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Our ref: SYDGE287887AC

Aqualand
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Attention: Sam Jenkins

Quality information

Revision history

Revision	Description	Date	Originator	Reviewer	Approver
v1 draft	Preliminary Advice	15 Oct 2021	PKW	PS	PS

Distribution

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v1 draft	1	PDF	Sam Jenkins	15 Oct 2021

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1. INTRODUCTION

Tetra Tech Coffey Pty Ltd (Coffey) has been requested by Aqualand Pty Ltd (Aqualand) to provide preliminary geotechnical advice on the design of tension piles to resist hydrostatic uplift of the proposed 5 level basement for the Barangaroo Central Development, together with interaction effects of the basement excavation on the adjacent Sydney Metro Barangaroo Station.

This draft report full fills Items 2 and 3 of our fee proposal SYDGE287887AA dated 01 June 2021.

2. DESIGN BASEMENT LEVEL

We understand that the proposed Barangaroo Central Development will have 5 below-ground basement levels with the top of the B5 slab at a level of RL -11.5 mAHD. Allowing for an approximate hydrostatic slab thickness of 1.3 m (TBC) at this stage, the base of the B5 slab is expected to be at approximately RL -12.8 mAHD.

3. DESIGN GROUNDWATER LEVEL

The highest astronomical tide (HAT) level measured at Fort Denison is about 2.1 m which corresponds to RL1.18 m AHD approximately. This HAT level does not allow for storm surges and climate change effects. For the Barangaroo South projects (Stage 1A Commercial and Stage 1B Residential), a long-term design groundwater level of RL 2.575 mAHD was adopted, and we understand from Taylor Thomson Whitting that they have adopted a level of RL 2.435 mAHD for preliminary design.

4. GEOTECHNICAL MODEL

The site of the Barangaroo Central Development is underlain by fill, alluvial sediments and minor residual soils overlying Hawkesbury Sandstone. From the available geotechnical data, the top of Class V, IV, III and II Sandstone[1] has been plotted as shown in Figures 1 to 4 that follow.

