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Ground anchor design and installation challenges within saturated Sydney basin alluvium: A case study for Sydney Metro - The Bays Station

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ABSTRACT: Excavation and retention of alluvium and sandstone was a significant design component for the construction of The Bays station box for the Sydney Metro West Central Tunnelling Package. The soil in the upper part of the station box comprises of deep saturated estuarine and alluvial sediments, sand, residual soil, and weathered sandstone, with a total thickness up to 22 meters. These soils were designed to be supported by anchored secant pile walls, founded within the sandstone bedrock. In this paper, the design process for the anchors is discussed, from the preliminary design to the testing requirements, as well as the description of different challenges that were encountered during the excavation of the station box in relation to the anchor installation and testing, and the mitigation measures adopted, including specific capacity assessments and installation of additional anchors.

1 INTRODUCTION

1.1 Project Description

The Sydney Metro West project represents a transformative addition to Sydney's public transport network, aiming to enhance connectivity and capacity between the city's central business district (CBD) and Greater Parramatta. Spanning 24 kilometers, this fully underground metro line is designed to double rail capacity along this corridor, offering a target travel time of approximately 20 minutes between the two centers.

A key component of this initiative is the Central Tunnelling Package, comprising 11 kilometers of twin tunnels between The Bays and Sydney Olympic Park, with 5 stations (Sydney Olympic Park, North Strathfield, Burwood North, Five Dock and The Bays). Awarded in July 2021 to a joint venture between ACCIONA Australia and Ferrovial Australia (AFJV), this $1.96 billion contract included the design and construction of these tunnels, with completion anticipated in 2025. The design was carried out by a joint venture between Jacobs Australia and TYPSA (JTJV).

Figure 1. Sydney Metro West – Central Tunnel Package General Layout.

The present paper focuses on the design and construction challenges of the anchored retaining wall constructed at The Bays, where some of the most challenging geological conditions along the project were encountered.

1.2 Geology

The Bays Station at the eastern end of the CTP alignment is located within a low-lying area of reclaimed land, 50m south of the current shoreline of Sydney Harbor (White Bay). Broadly across The Bays area, the geological sequence consists of the following:

- Fill: Uncontrolled anthropogenic fill of unverified origin, with thickness typically ranging from 0.5m to 1.0m.
- Alluvium: Predominantly of Holocene age, comprising interbedded clays and sands. It exhibits high variability and heterogeneity, ranging from soft, loose sands to stiff to very stiff clays, often with interbedded lenses and pockets. The alluvial layer extends to approximately 19 m depth at the western end of the station box.
- Residual Soil / Extremely weathered Sandstone: Soils were predominantly stiff or better clayey soils with variability in thickness that was interbedded with extremely weathered sandstone, that follows the stepped profile of the underlying sandstone bedrock.
- Highly to Slightly Weathered / Fresh Sandstone: Highly weathered sandstone transitioning to slightly weathered to fresh sandstone at depth, with some jointing and bedding planes evident at the upper part.

Additionally, the Great Sydney Dyke crosses the station box diagonally, trending approximately 120°, and intersecting the north and east wall. The dyke is comprised of variably weathered dolerite, with soil properties in its upper part and becoming less weathered and of high to very-high strength with depth, especially in the core. The complexity of this geology, represented in the Leapfrog model that has been included in Figure 2 (left) is described in detail in a separate paper [1].

1.3 Geometrical constraints

The station excavation was designed with a retaining system consisting of secant pile walls, with 1.2m diameter piles, spaced 1.5m between secondary piles, although other solutions (including D-walls and propped options) were explored during the design process. These piles are socketed into fresh sandstone and are laterally supported by active anchors. Due to the complexity of the geotechnical profile described, up to five levels of active anchors are installed in the area with the greatest alluvial thickness, located on the western side of the station box.

The high loads required by the system necessitate anchor lengths of up to 55 meters for the first row of anchors, located at the capping beam, with bond lengths up to 7 meters. Furthermore, geometric constraints imposed by the presence of TBM tunnels at the western wall (where the soil thicknesses reach more than 20m) required precise alignment of the tendon orientations, with adjustments to both vertical and horizontal inclinations to avoid geometric and structural interferences. The use of high-detail 3D geometric models, BIM modeling tools (using Revit, Aecosim and coordination software as Navisworks), and advanced analysis was essential to ensure a correct system design.

