering assessment demonstrated the following:

- The calculated maximum vertical displacement was limited to 1.8mm in the case of the existing rail tunnels and 5.3mm for the existing station for the modelled stress condition extremes.
- The calculated maximum change on induced stress caused by the construction of new development was limited to 0.13MPa around the crown of the rail tunnels and 0.44MPa in eth case of the station crown. These changes in stresses as caused by the proposed construction were considered to be comparatively low.

Based on the above the impacts of proposed development and construction at the site are expected to be relatively minor in terms of changes in ground stresses and the consequential deformation in the ground around the existing rail infrastructure.

The calculated ground deformation was benchmarked against published information and case history regarding to other deep excavations in Sydney Sandstone. These predicted movements were found to generally consistent with performance data, and within limits which have been accepted on other developments.

It is notable that the analysis adopted by WSP followed a two-dimensional approach. Therefore, the applied loadings were arguably more conservative than actual loading environment. Consequently, the predicted deformation and stress concentrations were also likely to be conservative.

The developer was again given consent to proceed based on the acceptability of predicted impacts as calculated through the two-modelling conducted by WSP.

CONCLUSIONS

Based on the significant experience of the authors, this paper presents the general approach and methodology that is typically applied by rail authorities in Sydney to protect their existing underground assets and planned rail corridors from surface building development. This is illustrated by reference to two case studies where the authors have provided technical support to rail authorities to review development applications.

The two cases that are presented are examples of developments which were of specific concern based on their proximity to major rail assets. Their construction if left unchecked had the potential to cause unacceptable impact to the underground support structurers such as cavern rock support. A collaborative approach was taken with the building owners to navigate the protection guidelines and standards of the rail authorities.

The assurance of acceptable impacts in these cases was given through undertaking several engineering assessments to demonstrate compliance nominated performance requirements and presented within suit of reports and plans. In specific cases compliance with these requirements must be monitored by all parties during construction through the establishment of a deed agreement.

Ultimately the aim of these processes is to secure to safe operation of rail infrastructure and avoid stifling future rail expansion through protected corridors.

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Technical Assessment of New Developments' Impact on Historical and Recent Tunnels in Melbourne

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ABSTRACT

In urban areas, an increasingly common and necessary solution for creating new transport infrastructure is through the use of tunnels. This paper looks at the way development has occurred around infrastructure in Melbourne and the different ways that this has been accounted for in the design of the tunnels themselves.

A systematic approach to assessing the risks posed by developments around tunnels and its assets has been developed, working within various legislated or planning frameworks relevant to Melbourne. This paper looks at the approaches taken in the design of historical and recent tunnels and how these affect the assessments. Older Melbourne rail tunnels were typically designed for the existing conditions, while the design of recent tunnels has incorporated allowances for additional loading that could occur from possible future building development. These design approaches affect the way that the assessments can be carried out. In the former case, assessments are essentially made on first principles basis, confirming that the additional loading is within the capacity of the tunnel structures. This often requires obtaining and interpreting the structural design information and conducting structural modelling. For the more recent tunnel designs, a phased assessment process becomes possible with a first pass comparing the effects of development loadings with the design allowances. Only in cases where the development effects exceed allowances, would it become necessary to conduct analyses estimating the current loadings in the tunnel, adding the development effects, and assessing whether these in combination are acceptable.

Finally, the paper considers some of the measures used to monitor the effects of developments when the assessments require additional validation including dilapidation surveys.

The assessment approaches in use, both for older and recent tunnels, have been effective and achievable in allowing for developments around tunnels to proceed without placing tunnel infrastructure at risk.

Key words: Historical Tunnels, Recent Tunnels, Change Allowances, Monitoring, Melbourne, New Development, Ground Conditions

INTRODUCTION

For transport efficiency in particular, infrastructure in modern cities is relying more frequently on tunnelling solutions. This facilitates the construction of road, rail, and water infrastructure without significant disturbance to existing built-up areas and addresses land availability issues in the short term. However, this can lead to a challenge to build future transport and building infrastructure while ensuring that the existing tunnelling infrastructure is protected from unacceptable adverse effects. Consequently, the changes in potential loading on a tunnel, resulting from new works in its vicinity, will need to be managed both cost effectively and systematically to minimise the risks while not imposing unreasonable restrictions or significant additional cost on the developers.

This paper looks at the different approaches that have been taken for older tunnels and new operating tunnels to assessment the risk posed by any new development in their vicinity. Older tunnels may have been only designed for the existing conditions. More recently, the additional loading allowances for possible further development around the tunnels has been included in the design. This paper elaborates on the assessment approaches for these two scenarios using examples from rail tunnelling infrastructure in Melbourne, the Melbourne Underground Rail Loop (MURL) and the Metro Tunnel Project (MTP).

MURL became an essential part of transport system and a tunnelling icon in Melbourne since its first station opened in 1981. MURL, as an example, illustrates the assessments of historical tunnels on a first principles basis. Structural analyses that incorporate assumptions about the existing loadings within the tunnels are often required to verify that the structural capacity of the concrete lining (both reinforced and unreinforced) and its serviceability.

MTP in Melbourne is one of the largest public transport projects under construction in Victoria. MTP is an example of recent tunnels where allowance for future developments has been made in the original design. Loading and unloading scenarios with physical clearances are discussed to demonstrate the approach in use for this more recent project. It is worth noting that the design allowances provide guidance to developers rather than defining development constraints or acceptance criteria. They are indicators of whether a development would likely create significant changes at the MTP infrastructure.

Finally, the paper outlines fundamental geotechnical requirements including testing and instrumentation regime, required to facilitate assessment of impact on existing tunnelling infrastructure.

IMPACT ON HISTORICAL TUNNELS IN MELBOURNE

The Melbourne Underground Rail Loop (MURL) has been an essential and integral part of the transport network of the City of Melbourne, and its tunnels have been in place under the CBD for approximately fifty years since its construction began in 1971. The MURL was designed to extend the railway networks in Melbourne CBD by connecting the existing Flinders Street and Spencer Street (now called Southern Cross) station with three new stations distributed around the CBD, Flagstaff, Museum (now renamed as Melbourne Central Station), and Parliament station. The loop comprised four single-track tunnels on two levels that allowed trains to run through the CBD and back out on their respective lines. Tunnelling works used either a hard rock tunnel boring machine (TBM), drill and blast, or road header.

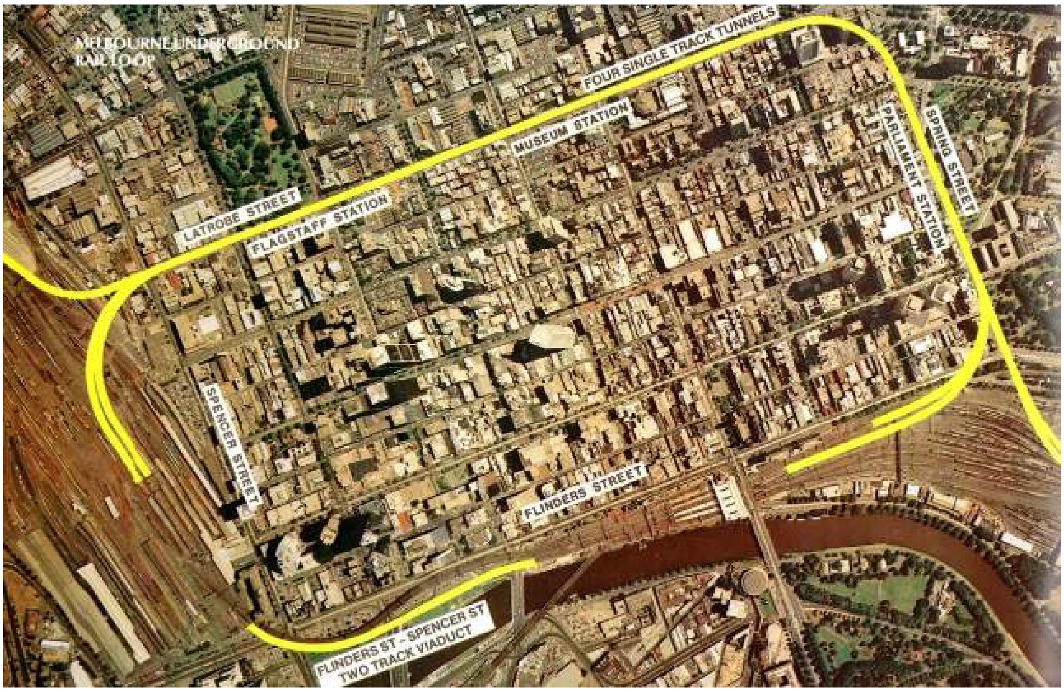


Figure 1. Melbourne Underground Rail Loop (Bishop).

MURL has been the catalyst for new growth in areas of the CBD. Developers continue to invest heavily in the CBD and building activity has significantly increased following the commissioning of the last station to be completed, Flagstaff. In 2022, it was reported that Melbourne's CBD remains a strong property market with 200 new development applications. Figure 2 provides an interesting contrast between the La Trobe Street of the time that Museum Station (now Melbourne Central) was under construction, and a contemporary view of the same area. The scale of development since the construction of MURL indicates how the assessment of impacts on the MURL tunnels due to new developments is critical both for the current tunnel owner Victorian Rail Track Corporation (VicTrack) and the developers.

Figure 2. La Trobe Street Looking Eastwards Over Queen St 1970s and 2019 (Bennett 2021).

Current Assessing Approaches For Melbourne Underground Rail Loop

The Protection Legisilation for The MURL Tunnels

The protection of MURL is provided by legislation, specifically Section 54 of the Transport (Compliance and Miscellaneous) Act 1983, and the relevant extract is included below:

Any person who proposes to develop any land along or in the immediate proximity of the Loop shall before commencing the development and without in any way limiting his obligation under any other Act to obtain any other approval or consent submit to Rail Track full details of his proposed development and shall comply with any conditions imposed by Rail Track which it thinks may be necessary for the protection of the Loop or the proposed development.

The Act is written in broad terms and provides VicTrack with strong powers to review and control proposed activities by adjacent developers. However, in terms of its application, there is no specific definition of what constitutes the immediate vicinity of the tunnels, nor specifically what would cause VicTrack to impose conditions. On the other hand, the protection of the development is considered in the Act as well. In practice, the purpose of the assessment is to minimise the consequences/risks for the tunnel owner while also not to imposing unreasonable restrictions or cost on the developers.

Technical Assessment for The MURL Tunnels

In terms of the current approach of the assessment, an indicative influence zone around the MURL, which is based upon the history of the assessments, has been adopted by VicTrack, using widths of approximately 40m from the centre of tunnels and 80m from the stations as indicative offsets of proposed developments that require review.

Most of the length of the MURL tunnels was designed with no allowance for future development. Over the history of assessments, criteria of tensile stress changes in tunnel structures have been developed based on the structural response of the tunnels to unloading and loading effects from developments.

The MURL tunnels were constructed using either a hard rock TBM, drill and blast, or road headers with primary and secondary linings. The primary supports were typically steel sets with timber blocking and lagging or shotcrete. Because it was an unsealed excavation, the ground water was drawn down to the invert level of the lower tunnels during construction. The secondary lining was cast in-situ concrete encasing the inner flange and the web of the primary support sets with, typically, light reinforcement only (with a few exceptions such as under the former Commonwealth Buildings at the corner of Spring Street and Victoria Street / La Trobe Street). The inner flanges of the steel sets were considered reinforcement as they were fully encased in concrete.

As a result of this construction technique and the uncertainties in the behaviour of the ground in the time between the installation of the two support systems, the secondary lining could be experiencing a range of states that fall between the following bounds:

- State 1: The primary lining is intact and supporting all the ground loads. The secondary lining is effectively unloaded (or carrying ground water pressures only); or
- State 2: The secondary lining is carrying the ground loadings partially and gradually through an initial delay in the relaxation of the ground, or through concrete creep and deterioration of components of the primary lining.

Consequently, both an unloading case from demolition of existing structures and excavation for basements and a loading case for the new development construction are usually considered. State 1 is typically deemed to be critical for an unloading case, while the State 2 is usually critical for the loading case.

As a first step to provide a quick assessment of the level of risk that a development poses for the tunnels, an analysis is conducted for review against stress change criteria. The following simplifications and assumptions are made to facilitate this preliminary assessment.

i) Unfactored loads from by the proposed works are adopted (effectively a serviceability assessment),

ii) The concrete lining is monolithic and un-reinforced (reflecting a conservative model for much of the lining),

iii) The contribution of any remaining primary support for the lining analysis is ignored,

iv) Tensile stress changes are transferred across the interface between the secondary lining and the ground,

v) Changes from adjacent works since the completion of MURL are considered.

Maximum tensile stress changes under short-term (construction phase) loading and long-term loading based on crack stresses have been adopted for the assessments.

In a scenario where the proposed development is within the zone of influence of the tunnel, a structural modelling based on the assumed scenarios related to existing loading case plus the additional loading, is required to demonstrate the tunnel still has the required structural capacity to cope with the additional loading. At this level of assessment, absolute stresses and crack widths are not estimated in the tunnel linings. The adopted criteria, in effect, control crack generation and crack opening and have proved to be sufficient over the history of developments where the stress changes have been below these criteria. The full structural and serviceability analyses would be conducted where the stress changes were beyond the criteria adopted for the first pass assessment.

An Example of Technical Assessment of MURL

A 33-storey commercial tower in Melbourne's CBD was proposed for a site currently occupied by a 7-level concrete-frame office building with basement car parking. The northern boundary of the proposed development was adjacent to the MURL tunnels (refer to Figure 3). In the area, the MURL assets comprise the four running tunnels stacked in two pairs, with the most affected tunnels being the Clifton Hill/City Circle Loop (upper tunnel) and Northern Loop (lower tunnel).

The existing building was constructed after the construction of the adjacent MURL tunnels and therefore its demolition had to be considered. The developer had already elected to limit the basement depth at the northern property boundary because of its proximity to the tunnels. Therefore, the basements at northern portion were to be two levels with a depth of approximately 5m below the ground, while four basement carpark levels typically extend to approximately 10m below the ground level over the remainder of the site.

Given that there would be an additional 1m of excavation below the existing level at the northern boundary and that vertical loads from podium columns onto the spreading foundation adjacent to the MURL tunnels were approximately two times higher than the existing, an independent technical assessment for the tunnels is initiated by VicTrack.

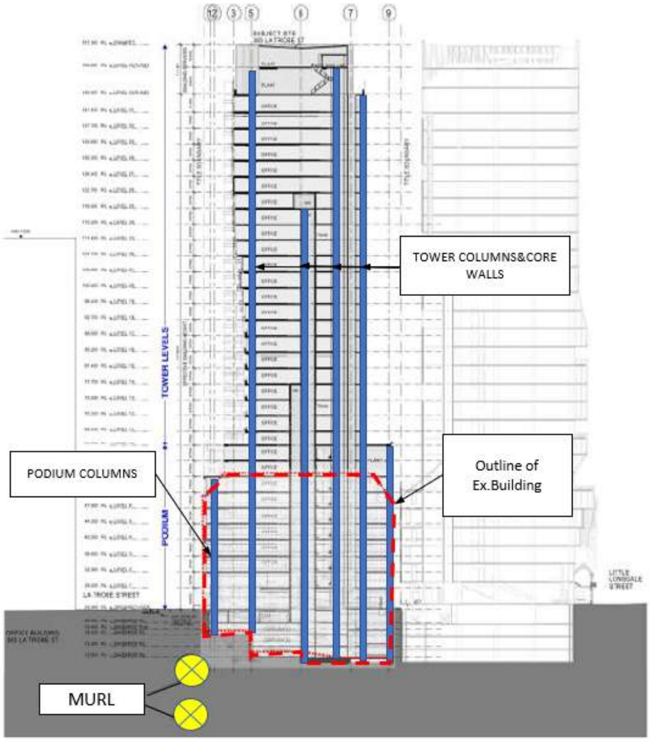


Figure 3. An Indicative Elevation of the Proposed Development also indicating the Existing Building Envelope

A PLAXIS 2D soil-structure model was developed to analyse the interaction of the proposed development with the MURL tunnels. The model simulated the history of ground stresses from the time of the construction of the MURL tunnels, including the construction of the tunnels, construction then demolition of the current building, and construction of the proposed development. The two tunnels closest to the proposed development and basement excavation were included in the 2D model and the cross section of PLAXIS model is shown in Figure 4.

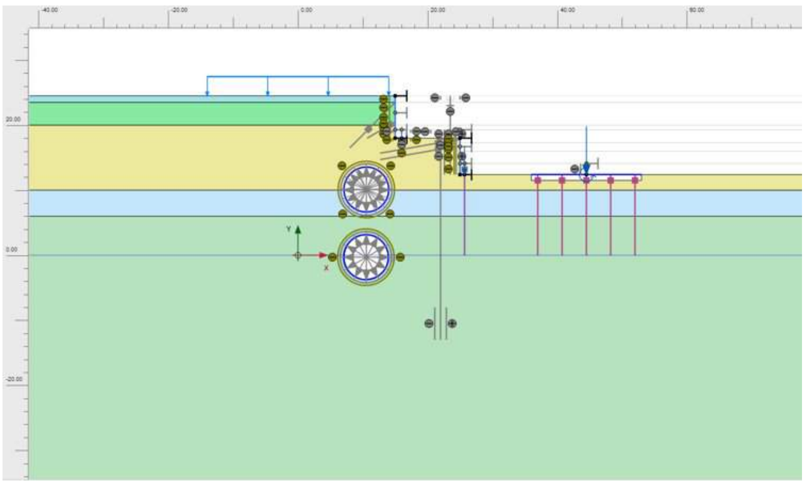


Figure 4. Cross Section of Plaxis Model

Because the model was 2D plane strain, it modelled variations in the ground profiles, ground properties and loadings perpendicular to the tunnels. Any out of plane effects, particularly with respect to the actual relatively limited extents of loading etc, in the direction parallel with the tunnel axes are ignored and would be, therefore, overestimated. Because 3D end constraints are ignored the analyses are conservative except perhaps at the mid-point of the site adjacent to the tunnel.

The excavation down to the proposed levels was identified as the critical short-term case (unloading case) and construction of the proposed development is the critical long-term case. The extreme fibre stresses of the concrete lining, calculated as (F/A ± My/I), were compared with the criteria for the tensile stress change. The analysis results showed that the maximum tensile stress changes under both short-term and long-term case were within the adopted criteria (shown in Figure 5).

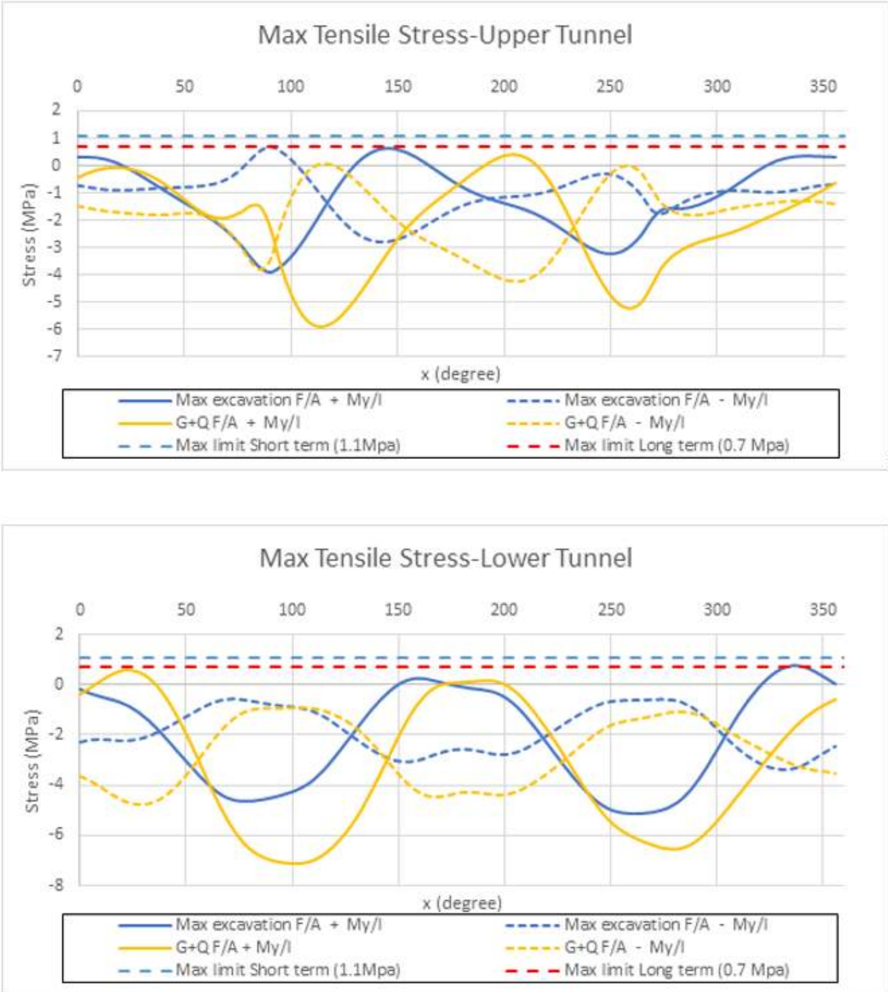


Figure 5. Calculated Tensile Stress Change vs Adopted Change Criteria

Discussion

Most historical tunnels in Melbourne have not included an allowance for further development, meaning that any significant increase of new developments in the vicinity needs to be carefully considered and analysed for their effects on the tunnel lining. Section 54 of the Transport (Compliance and Miscellaneous) Act 1983 used broad terms to protect the MURL tunnel and the affected developments. In response, VicTrack has determined an influence zone around its assets and adopted an assessment approach based on criteria of tensile stress changes of the concrete linings. It uses these as a basis for guidance to developers with proposals in proximity to the MURL assets.

It is realised that there is conservatism built into some assumptions and there is also a corresponding uncertainty in the current loads being carried by the concrete of the secondary lining. However, such an approach enables VicTrack to assess developments in a staged approach with the level of assessment related to the risk determined from the initially estimated effects, allowing technical assessments to be applied effectively. More complex and robust analyse would only be required in cases where the simplified methods leave uncertainty about the level of risk.

IMPACT ON RECENT TUNNELS IN MELBOURNE

The Metro Tunnel Project (MTP) in Melbourne, currently under construction, is one of the largest metropolitan rail infrastructure projects since the MURL was completed. It includes the construction of twin nine-kilometre tunnels with five underground stations connecting the south-eastern Pakenham and Cranbourne lines with the north-western Sunbury line. MTP will create a new north-south cross-city line that passes Melbourne Central and Flinders Street Stations and will also provide relief to the congested Swanston Street and St Kilda Road tram corridor. Like MURL, there are new developments and modification of existing structures in the vicinity of MTP. Accordingly, the Authority, Rail Projects Victoria (RPV) provides guidelines to assess the effects on MTP's assets due to the future developments. The same assessment approaches are applied for Suburban Rail Loop Project, which is a mega tunnel project in Melbourne at the reference design stage.

Current Assessment Approaches

Future Development Loadings

Additional allowances for the possibility of future development of the land above and adjacent to MTP have been incorporated into the design of the segmental lining of the MTP tunnels. The following three technical aspects have been considered in the vicinity of the tunnels:

- Unloading Case: Excavations over or adjacent to underground structures of the Metro
- Loading Case: Additional loading imposed after construction of the Metro; and
- Clearance of new development works from underground structures of the Metro.

The vertical unloading case is described as a limit of 7m excavation depth below the natural ground surface to allow for further development while maintaining a minimum 7m residual ground cover over the MTP tunnels. The minimum lateral clearance between the tunnels and deep excavations is also defined to consider the possibility of deep basement construction adjacent to the tunnels. Regarding additional load cases, the equivalent uniform unfactored load of 50kPa acting at a level 1m above the tunnel crown, plus the equivalent uniform unfactored load of 20kPa applied at ground level have been included in the MTP design.

The minimum physical clearance from foundations or footings of new developments to the tunnels has been specified so that the potential risk of damage to MTP tunnel structures can be controlled and mitigated.

With these MTP design allowances in place, it would be improbable that further development would be precluded by the presence of the Melbourne Metro. In some cases, re-arrangement or structural mitigation measures might be required to limit the effects of developments and to keep clear of the assets themselves. In summary, the clearances and loading allowances are shown in Figure 6.

It is also worth highlighting that the design allowances provide guidance to developers rather than defining development constraints or acceptance criteria. They are indicators of whether a development would be likely to create significant changes at the MTP structures. The MTP design allowances were also used in developing the Design and Development Overlay for referring a development, and RPV's initial assessment.

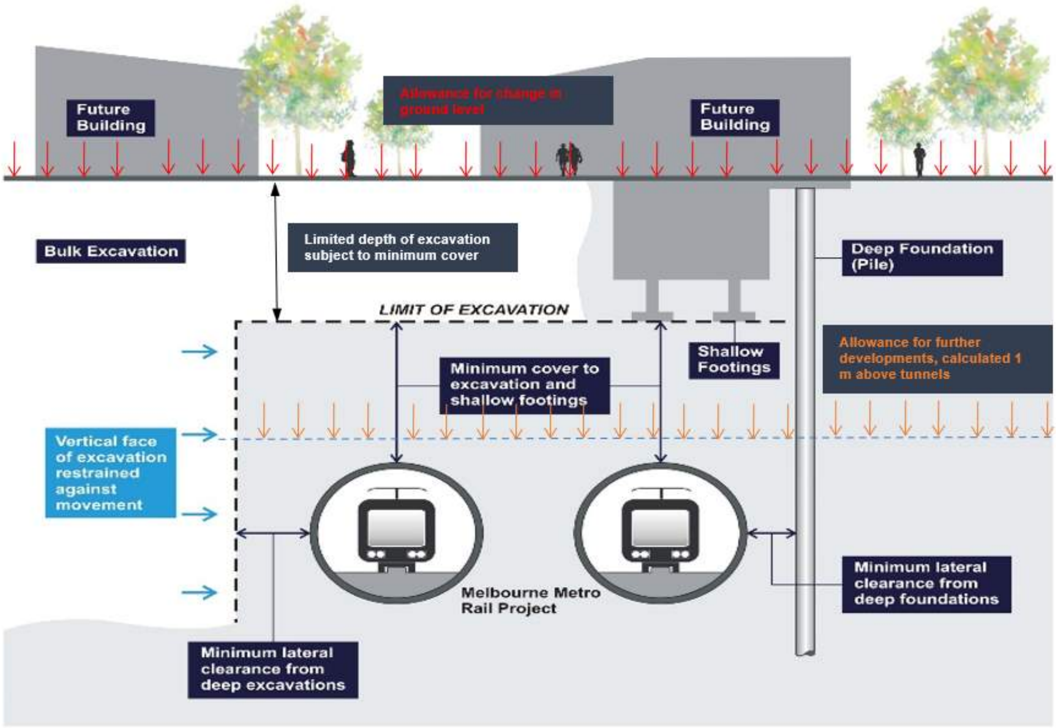


Figure 6. Calculated Tensile Stress Change V.S. Change Limitations (Modified from Figure 3-1 of Environment Effects Statement Appendix E)

Design and Development Overlay

The Design and Development Overlay (DDO) was placed in the planning schemes as a mechanism for alerting developers to the presence of the tunnels and other underground structures and formalising the referral process for assessment of applications for planning permits. The extent of the proposed DDO around the Metro Tunnel underground assets identifies the distance beyond which development loading would be unlikely to be of concern to MTP assets.

Figure 7) was derived from a series of 2D analysis results when an estimated ground stress changes from an offset loading representing a development was equivalent to the 50kPa allowance. Typical values of the best fit results from the 2D analyses for the segmentally lined tunnel were an offset of 15m and an angle from the horizontal of 35°. Terrain modelling is used to extrapolate the offset and slope surface from the tunnels to the intersection with the modelled surface, thus determining the DDO boundary in plan.

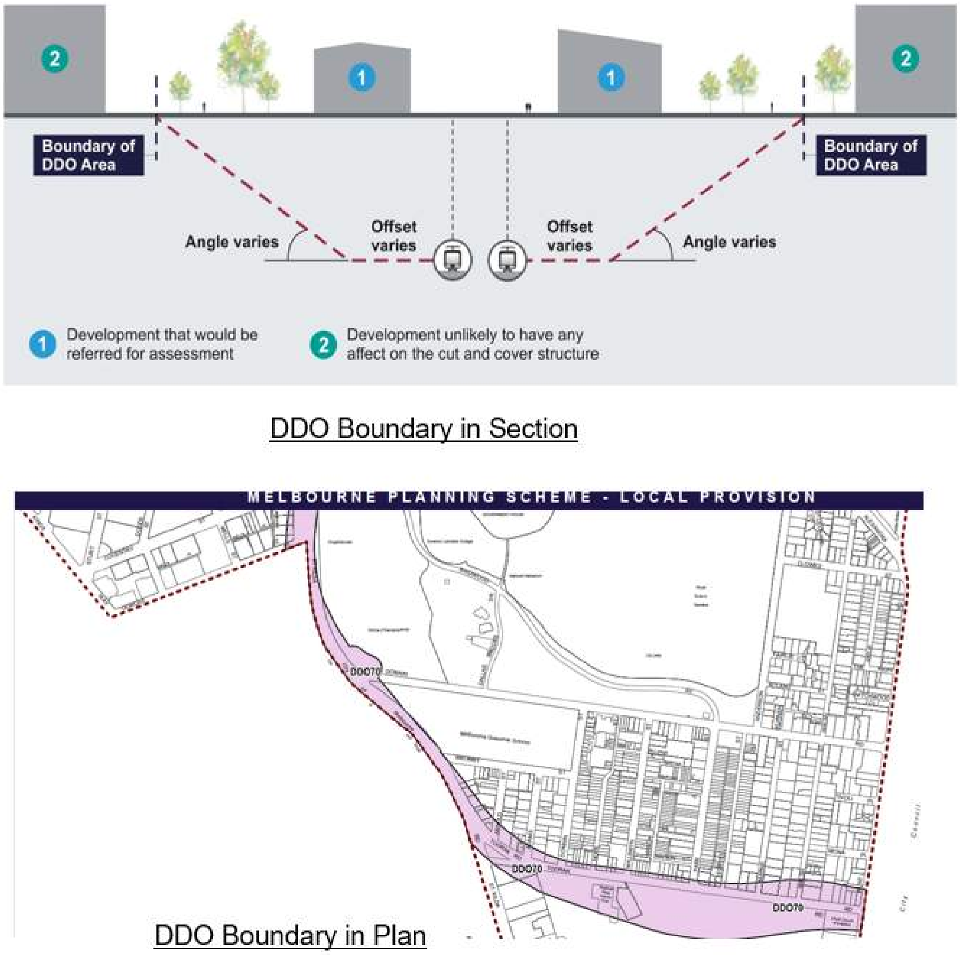


Figure 7. Example of DDO Boundary (Modified from Figure 3-1 of Environment Effects Statement Appendix E)

Design and Development Overlay

The purpose of the design and planning aspects developed during the MTP design, discussed above, were to facilitate a practical process for technical assessment of MTP assets as outlined in

Figure 8.

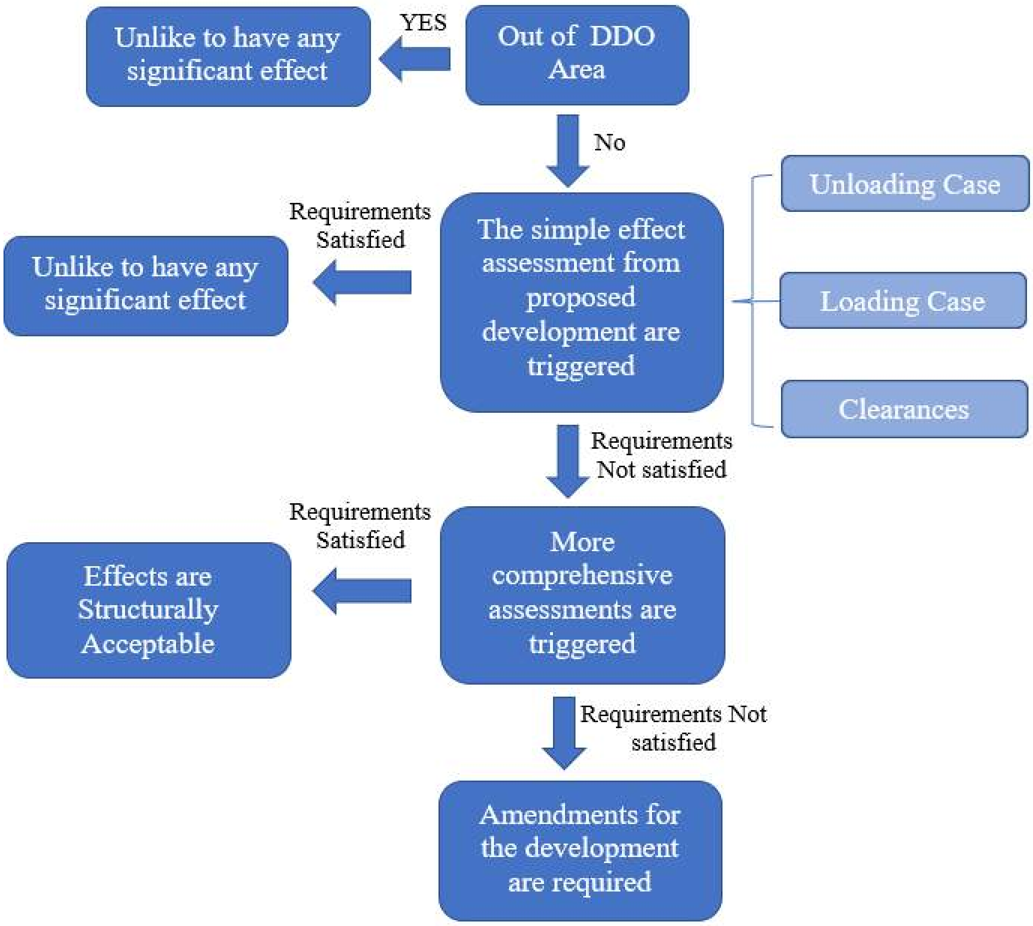


Figure 8. The current process for Technical Assessment

A Hypothetical Case of Technical Assessment of Development Effects on MTP

For reasons of project confidentiality, an example hypothetical scenario based on MTP is discussed in this section. A technical assessment for the redevelopment potential of an existing building site was carried out. The two MTP segmentally lined tunnels pass underneath the northern part of property, which means the site would be within DDO area and likely to affect the tunnels. The developers' preferred proposal for redevelopment was a building with ten storeys above ground level and three basements, which would result in a 185kPa unloading at the level of pile cap, and 120kPa reloading from the new structures. This represents unloading of 65kPa in the long term. The depth of excavation to the tunnel crown was 9.3m (refer to Figure 9).

Accordingly, the assessment criteria would have the following implications:

- Basement excavation depth immediately over the tunnels limited to 7m (typically two basements),
- A deeper basement outside this zone could extend down to the invert of the tunnels. The basement walls adjacent to the tunnels would need to be sufficiently stiff to limit the lateral displacement of the ground to 20mm. Temporary support of the wall could extend above the

tunnels but would keep 3m clearance from the tunnel.

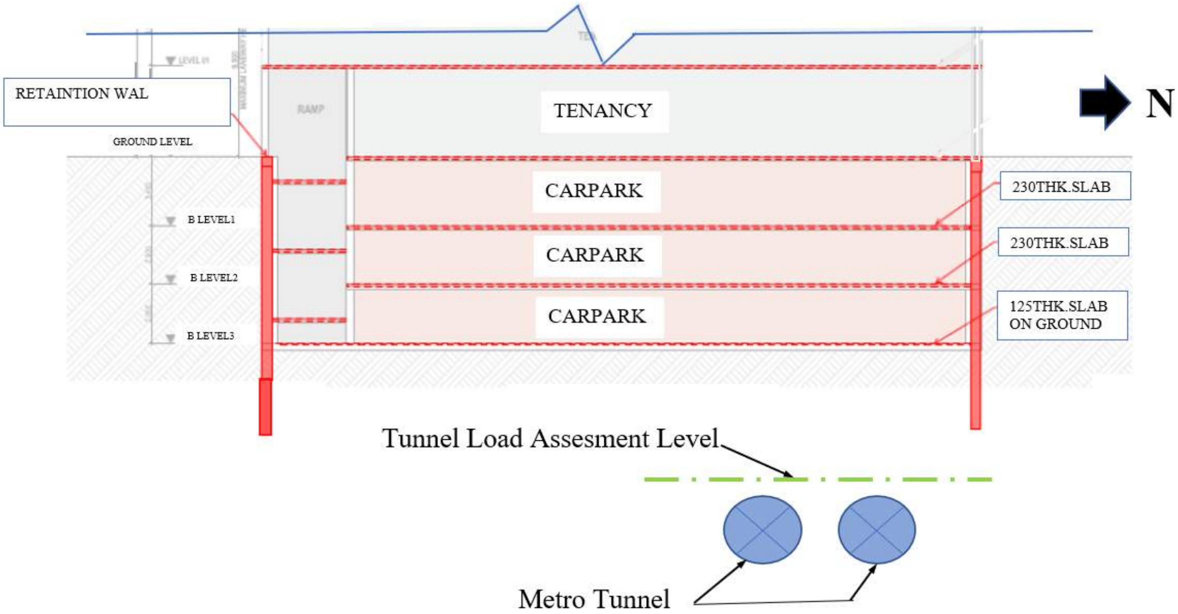


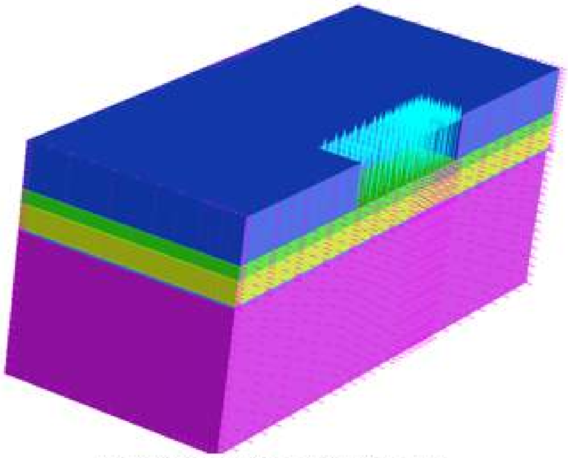
Figure 9. Partial Elevation of proposed Redevelopment

While the proposed development details did not match the configuration of unloading used in the original tunnel design for future developments, the effects of the proposed development were assessed to check whether they were equivalent to or less than the design case. Effectively, this was checking whether the effects from a limited extent of excavation for three basements would be less than the design case of a more extensive excavation for two basements.

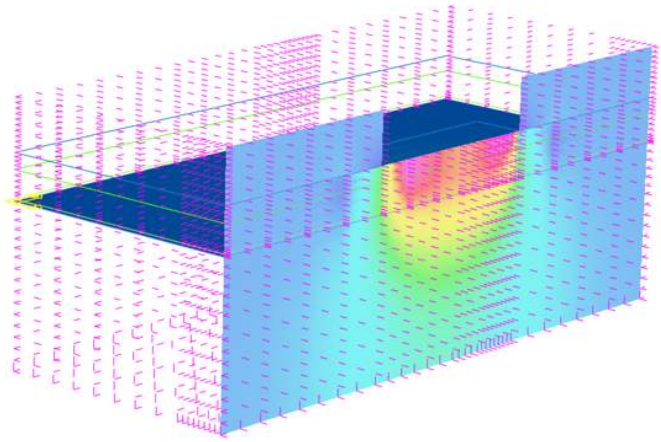
The analyses for the stress and deformation changes due to the loading and unloading cases were conducted in stages. At the initial stage, the analysis was a simplified manual method, using Newmark Charts, to provide an estimate of the effects of the excavations. The simplifications which allowed the analysis to be solved quickly meant that the results were indicative but very useful for identifying cases that are clearly acceptable or clearly not. However, in cases that are near the criteria, the results might not provide a basis for determining whether an effect was acceptable with sufficient robustness to convince the authority responsible for the tunnels. In this case, the manual method indicated that the excavations would lead to pressure changes of 10% more than the original design allowance, which was considered too close to confirm either acceptability or for raising concerns potentially leading to rejection.

Therefore, a more rigorous analysis was conducted. This comprised solving a half symmetrical 3D numerical model to determine the changes in ground pressures above the tunnels. The use of the model allowed better representation of the geometry of the surface and excavations as well as the different stiffnesses of the ground between the development and the tunnel (refer to Figure 10).

At this stage, the modelling still included simplifications that meant the model could be created and solved relatively quickly, using a few representative material properties that are readily available for the ground conditions. As was the case for the manual analysis, this modelling took the approach of comparing the pressures created by the excavations at the tunnels with the allowances made in the tunnel design for future developments. Unlike to the analysis for MURL tunnel, this analysis was able to check the development effects against the tunnel loading specification rather than explicitly assessing structural effects within the tunnel lining where the criteria are much more complex. The 3D model predicated that the unloading case would be 12% above the MTP design allowance, which was similar to the estimates from the manual analysis.



Half Symmetrical 3D Model



An example for the pressure distribution

Figure 10. 3D analysis model and one result of pressure distribution

Although unloading changes which exceed the design allowances for further development would not necessarily preclude the development’s basements proceeding, it would require additional design information on the proposed development, and further detailed assessment. This review process would be expected to take longer, and it should be noted that undertaking further assessments would not ensure acceptance by the authorities. Based on these results, the developer decided to revise the development to two basements to clearly fall within the design allowances discussed in the previous section.

Discussion

The inclusion of allowance for future development and the placing of the DDO approach provides a logical process both for the developers and the authorities in the technical assessment of the effects of proposed developments. These are clear guidance for both parties on how the risk levels for the MTP assets from the proposed developments would be assessed using the vertical stress changes above tunnel crown and clearances from the assets. This example shows a case where the MTP tunnels would not preclude further development in its vicinity but might require modification to assure the authority that the risks to the MTP assets are acceptable.