[1] Rock Classification based on Class V Sandstone

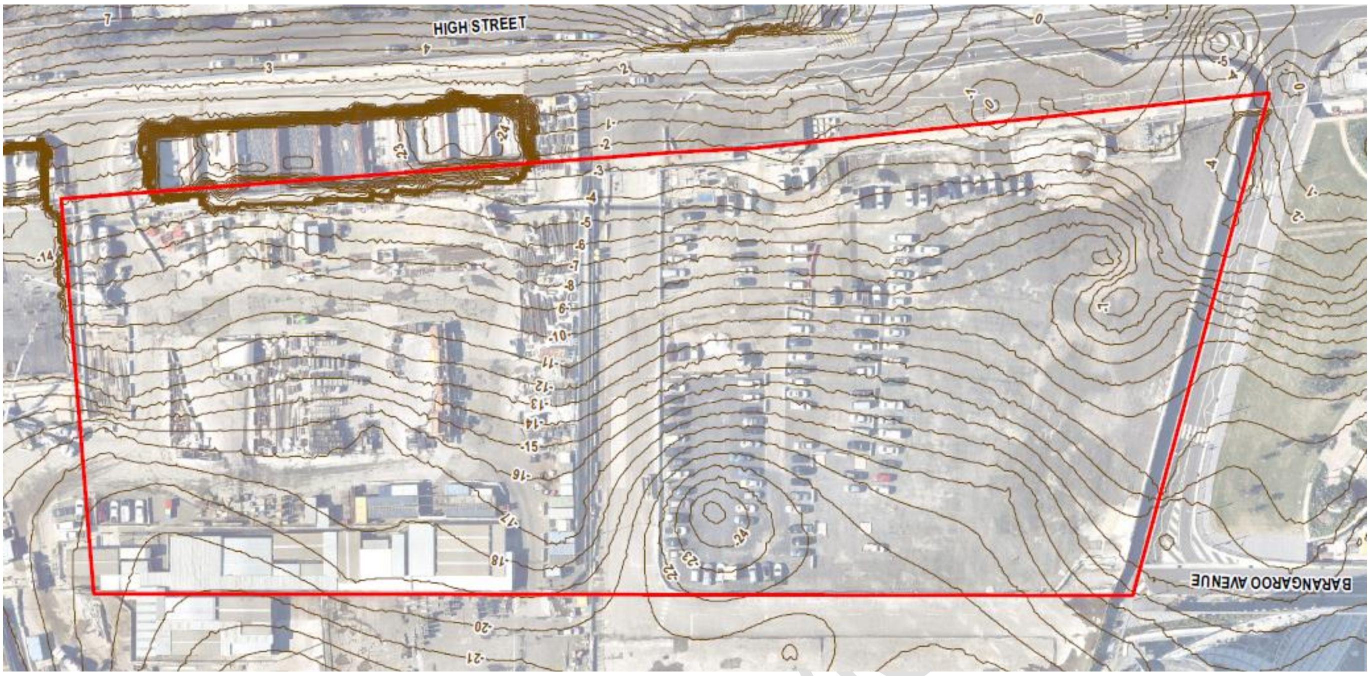

Figure 1 – Top of Class V Sandstone

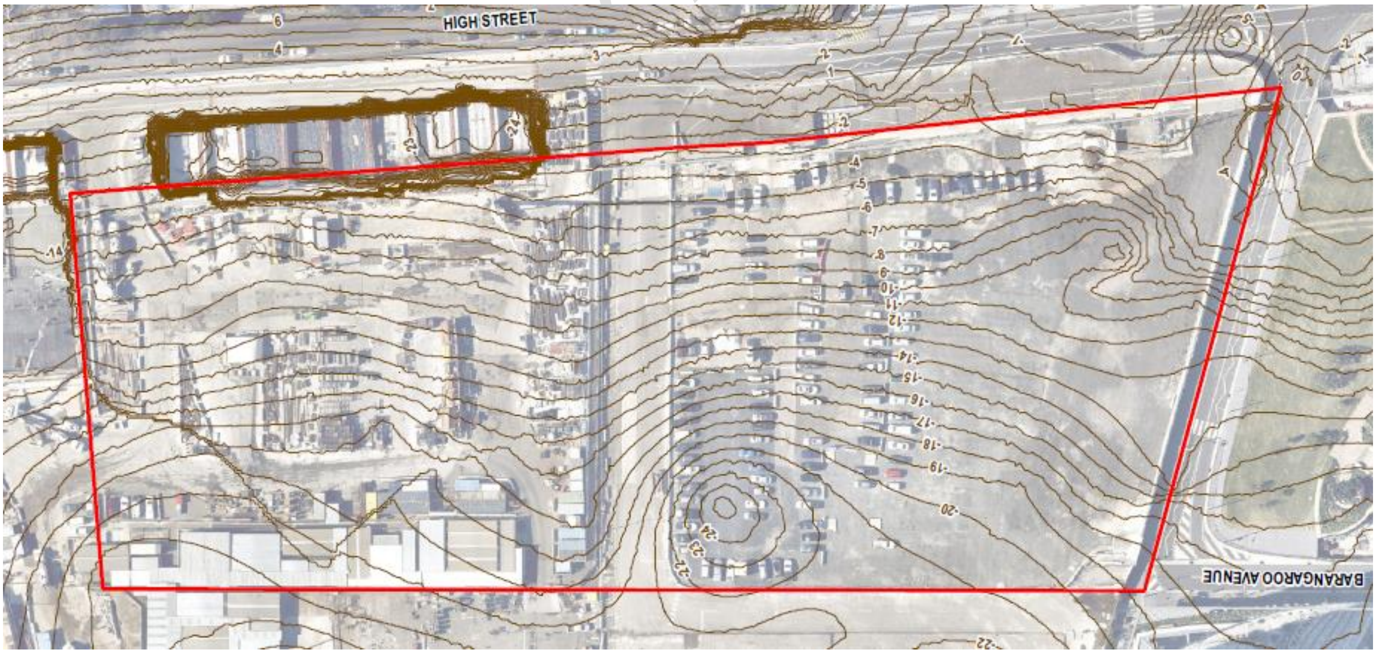

Figure 2 – Contours of Top of Class IV Sandstone

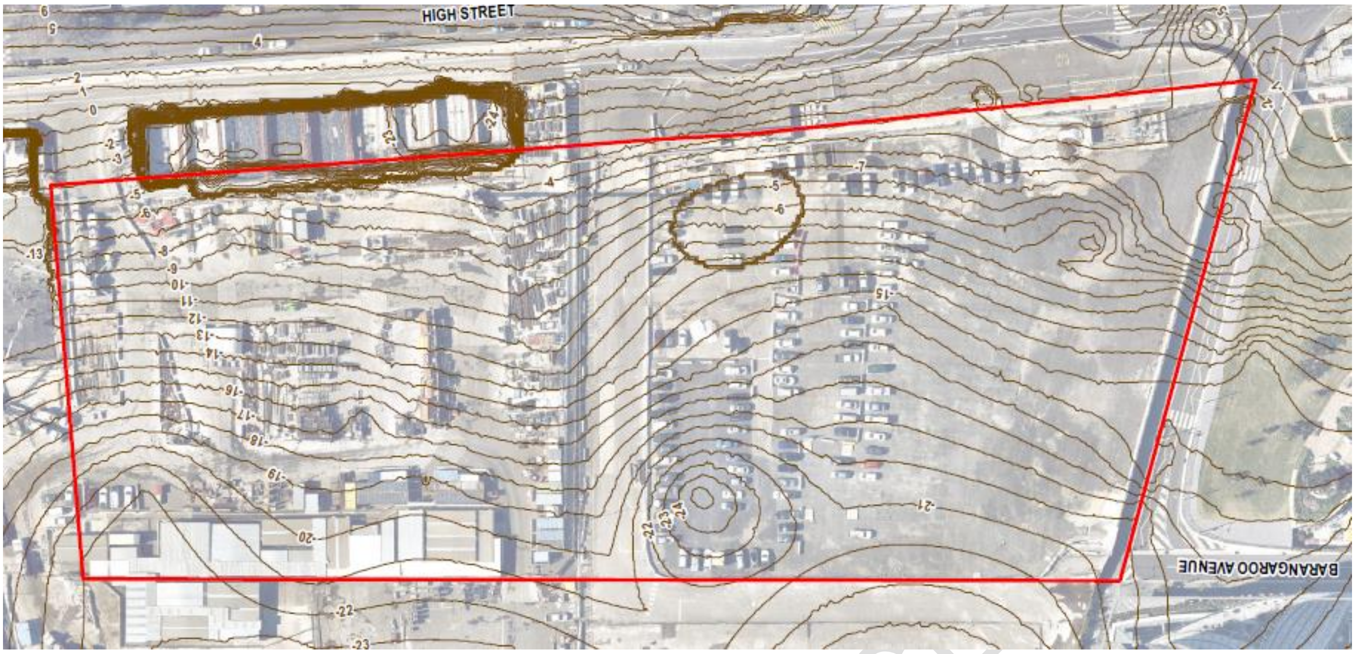

Figure 3 – Contours of Top of Class III Sandstone

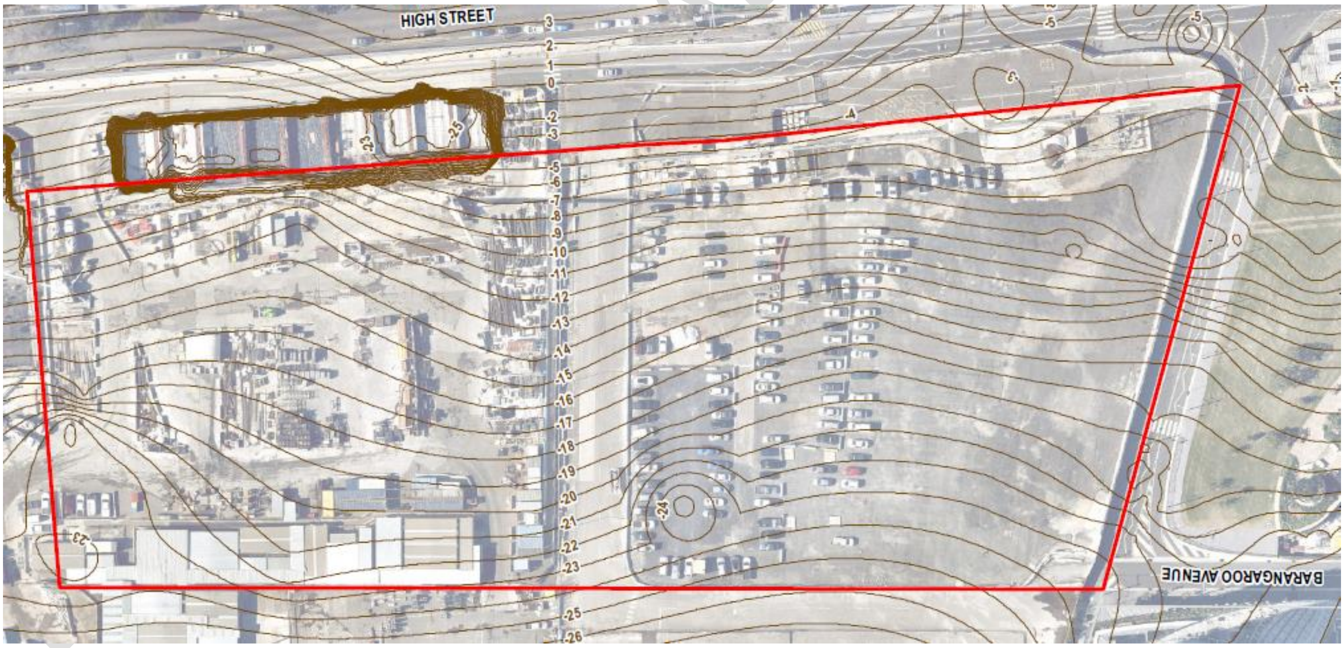

Figure 4 – Contours of Top of Class II Sandstone or better Rock.

The above rock contours are considered to be the most relevant in relation to tension pile design.

5. TENSION PILE DESIGN

5.1 DESIGN UPLIFT PRESSURE AND TENSION PILE LOADS

Using a maximum long-term design groundwater level of RL 2.575 mAHD, the long-term hydrostatic pressure at the base of the B5 slab will be approximately 154 kPa, whereas the use of RL 1.18 mAHD for the HAT level would result in a reduced pressure of 140 kPa.

The AS1170 requires the application of load factors of 0.9 to be applied to dead load resisting uplift and 1.2 to be applied to hydrostatic pressure loading. To avoid undue conservatism, we suggest that the following load factor be applied with corresponding ULS uplift pressures:

Table 1: Recommended Design ULS Load Factors and Uplift Pressure

Groundwater Level	Design Groundwater Level mAHD	Load Factor	Design ULS Uplift Pressure (kPa)
HAT	1.18	1.2	168
HAT + long-term Climate Change Allowance	2.575	1.1	169.4