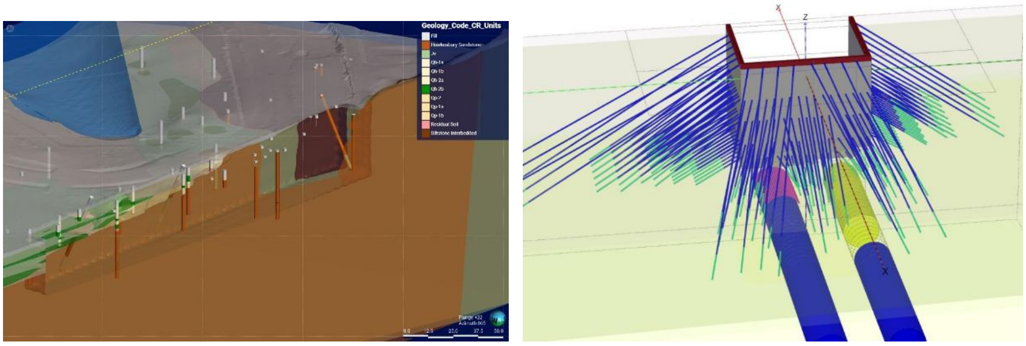

Figure 2- The Bays Station: Leapfrog model (left) and Plaxis 3D of the station box and TBM tunnels (right)

2 ANCHOR DESIGN

Grouted anchors with lengths in excess of 20m are not widely used and their interaction with the surrounding ground and modes of failure have not been extensively researched. The construction of the station box at The Bays for the Sydney Metro West Central Tunneling package comprised the drilling, installation and testing of anchors up to 58m in length, with inclination with the horizontal between 30° and 35° (as shown in Figure 3).The bond length of these anchors were designed for embedment in slightly weathered to fresh sandstone, however, the overburden above the rock head comprises of some man-made fill material but primarily of Holocene and Pleistocene alluvial deposits of non-homogenous clays and silty sands of up to 24m depth. In addition, the northern wall anchors had to pass through the Great Sydney Dyke before reaching competent sandstone.

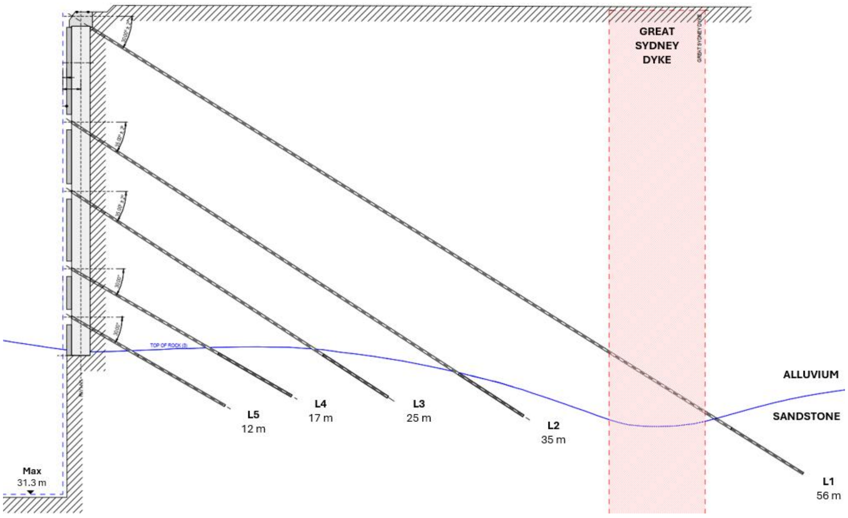

Figure 3. Anchored piled wall – Western part of the station box. Typical section

2.1 Anchored Pile Wall Description

Based on the results of the soil- structure interaction undertaking using FE analyses from which the axial forces experienced by the anchors were determined, active multistrand cable anchors were determined to be suitable for the secant piles. Where localized jointing is observed on the exposed sandstone rock faces where the secant piles do not extend to, passive bar anchors were proposed to be used to resist wedge movements, especially for areas around the dyke and the sandstone interface on the northern wall.
There are certain areas around the station box where geotechnical and geometrical conditions are complex because of the presence of deep-seated alluvium, the presence of the Great Sydney Dyke and the presence of the tunnel nozzles. This is summarized in Figure 4.

