

Deep Excavation Impact Assessment in Urban Environments: Predictions versus Reality

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ABSTRACT

To cater for the increasing population in Australia, we have had a property boom for several years. This has resulted in the demand for high-rise buildings with deep basement space. Within the Sydney metropolitan, these are preferably founded in rock strata. Invariably, proposed building developments are in close vicinity to other high-rise buildings, services and tunnels. In order to satisfy council's Development Approval (DA) requirements for new developments, rigorous and sophisticated numerical analyses are commonly executed to assess potential induced deformation and its consequent damage risks. Predictions of such performances require a sound understanding of rock behaviour and their defects; these include the appreciation of locked in-situ stresses and relief mechanism, defect orientations, inclusions and presence of fault zones. This is better understood by creating geotechnical models based on appropriate geotechnical site investigations and interpretation by geotechnical professionals. During construction phase, specific monitoring plans are implemented whilst bulk excavation is progressed to verify the design assumptions. This is achieved by providing monitoring equipment including the installation of geophones, inclinometers, extensometers and survey targets. This paper attempts to provide an insight into the geotechnical project life cycle for deep excavations in an urban environment from design phase to construction support and implementation of monitoring measures.

Keywords: Deep excavation, impact assessment, field monitoring, numerical model, rock engineering

1 INTRODUCTION

Studies to understand rock mechanics and its applications with deep excavation near tunnels have been a popular topic in Sydney since the late 1990s. Early texts have summarised case studies relating to the impact assessments of past building developments including the Genting Centre, Shangri-La Hotel and Green Square Station (Nye 2005). A more recent case study has also discussed the different approaches used to assess the proposed York & George building development which lay within a fault zone (Oliveira et al 2014).

The creation of a geotechnical ground model enables the design and development of economical solutions for ground bearing and underground infrastructures. This involves geotechnical site investigations to provide data to assist in developing the site geotechnical conditions. Inferred geological units are then adopted by understanding the regional geomorphology, geological setting and groundwater. To classify the different units, rock quality in the Sydney region is interpreted based on the Sydney Rock Mass Classification System (SRMCS) (Pells et al. 1978 and Pells et al. 1998). Appropriate design parameters are then carefully derived based on this classification system, which requires the understanding of rock behaviours and their defects; these include the appreciation of in-situ stress of rock and its relief, defect orientations, inclusions and presence of fault zones. Typical design parameters for tunnels and deep excavations are presented in Bertuzzi (2014) and Oliveira (2014). Oliveira and Wong (2012) have also provided a detailed discussion on relevant mechanisms that should be accounted for in deep excavation projects in Sydney. These papers emphasize the importance for designers to conduct their own assessment and adjustments to reflect the adopted method of analysis.

For geotechnical professionals, the services they provide to the principal contractor may vary dependent on the phase of involvement in the project life cycle. With deep excavations, it typically includes the design phase, construction monitoring planning and construction support services. The design phase may include the design of retention systems, remedial works and planning of construction methodologies. Whilst construction monitor planning assesses the impact of excavation

methods by installing instrumentation to measure movements, vibrations and noise. The scope and requirements are usually based on criteria specified by local authorities and key infrastructure asset owners (i.e. Sydney Trains and Roads and Martine Services).

Comparisons of predicted rock behaviour and actual performance are presented, based on experience on various projects.

2 GEOTECHNICAL PARAMETERS AND NUMERICAL MODEL

Geotechnical parameters used in numerical models are assessed based on a combined approach involving the understanding of the site geological history and processes, compilation of a geotechnical model, interpretation of in-situ tests from geotechnical site investigation and laboratory tests. Empirical methods from published text such as Carter and Bentley (1991) and Look (2007) are then commonly used to assist in deriving appropriate soil and rock parameters, which are consistent with previous experience in similar design conditions. The most influential engineering parameters on the behaviour of rock masses in deep excavation requires further calibration based on the results from the monitoring program.

Typical in-situ geotechnical parameters used in design of deep excavations are then assisted by processes of determining rock mass classes such as the SRMCS, which the author has used on projects in the Sydney region. Typical rock parameters for Sydney sandstone and shales can be interpreted from publications such as Bertuzzi (2014) and Oliveira (2014) for preliminary assessment. For other regions other than Sydney, the Q-system or RMR (Rock Mass Rating) system and local published research papers should be used.