Based on the above assessment, we recommend that a factored design ULS uplift pressure of 170 kPa be adopted, although this may be revised following review by Aqualand and TTW, and finalisation of the B5 slab thickness.

For preliminary design, we have adopted the range of uplift loads shown in TTW's Sketch SK.210923.a dated 23 September 2021, as follows:

Permanent Condition - 5,210 kN to - 9,265 kN

Temporary Condition - 5,558 kN to - 9,215 kN

5.2 RESISTANCE TO PILE TENSION LOADS

Resistance to pile tension loading will be via the following two mechanisms:

1. Piston pull-out resistance from shaft friction in uplift
2. Inverted cone pull-out of the rock mass

5.2.1 Piston Pull-out Mechanism

For the first of these two failure mechanisms, we have conducted some preliminary analyses to assess the effect of progressive failure mechanism (or unzipping effect) that is illustrated in Figure 5.

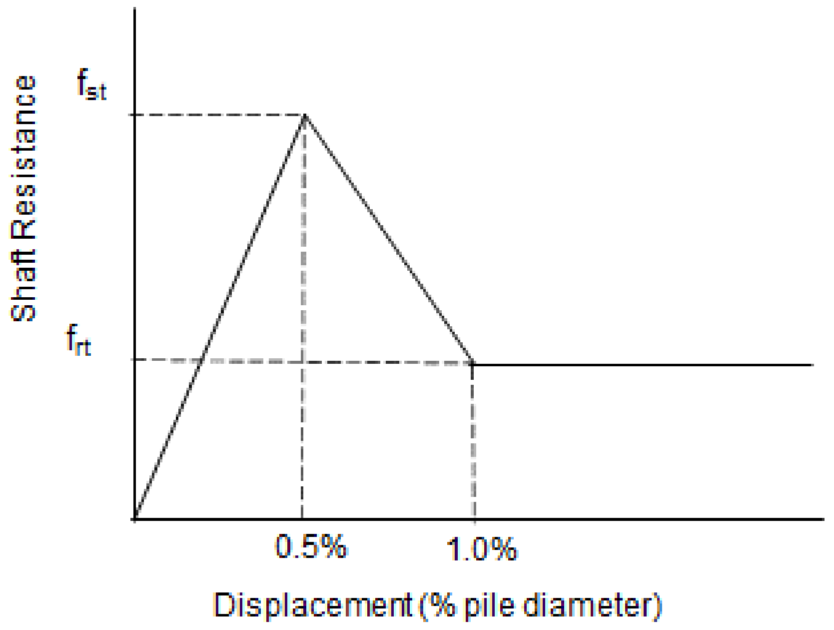

Figure 5 – Progressive Failure Mechanism

In the above figure, f_{st} is the peak shaft resistance (typically considered to be the ultimate shaft resistance of the pile under compression loading and assumed to occur at a displacement of 0.5%) for a rock socketed pile in medium strength rock or better, and f_{rt} is the residual shaft resistance in tension due to larger deformation concentrated at the top of the rock socket.