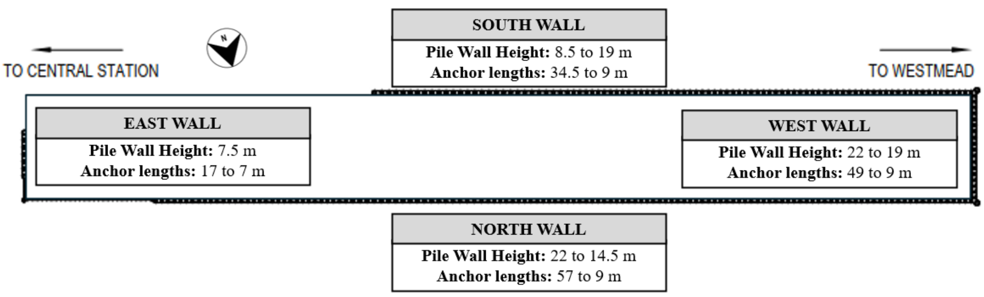

Figure 4. Station layout – Anchor distribution

It is noted that in the design of anchors, the effect of horizontal skewing is understood to increase the load component arising out of load deviation from the normal direction. The corresponding increase in load has been considered in the design of the anchor capacity.

2.2 Design considerations

The modes of failure for grouted anchors [2] working in tension are: failure within the rock mass (1), failure of the steel tendon or cables (2), failure of the rock to grout bond (3) and failure of the grout to tendon bond (4).

The anchor design verification has been undertaken in accordance with AS5100.3 to resist these modes of failures, including a potential gradual long-term deterioration as well [3].

For heavily loaded anchors, that require long bond lengths, it is noted that tests on conventional anchors have shown that the stress distribution along the bond length of an anchor is non uniform [2] [4]. This can cause that, for bonded lengths larger than 6m, a progressive debonding phenomenon (also known as the zipper effect) is noted to occur [5]. The bond stress concentration zone reaches the distal end of the anchor just before failure and this reduces the effective ultimate bond stress for anchors.

Keeping the above in mind, anchor pull tests (proof tests) were undertaken within the vicinity of The Bays Station to better understand the anchor resistance in relation to the ground conditions relevant to design. The undertaking of these tests aimed to provide information on (a) the bond capacity of an anchor at the anchor-grout interface and (b) the bond strength at the grout-rock interface to assess the required resistance for anchor working loads.

Five proof tests were undertaken by a specialist subcontractor. The tests were carried out in accordance with recommendations provided in TfNSW's guideline IC-QA-B114; Tests were carried out for anchors with 12 and with 15 strands and in seven load increments over eight cycles.

The ultimate bond stress values in sandstone and other pertinent anchor design parameters have been summarized in Table 1 below:

Table 1. Rock Mass Parameters for Rock bolts and Anchors

Rock Units	Ultimate Bond Stress* (kPa)	UCS (MPa)	Intact Rock Modulus (GPa)	Unit Weight (kN/m3)	Poisson's Ratio
Sandstone Class I	2000	30	6	24	0.2
Sandstone Class II	2000	20	5	24	0.2
Sandstone Class III	2000+	15	3	24	0.25
Sandstone Class IV	500	8	2	23	0.25
Dykes and Altered rock	250 – 500	8	2	24	0.25

*Failure along grout-steel interface governs in all cases.
+ Optimized from results of pull-out tests.

To account for progressive debonding, an effective bond stress adjusted by an efficiency factor (up to 15%) is often adopted for bond lengths longer than 7m and up to 10m maximum. However, the requirement for an efficiency factor is negated in those cases where bond stress is considered to be mobilized only when shear displacement occurs or in those cases where only residual values of bond stresses were considered for design.

2.2.1 Durability

The required design life for all anchors was a maximum of ten years, as per the requirements of the project's Particular Specification. Although these anchors are subject to the design and testing requirements of TfNSW specification B114, for anchors with a design life of 10 years or less, the corrosion protection requirements of B114 clause 3.2.1 do not apply.