GEOTECHNICAL INFORMATION REQUIRED

The analyses discussed in the earlier sections of this paper require reliable set of ground parameters in terms of both its strength and stiffness characteristics. The range of acceptable SLS or ULS effects is normally narrow and in the order of only few millimetres in terms of deflections and not more than 5-10% in terms of stress change. To accurately estimate this range of impact on tunnels from construction of developments using sophisticated finite element analysis, it is important that a good level of ground investigation data is made available in conjunction with establishing a robust instrumentation and monitoring plan to verify any assumptions. Even if previous ground investigation information is available, potential more recent changes in ground water regime can create significant differences in stresses. Designer’s working to assess impact of new developments on existing historic tunnels may not have a reasonable level of confidence in existing third-party geotechnical data. Therefore, even when historic site investigations are available, to assess impact on sensitive infrastructure, it is generally recommended to carry out additional verification site investigations with a focus on ground stiffness parameters, in-situ stress conditions and understanding ground water regime.

Geotechnical assessment make use of available historic and newly acquired ground investigation information to help assess impacts on the tunnel in terms of loading or unloading from the new development, ground deformations, induced vibration, ground-borne noise impacts, discharge of stormwater from the planned development, changes to groundwater levels, loss of support to rock bolts and anchors associated with the tunnel structure or loading of piles or anchors on the existing tunnel.

Geotechnical assessment includes geotechnical investigations, detailed engineering analysis and associated impact assessment utilising a systematic risk management system.

The developer carries out detailed geotechnical investigations of the soil or rock strata above, alongside and below the tunnels, as appropriate, to establish the existing ground conditions within the area affected by the proposed development.

Further ground investigations may also be carried out to assess any changes in ground conditions such as those associated with stress changes due to excavations or surcharging and importantly any changes associated with the ground water regime. The geotechnical investigation provides accurate geological profile of the sections where the new development is proposed and the sections beyond the footprint where the development can potentially impact the existing tunnel.

The geotechnical report includes factual and interpretative account of the existing in-situ stress state in soil and rock mass, joints, bedding planes and dykes besides associated soil and rock parameters. Any new intrusive ground investigation such as boreholes and CPTs carefully consider potential interaction of the exploratory holes or impact on the tunnel or associated systems. To determine accurate estimates of soil or rock stiffness parameters, pressure meter testing is carried out in addition to derivation of stiffness parameters from in situ SPT or CPT tests.

Geophysical testing methods might also be utilised as part of the geotechnical investigation works as these provide quick and economical means of supplementing information obtained by other more direct methods, such as boreholes, test pits and CPTs. These methods help in identifying local anomalies that might be difficult to identify through other exploratory methods.

A detailed instrumentation and monitoring plan alongside a contingency plan and early warning system is developed as part of the construction process of the new development next to the existing tunnels. Instrumentation to predict displacements, stress levels in structural elements and vibration levels are included as part of the instrumentation plan.

Physical inspections of the existing tunnel may be required by the developer accompanying representative from the owner’s organisation on regular basis during critical stages of construction.

The basic instruments used to monitor tunnels against adverse impact due to new developments might include the following typical instrumentation.

Table 1. Typical Instrumentation

Instrument for Monitoring	Purpose to Monitor
Water standpipe	Ground water changes
Piezometer	Ground water and changes in access porewater pressure over time
Inclinometer	Movements along depth
Extensometer	Displacements
Ground settlement pins	Surface movement
Building settlement markers	Building settlement
Vibration Sensors	Vibrations from construction
Crack-meter	Cracks in tunnel lining
Strain Gauges	Strain in tunnel lining
Vibration Sensor	Vibration in tunnel
Pressure Sensor	Pressure in tunnel lining

It is of utmost importance to collect sufficient baseline data for each of the monitoring parameter well before the construction works begin. It is not uncommon to utilise remote data loggers and associated warning systems involving visual and audio alarm system for monitoring purposes. This ensures real time monitoring and ensures health and safety of site staff.

Detailed pre and post construction dilapidation survey is a key requirement in establishing any adverse impact on the tunnel due to construction of an adjoining development. Dilapidation reports are typically developed before commencement of construction to understand pre-construction condition, and to summarise the post-construction condition of the asset. This is carried out even if there were no complaints or obvious damages caused during the construction phase. These surveys help ensure any un-noticed past or future damage or distress caused by other factors or construction is not construed to have been caused by the new development under review. Through the dilapidation survey, a robust record of any unintentional impact that the construction works may have caused on the tunnel can be fully documented and rectification can be made in a timely manner.

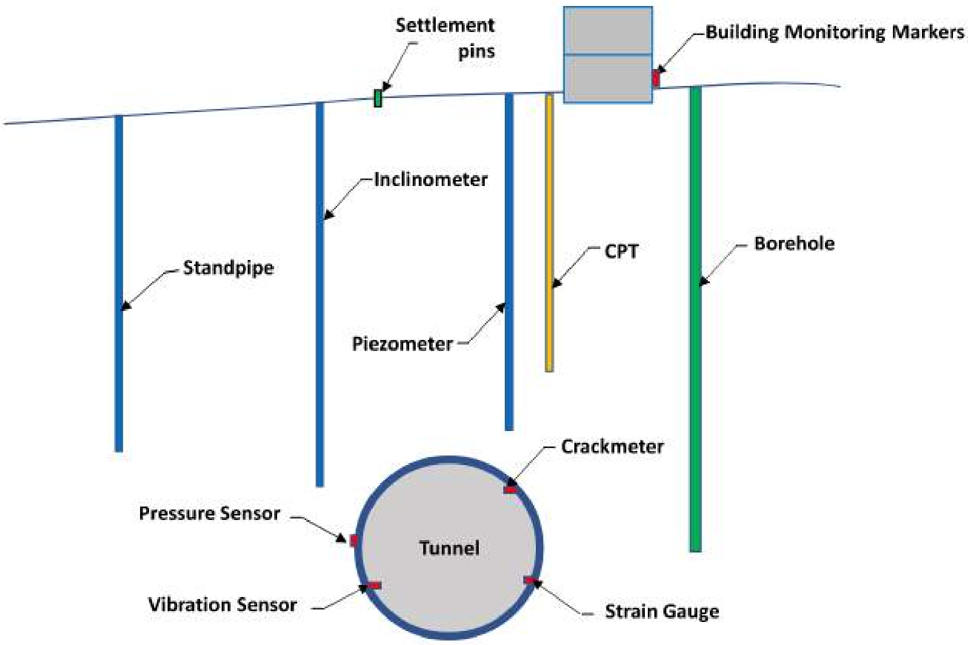


Figure 11. Typical Monitoring Instrumentation

CONCLUSION

It is understandable that there are different impact assessment approaches taken for historical tunnels which may not have specific allowances for future development, and for impact assessment on more recent tunnels where future developments have been recognised. Both types of approaches have the objective of ensuring the continuing serviceability of the tunnels without unnecessary constraints on the development. The Melbourne Underground Rail Loop (City Loop), which was a major investment in Melbourne's public transport system in 1980s, is taken as an example of historical tunnels of the former type to demonstrate the process adopted for impact assessment due to newly proposed development. This type of assessment often requires interpretation of original design information and detailed modelling analysis of stresses in the tunnel structural elements. As an example of the later tunnel types, the Metro Tunnel Project, a major rail project under construction, is presented. This example illustrates a logical assessment process based on the guidelines derived from allowances for possible further developments in design. This second type of assessment focuses on the pressure changes in the surrounded soil at required levels rather than the stresses changes in the tunnel linings.

Notwithstanding the two approaches mentioned above, the assessment for both must be conducted with a clear understanding of the ground conditions. Where there is not sufficient knowledge of the ground available, additional investigation work would be required.

Once a development has been assessed and approved construction commences, in cases of higher risk or uncertainty, it would be important to verify the assumptions made during the technical assessment using a robust instrumentation and monitoring regime along a detailed contingency plan.

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Confinements system for deep excavation with adjoining tunnels in the west of Mexico City

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ABSTRACT

In professional practice, the analytical design process requires simplifying the site conditions; however, with the support of the observational method during project construction monitoring, it is possible to make modifications in the construction process to guarantee the success of the engineering works. This article presents the case of an adaptation of the construction procedure of an excavation carried out in the west of Mexico City. The project presents several complexities such as topographic and stratigraphic irregularities, as well as complex limits such as a twin tunnel system and a highway with high vehicular flow. The excavation has a variable depth; however, the maximum vertical height for the excavation is 60 m according to the project levels. The initial soil mechanics study considered a confinement system using only anchors; however, during the development of the project, an adaptation was made in the area adjacent to the tunnels, which required a new design and a change in the construction procedure of the excavation using the "Top-Down" technique. The excavation and construction of the building was monitored and numerical modeling was carried out in an attempt to predict the behavior of the tunnel area. Finally, comparisons are presented to ensure that the excavation will generate the least possible disturbance to the adjacent tunnel area.

INTRODUCTION

This article presents a successful case of the construction of a building next to existing tunnels. The building project contemplates the construction of a 16-story superstructure and a 9-story basement. This project is located in the western part of Mexico City, which corresponds to a delegation colloquially known as Santa Fe. According to the geotechnical zoning defined in the Complementary Technical Standards for the Design and Construction of Foundations of the Federal District, the study site is located in the area known as Zone I, Lomas (hills), characterized by tuffaceous volcanic materials and igneous rocks.

The study site consists of a construction area of 5106 m^2, with irregular topographic conditions. The main problem of the terrain lies in the existing boundaries. To the west there are two parallel road tunnels; to the south there is the Tacubaya river bed and, finally, to the north, 33 m above the sidewalk level, there is the Mexico-Toluca federal highway (Figures 1 and 2).

The twin tunnels were built in the early 1990s (each 16m wide with an excavation height of 12m). The tunnels have an oblique geometry, which meant that the construction method was conventional, using mechanical advancement stages, shotcrete and secondary lining. The secondary lining was planned using reinforced concrete with a thickness of 50 cm.

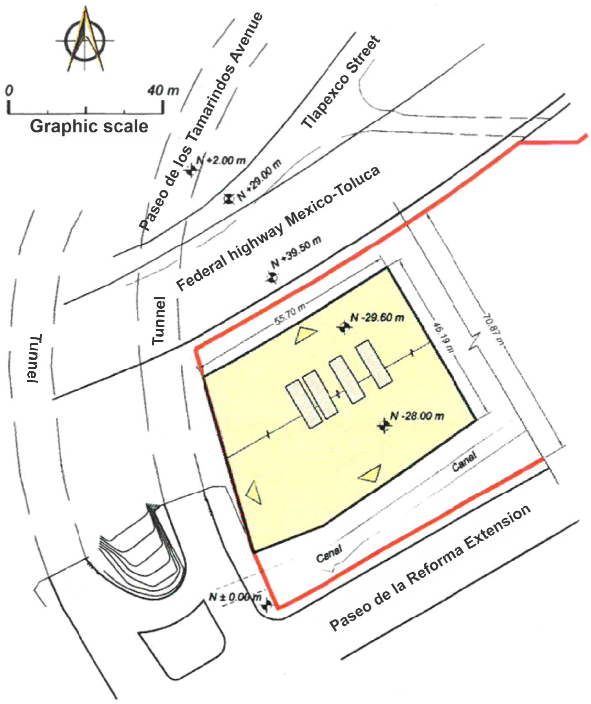


Figure 1. Site location map

Figure 2. Graphic description of the site and photograph of conditions before construction

STUDY SITE

Geotechnical site conditions

The stratigraphy of the site was defined from a geotechnical exploration campaign by means of standard penetration testing technique (SPT) and based on the geological study of the area. Four borings were made at different depths depending on their location due to the complexity of the topography; the borings were extended to reach below the level of the basements. Due to the firm to hard nature of the material, the laboratory work was limited to obtaining index properties.

From the geological survey it was possible to detect four main units according to their volcanic formation in the area. The description of each geological unit is as follows: in the upper part, slope deposits from lower volcanic formations were found; then there is a pumice volcanic eruption consisting of a layer of yellow pumice; underlying this layer are the so-called blue sands and, finally, three units known as Xolopo, which are pyroclastic flows of dacitic composition from the Totolapa dome whose remains are found south of the Santa Fé Shopping Center. Coresponding to the three events to the Xolopo, according to age and stratigraphic content, it was found that the first subdivision is composed of gravels and boulders packed in a pumiceous matrix, the second is constituted by a sandy silt matrix, similar to a hardened mud and the third is composed of a yellowish sandy matrix. At the bottom of the excavation levels, there is a rocky basement typical of a lava flow from north to south inclined according to the natural topography. Figure 3 shows a geological section of the site.

Additionally, a compilation of the information available in the literature near the property (Tamez, et al, 1997; Juárez 2019, Yama 2020) was made to define a simplified geotechnical model for the analysis as shown in Table 1.

Table 1. Summary of stratigraphic information.

Layer	Volumetric weight (γ) kN/m^3	Cohesion (c) kPa	Angle of friction (φ) °	Modulus of elasticity (E) kPa	Poisson's modulus (ν)
UG1: Slope deposit	18	10	28	66,000	0.30
UG2. Arenas azules	14	50	28	77,000	0.30
UG3. Xolopo (Tobas)	17	100	35	100,000	0.28

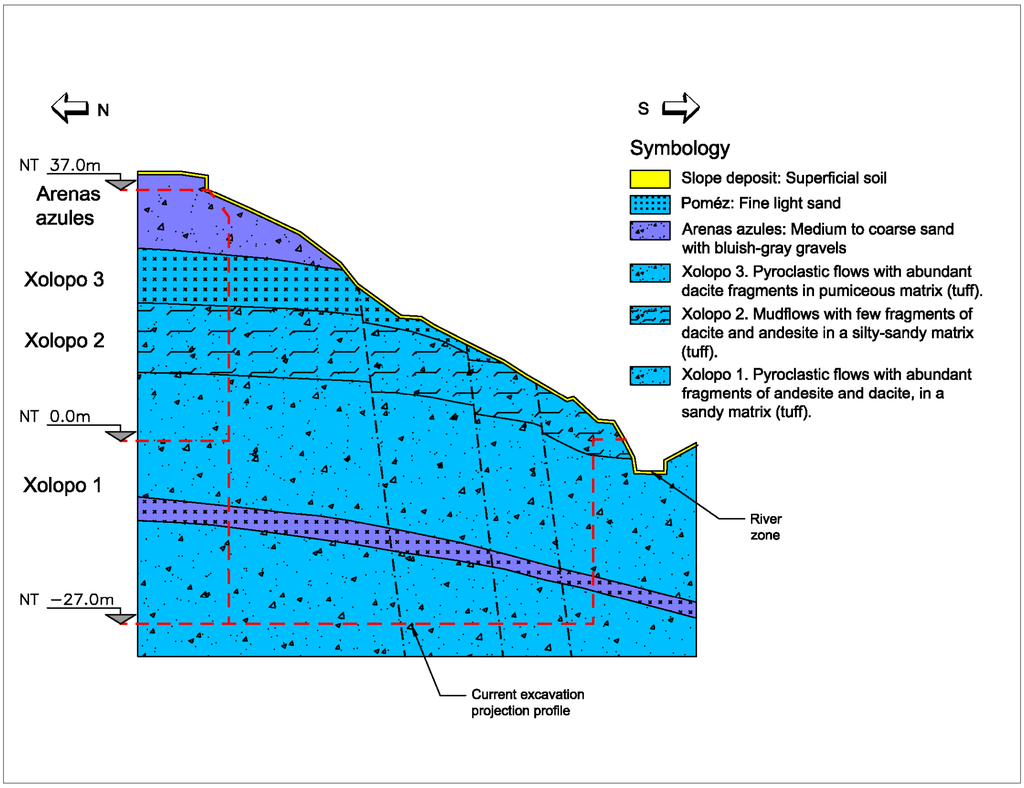


Figure 3. Geological profile of the site

Problems detected in the original design

The first design for the excavation next to the tunnels considered a confinement system using anchors as shown in Figure 4a, in the area adjacent to the tunnels with short anchors using them as ground improvement. However, this analysis, carried out by purely analytical procedures, does not take into account the presence of the tunnel and its proximity to the excavation limit.

On the other hand, the design that was carried out for the conception of the tunnels in 1990 (Tamez et al, 1997), contemplated that the definitive or secondary lining would work all its useful life in compression, so that the anchorage system would generate working stresses for the lateral excavation (Figure 4b).

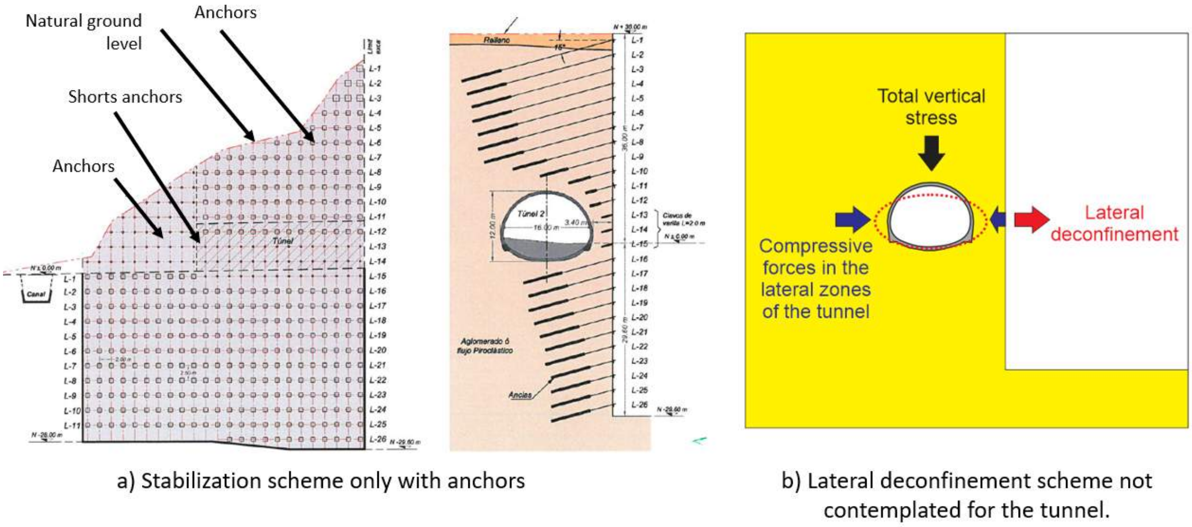


Figure 4. Schematic of the initial plan for excavation support

REVIEW OF STABILITY AND ADEQUACY OF THE CONFINEMENT SYSTEM.

Review of the initial project

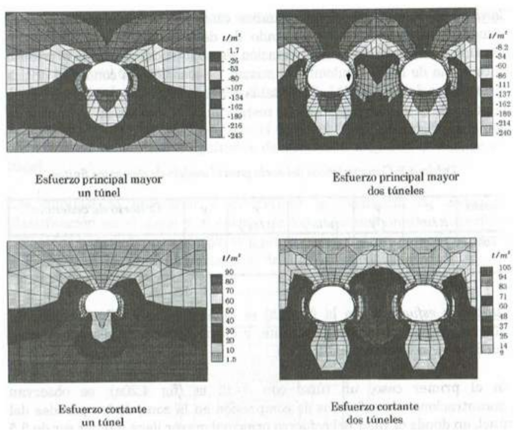
At the time of the on-site intervention, the lateral excavation of the tunnel had advanced to 70% of its height, i.e., there was already a partial deconfinement. Therefore, the first adaptation was the placement of a soil-cement platform to act as a cover for the laterally excavated area of the tunnel.

Once the work was stopped, a reinforcement by anchorage only was proposed. The initial designer considered that, due to the 3 m separation between the excavation and the tunnel, the confinement system in the area of interest should be modified with the placement of 6 short anchor lines of 2 m in length.

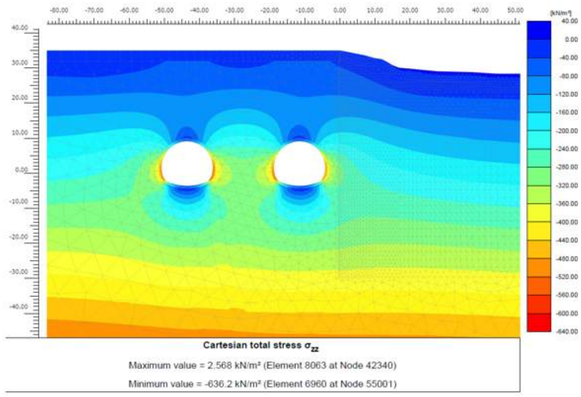
Since analytical methods involve many simplifications, in order to give a better approximation of the tunnel behavior with lateral deconfinement and to obtain a factor of safety of the overall slope stability, a first 2D finite element analysis was performed on the highest and lowest sections of the slope (Figure 5). These modelings were performed using the anchor-based restraint system proposed by the company in charge of the initial soil mechanics study. The results showed a loss of support in the lateral part of the tunnel, generating a failure by deconfinement of the tunnel. The constitutive model to represent the geotechnical units of the site was the Mohr-Coulomb model with parameters determined with the limited information that could be collected from the geotechnical exploration of the site.

Figure 5. Selection of cross-sections for finite element method analysis

To verify that the modeling was correct, calibration was performed with respect to the stress field that was calculated when the tunnels were designed in 1990 (Figure 6).



a) Results obtained in the literature when conceiving the tunnel design



b) Calibration result for the section with the presence of the tunnels.

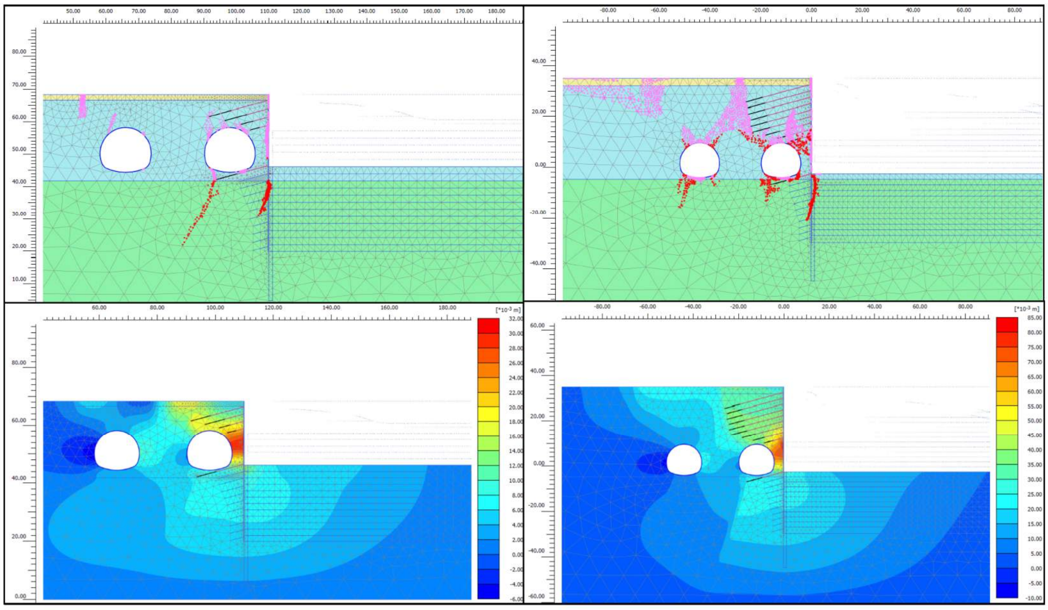
Figure 6. Model calibration with respect to the results obtained in 1990

The analysis was carried out considering the excavation stages of 3m in height and placing the corresponding anchor level according to the designer´s recommandations. From the lower section analyzed, the model did not converge, showing the collapse of the soil when reaching the lower part of the tunnel; on the other hand, the model collapsed in the highest zone at 3m above the level of the tunnel floor Figure 7 shows the plastic points that define the slope failure path when contemplating the existing tunnel. This result allowed to suspend the system that had been conceived for the support of the excavation with anchors only and to make a new proposal together with the structural designer to propose an alternative taking into account the advance that had already been achieved.

Approach for the new confinement system

Part of the proposal was to generate a confinement system, applicable to the existing conditions, in which the tunnel would not suffer deconfinement and permitting that the excavation could proeed with in the central core of the site. A shoring system was proposed against the structure itself, to minimize the impact on cost and execution time. Additionally, to support the deconfinement, a rigid element was required on one side of the tunnel, so a pile wall system was proposed, the diameter of the piles being defined from the soil thrust diagram in dynamic conditions; finally, these piles would be supported at the top by a capping beam to work uniformly.

This alternative would entail complex excavation systems throughout the site, the continuation of the excavation in the central core, the construction of piles previously to the placement of the structure's profiles, as if a "Top-Down" type excavation system were being considered. Figure 8 shows a schematic drawing of the approach of the structure to support the lateral thrust and Figure 9 shows a schematic drawing of the proposed excavation system, advancing by levels and constructing the mezzanine slabs to be connected to the pile wall and generate the confinement system.



Section 1 Section 2

Figure 7. Plastic points and horizontal displacements with alternative anchors

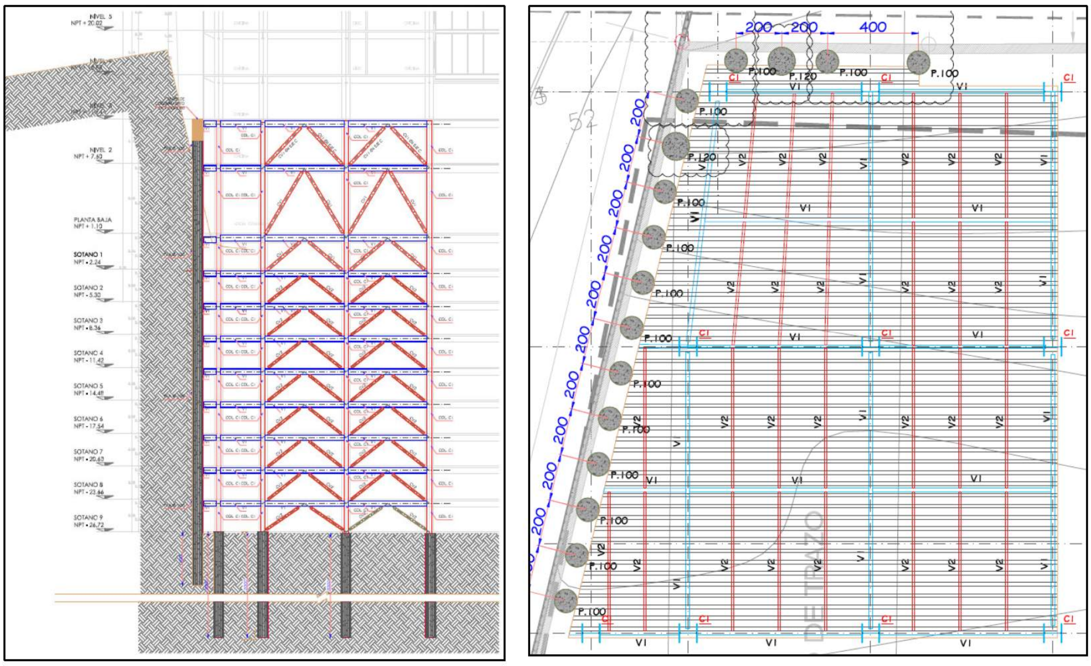


Figure 8. Project of the proposed solution alternative

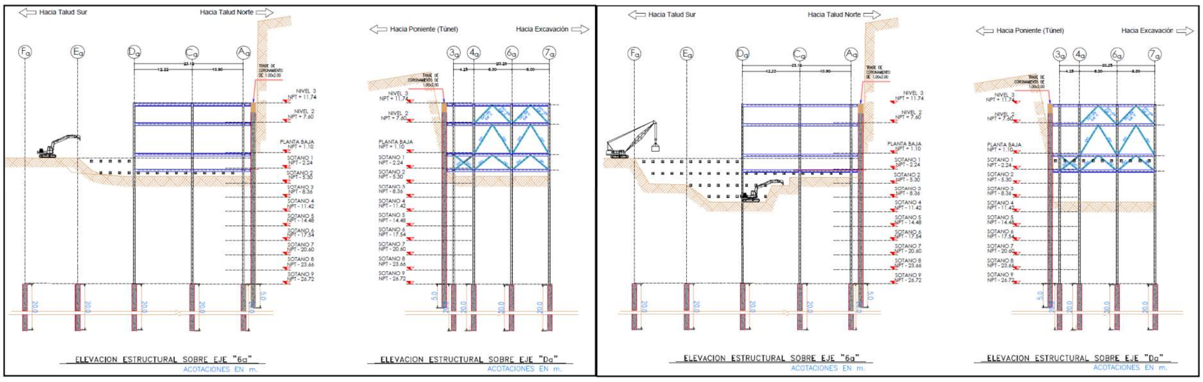


Figure 9. Excavation procedure diagram

Review of system adequacy

Once the confinement system was designed, a 2D finite element analysis was performed due to the urgency required to continue with the construction of the project.

Since the thrust diagram was structurally contemplated, the system was modeled in a similar way by placing loads in the position of each slab of the structure, equivalent to the thrust in the opposite direction to the thrust, to counteract the decompression.

As in the review, the modeling was performed in stages according to the analysis of the construction logistics. This resulted in satisfactory stages up to the lower part of the confinement system without collapse of the material (Figure 10). A comparison of the mechanical elements in the tunnel was made, giving as a result an increase of almost 60% in the magnitude of moment diagrams as shown in Figure 11. These loads were reviewed by the structural designer with the current conditions of the tunnel lining; it was concluded that the results obtained with the numerical model indicated a sufficient margin of resistance of the tunnel lining already built.

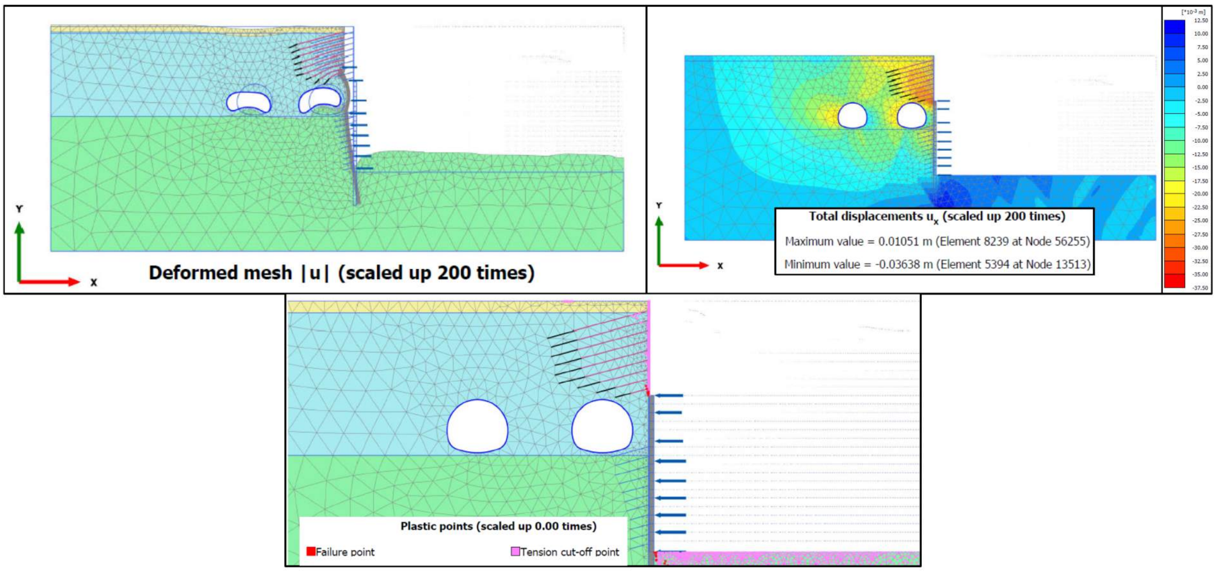


Figure 10. Results obtained for section 2 of greater height with the shoring system with the same structure

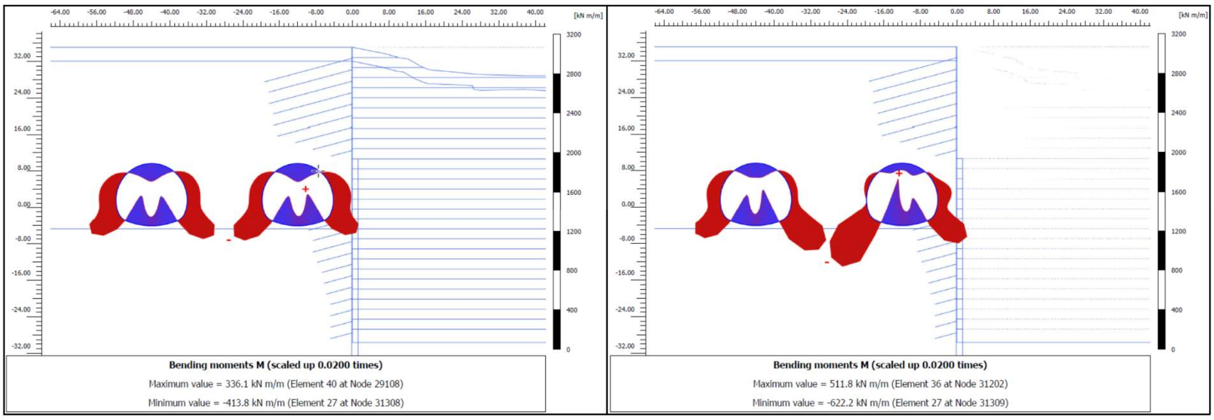


Figure 11. Comparison of moment diagrams with natural conditions with tunnels and with the effect of shored excavation with the structure

EXECUTION OF THE CONSTRUCTION PROCESS

Description of construction procedures prior to further excavation

Although the solution to the confinement of the tunnel boundary was planned and calculated, the construction procedure was a challenging activity that generated a large impact on the cost of the work.

When continuing with the excavation, the work space in the upper part above the soil-cement platform was reduced, preventing to have an advance with two drilling rigs and avoiding complications due to the depth of the piles in soils with great variability of the deep strata and with the presence of isolated rocks (Figure 12a).

It was necessary to consider that the steel elements that were embedded in the piles, weighted about 70 tons, so the lifting of these elements had to be done done in a single maneuver to prevent them from presenting any inclination inside the pile. This activity was essential to verify the verticality of each element placed in order to descend with the excavation without any torsion effect (Figure 12b).

a) Continuation of excavation in central core and construction of deep piles up to 45m according to project.

b) Lifting of steel profiles to leave in foundation piles

Figure 12. Pictures of excavation procedure, pile construction and lifting of steel profiles

To solve the problem of lifting the weight of the steel dowels, a crane with a capacity of 250 tons was used, considering that the width of the working platform did not allow the work of two machines.

The placement of these elements increased the cost of the work significantly as the crane was rented only per day to place the steel dowels, with only one machine working at a time on the narrow platform. The execution time to place all the dowels was about 4 months.

At the same time, as the placement of the pile wall at the edge of the tunnel proceeded, excavation continued in the center of the property, with vertical cuts with passive anchoring of the "Soil Nailing" type, placed only below the soil-cement platform, and in the areas that do not belong to the tunnel. The anchoring continued according to the initial design.

Once all the metal dowels were in place, the punching work began, joining the columns with the metal beams to begin to give strength to the structure. These metal profiles were punched against the capping beam with plates that were anchored to the beam and the piles according to each level of the project slab (Figure 13), in order to start working on the distribution of forces in the structure.

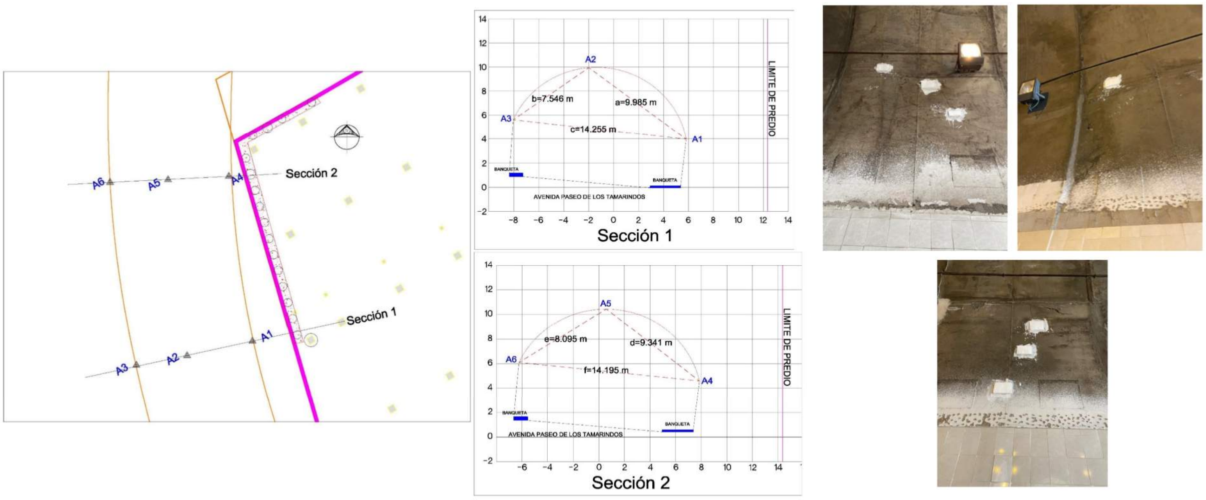
Figure 13. Continuation of the placement of dowels fastened with tie beams up to the crowning beam and piles.

Monitoring and instrumentation program

One of the measures adopted during the excavation works was the continuous topographic monitoring. Reference points were placed in the west zone to adequately control the behavior of the lateral support system.

Additionally, gypsum cores were placed in all the existing cracks inside the tunnel lining since the work was done in the 90's, having already suffered weathering effects, among others.

Finally, monitoring points were placed inside the tunnel which would help to check the convergences and divergences inside the tunnel (Figure 14). The monitoring program requested the measurements to be made every 3 days to identify possible problems not foreseen in the design of the adaptation.



Topographic monitoring points inside the tunnel

Gypsum witnesses

Figure 14. Monitoring and instrumentation systems

Continuation of the excavation with a matching system

In order to verify the safety of the excavation, it was decided to make a 3D model but now in finite differences (considering a higher solution speed in large models), using the parameters of the 2D finite element analysis. An analysis was performed almost at the same time as the construction. Figure 15 shows the schematic of the model that included the existing anchors, the piles, the abutting wall and the mezzanine slabs to support the structure.

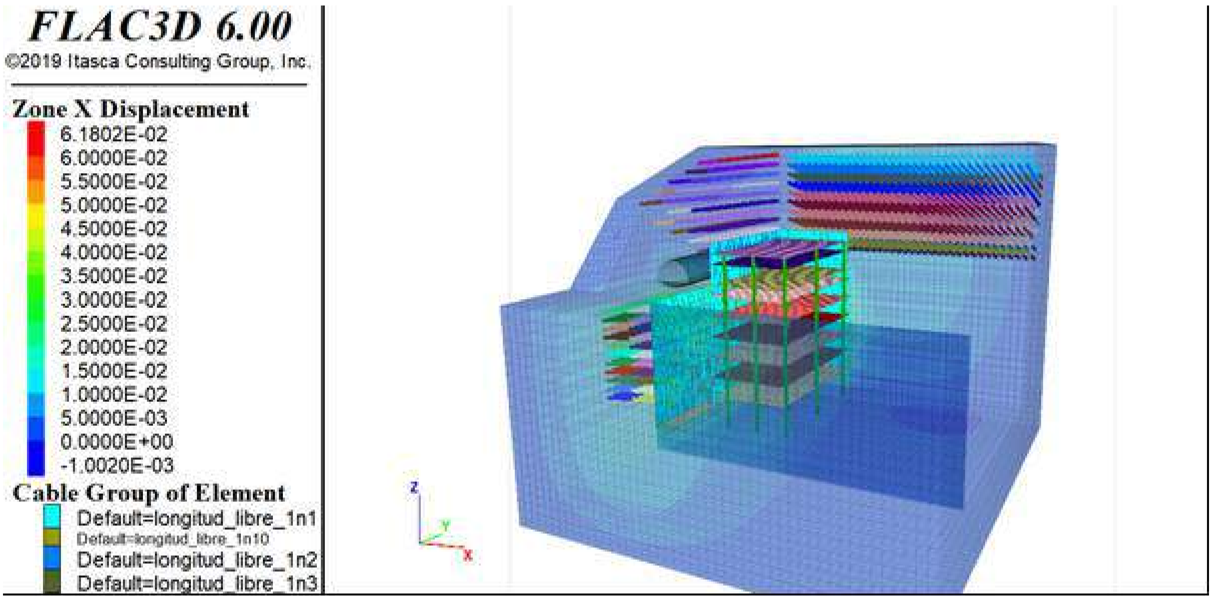


Figure 15. Finite-difference numerical model for 3D confinement system

From the results obtained, the maximum expected displacement is of the order of 3 cm in the lower part of the slope, where there is no problem in the tunnel, and in the area of the slope that covers the tunnel, displacements of the order of 1.75 cm are expected. The results obtained for the tunnel lining were also analyzed, where displacements in the order of 1 cm would be expected, which would not put the stability of the tunnel at risk according to the structural reviews. Some of the model results can be seen in Figure 16.

