GEOTECHNICAL CHALLENGES FOR CONSTRUCTION OF DIAPHRAGM WALLS AND FOUNDATION OF SYDNEY'S TALLEST BUILDING, CROWN SYDNEY HOTEL RESORT

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ABSTRACT

Crown Sydney Hotel Resort is the Stage 1C component of Barangaroo South and is being developed as a single high rise mixed use tower of 72 stories (271m high), rising over a multi-level podium and a 3 level basement car park (total 75 levels). The Crown Sydney Hotel Resort basement retaining wall comprised 33 diaphragm wall (D-wall) panels and 36 barrettes for the foundation of the main tower and more than 130 bored piles (including bored compression piles, bored tension piles, bored sleeved piles and permanent plunge column piles). AECOM were engaged as designers of the foundation works by Piling Contractors Bauer Australia Joint Venture (PCBAJV) who constructed the foundation works as the D&C foundation contractor. The depth of foundation elements varied from 25 m to 50 m below ground level.

AECOM provided an initial concept design followed by a detailed design services and then, during construction, full time on-site geotechnical inspection of the basement diaphragm walls and foundations. This paper will focus on the challenges of geotechnical verification of diaphragm wall panel, barrette and pile foundation construction and how these challenges were met. During fulltime site inspection, hydraulic trench cutter penetration rates various sandstone rock classes have been measured and compared with the borehole data. Rate of penetration of the piling rig into the various sandstone rock classes, rock quality and rock apparent temperature were closely monitored and recorded as part of verification of the socket requirements. Monitoring, data collection and comparing the data with available boreholes, allowed AECOM to develop a method to reliably check the rock socket compliance with requirements across the site. Other geotechnical observations and lessons learned during the inspection of the pile, diaphragm wall and barrette construction are also presented in this paper.

1 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. Crown Sydney Hotel Resort will be Sydney's tallest building with 75 floors (including basements) and a height of 271m. Furthermore, Crown Sydney Hotel Resort will be the first 6-star luxury hotel. The project is currently under construction and is expected to be completed in 2021. Crown Sydney Hotel Resort (the Crown development site) is surrounded on 3 sides by other areas of the Barangaroo development precinct and, on the western side, is adjacent to Darling Harbour (see Figure 1).

A full top down methodology for concurrent tower and basement construction was adopted in order to meet overall project deadlines for Stage 1C with all foundation elements constructed in-situ from ground level (see Figure 2). The perimeter retention walls and the tower and podium foundations were installed from ground level, after which the ground floor slab was fully constructed, with the subsequent, concurrent construction of the tower and podium occurring simultaneously with the excavation and construction of the basement works. In order to facilitate the top down methodology, plunge columns were selected to support the ground floor slab and the construction of the podium levels above, while basement excavations were taking place below. The plunge columns consist of a rolled steel I-Section inserted into a concrete bored pile. The plunge columns are embedded into and are supported by the concrete bored piles with cut-off levels below the lowest basement slab level (B3 Slab). The plunge columns have prefabricated collar plates that are connected to the basement level slabs as the basement is constructed.

Figure 1: Crown Sydney Hotel Resort location (Courtesy of Wilkinson Eyre Architect)

