GROUND CONTROL FOR A DEEP BASEMENT EXCAVATION IN SYDNEY’S GPO FAULT ZONE

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ABSTRACT

The use of the underground space in major cities often involves complex systems with significant interaction between new excavations and existing structures such as buildings, services and tunnels. Such a complexity significantly increases when excavating in poor quality or unstable rock masses. Such ground conditions can be found in a fault zone or areas with high locked-in horizontal stress, both commonly observed in Sydney. This paper presents the geotechnical design challenges and construction outcomes of a deep excavation for a 38-storey mixed-use tower with 3 basement levels in the Sydney CBD. The challenges included excavating in the immediate vicinity of heritage listed buildings and rail tunnels built circa 1930. Two distinct excavation zones were inferred during the geotechnical site investigation, including both poor rock mass related to the GPO fault zone and good quality sandstone. An additional challenge was imposed by a 14 m deep excavation of a vehicle lift shaft with unsupported horizontal spans of 10 m, i.e. without internal support such as anchors or struts. Details of the design approaches and methods of analysis are discussed. These included a 3-Dimensional (3D) finite element (FE) analysis for prediction of ground movements and impact assessment. A structural frame model was used to simulate the effects of waler beams in 3D; hence it could be used as an input in the 2-Dimensional (2D) finite element model of the excavation. Finite Element Limit Analysis (upper and lower bound theory) was also adopted to estimate global factors of safety. A comparison between Class A predictions of ground movements developed during design and impact assessment stages and onsite measurements taken during and after excavation will be discussed. These include field data from two inclinometers, one horizontal extensometer installed in the vicinity of the rail tunnels and survey targets around the perimeter of the excavation. Photos taken during construction are presented to illustrate the challenges and successful outcome. These include some snapshots of the GPO fault zone, completed excavation with multiple ground support and ground control measures.

1 INTRODUCTION

The project development known as the ‘York and George’ is currently under construction and will consist of an architecturally iconic 38-storey mixed-use tower in the heart of Sydney’s Central Business District. The development site is an amalgamation of four existing properties at 38 – 48 York Street and 379 – 385 George Street (Figure 1). The combined site is irregular in shape and is bounded by heritage listed buildings to the north and to the south and York and George streets to the west and east respectively.

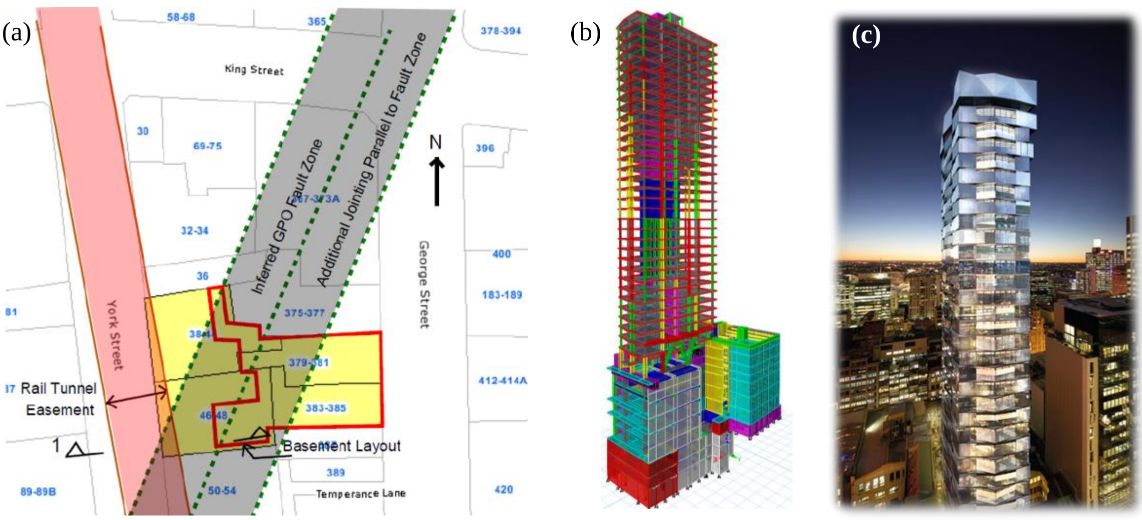

Figure 1: York and George development in Sydney CBD: (a) location plan with old basement layout (b) model illustrating the structural concept (c) artistic impression.

The development involves the demolition of the existing George Street buildings within the site (Figure 2a) but maintaining the existing heritage listed buildings on York Street, known as Carlton House and Spiden House (Figure 2b). A thirty-eight level tower is being constructed adjacent to George Street and some parts will compass over the top of the York Street heritage listed properties (Figure 1b).

Figure 2: View of heritage listed buildings: (a) on George Street - to the north and south of the site - after demolition (b) within the development site on York Street.

The feasibility of the development required the excavation of a minimum three basement levels, not only in the vicinity of the heritage listed buildings but also in close proximity to rail tunnels built in the 1930's that are used for the current North Shore and City Circle lines under York Street (Figure 1a).

The closest edge of the basement was excavated within less than 7 m away from the nearest tunnel wall extrados, i.e. a maximum 7 m wide rock pillar separates the tunnel wall from the excavation face (Figure 3), and part of the building is founded on top of the tunnels (Figures 1 and 2). To further complicate the site conditions, a 30 m wide fault zone (the GPO Fault Zone) was known to traverse the site.

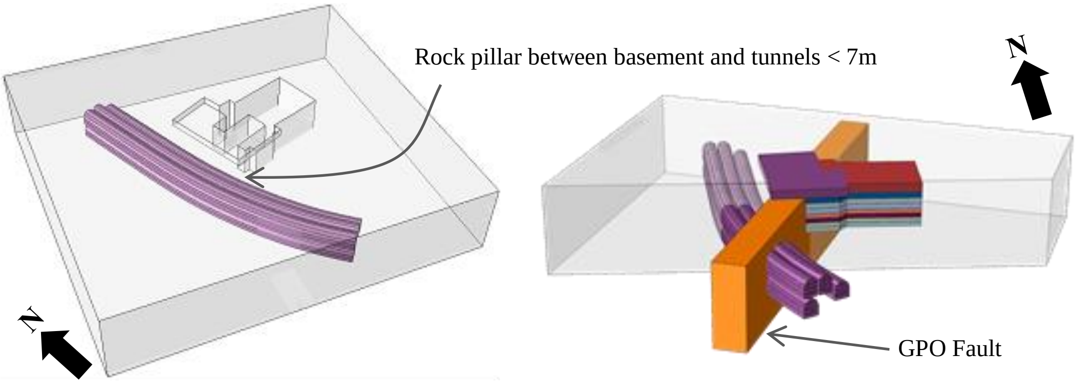

Figure 3: 3D view of the excavation and tunnels (after Oliveira et al. 2014).

