A CASE STUDY OF DEEP EXCAVATION AND SHORING DESIGN FOR SYDNEY METRO NORTHWEST

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ABSTRACT

This paper provides an overview of the Sydney Metro Northwest (formerly the North West Rail Link) project and the underground station excavation retention design and construction works, including the key requirements set out in the scope of works and technical criteria (SWTC). Based on the assessment of the geological conditions a soldier piled wall shoring system was adopted for all five new underground stations and one of the two services facility shafts for ease and speed of construction. During the Castle Hill Station excavation a new planar wedge instability mechanism was considered to be credible based on the additional geological data, with the original three-dimensional block instability being no longer suitable. This led to redesign of the south wall stabilisation works based on the updated geological model and input parameters. The instrumentation and monitoring plan was also adjusted to ensure the required additional support provided would be adequate for the safety of the station box excavation. The monitored lateral movements at the capping beam and at the inclinometers were within the trigger values, indicating that the retention system constructed was robust.

1 INTRODUCTION

The Sydney Metro Northwest project is Stage 1 of the Sydney Metro project. It is being delivered in three contracts: Tunnels and Station Civil (TSC) Works, Station Viaducts and Civil (SVC) Works and Operation, Trains and Systems (OTS) Works. A preliminary alignment of Sydney Metro is shown in Figure 1.

Figure 1: Planned Sydney Metro route (After TfNSW, website information)

The TSC contract consists of approximately 15 km of twin running tunnels, five new underground stations at Bella Vista, Norwest, Showground, Castle Hill and Cherrybrook, one crossover cavern next to Castle Hill Station and two services facility shafts at Cheltenham and Epping. There are also nozzle enlargements at excavated station boxes.

The TSC contract was awarded in June 2013 to the CPB Contractors, John Holland and Dragados (CPBJHD) joint venture (formerly Thiess, John Holland and Dragados). Arcadis was the design lead for stations and services shaft excavation shoring and site civil and road works, CH2M Hill (formerly Halcrow) the tunnel designer for the permanent tunnel elements, including segmental lining, cross-passages, caverns and mined tunnels, and Pells Sullivan Meynink as geotechnical consultant. APP was the project Independent Certifier for this TSC project.

This paper presents a case study of the underground station excavation design and shoring with an emphasis on the geological challenges encountered and subsequent solution considered during construction of Castle Hill Station.

2 KEY FEATURES AND PERFORMANCE REQUIREMENTS

The Sydney Metro Northwest requirements for the TSC contract were set out in the tender documents prepared by Transport for New South Wales (TfNSW). The key features and performance requirements that pertain to the TSC works are summarised below:

- 15 km twin tunnels between Epping and Bella Vista – 6 m internal diameter
- Concrete lining – minimum compression strength 40 MPa, with options including: a) reinforced concrete linings; b) steel fibre reinforced concrete linings; c) sprayed concrete linings; d) unreinforced concrete linings; and e) segmental precast concrete lining
- Stations must be excavated and stable for 10 years and handed over to OTS contractor for station permanent works
- Permanent ground support must be provided by concrete linings for the nozzle enlargements and the crossover cavern
- Use of rock bolts/anchors for permanent ground support is not permitted
- Specific key structural elements shall be designed as either drained or undrained structure as shown in Table 1.

Table 1: Specific requirements for structural elements

Asset	Drained/Undrained
Running tunnels, cross passages	Undrained
Cross passages with sump	Undrained
Nozzle enlargement	Undrained
Crossover cavern	Undrained
Station excavations	Drained
Services facilities shafts	Drained

- Water tightness – running tunnels, caverns and nozzles, Grade B: water indications limited to minor damp patches with no visible flow of water or water drips; cross passages, and cross passages with sumps, Grade A: watertight with the complete absence of any leakage, seepage and damp patches. The relevant groundwater seepage limits are presented in Table 2

Table 2: Seepage limits for structures

Asset	Groundwater Seepage Limits
Running tunnels, nozzle enlargements, crossover cavern	2 ml per hour per m^2 of concrete lining surfaces; and 10 ml per hour per m^2 of lining surfaces for any 10 m length of concrete lining
Station excavations and services facilities shafts	15,000 litres in any 24 hour period, measured over any square with an area of 10 m^2, at any and all locations within the side and bases of shafts and excavations, with the total seepage volume limit for each excavation specified in Table 5.4 of Scope of Works and Technical Criteria (SWTC), (TfNSW, 2013)

- The TSC works must be protected to prevent flooding of running tunnels from probable maximum flood event

- Station excavations and services facility shafts must be protected to prevent flooding from the 100 year ARI flood event.