Because there is little or no published information on the ratio of f_{rt}/f_{st}, we have conducted some parametric analyses using f_{rt}/f_{st} ratios of 0.25, 0.5 and 0.75 for a rock socket in medium strength rock and for different rock socket length (L) to diameter (D) ratios. The results of the parametric analyses are shown in Figure 6.

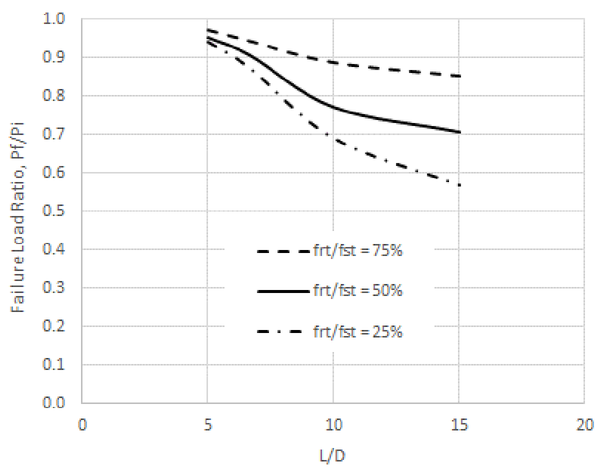

Figure 6 (a) Ratio of Pf/Pi

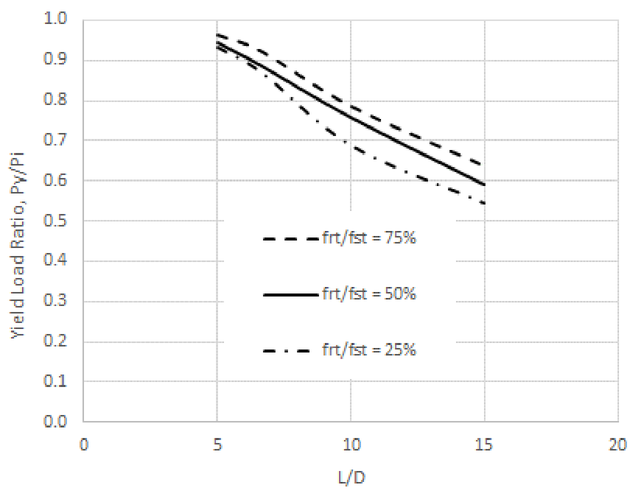

Figure 6 (b) Ratio of Py/Pi

In Figure 6, the different symbols are explained below:

Pi = Failure load if there is no difference $f_{rt}/f_{st} = 1$ (i.e. no reduction or if tension = compression shaft friction)

Pf = Failure load with $f_{rt}/f_{st} < 1$

P_y = Load at which yielding (i.e., non-linear behaviour) becomes significant ($P_y < P_f$ but typically quite close to P_f) for rock sockets in medium strength rock

In addition to the above progressive failure mechanism, there may also be a reduction in shaft friction due to the Poisson's ratio effect of pile diameter reduction under high tension loading, thus reducing the lateral stress of the rock against the rock socket in such circumstance. On the other hand, the application of reduction factors to account for both of these mechanisms may be overly conservative for rock socketed piles constructed with appropriate socket roughness.

Based on the above assessment, we recommend that a reduction factor of 0.7 for L/D ratio ≤ 10 be applied to the ultimate shaft friction values in compression, for the design of rock sockets in tension. We note that although higher reduction factors are possible for low L/D ratios, there is less redundancy for short rock sockets, and higher reduction factors should only be applied in the detailed design stage with sufficient assessment of risk.

Recommended preliminary design values for compression and tension pile are provided in Table 2 below.

Table 2: Recommended Preliminary Pile Design Values

Sandstone Rock Class	Buoyant Unit Weight (kN/m³)	Ultimate Shaft Resistance (kPa)		Ultimate End Bearing Pressure (kPa)	Young's Modulus (MPa)	
		Compression	Tension	Compression	Axial Response	Lateral Response
V	12	120	85	3,000	70	50
IV	13	400	280	6,000	400	280
III	14	1000	700	35,000	1000	800
II or better	14	1800	1260	80,000	1800	1800

5.2.2 Inverted Cone Pull-out Mechanism

For horizontally bedded rock under axial pull-out approximately perpendicular to the bedding plane, the following inverted cone angles may be adopted:

Class V and IV Sandstone 90º (i.e., ±45º from the vertical)

Class III Sandstone or better 120º (i.e., ±60º from the vertical)

These cone angles may be used together with the buoyant unit weight of the rock mass shown in Table 2, times 0.9 to assess the limit state resistance against the design ULS uplift load for the inverted cone pull-out mechanism.

However, due to the column grid spacing (typically at 8.4 m centres) and piles beneath the columns, the inverted cones will likely intersect except for the rock wedge beyond the perimeter of the basement. Therefore, the rock wedge weight will be limited to approximately the thickness of rock within the pile depth.

5.2.3 Geotechnical Strength Reduction Factor

The geotechnical strength reduction factor (ϕ_g) is a function of pile load testing and other factors, such as geological complexity of site, extent of ground investigation, level of construction control etc. Assuming sufficient pile load testing will be conducted for the project, a preliminary geotechnical strength reduction factor of 0.7 may be adopted at this stage.