Other considerations include interpreting the relationship between “*at-rest*” ground conditions and the in-situ stress relief during excavation of the deep basement. Especially with deep excavation in Sydney, the assumption of a single linear relationship with depth may not be appropriate. This is due to the existence of high tectonic locked-in horizontal stress at shallow depths (Oliveira and Parker 2014) and the influence of rock masses of varying stiffness.

Simulation of the excavation processes can be replicated by commercially available Finite Element Analysis (FEA) software with geotechnical capabilities. In Sydney, extremely weathered sandstone (i.e. rock mass with soil like properties) are modelled as isotropic elastic – perfectly plastic materials following the Hoek-Brown failure criterion, whilst weathered to fresh sandstone are modelled as transversely isotropic elastic – perfectly plastic material with failure through either the intact rock (matrix) or ubiquitous joints. An example of a numerical model is depicted in Figure 1. Emphasis should be placed on adopting appropriate engineering design parameters to determine ground deformation and relevant structural actions.

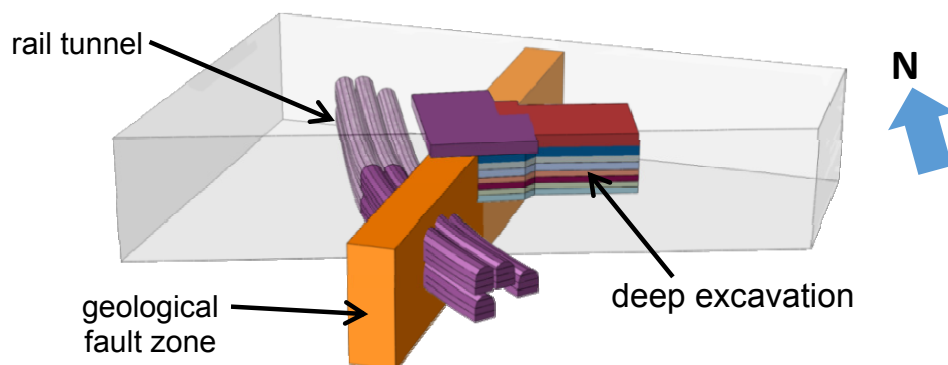


Figure 1. Typical 3-dimensional numerical model used to predict excavation performances on the York & George Street project site (after Oliveira et al. 2014)

3 EXCAVATION AND VIBRATION MONITORING PLAN

Deep excavations in urban areas can be relatively complex due to the presence of existing services, adjacent building structures and/or tunnels. To protect these assets and to verify the design, monitoring plans are devised including Geotechnical Excavation Monitor Plan (GEMP) and Vibration Monitor Plan (VMP).

The objectives of both GEMP and VMP are to monitor the behaviour of the excavation and its effects on adjacent infrastructures and services. These are then validated with predicted performances of the deep excavation considered during DA stages. Certain limit criteria are set so that excavation procedures can be tracked and modified if recordings exceed acceptable levels. These monitoring plans are primarily in place to provide safety during construction and to the public, and mitigate risk of damages to adjacent structures and infrastructures.

3.1 Geotechnical Excavation Monitoring Plan

GEMP involves various tasks to satisfy the objectives of the project. The procedures can be described in conjunction with excavation processes of the development. The excavation processes include pre-excavation works, installation of geotechnical instrumentation, performing geotechnical monitoring throughout the bulk excavation period, reporting as excavation progresses and conclusive summary. This is depicted on Figure 2. To confirm anticipated lateral displacement performances, geotechnical monitoring is assessed at each incremental drop of bulk excavation and compared to a set of criterions. The set of criterions can be in the form of excavation deformation trigger levels and response as listed below:

- *Alert level (or green light)*: Safe working value less than the agreed level;
- *Alarm level (or amber light)*: Working value where caution is advised but less than the agreed level. Frequency of monitoring to be increased;
- *Action level (or red light)*: Value where agreed level has exceeded. Works will stop, monitoring will continue and a tailored contingency action plan will be devised.

The agreed level is semi-empirical method derived from a combination of numerical predictions and tolerable displacement limits from adjacent structures/services. Based on semi-empirical methods and past experience with deep excavations in Sydney, has shown that lateral deformations in deep basement excavations typically vary between 0.5-2 mm/m of rock depth excavation in practice (Pells 1990, Walker 2004 and Oliveira and Wong 2012).