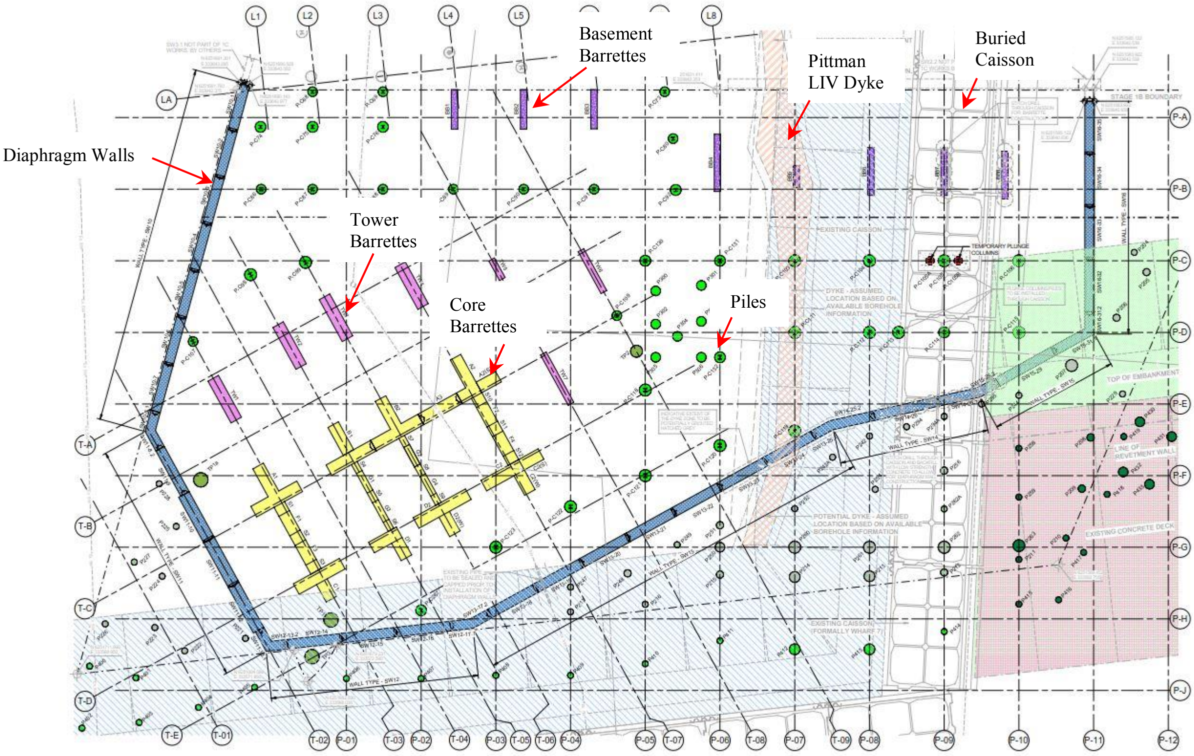


Figure 2: Crown Sydney Hotel Resort basement foundation plan

2 GROUND CONDITIONS

The development is located at a site that has undergone significant alteration in the past century. Prior to the early development of Sydney, the site was a part of Sydney Harbour and completely under water. Earliest development and use of the site had finger wharfs being constructed and, later, reclamation and conversion of the site to a container wharf has seen the site backfilled with highly variable, uncontrolled fill containing slag, concrete, bricks, wood and steel mixed in with sandy and clayey soils. The nature and type of fill and the methods of those times meant that the material was very porous with potentially large voids. The originally marine influenced erosional environment has led to buried sandstone cliffs across the site, resulting in rock levels that vary significantly over short distances across the site. Furthermore, the Pittman LIV Dyke, a caisson wall, buried rock revetments running across the site, and a suspended wharf deck all added further complexity to the site geology and challenges to the foundation design and construction methodology.

3 APPROACH TO ASSESSMENT OF GROUND CONDITIONS

In addition to the available borehole information, targeted ground investigations were conducted to better understand the ground conditions at the site, in particular along the alignment of the perimeter retaining walls and at the tower core foundations, and to also further assess the location and geotechnical properties of the Pitman LIV Dyke. In addition to previous site investigation works conducted by Lendlease and Crown for the tender, PCBAJV and AECOM carried out additional geotechnical investigations comprising more than 60 geotechnical boreholes up to 57.5m depth (Crown Sydney Hotel Resort; Geotechnical Investigation 2016). The site is underlain by fill and alluvium overlying Triassic age Hawkesbury Sandstone. The ground investigations confirmed that the fill and alluvium layers are highly variable in nature and in thickness. As a result, the rock level variation is considerable and in addition a few buried cliff lines with significant drops were identified at the site.

Figure 3 below shows the rock level variations and possible buried cliff lines along the diaphragm wall alignment. Even with the substantial amount of geotechnical information available (Figure 4), it was not possible to determine the subsurface conditions at the location of each foundation element or address all existing features, subsurface conditions or ground behaviour during the trench excavations.