5.2.4 Indicative Tension Pile Lengths

For a design ultimate uplift pressure of 170 kPa as discussed in as discussed in Section 5.1, the thickness of buoyant rock mass to resist this uplift pressure below the base of the B5 slab is assessed to be approximately 1.3 m of B5 slab, 1 m of Class IV Sandstone, 2 m of Class III Sandstone and 8.5 m of Class II Sandstone, giving a total rock socket length of 11.5 m. This assessment is made using total weight above the base of the slab and buoyant weight below the slab as follows:

[(1.3 x 24) slab + (1 x 12) IV Sst+ (2 x 14) III Sst+ (8.5 x 14) II Sst] x 0.9 = 171 kPa > 170 kPa

This gives a total rock socket length of 11.5 m. This is then checked against the piston pull-out mechanism using the ultimate shaft resistance values in tension shown in Table 2 and a geotechnical strength reduction of 0.7 as follows:

π.d.[(1 x 280) IV Sst + (2 x 700) III Sst + (8.5 x 1260)] x ϕ_g (i.e., 0.7) > 9265 kN (max. tension load)

Solving the above equation, the minimum pile diameter (d_{min}) is less than 0.5 m for the piston pull-out mechanism. This calculation is only a theoretical one to show that the design of the tension piles will not be governed by the piston pull-out mechanism, but rather sufficient length of pile to engage a sufficient thickness of rock beneath the B5 slab in the rock mass pull-out mechanism. Obviously, the minimum pile diameter will be governed by structural design requirements. For the relatively large tension loads, the amount of reinforcement in the pile will need to be substantial and the connection to the slab and column above will require extra consideration. For piles founded in Class III or II Sandstone, the compression capacity will be large and unlikely to govern the design other than for serviceability requirements to limit settlement and differential settlement, which are matters of analysis and design.

6. INTERACTION WITH SYDNEY METRO BARANGAROO STATION

6.1 SYDNEY METRO PROTECTION ZONES

The specific reserve (protection) zones of the Sydney Metro Barangaroo Station are shown in Figure 3 Appendix D of the Sydney Metro Station Protection Guidelines (SMCSWSBR-MET-SBR-ID-REP-000030.01.AFC.01.02), and this figure is reproduced as Figure 7 overleaf.

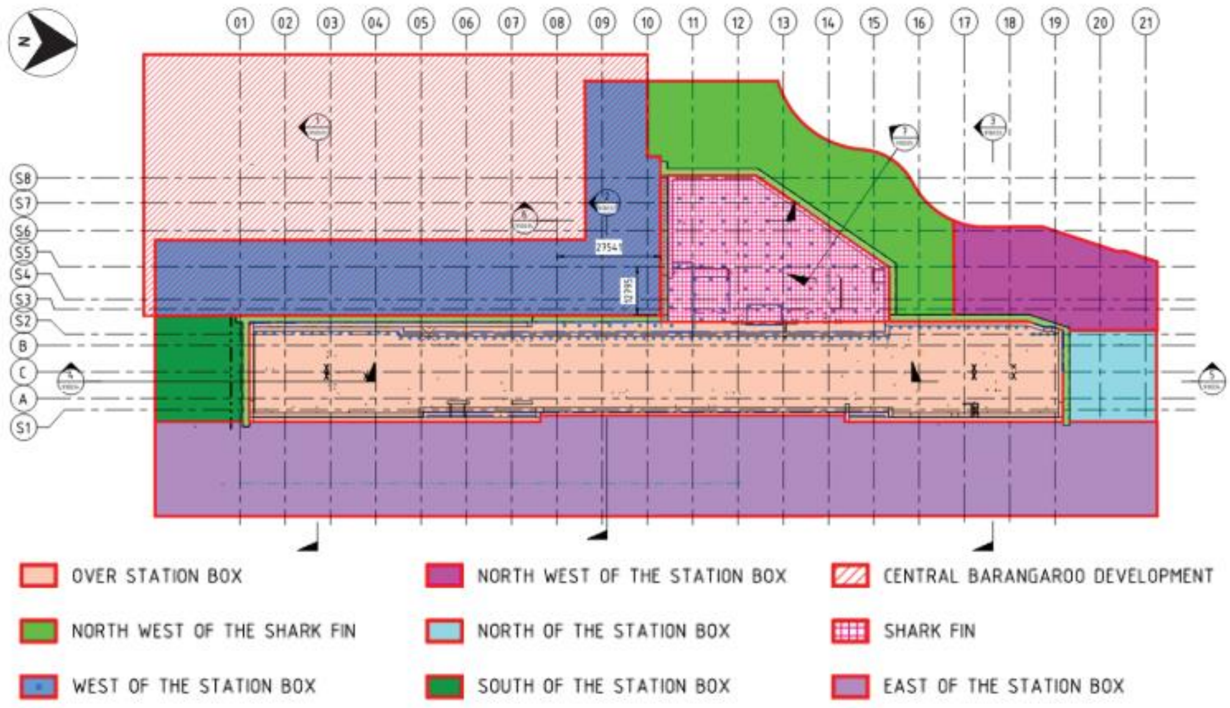

Figure 7 – Specific Reserve Zones from Figure 3 Appendix D of the Sydney Metro Station Protection Guidelines (SMCSWSBR-MET-SBR-ID-REP-000030.01.AFC.01.02 dated 11 Feb 2021)

The blue area marked as "WEST OF THE STATION BOX" is where interaction between the Barangaroo Central Development and the Sydney Metro Barangaroo Station will be relevant.

Sections 1, 2 and 6 are reproduced in Figures 8 to 10 which indicate the 1st and 2nd Reserves of the protection zones.