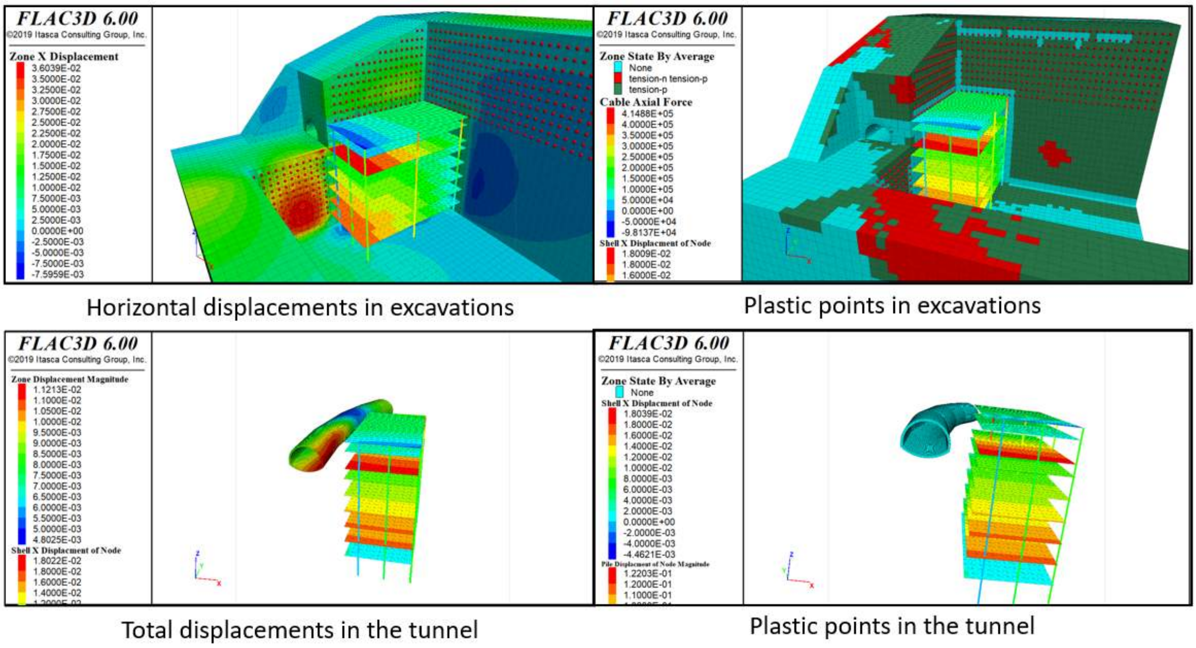


Figure 16. Results of displacements and plastic points for the 3D finite differences model.

Once the placement of the skeleton of the structure was completed, it was decided to start the excavation by removing the soil-cement platform, agreeing with the site management to work at the same time with the placement of the mezzanine slabs at the available levels (Figure 17). In accordance with the need to advance the project, it was allowed to raise the structure in the central core, so that as the excavation progressed, the structure would also become more rigid, offering greater stiffness in the confinement system.

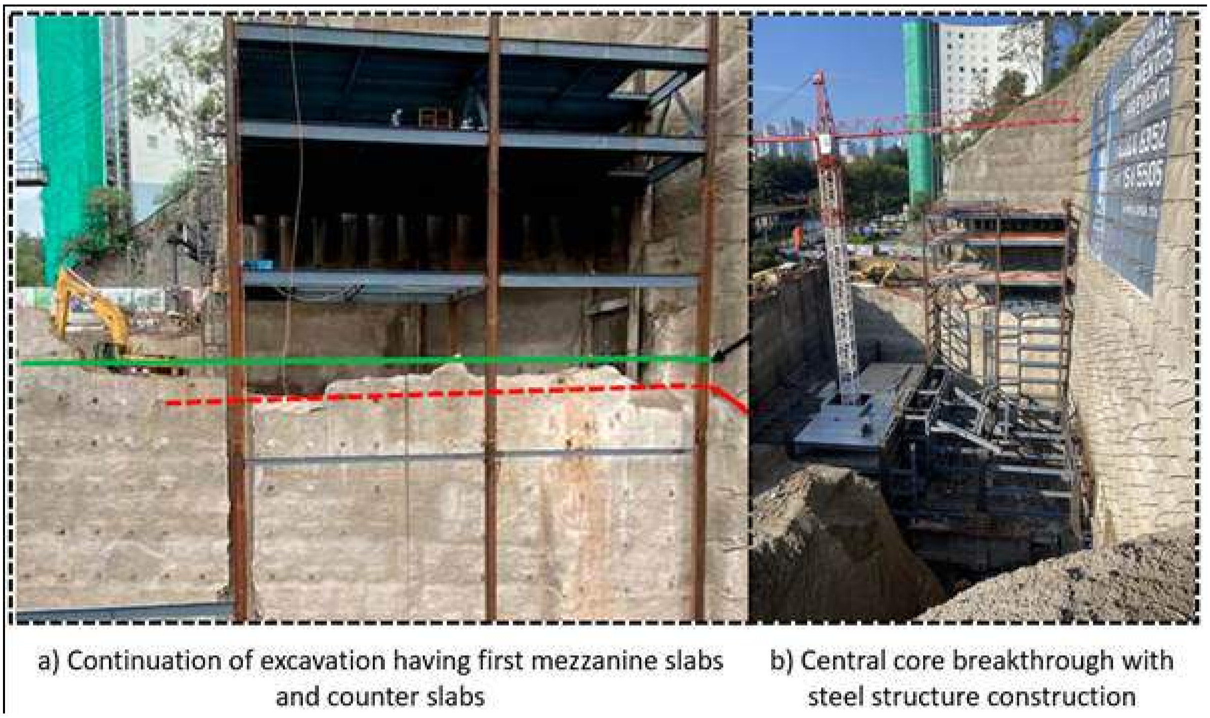


Figure 17. Continuation of excavation and construction of shoring system

Subsequently, the procedure became systematic, advancing two levels of excavation at a time and pouring slabs with counter-veneering (Figure 18) until reaching the bottom of the excavation.

Figure 18. Pictures of the different excavation advances with proposed confinement system

During the excavation period, monitoring continued, without showing any deformations outside of what was calculated and without cracking or damage to the interior of the tunnel.

Finally, to date, the construction of the superstructure has continued and no damage has occurred to the structure of the building or the tunnel. Considering the material nature, it can be assumed that current behavior represents the long term. Therefore, the stucture and the tunnel will continue steady lifetime.

CONCLUSIONS

Some analytical design methods tend to make many simplifications and omit or ignore relevant details, so it is important to have constant monitoring during the work and ensure that the design proposed with the support of more sophisticated models.

It was possible to detect in time the lack of calculations and initial considerations, to later complement them with finite element analysis to propose the adequacy in the excavation stabilization project.

To contain the horizontal thrusts due to stress relaxation around the tunnel, a lateral shoring system was used that was sufficiently rigid to transmit the lateral forces to the structure.

The omission in the initial analyses generated a cost overrun which was borne by the construction company; however, the cost of the work was affected by the procedure requested to preserve the safety of the project.

According to the approach of the laterally supported diaphragm wall with the structure, a good performance was observed provided that the construction procedure included the minimum elements necessary to guarantee the transfer of forces from the slope to the structure.

From the topographic measurements and the continuous monitoring inside the tunnel, it is concluded that the lining has not presented deformations that put its stability at risk.

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A Case Study- Tunnelling adjacent to major water transfer tunnels

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ABSTRACT

As demands for infrastructure continue to grow to meet our expanding population and cities, interaction with existing assets is frequent. A key aspect in developing new infrastructure is recognising these fundamental interactions early in the project development process and developing robust design solutions that minimise and control impacts. WestConnex is part of an integrated transport plan to keep Sydney moving - easing congestion, creating jobs and connecting communities. The new motorway will support Sydney's long-term economic and population growth. The M4-M5 Link Tunnels are the final and most critical component of WestConnex. The project involves designing and constructing twin 7.5 kilometre tunnels linking the M4 Tunnels at Haberfield with the M8 at St Peters and accommodates up to four lanes of traffic in each direction.

Sydney Water Assets in the vicinity of the M4-M5 Link Tunnels included the City Tunnel and the Pressure Tunnel. These are critical potable water transfer tunnels between Potts Hill Reservoir in Sydney's west and Waterloo, east of the city. The two tunnels were constructed in the mid and early 20th Century. The alignment of the M4-M5 Link Tunnels cross underneath the City Tunnel and over the Pressure Tunnel in the vicinity of the existing Newtown Station.

As part of this paper, the authors will present the following:

- *a case history documenting the history and construction of the transfer tunnels*
- *key drivers for the Asset Owner*
- *the design solution and construction methods utilised for the crossings,*
- *the instrumentation and monitoring regime which was developed.*
- *observations from the monitoring data obtained during construction and the outcomes from a back analysis undertaken to verify the design assumptions.*

INTRODUCTION

WestConnex is part of an integrated transport plan to keep Sydney moving - easing congestion, creating jobs and connecting communities. The new motorway will support Sydney's long-term economic and population growth.

In 2018 the joint venture of Acciona, Samsung and Bouygues (ASBJV) was awarded the design and construct contract for the WestConnex M4-M5 Link Tunnels project, which was completed in 2023. The project was constructed in partnership with ASBJV and the owner and operator of WestConnex, which is owned by a consortium led by road operator, Transurban. A joint venture between Jacobs and Aurecon undertook the design of the tunnels.

The project involved designing and constructing twin 7.5 km tunnels linking the M4 East at Haberfield with the M8 at St Peters and accommodating up to four lanes of traffic in each direction.

The M4-M5 Link Tunnels crossed below the City Tunnel and over the Pressure Tunnel in Newtown, marked in Figure 1, are major Sydney Water assets.

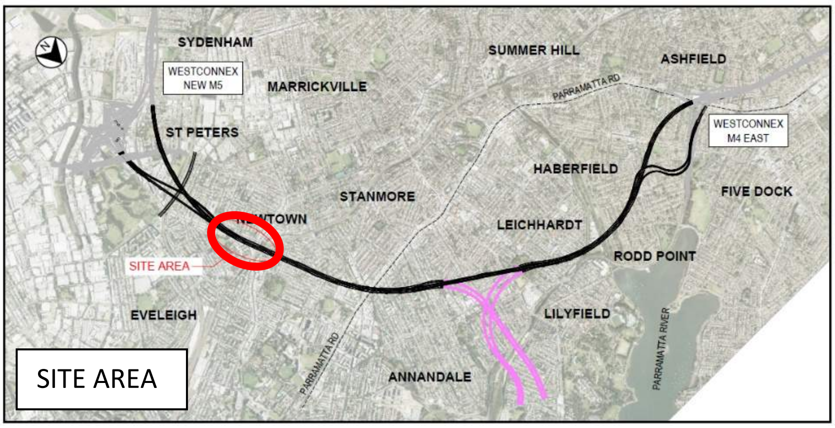


Figure 1: M4-M5 Alignment

A high level framework with key design criteria was introduced at planning stage. Protocols were then established during the design stage so that while the main tunnel headings were within the 150 m wide interface zones on either side of the water tunnels, ground and tunnel movements were monitored, reviewed and compared against predictions. The results were presented and discussed at regular interface meetings.

Details of the City and Pressure Tunnels

The City and Pressure Tunnels are operational and supply potable water from the Potts Hill Reservoir in Sydney's west to Waterloo in the city's eastern suburbs. Given the age of the tunnels, accurate historical records of as constructed materials and original construction methods utilised were a challenge to identify. Assessments were carried out based on best available information and sensitivity checks.

City Tunnel

The City Tunnel is approximately 17 kilometres in length and 30 to 80 metres below ground. The tunnel is horseshoe-shaped, as shown in Figure 2 and was constructed using drill and blast techniques. A 2.1 metre diameter cement-lined mild steel pipe was installed in the tunnel and surrounded with concrete.

The 12-ft (3.6metre) steel pipes lining the tunnel comprised a bell-shaped socket at one end to fit over a slightly splayed spigot. The pipe lengths were fully welded throughout the section.

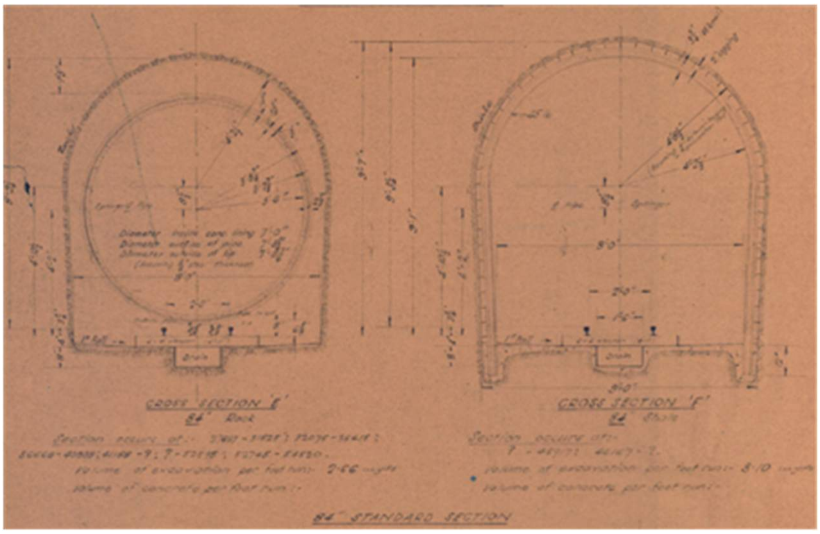


Figure 2: City Tunnel Standard Section details

Pressure Tunnel

The Pressure Tunnel is 16 kiolmetre long and between 46 and 118 metres below ground level. It is approximately 3.8 metres in diameter and was excavated by drill and blast techniques during the 1920s and 1930s. The tunnel has been fitted with an internal steel lining of 2.515 m diameter, which like the City Tunnel, is a cement-lined steel pipe, as shown in Figure 3.

The lining for the Pressure Tunnel was constructed using 3.6 m long mild steel tubes with 12 millimetre wall thickness, and each tube section was connected by sockets incorporated into each pipe. The socket connections resulted in a 1/2" to 5/8" (12 – 13 millimetre) annulus gap around the entire circumference which was sealed with a rubber gasket and filled with lead caulking, as shown in Figure 4.

The Pressure Tunnel was also "gap grouted" to eliminate the air gap between the original lining and the rock. Furthermore, when the steel pipe was installed, it was designed with an 8" (20 centimetre) gap above the concrete backfill and the original lining to provide relief of the external groundwater pressure.

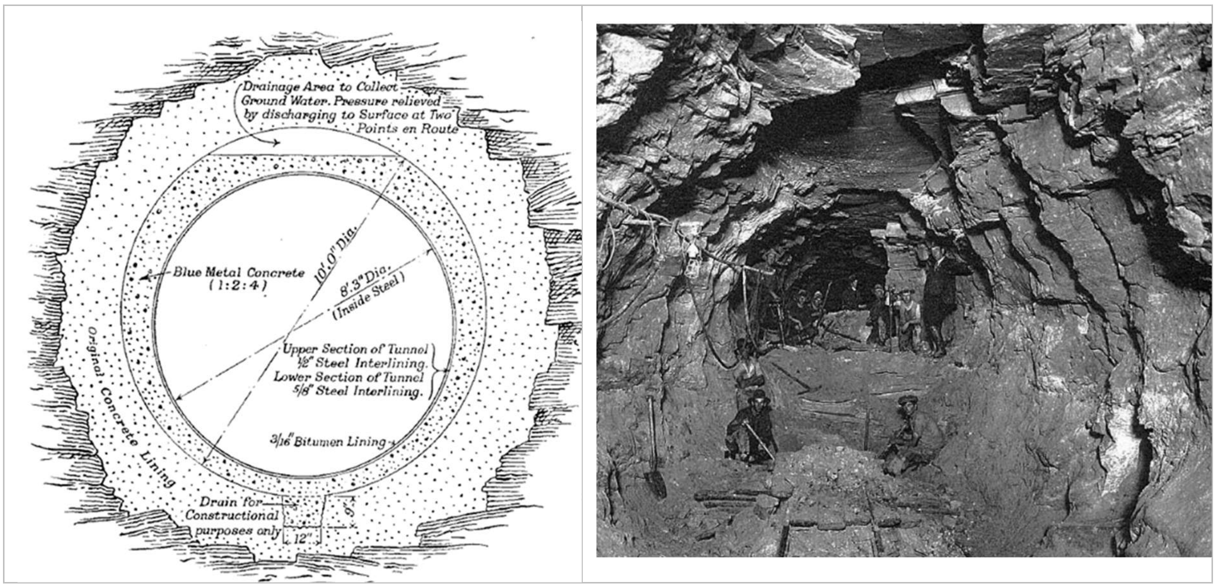


Figure 3: Sydney Water Pressure Tunnel details (G Haskins, 1932)

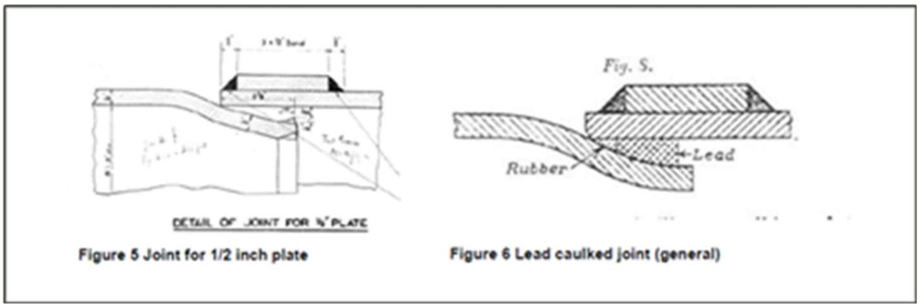


Figure 4: Indicative Joint Details of Pressure Tunnel. (Aecom, 2017)

Movement Criteria and Design Requirements

The permissible movement limitations defined in the project requirements were as follows:

- City Tunnel
    - displacement of 15 mm in any direction of any part of the lining of the Sydney Water City Tunnel; and
    - angular distortion of 1:1500 at any location along the length of the Sydney Water City Tunnel.
- Pressure Tunnel
    - displacement of 10 mm in any direction of any part of the lining of the Sydney Water Pressure Tunnel; and
    - angular distortion of 1:2000 at any location along the length of the Sydney Water Pressure Tunnel.

However, in addition to the above, Sydney Water required an independent assessment to ensure no adverse impacts are induced. Hence, detailed assessments were undertaken to demonstrate that the movement criteria were satisfied and appropriate risk mitigation measures were implemented. As part of the impact assessment, the scenarios which needed to be considered were as follows:

- collapse of rock tunnel lining (i.e. mild steel lining and concrete encasement); of the Sydney Water Tunnels;
- overstressing mild steel lining and/or disjointing of the internal cement lining;
- buckling of the steel lining;
- impact of construction vibration on tunnel lining;
- disturbance to the lead/rubber joint of the Pressure Tunnel; and
- structural assessment of steel liner for stability, strength, serviceability and durability, including impacts on the cement lining.

Geological Setting

At the tunnel horizon of the mainline tunnels, the ground profile was expected to comprise Hawkesbury Sandstone Class I and II with small proportions of Class III. At the Pressure Tunnel crossing, the mainline tunnel is closer to the Mittagong Formation, which was expected to be present within the support zone above the tunnel crown.

The City Tunnel was located within the Ashfield Shale and the Mittagong formation, as shown in Figure 6. Approximately 10 metres of the Hawkesbury Sandstone and the lower sandy Mittagong formation separated the City Tunnel from the Mainline Tunnels. The Pressure Tunnel was also located within Hawkesbury Sandstone.

Groundwater data indicated a groundwater level of approximately 4 to 5 metres below the surface at both tunnels.

DESIGN AND CONSTRUCTION OF THE M4-M5 LINK TUNNELS

The M4-M5 Link tunnels were constructed sequentially for excavation and support installation. The excavation was undertaken using roadheaders and support comprised a combination of permanent rockbolts and sprayed concrete, as shown in Figure 5.

City Tunnel

The M4-M5 Link Tunnels crossed below the City Tunnel. The clearance between the tunnels was 14.7 metres above the ramp and a minimum of 11.5 metres above the mainline tunnels. In order to minimise the effects of ground movement, the road alignment was developed to minimise the cross-sectional area of the tunnels at the crossing such that the City Tunnel was located above the pillar of the Y-junction as shown in Figure 6, rather than further north where the span of the cavern was greater.

Initially, the northbound tunnel was advanced from south to north in a split heading sequence, leaving a temporary central pillar in place. This pillar was progressively removed to open up the full span of the cavern, as shown in Figure 7.

Pressure Tunnel

The M4-M5 Link Tunnels corridor crossed over the Pressure Tunnel and the clearance between the tunnels was approximately 7.6 meters as illustrated in Figure 8.

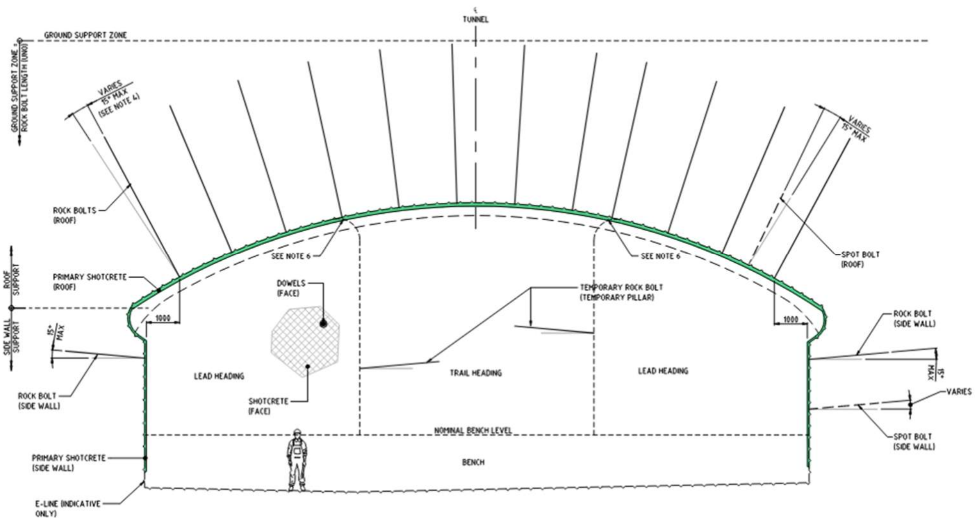


Figure 5 Typical M4-M5 Link tunnel profile and support

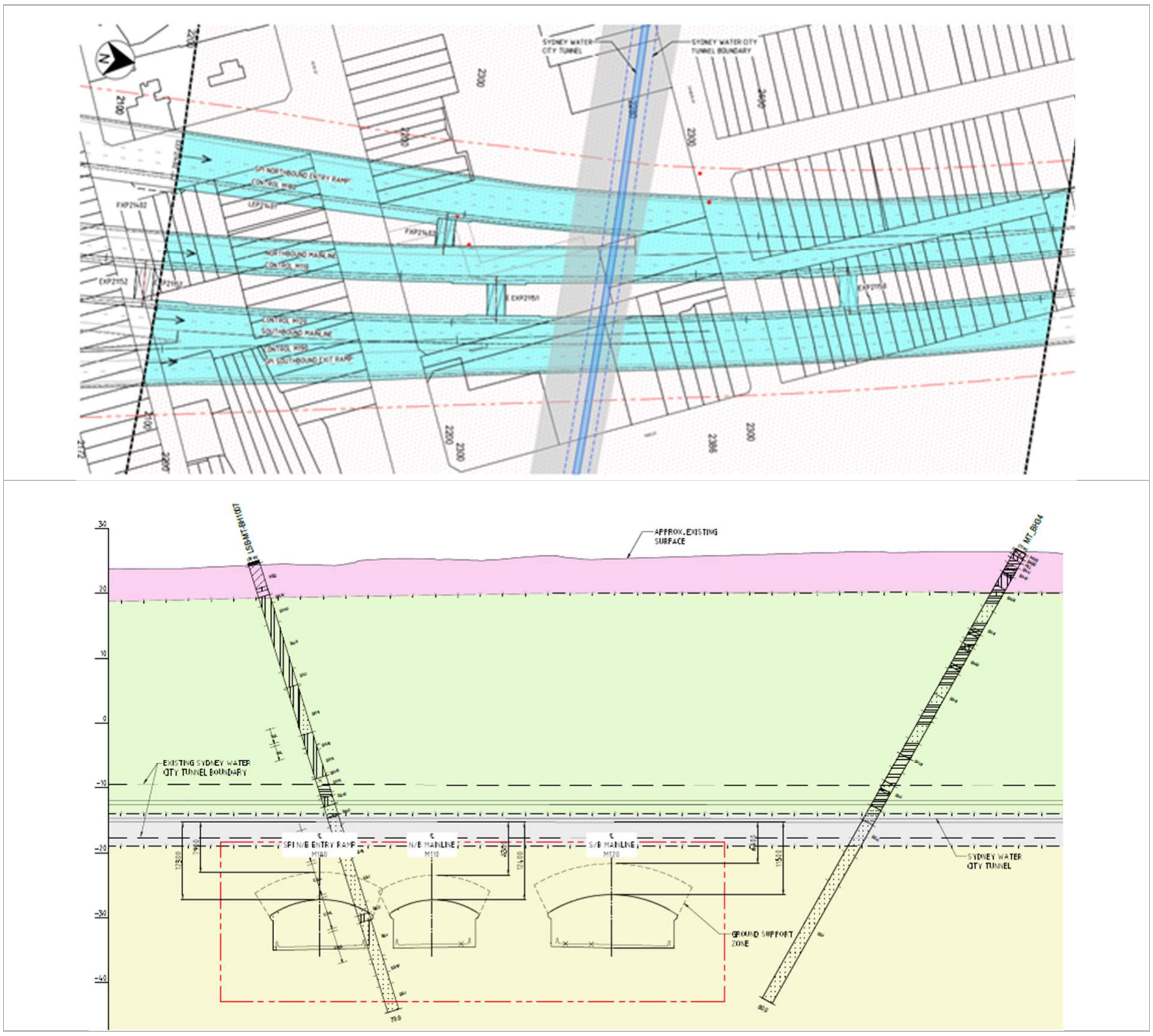


Figure 6 Plan and Section of M4-M5 Link Tunnels and City Tunnel

Figure 7 Construction of Cavern (temporary pillar removal) adjacent to the City Tunnel

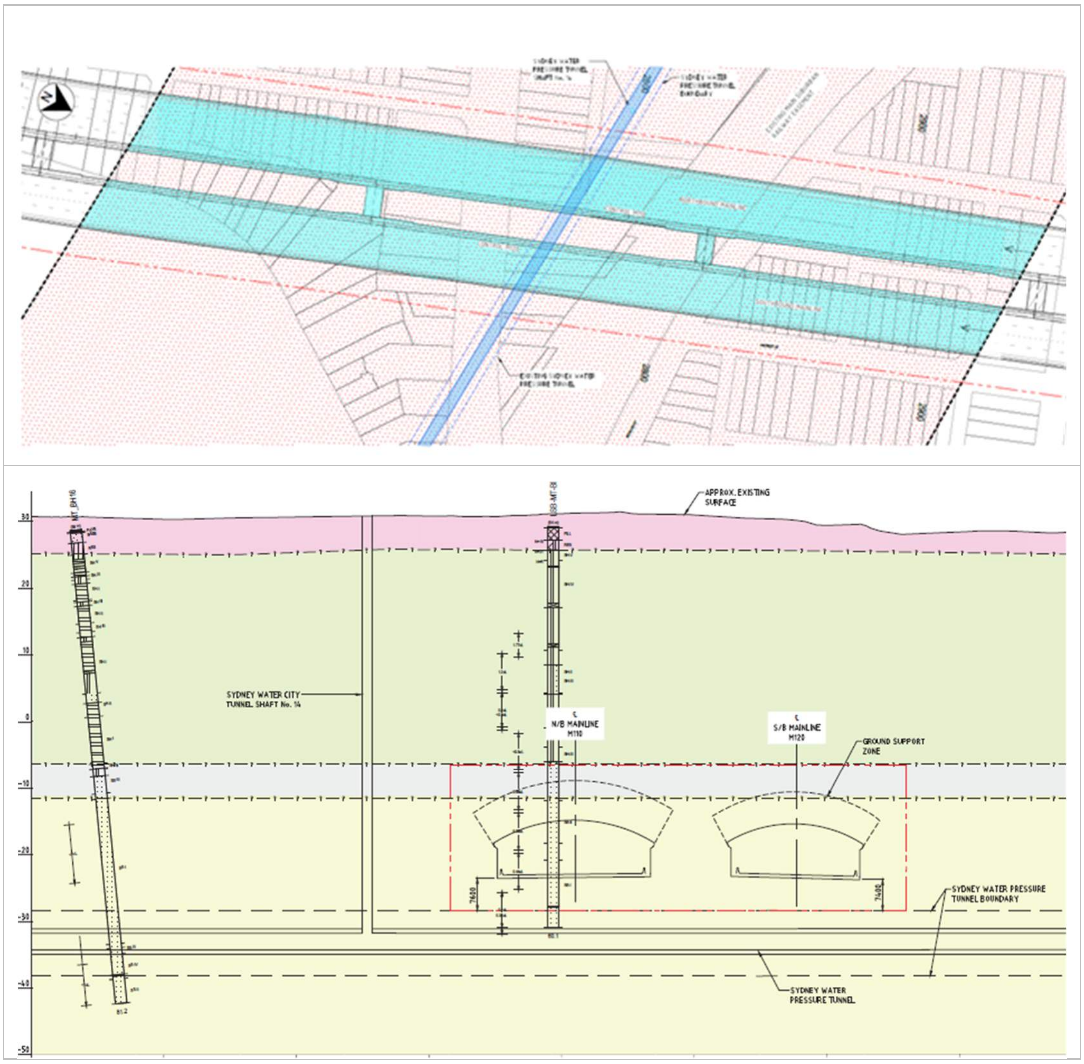


Figure 8 Plan and Section of M4-M5 Link Tunnels and Pressure Tunnel

GROUND MOVEMENT IMPACT ASSESSMENT

A 3D numerical analysis was undertaken for both the City and Pressure Tunnels crossings to assess predicted ground movements. Three scenarios were examined to consider the effect of varying ground conditions using a set of realistic assumed ground conditions. These were as follows:

- Case 1 - (Base Case) where the tunnel face and support zone are within Class I Hawkesbury Sandstone
- Case 2 - where the tunnel face and support zone are within Class II Hawkesbury Sandstone
- Case 3 - where the tunnel face and support zone are within Class I Hawkesbury Sandstone, apart from an approximate 5 m thick band of Class III sandstone is assumed above and below the tunnel crown.

The effects of drill and blasting operation during the City and Pressure tunnels excavation were simulated with a disturbance zone of 2 metres around the outer layer of the concrete structure.

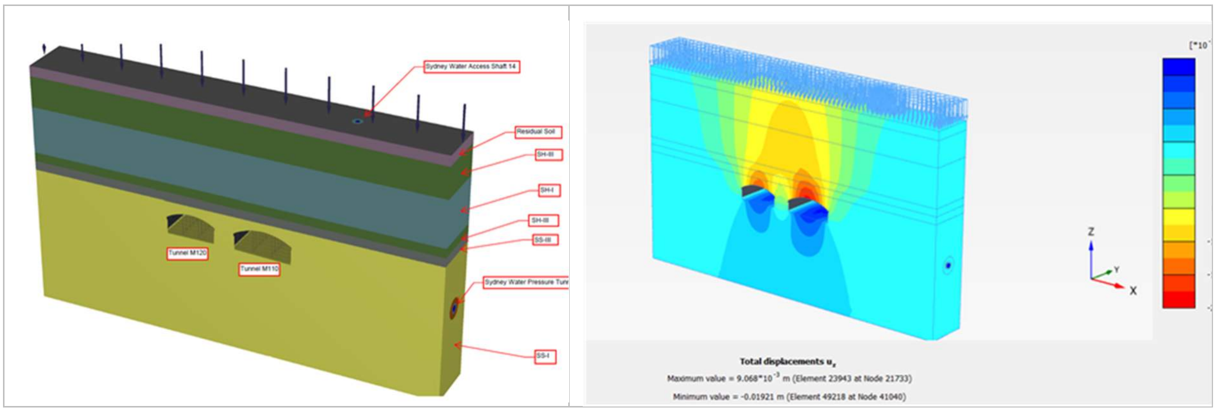


Figure 9: 3D Model of Pressure Tunnel and vertical displacement contours

The acceptable limits predicted through the analyses and the most onerous cases were used to define the critical limits.

Table 1 Three-Dimensional Numerical Geotechnical Analysis Results

Asset	Movement Parameters	Case - 1	Case - 2	Case - 3	Limiting Criteria	Critical Capacity Ratio
City Tunnel	Max. total Vertical Movement (mm)	10	15	15	15	28%
	Max. Differential Movement (mm)	1	2	2		
	Max. Angular Distortion	1: 3600	1: 2250	1:1895	1: 1500	N/A
	Max. total Horizontal Movement (mm)		2		4	51%
Pressure Tunnel	Max. total Vertical Movement (mm)	4	6	4	10	48%
	Max. Differential Movement (mm)	1	<2	<2		
	Max. Angular Distortion	1:2922	1:2000	1:2541	1:2000	N/A
	Max. total Horizontal Movement (mm)		1		2	48%

Case 1 represented the most probable geological conditions that were anticipated to be encountered during construction. Cases 2 and 3 served as sensitivity analyses for ground conditions and stresses. Cases 2 and 3 indicated movement of approximately 15 millimetres on the City Tunnel. However, these were not the governing cases and were considered to have a low probability of occurrence. As a result, it was expected that the movements induced during construction would be below 15 millimetres. Movements induced on the Pressure Tunnel were expected to be heave due to the unloading effect from the mainline tunnels. The magnitude of the movement was expected to be in the order of 4 to 6 millimetres.

City Tunnel

A three-dimensional plate structural model was used to assess the stresses in the steel lining and the compressive limit of the cement lining. The model included two "shells" representing the steel lining and the cement lining. These were assumed to be fully bonded to conservatively achieve the largest stress transfer between the steel and the cement lining.

The analysis showed that the maximum induced stress in the steel liner due to a 0.09° rotation was less than the tensile stress capacity of the steel, reduced by a capacity reduction factor of 0.9.

Table 2 Results of the assessment of the City Tunnel 3D plate structural model

Parameter	Predicted rotation of 0.04º	Acceptable rotation of 0.09º	Allowable Limit
Cement Lining Axial Force (kN)	67	147	153
Steel lining Stress (MPa)	86	190	225

The effects of excavating the M4-M5 Link Tunnels were explored further using closed-form solutions to simulate the settlement slope using the approach by Loganatham and Poulos (1998). Based on the results extracted from this analysis, the stresses at the steel pipe's top and bottom were determined. The compressive stress induced on the steel and the cement lining was assessed within acceptable limits.

The steel lining was also checked for the hoop stresses developed when the pipe was operational and subjected to an internal pressure of 710 kPa, assuming a 71 metre water head. The stresses were within acceptable limits, as illustrated in Table 3.

Finally, the lining of the City Tunnel was also checked against buckling. Buckling could only occur when the pipe is empty of water, as when the pipe is operational, the liner is subjected to hoop tensile stresses. The check was carried out assuming a pressure on the City Tunnel based on the known groundwater level. The M4-M5 Link Tunnels will lower the groundwater level. Hence the buckling scenario was not a governing condition.

Table 3 City Tunnel Results of Impact Assessment

Parameter	Maximum Assessed	Allowable Limit	Maximum Assessed	Allowable Limit
	X-Z Direction		X-Y Direction	
Rotation (degrees)	0.04	0.09	0.01	0.09
Compressive Stress – Cement (MPa)	1.9	10	2.35	10
Compressive Stress – Steel (MPa)	19.2	165	23.51	165
Tensile Stress – Cement (MPa)	0.46	3	0.69	3
Tensile Stress – Steel (MPa)	4.6	165	69.5	165
Hoop Stress – Steel (MPa)	56	165	56	165
Von Mises Yield Criterion for Steel (MPa), LFS-1.2	78.4	225	81.4	225
Von Mises Yield Criterion for Steel (MPa), LFS-1	67.5	165	70.6	165
Pressure applied on steel lining – buckling check (kPa), LFS-1.2	520	1250	520	1250
Localised buckling Check -Steel (MPa)	19.2	116	23.51	116
Cracks width (upper limit) - Longitudinal Stress (mm)	<0.1	1.3		
Cracks width (upper limit) - Hoop Stress (mm)	0.7	1.3		
Weld Stress (MPa)	122	170		

Pressure Tunnel

The stresses in the steel and cement linings were assessed similarly to the City Tunnel, using a three-dimensional plate model comprising two bonded shells representing the steel pipe and cement lining. The model was deformed to induce a rotation and determine whether a 0.06° constituted an acceptable criterion for joint rotation.

From the analysis, the maximum induced stress in the steel liner due to a rotation of 0.06° would be less than the tensile stress capacity of the steel, reduced by a capacity reduction factor of 0.9.

The performance of the Pressure Tunnel lead caulked joints was assessed using a detailed 3D numerical model. It was concluded that it was reasonable to assume a 0.06° rotation as an acceptable upper limit even though the predictions indicated the expected rotations were in the order of 0.03°, i.e. with a factor of safety of approximately 2.

Table 4 Results of the assessment of the Pressure Tunnel 3D plate structural model

Parameter	Predicted rotation of 0.03°	Acceptable rotation of 0.06°	Allowable Limit
Cement Lining Axial Force (kN)	52	102	103
Steel lining Stress (MPa)	62	122	225

When the steel liner was installed, the annulus between the liner and concrete backfill was gap grouted, as shown in Figure 3, and a drain was created in the tunnel's crown. This provides a pressure relief mechanism and prevents high-induced stresses on the steel liner due to external groundwater pressure or unbalanced external water pressure when the tunnel is dewatered for maintenance.

A 3D numerical model was developed (see Figure 10) to assess the limiting joint rotation of the spigot and socket joints in the steel liner and to examine the likelihood of the external pipe coming into contact with the unsupported spigot. It also was used to assess the possible movements that could occur before the water-tightness of the joint was compromised.

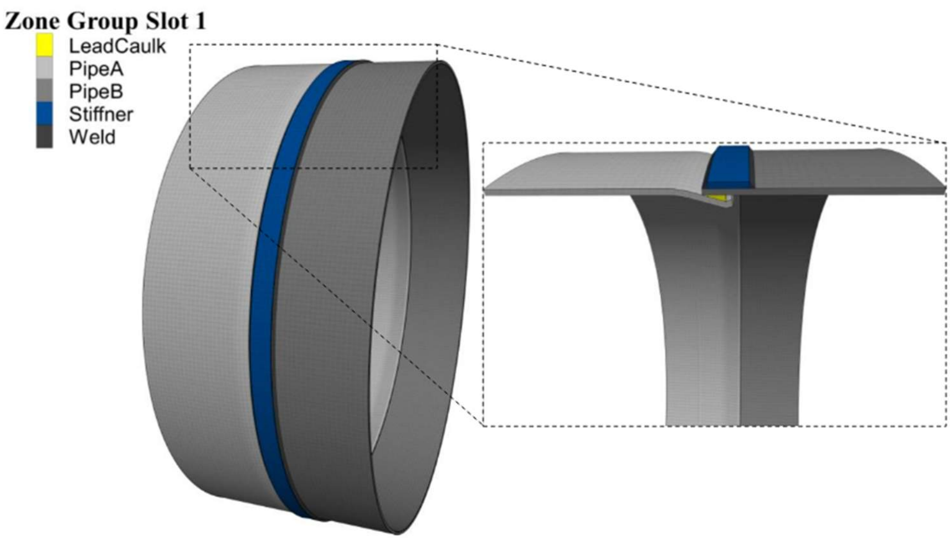


Figure 10 Geometry of 3D Joint Model

The modelling results indicated that the risk associated with the edge of the external pipe coming into contact with the unsupported spigot is low because it would have been separated during the manual caulking during the construction of the joints. Even when the joint was subjected to an axial inward displacement of 0.5 millimetres (compression), significantly over the predictions, combined with a maximum rotation of 0.06°, the risk was considered low, as confirmed by the negligible changes in Von Mises stresses in the pipe.

The numerical model also indicated that for a maximum rotation of 0.06º, the lead caulking would still be in contact with both sides of the joint and without the material yielding, which meant that the rubber gasket would still be in compression and with likely acceptable water-tightness performance. Beyond this rotation, some loss of contact in the lead caulking could occur, with a potential increase in leakage. As a result, a rotation of 0.06º was set as the upper limit despite the expected rotations in the order of 0.03º, i.e. with a factor of safety of approximately 2.

Table 5 Pull out and Joint Rotation for Pressure Tunnel

Parameter	Case 1 – M4-M5 Tunnels within SS-I	Case 2 – M4-M5 Tunnels within SS-II	Case 3 – M4-M5 Tunnels within SS-I with 5 m band on SS-III on roof of tunnels.	Acceptable limit
Calculated Joint Rotation (º)	0.02	0.03	0.02	0.06
Calculated Joint Pull-out (mm)	0.9	1.3	0.92	2

Table 6. Pressure Tunnel Results of Impact Assessment

Parameter	Maximum Assessed	Allowable Limit	Maximum Assessed	Allowable Limit
	X-Z Direction		**X-Y Direction**	
Rotation (degrees)	0.03	0.06	0.003	0.06
Compressive Stress – Cement (MPa)	0.19	10	0.116	10
Compressive Stress – Steel (MPa)	1.96	165	1.161	165
Tensile Stress – Cement (MPa)	N/A	N/A	N/A	N/A
Tensile Stress – Steel (MPa)	N/A	N/A	N/A	N/A
Hoop Stress – Steel (MPa)	85	165	85	165
Von Mises Yield Criterion for Steel (MPa), LFS-1.2	102.6	225	102.6	225
Von Mises Yield Criterion for Steel (MPa), LFS-1	85.6	165	85.6	165
Pressure applied on steel lining – buckling check (kPa), LFS-1.2	624	670	624	670
Localised buckling Check -Steel (MPa)	1.96	116	1.96	116
Cracks width (upper limit) - Longitudinal Stress (mm)	<0.1	1.3		
Cracks width (upper limit) - Hoop Stress (mm)	1.2	1.3		

Instrumentation and Monitoring

Sydney Water required that monitoring be undertaken for a minimum period of three months before the commencement of works in the Interface Zone to establish baseline data. The monitoring also needed to continue after completing works within the Interface Zone.

