Numerical Analysis of a Top-Down Constructed Deep Basement with Diaphragm Walls in Barangaroo, Sydney - A Case Study

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ABSTRACT: *Crown Sydney Hotel Resort, is located at Barangaroo South alongside Sydney Harbor on the western side of Sydney's business district. This building is being developed as a single mixed-use high-rise tower of 72 stories, rising over a multi-level podium and a 3-level basement car park. The tower basement is a 13 m deep excavation with irregular geometry made using top-down construction in highly variable ground and bedrock profiles. The excavation has been retained by the construction of 33 diaphragm wall panels, which resist and transfer the out-of-balance soil and water loads to the 36 internal barrettes by means of multiple levels of slab diaphragms.*

2D and 3D numerical methods have been adopted to simulate the complex interaction between the soil and the basement of this structure. The 3D model has been used to capture the 3D effects of the deep excavation and asymmetric wall geometry, including soil pressure changes at the corner panels, potential arching, out-of-balance soil loads, and group effects. A detailed instrumentation and monitoring plan have been developed to assess the performance of the basement during construction. This paper presents the design methodology and the numerical modeling adopted to carry out a robust geotechnical design and successful modeling of the soil-structure interaction. It explains how the numerical model has been validated and calibrated. The numerical predictions are compared with the data that has been collected from the web-based, real-time instrumentation and monitoring during the construction phase. The approaches that have been used to mitigate the associated risks with the design are also discussed in this paper.

KEYWORDS: Diaphragm wall, deep excavation, top-down construction, numerical analysis, 3D effect, corner effect, instrumentation, monitoring, tall buildings.

SITE LOCATION: Geographic Database

INTRODUCTION

The Barangaroo South Precinct is a 22-ha land parcel located on the north-western edge of the Sydney CBD. Crown Sydney Hotel Resort is located on Stage 1C of Barangaroo South (see *Figure 1*). Crown Sydney Hotel Resort is Sydney's tallest single mixed-use tower (271 m) with 75 floors including a 3-level basement. This building is surrounded on three sides by other areas of the Barangaroo development precinct and, on the western side, is adjacent to Darling Harbor (Azari et. al 2019).

The basement of the tower comprises a 13 m deep excavation which has been retained by 33 diaphragm wall (D-Wall) panels. D-Walls were designed to act as structural elements with the following functions:

- Resist and transfer the earth pressures, including significant global out-of-balance earth pressures to internal barrettes by means of multiple levels of slab diaphragms.

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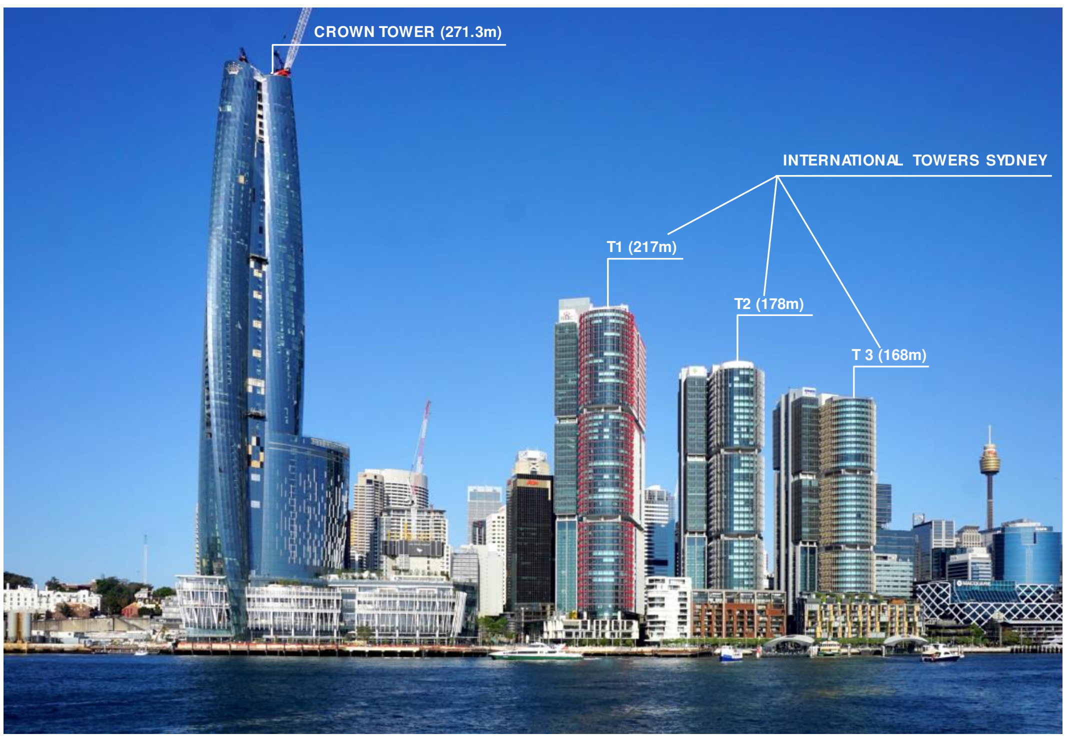
(a)



(b)

Figure 1. (a) Location of Barangaroo development precincts; and (b) Crown Sydney Hotel Resort (October 2020).

- Transfer high vertical and lateral loads from the building above (including wind and earthquake) during and post construction to class IV sandstone or better.

- Prevent the inflow of groundwater (GW) into the basement during construction and in the permanent state.

A full concurrent top-down basement and bottom-up tower construction methodology was adopted in order to meet overall project deadlines for Stage 1C with all foundation elements constructed in-situ from ground level. Plunge columns were used to share the tower loads and to facilitate the top-down basement construction. The Perimeter Retention Walls (PRWs) and the tower and podium foundations were installed from the ground level, after which the ground floor slab was fully constructed. Subsequently, construction of the tower and podium structure was carried out simultaneously with excavation and construction of the basement works (Azari et. al 2020). The configuration of D-walls and foundation elements is presented in *Figure 2* and *Figure 3*.

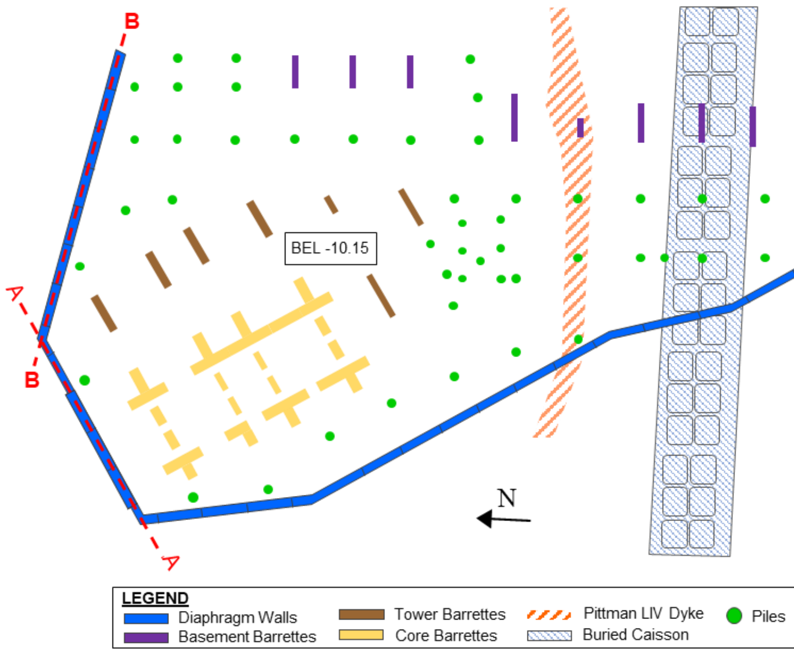


Figure 2. Crown Sydney Hotel Resort basement foundation plan.