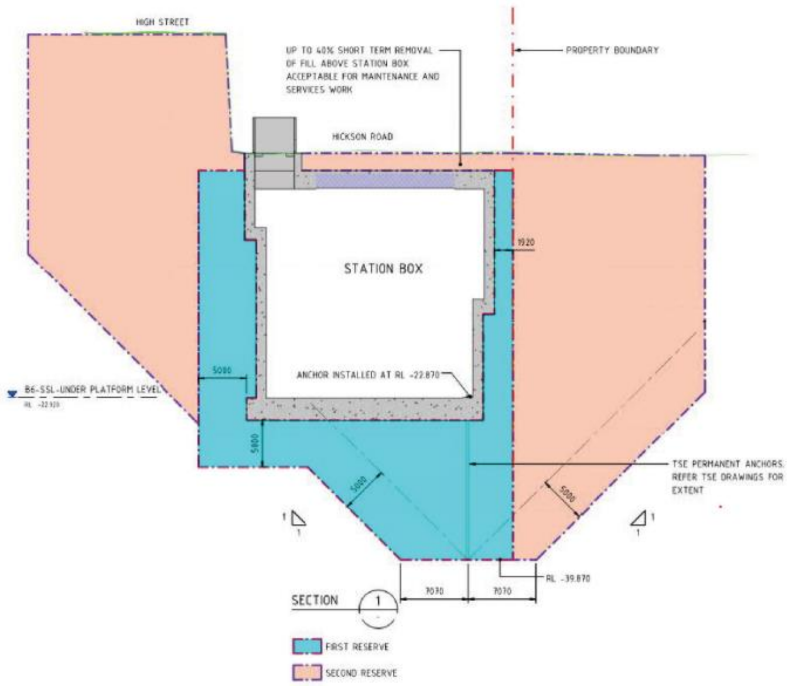

Figure 8 – Section 1 (see Figure 7) from Figure 11 Appendix D of the Sydney Metro Station Protection Guidelines (SMCSWSBR-MET-SBR-ID-REP-000030.01.AFC.01.02)

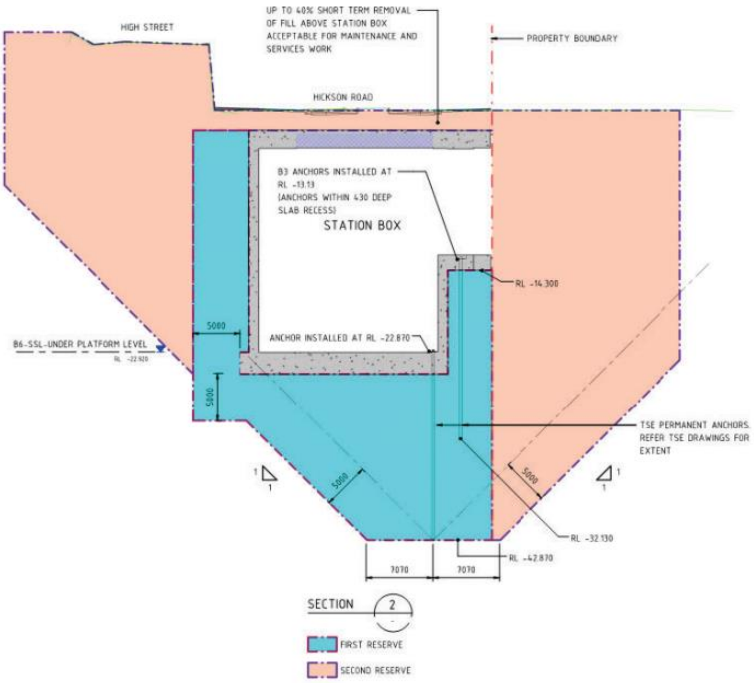

Figure 9 – Section 2 (see Figure 7) from Figure 6 Appendix D of the Sydney Metro Station Protection Guidelines (SMCSWSBR-MET-SBR-ID-REP-000030.01.AFC.01.02)

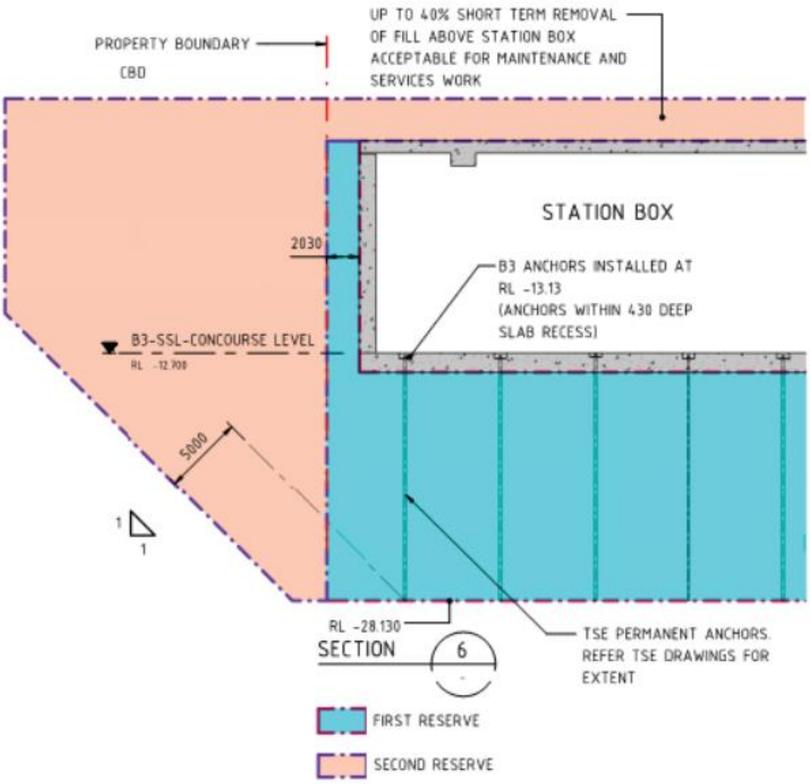

Figure 10 – Section 6 (see Figure 7) from Figure 4 Figure 3 Appendix D of the Sydney Metro Station Protection Guidelines (SMCSWSBR-MET-SBR-ID-REP-000030.01.AFC.01.02)

A more detailed plan of the 1st and 2nd Reserves is shown in Appendix L of the Structures Interface Report (SMCSWSBR-MET-SBR-ST-REP-000032.C.INF.C.02 dated 11 Feb 2021) which is reproduced as Figure 11 below.