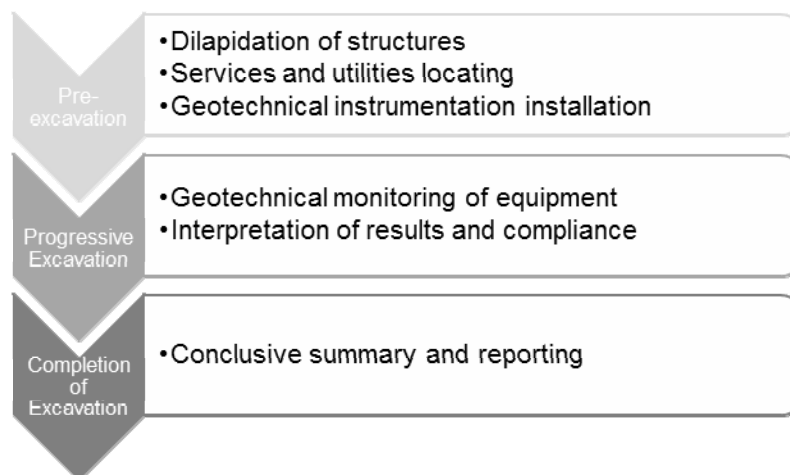


Figure 2. Construction excavation process with GEMP and VMP involvement

3.2 Vibration Monitoring Plan

Similarly, VMP's utilised during bulk excavation, may work in conjunction with the GEMP. The works involve the monitoring of peak particle velocity (PPV) caused by heavy machinery during deep excavation. By doing so, the procedure provides a measure of risk mitigation and early warning signs of potential damages. BS 5228-4:1986 *Noise and vibration control on construction and open sites* recommends a set of trigger level range between 10-25 mm/s and values may be tailored to specific projects. In some instances this may be low as 6.5-12.5 mm/s for structures over or adjacent to tunnels. This is in accordance with Transport for NSW Standard for *Miscellaneous Structures*: T HR CI 12070 ST to prevent the potential damages to rail tunnel lining. If recordings surpass the trigger level, modification of equipment and excavation methodology would be required.

4 CASE STUDY PERFORMANCES

Three projects are presented where deep excavation induced movements have been assessed by FEA for the purpose of 'Class A' predictions (i.e. prior to construction and based on inferred models and parameters). Lateral displacements are horizontal movement occurring near top of a deep excavation face. These are presented in conjunction with results from geotechnical monitoring summarised in Table 1 and Figure 4. The below mentioned cases were assessed and designed by Coffey Geotechnics Sydney Office. Details of each case study have been summarised with the following key points; site description, excavation support, types of monitoring plans and field measurement results.

4.1 York and George Street, Sydney

- *Site description:* The York and George Street project is a mixed use development located within the Sydney CBD. The geological units vary across the site and can be separated into two zones. The rock mass on the York Street side lies within a fault zone and comprises low quality rock to great depths. The rock mass along the George Street comprises of fill overlying weathered to fresh sandstone with rock mass quality increasing with depth.
- *Excavation support:* Lateral support comprised of soldier pile walls with a combination of anchoring and propping support along York Street (refer Figure 3) and deep open cut along George Street. These were devised to address the variable quality of rock mass encountered, from poor to good. Sensitive neighbouring structures in close vicinity to the project site included adjacent commercial buildings, frequently used roads (York Street and George Street) and existing rail tunnels. Further details of the project are presented in Oliveira et al (2014) and Oliveira and Chan (2016).
- *Monitoring plans:* A VMP was developed with vibration monitoring in the rail tunnel and neighbouring building. The tolerable surface vibration limit near commercial building was determined from an acoustic logic assessment with references to BS 5228-4:1986. This assessment enabled the development of vibration trigger levels for alert and action of 15 and 25 mm/s respectively. The neighbouring rail tunnel is an asset of Sydney Trains (Transport for NSW) and required to comply with alert and action trigger levels of 6.5 to 12.5 mm/s respectively from T HR CI 12070 ST. Geophone sensors with monitor readout unit were installed at the north-west (near a commercial building) and south west corner (near the rail tunnel) and maintained throughout the excavation period. GEMP was also enforced with two inclinometers in the soldier pile walls (Spiden and Carlton House), survey monitoring at 10 m centres located at the top of the capping/excavation and a horizontal extensometer installed along the excavation face closest to the rail tunnel. The excavation monitoring plan regime was to ensure that behaviours were as predicted with numerical model and within tolerable limits.
- *Field results:* A maximum PPV occurrence of 11 mm/s near the commercial building and 1 mm/s within the tunnel was recorded during the bulk excavation period. This was considered to be within acceptable levels. Construction inspections were also called upon for observation of ground conditions of the excavation face, piles and footings to ensure compatibility with design. With an approximately 15 m excavation depth, the predicted performance from FEA indicated lateral displacements between 10-22 mm acting at the top of the open excavation and soldier pile walls. Results from inclinometer measurement indicated a maximum lateral displacement of 14 mm (i.e. were well within the expected movement range for deep excavations).