3 GEOLOGICAL CONDITIONS

The geotechnical investigation were carried out in stages with the majority of field works carried out prior to tender and supplied to the tender team by TfNSW. Additional investigations were carried out by PSM. The geological conditions, geological models and design parameters were presented in the geotechnical interpretative report prepared by PSM (PSM, 2014). Figure 2 shows the geological profile along the tunnel alignment of the Sydney Metro Northwest.

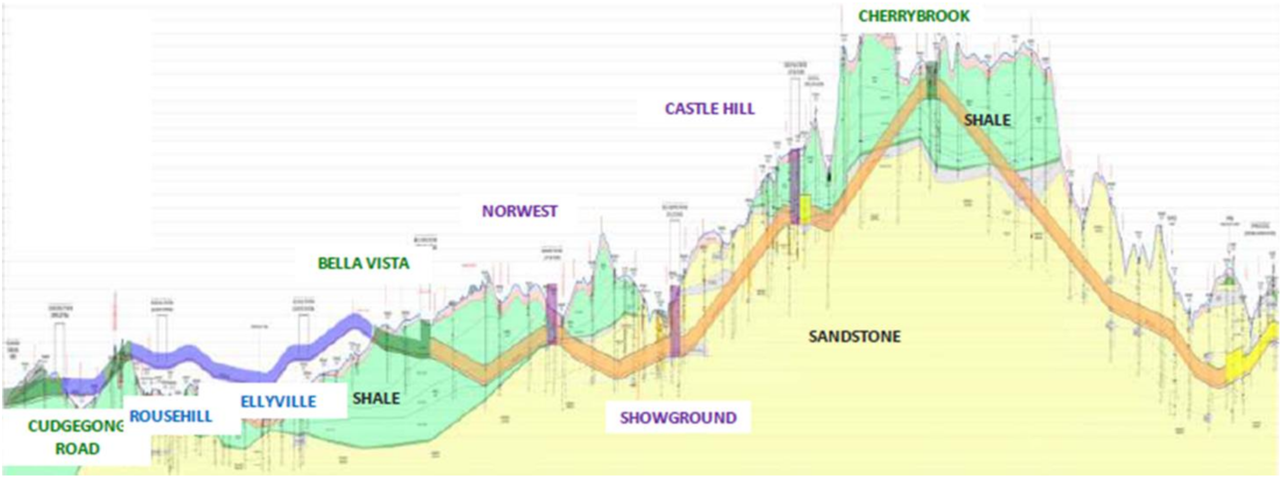

Figure 2: Generalised geological profile along the tunnel alignment (After Rudd and Porkinghorne, 2013)

A summary of the general geological conditions along the corridor for tunnelling, station and shaft excavations is summarised below:

- Tunnels - through primarily sandstone (~70%) and shale or siltstone (~30%)
- New underground station excavations – through residual soil, shale/siltstone and sandstones
- Geological features such as a faults zone that may be encountered by tunnelling, and station excavations
- High horizontal stresses assessed through site specific testing
- Hydrogeological conditions along the tunnel alignment.

Particular issues that had significant impact on the tunnel and station construction are described as follows:

- Dural Dome Fault and other faults at and in the proximity of the proposed Castle Hill Station and crossover cavern north of the station box
- Potential high flood level at the Epping Services Shaft due to the close proximity to Devlins Creek
- Shallow cover at the Cherrybrook Station and Bella Vista Station sites where TBMs were launched
- Shallow cover underneath the creeks, such as Devlins Creek and Cattai Creek
- Station excavation impacts on existing roads and infrastructure, such as Old Northern Road near Castle Hill Station.