A comprehensive monitoring regime was developed, which monitored ground movements within on the approach to the Sydney Water tunnels to validate the design predictions and hence confirm the behaviour of the tunnel support system.

The monitoring instruments comprised a combination of the following:

- Instruments installed from the surface
  - triple head extensometers, inclinometers and geophones.
- In-Tunnel Instruments
  - triple head extensometers
  - convergence arrays
- Instrumentation within the Pressure Tunnel
  - strain gauges were located at third points around the steel lining at the joints and midpoints, orientated to measure the joint opening and hoop deflection.

BACK ANALYSIS OF THE M4-M5 LINK TUNNELS

In advance of the tunnel headings crossing the City and Pressure Tunnels, a detailed back analysis was undertaken, which replicated the construction sequences and was used to validate the performance of the support systems and the impact assessment.

3D numerical models were used to compare the predictions derived from the back analysis with the measured ground movement monitoring data in the vicinity of the City Tunnel and Pressure Tunnel. This enabled the validation of the modelling parameters and movement predictions.

Numerical modelling

3D continuum numerical models were developed using the Plaxis 3D finite element program to assess ground deformation due to tunnelling, as shown in Figure 11 and Figure 12. The rock mass was modelled as an elastic-perfectly plastic continuum material with a Hoek-brown failure criterion, while near-surface material was modelled as an elastic. Rock mass discontinuities such as bedding planes and sub-vertical joints were not explicitly included. The rock-shotcrete interface was modelled as an elastoplastic material with the Mohr-Coulomb failure criterion. The construction sequence of tunnels in the vicinity of the City and Pressure Tunnels were simulated for advance length and heading heights.

The major horizontal in-situ stress (σ_H) direction within the Sydney basin is typically 20° east of north (Pells 2002), and the direction of minor horizontal in-situ stress (σh) is normal to σ_H. The in-situ stresses measured in the vicinity of the City and Pressure Tunnels indicated that major horizontal in-situ stress orientation was along the longitudinal tunnel alignment. The relationships between horizontal and vertical in-situ stresses proposed by Oliveira & Parker (2014) were adopted to capture rock mass quality and stiffness varying with depth.

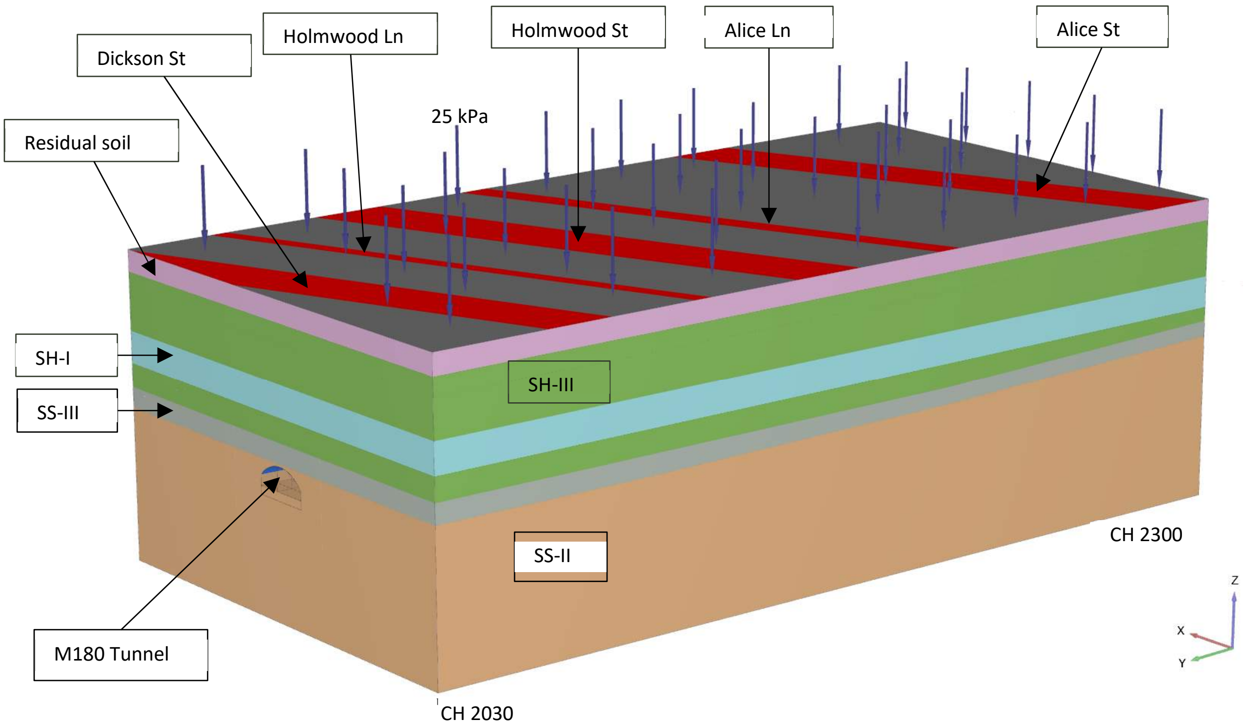


Figure 11: 3D Finite element model of M180 tunnel between CH 2030 and CH 2300 for City Tunnel interface

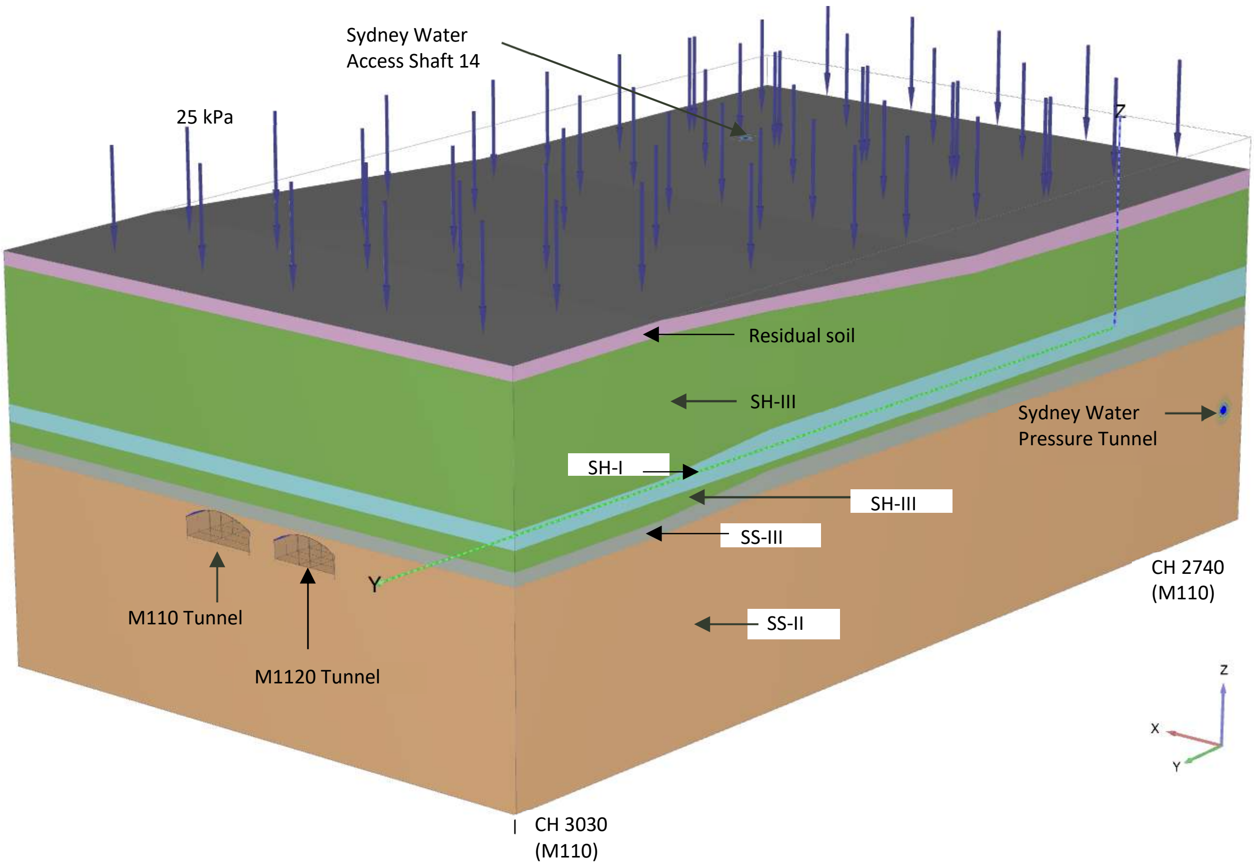


Figure 12: 3D Finite element model of M110 and M120 tunnels between CH 2740 and CH 2300 on M110 for Pressure Tunnel interface

Model validation

In-tunnel movement

Figure 13 shows a typical configuration for the in-tunnel monitoring targets. These targets were installed 2.5 metres from the excavation face. Hence when comparing results with the numerical model predictions, the displacements that occurred before installation need to be removed. The numerical analysis results show that 30% of total displacement typically occurred before in-tunnel targets could be measured. Hence the predicted values only show the portion of displacements post-commencement of measurement.

A comparison of the maximum predicted (adjusted) and measured displacements from the in-tunnel monitoring points at the City and Pressure Tunnels is presented in Table 7 and Table 8. The comparison generally demonstrated that the modelling approach adequately represented the overall tunnel behaviour with the magnitude of crown displacements comparable with the model predictions, i.e. measured values were typically 30-70% of the predictions. It should be noted that localised differences between predicted and measured at the excavation boundary are to be expected due to rock discontinuity effects.

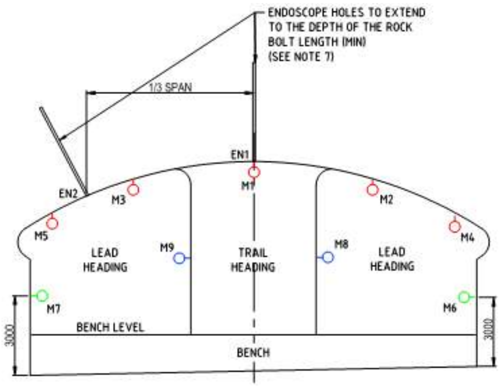


Figure 13. Typical in tunnel monitoring points

Table 7. Comparison between predicted and measured in-tunnel displacements at CH 2148 – City Tunnel interface

Displacement	Vertical (mm)					Horizontal (mm)	
Target	M1	M2	M3	M4	M5	M6	M7
Predicted (before targets installation)	6.5	2.6	1.3	2.6	1.4	5	3.7
Predicted (total)	15.8	13.8	13.5	9.3	8.6	7.8	7.2
Predicted (adjusted)	9.3	11.2	12.2	6.7	7.2	2.8	3.5
Measured	4.8	9.1	19.2	4.1	6.0	0.7	2.3
Ratio between measured and predicted	52%	81%	157%	61%	83%	25%	66%

Table 8: Comparison between predicted and measured in-tunnel displacements at CH 2972 (M120) – Pressure Tunnel interface

Displacement	Vertical (mm)					Horizontal (mm)	
Target	M1	M2	M3	M4	M5	M6	M7
Predicted (before targets installation)	17	6.6	5.8	4.9	4.4	4.9	8.1
Predicted (total)	26.6	21.3	21.5	13.6	11.8	3.9	11.2
Predicted (adjusted)	9.6	14.7	15.7	8.7	7.4	1.0	3.1
Measured	4.3	9.3	14.5	4.7	3.3	1.0	7.6
Ratio between measured and predicted	45%	63%	92%	54%	45%	100%	245%

Surface extensometer measurements

Figure 14 (a) shows the predicted vertical displacement profile at a surface extensometer located approximately 21m east of the centreline of the M180 tunnel. The measurement points P1, P2, P3 and P4 along the extensometer corresponded to City Tunnel's location and are highlighted in Figure 14 (a). The predicted differential vertical displacement (i.e. between the head of the extensometer and a target measurement point) has been calculated from Figure 14 (a) and compared with measured differential vertical displacement (see Figure 14 (b)). The results show that the measured differential vertical displacements were less than predicted.

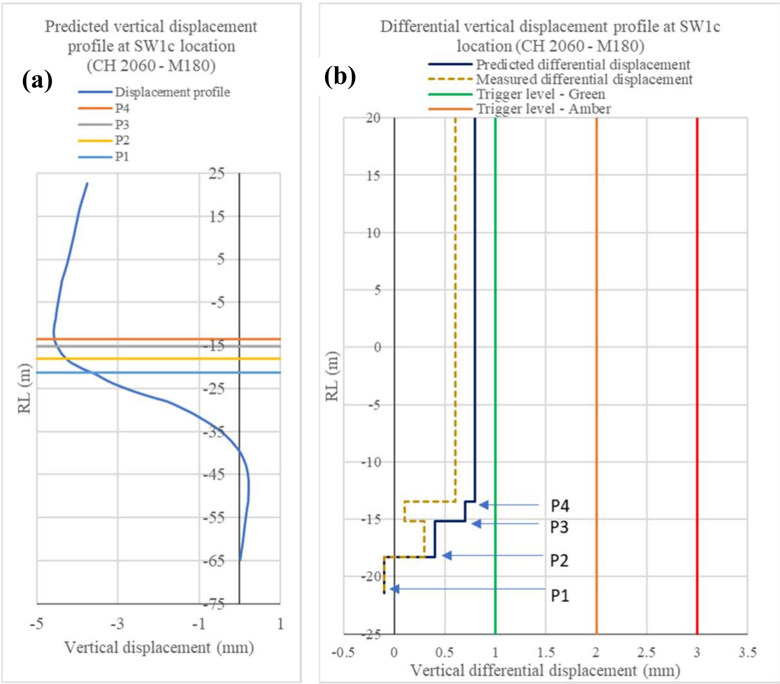


Figure 14. (a) Predicted vertical displacement profile at SW1c surface extensometer location with respect to measurement points P1, P2, P3 and P4 and
(b) Comparison between predicted and measured differential vertical displacement at location P1, P2, P3 and P4

Similarly, in the Pressure Tunnel, the results show that the measured differential vertical displacements/extensions are less than predicted (see Figure 15). The extensometer showed that there was a minimal displacement of the measurement points below the invert of the mainline tunnels as expected, i.e. less than 0.5 millimetres. The point at RL -9.9m (P4), which is aligned with the rockbolt horizon, also indicated negligible movement i.e. less than 0.5 millimetres. The reason for the point at RL-9.9 metres exhibiting such a small displacement when the movement is compared with the tunnel convergence that generally correlated well predictions could be due to the rock conditions as most of the movement probably occurred close to the boundaries of the excavation. Hence, the movement of the point, which was 4 to 5 metres radially from the excavation, was not picked up.

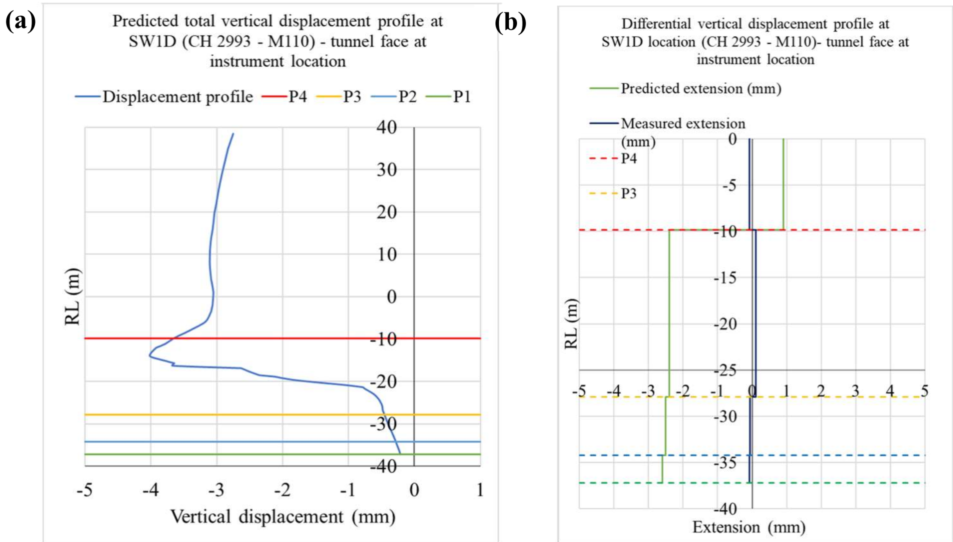


Figure 15: (a) Predicted vertical displacement profile at SW1D surface extensometer location with respect to measurement points P1, P2, P3 and P4 and
(b) Comparison between predicted and measured differential vertical displacement at locations P1, P2, P3 and P4 at the tunnel face.

Surface inclinometer measurements

Figure 16 shows an example of the comparison between the predicted and measured horizontal displacements of one of the inclinometers at three different tunnel heading face locations. The results show that the predicted horizontal displacements are generally less than those measured and the trends of displacement profiles between predicted and measured are comparable as the face of the heading passes by the instrument location.

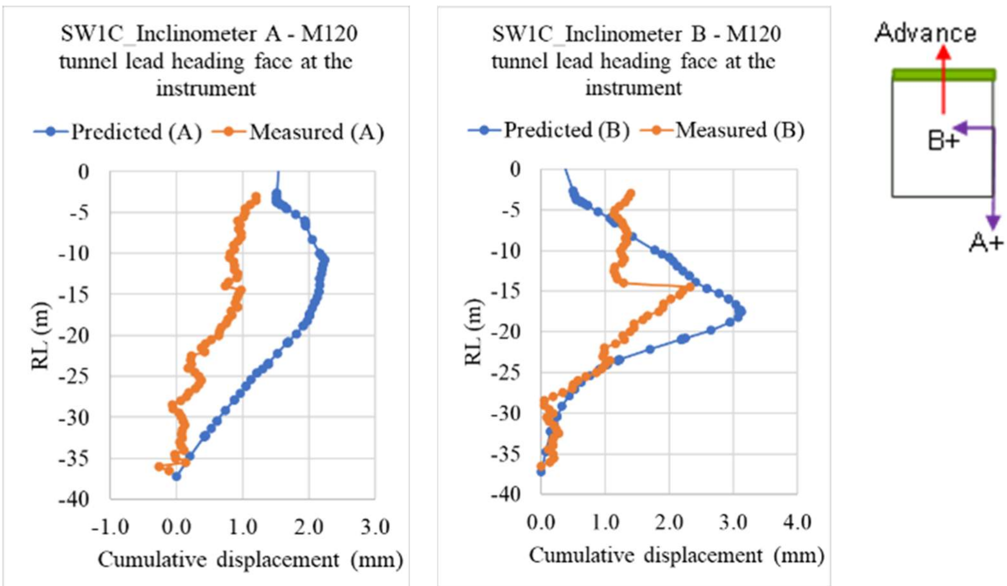


Figure 16: Comparison with predicted and measured horizontal displacements from SW1C inclinometer at the tunnel face.

REVIEW OF MONITORING DATA DURING CONSTRUCTION

Throughout the construction period, monitoring data was continuously reviewed and compared against the predictions whilst the tunnel headings were within the interface zones.

City Tunnel

The locations of the various types of monitoring at the City Tunnel are shown in Figue 17, and the movements that occurred at the highlighted extensometer are in Figure 18. There are distinct stages at which the displacement rate changed as the tunnel excavation progressed, as described below.

Before the lead heading reached the extensometer, i.e. from A to B, a rate of displacement of 0.03 millimetres /day occurred.

Once the trail heading passed the instrument, i.e. from B to C, there is a marked increase to the rate of displacement to approximately 0.9mm/day which gradually reduces once trial heading has reached a distance of approximately three times the tunnel span ahead the instrument, i.e. from C to D.

From D to E, there was again an increase in the displacement rate to approximately 0.08 millimetres /day when the temporary rock pillar was approximately at a distance. 30 metres from the permanent pillar nose (equivalent to the full span of the cavern). This increase gradually reduced as removal of the temporary pillar continued, ie from E to F.

The induced displacements trends measured compared well with the predicted displacements and design assumptions. The comparison also demonstrated that the 3D finite element modelling approach adopted for the design adequately represented the tunnel behaviour.

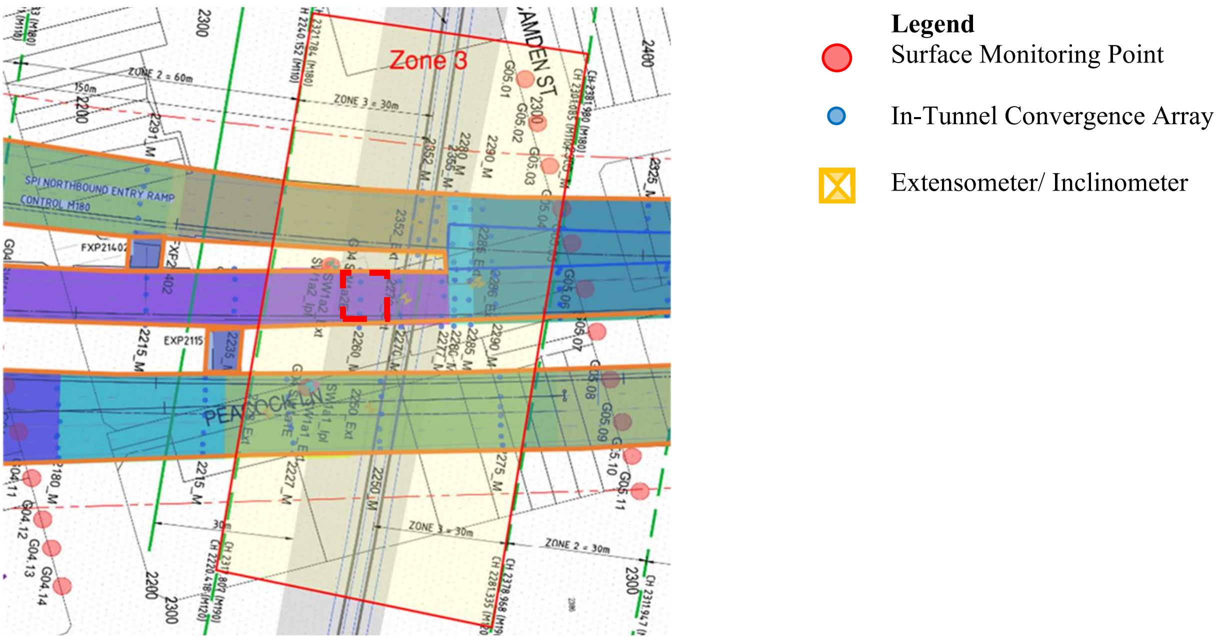


Figure 17: Monitoring locations at the City Tunnel

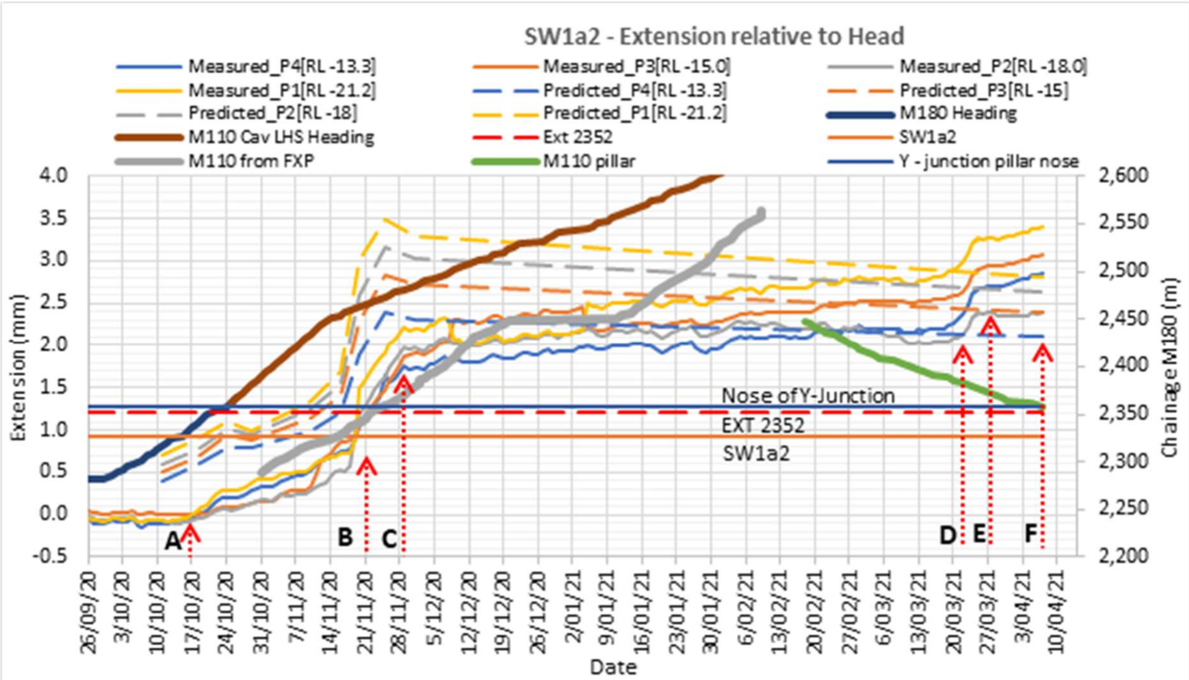


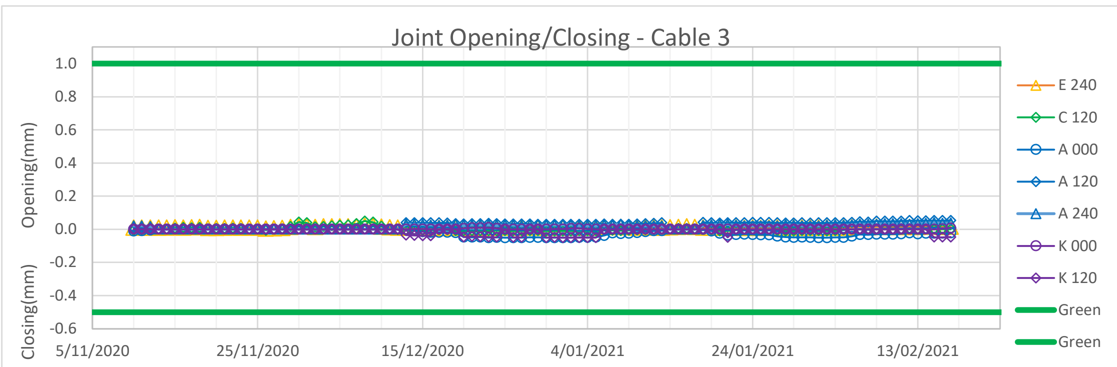
Figure 18: Comparison between measured and predicted relative vertical displacements along SW1a2 extensometer measuring **points with respect to time and excavation stages**

Pressure Tunnel

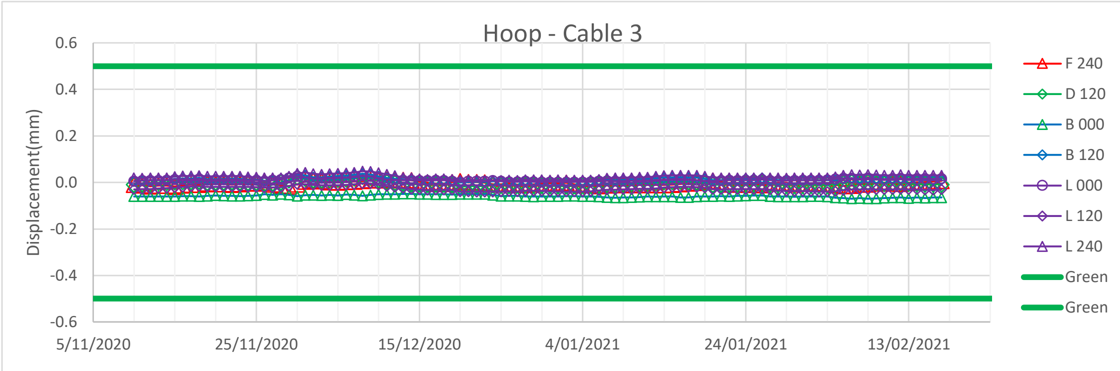
In a similar manner to the City Tunnel, the data from the instruments was monitored and reviewed throughout construction and the trends compared well with predictions. In addition, as already noted above, strain gauges had been installed to monitor joint opening and rotation together with hoop/ circumferential movements. Data was provided in real-time (every 6 minutes) and smoothed to remove peaks by taking a daily average. An example of the typical output is shown in

Movements were typically in the order of 0.1 to 0.2 millimetres and varied by 1/100$^{\text{ths}}$ of millimetres, which was substantially less than predictions.

Comparing the predicted and measured data demonstrated that the effects of the tunnelling works on the Water Pressure Tunnel were within the predicted ranges and less than the limiting criteria.



(a)



(b)

Figure 19: Pressure Tunnel Strain Gauge data (a) and (b)

CONCLUSION

A key aspect in developing new infrastructure is recognising the fundamental interactions with existing assets early in the project development process. As a result, an tunnel alignment was developed by the design and construction team which recognised and maintained adequate clearances to the City and Pressure Tunnels. Furthermore, a comprehensive impact assessment was undertaken which in conjunction with the monitoring regime developed, enabled the asset owner Sydney Water to be part of a continuous review of the monitoring data during construction so that concerns and issues were addressed expeditously. This process was framed by a set of protocols which clearly set out the roles and responsibilites and lines of communication to be followed during construction.

As a result, construction of the M4-M5 Link Tunnels was able to progress smoothly and efficently across the interface zones with the the City and Pressure Tunnels.

Acknowledgements

The authors of this paper would like to thanks the Asset Owner Sydney Water, the Client Transurban, and the construction joint venture Acciona Samsung Bouygues Joint Ventrure for their permission to publish this paper as well as the hard work and effort of the team involved

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Modelling considerations for the impact of loading on brittle brick lined oviforms

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ABSTRACT

Buried infrastructure can be adversely impacted by stresses and strains induced by excavations and foundation loadings from new works. Ground-structure interaction can be readily modelled for continuous or semi-continuous conduits such as steel and concrete pipes. Modelling more complex and brittle structures such as stone and masonry arches and masonry oviform is more challenging. This paper discusses the behaviour of such structures, such as their capacity to resist large strains without collapse and serviceability at lower strains, using the specific example of a brick oviform.

A literature review of papers describing the behaviour of masonry structures and buried conduits, and asset owner guidelines for impact assessments is presented. Common limits applied to strain and cracking of buried structures based on the literature and numerical analysis are summarised.

Simplified modelling techniques are discussed that involve assigning equivalent stiffnesses to continuum models to represent structures made up of discrete elements. The results of detailed modelling of bricks and mortar joints as discrete structural elements is compared with simplified continuum models to assess the equivalent stiffness of continuum structural elements.

The results show that adopting simplified continuum models with equivalent stiffnesses is practicable. Benefits of adopting simplified models include reduced modelling time, and mitigation of the risk of errors in discrete element modelling due to the complexity of the geometries and parameter selection.

INTRODUCTION

Construction works in urban areas has the potential for substantial impacts on third party buried utility infrastructure and the risks are often greater in mature cities with old utilities of uncertain construction, integrity and structural capacity. Ground-structure interaction can be readily modelled for continuous or semi-continuous conduits such as steel and concrete pipes. Modelling of more sensitive and brittle structures such as stone and masonry arches and masonry oviform that are often more than 100 years old is more challenging.

Utility owners are concerned that works can cause gradual loss of serviceability that could lead to increased maintenance requirements, compromise the water tightness or even causing collapse of brittle masonry or stonework. This is of particular concern when construction activities such as traffic by heavy construction plant, crane outrigger loads, stockpiled materials or eccentric excavations subject the asset to more adverse short-term conditions than the long-term loads. Due to inherent heterogeneity and anisotropy of materials, it can be difficult to demonstrate the stability of brittle conduits under short term loads and serviceability of these assets under long term loads.

The paper focuses on modelling of a masonry oviform such as shown in Figure 1, demonstrating their capacity to withstand substantial loads without collapse and their serviceability behaviour at lower strains.

Figure 1. Typical brick oviform section (Source: Powerhouse museum)

Simplified modelling techniques are discussed covering both micro-modelling of transverse sections, where individual brick elements and mortar joints are modelled as discrete elements and macro-modelling, where equivalent stiffnesses are assigned to continuum models to represent structures made up of discrete elements. The results of detailed modelling of bricks and mortar joints as discrete structural elements is compared with simplified continuum models to assess the equivalent stiffness of the continuum structural element.

The objective of this comparison or discrete element and continuum modelling is to simplify the modelling required to assess brittle masonry or stonework performance, if practicable. Discussion is also provided on the longitudinal performance of conduits and simple methodologies to assess longitudinal strains that are often critical where excavation parallel to an asset induces significant displacements.

BACKGROUND AND LITERATURE REVIEW

Masonry structures are sensitive to significant tensile strains. A literature review has been conducted to understand the maximum tensile strain that can be tolerated by masonry structures. There are few published papers on the behaviour of buried masonry structures such as brick oviform. However, there are numerous publications that investigated the critical threshold tensile strains on masonry buildings for the onset of cracking based on laboratory experiments and field observations.

Burhouse (1969) observed that the onset of visible cracking occurs between 380με (microstrain) and 600με, while Polshin and Tokar (1957) gives this limit as 500με. Boone (2001), assumes a value of tolerable strains between 100με to 300με after reviewing data from over 100 case histories of damage to masonry bearing walls and masonry in-fill walls.

Burland et al (1978) and Burland et al (1974) classified the visible damage of above ground masonry walls with respect to ease of repair into different damage categories. This was based on previous work conducted by the U.K National Coal Board, which published a classification based on experience of subsidence damage categorised based on crack width. Hairline cracks with widths less than about 0.1 mm were considered to have negligible impact on structural performance. This work forms the foundation of building damage assessment methods currently used for preliminary assessments (Mair et al , 2016).

Mair et al (2016), classifies limiting tensile strain of less than 500με as negligible, based on the case histories analysed by Boscardin and Cording (1989).

Chen et al (2016) reports experimentally analysed cracking behaviour of masonry arches and suggests cracks initiate in masonry at 56% of failure load. The report also highlights that stress strain behaviour of the masonry arch was linear until initial cracking. Heydarpour (2019) adopted allowable tensile stresses in masonry as 550 kPa. This value was adopted by applying a strength reduction factor of 0.6 to the specified modulus of rupture for masonry structures (TMS 402-13/ACI 530-13/ASCE 5-13) developed by Masonry Standards Joint Committee, USA.

Based on both field observations and laboratory experiments, as reported by multiple authors, the acceptable level of tensile strain before cracking occurs in masonry structures ranges from 100 με to 550με. Strains below 500με and crack widths less than 0.1 mm are considered insignificant (Boscardin and Cording 1989, Mair et al 1996). It is important to keep in mind that this conclusion is based on observations and laboratory tests for above-ground masonry buildings. Hence, using a maximum allowable strain level of 500με should be used cautiously for buried masonry sewers New (2017), as it may exceed the serviceability limit for brick sewers, even if it is satisfactory for above ground masonry walls.

The Thames Water guide on piling, heavy loads, excavations, tunnelling and dewatering, Thames Water (2009) sets a limit on the increase in tensile strain and compression stress limits of:

- Tensile strain – 500με
- Compression – 25% or allowable stress.

The Sydney Water Specialist Engineering Assessment procedure, Sydney Water (2021) sets tensile strain and crack threshold impact criteria for high-risk masonry assets of:

- Tensile strain – 250με due to longitudinal and transverse effects.
- Maximum crack width – 0.2 mm.
- Maximum depth of crack – 1/5 of section thickness.

TRANSVERSE MODELLING METHODOLOGY

Individual bricks typically exhibit isotropic properties. However, masonry structures in general display anisotropic behaviour due to the mortar joints between bricks. Although brick units are initially bonded together by mortar, this may not be the case for old structures since mortar joints are often relatively weak and/or brittle. For the masonry oviform considered in this paper, stability is maintained by the general state of compression across the mortared joints. Cracks are most likely to develop across mortared joints or existing cracks will open if tensile stresses develop in the masonry. A realistic modelling methodology should be able to account for cracking at lower tensile stresses and be able to re-distribute stresses within the structure to keep it under compression. Lourenco (1996) identified the following modelling strategies, depending on the level of accuracy and computing power available.

1. Detailed micro-modelling: Individual brick units and mortar joints are represented by continuum elements and brick-mortar interfaces are represented by interface elements. The nonlinear behaviour of brick and mortar can be analysed, but this approach is computationally demanding.
2. Simplified micro-modelling: Brick units are represented by continuum elements with brick-mortar interface represented by interface elements. This does not explicitly model the properties of the mortar. However, the approach can represent the discontinuous anisotropic nature of a brick oviform with a reasonable level of computational effort.
3. Macro-modelling: Bricks, mortar and unit mortar interfaces are represented by continuum elements usually by advanced numerical models which can represent the anisotropic, nonlinear behaviour of masonry structure with reasonable accuracy. This approach does not make a distinction between brick units, mortar and joints, but treats the masonry as an anisotropic composite.

The modelling strategies adopted for comparison in this study are: (2) the simplified micro-modelling and (3) Macro-modelling, both using Finite Element Method analyses.

Finite element calculations were carried out in plane strain. The behaviour of mortar and brick-mortar interface was represented by interface elements which allow the oviform geometry to translate, rotate or fail along the interface. Brick units may slide relative to adjacent bricks at joints if joint capacity is exceeded. The modelled geometry for a brick oviform is presented in Figure 2.

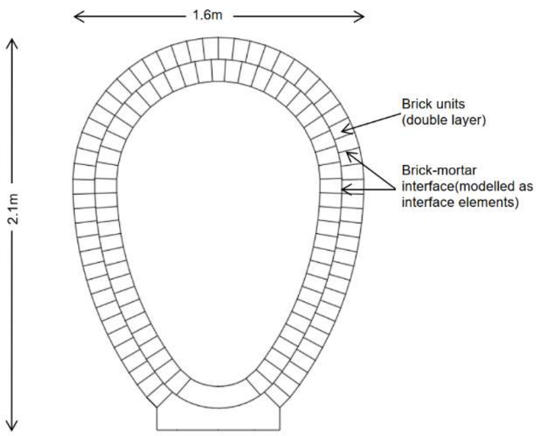

Figure 2. Brick oviform geometry adopted for finite element modelling

TRANSVERSE LOAD SCENARIOS AND MODELLING PARAMETERS

Load Scenarios

Load scenarios adopted for the models are described in Figure 3. Static loading conditions are assumed, and no dynamic analysis was performed. An 8 m wide surface load was applied to the ground surface above the oviform. The position of the load was varied from directly over the oviform centreline to an offset of up to 1 m.

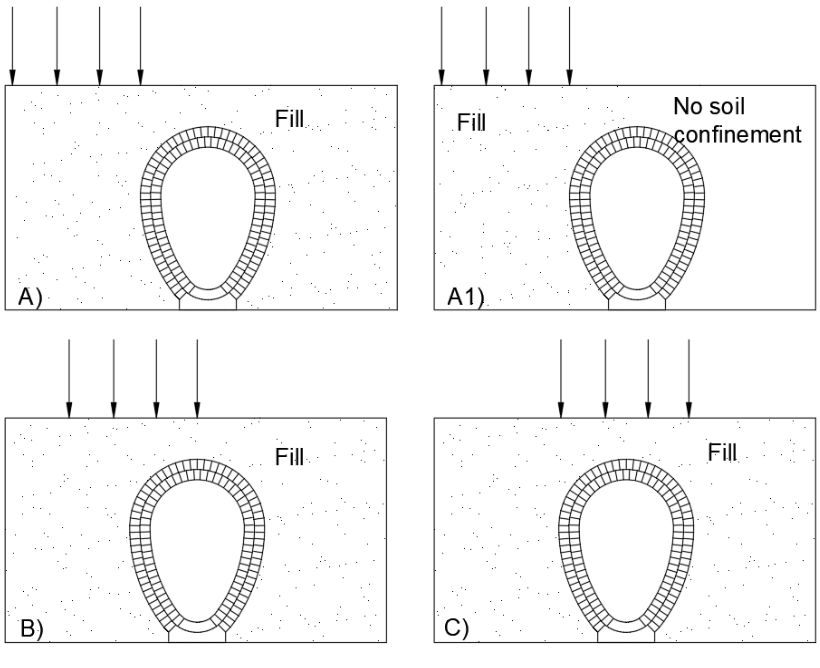

Figure 3. Loading conditions adopted for numerical models

Modelling Parameters

Soil Properties

The oviform base is assumed to be founded on an incompressible foundation with an isotropic soil mass to the sides and 2 m thick cover over the top of the oviform. Typical trench backfill parameters were selected, assuming the fill around the oviform is well compacted and has similar stiffness and strength to the surrounding soils. A drained, linear elastic model was assumed for the soil with the parameters in Table 1.

Table 1. Soil parameters assumed for the finite element analysis

Parameter	Symbol	Unit	Assumed Value
Young's modulus	E'	MPa	20-70 [50]
Cohesion	c'_{ref}	kPa	1
Friction angle	φ'	°	30

Masonry Properties

Table 2 shows the range of strength stiffness values adopted for the analysis with typical values adopted provided in square brackets.

Table 2. Brick and mortar parameters adopted for the finite element analysis

Parameter	Symbol	Unit	Mortar	Brick
Young's modulus	E'	MPa	-	10,000
Cohesion	J_{coh} / c'_{ref}	kPa	10-100 [50]	1
Friction angle	J_{fric}	°	25-35 [30]	-
Joint tensile Strength	J_{ten}	kPa	0	-
Joint normal stiffness	K_N	GPa/m	1-50 [20]	-
Joint shear stiffness	K_s	GPa/m	1-20 [8.3]	-

Historical masonry structures typically exhibit low bond strength characteristics. The bond between the masonry units dominates the behaviour of the structure such as the formation or opening of cracks, causing redistribution of stresses and development of collapse mechanisms (Sarhosis 2012). The behaviour of the model is predominantly influenced by the joint mechanical properties. The potential joint behaviour under loading is influenced by three main parameters:

1. Joint normal stiffness K_N, that defines normal displacements
2. Joint shear stiffness K_S, that defines elastic shear displacements and
3. Tensile and Shear strength properties of the joints that defines joint failure (J_{ten}, J_{coh}, J_{fric}).