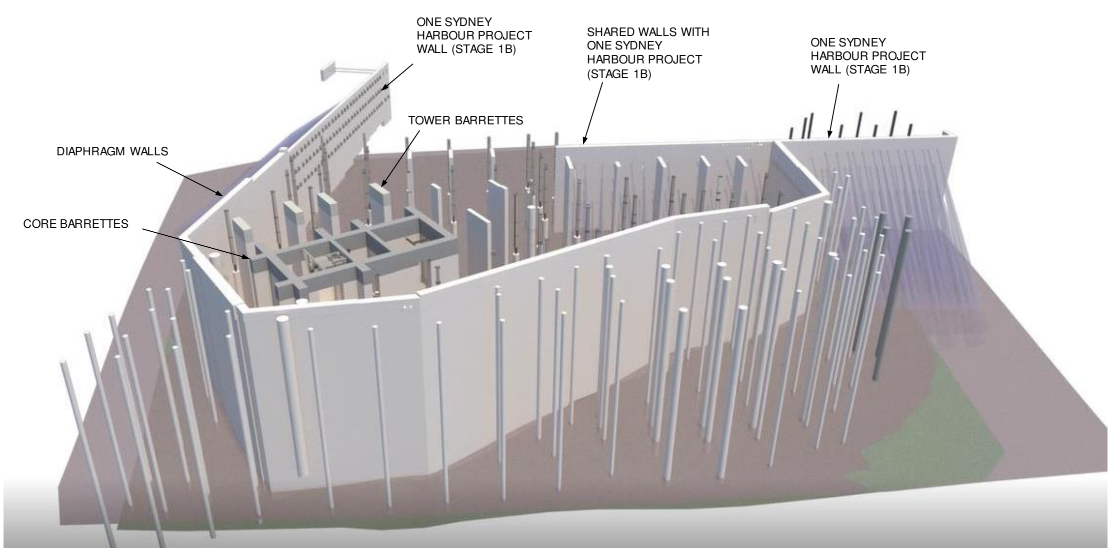


Figure 3. A 3D view of the Crown Sydney Hotel Resort basement foundation.

Due to the irregular and asymmetric shape of the excavation and complex interaction between the soil and foundation elements, a combination of two-dimensional (2D) and three-dimensional (3D) finite element analyses were adopted for the geotechnical design and to assess the performance of the basement for temporary and permanent conditions.

This paper presents a summary of the site investigations and ground conditions. It also describes the steps and methodologies taken to undertake, validate, and calibrate the numerical analyses. The numerical analyses results are presented and compared with the recorded data from the instrumentation and monitoring plan. The paper furthermore explains the sensitivity analysis approaches that have been used to mitigate design risks.

HISTORICAL DEVELOPMENT AND GROUND CONDITIONS

Historical Developments

Barangaroo is a 22-hectare parcel of land that has historically been developed and used for a variety of productive port facilities over the last 150 years. Barangaroo was occupied by a gasworks and was also used for other industrial, commercial, and maritime activities from 1840 to 1925. A series of timber finger wharves were then built across the Barangaroo precinct, with several extending over the Crown Sydney Hotel Resort site (the Site) as shown in *Figure 4*(a). In the 1970s, a seawall comprising of caissons (sand-filled precast concrete segments) was built at the Barangaroo precinct, creating wharves for ship berthing (*Figure 4*(b)). These caissons were constructed over a gravel platform with the sea floor dredged prior to placement. Following the placement of the seawall, the precinct (including the Site) was backfilled with uncontrolled fill and paved. The Site was subsequently used as a shipping container terminal. Since the construction of a new port at Port Botany in 1979, the terminal activities and container traffic declined. By the 1990s, container ships no longer berthed in Sydney Harbor (Kane 2017). Prior to the precinct's redevelopment, Barangaroo was used as an open space for mass gatherings and as a temporary passenger terminal for cruise liners (*Figure 4*(c)). The redeveloped Barangaroo precinct is illustrated in *Figure 4*(d).

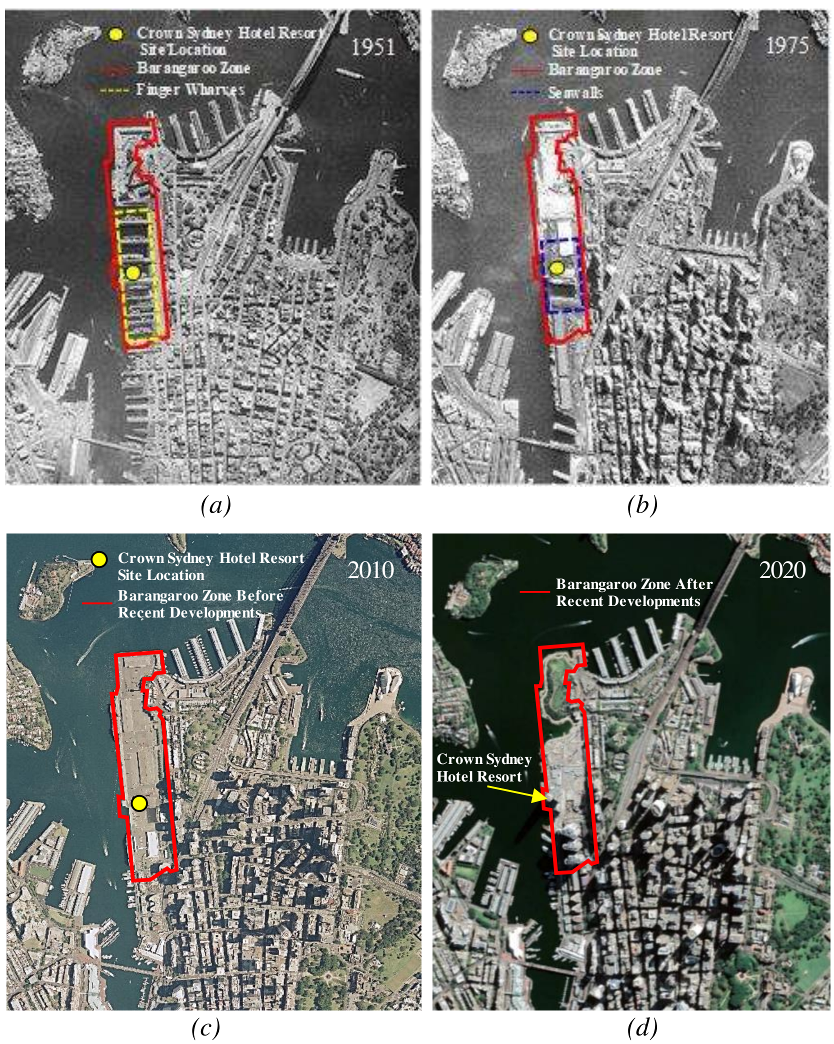


Figure 4. Aerial views of the Barangaroo Site showing historical developments; (a) 1951; (b) 1975; (c) 2010, and (d) 2020.

Ground Conditions

The ground conditions at the site have been characterized based on a comprehensive site investigation program including more than 60 geotechnical boreholes up to 57 m depth, previous site investigation data, and available publications. The results of the desktop study and site investigations indicate that the site is overlain by highly variable fill with possible voids and obstructions such as buried steel piles, timber piles, steel scraps, etc. As previously discussed, subsurface caissons are also present at the Site. The fill is underlain by alluvial sediments and residual soils. Underlying the soil units is Hawkesbury Sandstone, which is typically a medium to coarse grained quartzose sandstone. The sandstone was classified with the classification system presented in *Table 1* (Pells et al. 1998). The rock classes were derived for each cored borehole using the log descriptions, Uniaxial Compressive Strength (UCS), and Point Load Tests (PLTs). A focus of the drilling was to identify the top of rock and rock weathering and classes with depth.

Table 1. Pells et al. 1998. Sandstone Classification.

Rock Class	*Rock Mass Parameter*		
	UCS (MPa)	*Allowable Seams*	*Defect Spacing (mm)*
Sandstone Class V	>1	n/a	n/a
Sandstone Class IV	>2	<10%	>60
Sandstone Class III	>7	<5%	>200
Sandstone Class II or better	>12	<3%	>600

The sandstone at Barangaroo is known to dip towards the west (i.e., into Darling Harbor). Typically, in Sydney Harbor, rock faces develop as a series of sub-horizontal benches separated by sub-vertical cliff lines. Three typical sections (i.e., Sections A-A, B-B, and C-C) of the ground profile at the Site are illustrated in *Figure 5*-7.