4 TENDER STRATEGY AND STATION DESIGN CONCEPT

Given the characteristics of the geological conditions and the watertightness requirements, segmental lining was chosen for the running tunnels. A number of workshops were held on the need for tunnelling boring machines (TBMs) to achieve the construction program. One of the key dates was to launch the first TBM in October 2014, which was critical to the project. The other factors considered were the location of tunnel portals and the crane loading pads for TBMs at the launching stations.

A number of potential options, including open excavation and soil nail walls, for the station excavations were investigated at tender stage. It was found that soldier piled walls with ground anchors was the best solution for station excavation due to constraints of time and excavation space. The soldier piled wall system was adopted for all five new station excavation supports and the facility shaft at Epping. The soldier piles were installed below the base of the bulk excavation level at Bella Vista and Cherrybrook stations where excavations were through soil and weak rock. The piles were socketed in the competent sandstone above the final excavation levels at Norwest and Showground stations. At the Castle Hill Station site the soldier pile arrangement was more complex due to the presence of the Dural Dome Fault (DDF) and other faults intersecting the station, which will be described in more detail in the following sections.

This station excavation support strategy was primarily driven by the ease and speed of box excavation and shoring as compared to other options that might require on-site testing and waiting time. The access to the crossover cavern on the eastern side of Castle Hill Station was on the critical path. The cavern had to be completed for the two TBMs to pass through on their way from the launch site at Bella Vista to the Cherrybrook Station site.

5 CASTLE HILL STATION DETAILED DESIGN

The proposed Castle Hill Station involved excavation of a NE–SW orientated box of some 20 m × 210 m to a depth of approximately 25 m. Detailed design was based on a long and short piled wall with multiple levels of ground anchors for the station box. Figure 3 presents the plan view of the excavated station box while Figure 4 shows piles, the ground anchors, the final excavation level and the crane loading platform at both ends of the excavation. The long piles were socketed below the base of the excavation and the short piles were keyed in Class II shale or better material.

Figure 3: Plan view of constructed Castle Hill Station

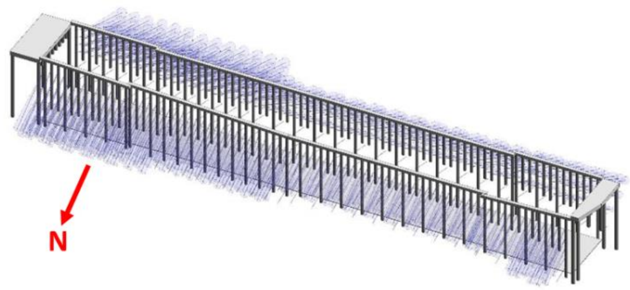

Figure 4: Soldier piled wall support at Castle Hill Station

A summary of station box excavation support for the south wall is presented in Table 3.

Table 3: Summary of station box excavation support for the south wall

Pile ID	**Approx. Length (m)**	**Short Pile (SP) Toe Level (m) & Spacing**	**No. of Rows of Rock Anchors through SP and Inclination to Horizontal**	**Long Piles (LP) Toe Level (m)**	**No. of Rows of Rock Anchors through LP and Inclination to Horizontal**	**Rock Support below the Toe of Short Pile**
CSH1 to CSH13	31	RL128.5 2.5 m c/c (Alternate pile long)	4 (9 × 15.2 mm strand) at 20°	RL113.5 2.5m c/c (Alternate pile short)	4 (9 x 15.2 mm strand) at 30°	8 m long 32 mm dia. rock dowel at 1.25 m (V) × 1.75 m (H) plus 150 mm shotcrete
CSH13 to CSH28	35	RL128.5 2.5 m c/c (Alternate pile long)	4 (9 × 15.2 mm strand) at 20°	RL113.5 2.5m c/c (Alternate pile short)	4 (9 x 15.2 mm strand) at 30°	8 m long 32 mm dia. rock dowel at 1.25 m (V) × 1.75 m (H) plus 150 mm shotcrete
CSH28 to CSH40	30	RL128.5 2.5 m c/c (Every 3rd pile long)	3 (8 × 15.2 mm strand) at 20°	RL116.0 7.5m c/c	3 (8 x 15.2 mm strand) at 30°	8 m long 32 mm dia. rock dowel at 1.25 m (V) × 1.75 m (H) plus 150 mm shotcrete
CSH40 to CSH49	23	RL128.5 2.5 m c/c (Every 3rd pile long)	3 (8 × 15.2 mm strand) at 20°	RL116.0 7.5m c/c	3 (8 × 15.2 mm strand) at 30°	8 m long 32 mm dia. rock dowel at 1.25 m (V) × 1.75 m (H) plus 150 mm shotcrete