Tensile capacity of the brick-mortar interface is taken as zero. Normal and shear stiffness parameters are estimated based on the mortar stiffness.

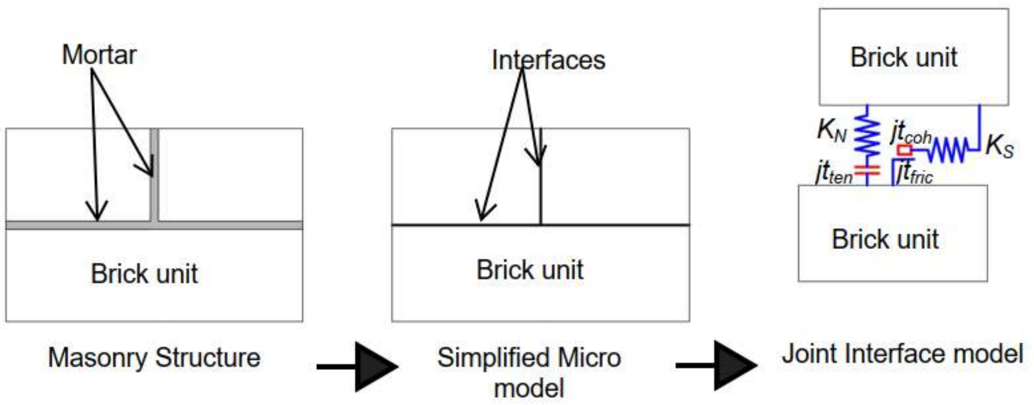

Figure 4. Simplified micro-modelling of masonry structures (K_N: Joint normal stiffness, K_S: Joint shear stiffness, J_{ten}: Joint tensile strength, J_{coh}: Joint cohesion, J_{fric} : Joint friction angle) (Idris et al 2009, Lourenco 1996, Al-Heib 2012)

Based on previous works of Rots (1991), Rots (1997), Sarangapani et al (2005), Sarhosis (2012) reports stiffness of mortar in the range of 1 GPa to 10 GPa. This results in joint normal stiffness of the mortar in the range of 10 GPa to 100 GPa for a 10mm mortar thickness. It is important to note that the range of stiffness values reported in literature is for new mortar, hence a reduction factor should be considered for older structures.

SIMPLIFIED TRANSVERSE MICRO-MODELLING RESULTS

The modelling results indicate that soil failure occurs before oviform failure and this only happens under relatively large loads. The oviform masonry can withstand substantial loads without collapsing when confined. This is because the brick units can resist high compressive loads and the oviform shape is effective at transferring forces without creating tension in the brick-mortar interfaces. Figure 5 shows the deformed shape of oviform under applied surface loads for Model A. The models indicate cracking at the brick-mortar interface under flexural loads, however, the crack width is less than the width of a single brick unit, with the section composed of two brick widths. Both Model B and Model C had similar responses and were able to withstand substantial loads without failure.

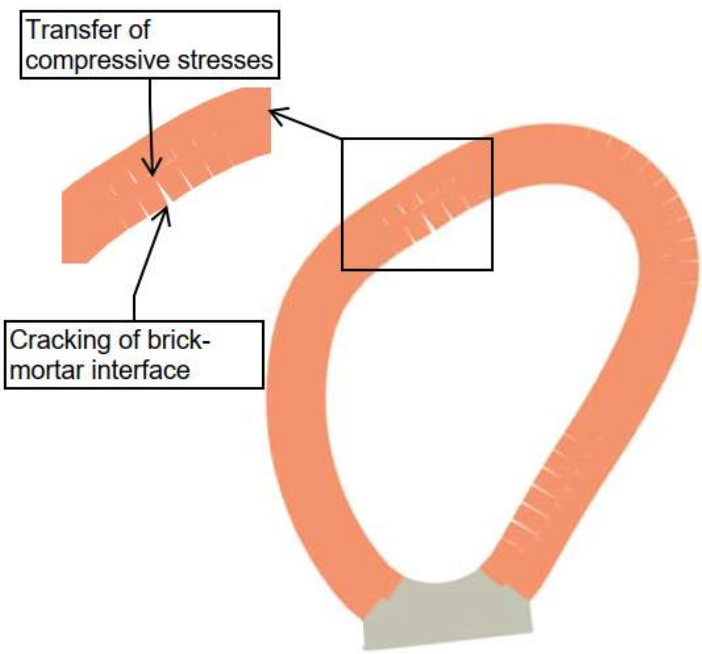

Figure 5. Deformation behaviour of oviform with soil confinement around the structure

The Models are only stable when confined and removal of confining stresses by excavating the soil surrounding the structures results in failure. Figure 6 shows oviform failure as soon as the confinement on one side of the Oviform is removed. This highlights the importance of maintaining confinement around an Oviform.

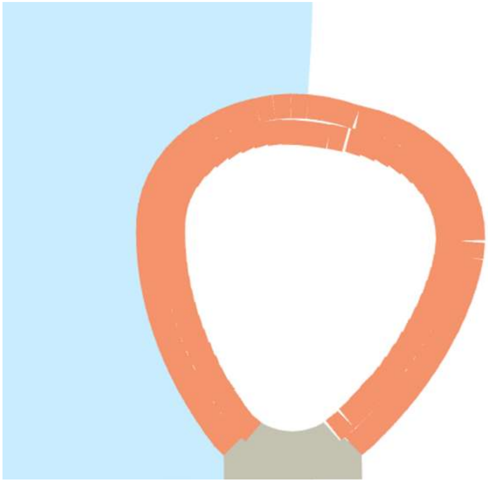

Figure 6. Failure of oviform after removing the confining stresses on the right-hand side.

Figure 7 shows the normal stresses at the brick-mortar interface for Load case A in Figure 3, with some areas displaying stress concentrations and others showing minimal stress. Rotation and translation of brick units leads to re-distribution of stresses within the structure. Eccentric loading causes flexural stresses and tensile cracks forming because the brick-mortar interface has no tensile strength. The stress redistribution causes the axial thrust line to change. Where the line of thrust remains within the brick structure the oviform remains intact.

The redistribution of stresses leads to a compressive stress concentration at the brick-mortar interface at some locations. If those normal compressive stresses are larger than the compressive strength of the brick units, they will crush and this will result in a compressive failure mechanism.

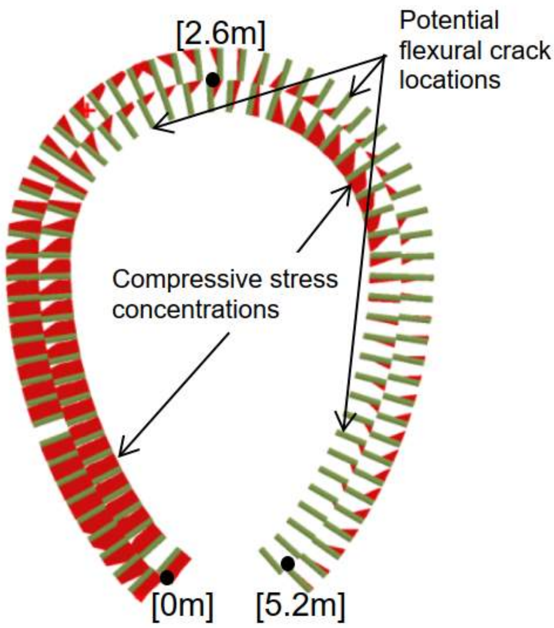

Figure 7. Normal stresses generated in brick unit-mortar interface. Distance along the centreline of the oviform is shown in square brackets.

Figure 8 shows the vertical and horizontal distortions of the oviform under the modelled surface load scenarios. The oviform distortions are calculated as the ratio of the differential displacement to the maximum span in the horizontal and vertical directions. The vertical differential is calculated as the differential displacement between the invert and obvert of the oviform and horizontal differential is calculated as the differential displacement between the right-hand side and left-hand side of the oviform through the location having the longest span.

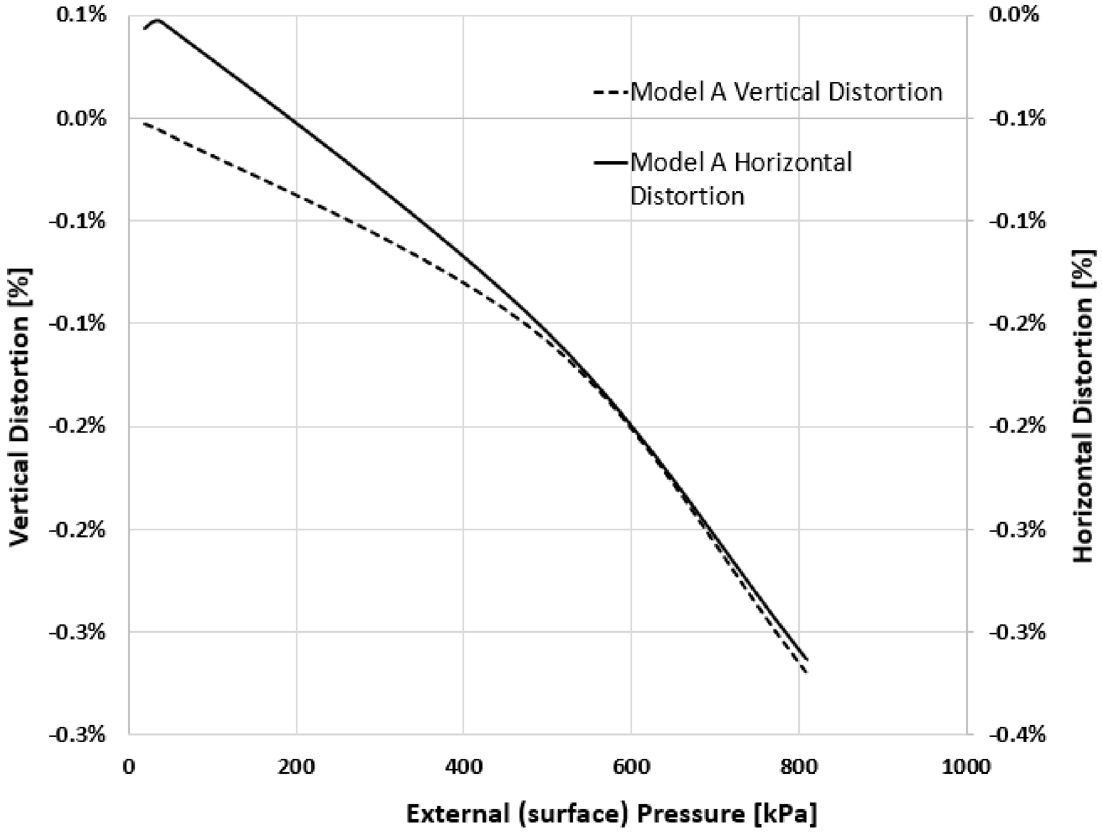

Figure 8. Vertical and horizontal distortions at (soil) failure

TRANSVERSE MACRO MODELLING WITH EQUIVALENT STIFFNESS

To avoid the complexity of discrete modelling of masonry, using a continuum model is preferable, if there can be confidence that parameter selection in a continuum model produces equivalence of deformations. In this study continuum modelling was carried out to match the deformation profile generated by the discrete model and then estimating the tensile strain generated in an equivalent, linear-elastic continuum structural model. The elastic continuum model had the same geometry and modelling sequence as the discrete model. All the brick mortar interfaces were replaced with an elastic continuum material. The stiffness of this elastic continuum material was then varied to match discrete model deformations.

An equivalent linear elastic stiffness was estimated by matching the total centreline deformation along the oviform across section under the load scenarios as shown in Figure 9. Total deformations at the crown of the oviform was matched with the equivalent elastic model under serviceability conditions. The centreline distance plotted in Figure 9 was measured from the base of the oviform clockwise as shown in Figure 7. For typical material parameters, the stiffness of the equivalent linear elastic material is about 2 GPa. This value is closer to the stiffness of the mortar than that of the individual bricks.

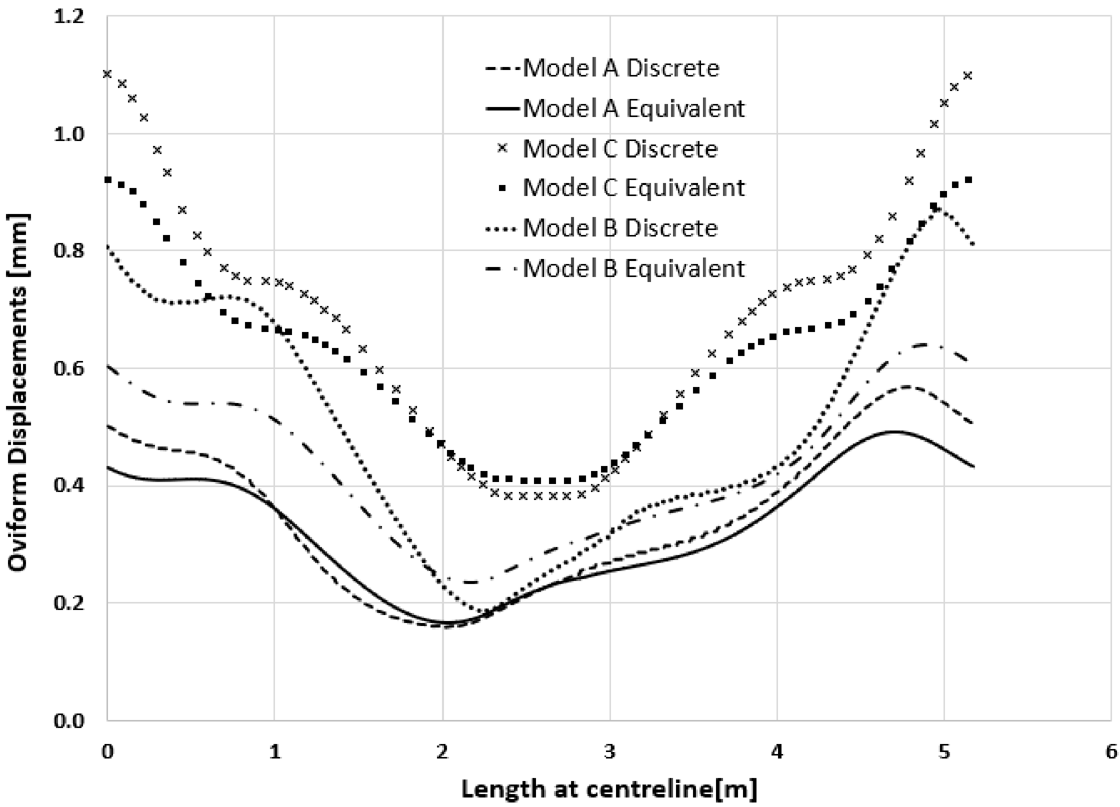

Figure 9. Oviform deformation along the inner edge for discrete model compared with continuum model with elastic stiffness of 2GPa

Figure 10 shows a comparison of cracking at the brick-mortar interface due to flexural stresses in the discrete model to the tensile strains generated in an equivalent continuum model. Cracks are only generated in locations that are under tension in the discrete model. For identical loads, an average crack width opening of about 0.14 mm can be observed near the crown of the oviform, and the equivalent continuum model shows a tensile strain of about 500με.

The results indicate that the equivalent linear elastic model predicts maximum tensile strains at the same locations where the discrete model shows maximum crack width opening. Note that in Figure 10 the distortion of the discrete model is exaggerated to show the crack locations.

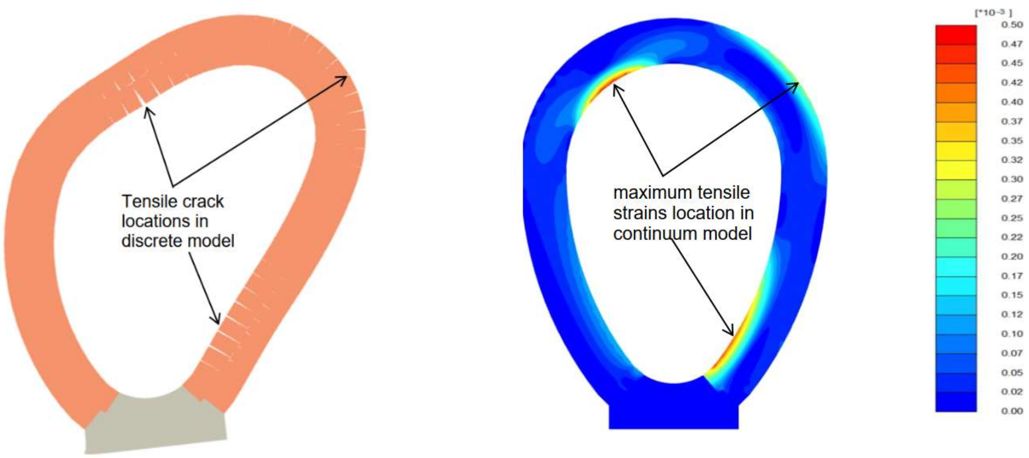

Figure 10. Comparison of tensile crack location of discrete model with tensile strains location of continuum model.

For these analyses, a surface load of 200 kPa was applied eccentrically 2m above the oviform. This loading magnitude is significantly higher than the typical construction loads that are commonly applied. However, the load transfer also depends on the strength of the surrounding soil. Smaller surface loads could induce an identical response if the surrounding soil is considerably weaker.

Figure 11 shows a comparison of the evolution of crack width opening near the oviform crown under increasing surface load to the tensile strains developed in an equivalent continuum model. The plot gives an indication of expected crack width in masonry structures based on tensile strains generated on a simplified equivalent model. The crack width opening under loading is non-linear, with crack widths increasing under higher loads. However, for a continuum model, evolution of strains will be linear due to the limitations of the elastic model.

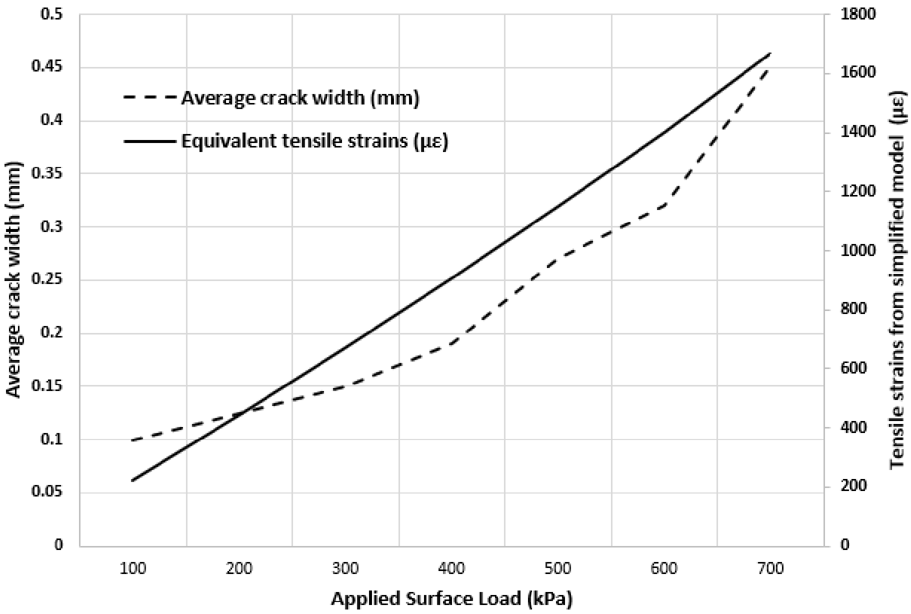

Figure 11. Comparison of tensile strains generated in a simplified model with cracks widths generated in a discrete model for identical loading conditions

Cracking of mortar joints could lead to increased permeability of the structure. Based on a simplified continuum model and using Figure 11, a preliminary estimate of level and extend of cracking can be evaluated. These crack predictions can be compared with compliance requirements or to formulate rectification measures based on the predicted extent of damage.

LONGITUDINAL MODELLING

Masonry conduits such as the oviform modelled in this paper are continuous linear structures that are also vulnerable to the differential longitudinal ground movements. The following section gives a brief overview of a longitudinal modelling approach for masonry oviform.

In transverse modelling it is assumed (conservatively) that the bricks have been laid such that the joints of multiple layers form a continuous plane of weakness through the oviform cross section. In the longitudinal plane the joints are more likely to be staggered and this will produce a stiffer structure than one where a non-staggered brick pattern is laid longitudinally. However, for a linear structure, the loading is predominantly flexural, as opposed to compressive in the transverse direction. Oviform bending stiffness in the longitudinal direction may be low due to the lack of tensile strength at the brick-mortar interface. Therefore, a conservative approach is to omit the longitudinal stiffness contribution in calculations.

The sum of longitudinal flexural strains and axial tensile strains, if any, is the total longitudinal strain along the asset. Because of the weak and aging mortar that results in low tensile strength of masonry conduits such as oviform, the total longitudinal tensile strains is a critical factor when considering impacts for adjacent works. Flexural strains can be determined from the conduits curvature, which is calculated from the double differential of displacement. This means that the structure is more sensitive to sudden changes in displacement than to the overall magnitude of displacement. Longitudinal modelling methodology to be adopted can be summarised in following steps.

1. Prediction of ground movements: A longitudinal ground movement profile should be generated. If there are multiple construction activities that could potentially induce ground movements, predicted ground movements should be the superposition of all ground movements affecting the structure. Depending on the complexity of the loading and alignment of the asset, ground movement profile can be generated using analytical solutions or from 2D/3D modelling software.
2. Estimation of curvature: Curvature can be directly calculated by double differentiating the displacements. However, for simple geometry and load conditions the displacements can often be assumed to fit a Gaussian curve which offers the advantage of being able to apply simple analytical formula. If the alignment and loading conditions are complex, it may not be possible to fit displacements to a Gaussian curve and calculating double derivative of displacements is preferred in that case.
3. Calculation of flexural strains from ground curvature: For a longitudinal section undergoing flexural deformation, strains can be calculated by multiplying the curvature of the structure and depth of neutral axis. Masonry is supposed to have no significant tensile strength, hence the neutral axis should be taken at the extrados of the structure. Hence for a masonry oviform, flexural strain can be calculated by multiplying the curvature of the structure with height or width of the oviform depending on the orientation of deformation profile.
4. Estimation of axial (tensile) strains: Settlements could induce tensile horizontal ground strains. The total tensile strain is the combination of this axial tensile strain and longitudinal bending strain. However, transfer of tensile strain especially at shallow depth is questionable as noted by New (2017), hence can be ignored in preliminary assessments. Depending on the criticality of the asset, tensile strains can be included in a detailed final assessment.

CONCLUSIONS

Conduits such as masonry oviform display anisotropic behaviour due to the mortar joints between bricks. The discrete modelling of a masonry oviform section shows that under confinement the oviform can withstand substantial loads without collapse. Although localized cracking is expected to occur at small deformations, the oviform shape allows it to remain in a state of compression. This is due to the rotation of the bricks creating a hinge mechanism that transfers compressive forces through a narrow surface area. The presence of cracks further increases the ductility of the oviform.

The analysis suggests that masonry oviform can be modelled as linear elastic continuous structures with equivalent linear elastic stiffness comparable to that of the mortar between bricks. Such models result in deformation behaviour consistent with models that consider individual bricks and the mortar joint interfaces. This simplifies the modelling process by eliminating the need to create discrete elements for brick units and to carefully select material and interface properties and hence reduces the potential for errors in the analysis.

The results show that the equivalent linear elastic model predicts the locations of the maximum tensile strains, which coincide with the maximum crack width openings in the discrete model. A comparison between the crack width opening in discrete model and simplified equivalent continuum model provides an estimation of the crack width that can be anticipated in masonry structures based on the tensile strains calculated in the equivalent model.

Oviform tunnels are continuous linear structures and are also sensitive to longitudinal movements. A brief overview of a longitudinal modelling approach was also presented.

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Basement excavation analysis – potential impact on the North Georges River Sewer tunnel

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ABSTRACT

The Sydney Water North Georges River Sewer (NGRS) tunnel was constructed in 1942 in slightly weathered/fresh Hawkesbury Sandstone. The unreinforced concrete lining has a near flat arched roof profile with vertical walls and flat but slightly sloping invert. The concrete arch is 380mm in thickness and the walls and invert around 230mm. The tunnel crown is 15m below the surface and 3m below the basement excavation. The two adjacent building basements are to be excavated to a depth of 12m. The horizontal separation between the tunnel and basement wall is 8m. Even though the tunnel and concrete lining are now 80 years old, it is in remarkably good condition. The sewer services 1 million people so it is a vital piece of infrastructure. A site investigation with borehole logging, sampling and testing was used to develop a geological profile which was then incorporated into numerous Finite Element models. The tunnel concrete lining was the focus of the analysis given the potential for cracking should there be excessive ground movements and/or external groundwater pressure changes. A hypothetical scenario with a high-water table was also analysed. As predicted and then measured on site, there are two water tables. One perched well above the tunnel overlying the sandstone rock and the other a few metres above the tunnel crown. The analyses demonstrated that the differential movement across the tunnel section is likely to be small, in the order of 0.3mm or less. However, if any cracking did occur it would likely be in the corners of the tunnel profile, due to the development of stress concentrators.

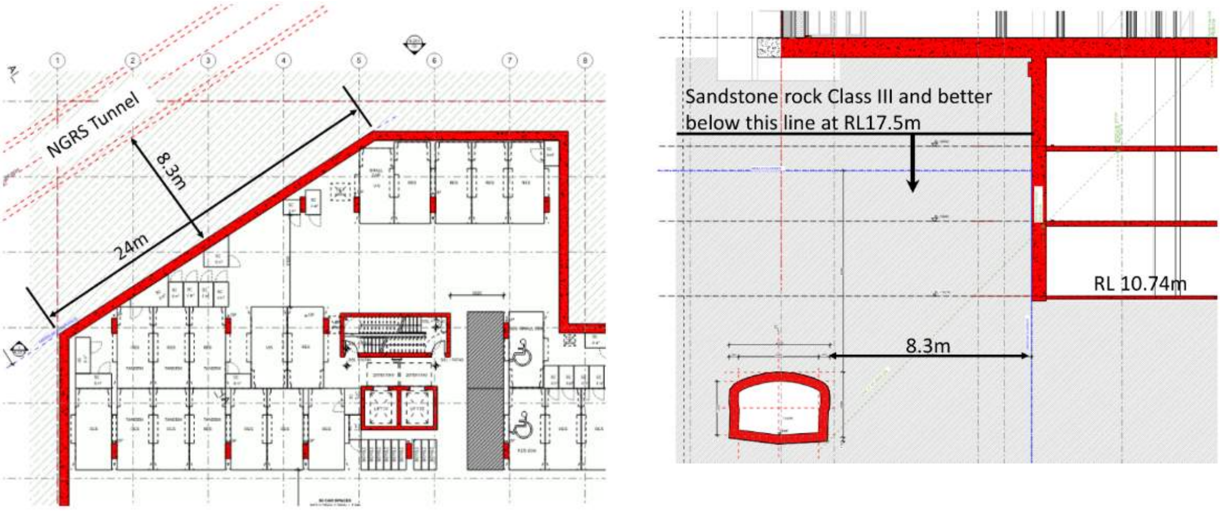
INTRODUCTION

This paper describes the process that was followed to assess the potential impact of a deep basement excavation and building tower near an unreinforced concrete lined sewer tunnel. The analyses and associated report were subsequently approved by Sydney Water to allow the building developments to proceed. The sewer tunnel services 1 million people, so it is a vital piece of infrastructure. Apart from 2D FE analysis to determine the impact of rock removal on displacements of the ground and induced stresses in the lining, changes in potential height of the water table were a concern for Sydney Water. The rectangular profile of the tunnel also meant that stress concentrators could develop in the tunnel corners even for small rock mass displacements. Perhaps unusually for a tunnel of this size today, the tunnel invert was flat, giving rise to the possible impact of a changed water table height both on flexural stresses and potential uplifting of the concrete invert. The location of the longitudinal construction joints also had a significant impact on the assessment of the latter.

DEVELOPMENT DESCRIPTION

The Stage 1B (6-16 Victoria Street at Kogarah) residential development is a 12-level building with a 3-level basement excavated predominantly in Hawkesbury Sandstone. A sewer tunnel, the NGRS, traverses the site under the north-west corner as shown on the figures below. Geotechnical investigation and special engineering assessment reports were prepared for both Stage 1A and 1B. However, the potential for impact from Stage 1B on the tunnel was greater than Stage 1A, hence this paper only refers to Stage 1B investigation and analysis.

Figure 1: Tunnel alignment and piezometer and inclinometer locations



a) Plan **b) Section**

Figure 2 – Extracts from architects' drawings adjacent to the tunnel.

Because the basement would be fully drained, it was concluded that the water table will not rise above the bottom of the basement and hence the only likely change to the loading on the tunnel lining would be due to changes in ground loading being principally the result of removal of the rock forming the basement. Notwithstanding the fully drained basement impact on the water table, Sydney Water were still concerned about the water table impacts, so further investigation of the groundwater regime was addressed, including the installation of two piezometers as discussed later in this paper.

TUNNEL PROFILE

The Sydney Water NGRS tunnel was constructed in 1942 in slightly weathered/fresh Hawkesbury Sandstone. The unreinforced concrete lining has a near flat arched roof profile with vertical walls and sloping invert (i.e. a tunnel with an excavated rectangular profile 2.8m high and 4m wide). The concrete arch is 380mm in thickness and the walls and invert around 230mm. The tunnel crown is 15m below the surface and 3m below the basement excavation base. There is a pillar of rock 8m wide separating the closest tunnel wall to the basement boundary wall (at Victoria Street). The building's basement depth is 12m. A condition survey of the tunnel was made by consultants including a video of a walk through. Even though the tunnel and concrete lining are now 80 years old, it is in remarkably good condition.

The photograph in Figure 3 was extracted from this video. A dimensioned sketch of the tunnel section is shown in Figure 4 below. The dimensions of the structural elements of the tunnel are likely to be greater thickness than shown. Drill holes from the July 2021 dilapidation inspection showed a wall thickness of 380mm even without drilling through the wall to the rock.

Figure 3 – Photographs - 2021 inspection

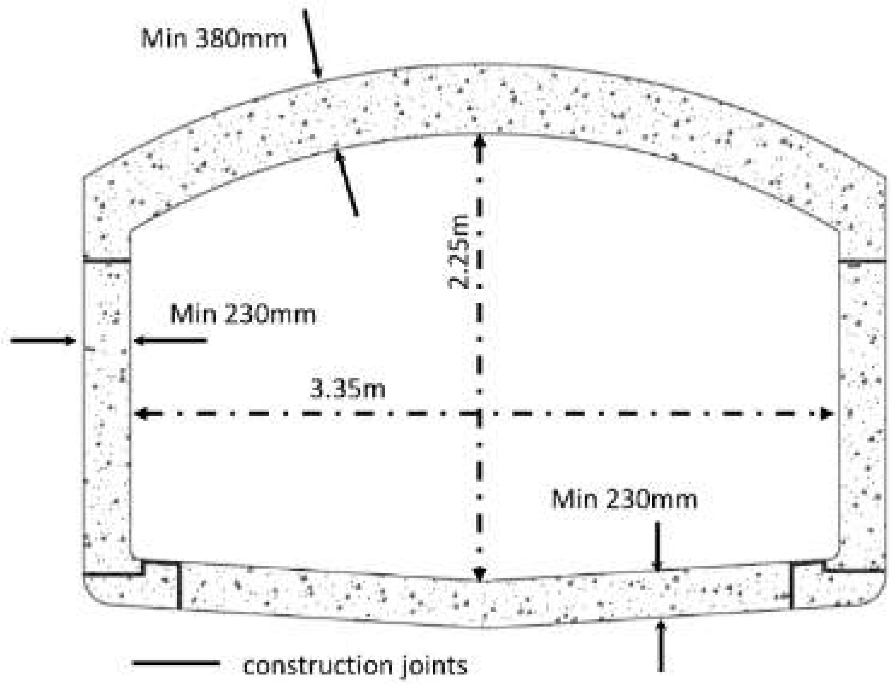


Figure 4 – Tunnel section

GEOLOGICAL PROFILE

Within the boundary of the Stage 1B development there have been 4 boreholes drilled. BH04, BH05, BH06 and BH101. Also, DCP04 (one of several Dynamic Cone Penetrometer tests, the other DCPs are at the borehole locations) is located near the tunnel alignment and confirms (with the boreholes) that the sandstone rock strata surface is consistently only a few metres below the surface. Where the sandstone is initially weathered, it rapidly increases in strength with depth. Additional boreholes drilled in April 2022. BH04, BH05 and BH06 were drilled to a depth of around 7m.

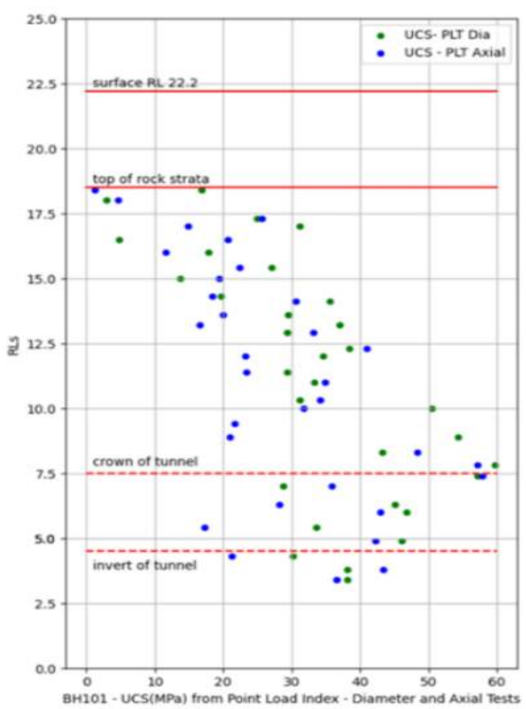


Figure 5: UCS values with depth

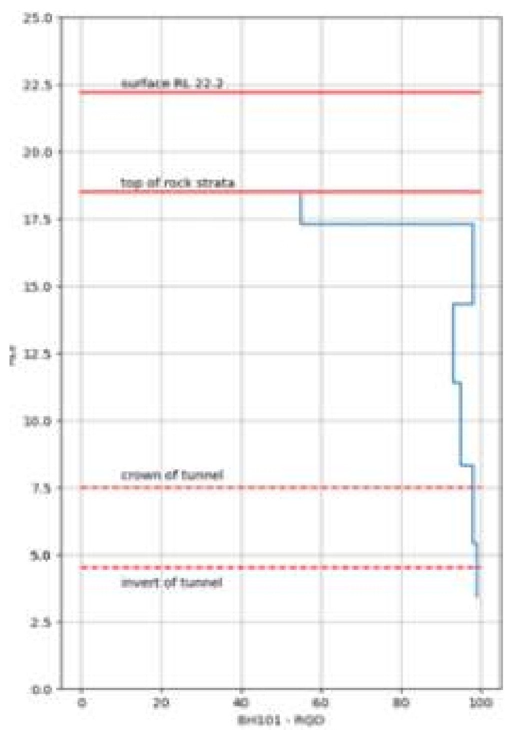


Figure 6: RQD values with depth

However, BH101 was drilled to a depth of 19m (RL3.35m) with the base of the borehole slightly deeper than the invert of the NGRS tunnel (at RL4.5m). All the boreholes found moderately weathered sandstone (Class III using the Sydney Rock Mass Classification System) rock between 3.2 to 4.6m below the surface. BH05, BH06 and BH101 found slightly weathered to fresh sandstone (Class II) between 5.2 and 7.5m below the surface. BH101 being deeper, from 7.5m depth, cored only fresh sandstone (Class I). Figures 5 and 6 above are of UCS and RQD values with depth respectively.

GROUNDWATER PROFILE

A high-water table given the invert of the tunnel was flat and unreinforced was a concern. A piezometer was not installed during the initial site drilling. In April 2022 two piezometers were installed on the site adjacent to the tunnel alignment, the first piezometer terminates within the more permeable surface overburden and the second at the depth of the tunnel in the sandstone rock (with a watertight clay plug at the interface with the overburden to isolate the two water tables). The second deep piezometer readings show a water table level just below the proposed building basement invert level (RL10.7m), this being 6m above the underside of the tunnel concrete invert. Once the building is complete and occupied, the building's weight will counter the weight of the rock removed to form the basement. The overall weight of the ground removed will not be completely offset by this future loading and as such will not impact the existing tunnel.

Assessments were carried out on the tunnel unreinforced concrete invert, and identified that it would fail under an external pressure head of 7m. The likely explanation for its actual non-failure is that there is actually very little water pressure acting on the tunnel lining. One possible reason is because the hydraulic conductivity of the tunnel lining is higher than the hydraulic conductivity of the surrounding rock which has few rock defects. The hydraulic conductivity of the tunnel lining is controlled by existing concrete cracks and the construction joints in the tunnel lining (which are both longitudinal and circumferential). The tunnel is effectively a drainage gallery. Reference was made to (Tammetta et al 2004 and Yuqi Tan et al 2018) to try and quantify this but there was insufficient parametric information.

Also, given the topography of the site location (it is higher than the surrounds), water runoff will be very efficient over the surface and along the approximately 3m of permeable ground overlying the rock strata. It also apparent from the piezometer install in April 2022 and rainfall records that the water table levels around the tunnel are not sensitive to rainfall. The source of groundwater around the tunnel could in fact be kilometres away.

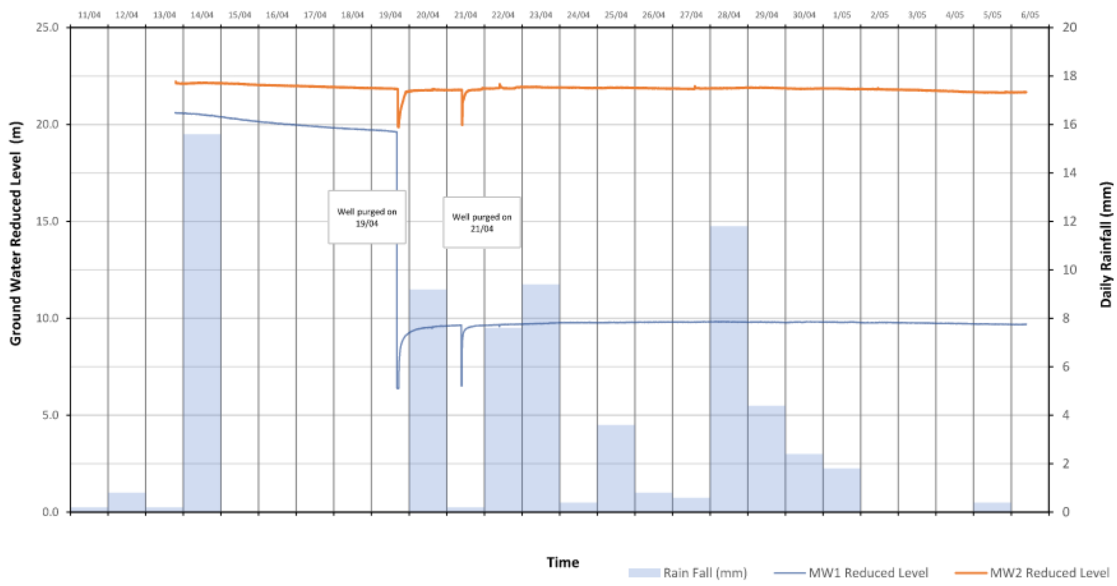


Figure 7. Perched and lower water table levels and rainfall data, just after and days after

installation of the piezometer (lower values after stabilisation)
SYDNEY WATER CRITERIA

Figure 5 from the SEA procedure for the exclusion zone around the tunnel has been reproduced below. Notably this figure provides for both a horizontal and vertical clearance from the tunnel to the building works of 2xD where D represents the maximum tunnel dimension. The limit lines, based on the dimensions of the tunnel, are marked in blue on Figure 2 of the report. The building has been designed to maintain the clearances required (Sydney Water 2015).

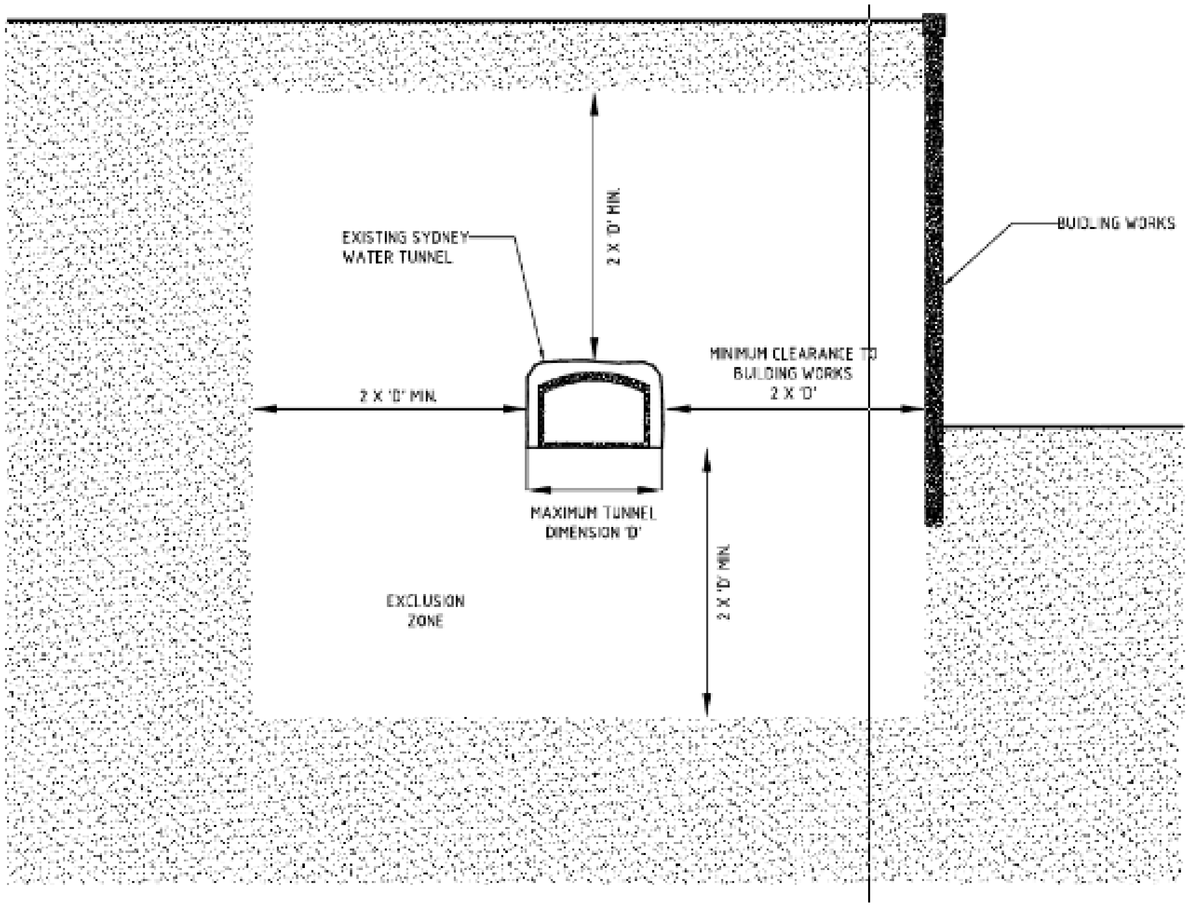


Figure 8 – Minimum clearance to building works for direct buried assets.