In general, the geotechnical conditions in Sydney are favourable for the foundation of high-rise buildings due to the good quality sandstone at shallow depths. However, moderate lateral displacements are typically anticipated for deep excavations even in good quality sandstone due to the relief of the relatively high horizontal stresses known to exist in the Hawkesbury Sandstone. An added challenge to the proposed site was the excavation within poor rock classes of the GPO fault zone.

As a result, part of the development approval (DA) involved the establishment of an excavation protection strategy for the neighbouring buildings and the old rail tunnels. This required the assessment of the potential impacts induced by the proposed deep excavation (Class A predictions), design of ground control measures and monitoring strategy to confirm the predictions and reduce geotechnical risks.

2 IMPACT ASSESSMENT AND CLASS A PREDICTIONS

The overall geotechnical studies for the development included a geotechnical site investigation to assess the subsurface conditions and development of the geotechnical model. This information was then used to model the excavation, assess the potential impacts and to develop the protection strategy.

The impact assessment involved geotechnical analysis that considered the effects of the proposed development including basement excavation and building loads, and the effects of key ground behaviour aspects considered relevant for deep excavations in Sydney as discussed by Oliveira and Wong (2012) and Oliveira and Parker (2014). These factors will be further discussed in the paper and includes:

- Anisotropy (transverse isotropy) of the bedded sedimentary rocks
- Loading versus unloading-reloading stiffness values
- Correction of the natural stress field based on rock mass quality

2.1 GEOTECHNICAL MODEL

Based on the results of a geotechnical site investigation, four distinct geological or geotechnical zones were inferred within the subject site. An east-west section is presented in Figure 4. The rock quality is classified in accordance with the Sydney Rock Mass Classification System (Pells *et al*. 1978 and Pells *et al*. 1998), but Rock Mass Rating (RMR) values are given for comparison purposes. The groundwater level was inferred to be below the tunnel invert levels as the railway tunnels act as a subsurface drain for groundwater.

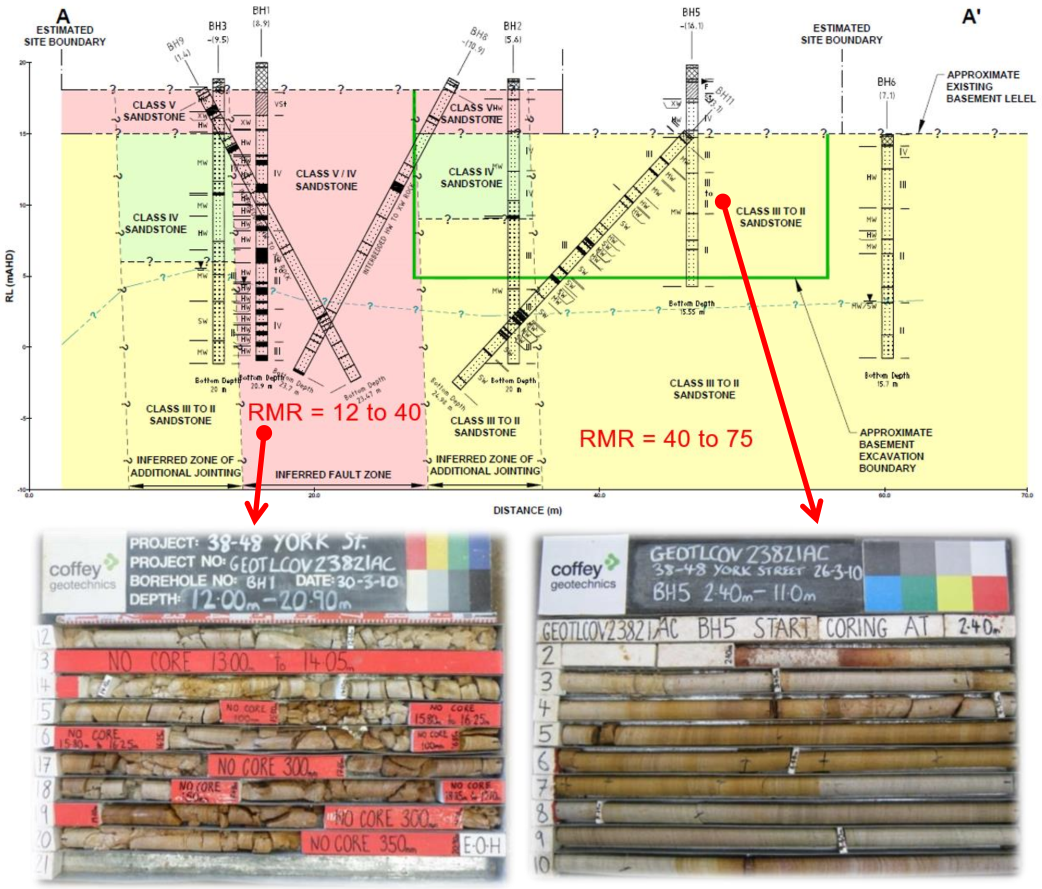

Figure 4: Inferred geotechnical model based on geotechnical site investigation from 2010-2011.

2.2.1 Design parameters

Assessment of ground movements resulting from rock excavations in Sydney are typically carried out considering an equivalent continuum approach where the effects of the rock discontinuities are not explicitly modelled. Rock mass is typically assumed as an isotropic continuum where only one elastic modulus is assigned. This was also the assumption initially adopted during preliminary modelling study for the subject project (Tey, 2012).

Besides the conceptual simplifications of such an approach, thus, ease of analysis compared to a fully discontinuum approach, one of the main motivations for the use of the equivalent continuum approach in Sydney is perhaps the well-established and tested foundation design parameters for the sandstone and shale in Sydney as originally proposed by Pells *et al*. (1978).

However, it is important to consider that such design parameters were originally developed for foundation design and that sedimentary rocks are in many cases approximately transversely isotropic material. The material behaviour is based on the preferred orientation of particles, bedding textures and bedding partings. Therefore, it is possible to account for such anisotropy assuming a transversely isotropic model where the plane of isotropy is taken into consideration. In such a model, the axis of rotational symmetry normal to the plane of isotropy is assumed to be a principal direction of elasticity and any two perpendicular directions in the isotropic plane are also assumed principal directions of elasticity. As a result, the elastic properties can be represented by two elastic components: E_3 (e.g. E_v) perpendicular to the plane of isotropy and $E_1 = E_2$ (e.g. E_h) in the plane of isotropy.

Oliveira and Wong (2012) presented a discussion on the deformation characteristics of the bedded sedimentary rocks such as those in the Sydney Basin which typically present different vertical and horizontal rock mass modulus values (E_v, E_h), including unload-reload modulus. In other words, both Sydney sandstone and shale typically present a transversely isotropic elastic behaviour.