It can be noted that the piles were designed to be 2.5 m centre to centre along the perimeter wall with a capping at the top. The capping was set at around RL142 m, with short piles to be terminated at RL 128.5 m and the long piles at 7.5 m centre to centre. There were four rows of ground anchors for the eastern part of the station and three rows of ground anchors for the western part of the station. There was shotcrete between the soldier piles, which were to be installed in stages. The long piles were taken below the excavation level to take account of the potential large scale rock wedges resulting from the uncertainties of the DDF and NW faults at the detail design stage. Beyond the toe of short piles, rock dowels were to be installed to ensure the wall stability. Further details will be discussed in the following sections. An instrumentation and monitoring plan and a traffic management plan were considered in the detail design to minimise any adverse impact of excavation on the existing Old Northern Road.

6 CONSTRUCTION STAGE REDESIGN

Following installation of the piles, capping beams and some subsequent excavation within the Castle Hill Station box, the characteristics (plan location, dip and dip direction, and strength) of the DDF were revised based on the additional geotechnical information that became available during station excavation.

6.1 GEOLOGICAL MODEL AND DESIGN PARAMETERS

The general geological profile adopted for the detail design without the DDF intersecting the south wall of the station box is presented in Table 4:

Table 4: Generalised geological profile at Castle Hill Station

Consistency or Rock Class	Depth to the base of Layer (m)	Thickness (m)
Stiff to hard, medium to high plasticity	1 to 2	1 to 2
Class IV to V shale	6 to 9	4 to 7
Class III shale	9 to 12	3 to 5
Class I to II shale	27 to 30	15 to 18
Class I to II sandstone	>27 to 30	>30+

Note that the original DDF was further to the southern side of the station box wall and there was not much extra load on the stability of the south soldier piled wall with ground anchors.

The presence and persistence of the DDF and the adjacent fractured zone were updated by PSM after drilling of the inclinometer holes was completed in early April 2014. Arcadis carried out a preliminary concept design using the identified DDF information for the wall stabilisation works. A meeting between CPBJHD, PSM and Arcadis was held to highlight the critical input parameters for the wall stabilisation design. It was agreed that the DDF would be such that a planar wedge sliding mechanism was credible and should be taken into account in the southern face excavation support design. Arcadis noted that the DDF inferred on 6 June 2014 was extended to around pile CSH45 which is about 115 m from the east portal.

Given the extent and large capacity of ground anchors required for the DDF stabilisation work, Arcadis requested further geotechnical investigations to delineate the DDF plane location and confirm the day lighting points in elevation along the southern face of the station excavation. PSM subsequently carried out field intrusive works with CPBJHD approval and provided updated geological models for detailed redesign.

The additional testing locations with ATV scanning (Cyan colour) are presented in Figure 5. A series of technical memorandum were issued by PSM to CPBJHD and Arcadis progressively for the stabilisation redesign of the south wall against the rock wedge instability along the DDF during June 2014. The DDF was assessed to be closer to the southern face of the excavation and the toe of the DDF plane would daylight above the base of the excavation along the south wall. Figure 6, based on PSM’s memorandum dated 11 July 2014, shows an elevation along the south wall. It can be seen that the DDF assessed in early July 2014 is closer to the south of station box wall and shorter than that identified in 2013. Figure 6 indicates that the extent of the DDF was revised to be only up to pile CSH35 from the city end.