Section 4.5.4 Structural Criteria (Reference 4) lists items that must be satisfied when assessing the likely impacts of the building development on the existing NRGS tunnel. Given the low number of defects and high strength of rock surrounding the tunnel and its distance from the excavation the majority of the issued raised in the list of items do not apply to this tunnel. As the analyses provided shows the maximum ground displacement around the tunnel is of the order of 0.5mm or less. The maximum displacements and stress changes in the tunnel lining will be during unloading of the site (due to the basement excavation). Reloading of the ground due to the building construction partially replaces the ground load removed (as per below). Bulk weight of the basement excavation = 1200m^2 x 22kN/m^3 x 12m = 316,800kN or 264 kPa applied at the base of the excavation. Area of each floor including the ground floor above the basement 1660m^2. The average floor loading: Basement Carpark (DL + LL) = 10kPa/level (3 levels, hence total = 30kPa, gives 1200 x 30 = 36,000kN. Tower loading (DL + LL) = 10kPa (13 levels, hence 130kPa, gives 1660 x 130 = 215,800kN). Total building weight = 215,800 + 36,000 = 251,800 kN < the 316,800 kN of the excavated basement material (even with a margin of error of 20% added on to the weight of the building the building weighs less than the bulk excavation).

CONSTRUCTION VIBRATIONS

Given the quality of the rock the expected construction equipment would include bulldozers, hydraulic rock breaker, saw tooth cutter wheels and a bored piling rig as used on other Hawkesbury Sandstone rock sites.

Regarding potential ground vibrations due to excavation and piling works we provide some background information from a past and recent Environmental Impact Statement (EIS).

Hydraulic rock-breakers: The table below sets out typical ground vibration levels at various distances from rock-breakers operating in hard sandstone. Use of smaller machines can reduce levels of vibration significantly (New Southern Railway Impact Statement – 1994).

Table 1: Rock-breaker vibration levels (mm/sec) by distance

Plant Item	Vibration level (mm/sec) at a given distance					
	5m	10m	20m	30m	40m	50m
Heavy rock-ham.	4.5	1.3	0.4	0.2	0.15	0.1
Light rock- ham.	1	0.3	0.1	0.05	0.01	-

In the table above heavy and light rock-hammers have not been defined, however, the following data taken from the Western Harbour Crossing EIS (2020) does give minimum distances to sensitive structures (with a Peak Particle Velocity limit of 2.5mm/sec).

Table 2: Rock-breaker vibration levels (mm/sec) by distance (bulldozer and bored piles)

Plant Item	Recommended minimum working distance – cosmetic damage	
	Details	from Structures(m)
Large rock-hammer	1600 kg ham.- 18 to 34 tonne excavators	30
Medium rock-ham.	900 kg hammer – 12 to 18 tonne	15
Small rock-ham.	300kg – 5 to 12 tonne	5
Bulldozer	D10 with ripper	10
Pile boring	less than or equal - 800mm dia.	5

The 3mm/sec PPV limit set by Sydney Water at the NRGS tunnel will not be exceeded as the distance of the tunnel from the closet excavation boundary is at least 8m and provided that the appropriate equipment is used. There is considerable experience in excavating near sensitive structures in sandstone rock in Sydney.

Irrespective of the above data, ground vibrations are monitored on site.

GROUND STRUCTURE INTERACTION ANALYSIS

A series of finite element analysis (FEA) models have been developed with the intention of determining firstly whether there is any risk to the functionality of the tunnel and the tunnel unreinforced concrete lining and secondly to determine the parameters that have the most influence on this assessment (FE software Rocscience's RS2). These were supplemented with hand calculations for the structural capacity of the tunnel invert for external water pressure.

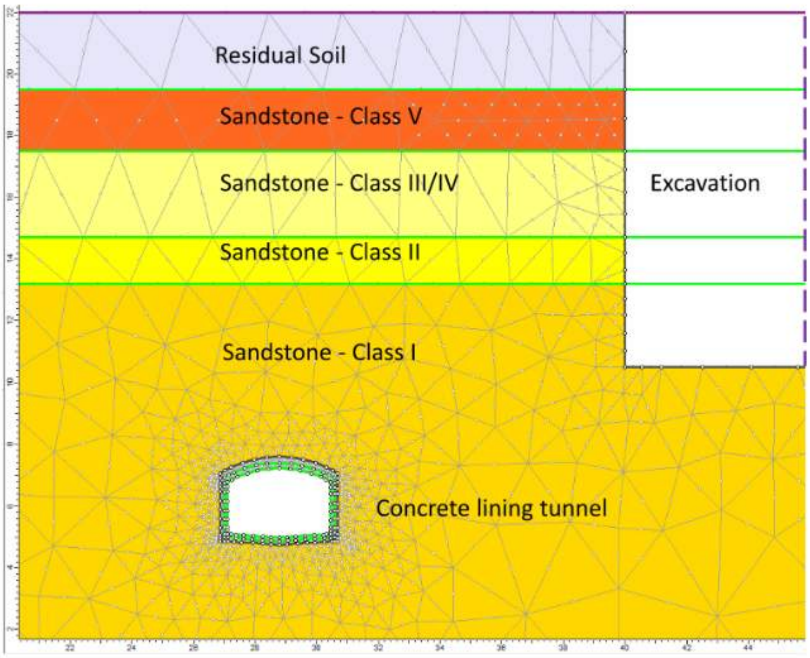


Figure 9: Section of FE grid and material strata used in analysis.

The rock has been taken as elastic because displacements/stress changes in the rock mass are so small (compared to the intact strength of the rock) that linear elastic parameters are very realistic and representative. FEA carried out with a range of rock modulus between 2000MPa and 4000MPa in the Class I/II which surrounds the tunnel with no significant change (this variation has more potential impact on the results of the analysis). The tunnel is below and 8m away from the basement excavation in a zone considered for all practical purposes isolated from stress changes on the excavation boundary (i.e. particularly at the lower corner).

Table 3: Basic material parameters and type.

#	Strata	Weight (kN/m^3)	Modulus E(MPa)	U	Material Type
1	Residual soil – sandy stiff clay	20	50	0.3	elastic
2	Sandstone Class V	22	75	0.3	elastic
3	Sandstone Class III/IV	23	500	0.25	elastic
4	Sandstone Class II	24	900	0.25	elastic
5	Sandstone Class 1 – lower bound	24	2000	0.2	elastic
6	Sandstone Class 1 – upper bound	24	4000*	0.2	elastic
7	Blast Damaged – Sandstone Class 1	24	25% of intact	0.3	elastic
7	Tunnel Lining Concrete	24	20,000	0.2	elastic
8	Tunnel Lining Concrete* see Table 2	24	20,000	0.2	plastic

*--the upper bound value of 4000MPa is likely to be higher based on back calculation of measured deformations in more recent published tunnel data in Class I/II Sandstone.

Table 4: Failure criteria for concrete when plastic material.

Type	Lower Bound Data	Upper Bound Data
Material Type	Plastic	Plastic
Peak Strength		
Peak Tensile Strength (MPa)	0.5	2*
Peak Friction Angle(degrees)	34	34
Peak Cohesion (MPa	0.5	0.5
Residual Strength		
Residual Tensile Strength	0	0
Residual Friction Angle	30	30
Residual Cohesion (MPa)	0.5	0.5
Dilation angle(degrees)	0	0

*-- given the apparent durability of the tunnel concrete lining (>80 years old) it is likely the characteristic strength is greater than 20MPa and therefore the tensile strength is more likely to be at least 2MPa.

Table 5: Where construction joints in tunnel lining FEA model – parameters

Failure Criteria	Tensile Strength (MPa)	Peak Cohesion (MPa)	Peak Friction Angle (deg)
Mohr Coulomb	0	0	45

Table 6: List of FEA models

#	Description
1A	Concrete Lining – elastic, Sandstone Class 1 E = 2000MPa, H:V stress ratio 2:1
2A	Concrete Lining – elastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 2:1
2AB	Concrete Lining – elastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 5:1
3A	As above but with construction joints added
4A	Concrete Lining – plastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 1:1 Mohr Coulomb Failure Criteria = Upper Bound peak tensile strength 0.5MPa
5A	Concrete Lining – plastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 1:1 Mohr Coulomb Failure Criteria = Upper Bound peak tensile strength 2MPa
6A	Concrete Lining – plastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 2:1 Mohr Coulomb Failure Criteria = Upper Bound peak tensile strength 2MPa
7A	Concrete Lining – plastic, Sandstone Class 1 E = 4000MPa, H:V stress ratio 2:1 Mohr Coulomb Failure Criteria = Upper Bound peak tensile strength 2MPa *Blast damaged rock around tunnel perimeter, E = 1000MPa*
8A	Foundation FE grid used for all models
9A	Zone of blast damage rock around tunnel perimeter.

If there was an external water pressure head acting on the tunnel invert (which has a 3.35m clear span between the walls) at the level indicated by the piezometer records, then the tensile bending stress calculated would have exceeded the 1 MPa Sydney Water criteria. This is demonstrated by both hand calculations and by FEA models where there is no bond between the concrete lining and the surrounding rock. The tunnel lining was modelled using both continuous FE elements and other models using beam elements (for average axial, bending moments and shear loads).

Figure 10: Exposed geology of a nearby site

SUMMARY AND CONCLUSIONS

The key assumption is that the tunnel concrete lining has no rock load in its initial state after its construction. Then the only load that is applied to the lining is a result of the subsequent basement excavation (causing ground relaxation or uplift). There is potential for ground water loading.

There are 2 other important issues, namely the in-situ stress in the ground and the modulus of the Sandstone Class 1.

While there are numerous references quoting high in-situ ground stress most of the actual data is based on mining industry measurements taken at depths far greater than normal civil engineering projects. While it is true that evidence of high in-situ stresses exists (from observation) in Sydney CBD projects it is not a given that they exist on every site. Also given the quality of the Sandstone Class 1 rock on most sites with deep basement the potential consequences of high in-situ are not always observable. As the building site is located on a ridge, it is likely that it does not have high locked in tectonic stresses.

Nonetheless the FEA runs include H:V of 1:1 and 2:1 to assess the sensitivity of changing this ratio (and where H:V is the ratio of the undisturbed in-situ stress ratio of the horizontal ground stress to the vertical stress). We included one example FEA model analysis with a H:V ratio of 5:1 for comparison purposes only.

The overriding parameters that result in the very small displacements around the tunnel is firstly the quality of the Sandstone Rock - Class 1, which has widely spaced defects and which surrounds the tunnel and extends up the wall and across under the basement excavation. The second parameter is the location of the tunnel relative to the basement excavation. The tunnel crown is 3m lower than the base of the excavation, and the closest tunnel wall is almost 9m clear of the basement excavation boundary.

So even though the FEA models' analyses are very different these two parameters dominate the results with regards to determining the potential impact on the tunnel lining.

Also given the quality of the rock it can be assumed that during the excavation phase, apart from barring down loose rock wedges or blocks because of the drill & blast method used, that the tunnel was for all practical purposes self-supporting. Even as late as the mid-1960s this was the approach used by Sydney Water in drill & blast tunnels.

The identified risks to the NGRS tunnel are extremely low for the following reasons.

1. The rock surrounding the tunnel has defects that are widely spaced, and the intact rock is of high strength being Class I/II sandstone.

2. The location of the tunnel crown below the basement excavation level by at least 3m and its lateral offset location of 8m from the basement excavation boundary.

3. Potential ground vibration impacts can be managed by using appropriate construction methods and monitoring and using appropriate excavation plant.

4. The FE analysis predict minor ground movements at the level of the tunnel. Stress concentration points may develop in the upper and lower corner of the tunnel lining. Concrete cracking would also be partially mitigated by movement at the construction joints. Cracks widths will be minimal if even visible/measurable in this tunnel environment.

5. External water pressure - the building basement excavation, because it is drained, will maintain the existing the water table level over the crown of the tunnel whatever the rainfall or ground conditions. Actual piezometer readings and the groundwater load calculations, when considered together, confirm that the tunnel lining has not been or is unlikely to be impacted by external ground water pressure.

6. Potential structural element instability (walls and invert) due to new cracks are not expected to develop nor is there expected to be any change in the groundwater regime. Clearly if there were stability issues with this 80-year-old tunnel, they would have developed previously. The basement excavation is not expected to change the tunnel environment to any significant degree.

7. Any pile loads directly above the tunnel will dissipate into the intact rock mass well above the rock surrounding the tunnel and not impose loads onto the tunnel lining itself.

8. The dilapidation survey concrete coring of the tunnel wall confirms, in our opinion, that the drill and blast excavation method used to excavate the tunnel with the inherent overbreak in the rock would result in a lining concrete thickness greater than the design thicknesses shown on the Sydney Water sketches provided.

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Optimised Foundation Design for Multi-Level Car Park Above Twin Rail Tunnels: A Novel Approach Using Reinforced Concrete Piles and Grouting Strategy

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ABSTRACT

Cherrybrook Station is one of the metro stations on the Sydney Metro Northwest project. The station has a multi-level car park positioned over twin tunnels and adjacent to the metro station box. However, the twin tunnels were not originally designed to accommodate any additional load from the multi-level car park. To address this challenge, the foundation design for the multi-level car park was developed such that the axial loads are transferred to the high-strength rock beneath the twin tunnels, while allowing the ground above the tunnels to provide lateral support for resisting horizontal loads. The foundation design involves the use of permanently cased cast-in-place reinforced concrete piles constructed in an oversized bored hole, with different grouts used to fill the annulus between the ground and pile at different depth along the pile. A low-strength grout was used near the tunnels to reduce the load transferred to the ground and tunnels, while a high-strength grout was used near the top of the pile to provide a firm connection with the surrounding ground. This paper presents the details of the foundation design that addresses the constraints and challenges associated with constructing a car park over existing twin tunnels, and how interfaces with existing and future works were considered in the design.

INTRODUCTION

Sydney Metro Northwest is a significant infrastructure project in Australia, aimed at connecting the north-west growth areas of Sydney to Chatswood through a fully-automated metro rail system, encompassing 8 new stations and 23km of rail track. The $8.3 billion project was delivered under three contracts, including the Tunnels and Station Civils (TSC) contract which involved the construction of 15km twin tunnels, station box excavations, and temporary ground support. The Operations, Trains and Systems (OTS) contract included the construction of the permanent station box, commuter car park, metro trains infrastructure, and rail system. The Surface and Viaduct Civil (SVC) contract covered the construction of the 4km elevated skytrain viaduct between Bella Vista and Rouse Hill.

Cherrybrook Station is located adjacent to Castle Hill Road between Franklin and Robert Roads. It is an open-air structure within a terraced cutting and includes a multi-level commuter car park (MLCP) located at the city end of the station as shown in Figure 1, which was constructed as part of the OTS contract. The car park is situated above Sydney Metro Northwest twin tunnels and adjacent to the station box, which were constructed as part of the TSC contract. One of the key challenges in designing the MLCP was to develop a foundation design that addresses the complex interfaces with the existing TSC infrastructure while optimising the arrangement for the MLCP superstructure by limiting the spans between columns.

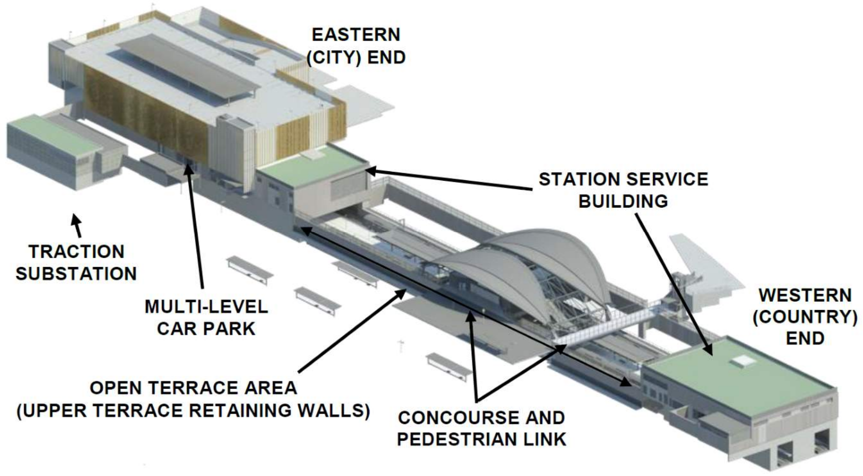


Figure 1. Architectural rendering of the Cherrybrook Station, with the multi-level car park situated at the eastern end of the station above the twin tunnels.

GROUND CONDITIONS

Geological face mapping of the east wall of the station box indicates that the ground conditions at the location of the MLCP typically consist of 1-2m of residual soil underlain by up to 8.0m of Ashfield Shale (Regentville Siltstone), which is generally weathered or heavily jointed/fractured (Class V/IV/III). Beneath the Regentville Siltstone, up to 6.0m of Kellyville Laminite and Rouse Hill Siltstone (Class II/I) is encountered, reaching the station box excavation floor at approximately RL160.8mAHD. No sandstone was encountered during the excavation of the station box, although it has been identified below depths of approximately 35m in the surrounding boreholes.

Faulting has been identified at the eastern end wall of the station excavation, mapped as a band of heavily jointed and fractured rock (SHA-IV/V), up to 2.5m thick, with the primary fault dipping at 040° with a dip direction (mag) of 282°. This fault line intersects the wall at approximately RL167mAHD and reaches the base of the excavation approximately 2.5m from the south side of the eastern end wall, as shown in Figure 2.

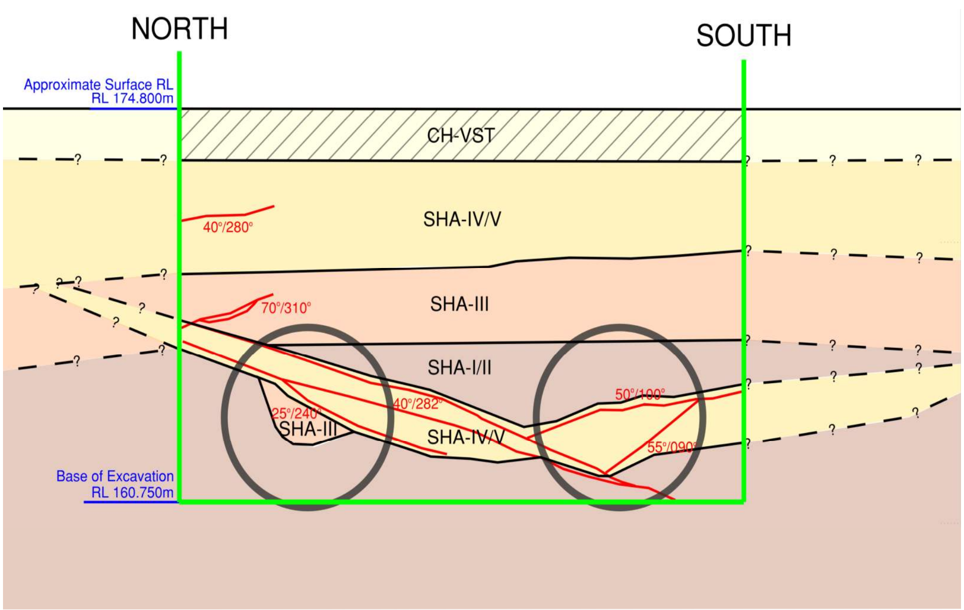


Figure 2. Geotechnical mapping and interpreted section at the Eastern portal below the multi-level car park.

EXISTING AND FUTURE INFRASTRUCTURES

The design of the foundation for the multi-level commuter car park (MLCP) at Cherrybrook Station had to take into consideration the existing infrastructure that was constructed during the Tunnels and Station Civils (TSC) works, including the twin rail tunnels, nozzle enlargement, and face support for the station box's east end wall. The design also had to accommodate a cut wall (soil nail wall) along the eastern side of the car park constructed as part of the OTS contract, extending 4.2m below the top of the pile.

Rail Tunnels and Nozzle Enlargement

The twin tunnels were constructed as part of the TSC works and are made of precast segmental lining with a typical outer diameter of 6985mm (I.D. minimum of 6000mm). Within the multi-level car park area, the twin tunnels are spaced approximately 4.5m apart, with their crowns at around RL167.3mAHD, resulting in a clear cover of approximately 6.7m to the car park foundation level. At the station box interface, there is a nozzle enlargement with an outer diameter of 7610mm, which extends around 12m from the station box's east end wall. The nozzle crown is supported by temporary rock bolts that are 3600mm long and arranged in a 5/6 alternating pattern at 1000mm longitudinal centres. The nozzle pillar supports consist of temporary glass reinforced polymer (GRP) rock bolts that are 3000mm long and arranged in a 3/2 alternating pattern at 1000mm centres (Figure 3).

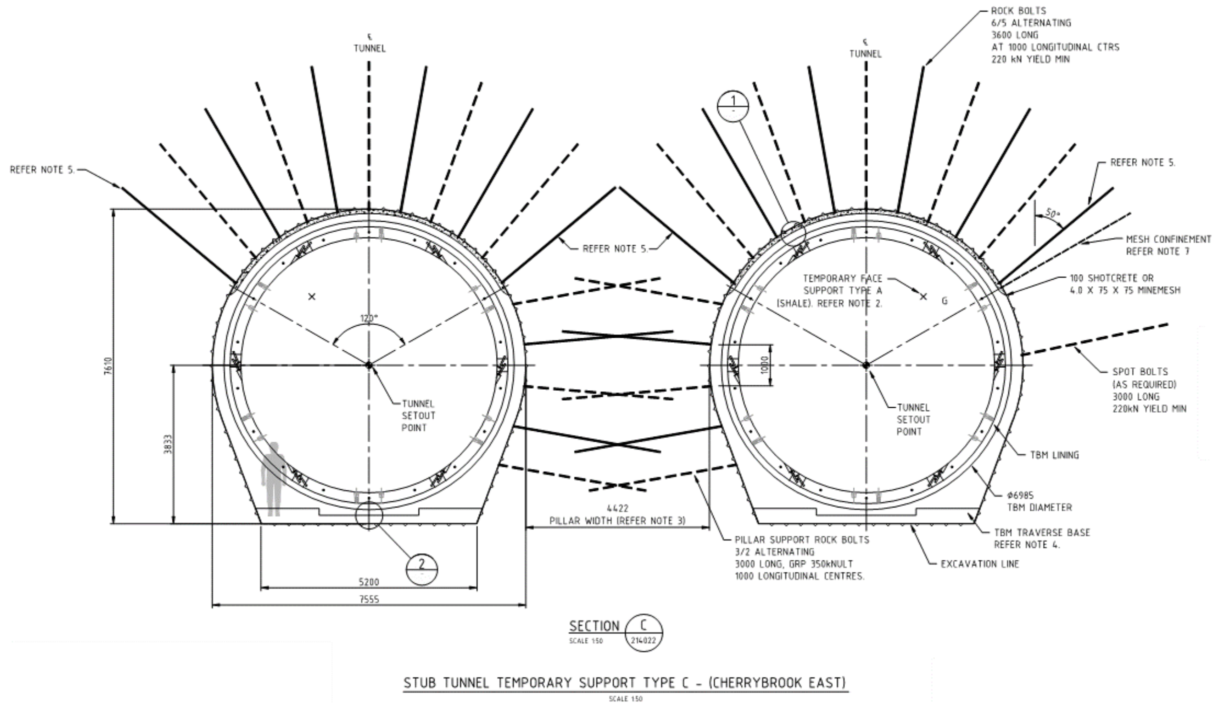


Figure 3. Temporary support arrangement for the nozzle structure at the Cherrybrook eastern end wall.

Eastern End Wall of Station Box

Due to the presence of the fault-affected zone, a band of class IV/V shale, additional temporary ground supports were required as part of the TSC works to support the eastern end wall of the station box excavation. These TSC supports are critical interface to the MLCP foundation design, which consists of fourteen 13m long post-tensioned ground anchors. Among them, ten were installed between and parallel to the running tunnels at a 10-degree angle from horizontal, while the remaining four were installed above the northern half of the upline running tunnel, between RL168.5mAHD and RL168.8mAHD, at a 10-degree angle from horizontal and at a plan angle of between 25-30 degrees outward from the upline TBM alignment. These anchors can be seen in pink in Figure 4.

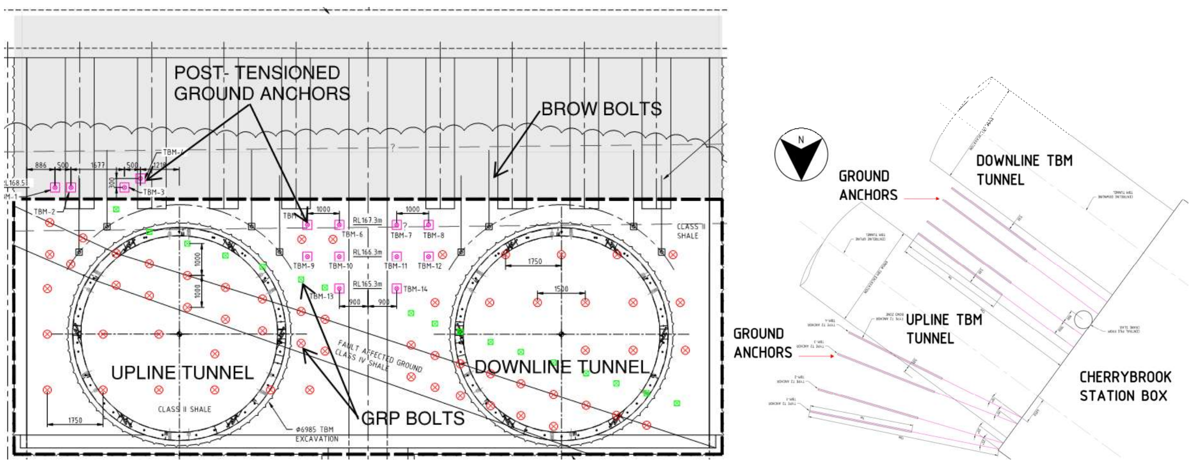


Figure 4. The existing TSC infrastructure within the footprint of the MLCP at the eastern end of station: (Left figure) elevation of the east wall of the station box excavation and (Right figure) plan view of the running tunnels and ground anchors beyond the eastern end wall.

Soil Nail Wall Adjacent to MLCP

On the eastern side of the multi-level car park at Cherrybrook Station, there is an approximately 73m long retaining wall that runs along the entry ramp and one side of the car park, as shown in Figure 5. This retaining wall consists of a 1.75m high reinforced concrete L-shaped wall constructed on top of the soil nail wall. A service tunnel connected to a vertical tunnel dropper, which runs from the eastern end wall and connects to the station traction substation, passes vertically behind the soil nail wall. The MLCP piles are located at a plan offset of approximately 4m from the face of the soil nail wall and are less than 0.5m away from the vertical service tunnel dropper. According to the construction program, the car park piles were to be constructed before the excavation of the soil nail wall and the service tunnel dropper. Therefore, measures were required to isolate the car park piles from the ground movement induced by the excavation of the soil nail wall.

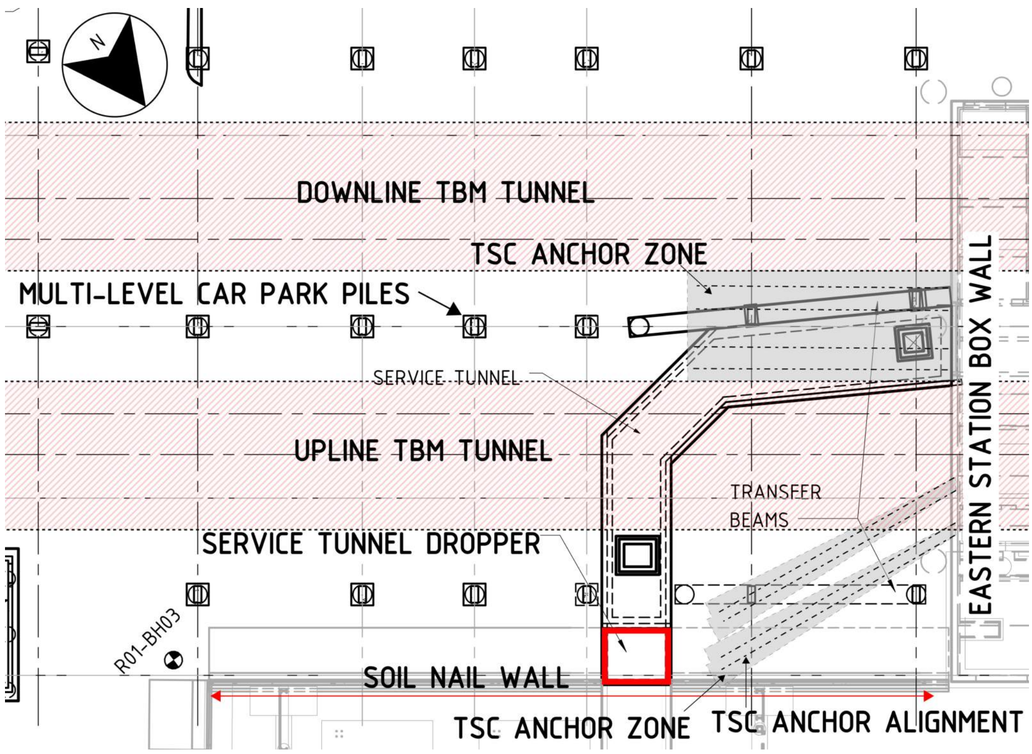


Figure 5. Plan view of soil nail wall and the vertical service tunnel dropper next to the car park.

MULTI-LEVEL CAR PARK FOUNDATION DESIGN

Foundation Layout

The foundation layout for the multi-level car park (MLCP) was carefully planned, with the car park columns and piles located between and on either side of the tunnels, as shown in Figure 5 and Figure 6. These MLCP piles have been positioned at an approximate clear distance of 2m from the existing twin tunnels. Post-tensioned ground anchors were installed as part of the TSC works, between and parallel to the running TBM tunnel and above the northern half of the upline twin tunnels, adjacent to the eastern end wall, as shown in Figure 5. The construction tolerances allowed by the TSC works were taken into account to assess the corresponding anchor zone, as shown in Figure 5. To mitigate the risks of clashing with the TSC anchors, horizontal structural transfer beams were designed to span over these ground anchors, as shown in Figure 5. Furthermore, the existing tunnels and the car park piles were accurately incorporated into the station BIM model to achieve the correct positioning of the piles and to further reduce the risk of clashes.

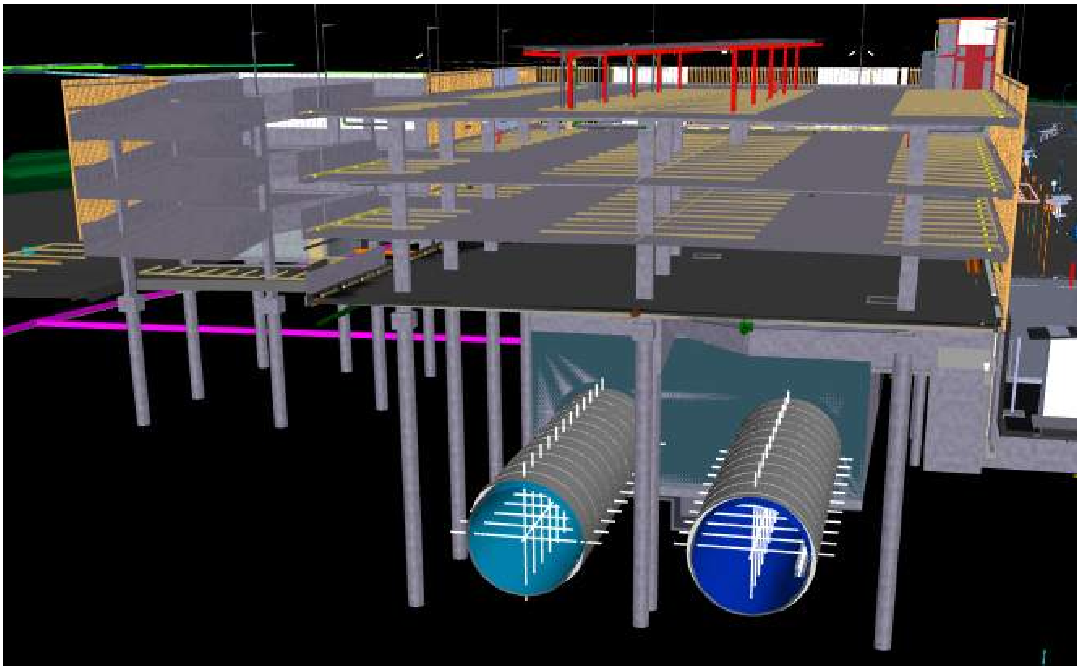


Figure 6. Section cut through the BIM model showing the car park structure and foundations in relation to the twin rail tunnels, looking towards the west.

Pile Design Concept

Piling adjacent to existing tunnels is becoming more common in urban areas, and guidelines have been developed by various asset owners specifying the minimum offsets of piles from tunnels to mitigate the risk of piles impacting existing tunnel infrastructure. For example, London Underground Limited (LUL) requires a minimum offset of 3m for bored piles and 15m for driven piles installed adjacent to LUL tunnels (Chudleigh et al., 1999). Furthermore, pile designs that reduce the impact of pile loading on tunnels typically involve positioning the toe of piles below the tunnel invert level and adopting specific pile design measures (Fellenius, 1998; Azad et al., 2014). These pile design concepts include:

- Double steel casings with a clear gap to fully isolate the pile from the ground.
- Single permanent steel casing coated with a friction reducing compound, such as bitumen.
- Placing a single casing in an oversized hole and grouting the gap with a soft grout (Schroeder et al., 2004; Beadman et al., 2012; Schroeder, 2003).

There were no specific tunnel offset requirements for the multi-level car park (MLCP) piles, and initially, the typical approaches outlined above were considered. However, double casing and soft grout arrangements were found to be ineffective due to the significant bending moments at the pile socket caused by lateral loading. This is because the unsupported length along the double casing causes more bending moments, despite the lateral loading being lower compared to a commercial office building. Additionally, this solution is relatively expensive. Moreover, the client preferred not to use bituminous coatings due to the challenges associated with applying a uniform coating and handling the casings without damaging the coating. As a result, a novel pile design was developed (as shown in Figure 6), which ensures that substantial axial loads are transferred to the high-strength rock below the tunnels, while allowing the ground above the tunnels to provide lateral support to resist horizontal loads.

The pile design involved the use permanently cased cast-in-place reinforced concrete piles constructed in an oversized bored hole, with the gap between the ground and pile being filled with one of two types of grouts:

- Next to the tunnels, a very low-strength grout with low compressive stiffness is used to significantly reduce the vertical load transferred to the ground and onto the tunnels.
- Within 4m of the top of the pile, a high-strength grout is used to provide a firm contact between the pile and the surrounding ground, thus providing significant lateral support to the pile.

Furthermore, this pile arrangement has the added benefit of being able to isolate the pile from the surrounding ground during the excavation of the adjacent soil nail wall and vertical service tunnel dropper, to reduce any movement induced by soil nail wall excavation from affecting the piles. This was achieved by leaving the outer annulus as a void until the monitoring data from the soil nail wall construction confirms that the ground movements have stabilised. Following this, the annulus can be grouted as per the final design arrangement.

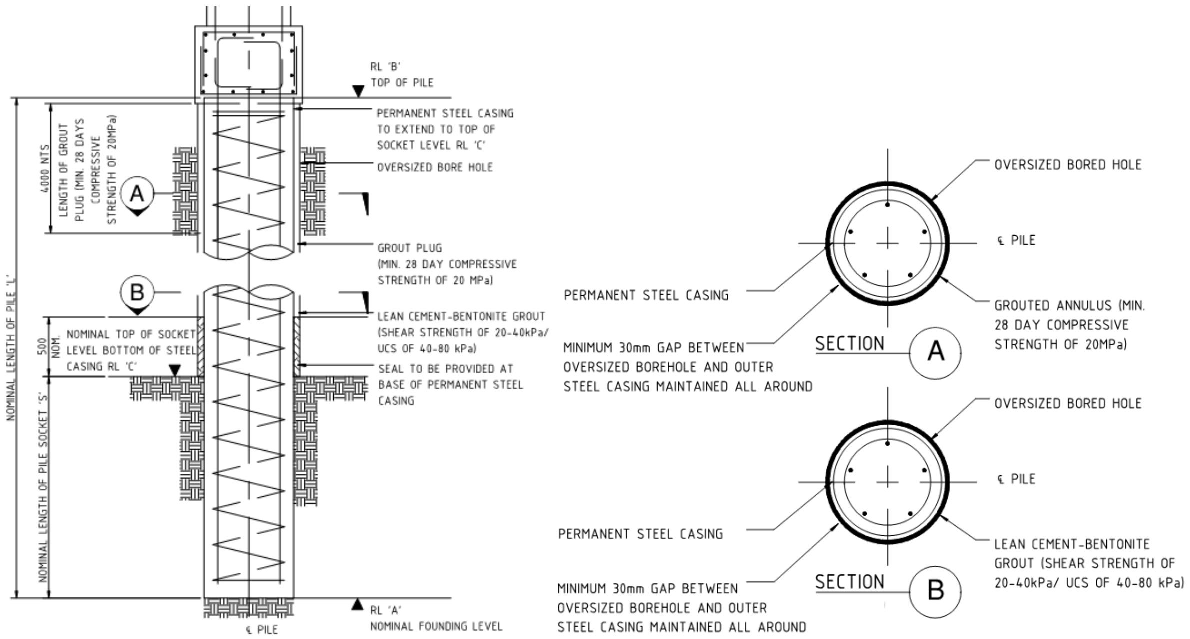


Figure 7. Section and details of the pile design developed for the MLCP (a) Section showing the pile design concept (b) Details of the pile design concept.

Analysis

The pile design was design and substantiated using several methods. Firstly, an axisymmetric finite element analysis was performed to demonstrate the efficacy of the grouting strategy in limiting the axial load transferred to the ground above the pile socket level. Then, laterally loaded pile analysis using LPile was conducted to assess the lateral performance of the piles to demonstrate the effectiveness of the hard grout within the top 4m of the pile in providing lateral restraint. Finally, a plane-strain finite element model was used to assess the interaction of the piles with the running tunnels. For comparison, the analyses were also carried out for two other pile designs: a standard pile with no casing, and a pile that is isolated from the ground above the pile socket level using a widely used double casing arrangement with a clear gap in the annulus between the inner and outer casing. The analysed cases are summarised in Table 1. The analysis considered the worst case loading conditions from all of the piles, as summarised in Table 2. For type A and C piles, the length of the pile casing considered is 14.35m, which extends from the top of the pile to below the tunnel invert level.

Table 1. Pile design types analysed.

Pile Type	Description
Type A (Double Casing)	Pile isolated from the ground via a double steel casing arrangement – In the axisymmetric finite element (FE) analysis, a gap is specified between the pile and the surrounding ground, and only the pile socket is in direct contact with the surrounding ground. In the LPile analysis, no lateral restraint is considered above the top of the socket level.
Type B (No Casing)	Pile without steel sleeve or casing – In the FE and LPile analysis, the entire length of the pile is in direct contact with the surrounding ground.
Type C (Single Casing with Grouts infill)	Pile with a single casing and grout infill in the pile annulus between the casing and oversized hole – In the FE and LPile analysis, a low-strength soft grout was modelled between the pile and the surrounding ground above the pile socket level. A 4m high-strength hard grout was modelled above the soft grout in the FE analysis. In the LPile analysis, the soil and rock adjacent to the pile hole within the top 4m were modelled.

Table 2. Design load cases considered.

Pile Diameter (mm)	Pile Length (m)	Design Socket	Level of Pile Sleeve	Ultimate Limit State Loads			Serviceability Limit State Loads		
				Axial (kN)	Shear Force (kN)	Bending Moment (kNm)	Axial (kN)	Shear Force (kN)	Bending Moment (kNm)
900	17.35	3m SHA-II	14.35	5100	400	0	4050	100	0

Table 3. Ground parameters adopted for finite element analysis.