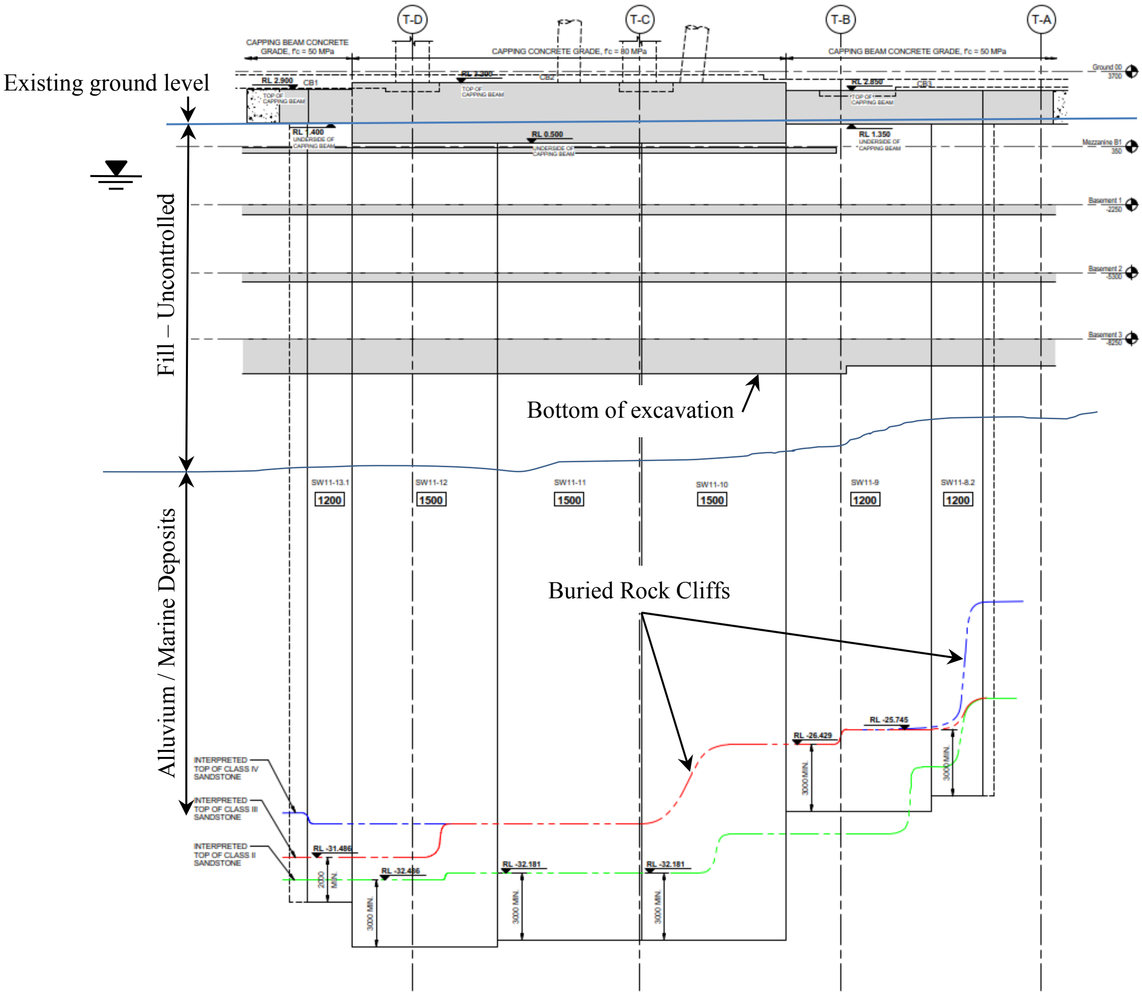


Figure 3: Typical diaphragm wall elevation showing ground conditions, rock level variation and buried cliff lines

Also, as the basement foundations were subject to large uplift pressures due to permanent hydrostatic loads, the rock socket verification was one of the key aspects of the design and construction of the diaphragm walls and piles. Therefore, PCBAJV and AECOM established an approach to reliably check the rock socket requirements where there was not enough borehole data at some of the pile and diaphragm wall locations. To achieve this for diaphragm walls and barrettes, geotechnical boreholes were drilled at nearly all panel locations prior to detailed design. Then, during construction, AECOM provided a full time on-site geotechnical engineer to observe, monitor, record and inspect excavation of diaphragm wall panels and barrettes to establish a correlation between grab and hydraulic cutter excavation and ground conditions recorded in the borehole logs. A similar process was followed for piles, where a number of targeted geotechnical boreholes were drilled at the exact location of specific piles prior to the construction. These piles were then constructed first monitoring and recording installation to establish a correlation between the drilling performance and ground conditions in the borehole logs. Then during full-scale pile construction, the AECOM geotechnical engineer attended installation of all piles and was able to verify that ground conditions complied with the design assumptions using the correlations established earlier.