The Sydney region is intersected by a number of basalt and dolerite dykes. The Pittman LIV Dyke mapped, in the proximity of the Site, trends in an east-west direction. The dyke comprises extremely weathered to fresh dolerite. The width of the dyke varies between 3 m and 5 m, including offshoots, and extremely weathered dolerite seams (i.e., up to 100 mm thick) intrude into bedding partings at contact with sandstone. An altered zone approximately one meter wide occurs on either side of the dyke. The location of the Pittman LIV Dyke in relation to the Site is depicted in Figure 2.

Groundwater

The design groundwater level was considered at RL +1.5 m on the active side of the D-Walls for the temporary case (i.e., during construction). This level considers the Highest Astronomical Tide (HAT) level, recorded groundwater tidal fluctuations, and recorded water levels at Fort Denison from 31 May 1914 to 31 December 2006 (Watson et al. 2008).

The design groundwater level behind the D-Walls was taken as RL + 2.335 m for the long term, with the basement designed as a fully tanked structure. This groundwater level includes the anticipated increase in water levels associated with a 1 in 100-year extreme weather event as well as an allowance for global warming. The unit weight of water was taken as 10.05 kN/m^3 to allow for the salinity of the water.

GEOTECHNICAL DESIGN AND NUMERICAL ANALYSIS

Analysis Methodology

The Crown Sydney Hotel Resort basement was designed to serve several functions, including resisting and transferring the global out-of-balance earth pressures to internal barrettes (i.e., core, tower, and basement barrettes) by means of D-Walls and basement slabs. The geotechnical design of the basement was conducted to analyze the soil-structure interaction (SSI) and determine displacements, bending moments, and shear forces in the D-Walls, basement slabs, and barrettes for the temporary and the long-term conditions. The settlement of the adjacent ground surface due to the excavation of the basement was also estimated.

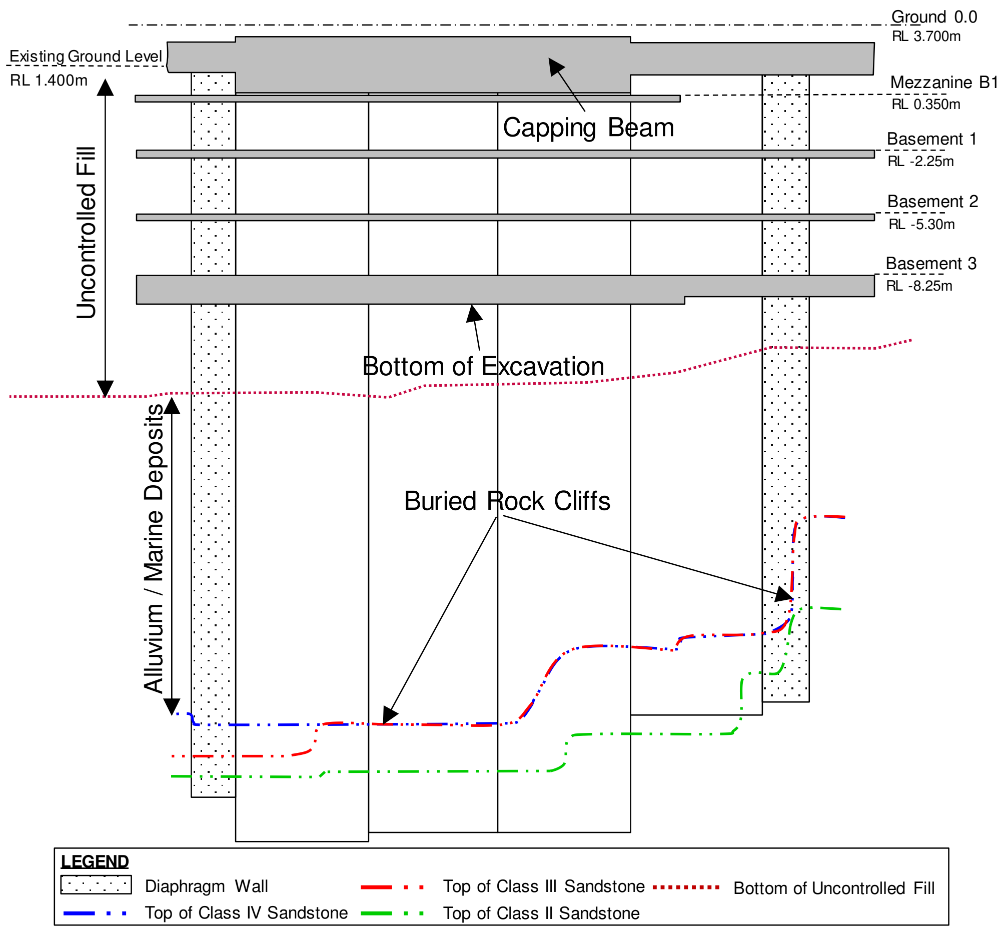


Figure 5. A typical section of the ground profile at Site location – Section A-A.

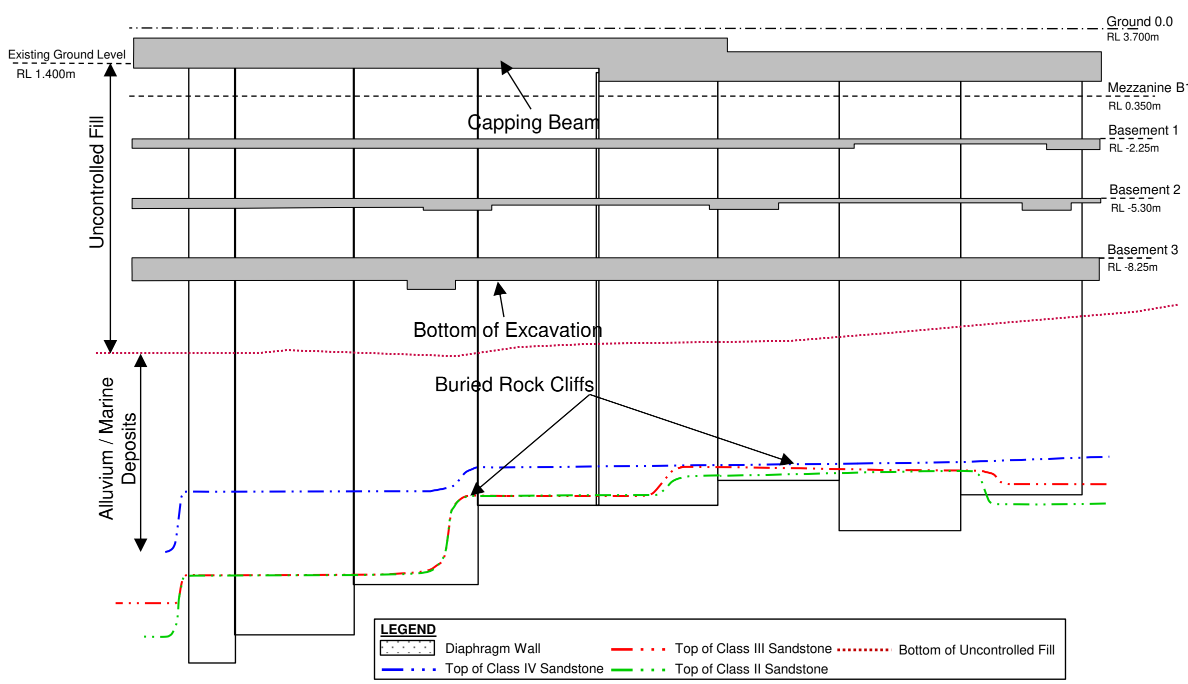


Figure 6. A typical section of the ground profile at Site location – Section B-B.