Material	Constitutive Model	Unit Weight (kN/m^3)	Elastic Modulus (MPa)	Poisson's Ratio, ν	Hoek-Brown (HB) Criterion			s_u (kPa)	Ultimate Lateral Resistance for LPile (kPa)
					mb	s	a		
Very Stiff Clay	Mohr Coulomb	20	40	0.3	–	–	–	*	–
SHA-V	Gen. HB	21	50	0.3	0.459	0.0001	0.544	–	1,500
SHA-III	Gen. HB	23	300	0.3	0.939	0.0013	0.511	–	5,000
SHA-II	Gen. HB	24	700	0.2	1.341	0.0039	0.506	–	30,000
Pile	Elastic	24	32,000	0.2	–	–	–	–	–
Soft Grout	Mohr Coulomb	19	10	0.3	–	–	–	40	360
Hard Grout	Elastic	24	32,000	0.2	–	–	–	–	–

* Modelled as drained condition with an effective stress parameter c' = 10kPa and φ' = 28°

Axisymmetric FE Analysis

The settlement and load transfer behaviour of the three types of piles under axial loading was assessed using an axisymmetric finite element (FE) analysis in Rocscience RS2. The pile was modelled in the centre of the axisymmetric model as a solid cylindrical element with a radius of 450mm surrounded by a soil column, as shown in Figure 8. The total length of the pile modelled was 17.35m with a pile socket of 3m into high-strength class II shale. The tunnel has not been modelled in the axisymmetric model. The mesh consists of 6-noded triangular elements and the distance to the horizontal boundary was chosen to reduce the boundary effect on the pile load transfer behaviour. As the car park piles are widely spaced at a distance greater than 5m (more than 3 times the pile diameter), it is expected that the pile group interaction effect will be minimal. The subsurface profile was modelled and the ground parameters adopted are summarised in Table 3. The rock layers were modelled using the generalised Hoek-Brown model, while the soft grout and very stiff clay were modelled using the Mohr-Coulomb model. The pile was modelled using a linear elastic model.

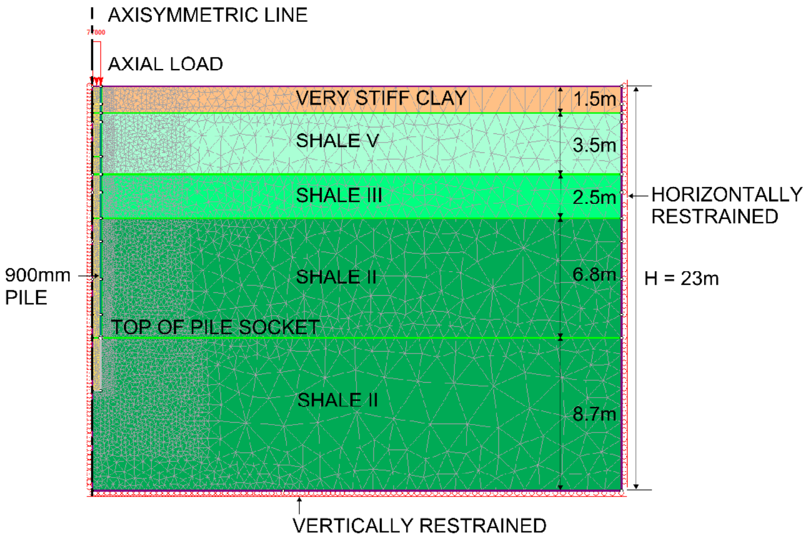


Figure 8. Finite element mesh with 6-noded triangular elements used for the axisymmetric analysis.

Figure 9 and 10 show the calculated ground deformation contour and the mobilised skin friction. In Figure 10, the location of the TBM tunnel has been overlaid for easy comparison.

The results show that the type B pile, i.e., without steel sleeve or casing, transfers the axial load to the surrounding ground via skin friction, with a computed greenfield ground movement at the location of the tunnel of up to 10mm. The results also show that despite the high-strength grout plug that is modelled in the type C pile, the skin friction mobilised within the soft grout is very comparable to that mobilised in the type A pile using a conventional physical isolation method via double steel casing. The results indicate that the high-strength grout plug, which is required to resist the lateral load, has not caused significant axial load transfer to the surrounding ground. The computed ground deformation with both the type A and type C piles at the location of the tunnel is negligible (less than 1mm).

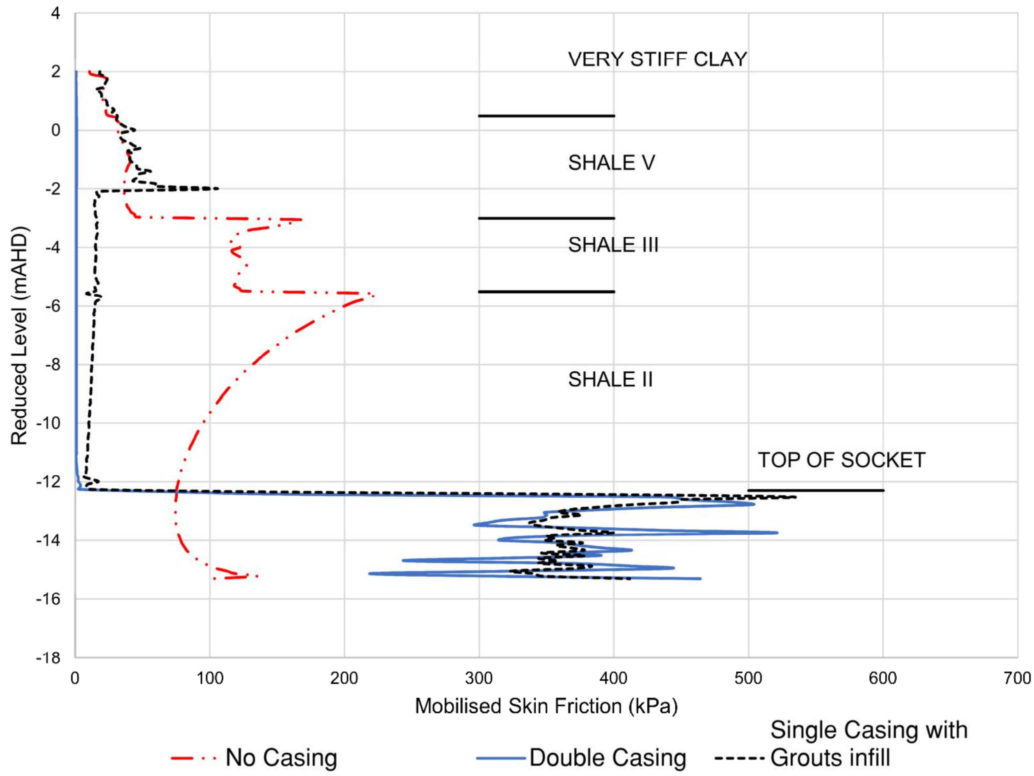
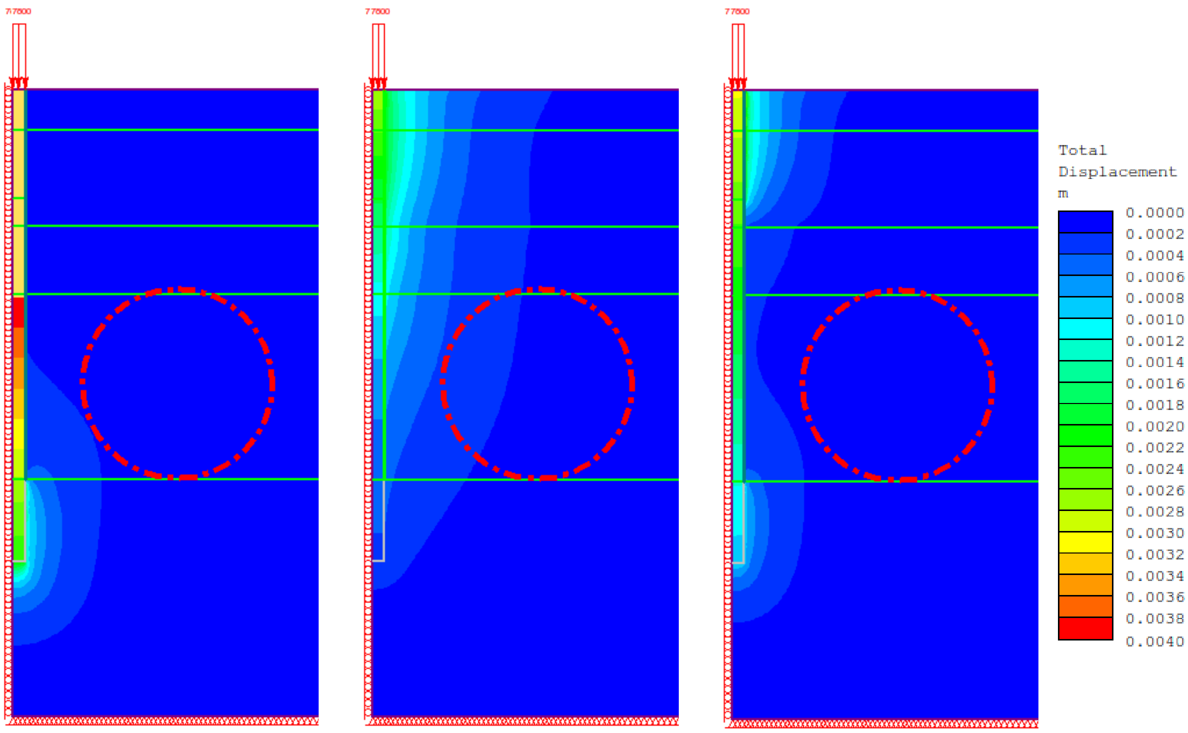


Figure 9. Mobilised Skin Friction from axisymmetric FEA.



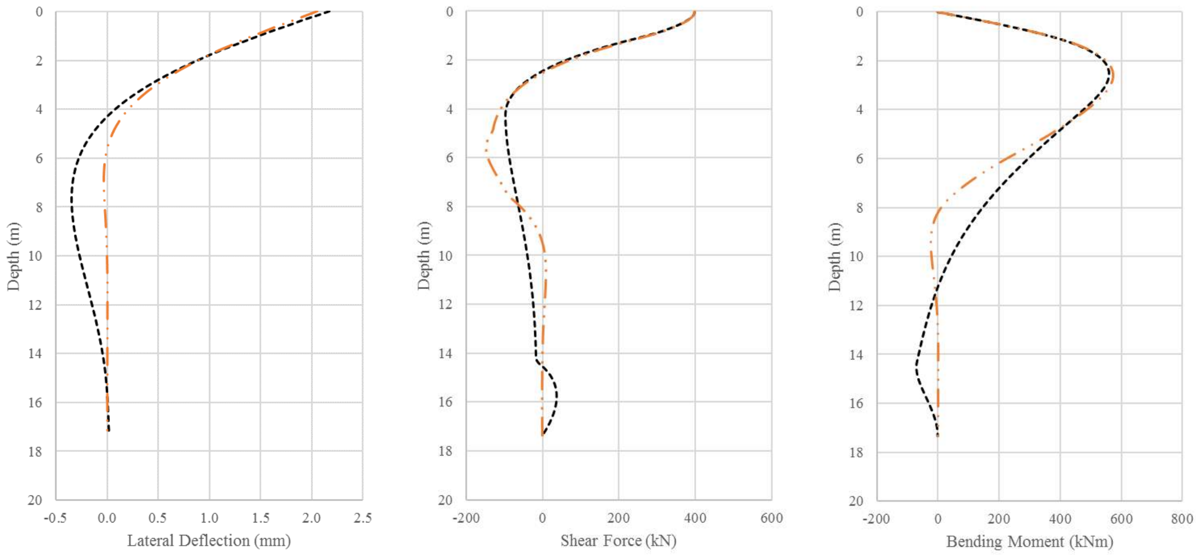
(a) Double casing (Type A) (b) No Casing (Type B) (c) Single Casing with Grouts infill (Type C)

Figure 10. Total displacement profiles from axisymmetric FEA. The location of the tunnel has been overlaid only for easy visual comparison (tunnel not modelled in axisymmetric FEA).

Lateral Pile Load Analysis

The lateral performance of the three types of piles was analysed using Ensoft LPile, with the soil and rock modelled as linear-elastic perfectly plastic materials, utilising a custom p-y curve derived from the ultimate lateral resistances summarised in Table 3. The pile performance was assessed under both ultimate limit state (ULS) loads and serviceability state loads (SLS), as detailed in Table 2. For the Type C pile, it is considered that the lateral loads are directly transmitted into the surrounding ground through the hard grout, resulting in lateral restraint being governed by the ultimate lateral resistance of the surrounding ground within the top 4m. Conversely, for the Type A piles, a clear gap between the inner and outer casings implies no lateral restraint above the socket level.

The lateral analysis results of the Type A piles indicate that the bending moments generated at the top of the pile socket due to the 400kN lateral load can exceed 11300kNm, rendering the pile size and reinforcement requirements prohibitively expensive. On the other hand, the lateral analysis results of the Type B and Type C piles, including displacement, shear force, and bending moment, are presented in Figure 11. These results suggest that the majority of lateral loads are dissipated within the top 4m of the pile for both pile types, with similar responses observed.



(a) Displacements (SLS) (b) Shear Force (ULS) (c) Bending Moment (ULS)

Figure 11. Lateral pile analysis results for the Type B (- -) and Type C (─ · · ─): (a) lateral deflections, (b) shear force, and (c) bending moment.

Plane Strain FE Analysis

An assessment of the impact of the piles on the permanent lining of the running tunnels has been carried out using a 2D plane strain finite element (FE) analysis, which is considered sufficient in this instance as Schroeder (2003) demonstrated that an accurate assessment of the impact of a row of piles adjacent to rail tunnels can be obtained from 2D analysis. Figure 12 shows the finite element mesh used in the analysis. The permanent lining was modelled as a 250mm thick Timoshenko beam element. A multi-stage stress analysis was undertaken, including the construction of the tunnels and activation of the tunnel lining, followed by the installation of the car park piles and application of the axial load to the car park pile. Due to the out-of-plane spacing of the piles and the nature of the plane-strain analysis, the pile properties and axial loads were divided by the out-of-plane spacing. An assessment of the impacts on the tunnel lining was carried out for only the ultimate limit state (ULS) load case. The displacement of the tunnel lining and the increment in the bending moment and axial load of the tunnel lining were calculated by comparing the values before and after the loads were applied, and the results are summarised in Table 4.

The analyses demonstrate that under the ULS loads, the distortion of the tunnel would be less than 2mm, the increment in the maximum shear force is less than 6kN, and the increment in the maximum bending moment is less than 4kNm. These small increments due to the pile loading are not considered to have a meaningful impact on the long-term performance of the tunnel lining.

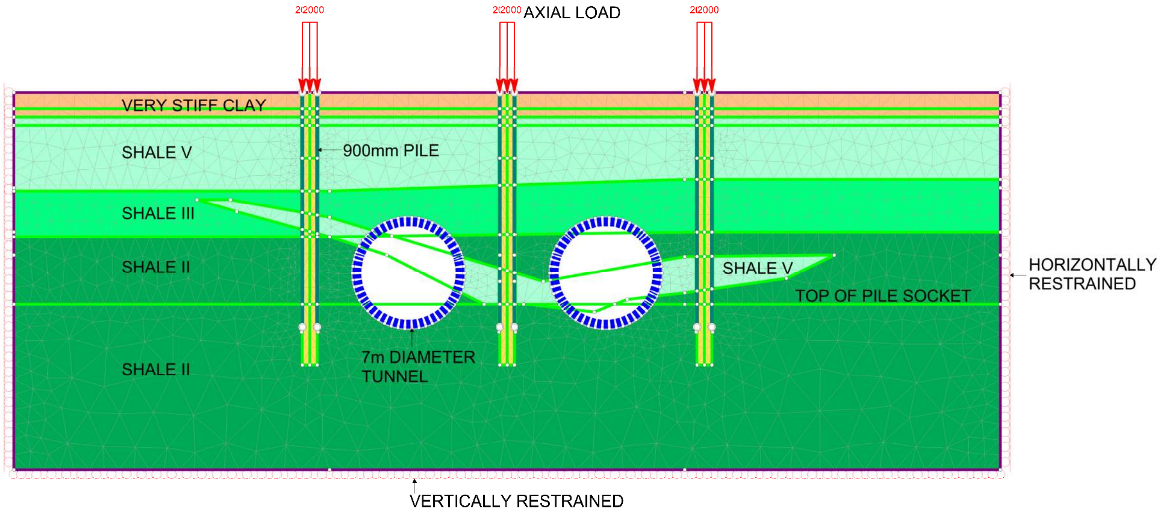


Figure 12. Finite element mesh with 6-noded triangular elements used for the plain strain analysis.

Table 4. Summary of analysis results.

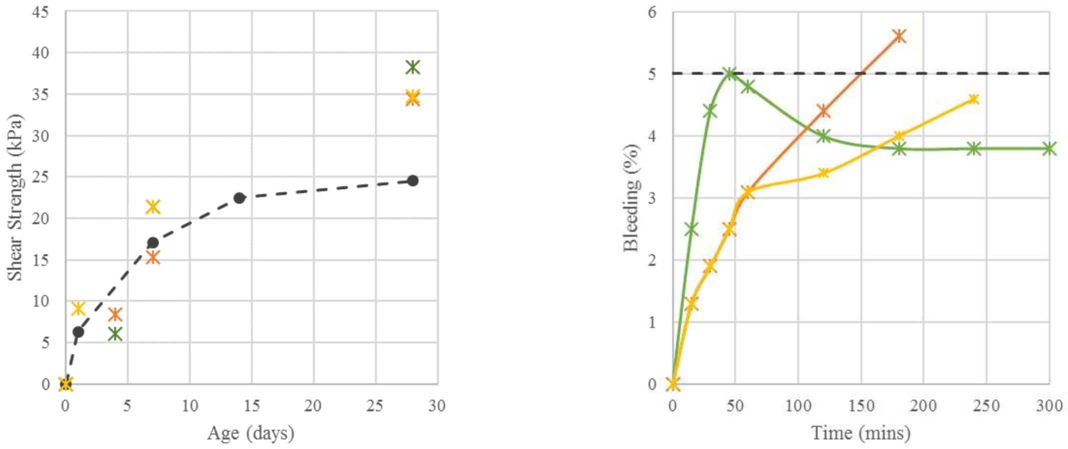
Tunnel Lining Property	Increment Due to Pile Loading
Maximum shear force	< 6kN
Maximum bending moment	< 4kNm
Maximum displacement	< 2mm

CONSTRUCTION OF FOUNDATIONS

A brief overview of the construction sequence and construction techniques implimented on site are presented in this section.

Soft Grout Mix Details

Prior to the construction of the piles, several trial mixes of soft grout were tested with the primary aim of meeting the design criteria of a 28-day shear strength between 20-40kPa without excessive bleed. The soft grout mix ultimately selected was a lean cement-bentonite mix with a cement:bentonite:water ratio of 1.67:10:1 by mass. The 28-day shear strengths obtained during testing averaged around 36kPa, which fell within the design criteria (Figure 13). The final bleed of the mix averaged 4.6%, with the maximum value obtained being 5.3%. These results were considered acceptable for the intended purpose of the soft-grout and were in line with the available literature on such grouts (Azadi et al. 2017) and the design target bleed of <5%.



(a) (b)

Figure 13. Lean cement-bentonite mix with a cement:bentonite:water ratio of 1.67:10:1 (a) Shear strength test results, with anticipated strength based on mix design (dashed line) (b) bleeding test results, with design target (dashed line).

Construction Sequence

The construction sequence was summarised as follows:

A. Drilled a 1050mm diameter oversized hole to the top of socket level.

B. Placed a 900mm steel case centrally within the oversized hole and fixed it in place with a 500mm deep concrete seal, ensuring a minimum 30mm outer annulus between the steel casing and the surrounding ground was maintained.

C. Placed soft grout (lean cement-bentonite grout) from bottom to top in the outer annulus, extending from the top of the concrete seal to 4m below the top of the pile.

D. Bored the pile socket to the pile founding level.

E. Placed pile reinforcement and concrete up to the pile cut-off level.

F. Once the soft grout reached 50% of the nominated strength or after a minimum of 4 days (whichever was sooner), placed higher strength grout from bottom to top in the top 4m of the annulus up to the pile cut-off level.

Steps A to C were captured in Figure 14, where the 14.35m long permanent steel casing for a pile was craned into the oversized 1050mm diameter hole by the tower crane, the pile hole was protected with a plywood cap, and the casing was fixed in place with a nominal 500mm grout plug at the base of the annulus.

This approach allowed multiple piles to be partially constructed (up to step E) before ordering a single batch of high-strength grout to complete the pile construction. Piles along the soil nail wall did not have any grout placed in the outer annulus until after the completion of the soil nail wall and confirmation that the ground movements had stabilised.

(a) The 14.35m long permanent steel casing for a pile is craned into the oversized 1050mm diameter hole by the tower crane

(b) The pile hole is protected with a plywood cap and the casing is fixed in place with a nominal 500mm grout plug at the base of the annulus.

(c) A high-strength grout is mixed and placed into the annulus via a lay-flat grout tube

Figure 14. Installation of the permanent steel casing in the oversized borehole.

CONCLUSIONS

In conclusion, the design of the foundations for the multi-level car park structure at Cherrybrook Station successfully addressed various interfaces with existing and future infrastructure, such as the twin rail tunnels, ground anchors, and adjacent soil nail retaining wall. This was achieved through the development of a novel pile design that involved permanently cased, cast-in-place reinforced concrete piles constructed in oversized holes, with the outer annulus filled with one of two grouts.

In the vicinity of the running tunnels, a soft grout with a maximum shear strength of 40kPa was used to reduce mobilised skin friction and axial load that could impact the running tunnels. Within 4m of the top of the pile, a high-strength grout was used to resist lateral loads from the car park superstructure, significantly reducing shear stress and bending moment induced in the piles. This allowed for optimisation of pile size and quantity of steel reinforcement. Additionally, the placement of the soft and hard grouts could be delayed to prevent lateral ground movement due to soil nail excavation adjacent to the pile from impacting the car park pile.

Overall, this pile design provides a cost-effective alternative to the conventional double sleeve arrangement, while effectively managing the risk of impacting the existing tunnels.

ACKNOWLEDGEMENTS

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Feasibility study on the effect of a building development near tunnels at Sydney CBD

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ABSTRACT

In the absence of published guidelines on how to consider the effect of developments on road tunnels, this paper presents a case study where the impact of a proposed development on an underlain road tunnel was assessed.

As part of the Transport Asset Revitalisation Project at Woolloomooloo, SMEC Australia Pty Ltd carried out a geotechnical feasibility study for a proposed development of multi-storey residential buildings. The site formed part of the land acquired by Transport for New South Wales (TfNSW) for the construction of the Eastern Distributor (ED) and Cross City Tunnel (CCT) projects and now it is intended to revitalise the residual land that may be suitable for repurposing for the multi-storey residential buildings.

The desktop study was conducted to define geotechnical design parameters, structural strength and stiffness of the ED tunnel elements, and to understand the ED tunnel easement restrictions. Then numerical modelling for two analysis cross-sections was conducted to assess the impact of the proposed buildings on the ED tunnel. Based on the analysis results, load and excavation limits for the development were recommended. Finally, the missing but crucial sets of information were listed and further considerations for the detailed design stage were proposed.

INTRODUCTION

As part of the Transport Asset Revitalisation Project (TARP) at Woolloomooloo, SMEC Australia Pty Ltd (SMEC) carried out a geotechnical feasibility study on the proposed development of multi-storey residential buildings. The site formed part of the land acquired by the client, TfNSW, for the construction of the Eastern Distributor (ED) and Cross City Tunnel projects and now that these projects are complete and operational, revitalisation of the residual land to repurpose is on the horizon.

With an integrated consideration of architectural, structural and landscape form to achieve a suitable feasible yet iconic massing presentation, the clarification of the allowable loadings and structure interaction with the ED and CCT tunnel and ramp structures was required by TfNSW to support the clarity of options and concepts being considered.

Although there are standards and guidelines such as T-HR-CI-12051-ST that clearly set out the development requirements including allowable loadings and offset to the existing rail tunnels, there are no or limited published guidelines on how to consider the effect of the proposed developments on road tunnels. In the absence of such guidelines, a site-specific feasibility study has been carried out to define the loadings and offset requirements for the proposed development over the ED tunnels.

This paper focuses on the northern part of the site interacting with the ED tunnel (Building A) and covers the desktop study performed to understand the underground condition, to define concept geotechnical design parameters, structural strength and stiffness of the ED tunnel elements, and understand the ED tunnel easement restrictions.

A feasibility assessment of excavation and surcharge limits was then performed based on the findings of the desktop study. To this end, numerical analyses were conducted and the results were compared against the developed assessment criteria. Load and excavation limits were then set accordingly.

PROJECT DESCRIPTION

Site Location and Orientations

The proposed development was a complex of multi-storey residential buildings located at Woolloomooloo, Sydney. The focus of this paper is on Building A development and its potential effects on the ED tunnel. Figure 1 shows the site location and the extent of the proposed Buildings A and B.

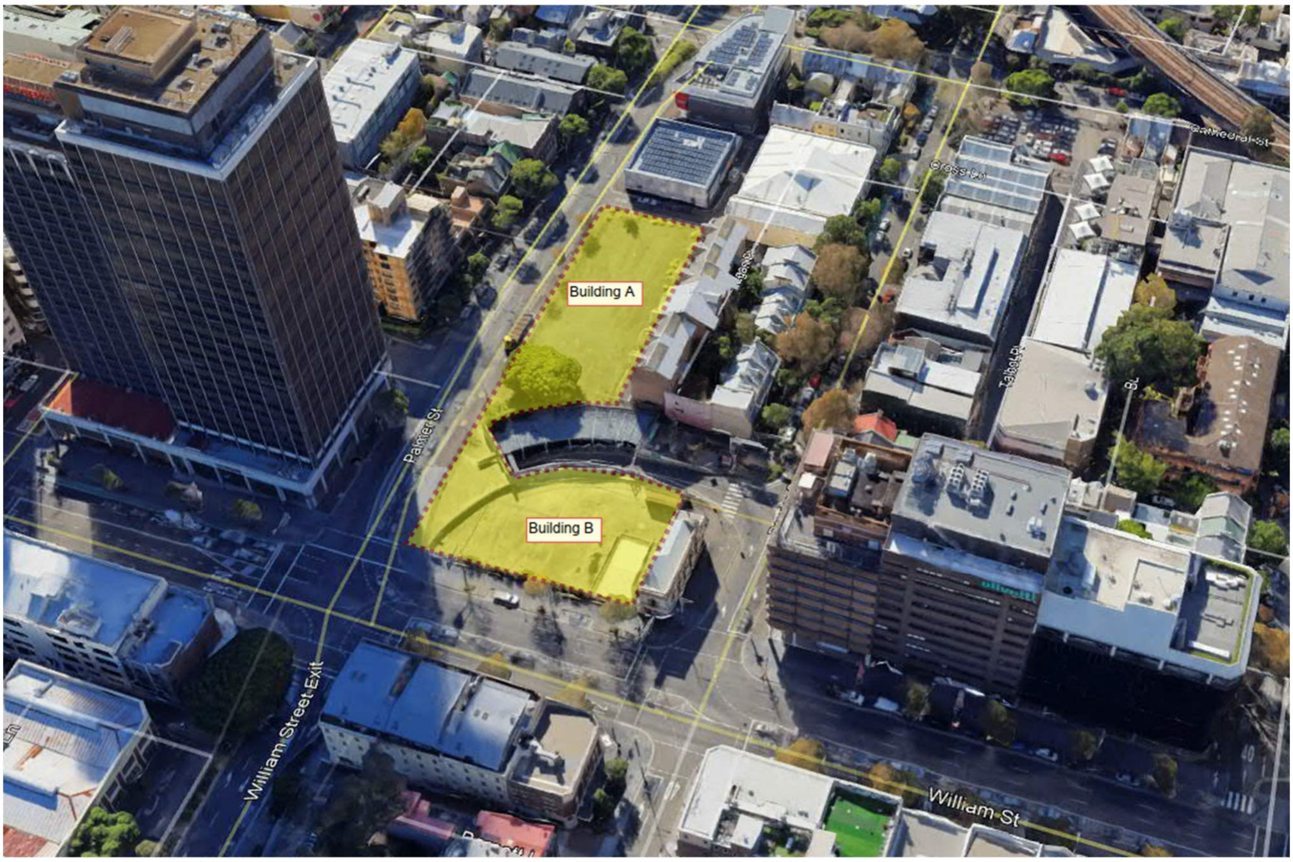

Figure 1. Site location showing proposed Buildings A and B.

Desktop Study

The desktop study involved gathering and compiling available technical information to identify gaps in the data so that further information can be sought from client and identify limitations of the study if such information is not available. Several documents including geotechnical investigation reports, as-built information for the ED tunnel and survey information have been reviewed as part of the desktop study as summarised below.

ED Tunnel Structure

Based on the as-built drawings, the portion of the ED tunnel running within the boundaries of this project is a double-decker profile with a flat crown which is supported by pattern rock bolts and steel fibre-reinforced shotcrete (SFRS) liner. The support system extends only at the crown and the curved upper corners of the tunnel profile.

The liner comprised of 135mm thick SFRS layer and a 40mm thick cover layer. The geometry indicates the liner is not a load-bearing structural element and is placed for local stabilisation purposes.

Pattern rock bolts are spaced in a 1.75 m by 1.75m grid, inclined at the corners (45 to 75 degrees to the horizontal plane) and vertical at the middle section of the crown as shown in Figure 2. The rock bolts are 23mm nominal diameter of high capacity type with 450kN minimum ultimate capacity. Corner rock bolts are pre-stressed at 200kN, and the middle bolts are passive with a plate seating load of 50kN.

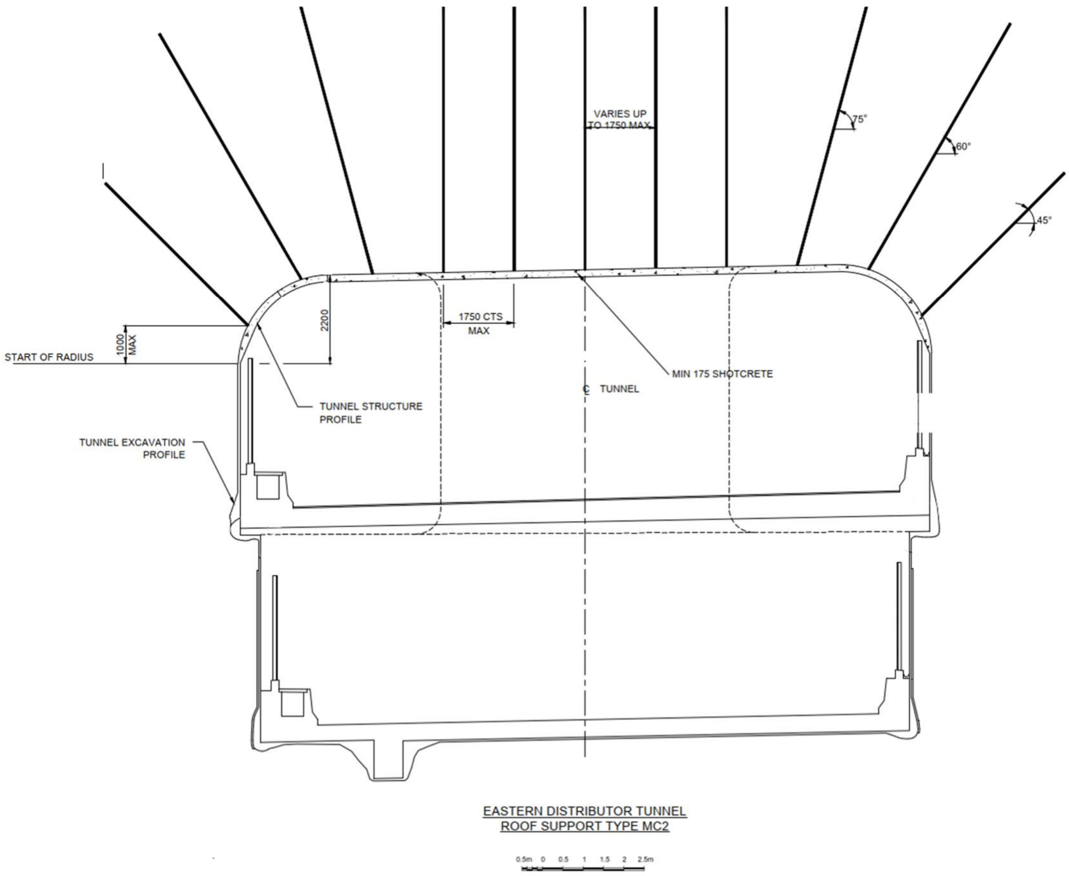

Figure 2. The ED tunnel typical cross-section showing - geometry, roof support and tunnel liner

PROPOSED ASSESSMENT CRITERIA

Considering there are limited guidelines available for the development near road tunnels, it is required to carry out site-specific assessments for the proposed development to propose design criteria or guidelines to ensure that the impact of such development on the tunnels is kept within acceptable limits.

Considering that the required information regarding the existing condition of the ED tunnel support was not available, load limits for the proposed development works were developed so that these loads have a minimal impact on the ED tunnel, without compromising its performance. To this end, analyses have been carried out to assess the loading and unloading limits and the associated deformations of the liner and rock bolts. The imposed deformations were then compared to the inferred levels after the construction of the ED tunnel (pre-development condition).

In the absence of any specific design criteria, the following assessment criteria for stress and deformations were considered:

- Nominal changes in ED shotcrete lining structural actions and rock bolt forces. It was attempted to keep the changes in the actions between pre and post-development to a minimum.
- Rock bolts differential shear displacements along the rock beddings. - change is limited to less than 1mm.

Published test data on the performance of bolting systems and the protective sleeve under shear force, indicated that no deterioration in the protective sleeve was observed for shear displacements up to 25mm (Aziz et.al 2020 and Khaleghparast et.al 2023).

These criteria were set to limit the impact on the ED tunnel to practical extents based on the sensitivity analyses.

NUMERICAL MODELLING

The numerical modelling was carried out using commercially available finite element modelling software RS2 at two cross sections selected at the Building A site. Sections are located at the ED tunnel portal (SEC-1) and 40 meters away to the South (SEC-2) as shown in Figure 3.

Geotechnical Model

Based on the available site survey, the site is located on a north-facing hill with ground level varying between RL 12 to 17m. The ground condition was assessed based on information from the geotechnical investigation report. The investigation revealed a generalised subsurface profile comprising surficial fill (0.5 to 2 m deep) overlying sandstone bedrock. Considering the available information on geological faults and dykes in Sydney CBD (Pells et.al 2004), it is probable that the southern zone of the site is affected by the Woolloomooloo Fault Zone. However, apart from recorded rock joints and beddings from the boreholes, there is no evidence in the supplied geotechnical investigations that this site is affected by any geological structures. However, geological mapping conducted during ED construction will provide more details of the subsurface conditions and the rock mass defects. This is highly recommended to acquire and use such information for the next phase of design development.

The reliability of the geotechnical model developed for the assessment may be limited due to the absence of cored borehole logs within the footprint of building A. The subsurface conditions assumed for building A are based on the interpretation of cored borehole logs obtained within the footprint of building B, using Pell's rock classification (1998). It should be noted that there may be differences in subsurface conditions between the two buildings, and as such, the applicability of the ground model presented in Table 2 could be changed.Based on the geotechnical investigation factual report supplied by the client, only one borehole is located between buildings A and B. Ground conditions inferred from this borehole indicate that Class III sandstone extends to approximately RL 0m. Actual ground conditions at the building core locations should be confirmed by cored boreholes prior to further design development.

Table 1 - Summary of Ground Model – Building A

Geological Units	Top RL of Unit*, mAHD
Fill material	Variable
Sandstone - Class II	Variable
Sandstone - Class III	12.5
Sandstone - Class II	10.5
Sandstone - Class III	6.5
Sandstone - Class II	3.5
Sandstone - Class III	1.5
Sandstone - Class II	-2

Soil and Rock Parameters

In the absence of rock defect mapping data, rock mass parameters are adopted to develop numerical models. The spacing and properties of the rock mass beddings are inferred based on the reported information in borehole logs. Based on the experience with similar rock units, the design parameters are adopted as summarised in Table 3.

Table 2 - Geotechnical Model-Material Properties

Parameter	Unit	Sandstone*	
		Class III	Class II
Unit weight	kN/m³	24	24
Effective cohesion, c’	kPa	300	500
Effective friction angle, φ’	degree	50	50
Tensile strength	kPa	40	100
Poisson’s ratio	-	0.25	0.25
Elastic modulus	MPa	1000	2000
Unit weight	kN/m³	24	24
Effective cohesion, c’	kPa	300	500
Effective friction angle, φ’	degree	50	50
Tensile strength	kPa	40	100

* *Rock classification according to Pells et.al (1998)*

Significant locked-in horizontal stresses at magnitudes beyond the corresponding overburden pressure are known within the Sydney Basin and have been experienced in the design and construction of previous major underground infrastructure projects and developments. The in-situ stress levels within the rock mass had a significant impact on the design and construction of the ED tunnel and plays a key role in this assessment as well.

For the purpose of this numerical modelling, the in-situ stress field is adopted as below:

$\sigma_H = 1.0$ MPa + 3.5 σ_v (Sandstone Class II)

$\sigma_H = 0.5$ MPa + 2 σ_v (Sandstone Class III)

The ratio of major to minor horizontal stress:

$\sigma_H / \sigma_h = 1.5$

Considering the site orientation, the major horizontal stress is applicable in the north-south direction.

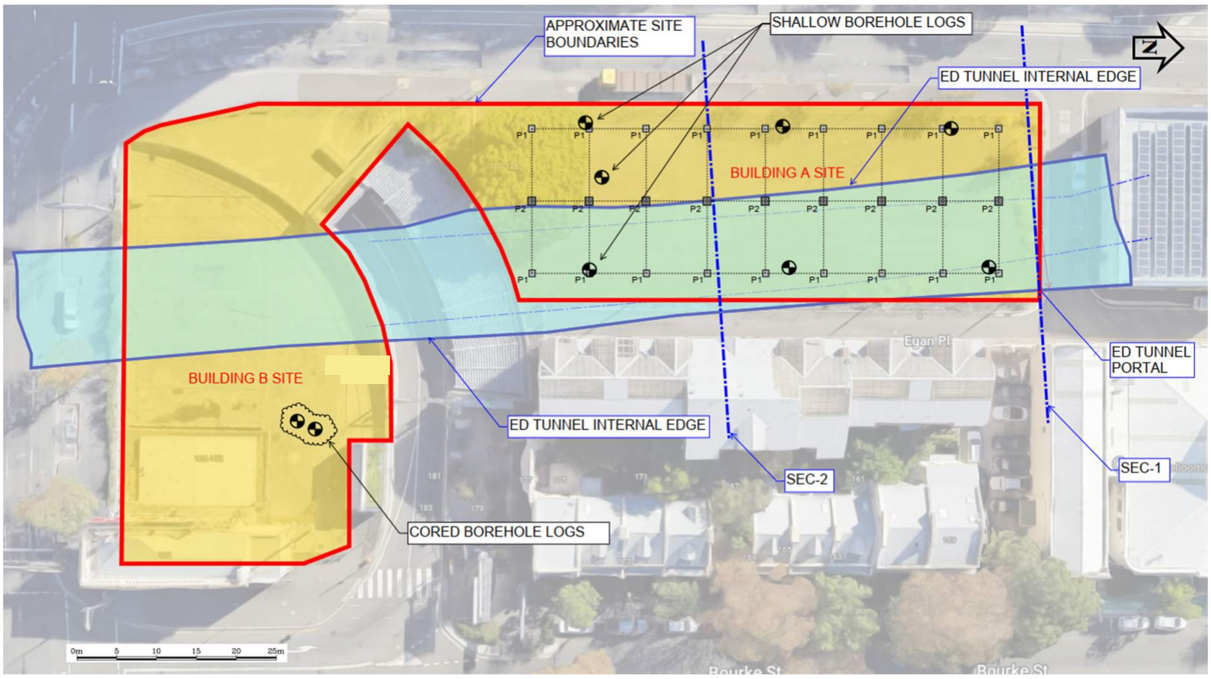

Figure 3. Location of existing geotechnical investigations, building column positions, and FE analysis sections

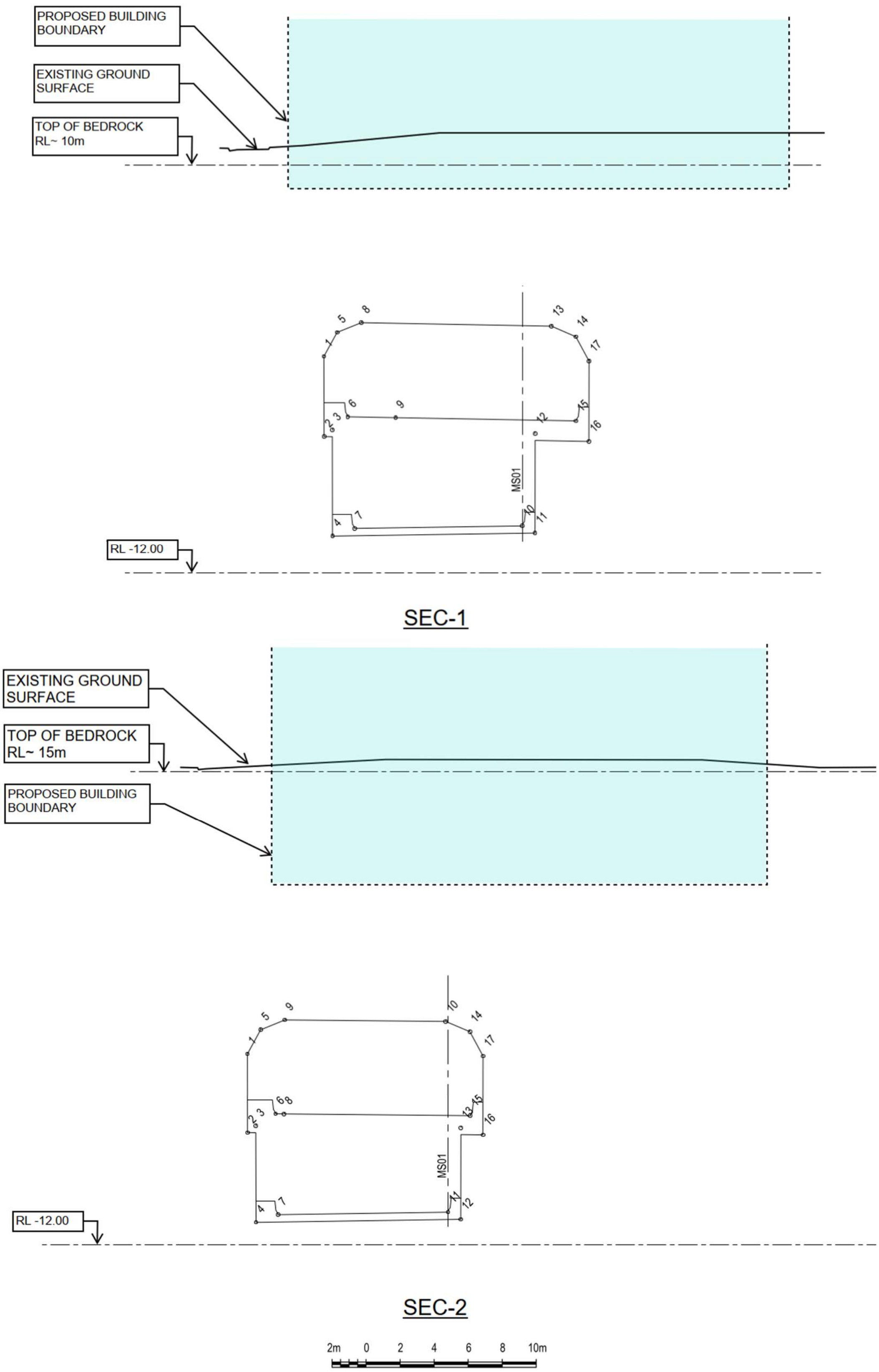

Figure 4. Site cross-sections showing the position of the ED tunnel with respect to the proposed development

Loading

For the purpose of calculating structural loads the following assumptions were made:

- A regular structure was assumed with no significant transfer floors between levels
- The structure consisted of post-tensioned floor plates and reinforced concrete columns and shear walls.
- The basement is drained and no hydrostatic forces are to be considered.
- Loads were determined based on the following usage assumptions and distributed evenly throughout the building:
  - Maximum 1 commercial level (Building A) and 2 commercial levels (Building B).
  - 1 car parking level (Building A) and 2 car parking levels (Building B).
  - 1 heavier floor plate for roof/plant.
  - Remaining levels residential.