The differences between the vertical and horizontal moduli are a result of the sedimentary deposition of the intact rock matrix combined with the differences in degree of fracturing in these two directions. Both sandstone and shale are often more fractured in the vertical direction (i.e. along a vertical scanline) than in the horizontal direction (i.e. along a horizontal scanline). This induces a horizontal rock mass modulus that is often 1.5 to 2 times the value of the vertical modulus. In addition, the presence of horizontal seams and horizontal clay-infilled joints is typically more pronounced than sub-vertical infills or seams, also contributing to a lower vertical rock mass modulus. For these reasons, Pells *et al*. (1978) noted that the modulus values suggested for the Sydney classification system are for vertical loading for foundation design.

Oliveira and Wong (2012) also stated that it is also possible to observe in some of the test results of the database used by Pells *et al*. (1978) that the unload-reload modulus may be 1.5 to 2 times stiffer than the loading modulus. This higher unload-reload modulus is associated with closure of micro-cracks within the intact matrix and joint closure. This mechanism is also often ignored in typical assessments in Sydney or at least not explicitly accounted for when selecting rock design parameters. Nevertheless, considering that the main stress path associated with excavations is an unloading of the major principal stress (horizontal), thus, a reduction of the deviator stress will have a significant impact on the excavation performance.

Based on the above assessment, Class V and Class V/IV Sandstone, including the material in the GPO Fault zone, were modelled as isotropic elastic – perfectly plastic materials following a Hoek-Brown failure criterion. This assumption was to account for the fault zone's more fractured rock mass structure. The units of Class IV Sandstone or better were modelled as transversely isotropic elastic – perfectly plastic materials with failure through either the intact rock (matrix) or ubiquitous joints representing the three main joint sets (bedding and two sub-vertical joint sets).

The vertical components of the elastic modulus, E_v, are those typically adopted for foundation design in the Sydney, which are basically values assessed under loading condition. The horizontal component of the elastic modulus, E_h, is assumed approximately 1.5 to 2 times the value of the vertical modulus. The rock model adopted for the Class IV Sandstone or better also allows for different modulus if the rock is unloaded-reloaded (E_{urv} and E_{urh}). The unload-reload moduli were assumed to be twice the value of the corresponding loading modulus. The adopted parameters for the geotechnical units used in the numerical analysis are presented in Tables 1 and 2.

Table 1: Parameters adopted for rock units.

Sandstone Unit	**σ_{ci} (MPa)**	**m_i**	**GSI**	**Intact rock ϕ (deg)**	**Intact rock *c* (kPa)**	**σ_{ti} (kPa)**	**E_v / E_{urv} (MPa)**	**E_h / E_{urh} (MPa)**
Class V	2	17	45	-	-	-	150/150	150/150
Class V/IV	4	17	45	-	-	-	200/200	200/200
Class IV	4[1]	17[1]	100[1]	49	780	235	350/700	700/1400
Class III/II	7[1]	17[1]	100[1]	52	1200	700	900/1800	1500/3000

1) Hoek-Brown parameters for the intact rock, i.e. a GSI = 100, have been used to estimate the corresponding friction angle, ϕ, cohesion, *c*, and tensile strength, σ_{ti}, of the matrix within the anticipated range of confining stresses. Rock defects have been modelled through three sets of ubiquitous joints.

2) A Poisson ratio equal to 0.25 was adopted for Sandstone units Class V and Class IV/V, and a value of 0.2 was adopted for Class IV and Class III/II Sandstone.

Table 2: Parameters adopted for joint sets.

Defect	Dip Direction (º)	Dip (º)	ϕ[1] (º)	c (kPa)	Dilation angle (º)	σ_t (kPa)
Bedding	0	0	35 / 32	10	6	0
NNE	110	90	35 / 32	10	6	0
ESE	20	90	35 / 32	10	6	0

1) First value corresponds to joints within Class III/II Sandstone and second to joints within Class IV Sandstone.

For analyses where a transversely isotropic elastic-plastic model was not available, the most predominant parameter related to the failure mechanism under assessment would be selected. For example, for the design of the soldier pile walls later discussed in Section 2.3 the unload-reload horizontal modulus would be predominant. However, for the Class IV Sandstone it was considered appropriate to adopt the horizontal loading modulus to account for the effect of the existing footing loads.

2.2.2 In-situ stress

A number of studies have demonstrated the existence of the large horizontal stresses that are locked into the sandstone rock in Sydney. The major principal stress, σ_1, is approximately horizontal with a trend of 20º from the true north (i.e. NNE) and approximately oriented in the same direction of some of the major fault zones observed in Sydney, such as the GPO Fault Zone.

Without site specific data, the in situ stress condition in Sydney have been often estimated from the equations proposed by Pells (2002) as represented by the solid blue line in Figure 5 and compared against some stress measurements in the Sydney Metropolitan area. It can be noted that this relationship represents approximately the average of the measurements and it is close to a stress measurement near to the development site shown by the red square.

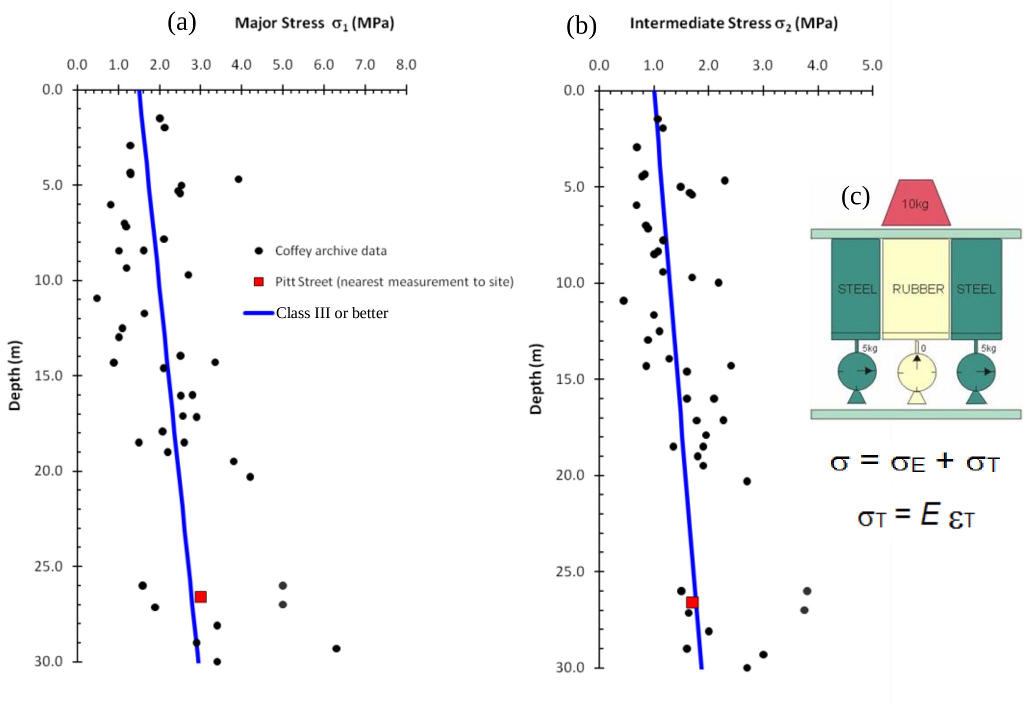

Figure 5: Principal stress database used in analysis (after Oliveira et al. 2014): (a) Major (b) Intermediate or minor horizontal (c) Analogue model for correction based on rock quality (after Nemcik et al.; 2006; Oliveira and Parker, 2014).