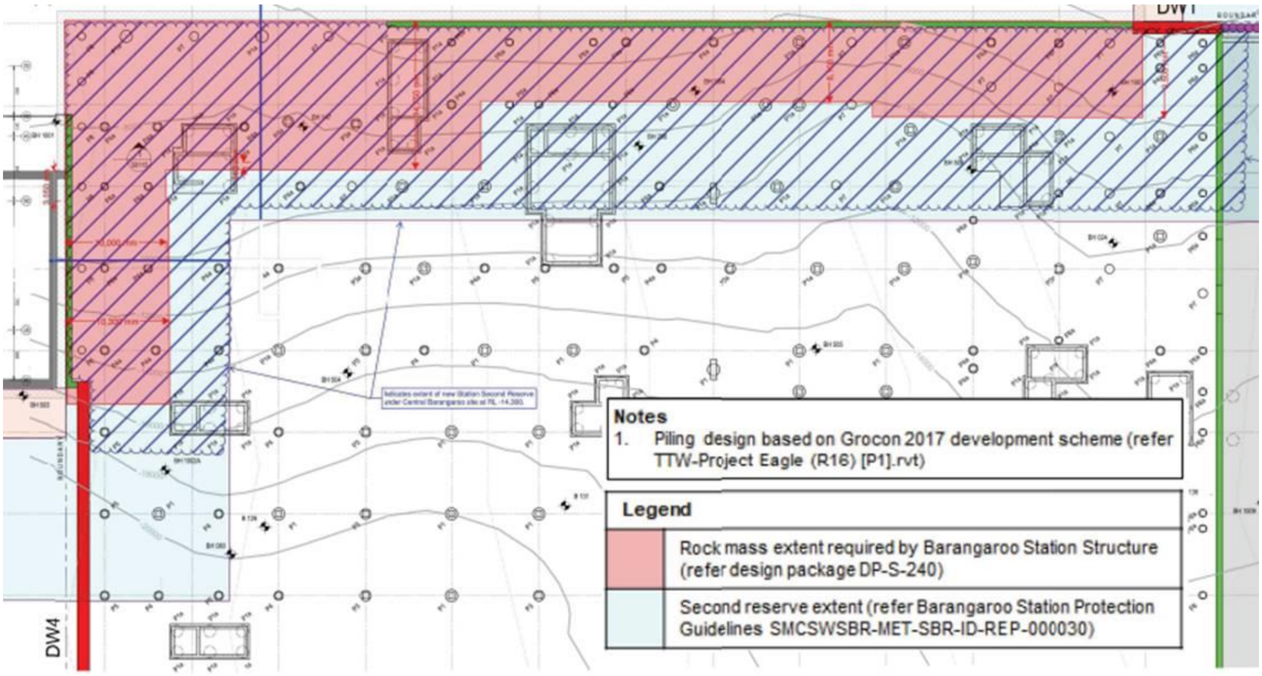

Figure 11 – Appendix L of the Structures Interface Report (SMCSWSBR-MET-SBR-ST-REP-000032.C.INF.C.02 dated 11 Feb 2021)

6.2 IMPLICATION OF THE PROTECTION ZONES

The implications of the Sydney Metro protection zones on the Barangaroo Central Development are as follows:

1. The rock within the orange zones in Figures 8, 9 and 10 and/or the pink zone of the plan projection shown in Figure 11 to the level of Hickson Road may be excavated with permission from the Sydney Metro Authority and the supporting engineering assessments.
2. No piles shall be located within the First Reserve (i.e., blue zones in Figures 8, 9 and 10).
3. Piles may be located within the Second Reserve (i.e. orange zones in Figures 8, 9 and 10). However, these piles will need to have their shaft section within this zone isolated from the rock socket side walls by the use of a temporary or permanent casing during pile construction.

6.3 INTERACTION OF GROUND ANCHOR DESIGN

An extract of PSM's design memorandum[2] on permanent rock anchors to resist buoyancy forces is presented in Figure 12 below.

10. CAPACITY OF PERMANENT VERTICAL ANCHORS

We understand the purpose of the proposed vertical anchors shown in the Metron drawings is to resist uplift forces due to groundwater pressures acting on the base of the tanked structure.

We recommend that the bond zone of the vertical anchors be entirely within Class I/II sandstone. The anchor bond zone may be proportioned based on the grout to rock bond strengths provided in Section 8.2.

Figure 4 shows a simplified approach to the assessment of uplift capacity of the anchors due to the "cone pull-out" failure mode.

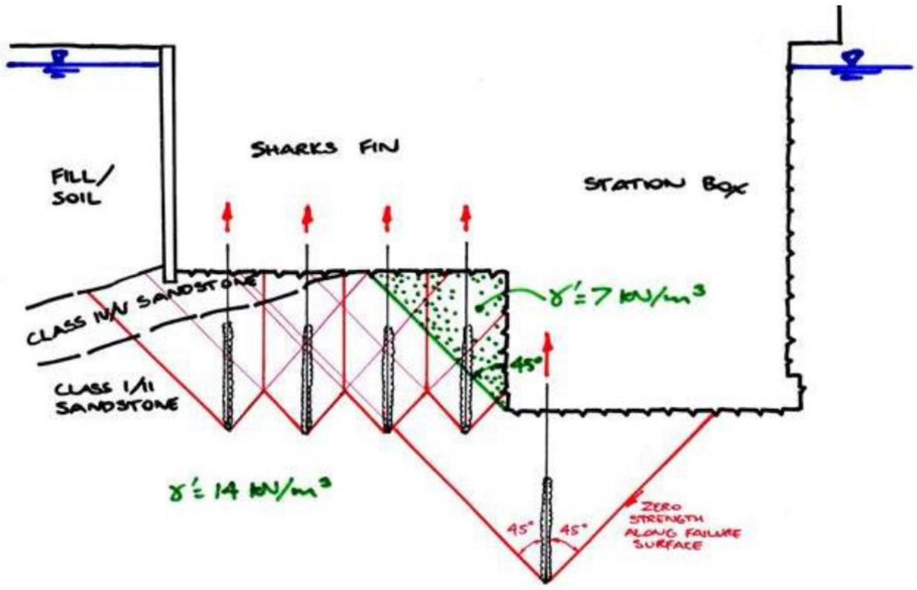

Figure 4 - A simplified approach for assessing uplift capacity

This approach may be summarised as:

- The nominal failure zone comprises an inverted 90° cone with the tip at the distal end of the bond length;
- The cone is truncated at the rock surface (taking into account the potential for future excavations);
- Zero shear strength is assumed along the failure surface;
- The failure zone is also truncated where it intersects other failure cones (i.e. each volume increment of rock may only contribute to resisting cone pull-out from one anchor);
- The buoyant weight of the rock within the failure zone constitutes the uplift capacity due to cone pull-out (i.e. 14 kN/m^3); and
- Next to a vertical cut (i.e. at the east side of the Shark's Fin), the uplift capacity of the rock is lower due to reduced horizontal stresses and the possibility of sub-vertical jointing. A lower unit weight is assigned to this region as indicated by the green zone in the above sketch.

Figure 12 – Extract from Section 10 of PSM Design Memorandum SMCSWTSE-JPS-SBR-GE-DAN-502205-A dated 25 October 2018

[2] PSM Design Memorandum SMCSWTSE-JPS-SBR-GE-DAN-502205-A dated 25 October 2018, contained within document SMCSWTSE-JAB-SBR-ST-RPT-030018, Rev 0, dated 19 July 2019.

We note the following points:

- The section shown in Figure 4 of the PSM memo is a South-North section through the "Shark's Fin" area located at the northern boundary of the Barangaroo Central Development (see Figure 7 for location).
- For the eastern boundary of the Barangaroo Central Development, PSM indicated in Sections 13.3.1 to 13.3.3 that "*Impacts relevant to the proposed Grocon excavation are not included*". As such, the reliance of the rock above the proposed Barangaroo Central Development within the First and Second Reserve would need to be considered in the design.
- PSM has adopted an inverted cone angle of 90º only, rather than the 120º which we consider to be appropriate for our site. However, PSM's design assumption is advantages to the Barangaroo Central Development as this reduces the zone of impact of the rock ledge relied upon by the permanent anchors of the Sydney Metro Station.
- PSM's anchor design was based on an allowable grout to rock bond stress of 1,000 kPa and an ultimate bond stress of 2,000 kPa in Class I/II Sandstone. We consider these design values to be reasonable and appropriate for use in the Barangaroo Central Development, subject to satisfactory suitability and proof load anchor testing.
- PSM recommended bond length be at least 5 m in Class I/II Sandstone (note: the actual anchor lengths (including free lengths) constructed are longer as inferred from the rock wedge projections shown in Figure 11.

Temporary dewatering to reduce the buoyancy forces will be required until the additional ground anchors, foundation tension piles and the tank basement of the Barangaroo Central Development are constructed.

7. LIMITATIONS

The preliminary advice contained in this report is based on a desk study of available existing geotechnical data. The advice is subject to change when further geotechnical investigations become available.

This report provides preliminary advice on the design of tension piles to resist uplift hydrostatic loading beneath the proposed B5 basement slab (Barangaroo Central), interaction with the Sydney Metro Station at Barangaroo in relation to existing rock anchors installed, considerations to bulk excavation to B5 due to the Sydney Metro protection zones and rock wedge relied upon by the existing permanent anchors.

Unbalanced lateral forces from groundwater and earth pressures are not covered in this preliminary advice, which will be addressed in our compiled preliminary geotechnical desk study report in due course.

This report must not be reproduced except in full and must be read in conjunction with the attached 'Important information about your Tetra Tech Coffey Report'.

For and on behalf of Tetra Tech Coffey

Patrick Wong
Senior Consultant

Important information about your Tetra Tech Coffey Report

These notes have been prepared by Tetra Tech Coffey to help you to interpret and understand the limitations of this report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Tetra Tech Coffey and applies only to this project and site. Project criteria typically include the general nature of the project; its size and configuration; the location of any structural elements on site; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Tetra Tech Coffey to assess how factors that changed subsequent to the date of the inspection or report affect the report's recommendations. Tetra Tech Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Reliance on external data

External data is often relied upon for Tetra Tech Coffey to generate our reports. This data includes items such as survey data, drawings, reports and geotechnical data. It is the responsibility of the client to ensure that all relevant data is provided to Tetra Tech Coffey. We are not able to independently verify external data and therefore cannot accept responsibility for problems due to the accuracy of external data.

Your report is prepared for specific purposes and persons

This report has been prepared by Tetra Tech Coffey for the Client and may only be used and relied on by the Client for the purpose as set out in proposal and within this report.

To avoid misuse of the information contained in your report, it is recommended that you confer with Tetra Tech Coffey before passing your report on to another party. Your report should not be applied to any project other than that originally specified at the time the report was issued.

The conclusions and recommendations in this report are based on the conditions and other information available to Tetra Tech Coffey at the time of preparing this report. Tetra Tech Coffey has no responsibility or obligation to update this report due to and subsequent change to these conditions or information.

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Tetra Tech Coffey to work with other project design professionals who are affected by this report. Tetra Tech Coffey can explain the report implications to the design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report

This report as a whole presents the findings of our investigation or design and the report should not be copied in part or altered in any way. Figures, tables, drawings, etc/ are customarily included in our report and are developed by our engineers based on their interpretation of input data. Information should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Rely on Tetra Tech Coffey for additional services

Tetra Tech Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from concept to operation. It is common that not all approaches will be necessarily dealt with at the time of this report due to the project status. As the project progresses through design towards construction, speak with Tetra Tech Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.