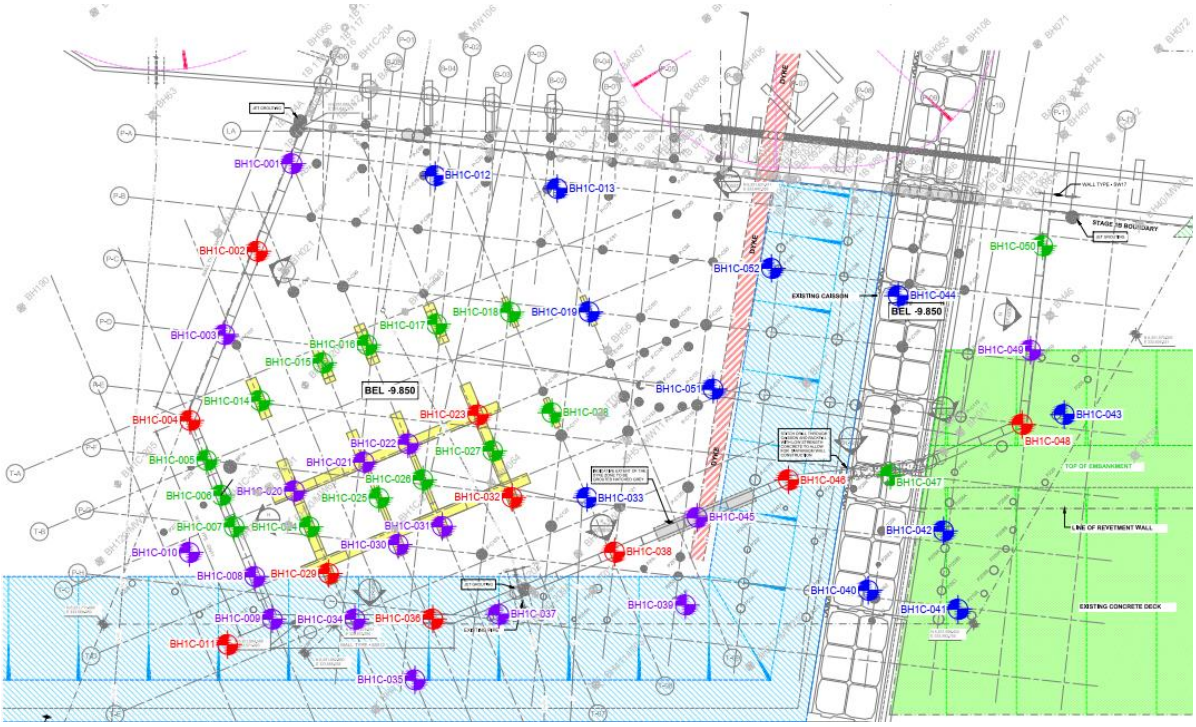


Figure 4: Locations of the targeted geotechnical boreholes (after Crown Sydney Hotel Resort 2016) in addition to the previously available boreholes

4 CONSTRUCTION METHODOLOGY

4.1 DIAPHRAGM WALLS AND BARRETTES

The diaphragm wall panels and barrettes were constructed by excavating the panels under bentonite to the required depth using clamshell grabs and hydraulic trench cutters (Figure 5). The diaphragm wall was constructed in a series of panels. Panels were constructed in a planned sequence, with every second panel constructed initially as primary panels. This is followed by construction of the secondary and closing panels between the previously constructed primary panels.

Due to the frequent variation of groundwater level (tidal effect) at the site, the variable depth of fill, and the high soil permeability, ground treatment was carried out along the diaphragm wall alignment and every barrette location to mitigate the risk of ground collapse and loss of bentonite during the diaphragm wall and barrette excavation and construction. Pre-treatment consisted of grouting at 2.5m centres with a flowable quick-setting grout to help bind the fill, fill any voids and reduce porosity

Due to the ground conditions, the excavation of diaphragm wall panels and barrettes was carried out in two stages. A clamshell grab is used to excavate fill, alluvium and weak or highly weathered rock in each panel. Upon reaching refusal of the clamshell grab, the grab is exchanged with the hydraulic trench cutter and the excavation process is continued into the rock using a hydraulic trench cutter to the designed level.

During excavation, the sides of the excavated trench are supported by bentonite slurry. Upon completion of the excavation, the working bentonite slurry is replaced by fresh bentonite slurry prior to installing the reinforcing cage and concreting. The panel is dipped with a weighted tape to verify that it has been excavated to the required depth. The steel reinforcement cage is lowered into the cleaned slurry and concrete is then tremied into the trench (Figure 6). As the concrete level rises, displaced bentonite is drawn off and pumped back to the bentonite plant for treatment and re-use.

(a) (b)

Figure 5: (a) Clamshell grab; (b) hydraulic trench cutter

(a) (b)

Figure 6: (a) Lowering steel cage; (b) pouring concrete

4.2 PILES

Similar to what was done for the diaphragm walls, due the variation of groundwater level (tidal effect) at the site, the variable depth of fill, and the high soil permeability, ground treatment was carried out at every pile location to mitigate the risk of ground collapse and loss of bentonite during the piling. Pre-treatment consisted of grouting the zone of the pile with a flowable quick-setting grout to help bind the fill, fill any voids and reduce porosity. Thereafter, the following stabilizing techniques were used:

- Single wall temporary casing with bentonite slurry
- Single wall permanent casing with bentonite slurry
- Temporary double wall segmental casings

Pile construction commenced by stabilizing the top section of pile hole using appropriately sized temporary or permanent casing. The starter casing was then advanced into the ground applying a combination of rotary torque and crowd force directly from the drilling rig. The starter casings were fitted with either interchangeable or permanent teeth (depending on the casing type) in order to advance the casing through hard soils (Figure 7).

Figure 7: (left) Single wall casings with permanent teeth, and (right) double wall casing with interchangeable teeth

(a) (b) (c)

Figure 8: (a) Auger, (b) drilling bucket and (c) core barrel

To excavate to the design depth a combination of drilling tools (e.g. Auger, Drilling Bucket and Core Barrel) were used (Figure 8). Initially, the auger was used to establish the location of the pile and excavate through the fill above the groundwater level. The Drilling Bucket was used for most of the pile excavation in the fill below groundwater level and when the bentonite slurry was introduced into the pile shaft. The sandy to clayey fill had inclusions of slag, concrete and steel over significant lengths of the pile shafts (Figure 9). In addition, the piles had to be constructed through existing buried caissons. A combination of Core Barrel and Drilling Bucket was used to core through these zones and also into the high strength rock that formed the pile socket. The Core Barrel initially cuts the core and the Drilling Bucket is then used to collect and bring out the rock fragments.