Despite the successful application of the relationship above, it is important to note that using a single relationship would ignore the effect of rock mass quality which may play a significant role in the amount of tectonic stresses that are locked in the rock. As a result, when building a geomechanical model, the geotechnical engineer needs to account for the inter-relationship between stress and ground conditions, i.e. rock mass quality – fracturing, strength and stiffness (Oliveira and Wong, 2012), particularly for a site with distinct rock mass qualities. The correction procedure is described in details in Oliveira and Parker (2014) and it is based on the simple analogy presented in Figure 5c taking into account estimates of the tectonic stress, $\sigma_{T.}$

As presented in Oliveira *et al* (2014), the stress relationships adopted in the current assessment were:

- σ_1 = 1.5 MPa + 2σ_v for Class III or better
- σ_1 = 0.8 MPa + 1.2σ_v for Class IV
- σ_1 = 0.3 MPa + 1.2σ_v for Class IV/V (Fault Zone)
- σ_2 = 0.3 MPa + 1.2σ_v for all zones

The major principal in situ stress condition initialised in the numerical model is shown in Figure 6. The lower stress within the fault zones can be noted as described by the analogy of Figure 5c.

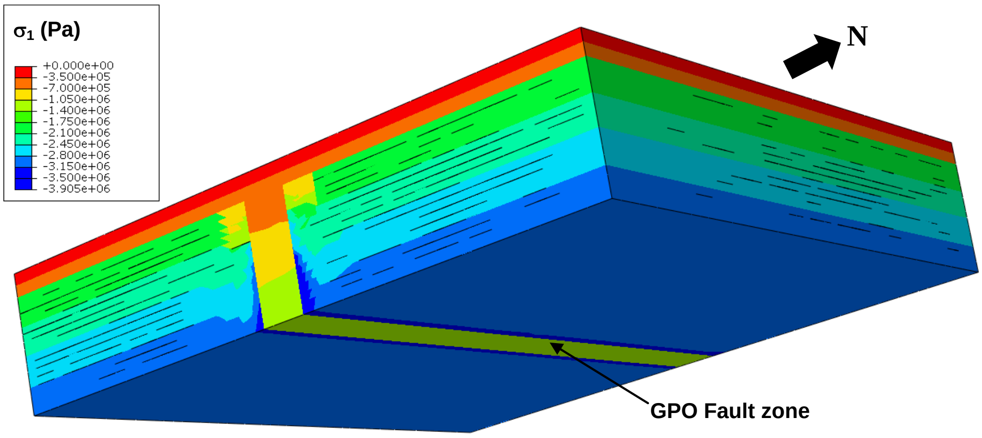

Figure 6: Major principal stress applied in the model (after Oliveira et al. 2014).

2.2.2.1 Effect of existing tunnels

Besides the existing heritage building basements, the rail tunnels construction methodology would also had an impact in the assessment as it affects the stress condition in the vicinity of the proposed excavation. As a result, reasonable steps were taken to reduce the inherent uncertainties related to the old tunnels and initialising the model with the effect of the tunnels.

A thorough literature search was carried out to identify the construction methodology of those old tunnels. Unfortunately, no official document from the Rail Authority could be found describing their construction. However, two papers published by Fraser (1930) and Humphries (1931) provided some useful information. These papers describe some of the methods of tunnelling as used in the construction of the City of Sydney underground railways which include the City Circle and North Shore lines.

According to Fraser (1930), for the area under assessment, single track tunnels have been excavated by drill and blast techniques using the top heading and bench method. Similar methodology was applied to double track tunnels with some differences in the temporary support (timber lagging). Due to a number of superimposition of tunnels, interaction of the permanent lining, transition from double track tunnels to single track and vice-versa, all four tunnels were likely excavated and lined simultaneously, i.e. at the same time.

Figure 7: Photos showing timber lagging, curved rails and concrete lining used in double track tunnels in the City of Sydney railway tunnels (Fraser, 1930).

The arch tunnels were temporarily supported by timber lagging and/or old 80 lbs/yd rails as illustrated in Figure 7. Fraser (1930) also states that the arch tunnels were all-round concrete lined, including the bottom beneath the roadbed. This lining was carried out in four stages: footings to sidewalls, sidewalls, arches then roadbed. The concrete pouring lengths of theses stages varied from 5 to 10 m before the next one was poured. Therefore, considering the excavation method using drill and blast, the timing for temporary support and concrete lining it seemed reasonable to assume that all ground movements and stress redistribution due to tunnelling were completed by the time the lining was built and hence the concrete lining was essentially unloaded after construction of the tunnels. Such construction sequence was then included in the 3D model.

2.2 PREDICTIONS DURING DEVELOPMENT APPROVAL (FLEXIBLE SHORING)

During development approval (DA) studies, it was assessed that a flexible shoring system could be a feasible ground control solution for the excavation. This would involve a pattern of rock bolts and/or soil nails with shotcrete facing. However, such method would have to allow for potential underpinning of the existing building footings on the excavation boundary which would in turn result in a slow excavation process. At early stages of the project, there were uncertainties relating to the type of shoring system and ground control measures to be adopted (unknown at the DA stage, i.e. yet to be tendered). It was then considered appropriate to adopt such a solution for the purpose of DA impact assessment. This would provide an upper bound estimate of induced ground movements and likely impacts when compared to more robust solutions such as anchored piles within the fault zone. As a result, a 3D numerical analysis was performed using the commercially available Finite Element Method (FEM) computer program Abaqus\FEA (Version 6.12.3), assuming a flexible shoring system by using rock bolts and soil nails with a shotcrete face. The results are provided in Figure 8.