Due to the proximity of the station box to White Bay, generalized or localized corrosion arising as a result of chloride ingress from saline groundwater, acid sulfate attacks and microbially influenced corrosion is anticipated during the design life of all active and passive anchors. Saline intrusion modelling was therefore conducted for the station box piles and anchor elements. The results of the analysis indicate that seawater is expected to affect the station box excavation within 2 years of construction, with the highest exposure anticipated for the northern wall.

Corrosion protection was therefore provided through close fitting plastic sheaths with corrosion inhibiting grease or lubricant over the free end. Within the bonded end, the tendons were designed to be fully encapsulated with grout, with a minimum grout cover of 20mm.

2.2.2 *Category/Reduction factors based on AS5100.3*

For serviceability limit state design, the fundamental approach is that all applied loads are increased by a load factor (ψ) to give a design action with a very low probability of occurrence, which the resistances are reduced by a strength factor (φ) to give a design strength (R*) which must be greater than or equal to the design action [6]. Multiplying a design value by a strength or capacity factor implies that the variance of a given parameter will be directly proportional to the characteristic value of that variable and is therefore constant for a wide variety of possible actions and resistances.

Anchor verifications have been undertaken in accordance with AS5100.3, with relevant dispensations made as per requirements provided in the project's Particular Specifications with calculations to ensure that the anchors will not fail under any of the three failure mechanisms identified previously. The design process is summarized in Figure 5.

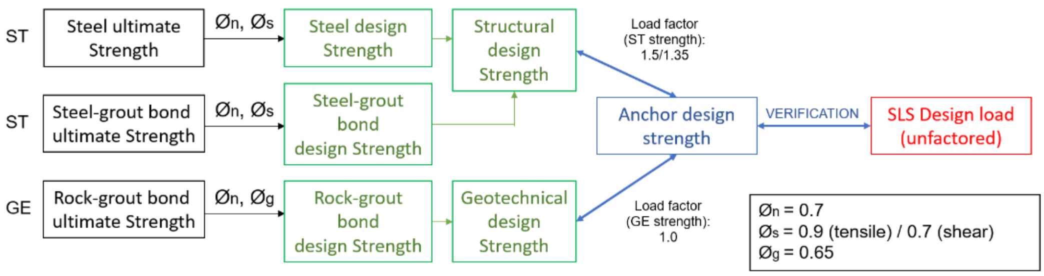

Figure 5. Anchor design flowchart (As per AS5100.3 and project specifications)

3 ANCHOR INSTALLATION

3.1 *Anchor installation in alluvium: Challenges*

The execution of ground anchors in secant pile walls within heterogeneous alluvial deposits composed of clay and sand, and below the groundwater table, presents notable geotechnical and construction risks.

In saturated sands and loose alluvial materials, the risk of borehole collapse or uncontrolled water inflow during drilling is significant. This can compromise the integrity of the borehole, affect anchor alignment, and may result in loss of ground or the need for additional stabilization measures. Additionally, the migration of soil particles (especially sands) due to the water inflow through the hole created in a watertight structure can cause unexpected settlement and, if ignored, potentially collapse.

During the excavation of The Bays station, excessive settlements were detected in some areas, mainly in the western and southern wall. As shown in Figure 6, the movement was appreciated to accelerate during some periods and stabilize in others (as shown in the settlement markers).

It was determined that the cause of the settlement was due to the soil migration caused by the adopted drilling methodology. This was consistent with some site observations reporting high water inflows through the anchor holes, in some cases accompanied by an influx of some fine-grained soils, and this settlement observations correlated with the timeline of anchor drillings for anchor rows two and three (the two first rows under the groundwater level).

Several Monitoring Action Team (MAT) meetings were held, including representatives from the construction and design team, to analyze the information available and determine the best course of action to correct the drilling methodology and prevent further movements during the excavation of the subsequent rows. Subsequent corrective measures included the adoption of blast bags to minimize migration of fines during anchor drilling and continuous ground monitoring during anchor drilling and investigations.