Rock commencement and final founding levels were confirmed by the geotechnical representative on site to confirm the rock socket requirements. During excavation, bentonite slurry was used to stabilize the hole. After completion of excavation, the base of the shaft was cleaned with a cleaning bucket and then the working bentonite slurry was completely replaced by fresh bentonite slurry. To assure that pile base was clean, bentonite slurry was tested to meet sand content requirement of less than 2%. The piles were dipped with weighted tape. The steel reinforcement cage was then lowered into the bentonite slurry and concrete was tremied into the hole (Figure 10).

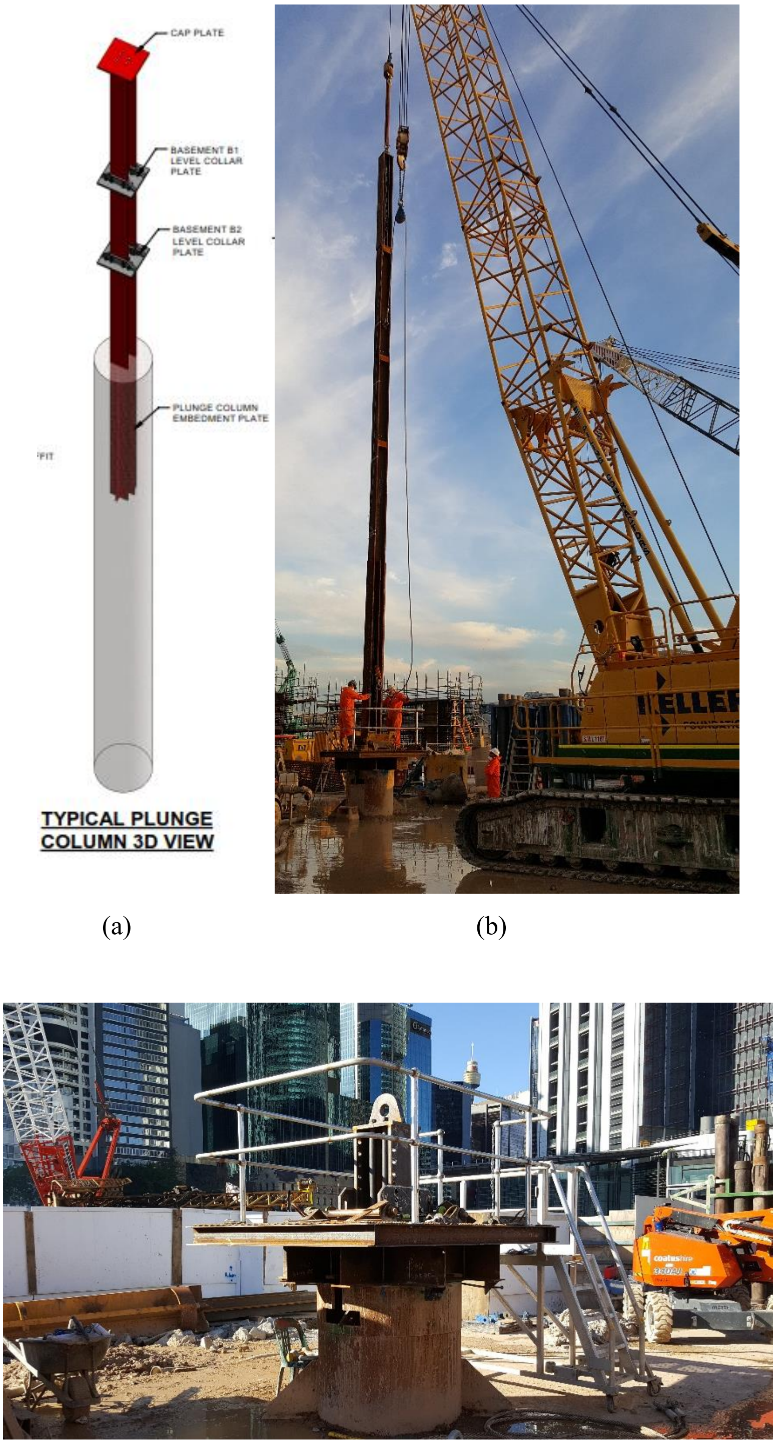
(a) (b) (c)

Figure 9: Examples of obstructions hit during construction

(a) (b)

Figure 10: (a) Lowering steel cage into bentonite slurry, (b) pouring concrete

Plunge columns were installed in selected basement piles as part of the top-down construction method to support the ground floor slab. To install the plunge columns, the I-Section column was inserted into the concrete of the completed pile no later than three hours after the concrete pour. The I -Section column was then suspended from a support platform positioned on top of the casing to precisely locate the plunge column. The support platform was left in place until the concrete reached the required strength to support the column. The shaft above pile cut-off level was then backfilled with stabilized sand and the support frame and casing were then removed. Figure 11 shows a 3D view and installation process of the plunge columns. Electronic tilt-meters were installed on the plunge columns during installation to maintain the strict verticality positional tolerances.