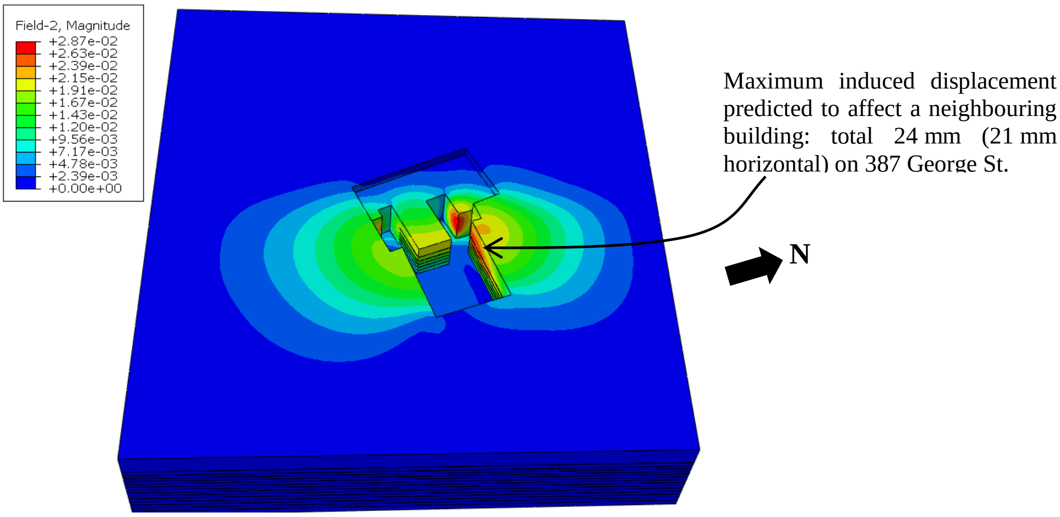

Figure 8: 3D model total displacements predictions (displacements from existing basement and tunnels removed).

2.2.3 Damage risk classification

A damage risk classification for the surrounding existing buildings was carried out based on the predicted excavation induced ground movements. The classification was based on industry accepted classification methods such as CIRIA PR30 (1996) and Boscardin and Cording (1989) that allows for the assessment of induced horizontal strain. The use CIRIA PR30 (1996) was also a request from the structural advisors and peer reviewers of one of the neighbouring buildings.

The CIRIA PR30 (1996) classifies the damage risk in four categories based on the assessed maximum settlement and settlement distortion/slope. Risk category 1 is described as negligible with unlikely superficial damage. Risk category 2 is described as a slight risk with possible superficial damage of unlikely structural significance, i.e. aesthetic only and of easy repair. Risk categories 3 and 4 are described as moderate and high respectively, with possible structural damage. However, it is important to note that the CIRIA document only focuses on settlement values (absolute and distortion) as it was developed for the assessment of tunnelling effects in soft ground. For excavations in Sydney sandstone, most of the ground movements are sub-horizontal. Therefore, the CIRIA PR30 risk classification alone may not be appropriate. A more appropriate classification method should also account for the assessment of induced horizontal strain, for example that proposed by Boscardin and Cording (1989).

Based on the maximum predicted settlement and angular distortion (less than 1:500), an assessed Risk Category 1 (Negligible) according to the CIRIA PR30 (1996) was selected. With respect to estimated horizontal displacement, the maximum predicted value was approximately 21 mm (Figure 8). Despite such relatively high value, it is differential movement, or more importantly, distortion that induces damage. The assessed horizontal or tensile strain was assessed at approximately 0.14% (1:700). Based on this additional criterion, the damage risk could potentially be within a Risk Category 2 (Slight) with possible superficial or aesthetic damage. This would unlikely cause adverse impact to the structural integrity of the neighbouring building. In other words, minor impact to non-structural elements such as finishes and partition walls may occur in the form of cosmetic cracking which can be easily remediated and therefore typically considered an acceptable risk.

Although the 3D model used is the same as for the buildings, details regarding the potential impact on the rail tunnel lining will not be presented here. The reader is referred to Oliveira et al. (2014) for a detailed description of the tunnel impact assessment.

2.3 POST-TENDER GROUND CONTROL STRATEGY

2.3.1 Anchored Soldier Piles

During tender, the successful building contractor decided to reduce the construction program risks associated with a flexible shoring within the GPO fault zone. The main concern with the flexible shoring system was that it could slow the excavation program due to the potential need for foundation underpinning. As a result, the designers and the contractor preferred to adopt a soldier pile wall solution within the GPO fault zone using temporary ground anchors (where possible) such that the material underneath the existing foundations would be confined during excavation. Although a full design of the shoring and prediction of the pile behaviour was carried out before construction, it was considered unnecessary to update the 3D analyses. It was expected that in areas of short horizontal spans, 3D effects would be predominant, and in areas of poor rock (fault zone), the soldier pile wall would potentially reduce or control ground movements such that the 3D analysis carried out during DA stage would provide an upper bound limit.

The soldier pile wall was designed with 600 mm bored piles spaced at approximately 1.2 m to 1.5 m centre to centre and rock socket varying from 1.5 m to 2 m. The exceptions were piles under columns where larger diameter piles with deeper rock sockets were adopted. Three to four rows of ground anchors were used where possible, with the exception of the vehicle lift shaft area were a clear span of 10 m was necessary. Initially 4 x 15 mm strand anchors with a lock-off load of 480 kN (i.e. 80% of the allowable capacity) were adopted at a spacing of 2.4 m to 3 m horizontally, i.e. every second pile, and connected by temporary waler beams. The ground anchors were later installed at every pile with no walers which allowed a reduction to 2 x 15 mm strands with half of the lock-off load.

The commercially available 2D FEA analysis program Optum G2 was selected for the design of the soldier pile walls. Optum G2 is a comprehensive finite element program for geotechnical stability and deformation analysis. It uses state-of-the-art numerical algorithms that lead to significant efficiency and robustness. One of the main advantages of Optum G2 is the availability of Finite Element Limit Analysis which provides an efficient means of assessing the stability of geo-structures by means of factors of safety without the need for exhaustive load-displacement analysis. This approach has the significant advantage of identifying failure mechanisms through the concepts of upper and lower bound limits. In other words, the most likely failure mechanism is identified as part of the analysis without the need for predefining it as would be necessary for Limit Equilibrium analysis.

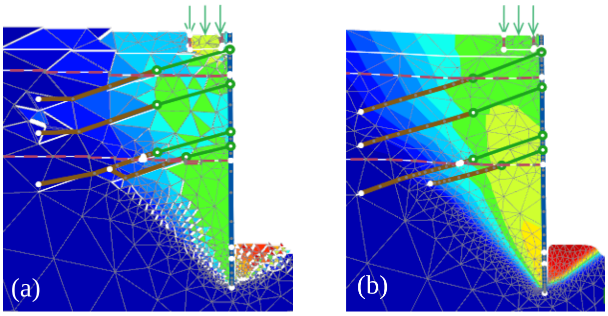

Figure 9: Global failure mechanism assessed using FE Limit Analysis with mesh adaptivity (note mesh refinement near failure surface): (a) Lower bound Fos = 2.48 (b) Upper bound FoS=2.52.