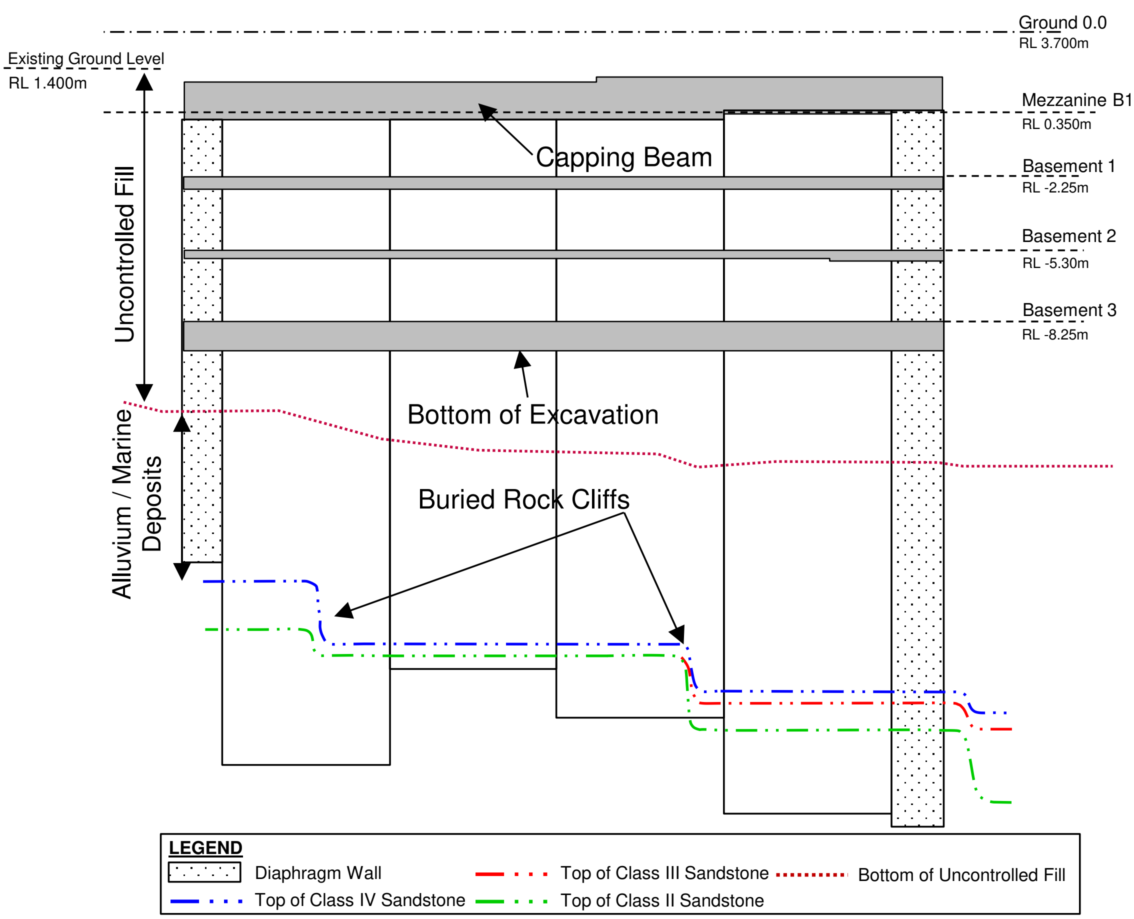


Figure 7. A typical section of the ground profile at Site location – Section C-C.

The Finite Element (FE) approach was used to carry out the SSI analysis of the asymmetric excavation, highly variable ground profile, and complex configuration of the structural elements. PLAXIS 2D and 3D software was used to carry out the numerical analysis. This analytical method was also used to determine the earth pressures and validate the results of the FE analysis. The geotechnical design and numerical analysis approaches are summarized in *Table 2*.

The earth pressures extracted from the PLAXIS 2D and 3D models were used as input for the structural model, which allowed for an assessment of the out-of-balance earth pressure in the structural model. An iterative geotechnical/structural analysis procedure allowed validation of the adopted values for the modulus of subgrade reaction. The predicted structural behavior of the basement elements, including displacements and structural actions, were also extracted from the PLAXIS models and used in this iterative process for the calibration of geotechnical and structural models as well as for the preparation of the instrumentation and monitoring plan.

Table 2. Adopted geotechnical design and numerical analysis approaches.

Analysis Approach	Function
2D FE Analysis (PLAXIS 2D)	• To assess the out-of-plane SSI for a typical panel in each D-Wall. • To analyze panels with increased socket lengths. • To carry out sensitivity analyses.
3D FE Analysis (PLAXIS 3D)	• To assess the effects of the irregular basement shape including 3D effects, out-of-balance soil loads, corner panels, and potential arching effects.

Rock Socket Design

The D-Walls and barrettes were socketed into Sandstone Class III or better. The adequacy of socket lengths against the lateral bearing capacity of the rock socket was assessed using the approach described in the Hong Kong Geoguide 1 (GEO 1993). Geotechnical strength reduction factors in the design were determined in accordance with the requirements of AS 2159 (2009). The socket depth also checked for uplift, and combined vertical and lateral forces in both temporary and permanent cases.

Numerical Models

The PLAXIS 3D model was developed based on the results of the geotechnical site investigations, excavation geometry, and configuration of the structural elements. The vertical boundary of the numerical model was considered as fixed and free in the horizontal and vertical directions, respectively. Ten-node tetrahedral volume elements with a reasonable mesh size distribution were adopted in the model.

The D-Walls, slabs, and internal barrettes were simulated using plate elements in the 3D model. Interface elements with a reduction factor of 0.7 were modeled between plate elements and soil and applied to the soil strength parameters. D-Wall panels were modeled with widths of 6 to 8 m and thicknesses of 1.2 to 1.5 m. The D-Wall panels were connected by deep capping beams. A beam element was used to model the capping beams for the D-Walls and the core. The connection type between the slabs and D-Wall/barrette elements was modeled as a fixed connection. The joints between the D-Wall panels were modeled by releasing the vertical shear and the bending moment between panels. The joints in the model consider the effect of interaction and compression between the panels. A fixed joint connection was also assessed for sensitivity checks.

The D-Walls were modeled as plate elements in PLAXIS 2D and the strength reduction factor of 0.7 was used for the interface element between soil and plate elements. In the 2D model, the slabs were modeled as anchor elements, where the stiffness of the anchors was derived from calibration with the PLAXIS 3D model.

The Hardening Soil (HS) and Mohr Coulomb constitutive models were used to simulate the behavior of the soil and rock materials, respectively. The schematic view of the PLAXIS 3D model is illustrated in *Figure 8*.

Construction Staging

The basement of the Crown Sydney Hotel Resort was constructed adopting a top-down methodology to meet the project's timeframe. The adopted construction staging in the numerical analysis is summarized and presented in *Table 3* and *Figure 9*.

Table 3. Adopted construction staging in the numerical analysis.

Stage No.	Description
0	Generation of in-situ stress conditions
1	Install diaphragm walls and barrettes
2	Apply surcharge of 25kPa
3	Maintain GWL to RL +0.5 m and excavate to RL +1.5 m
4	Install GF slab
5	Lower GWL to RL -4.5 m and excavate to RL -3.5 m*
6	Install B1 slab
7	Lower GWL to RL -11.65 m and excavate to RL -10.65 m*
8	Install B3 slab
9	Install B2 slab
10	Reduce wall stiffness for long-term effects and raise GWL to RL 2.335 m