(a) (b)

(c)

Figure 11: (a) Typical plunge column 3D view, (b) lowering plunge column into concrete, (c) Suspended I-Section

5 GEOTECHNICAL VERIFICATION

5.1 DIAPHRAGM WALLS AND BARRETTES EXCAVATION

As mentioned previously, boreholes were drilled at nearly all diaphragm wall panels and barrettes. AECOM provided a full time on-site geotechnical engineer to inspect rock sockets to ensure that they met design requirements. Rock socket requirements for D-walls and barrettes were:

(a) achieve specified socket roughness (R3 in accordance with Pells et al. 2002),

(b) clean panel bases,

(c) Socket to be founded in specified rock class.

These are discussed below.

(a) Socket roughness

Prior to commencement of construction, PCBAJV demonstrated that their construction methodology, based on using hydraulic cutters with protruding teeth was able to achieve the required socket roughness (Figure 12).

Figure 12: Hydraulic cutter with protruding teeth

(b) Base cleanliness

To achieve a clean panel base, the hydraulic cutter was kept at the excavation base level whilst pumping the debris and working bentonite out by reverse circulation techniques 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.

(c) Founding Material

As discussed in section 3, boreholes were drilled at nearly all diaphragm wall panels and barrettes. At panels where boreholes were available, the borehole information was used to define the final depth of the panel. 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.

The comparisons indicated that the clamshell grab refusal generally occurred in sandstone class V or low strength sandstone class IV (Rock classification as per Pells et al. 1998). It should be noted that the quality of rock being excavated, the freefall weight of the clamshell grab and the clamshell grab operator are the key factors that have considerable impact on the depth of penetration that can be achieved in rock. For instance, clamshell grab refusal could be at shallower depth as a result of boulders and/or a lens of harder rock.

Comparing borehole logs with recorded site observations indicated that the hydraulic trench cutter penetration rate reduced with increase in rock strength. The penetration rate halved from approximately 2m per hour for sandstone class IV to 1m per hour for sandstone class III. The indicative average rate of penetration in the sandstone rock classes versus point load tests results is depicted in the Figure 13. Due to variations in rock characteristics (e.g. bedding planes, weak zones, and possible localized higher strength bands) the average penetration rates are indicative and can vary from site to site.

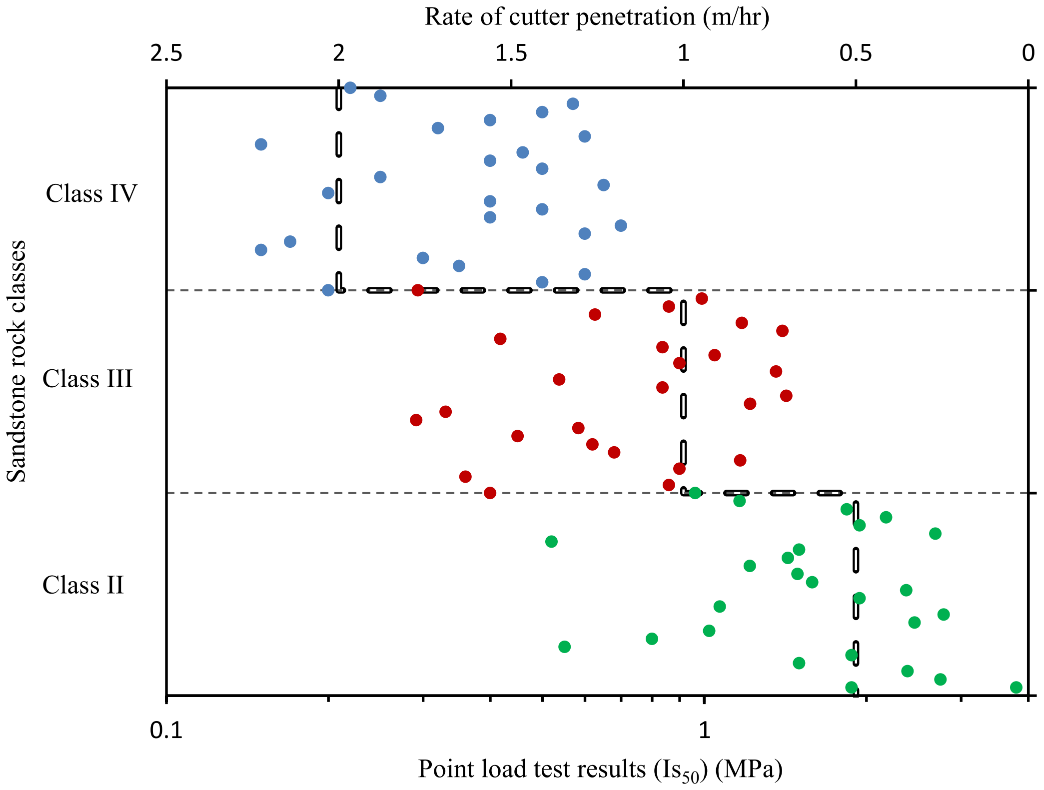


Figure 13: Hydraulic trench cutter observed penetration rate into Sydney sandstone rock classes