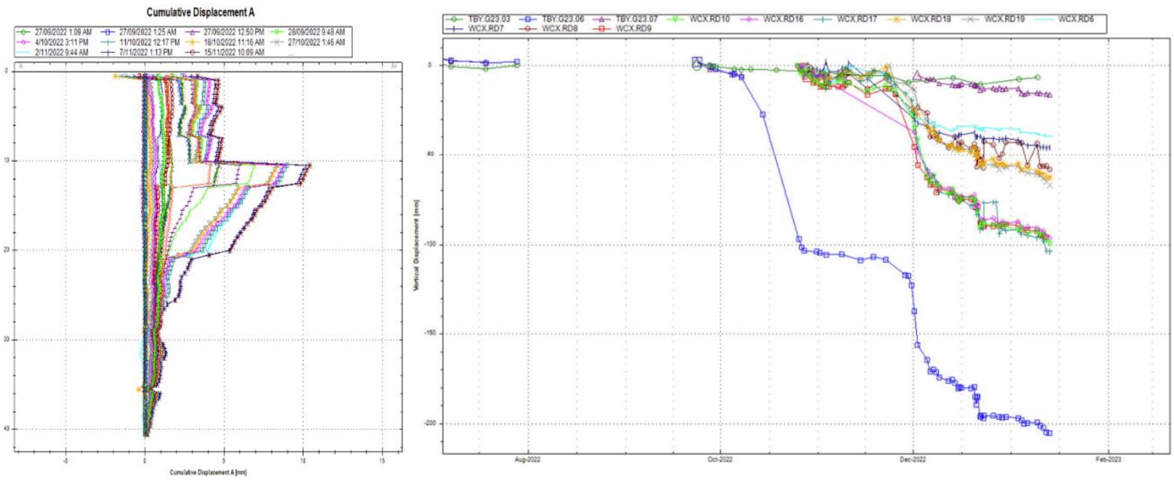

Figure 6. Up–Inclinometer readings (per date) and Settlement marker readings.

3.2 Other challenges during installation

Due to the characteristics of the soil material and the length of the required pilings, several challenges were faced during the execution of the anchors.

- Clash with reinforcement: The design of the pile was contemplated taking several measures into account to avoid reinforcement clashes, including specifically positioning the reinforcement cages with respect to the excavation, and the adoption of one sacrificial bar. In spite of these measures, there were some occurrences of this type of clash, most of which involved the entrapment of anchor drilling bits. Repositioning of anchors and a reassessment of pile capacity was undertaken in these instances.
- Temporary casing retrieval issues: In some instances, the retrieval of the temporary steel casing for piles was hindered due to unknown obstructions encountered in the anthropogenic fill layers. In those instances, the anchor installation and the overall behavior of the pile wall was reanalyzed to assess how the pile capacity reduction due to the temporary casing being in place affected the overall structural design.
- Concrete pouring issues: Due to weather conditions, pile lengths and other variables, some defects were identified during pile integrity testing, or when the station box excavation progressed. The effectiveness of the original anchor design and the structural performance of the pile wall needed to be reanalyzed in some situations.

The presence of the problems described above required different actions from the design perspective, including the relocation of the anchor or a redesign of the wall section considering a different anchor arrangement. The implementation of these measures is described in section 5.

4 ANCHOR TESTING

4.1 Active anchor testing requirements

Proof testing, load testing and acceptance testing were undertaken as per the requirements cited in Annexure B114/L – Testing Procedures for all multistrand anchors and bar anchors (steel and GFRP) for the following requirements:

- Proof tests: Carried out to provide information for the final design (anchor capacity at grout/ground, grout/tendon or duct/grout interfaces). These tests were carried out prior to ground anchor work commencing at the site. Five tests were undertaken.
- Load tests: These were carried out in a similar approach to the Suitability tests presented in TfNSW Specification B114 to verify the performance of represented working anchors, with

a minimum of six load cycles for the 2% of the total number of anchors tested for each distinct anchorage ground condition. Approximately 15 anchors were tested.

- Acceptance tests: These were carried out on all remaining anchors to demonstrate the short-term ability of the anchor to support the proof load with sufficiently low creep rate or load loss, and the efficiency of load transmission to the bonded length.

4.2 Failures during anchor installations

During anchor installation and testing, sixteen instances were noted where acceptance tests failed. These were noted to be either instances where the rate of residual load loss exceeded 1% during acceptance tests or when the strand displacement or elongation was higher than the predicted theoretical extension.