* includes 0.5 m over-excavation allowance to account for unplanned excavations

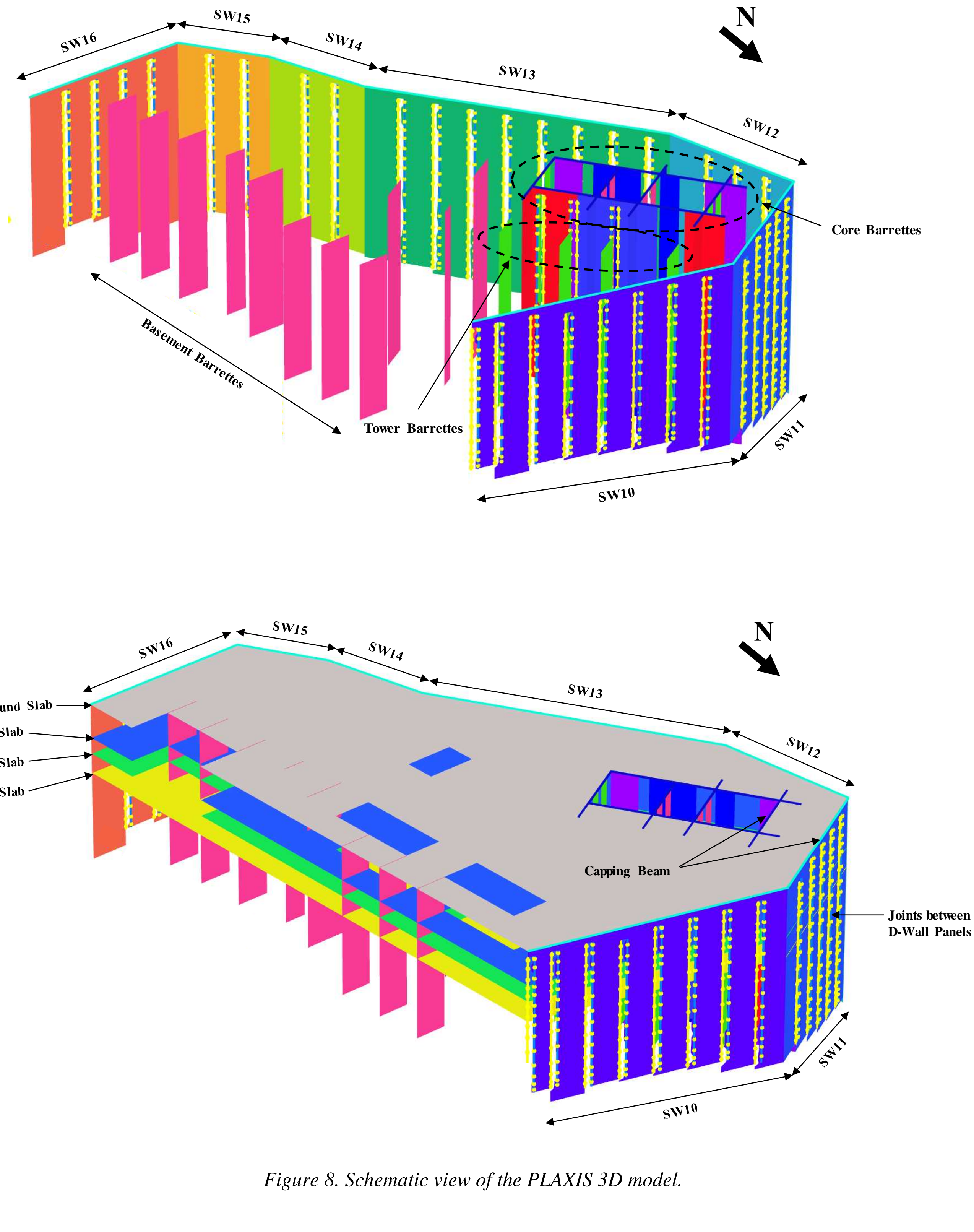


Figure 8. Schematic view of the PLAXIS 3D model.

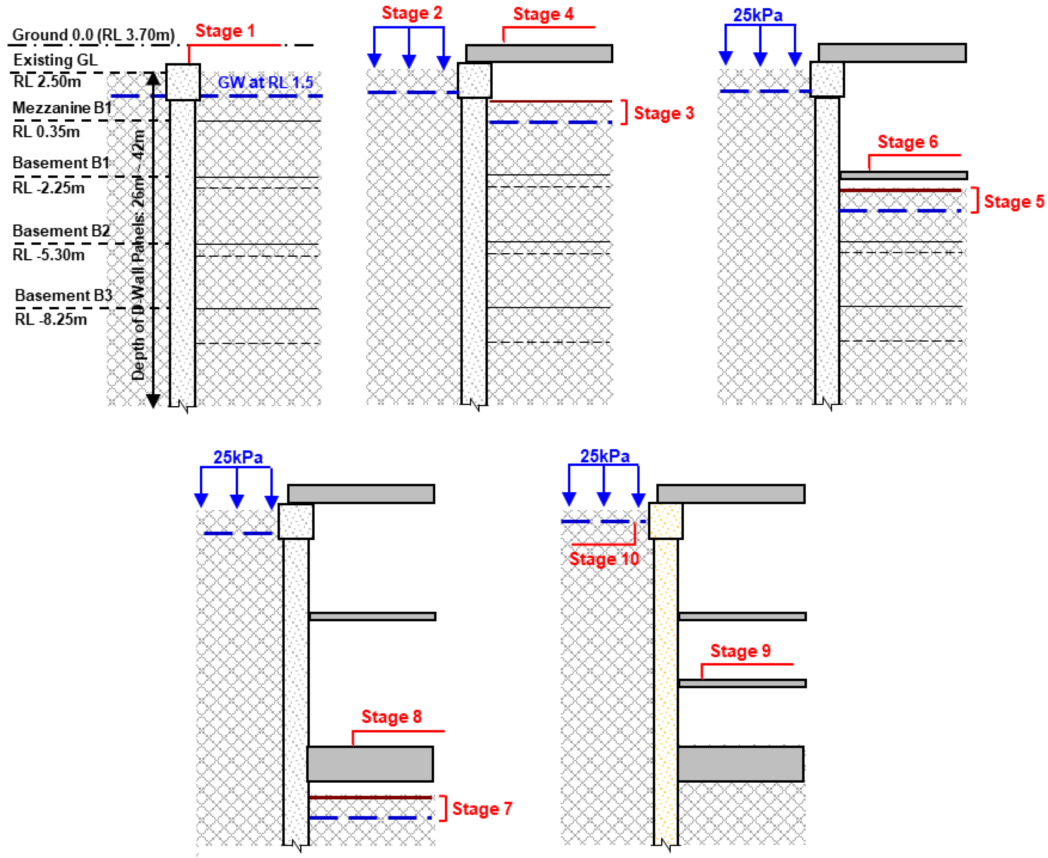


Figure 9. Adopted top-down construction staging in the numerical analysis.

Soil and Rock Parameters

The Hardening Soil (HS) Model was adopted for soil units in the numerical analysis. The aim of using the HS Model was to more realistically simulate the unloading behavior of the soil units and estimate the ground movements due to the excavation. The HS Model describes the soil stiffness more accurately by using three different input stiffnesses: secant stiffness in drained triaxial test (E_{50}^{ref}), tangent stiffness for primary oedometer loading (E_{oed}^{ref}), and unloading/reloading stiffness (E_{ur}^{ref}). In the HS Model, the total strains are calculated using a stress-dependent stiffness. The plastic strains are calculated by introducing a multi-surface yield criterion (Schanz, T. et. al 1999).

It is important to understand that the HS constitutive soil model is a simplified mathematical form of soil behavior. Consequently, it does not capture all soil behavior but rather approximates it. In this regard, the limitations (and the implications of these limitations on the design) associated with this approximation should be clearly understood. The limitations of the HS Model are that it is unable to simulate strain softening/hardening and thus under- or overestimates deformations that occur at very small strains.

In addition to the limitations of the HS Model, the incorrect selection of parameters will also lead to erroneous results. As such, it is recommended that model inputs be based on the laboratory and field tests such as a triaxial test, oedometer test, self-boring pressuremeter test, or other tests at relevant stress and strain ranges where practical. These results should then be used to simulate the test results for calibration and validation of the soil model, prior to the simulation of the actual problem (Calvello, M. & Finno, R. J. 2004, and Rodríguez-Rebolledo et al. 2019). The adoption of input parameters should also be carefully selected with regards to the strain and stress ranges that the soils will experience in reality. Sensitivity analysis and comparison with other calculation methods should also be used to gain an understanding on the sensitivity and/or importance of the respective parameters adopted. The importance of previous experiences and engineering judgment should furthermore be brought to bear when validating the analysis.