Figure 9 presents an example of the output for both Lower and Upper bound limit analyses carried out as part of the design. This included all structural elements for the piles and ground anchors. A FoS=2.5 was assessed for this case.

2.3.2 Propped Soldier Piles

As previously mentioned, the design of the vehicle lift shaft area imposed a particular challenge. A clear horizontal span of 10 m was required through the entire depth of the excavation, to allow for the lift. As a result, no internal props could be used in the permanent case. As the vehicle lift shaft was also located near the site boundary, permanent anchors were not permitted. The alternative was to design robust walers to span across the required length and assist in controlling the pile deflections during excavation (Figure 13a). Four rows of 610 UB 125 waler beams were designed to span over 10 m in the vehicle lift shaft area and supported at the opening by permanent universal columns props.

The horizontal stiffness provided by this scheme was estimated by modelling the structural frame formed by the walers and end props. The horizontal stiffness of waler was then modelled in the geotechnical FE as fixed anchor elements (Figure 10).

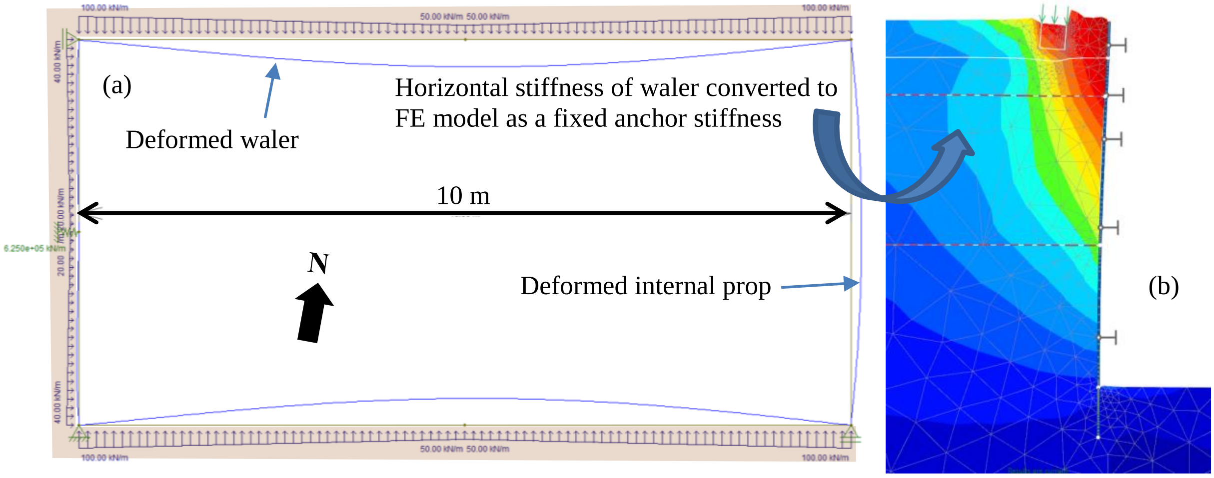

Figure 10: Design of vehicle lift shaft excavation with 10 m span: (a) Structural Frame Analysis (b) FE geotechnical model.

The design of the soldier pile wall provided a second round of ground movement estimates which could be used to confirm the performance of the piles. This was then compared to the measurements from the inclinometers installed within the GPO fault zone.

3 CONSTRUCTION PERFORMANCE

A number of monitoring points were adopted as part of the excavation protection strategy as shown in Figure 17. These included:

- Two inclinometers located within the GPO fault zone.
- One horizontal extensometer installed in the vicinity of the rail tunnels
- Survey targets along the perimeter of the excavation and neighbouring building walls.

Excavation commenced in October 2015 and completed April 2016. Figures 11 to 14 depict some of the construction details. They show the rock condition within the GPO fault zone with significant bleaching along rock defects and clay infill (Figure 11 and Figure 12), the vehicle lift shaft excavation (Figure 13a), the zone of increased fracturing near the main fault zone (Figure 13b) and good quality sandstone on the George Street side (Figure 13c).

Figure 11: View of the first stage of excavation within the GPO fault zone.

Figure 12: View of the second stage of excavation within the GPO fault zone.

Figure 13: Details of the competed excavation: (a) vehicle lift shaft within GPO zone (b) zone of additional fracturing induced by the GPO fault (c) good quality sandstone on George Street side.

Figure 14: View of the completed excavation. Note: Spiden House on the left (with vehicle lift shaft), excavation around York Street heritage buildings, Carlton House on the right with zone of increased fracturing at the end of soldier pile wall.

Figures 15 to 18 present a comparison between the Class A predictions carried out at different stages of the project prior to excavation and the measurements obtained at completion of excavation. Figure 15 presents the measurements at inclinometers 01 and 02. Inclinometer 02 was located at Spiden House within a pile near the vicinity of the tunnels as a protection measure to the rail lines. The main orientation of the readings was East-West. During the soldier pile wall design stage, no analysis was carried out for this section as it was considered that the short horizontal span would govern the movements (i.e. 3D effects). As a result, the previous 3D predictions carried out during DA stage was considered satisfactory which was later confirmed as illustrated in Figure 15a. Inclinometer 01 was located at the Carlton House excavation side, immediately behind the wall to capture North-South movements. The inclinometer was originally installed within a pile but damaged during drilling of the ground anchors. A second inclinometer was installed behind the piles. A 2D section was re-analysed as part of the soldier pile wall design, confirming the improvement provided by the soldier pile wall with reasonable comparison with the actual measurements.

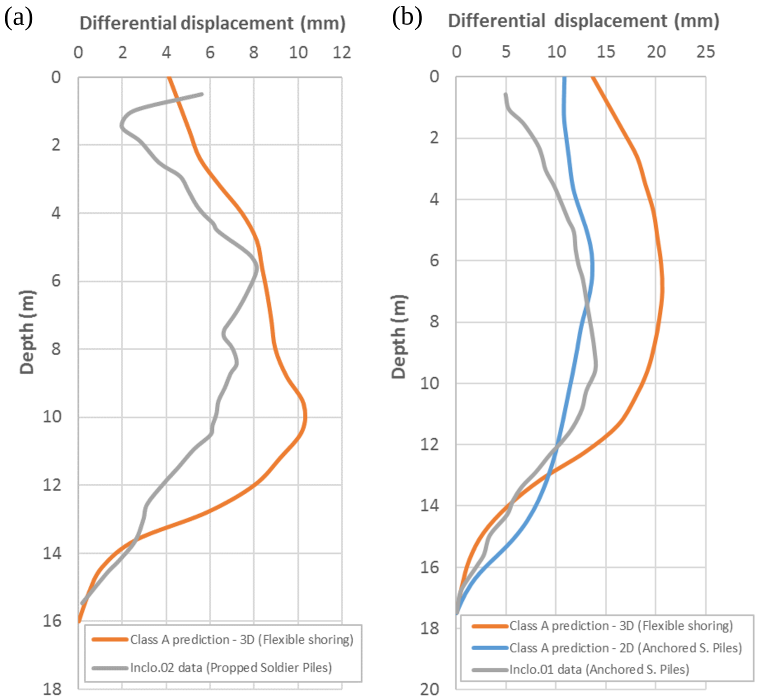

Figure 15: Comparison between Class A predictions at inclinometer locations and measurements: (a) Soldier Piles within the Spiden House (b) Soldier Piles within the Carlton House.