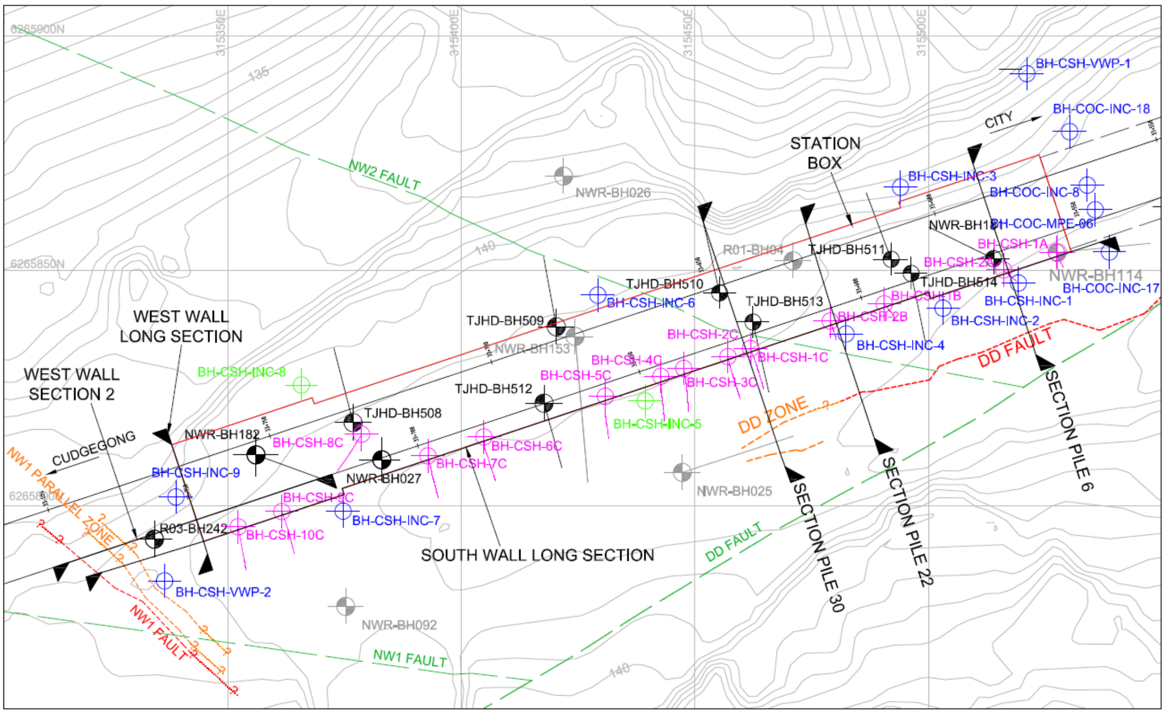

Figure 5: Plan View of Inferred Dural Dome Fault at Castle Hill Station (after PSM Report)

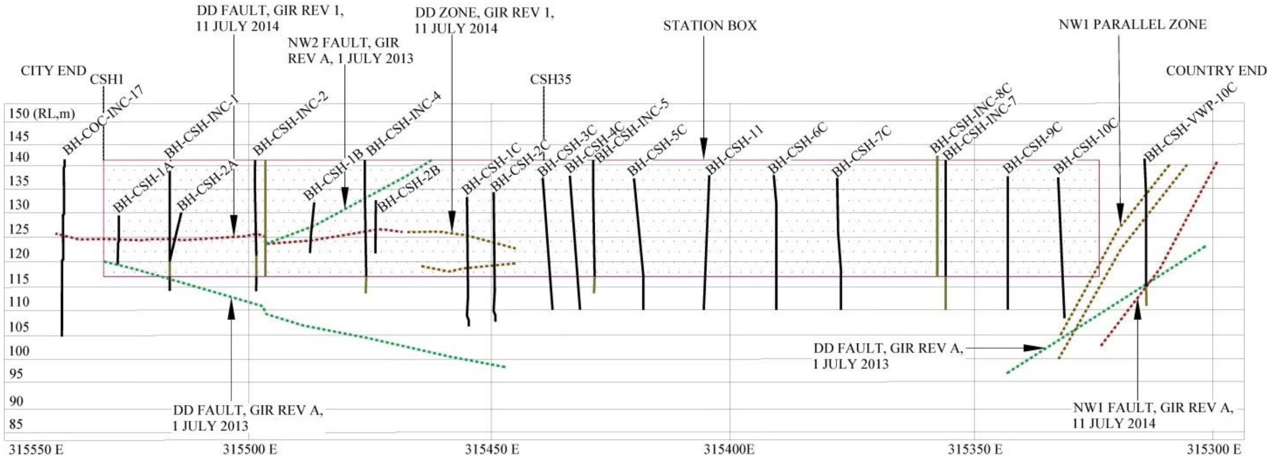

Figure 6: Inferred Dural Dome Fault along Southern Wall of Castle Hill Station (after PSM memo of 11 July 2014)

It was also confirmed by PSM that the geotechnical design parameters presented in the original Geotechnical and Hydrogeological Model report could be used for finite element analysis as the original design. A summary of adopted geotechnical design parameters is presented in Table 5. Note that the DDF was modelled as a one metre thick zone with a range of material properties considered for sensitivity assessment.