In this project, the sensitivity analysis was carried out adopting the range of elastic moduli presented in *Table 4.* The potential variation in modulus was deemed a critical parameter when considering deformation due to the uncertainty associated with the highly variable strength of the fill and alluvium materials. This analysis also helped us gain an appreciation for the potential impact of a differing modulus which may occur at differing strains and helped provide an appreciation of the deformation envelope that may exist but was not captured due to the limitations of the HS Model. The Mohr Coulomb (MC) Model was used to simulate the behavior of the rock. The adopted soil and rock parameters in the numerical analysis are also summarized in *Table 4.*

Table 4. Adopted soil and rock parameters in the numerical analysis.

Material	Model	Unit Weight, γ' (kN/m^3)	Cohesion, c' (kPa)	Friction Angle, φ' (degrees)	Elastic Modulus, E' or Secant Stiffness, E_{50}^{ref} (MPa)	Poisson Ratio, ν
Fill	HS	20	1	30	15-40*	-
Alluvium	HS	18	1	28	10-20*	-
Dyke	MC	20	5	25	60	0.35
Sandstone V	MC	23	20	35	80	0.3
Sandstone IV	MC	23	100	35	500	0.3
Sandstone III	MC	24	400	38	800	0.3
Sandstone I/II	MC	24	600	40	1600	0.2

* HS Model: $E_{oed}{}^{ref} = E_{50}{}^{ref}$ *and* $E_{ur}{}^{ref} = 3 \times E_{50}{}^{ref}$

CALIBRATION OF THE NUMERICAL MODEL

The PLAXIS 3D model was used to determine the total out-of-balance soil load on the basement. The consistency of the overall deformation of the floor diaphragm and the distribution of lateral forces in the resisting structural elements were assessed by comparing the results of PLAXIS 3D and 2D models. Then the structural models were calibrated against the PLAXIS 3D model. A snapshot of a 3D structural model is shown in *Figure 10* below.

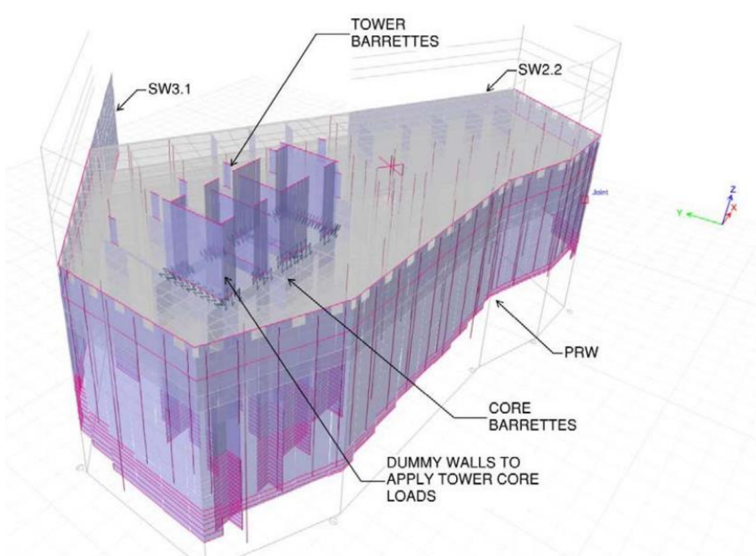


Figure 10. Snapshot of structural model.

RESULTS

The lateral earth pressure diagrams for each D-Wall at selected construction stages were provided to the structural team to be incorporated into the design. A typical diagram that includes multiple earth pressure plots for comparison is illustrated in

Figure 11. The components of soil pressures from water and 25kPa surcharge were provided separately to facilitate the application of various load factors on soil and water for design optimization.

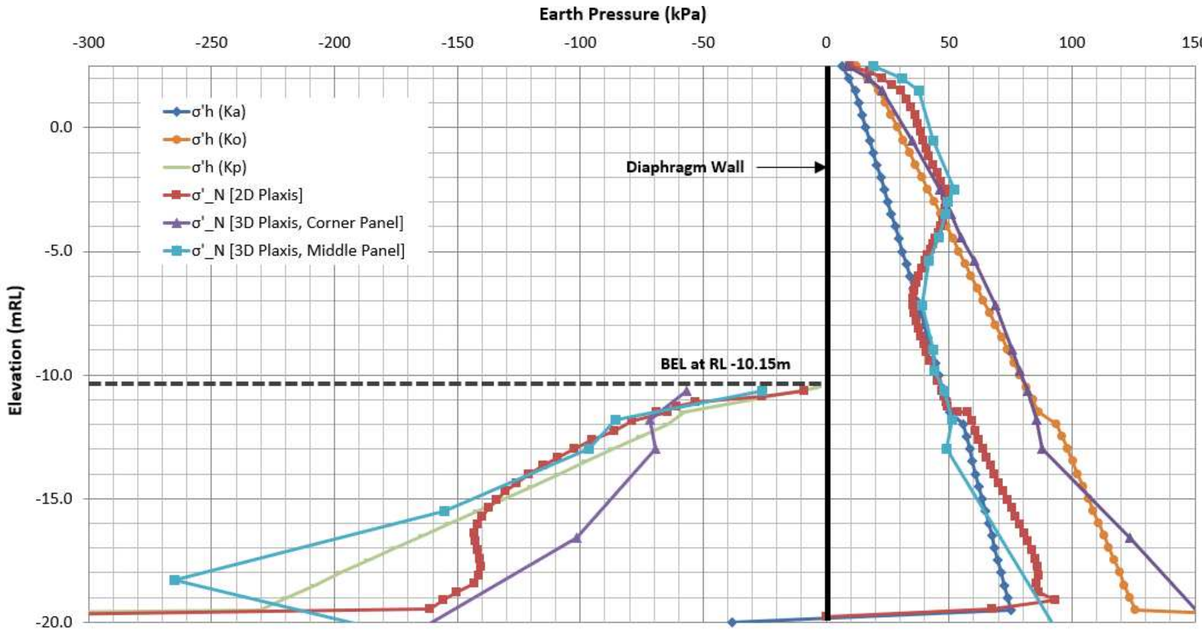
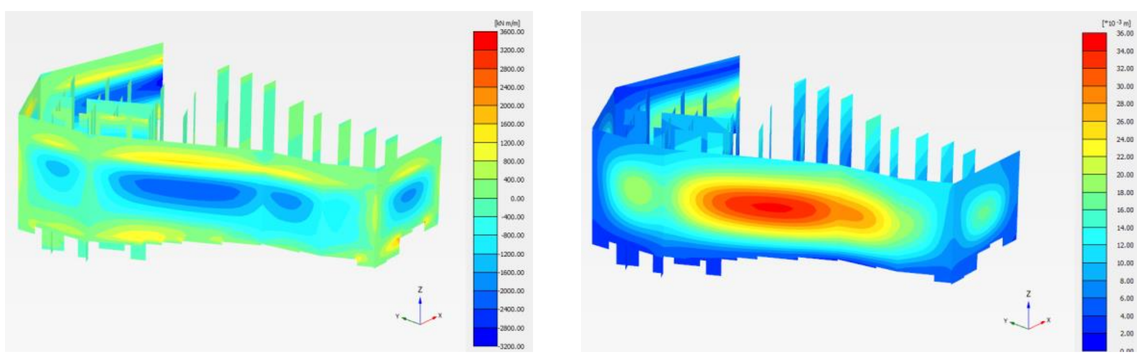


Figure 11. Typical lateral earth pressure diagram for a D-Wall.

The PLAXIS 3D outputs showing the structural behavior of the basement (i.e., D-Walls, slabs, and barrettes) were also provided for the assessment of the structural models and the calibration purposes. The PLAXIS 3D outputs included displacements, bending moments, and in-plane shear forces. Typical PLAXIS 3D outputs are presented in *Figure 12*.



(a) *(b)*

Figure 12. Example of PLAXIS 3D contour maps: (a) Out-of-plane bending moments and (b) Out-of-plane total displacements.

The deformations of D-Walls from PLAXIS 2D/3D models were used to prepare a detailed Instrumentation and Monitoring Plan (IMP).