Dyke – Two of the diaphragm wall panels crossed the Pittman LIV Dike and one of the basement barrettes was located in the centre of the Pittman LIV Dyke. Pittman LIV Dyke rock material was Dolerite. The final design depth was required to be determined considering available geotechnical information, on site observations and daily excavation records. The socket requirement was 1.5m excavation into equivalent of class IV sandstone or better. Since the barrette was amongst the last panels to be excavated, there was a substantial record regarding the rate of penetration in various rock classes. Also, various parameters that could affect the rate of hydraulic cutter penetration into various rock classes had been observed and captured (e.g. operator). The experienced hydraulic cutter operator was requested to provide feedback regarding rock strength while excavating. Using the available geotechnical information, penetration rate, cutter input and engineering judgement, the top of rock and adequate socket length could be determined.

Clay lenses- Despite the substantial number of boreholes across the site, unidentified clay lenses were encountered at some panels during cutting. Hard clayey material cannot be easily excavated by grabs and neither by the cutters when the clay lenses occurred within the rock layers. Hence, more ground investigations could reduce the risk of encountering unexpected clay lenses. The hydraulic trench cutter was fitted with teeth for dealing with rock and not with clay and hence the clayey material sticks to cutter teeth and builds up to reduce the efficiency of the cutters significantly. (Figure 14). As a result, the cutter needed to be removed from the trench and the teeth cleaned frequently causing delays. As a solution, a different type of hybrid cutting wheel with interchangeable teeth which is more suitable for both clayey material and sandstone can be used if the clay layers are thick and expected to be highly variable across the site. However, the hybrid cutting wheel is less efficient if the clay layers are not encountered and only rock is occurring.

(a) (b)

Figure 14: (a) Cutter with clean teeth; (b) cleaning cutter teeth from clayey material

5.1 PILE EXCAVATION

AECOM provided a full time on-site geotechnical engineer to inspect rock sockets to ensure design requirements were met. Rock socket inspection requirements for piles were for confirmation of;

(a) achievement of specified socket roughness (R3 in accordance with Pells et al. 2002),

(b) cleaning of pile bases,

(c) Socket to be founded in specified rock class.

These are discussed below.

(a) Socket roughness

Prior to commencement, the contractor demonstrated that their construction methodology, based on using augers and roughening tools with protruding teeth was able to achieve the required socket roughness. Also, the performance of the drilling and roughening method to achieve the required roughness and shaft adhesion values was confirmed by reviewing and assessing the pile load test results (O-Cell tests) on the adjacent site (Site A) with the same ground conditions by the same contractor.

(b) Base cleanliness

Pile base cleanliness was assured by the construction methodology described earlier in section 4.2 (i.e. using cleaning bucket, replacing working bentonite with fresh bentonite, measuring bentonite slurry sand content and dipping piles at various points).

(c) Founding Material

A number of boreholes were specifically located at pile locations so that various parameters (e.g. rate of penetration of the piling rig into various sandstone rock classes, rock quality and rock apparent temperature) could closely monitored and calibrated against the borehole logs.

Rock quality was assessed initially by visual inspection and examining hand specimens of the excavated rock from the pile shaft and comparing with borehole logs at the piles. The lower the strength of the sandstone, the easier to mould the excavated material by hand. The excavated material tended to become friable when the strength was closer to sandstone rock class IV. Higher strength sandstone (sandstone rock class II) was excavated as sharp fragments, friable and not remouldable. Rock temperature was qualitatively recorded and compared with ambient temperature and weather as an indicator of rock strength. Higher strength rock required more torque, pressure and time for the drilling rig to excavate and consequently tended to generate much higher temperature and water vapour (steam) intermittently (Figure 15). These initial assessments and comparison with borehole logs allowed the on-site geotechnical engineer to assess founding conditions for subsequent pile sockets.