As presented by Oliveira *et al*. (2014), the excavation induced ground movements on the extrados of the tunnel linings was predicted to be less than 4 mm. To confirm the Class A predictions and protect the rail lines, a horizontal extensometer was installed from the rock excavation face towards the rail tunnels. As a result, the propagation of excavation induced ground movements within the rock mass could be confirmed. As presented in Figure 16, the extensometer data reasonably confirmed the predicted behaviour with a 1-2 mm difference (on the safe side). It is important to note that the maximum ground movement as predicted by Oliveira *et al*. (2014) was at a different location that could not be monitored from within the excavation.

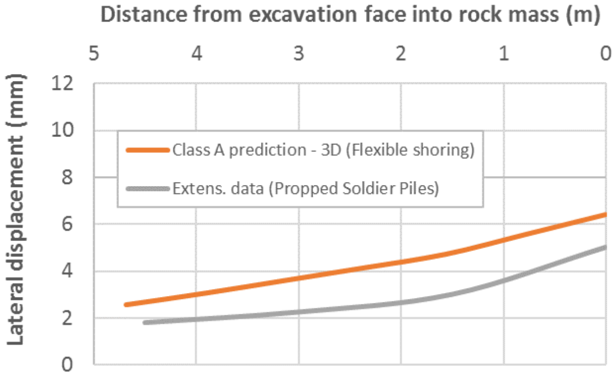

Figure 16: Comparison between Class A predictions at extensometer location and measurements.

Figure 17 presents a comparison between the 3D predictions and the survey target measurements. It shows a satisfactory excavation behaviour prediction with measurements generally between 50% and 100% of the Class A predictions but with a significant number between 75% and 100%. The only exception was the south-west face where some of the survey measurements were deemed in error. There is also good agreement in relation to the overall orientation of the ground movements. As a result, the predictions could be classified as "Fair" to "Excellent" in accordance with the Prediction Quality Classes proposed by Morgenstern (2000).

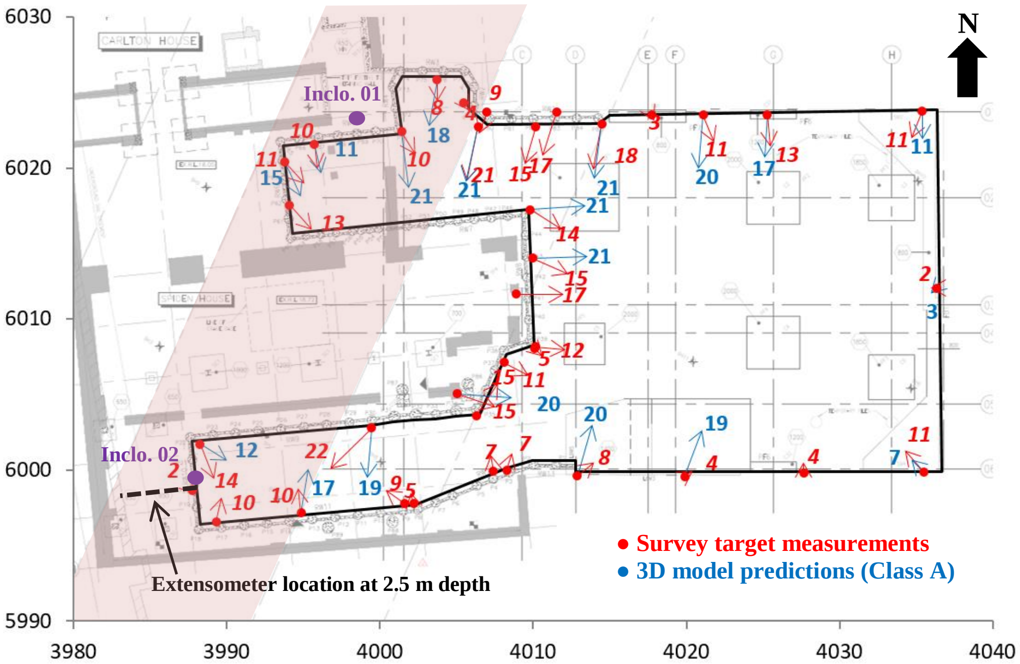

Figure 17: Comparison of the Class A predictions and survey target measurements.

Figure 18 shows a comparison between the survey target measurements, the Class A prediction ranges and the typical rates of movement observed in Sydney.

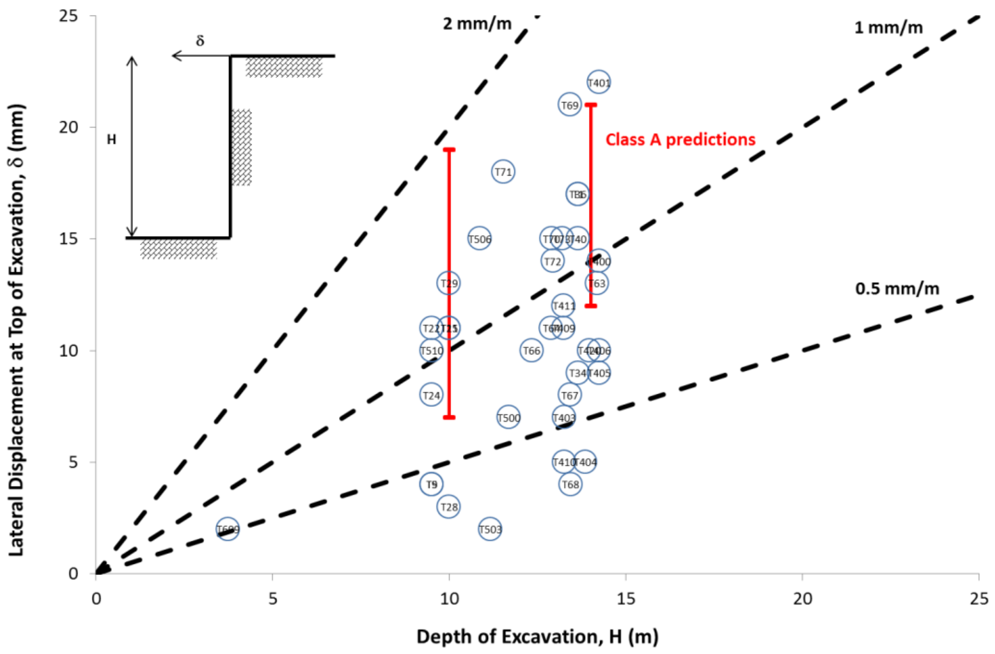

Figure 18: Comparison between survey target measurements and typical rates observed in Sydney.