SENSITIVITY ANALYSIS

A comprehensive sensitivity analysis was carried out to assess the impact of the following factors on the behavior of the basement:

- Soil strength parameters

- Ground water level
- Elastic modulus and modulus of subgrade reaction
- Dewatering levels
- Over-excavation
- Structural stiffness
- Surcharge

The modulus of subgrade reaction is not an intrinsic property of the soil or rock and depends on applied loads and material displacements. The modulus of subgrade reaction was derived from geotechnical analysis (i.e., PLAXIS models) and used in the structural analysis models as an input parameter for the short-term and long-term analyses. The outputs of the structural and geotechnical models were calibrated to confirm that the modulus of subgrade reaction values was appropriate for the design load and displacement conditions. The sensitivity checks indicated that the performance of the in-ground structures under loading is not significantly affected by changes in the modulus of subgrade reactions in the range of 50% to 200%.

The groundwater level inside the excavation was taken in the geotechnical design as 1 m below excavation level at each stage of construction. The sensitivity analyses were undertaken to assess the effect of variations in dewatering levels during construction on deflections and bending moments. The results of sensitivity checks indicated that dewatering to 4 m below excavation could result in a 10% to 15% increase in bending moments during the construction phase to mitigate the risk associated with the variations in dewatering levels. The monitoring wells were installed and ground water levels were constantly measured during the construction phase to ensure that the ground water level was maintained within the design limit (i.e., 1 m below excavation level) and mitigate the risks associated with the variations in dewatering levels.

INSTRUMENTATION AND MONITORING PLAN

The Instrumentation and Monitoring Plan (IMP) was prepared based on a comprehensive parametric study, sensitivity checks, and risk management considerations. The purpose of the IMP was to validate the actual performance of the basement during construction and compare it with the design estimates for design validation and construction risk mitigation. The measured parameters during the construction phase are listed below.

- D-walls, Core/Tower, and Basement lateral deformation
- Barrette vertical levels
- Tilt across the centerline of the core and tower barrettes
- Localized PRW lateral movement and capping beam level changes
- Groundwater levels inside and outside the excavation

The instrumentation locations were positioned based on the results of the geotechnical/structural numerical analysis and the following criteria.

- Maximum calculated movements/structural actions at critical locations
- Loading conditions and performance sensitivity
- Anticipated global behavior of the basement structure

A remote data logging system was used at the basement site, which enabled the real-time collection of data. The layout of installed instrumentation is presented in *Figure 13*.

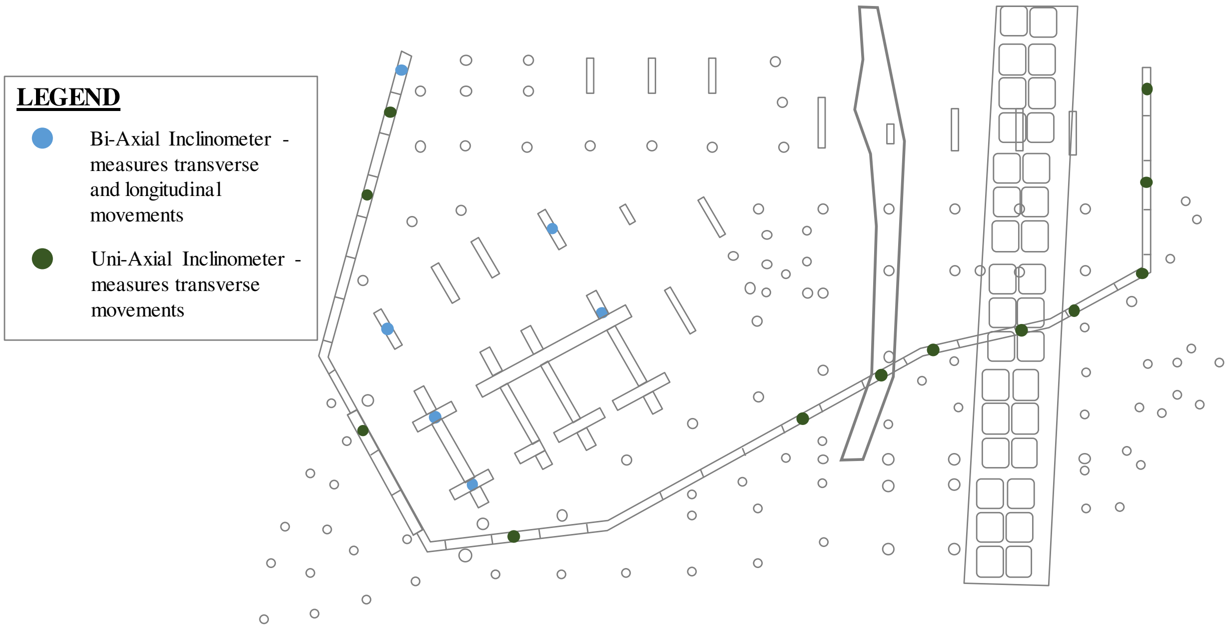


Figure 13. Layout of installed instrumentation across the Crown Sydney Hotel Resort basement.

The adopted monitoring trigger categories are summarized in *Table 5*. The trigger levels at each construction stage were assessed based on percentages of the calculated values from the numerical analysis.

Table 5. Adopted monitoring trigger categories.

Trigger Level Category	Condition	Function
Blue	Monitored parameters < 50% of calculated	Working level
Green	Monitored parameters ≥ 70% of calculated	Review level 1
Amber	Monitored parameters ≥ 100% of calculated	Review level 2
Red	Monitored parameters ≥ 150% of calculated	Alarm level

A comprehensive response plan was adopted if the reading on any instrument reached or exceeded the relevant trigger level category. The response plan consisted of a series of actions such as sending notifications to stakeholders, verification of movements, engineering review, and implementation of measures to control the movements and mitigate the associated risks.

CONSTRUCTION OF D-WALLS AND BARRETTES

The D-Walls and internal barrettes were constructed by Piling Contractors and Bauer Australia Joint Venture under a detailed inspection program developed by AECOM and Piling Contractors and Bauer Australia Joint Venture to ensure that the rock sockets met the following design requirements:

- Achieve specified socket roughness (R3 in accordance with Pells et al. 2002)
- Clean panel bases
- Socket founded in specified rock class

The required socket roughness was achieved by using hydraulic cutters with protruding teeth (*Figure 14*).

Figure 14. Hydraulic cutter with protruding teeth.

Bentonite slurry was used during the construction to stabilize the excavation. After completion of the excavation, reverse circulation techniques were used to achieve a clean panel base. In this method, the hydraulic cutter was kept at the excavation base level whilst pumping the debris and working bentonite out, while fresh bentonite was pumped in at the top of the panel. The suction pump capacity was approximately 500 m^3/hr, which provided adequate power to remove debris from the panel base. A bentonite slurry sample was tested for each panel after replacing the working bentonite slurry with fresh bentonite to meet the less than 2% sand content criterion.

Where boreholes were available at the panel locations, the final depth of the panel was defined using boreholes information. The rate of penetration of the hydraulic cutter into various sandstone rock classes was closely monitored and recorded. Daily excavation records and site observations were compared with the borehole logs. Comparing borehole logs with recorded site observations indicated that there was a correlation between the hydraulic trench cutter penetration rate and rock strength. The penetration rate halved from approximately 2 m per hour for sandstone class IV to 1 m per hour for sandstone class III (Azari et al 2020). The indicative average rate of penetration in the sandstone rock classes was used to verify the final depth of the panels where borehole information was not available.

The construction of in-ground structure started in mid-2016. The construction started with building bentonite plant, working platform slab and D-Walls, and internal barrettes guide walls. The construction continued with installing D-Walls, internal barrettes, and piles. The in-ground structure construction finished in March 2018.

COMPARISON OF NUMERICAL ANALYSIS RESULTS WITH MONITORING RECORDS

The actual behavior of the basement has been compared with the predicted deformations from both the 2D and 3D numerical analyses by selecting monitoring readings from two inclinometers installed along the D-Wall panels SW13.24 and SW16.33 (see *Figure 15*). The deformations of the D-Wall panels are compared in *Figure 16*.