Table 5: Generalised geological profile at Castle Hill Station

ID	Description	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Friction Angle (Deg)	Young's Modulus (MPa)	Poisson's Ratio
Layer 1	Residual	20	2.5	26	50	0.35
Layer 2	Class IV/V shale	20	7.5	26	75	0.30
Layer 3	Class III shale	23	100	30	350	0.30
Layer 5	Class II/I shale	24	225	30	1350	0.2
Layer FZ	Fault Zone Class V/III shale	20/23	7.5/100	26/30	75/350	0.30

DDF	Dural Dome Fault	20	10	20	5	0.30

6.2 ANALYSIS METHODOLOGY

Based on the geological model, the southern face of the Castle Hill Station excavation was divided into four sections, as shown in Figure 7, to cater for the changing depth of the DDF along the south wall:

- Section 1 – Pile CSH 1 (eastern portal) to Pile CSH13, about 30 m long
- Section 2 – Pile CSH14 to Pile CSH24, about 30 m long
- Section 3 – Pile CSH24 to Pile CSH35, about 33 m long
- Section 4 – Pile CSH35 to Pile CSH80, about 112 m long.

These sections were selected based on the identified daylighting depth of the DDF along the south wall.

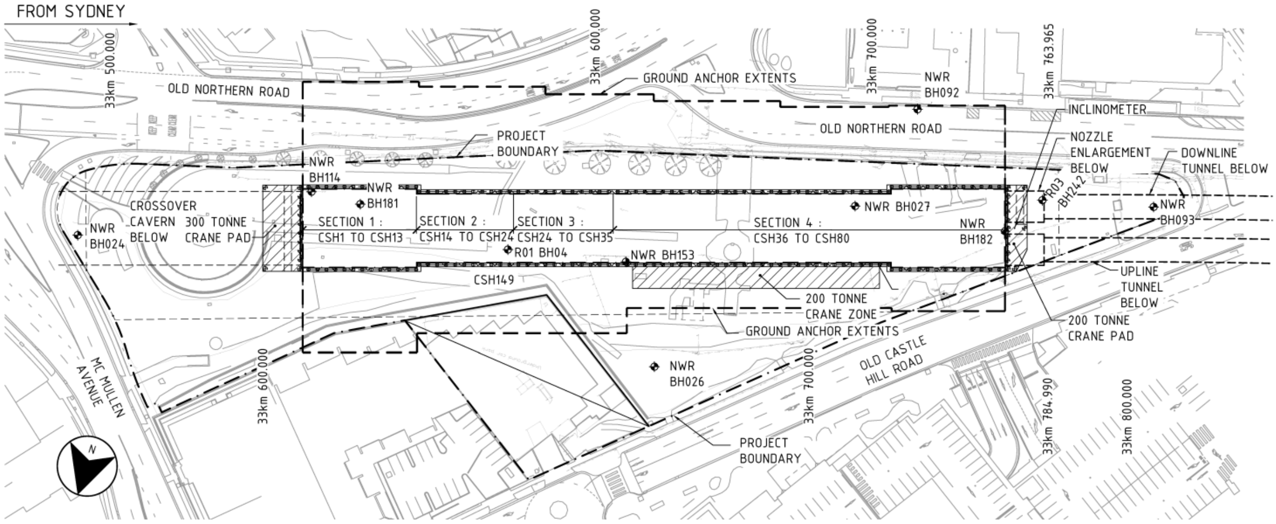

Figure 7: Station layout plan and sections analysed

The design philosophy for additional stabilisation work for the DDF was based the design basis report (Yang et al, 2000, Yang 2015 and Hyder, 2014), including two sets of analyses:

1. 2D planar rock wedge analysis using an in-house excel spreadsheet initially. It was then checked using the program Rocplane with a sliding plane along the DDF based on its corresponding shear strength parameters to assess the required additional reinforcement for wedge instability.