Figure 3. View of excavation faces for the York and George Street project – (a) York and George, Sydney – Carlton House; and (b) York and George, Sydney – Spiden House

4.2 Epping Road, Lane Cove

- *Site description:* This is a property development running adjacent to Epping Road located in Lane Cove. The geological unit of the site typically comprised fill overlying slightly weathered to fresh sandstone at shallow depths.
- *Excavation support:* Contiguous pile walls were used to retain all side of the basement excavation with anchoring support. The site is bounded by a busy road, commercial buildings and in close vicinity to a road tunnel.
- *Monitoring plans:* A VMP was developed for the project and geophones installed on the excavation face closest to the road tunnel. An initial VMP advising vibration triggers for alert and action levels of 10 and 25 mm/s respectively were issued to the respective stakeholders of the project. This was later reduced to 7.5 and 10 mm/s at the request of the asset owner of the road tunnel. These geophones were installed and monitored for the duration of the bulk excavation to ensure risk was limited as a result of the vibration by heavy machinery. A GEMP was also utilised during the bulk excavation of the site to confirm typical lateral behaviours based on experience of sites with similar conditions. To satisfy Roads and Martine Services (RMS) requirements, three inclinometers along the northern boundary were installation closest to the RMS road (Epping Road). These were accompanied with survey points at 25 m nominal centres. Measurements were to be recorded and assessed at each 1.5 m depth of excavation.
- *Field results:* For an approximate 15 m deep of excavation, the actual lateral displacements were 8 mm from the top of the excavation face. Vibration monitors at the north-east corner of site, located closest to the road tunnel recorded a maximum PPV of 4 mm/s. Both vibration and lateral displacement were within anticipated and tolerable limits.

4.3 Pacific Highway, Lane Cove

- *Site Description:* A property development located in close vicinity to a major intersection, along Pacific Highway in Lane Cove. The geological units comprised fill overlying residual soil, and then weathered to fresh shale whose rock mass quality increased with depth.
- *Excavation Support:* Soldier Pile Walls were used to retain all side of the basement. Anchoring was predominately used, with propping support used in areas with boundary restrictions. This development lay in close vicinity to a busy highway, existing neighbouring structures and road tunnel.
- *Monitoring plans:* As proposed in the VMP, an in-ground geophone sensor was installed at the site closest to the road tunnel to ensure vibration trigger levels of 10 and 25 mm/s for alert and action levels is not exceeded. A base station connected to the sensor included the functionality of housing the monitoring unit, remote access and alarm capabilities. This

enabled real-time monitoring and efficient response if trigger levels were exceeded. In accordance with the project specific GEMP, the plan aimed to reduce the risk in damaging adjacent structures. The setup included two inclinometers (along adjacent property and Pacific Highway), survey monitoring at 10 m centres along the pile cap and a vertical extensometer closest to the road tunnel were installed. Initiating the plan allowed the designers to verify its compliance with numerical predicted behaviours between 10-15 mm.

- **Field results:** The maximum recorded lateral displacement was 14 mm from top of the excavation for a 9.5 m deep excavation. Based on the inclinometer results, this occurred along the Pacific Highway alignment. The maximum recorded PPV was 13 mm/s during an isolated incident were daily assessment of vibration data subsequently carried out and the incident did not reoccur. Overall the vibration and displacement were well within tolerable limits.