Figure 15: Excavated rock with high temperature and water vapour

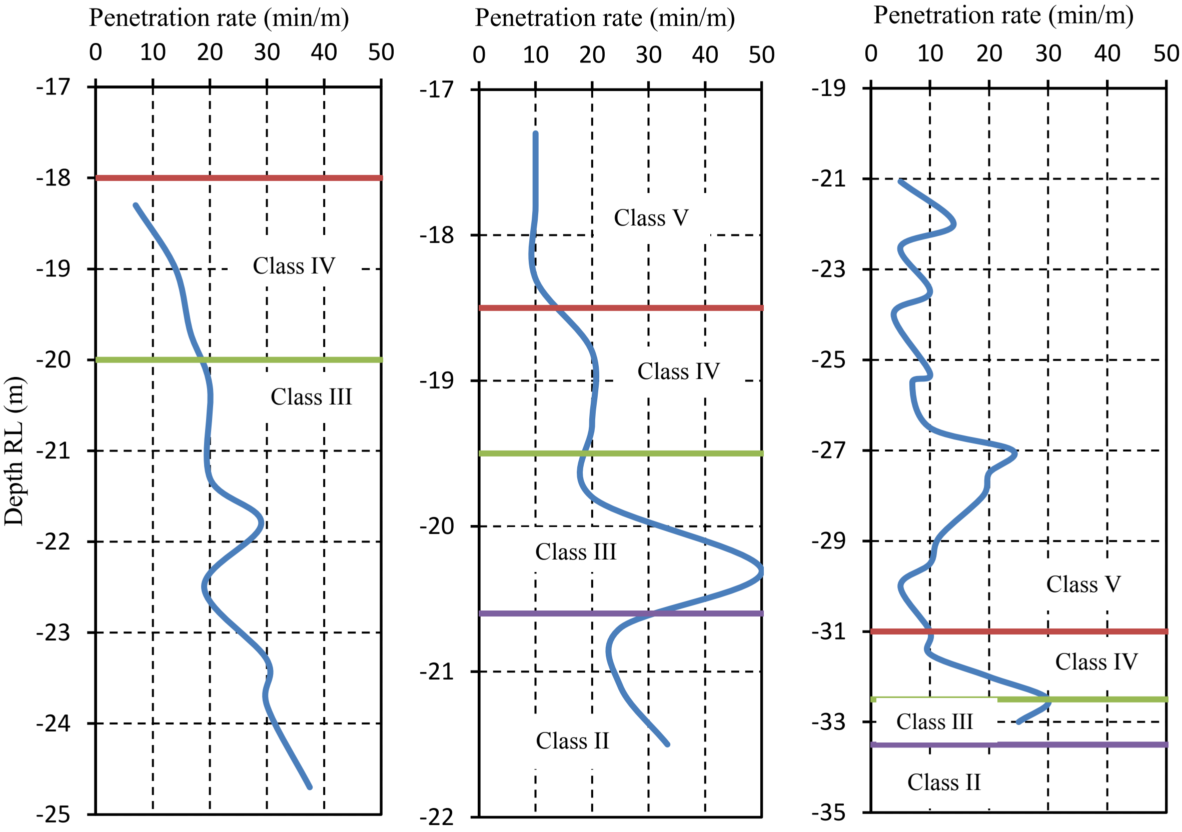


Figure 16: Example of excavation penetration rate in comparison with classified Sydney sandstone classes in nearby boreholes (piling rig) (after Azari et al. 2019)

The penetration rate for piles at borehole locations was monitored and measured to establish a baseline to assist the on-site geotechnical engineer to classify the rock on site. Figure 16 depicts three examples of excavation penetration rate compared with rock classification from nearby boreholes. Type of plant, operator experience, thin layers of high strength rock, clay lenses and tools efficiency play an important role in determining the penetration rate as can be seen in the spikes recorded in Figure 16. It is however evident that a meaningful relationship can be established between penetration rate and rock classes in Sydney sandstone for the particular conditions at this site with the higher strength sandstone requiring more time to excavate and resulting in a lower penetration rate.

6 CONCLUSIONS

Due to the highly variable thickness of fill and alluvium layers, and as a result of variable rock levels at the site and the limited geotechnical boreholes at the pile locations, an appropriate construction methodology as well as extensive site observation were required to achieve high quality construction.

To achieve adequate roughness in the diaphragm wall and barrette sockets proper excavation tools need to be employed. Subsequently, to clean the base of the panels, the hydraulic cutter is required to be kept close to the base and pump out debris and working bentonite. To ensure a quality concreting process, the process of pumping out the working bentonite while fresh bentonite is poured to the top of the panel needs to continue until the bentonite left in the panel has less than 2% sand content.

As part of an appropriate construction methodology, apposite drilling tools are required to achieve adequate roughness in the pile sockets. In addition, a cleaning bucket is required to clean the base of piles to ensure that they are free of coarse debris. It is then essential to also completely replace working bentonite (with high sand content) with fresh bentonite. Fresh bentonite with sand content < 2% avoids sand settlement at the base of the pile in addition to ensuring a quality concreting process as it has lower density and is easily displaced during concreting.

The design and construction intents were successfully achieved by close collaboration of AECOM design team, Piling Contractors Bauer Australia Joint Venture (PCBAJV) construction team and the fulltime geotechnical representative from AECOM.

The focus of this paper is purely on the construction aspects of the project and sharing the construction geotechnical lessons learned during the site inspections. A separate paper is under preparation on the design and performance of the basement and the foundations.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

Azari, B, Mirlatifi, S, Buys, H, Cullen, I (2019), Some Geotechnical Lessons Learned on Foundation Construction of Sydney's Tallest Building, Crown Sydney Hotel Resort, 13th Australia New Zealand Conference on Geomechanics, 1-3 April 2019, pp. 1157-1161.

Bertuzzi, B, Pells, P J N (2002), Geotechnical Parameters of Sydney Sandstone and Shale, Australian Geomechanics, Vol 37, No 5 December 2002.

Crown Sydney Hotel Resort (2016); Geotechnical Investigation 07 September 2016

Pells, P J N, Mostyn, G, Walker, B F (1998), Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics, December 1998.