For the purpose of comparing the numerical analyses with the monitoring readings, analyses were carried out applying a surcharge of 5kPa and considering no over-excavation to simulate the actual site conditions during the construction phase. The adopted elastic modulus for fill and alluvium materials is 40kPa and 20kPa, respectively. Both figures confirm that 3D numerical analyses reasonably predicted the behavior of the basement and there is a good agreement between the estimated D-Walls deformations and the inclinometers readings. Variations between the two can be related to uncertainties related to the properties of uncontrolled fill.

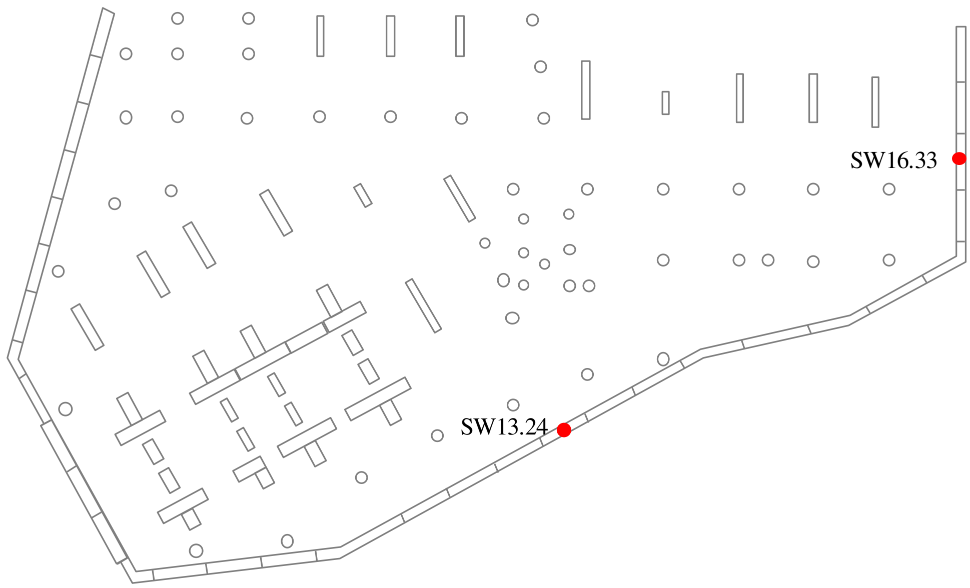
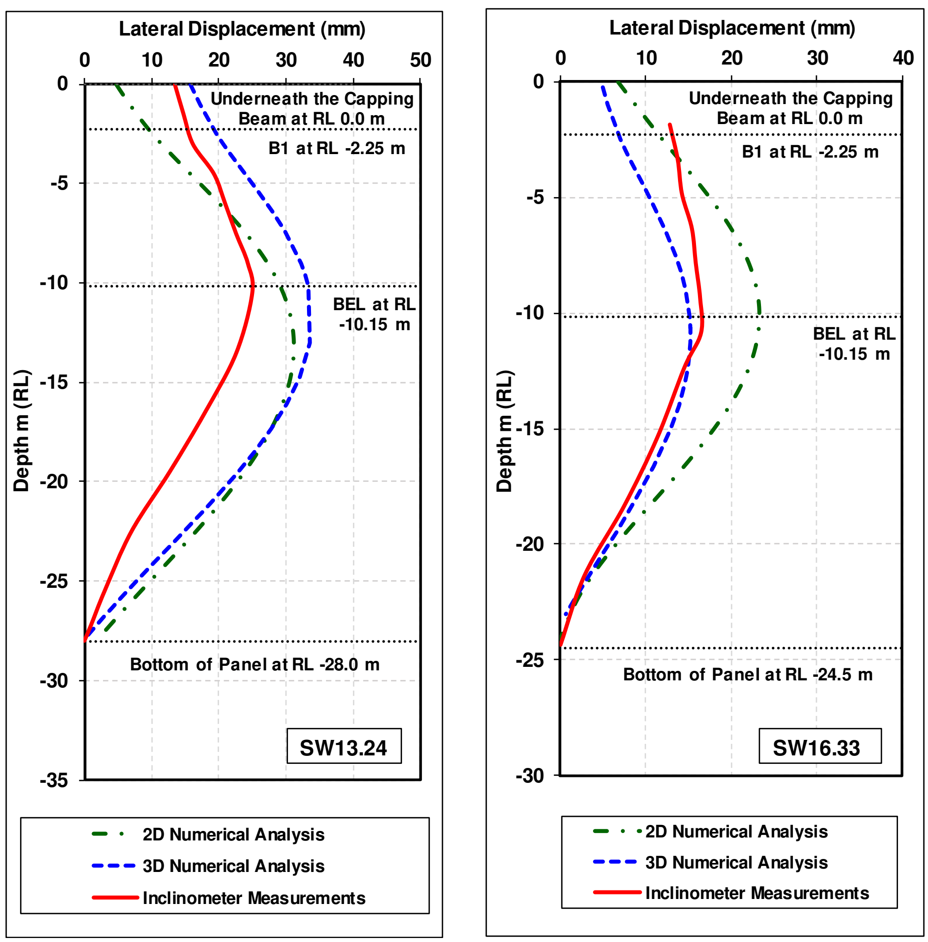


Figure 15. Location of selected inclinometers for comparison of recorded displacements with results of the numerical analysis.



(a) *(b)*

Figure 16. Comparison of inclinometer deformations with numerical analysis results; (a) SW13.24 & (b) SW16.33.

Panel SW13.24 is located away from the corners of the basement and has been less impacted by the corner effects (see *Figure 8*). Therefore, the behavior of this panel can be well modeled using a plane-strain model (i.e., 2D analysis), which is confirmed by the good agreement between the deformation curves from the 2D and 3D analyses in *Figure 16*(a). The *Figure 16*(b) shows that the 3D analysis is more accurate for the prediction of deformations at SW16.33, which is located between a corner and a fixed end of the basement and has significantly been impacted by 3D effects.

CONCLUSIONS

This paper presents the methodology adopted to carry out the numerical analysis and predict the behavior of the basement for the Sydney's tallest single mixed-use tower, with a complex configuration of the structural elements constructed in a highly variable and complex ground profile.

A combination of 2D and 3D numerical analyses were used to estimate the complex SSI of the basement. The 3D numerical approach was adopted to determine the total out-of-balance soil loads on the basement and assess the effects of the irregular basement shape, including the 3D effects and corner panels on the earth pressure and design action effects. The 2D numerical analysis was used to assess the out-of-plane SSI for the panels with plane-strain behavior, and to analyze the panels with increased socket lengths. The soil, water, and surcharge pressures from the 2D and 3D analyses were applied to the D-walls in the structural model, and the total out-of-balance soil force was calibrated to match the PLAXIS 3D reactions through the modification of soil and rock springs.

A comprehensive sensitivity analysis was carried out to assess the impact of variations in several parameters on the behavior of the basement, including soil strength, ground water level, modulus of subgrade reaction, dewatering levels, over-excavation, structural stiffness, and surcharge. The IMP was prepared based on the results of the numerical analysis and the sensitivity checks to monitor the actual performance of the basement during the construction. A comprehensive response plan was adopted by considering a series of actions when trigger levels were exceeded.

The perimeter retention walls were constructed under a detailed inspection program to ensure that the rock sockets achieved the required roughness and were founded in the specified rock class with clean panel bases. The excavations were stabilized using bentonite slurry, and the reverse circulation technique was used to achieve clean panel bases. The recorded penetration rate of the hydraulic cutter into various sandstone rock classes and the site observations were compared with the borehole logs to verify the rock quality in the panel sockets.

The comparison between the results of the numerical analyses and the inclinometer readings confirmed that the behavior of the basement aligned with the predictions in the numerical analyses. The results indicated that the behavior of the D-Wall panels that are less impacted by the corner effects can be well predicted by both 2D and 3D analyses. It was shown that the 3D numerical analysis is more accurate for the prediction of the deformation of the panels that are located close to the corners of the basement, which can significantly be impacted by 3D effects. Variations between the monitoring results and the predicted numerical deformations could be related to uncertainties related to the properties of uncontrolled fill as compared to the values adopted in the numerical analysis.

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