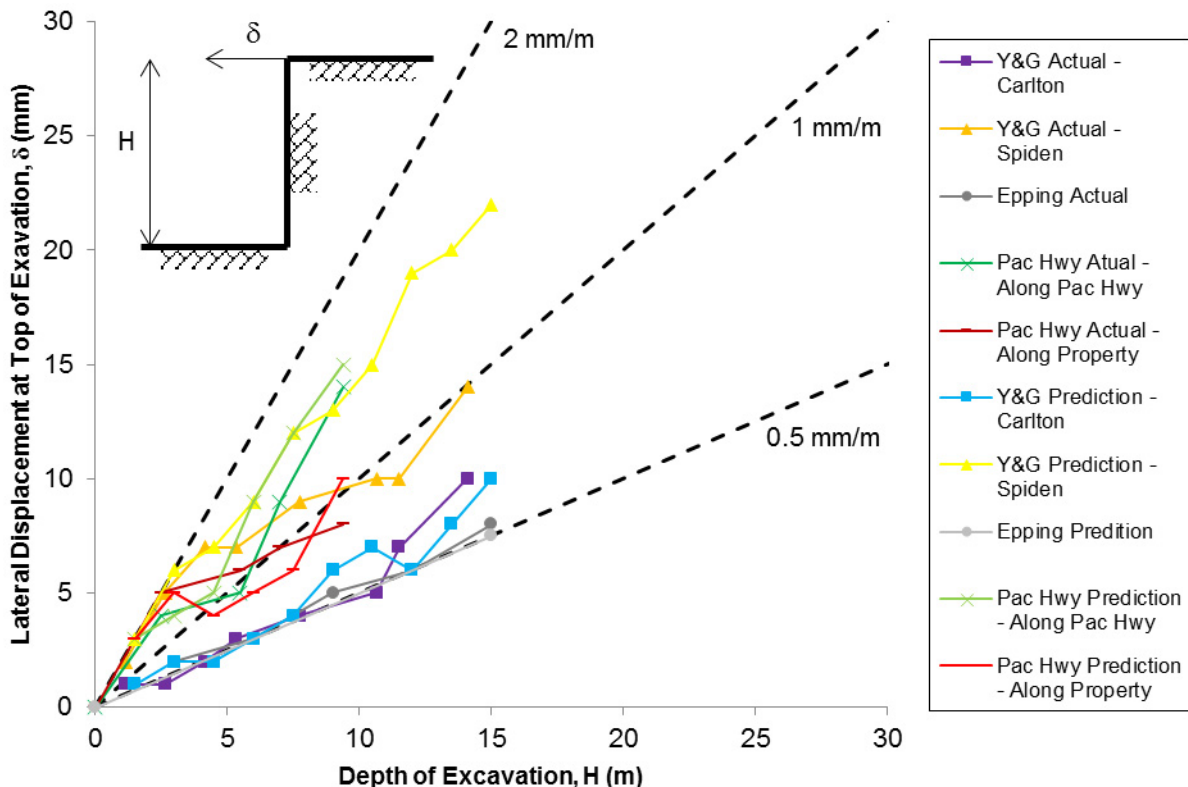


Figure 4. Summary of lateral displacement at top of excavation depth and length of excavation

Table 1. Summary of lateral performances and vibrations from case study

Site Location		Predicted Lateral Performance ^a (mm/m)	Actual Lateral Performance ^a (mm/m)	Vibration Limit ^b (mm/s)	Actual Vibration (mm/s)
York & George St, Sydney	Carlton House	0.6	0.6	15/25	11
	Spiden House	1.5	1		
Epping Rd, Lane Cove		0.5-2 ^c	0.5	7.5/10	4
Pacific Hwy, Lane Cove	Along Pacific Hwy	1.4	1.3	10/25	13
	Along Property	1	1		

^a Average lateral displacement interpolated from line of best fit and intercepting the origin.

^b Two values shown, provides a lower bound and upper bound vibration limit.

^c Lateral displacement range based on experience

5 SUMMARY AND CONCLUSIONS

An insight into deep excavation construction has been presented in the form of case studies within this paper. To predict its impact to important assets, FEA are carried out to assess the behaviours of deep excavation faces and its compliance with tolerable limits. Their performance is then confirmed by field measurements.

The variations in lateral displacement, especially where the differences are at the same site can be attributed to the changing geology, influences of high locked-in stress in rock masses from a dominant direction, changes to excavation sequence and the type of retention system (e.g. anchored Soldier Pile Walls, propped Soldier Pile Walls or open cut excavation) which can deviate the outcomes of the predictions.

A well-developed excavation plan is essential in mitigating risk associated with the deep excavation. It also provides a strategy and contingencies if limits are reached. Plans such as VMP and GEMP were used across all sites mentioned in this paper.

Measured displacements in the three case studies presented confirmed past experience in Sydney. The 'Class A' predictions have also generally confirmed the anticipated behaviour despite some over-predictions. It is important to note that these over-predictions are generally a result of some degree of conservatism in the assessment and considered beneficial to the project as no "factor of safety" are applicable for such serviceability assessment. Measurements of actual displacement below predictions allow for an excavation progress without the need for interventions and therefore beneficial from a construction program viewpoint. Nevertheless, some of the discrepancies between predictions and actual measurements are typically related to discrete features that are often difficult to capture in numerical models. These include uncertainties in the assessment of both in-situ stress conditions and rock mass deformation parameters which largely rely on engineering judgment to choose appropriate values.

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