Most of the acceptance test failures were observed at the southern wall of the station box, where anchor lengths were relatively shorter than the northern wall and geological anomalies were not observed. However, the total number of acceptance test failures only made up 2% of the total number of anchors that were installed.

The failures during acceptance testing can likely be attributed to the following factors:

1. Some field tests has shown discrepancies between theoretical and observed extensions in anchor strands occurring due to variations in the Youngs modulus values of steel tendons tested in the laboratory versus when the tests are carried out in field [7] [8] [9]. A possible explanation is that stressing multistrand tendons in the field takes longer than laboratory testing of individual strands, and during this time, plastic deformation occurs in the steel, yielding a larger extension and therefore correspondingly a lower Youngs modulus value.
2. In anchors with long, sheathed tendons enveloped in a protective grout column, friction in the free length of the anchor can be developed around the grip assembly of the anchor head which tends to reduce the measured extension. It is estimated that only about 70 percent of the applied prestress was transmitted along the entire tendon length [10]
3. Over drilling or under drilling the anchor installation borehole is also known to affect the free length extension of the anchor. In cases of under drilling, the strands may experience slack in the boreholes that may be incorrectly logged as excessive elongation.

5 MITIGATION MEASURES ADOPTED

5.1 Anchor capacity assessment

During acceptance testing, in some cases, the strands became damaged or began to extend beyond their theoretical extension limits and the anchor would not load to the test load.

In order to assess the as built conditions, the acceptance test results, the anchor drilling records, and grout compressive reports were reviewed. The acceptance test records provide invaluable information on the available bond length of the anchors; the bond length of the anchors was downgraded when the measured elongation exceeded 110% of the theoretical elongation, as per B114. In addition, the anchor drilling records provided information on where the rock was encountered in comparison to design assumptions.

The review provided information to reassess the remaining capacity of the anchors to support updating the numerical model and reassess the geotechnical capacity of both pile and anchors. The anchors are checked against the three failure mechanisms to ensure no failure will occur upon the changes to anchor forces and strands. Next, the as-built wall section is checked for its structural capacity to sustain the applied forces imposed by the soil and surcharge and satisfy the strength and safety requirements for those sections. If these checks are satisfactory then the anchors can be used as is, otherwise it was advised that an additional anchor was to be installed.

5.2 Additional anchor installation

Where the review of acceptance test results showed the anchor does not have the adequate capacity as required by design, or an unforeseen obstruction was encountered; a replacement anchors with same number of strands and same hole size and orientation as design and same bond length, 0.5m above/below the design level, was used. To not exceed the capacity of the pile or the maximum capacity of the installed anchors, the maximum excavation level before the installation and stressing of the replacement anchor was limited to 0.5m below the replacement anchor level. In other cases, the relocation of the anchor was required outside the prescribed allowance, requiring a verification of the final situation to check that the loads were within the limits of the pile wall. In all scenarios, the redundant anchor was decommissioned, and the hole was grouted and sealed against the water ingress.

6 CONCLUSIONS

The design and construction of the anchored retaining wall at The Bays Station for the Sydney Metro West project presented significant challenges due to the complex geological conditions and the presence of deep alluvial deposits and the Great Sydney Dyke. Furthermore, the high groundwater table posed further complication to anchor installation leading to occasional sediment and water inflow during drilling of anchors which led to unexpected settlement locally.

The implementation of secant pile walls supported by active anchors required diligent planning and innovative solutions to address issues such as anchor installation difficulties; Digital modeling tools and adjustments to anchor lengths and orientations were adopted to avoid structural interferences such as TBM and anchors.

To ensure the anchor performance, comprehensive proof testing, load testing, and acceptance testing were performed. The test results provided information in identification and mitigation of anchor failures. The mitigation measures included additional anchor installations and/or the remaining capacity assessments of anchors together with assessment of the as built sections to ensure structural integrity remains satisfactory.

Overall, the successful execution of this project highlights the importance of adaptive design strategies and collaborative efforts in overcoming geotechnical challenges using MAT meetings. The lessons learned from this study will contribute to future projects involving similar geological conditions and complex anchor installations.

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