Two different loading scenarios were considered for the numerical modelling based on the following two foundation conditions:

1. Spread loads applied to a stiff mat foundation assumed to be 600mm thick. For this scenario, four load cases were considered as tabulated in Tables 3 and 4 supplied by the structural engineer. These are correlated to different potential building heights within the development.
2. Locally concentrated column point loads representing columns at a grid of 9mx7.5m. For the column loading scenario, three building types of different heights were considered as shown in Table 4. Two types of single-column loads were considered (P1 and P2 as shown in Figure 3) representing internal and external columns. Single-column loads for each proposed building are presented in Table 4. Column loads were spread over 2x2m size single footings.

Locally concentrated loads at core/shear wall locations due to wind and earthquake actions were not considered as part of this feasibility study.

Table 3 – Surcharges for spread loading scenario (mat foundation) – Analysis Sections 1 and 2

Proposed Loading	Surcharge Load (kPa); SEC-1	Surcharge Load (kPa); SEC-2
Load case 1	30	50
Load case 2	50	110
Load case 3	75	160
Load case 4	110	180

Table 4 - Surcharges for column loading scenario (single footing) – Sections 1 and 2

Proposed Building	Single Column Load: Type-A (kN)	Single Column Load: Type-B (kN)
6-storey building	3200	4200
7-storey building	3650	4700
12-storey building	6000	7700

Modelling Stages

The construction of the ED tunnel is modelled in different stages to allow for locked-in stress relief before support installation. Considering the multi-stage profile excavation of the ED tunnel, 50% of rock locked-in stress is assumed to be released before the installation of the support system. Long-term stiffness is adopted for the liner in the last stage of ED construction. This stage is considered the benchmark for the pre-development condition. Any changes due to the proposed development in the site are then compared against the pre-development condition.

Different excavation and loading scenarios are investigated for the proposed development. The development is modelled in stages of basement excavation to the required RL, and then a range of surcharge loads is applied to model the proposed building loads.

The modelling stages adopted are as follows:

- ED Construction:
  1. Excavation of tunnel profile following the methodology indicated in the as-built drawings (upper part)
  2. Installation of rock bolts and liner
  3. Excavation of tunnel profile following the methodology indicated in the as-built drawings (lower part)
  4. Assign long-term properties for tunnel elements
- Proposed construction:
  5. Excavation to the various basement level (where applicable)
  6. Application of superstructure loads

ASSUMPTIONS

Considering all the available documents and with the acknowledgement of the missing pieces of information, the following key assumptions have been made to proceed with the assessment:

- Strata limits provided in the project Services Brief are acceptable
- Ground conditions observed in Building B are reasonably extendible to the Building A site and suitable for Building A assessments. This assumption has been made due to the absence of geotechnical data for rock conditions at the Building A site where all geotechnical boreholes were terminated at the bedrock level. No fault is located within the site boundaries. This assumption is made based on the geological faults and dykes in Sydney CBD (Pells et.al 2004).
- No existing defect is encountered with the tunnel support system. This assumption was made due to the lack of pre-development monitoring of the ED tunnel elements.
- Column loads with the single footing option were modelled two-dimensionally and it is assumed column loads are uniformly distributed perpendicular to the plane of 2D analysis.

ANALYSIS RESULTS

Liner Deformations and Structural Actions

The increment of liner deformations with respect to the ED tunnel's existing condition was found to be less than 4 mm in Section 1 and less than 2 mm in Section 2 under the studied cases. **Figure 5** shows the vertical displacement of the ED tunnel crown (edge to edge along the straight section) for different building surcharges. The change in the bending moment due to the proposed development was found negligible for all studied cases. Therefore, the proposed development under the investigated cases has minimal impact on the performance of the liner. This is considered reasonable as the SFRS liner does not play a role in the overall performance of the system, instead it assists to improve the local stability.

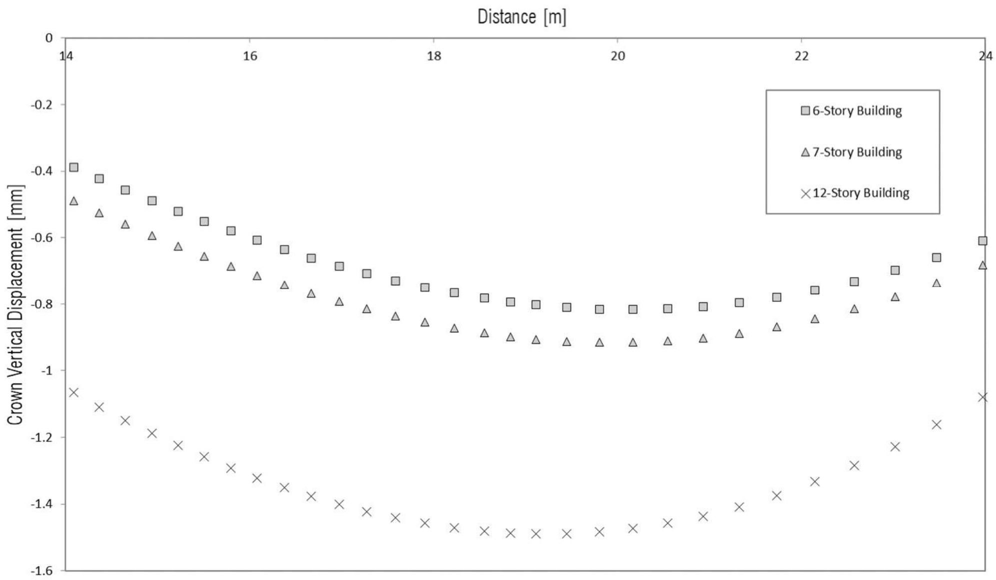

Figure 5. Vertical displacement over the ED Tunnel crown due to different building surcharges at section 2

Rock Bolts Loads

The axial load in rock bolts were investigated and it was observed that there was some load increase within the fixed length portion of the bolts, where they cross the rock bedding planes. However, the axial load in the bolt did not exceed the bolt pre-stress forces of 200 kN and 50 kN. **Figure 6**-a shows the axial force in a critical rock bolt for different loading cases. It can be seen that the increments of change with respect to the ED tunnel's existing condition are small. The largest changes happen at the intersection of the rock bolt with bedding planes.

Furthermore, a review of the shear displacement of bolts at the bedding plane was carried out. With close discussions with materials engineers on the associated durability issues, the maximum allowable shear displacement of 1 mm was adopted to reduce the risk of potential durability problems with bolts and their long-term performance. The total shear displacement before the development was in the order of 0.5 mm and additional displacement due to excavation and loadings from development was in the order of 1 mm. The total displacement is much less than the shear displacement of 10 to 15 mm required to damage the HDPE liner as reported in various laboratory test results (Aziz et.al 2020 and Khaleghparast et.al 2023).

The unloading effect of excavation depths was studied via sensitivity analyses. The results showed that excavations deeper than RL 11m may result in exceeding the allowable shear displacement of rock bolts.

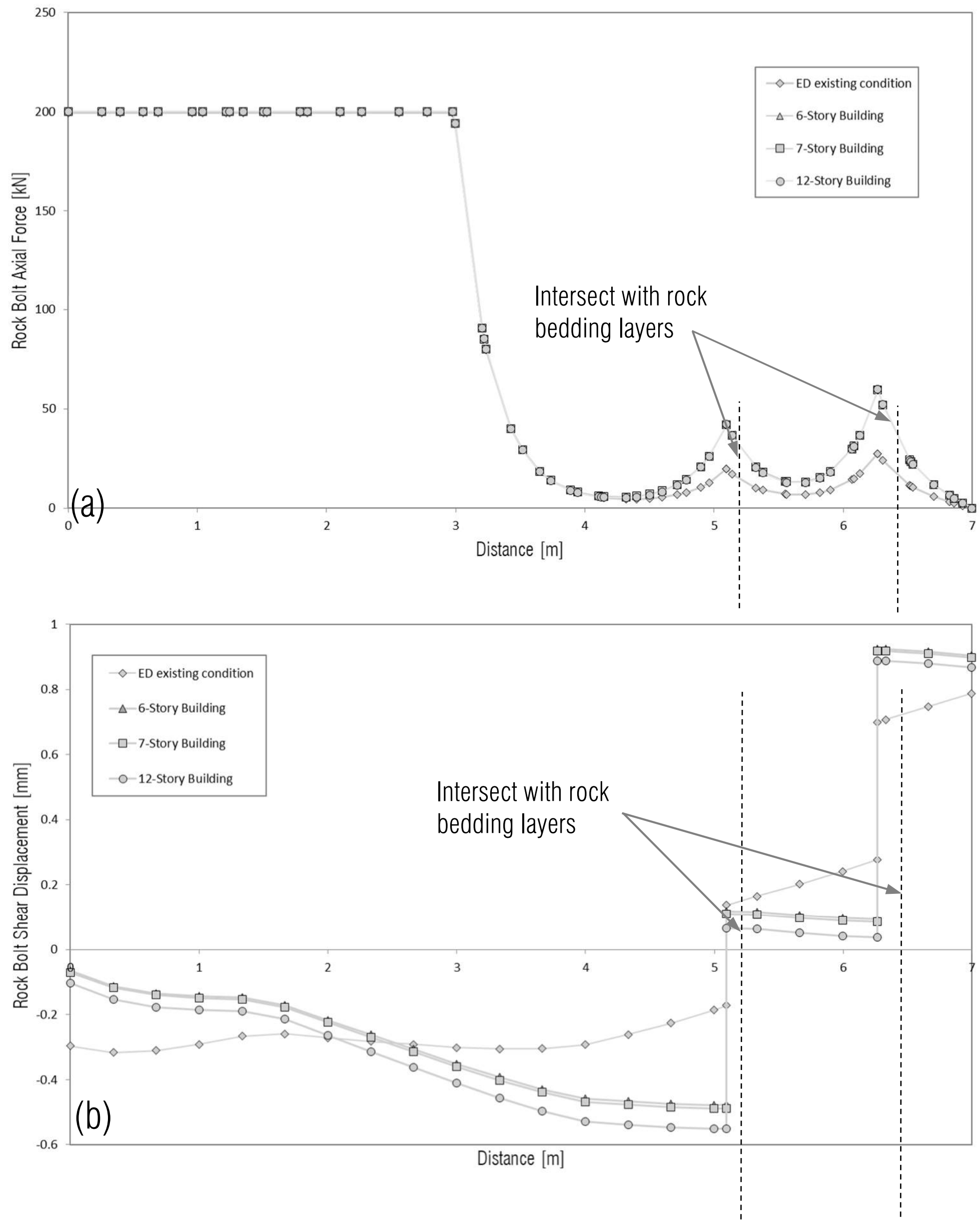

Figure 6. Axial force (a) and shear displacement (b) in one of the rock bolts.

Based on these criteria, the maximum allowable surcharge was developed for different cases as summarised in Table 4.

EXCAVATION AND SURCHARGE LIMIT

The ED tunnel easement allowed the excavation levels of up to RL 8.7m and 8.2m in sections 1 and 2 respectively. However, by considering the unloading effect of the excavation on the ED tunnel structural elements against the set assessment criteria, the maximum excavation for the proposed development was limited to RL 11m for both sections. Local excavation up to RL 10m was allowed to accommodate column footings.

Based on the assessed surcharge limits the use of pad footings with a minimum size of 2mx2m was allowed in both sections. In order to limit the stress transferred to the ED tunnel elements, a minimum footing thickness of 1m was recommended. The actual footing should obviously be designed and sized based on the local bedding material and the required structural adequacy. Column loads equivalent to a 7 and 12-storey building and spread loads of up to 70 and 140 kPa were assessed to be acceptable for the northern and southern parts of Building A, respectively.

The excavation and surcharge limits are summarised in Table 4.

Table 5 – Excavation and surcharge limits– Sections 1 and 2

Case number	Section	Excavation Limit - RL (m)	Foundation type	Surcharge Limit*
1	1	11	Mat foundation	70 kPa
2	1	11	Pad footing	7-Storey building column loads
3	2	11	Mat foundation	140 kPa
4	2	11	Pad footing	12-Storey building column loads

* *Refer to the Loading section for details*

CONCLUDING REMARKS

In general, for the feasibility study presented in this paper, the modelling, including checks of various scenarios, showed that the tunnel structure is sensitive to unloading due to the excavation of rock at the site as well as reloading during the construction of the new development. The structure of the rock mass and orientation of joints compared to the location of excavation and foundation loads is likely to have a significant effect on the stress/strain experience by the ED tunnel elements.

As discussed throughout the paper, there are outstanding works to be completed to elaborate the load limits provided in this study. The following are suggested as necessary for the detailed design stage:

1. It is strongly suggested that rock face mapping captured during the construction of the ED tunnel and the CCT as-built drawing sets be sought for and considered.
2. Actual ground conditions at the building core locations should be confirmed by cored borehole prior to further development. Investigations should allow for the determination of the structure of the rock mass and the orientation of joints.
3. Incorporating the Cross City Tunnel and ventilation geometry from as-built drawings to further review the proposed load limits.

4. A structural assessment should be undertaken of the acceptable impact of stresses and forces on the shotcrete liner, bolts and other structural elements.
5. Numerical modelling in three dimensions to account for the potential influence of the geometry of the site compared to that of the tunnels.
6. Numerical model staging should account for the construction of both the ED tunnel and CCT to incorporate effects due to locked-in stresses.
7. Detailed assessment including the updated ground conditions to be developed using details listed in points 1 and 2 above.
8. Agree on the scheme and allowable impacts on the tunnel elements with all the stakeholders.
9. Conducting pre-construction condition surveys in the ED tunnel and CCT.
10. Develop and install instrumentation and monitoring scheme.
11. The construction impacts should be considered and accounted for with appropriate construction controls to reduce the adverse impacts on the tunnel structure to the satisfaction of the stakeholders.
12. Detailed structural analysis across the site to determine structural design actions at footing level incorporating locally concentrated loads at core/shear wall locations and site-specific development requirements.

The authors wish to underscore that the prevailing expectation of zero impact on existing infrastructures by adjacent developments warrants reconsideration by asset owners and stakeholders. In rapidly evolving urban environments, this expectation increasingly appears unreasonable. To develop practical options for proposed developments that minimize the impact on existing infrastructure, more efficient and detailed analysis methods must be coupled with precise and unanimously agreed-upon assessment criteria. It is crucial to adopt a realistic approach and avoid unnecessary duplication of effort.

ACKNOWLEDGEMENT

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Barangaroo South – protecting a future rail corridor.

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ABSTRACT

The initial tunnel alignment of Sydney Metro West in 2014 was designated to pass under the completed Barangaroo South (International Tower 3) on an east-west alignment. While the tunnel alignment was later changed the tunnel protection corridor under the multi-level car park basement and building tower remains. The 7m diameter twin tunnels alignment, if they had been constructed, would have had to navigate between extensive rows of piles with around 1m offset from the sides of the TBM. The protection corridor runs from near Hickson Road down to the Darling Harbour foreshore approximately 160m. The surface topography of the sandstone rock surface dips towards the harbour. A diagram wall to rock was used around the perimeter of the site for the deep basements. There have been numerous historical wharfs constructed and abandoned along Darling Harbour at this site making the history of the site also of interest and creating another construction issue. This paper discusses the risk assessment that was undertaken for the proposed twin TBM tunnelling works and the approval process. The rail protection corridor is part overlain by a 4m deep structural concrete slab making surface access, should there be an issue requiring temporary access to the tunnel under construction from above, for all practical purposes impossible. Although the tunnels were never built the paper is important historically and for lessons learned, which can be applied to future protection corridors.

ORIGINAL SYDNEY METRO WEST ALIGNMENT

Figure 1 shows the original alignment for Sydney Metro West with a proposed tunnels running east-west under Barangaroo. This of course was later changed to the current alignment, part of which runs north-south under Hickson Road crossing the harbour just west of the Sydney Harbour Bridge.

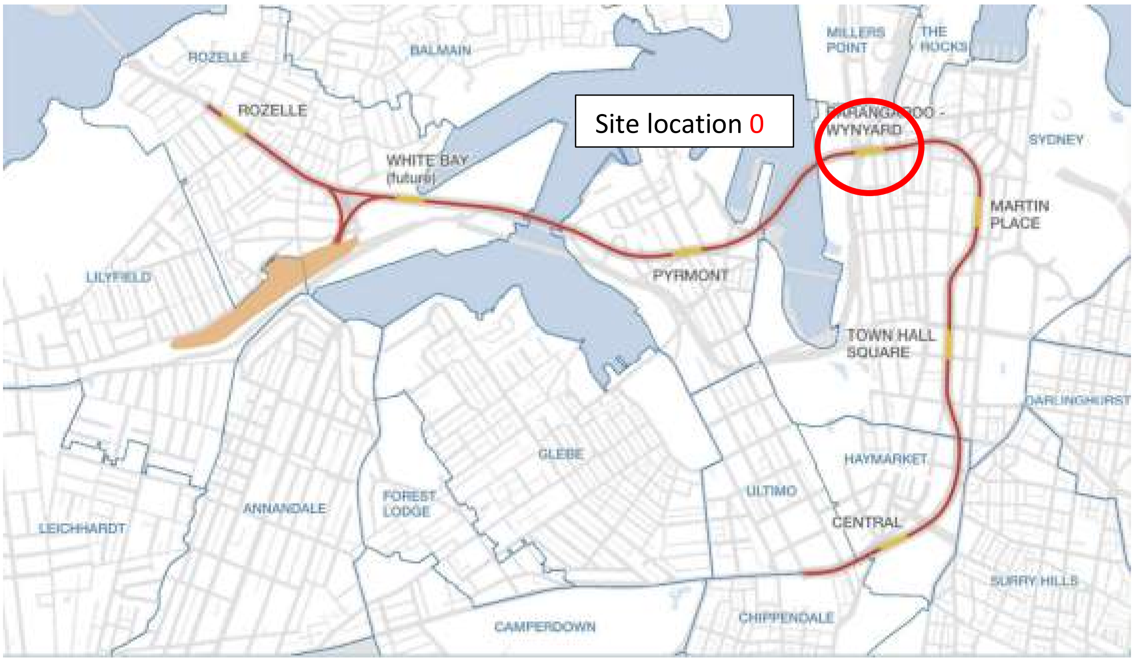

Figure 1: Superceded alignment of the Sydney West Metro – 2014

Note: refer to Transport for NSW websites for the actual Sydney Metro alignment.

HISTORY OF THE SITE

Darling Harbour has a history of marine structure development going back to the first settlement in Sydney two centuries ago. The foreshore along the waterfront has seen major changes as its development has encroached further into the waterway.

Figure 2: An aerial photograph Darling Harbour taken in 1937.

Figure 3: A more recent photograph of Barangaroo showing the proposed tunnel alignment.

The site from Hickson Road down to the foreshore has had numerous incarnations over time with numerous abandoned foundations from previous marine developments associated with this shipping berth. Typically, old timber piles and decaying concrete structures. For further project information (Wong et al, 2013)

GEOLOGICAL PROFILE

The geological strata overlying the site can generally be described as fill (Unit 1) overlying estuarine deposits (Unit 2A, as small lenses) overlying alluvium (Unit 2B) overlying residual soil and sandstone rock, ranging from Class V to I (shown as dark grey in Figures 4 to 6).

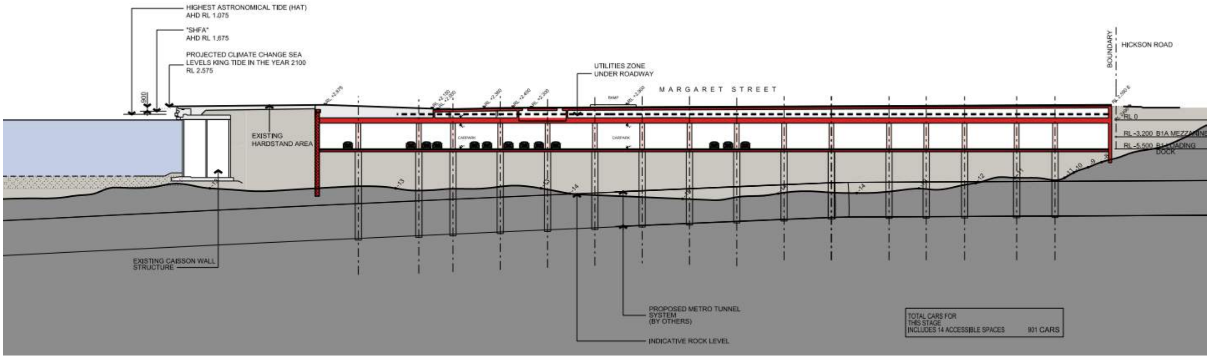

Figure 4: Long section of the tunnel along the south boundary of the site.

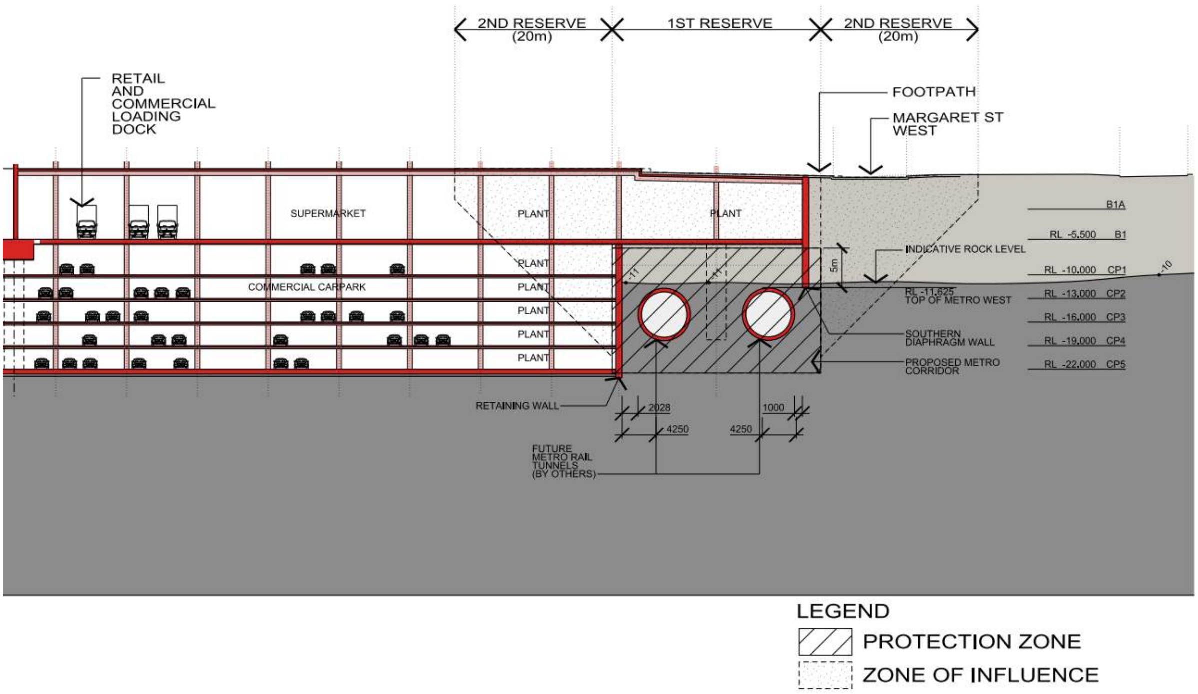

Figure 5. A typical section taken adjacent to section of deep basement.

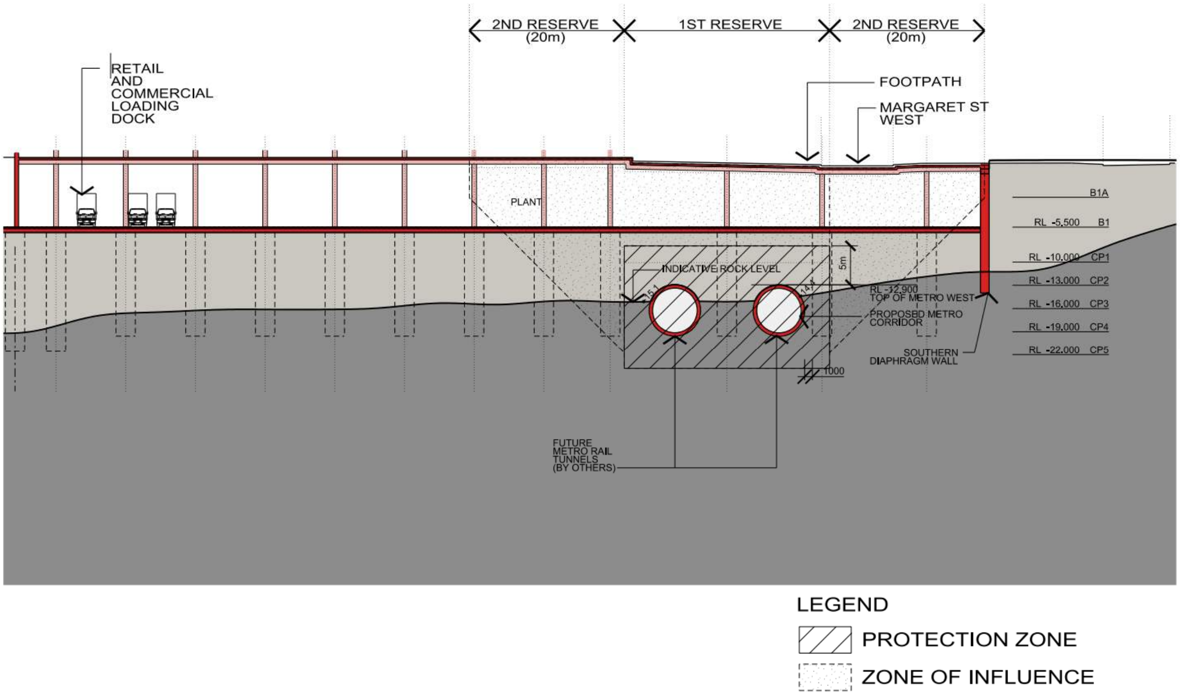

Figure 6: A section with a shallower basement adjacent to the proposed tunnels alignment.

The general scope of work carried out by Coffey, the geotechnical consultants, for the project consisted of the following:

- Verification of the anticipated stratigraphy,
- detailed assessment of marine/estuarine sediments and potential underlying residual soil,
- hydrogeological modelling,
- report and advice based on the Structural Engineers requirements,
- seismicity advice,
- acid sulphate soil potential and soil aggressivity,
- liquefaction potential of other than rock,
- material re-use assessment for use at Headland Park.

While the geotechnical data was assessed including the potential for high in-situ stresses in the rock a risk process which was carried out concluded that the highest potential risk was related to geometric set out (i.e. site survey control) and construction conformance. In each case mitigation measures were proposed, and they were generally industry standard.

CASE STUDIES

Part of the assessment process included identifying similar projects. Many of these were tunnels under older buildings, where a rail corridor had not previously been designated. In Sydney, however, at the Domestic Terminal, Sydney Airport a new 5 level multi-level car park had been constructed with the knowledge that a 10m slurry TBM would pass directly below it within 12 months of its completion (Nye, 1999). A somewhat similar situation at Barangaroo, except at the airport the piles terminated above the tunnel and in soft soil. For piles adjacent to a tunnel at the Shangri La Hotel (previously known as the ANA Hotel) building piles were very close to the unsupported sandstone rock tunnel walls with an unreinforced concrete arch (Baxter et al 1990). And again, at the Domestic Terminal, Sydney Airport, the 10m dia. slurry TBM passed with a few meters of newly installed deep bored piles supporting, and at the time under construction an elevated road viaduct (Nye, 1999).

PROTECTION CRITERIA

The proposal at the time of the assessment (and was delivered) for the basement of the building of interest included deep basements with excavations to and over the proposed Sydney Metro tunnel alignment and foundations that included bored piles, retention systems (e.g., secant piles and/or diaphragm walls) and possibly barrettes. The Sydney Metro tunnel corridor potentially would contain two tunnels with excavated diameters of 7m separated by a 7m wide pillar of rock.

Table 1. Summary of conditions guidelines (in 2014)

Protection Zone		Construction Activities	Conditions Guidelines
1st Reserve	Inside Protection Zone		Construction not permitted to directly encroach upon Protection Zone except where it can be demonstrated to the satisfaction of Sydney Metro that the encroachment will not have unacceptable structural or operational impacts on the metro corridor.
	Outside Protection Zone	Surface excavation	Engineering assessment required from developer where surface excavations are proposed directly above station caverns and crossover caverns.
2nd Reserve		Surface excavation	
		Foundations	Engineering assessment is not required if calculated bearing pressures are less than 150KPa for shallow footings and strip footings are less than 3m by 3m in plan. For all other shallow foundations an engineering assessment is required of the developer. Engineering assessment is not required from developer if loading from deep foundations (including shaft friction) is transferred to below the boundary of the influence zone. Engineering assessment required from developer where the above condition is not satisfied for deep foundations.
		Underground Excavation (e.g. tunnel/cavern construction), ground anchors and demolition activities.	Developers must demonstrate through an engineering assessment that loading from shallow foundations will not adversely impact the future Line 1 Metro.
		Geotechnical investigation and directional drilling	Assessment not required.

It is important to note that with the 1st Reserve, inside the protection zone, that penetration of the reserve was acceptable according to the guidelines *"where it can be demonstrated to the satisfaction of Sydney Metro that the encroachment will not have unacceptable structural or operational impacts on the metro corridor."* Refer to Figure 7, the dark area being 1st Reserve. The guidelines used have since been replaced by new guidelines issued by Sydney Metro and Sydney Trains from around 2017.

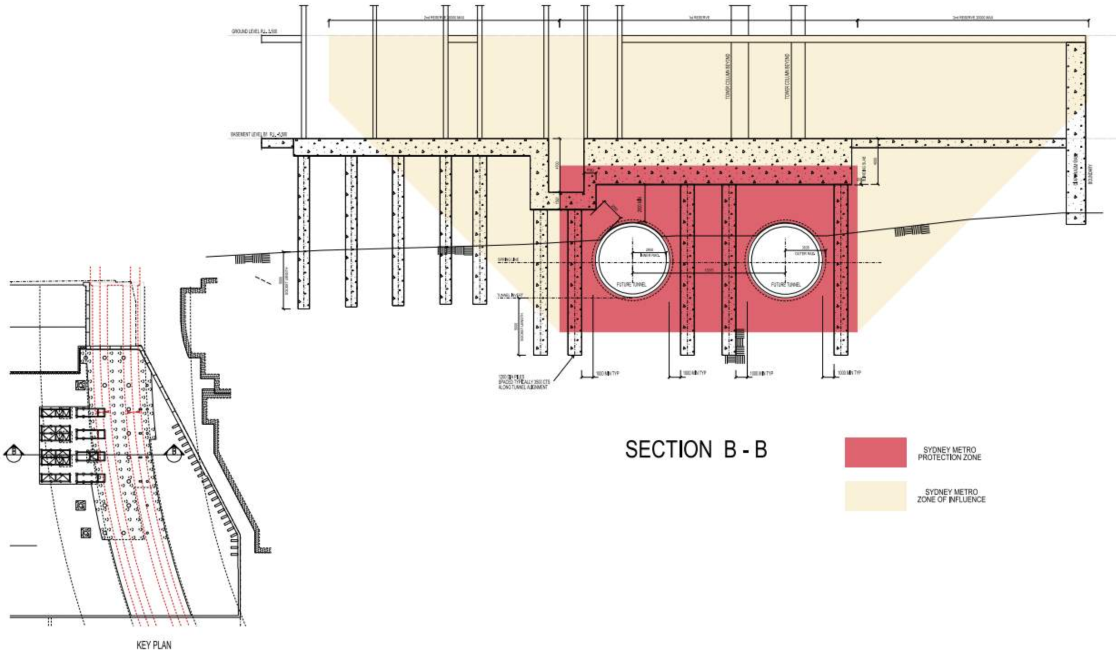

Figure 7. Tunnel protection zones at a typical section under a tower foundation

METHOD OF TUNNEL SUPPORT

It was taken for the assessment that the tunnel would be excavated by an Earth Pressure Balanced Tunnel Boring Machine (TBM). It was also expected that it would be, or would have operated through this section of ground, as a single shielded TBM where the TBM advanced by pushing off the last erected segmental concrete ring in the tail of the TBM i.e. there would be no side grippers. The tunnel support will consist only of concrete segments with the annulus between the outside of the ring and the ground created by the initial cut of the TBM and filled continuously by pea gravel/grout. The continuous grouting method relies on the grout not setting too quickly, otherwise there is a risk of grouting the TBM into the ground if it leaks forward of the tail and around the outside of the TBM and particularly if there is a TBM breakdown. Therefore, a slow setting and low strength grouted annulus behind the TBM would be used and to also prevent the full load coming onto the concrete ring initially. This may not occur until at least two tunnel diameters behind the tail of the TBM (say 14m). In sandstone rock, loading on the concrete segments would be negligible unless there is a localised rock block or wedge movement. The concrete segments, together with segment joint gaskets, provide a waterproof tunnel. Any ground loading from the fill above the tunnel crown would provide some loading which would be resisted by the passive resistance of the rock at the sides and below the tunnel invert. This will develop a uniform axial thrust in the circular ring and some minor bending moments. The concrete segments would be either reinforced with steel bars or steel or synthetic fibres. The grid of proposed concrete steel reinforced piles would in fact strengthen the rock mass, particularly across horizontal and inclined rock defects (prevent horizontal rock wedge slippage).

In the direction of vertical load, the pile elements are stiffer than the surrounding rock and would attract load to the pile, away from the rock, if any vertical downwards displacement, due to tunnelling, occurs. That is, any skin fiction at the top of the pile will be converted back into an axial load in the pile and transferred further down the pile length to a notionally new rock socket.

At the time of the assessment, it was understood that the building column loads could be around 50,000kN and the individual pile loads around 20,000kN.

The rock horizon will always be at or above the tunnel springline. This will reduce the likelihood of a squat developing in the segment lining circular profile, which could develop higher bending moments if the rock was below the springline.

The grid set out of proposed concrete steel reinforced piles would in fact strengthen the rock mass, particularly across horizontal and inclined rock defects (preventing horizontal slippage). In the direction of vertical load, the pile elements are stiffer than the surrounding rock and would have attracted load to the pile away from the rock, if vertical downwards displacement due to tunnelling occurred. That is, any skin friction at the top of the pile will be converted back into an axial load into the pile and transferred further down the pile length to the rock socket and base. The piles lengths above the rock horizon would also reinforce the fill material, although this is difficult to quantify.

RISK ASSESSMENT

Table 2. Abbreviated risk assessment table

A = Likelihood, B= Category, C = Risk (L,M,H,E(rare))

#	Description	A	B	C	Mitigation	A	B	C
1	Bulk excavation impact on surrounding rock mass	B	1	M	Highly unlikely. Moves will occur prior to tunnelling.	E	1	L
2	Temporary ground anchors	C	3	H	Design not to intersect tunnel alignment	E	2	L
3	Diaphragm wall intersection tunnel alignment	D	2	L	Construct mass concrete wall where intersection occurs	D	2	L
4	Impact on existing permanent water table	D	2	L	Diaphragm wall to retention existing water table	D	2	L
5	Loss of surface surcharge over tunnel, basement excavation	C	3	H	Minimum 2m overburden to be maintained above tunnel	C	2	M
6	Ravelling of ground at TBM face	C	2	M	Slurry of EPB TBM designed to prevent this	C	2	M
7	Risk TBM goes off alignment	D	4	M	Allow a 1m clearance in additional to pile placement tolerance	E	4	L
8	Risk building basement retention outside tolerance	D	4	M	Coordinate the Sydney Metro and Barangaroo survey grids	E	4	L
9	Stresses induced by Barangaroo basement	C	4	H	Transfer foundation loads below tunnel	E	1	L
10	Elastic movement of basement structures as tunnel passes	D	2	L	Insignificant movement	E	1	L
11	TBM breaks down under 4m slab	D	3	M	TBM design to enable to removal main bearing from within tunnel	E	1	L
12	Loss of soft ground above tunnel at face	D	2	L	TBM to be able to operate in pressurised mode	E	1	L
13	Change in water table	E	1	L	Barangaroo development unlikely to cause this	E	1	L
14	Flotation of tunnel lining	D	4	M	Around 10 m is needed in open ground. structural slab.	E	4	M

Risk assessment was made using AS/NZ 4360:2004 using Risk consequences, Likelihood and Matrix Tables. Table 2 above is a highly condensed version of the risk assessment table provided to gain the approval.

SUMMARY AND CONCLUSIONS

The proposed key elements of the structural design and construct criteria which are also applicable to future projects are as follows:

- The establishment and adoption of an integrated survey grid between the development at Barangaroo South and the Southwest Metro including the subsequent verification of Works as Executed drawings.

- The establishment of a 1 metre minimum clearance between the Southwest Metro tunnels and walls, columns or foundation elements associated with Bulk Excavation and Basement Car Parking. This is in addition to appropriate construction tolerances.

- Where required, the founding of all vertical structures associated with the building of interest a level below the zone of influence of the tunnels (or as agreed). The preliminary design shows the piles with their sockets founding below the tunnel invert.

- The piles are not isolated from the rock above the tunnel invert. Firstly, the steel reinforced bored concrete piles are stiffer than the surrounding rock which will facilitate the direct transfer of load through pile rather than into the rock. Secondly, if the rock is disturbed adjacent to the pile above the tunnel invert during tunnelling by the TBM this principle of load transference to the rock socket below still applies and will only be enhanced.

- Upon the completion of the Barangaroo South development, all the ground above the crown of the future metro tunnels under the slab spanning between the piles supporting Building C5 is retained. The minimum clearance from the underside of the slab to the crown of the future Sydney Metro tunnels will be 2m.

- The concrete segments are erected within the tail of the TBM shield. Pea gravel (followed later by high pressure grouting) or high pressure grouting alone from the within the tail shield of the TBM will fill the annulus formed between the surrounding ground the segmental lining. Grouting of the segments within or behind the tail shield of the TBM is an industry standard method of tunnel construction when using segments. Additional grouting of the ground can be achieved through cast-in or drilled holes in the segments, if required, to fill potential voids formed above the tunnel.

- Transport for NSW should ensure that when the tunnel is excavated under the building an additional level of tunnel construction surveillance is applied than used outside of the building footprint.

- The TBM can traverse beneath the load transfer slab above without the need for surface grouting during the tunnelling works and therefore no penetrations in the slab or structural elements adjacent to the tunnel are required. In the case of a 4m thick slab and depending on the building use in the basement above, this may be impractical to achieve anyway. Grouting of the ground surrounding the tunnel is in this case more efficiently carried out from within the tunnel. The integrity of the ground around the tunnel is required to be maintained to reduce lining deformation and tunnel lining flotation.

- While this alignment of Sydney Metro did not proceed, it got down to the wire with two remaining bidding contractors, this is an interesting engineering case study and if the protected corridor is not forgotten, maybe in the distant future it could be used. Perhaps even repurposed for something else? It joins a long list of abandoned projects under the Sydney CBD, for example the centrally located completed railway tunnels at both ends of the St James Station, completed in the 1920s.

It is worth repeating that although the tunnels were never built the paper is important historically and for the lessons learned and which can be applied to future tunnel protection corridors.

This projects assessment was carried out when the author was an employee of Mott Macdonald. Coffey & Partners at the time were the geotechnical consultants and Lend Lease were the developer and the structural engineers for the towers.

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