It is important to note that the measurements above were taken after the excavation was completed but while the ground anchors were still stressed. As a result, some further movements particularly within the GPO fault zone could be expected to occur upon construction of the basement slabs and decommissioning/de-stressing of the anchors.

3.1 CRACKING EVENT

As discussed in session 2.2.3, the damage risk was classified as Risk Category 2 (Slight) with possible superficial or aesthetic damage. This was considered unlikely to have an adverse impact on the structural integrity of the neighbouring buildings.

During construction, one cracking event was notified to the contractor. Minor defects were observed within the internal walls in some locations at 387 George Street. The main bulk excavation had been completed approximately 4-5 weeks prior to the notification, with an additional excavation for a new passenger lift at 387 George St completed about 2-3 weeks prior.

All survey target and vibration readings were within the prescribed limits in accordance with predictions. The observed cracks were typically less than 3 mm in width and in non-structural elements. Therefore, they were not considered to be of significant structural concern by the contractor and its structural consultant and designer.

It is interesting to note that visual observation of the outside face of the boundary wall of 387 George St indicated no noticeable signs of cracks and most of the damage was internal to the building. This and the measurements seemed to indicate that it was indeed the horizontal or tensile strain that governed the potential damage. As previously discussed a damage classification purely on CIRIA PR30 (1996) would not capture this mechanism.

4 CONCLUSIONS

A case study has presented on the development of a deep excavation project from the geotechnical assessments for DA stage through to detail design and construction monitoring. A number of lessons were learnt through the project with the following key points:

- The staged development of the excavation protection strategy proved successful with realistically safe predictions that have been confirmed during construction.
- The combined risk classification based on CIRIA PR30 and Boscardin and Cording (1989) proved to be consistent with observations. One cracking event was notified to the contractor. The observed damages were superficial and aesthetic in nature, which was considered consistent with a Risk Category 2 (Slight).

- The main key ground behaviour factors assumed relevant to capture the excavation performance proved to be satisfactory. These included the following assumptions:
  (a) Anisotropy (transverse isotropy) of the bedded sedimentary rocks and its effect on rock stiffness;
  (b) Loading versus unloading-reloading stiffness values; and
  (c) Correction of the natural stress field based on rock mass quality
- Class A predictions carried out during DA and design stages were confirmed during construction by comparing with the geotechnical monitoring measurements. The results satisfactorily confirmed the assumed rock behaviour with a generally good agreement in relation to the orientation of the ground movements and the magnitude of movements
- The observed excavation induced movement were generally between 50% and 100% of the Class A predictions with a significant number between 75% and 100%. As a result, the predictions could be classified as "Fair" to "Excellent" in accordance with the Prediction Quality Classes proposed by Morgenstern (2000). However, it is noted that some additional movements are expected to occur once the ground anchors are de-stressed and forces transferred to the basement slabs.

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6 REFERENCES

Bertuzzi, R., and Pells, P.J.N. (2002). *Geotechnical parameters of Sydney sandstone and shale*. Australian Geomechanics, Vol 37(5), pp 41-54.

Boscardin, M. D. and Cording, E. J. (1989). *Building Response to Excavation-Induced Settlement*. Journal of Geotechnical Engineering, ASCE, Vol. 115, No. 1, pp. 1-21.

CIRIA PR30 (1996). *Prediction and effects of ground movements caused by tunnelling in soft ground beneath urban areas*. Construction Industry Research and Information Association, Research Project 316/1992.

Oliveira, D and Wong, P. (2012). *Selection of Rock Mass Design Parameters for Assessing Excavation Induced Movements in the Sydney CBD*. In Proc.: ANZ 2012 Conference, Melbourne, 15-18 July, pp 789-795.

Oliveira D., Badelow, F., Wong, P. (2014). *Assessing the impact of a deep basement excavation on old rail tunnels in Sydney, Australia*. In Proceedings: World Tunnelling Conference 2014, Iguassu Falls-Brazil, 15-18 May.

Oliveira, D.A.F and Parker, C.J. (2014). A*n alternative approach for assessing in-situ stresses in Sydney*. Proceedings of the 15th Australasian Tunnelling Conference, 17-19 September 2014, Sydney.

Oliveira, D.A.F (2014). *An alternative view on geotechnical parameters for tunnel design in Sydney*. Australian Geomechanics Vol. 49(3), pp. 95-108.

Fraser, K.A (1930) Methods of tunnelling as used on the City of Sydney Underground Railway. J Inst Engrs Aust., Paper No 310, Vol. 2, pp. 45-57.

Humphries, A.H.D. (1931) *Flat top and Special Tunnel Construction. City of Sydney Underground Railway. Part 1. Details of construction and underpinning of buildings*. J Inst of Engrs Aust., Paper No 358 Vol 3 pp 206-220.

Pells, P.J.N., Douglas, D.J., Rodway, B., Thorne C.P., and McMahon B.K (1978). *Design loadings for shales and sandstones in the Sydney region*. Australian Geomechanics, Vol G8, pp 31–39.

Pells et al (1998). *Foundations on Sandstone and Shale in Sydney Region*. Australian Geomechanics, December, pp 17-29

Pells, P.J.N. (2002). Developments in the design of tunnels and caverns in Triassic rocks of the Sydney region. Int. Journ. of Rock Mech. & Mining Sciences 39, pp. 569-587.

Morgenstern, N.R. 2000. *Performance in geotechnical practice*. Inaugural Lumb Lecture, Transactions of the Hong Kong Institution of Engineers.

Nemcik, J, Gale, WJ & Fabjanczyk, MW (2006). *Methods of Interpreting Ground Stress Based on Underground Stress Measurements and Numerical Modelling*. Underground Coal Operators' Conference. Paper 39.

Tey, C. H. (2012). *3-Dimensional Numerical Modelling of a Deep Basement Excavation Adjacent to Rail Tunnels and Geological Fault Zone in Sydney CBD*. In Proceedings: ANZ 2012 Conference, Melbourne, 15-18 July, pp 789-795.