2. 2D Plaxis analysis using the rock mass model and associated parameters to assess the vertical and lateral movement of both short and long piles as well as the loads on rock bolts and rock dowels.

6.3 GLOBAL WEDGE ANALYSIS

Based on the persistence of the DDF and the planned excavation depth of the station box, a planar wedge sliding was considered credible. As such, a 2D limit state equilibrium analysis was performed to assess the additional stabilising force required by taking account of the originally designed rock anchors.

The design was carried out using the Australian Code AS5100.3. All the load factors were set to be 1.0 as per Clause 8.3.4 of AS5100. The wedge sliding instability along the DDF plane was identified to be the governing case. An equivalent geotechnical strength reduction factor of 0.65, equivalent to a minimum factor of safety (FoS) of 1.5, was considered in the limit state equilibrium analysis.

The pre-loads for the additional anchors were set to a level such that the minimum factor of safety of stabilising force against potential wedge instability was about unity, to control the potential movement of the wedge.

As the daylighting point of the inferred DDF along the southern face of the excavation varies along each section of the face, wedge analysis was carried out for selected typical cross-sections to calculate the required additional stabilisation force. The orientation angles of the original rock anchors through the short piles at 20 degrees to the horizontal and the long piles at 30 degrees to the horizontal were taken into account. The proposed additional ground anchors were between the original ones and at 15 degrees to the horizontal to provide more effective sliding resistance along the DDF while applying minimal vertical load on the DDF. The required anchor force per metre run was then converted to the required anchor force at 2.5 m centres along the face.

No groundwater was encountered during the additional borehole drilling and no groundwater seepage was observed during the excavation down to RL130m, except for one location which was due to potential leakage of a drainage pipe. Given the topographical setting of the Castle Hill Station site, no excess pore-water pressure built up along the DDF was considered in the wedge stability analysis. Nevertheless, horizontal drains were proposed to intersect the DDF to ensure negligible pore-water pressure at the DDF zone. These horizontal drains were additional to the vertical strip drains installed behind the shotcrete.

For Section 1 between Pile CSH 1 and CSH 13 (about 30 m long), the depth of the daylighting plane of the Dural Dome Fault varies between 16.9 m and 18.7 m based on the Vulcan cross-sections taken every two metres, with an average of 17.5 m. The average inclination angle of the inferred DDF plane to the horizontal ranges from 45 to 57 degrees with an average of about 52 degrees for the section. The average unit weight of the planar wedge is assessed to be about 23 kN/m^3. A surcharge of 20 kPa was considered in the wedge analysis. The current design considered three rows of additional ground anchors as follows:

- Row 1 – 9 x 15.2 mm strand anchors, with 16 m free length and 10 m bond length, at 2.5 m centres
- Row 2 – 9 x 15.2 mm strand anchors, with 16 m free length and 10 m bond length, at 2.5 m centres
- Row 3 – 16 x 15.2 mm strand anchors, with 16 m free length and 10 m bond length, at 2.5 m centres.

The calculated factors of safety under various load cases for Section 1 are presented in Table 6.

Table 6: Summary of calculated factor of safety for Section 1

Case Analysed	**Load Applied (kN/m)**	**Depth to DDF Plane (m)**	**Calculated FOS**
Case 1A-Preload	Pre-load to Approx. 90 kN/strand	17.5	1.07
Case 1B-Normal	Working Load of 150 kN/strand	17.5	1.81
Case 1C-Lower Bound (LB)	Working Load of 150 kN/strand	18.5	1.36
Case 1D-Earthquake (EQ)	Working Load of 150 kN/strand	18.5	1.22

A sensitivity assessment using the lower bound parameters and a potential increase of DDF plane depth of 1.0 m, as well as a larger inclination angle of DDF and slightly higher unit weight was carried out and the calculated FoS is greater than 1.3. An earthquake case using the lower bound parameters was analysed and it was found that a minimum FoS of 1.22 was achieved.

Further sensitivity analyses were also carried out to investigate the variability and reliability of the provided solution with respect to the potential changes in the input parameters.

A typical cross-section at pile CSH10 is shown in Figure 8.

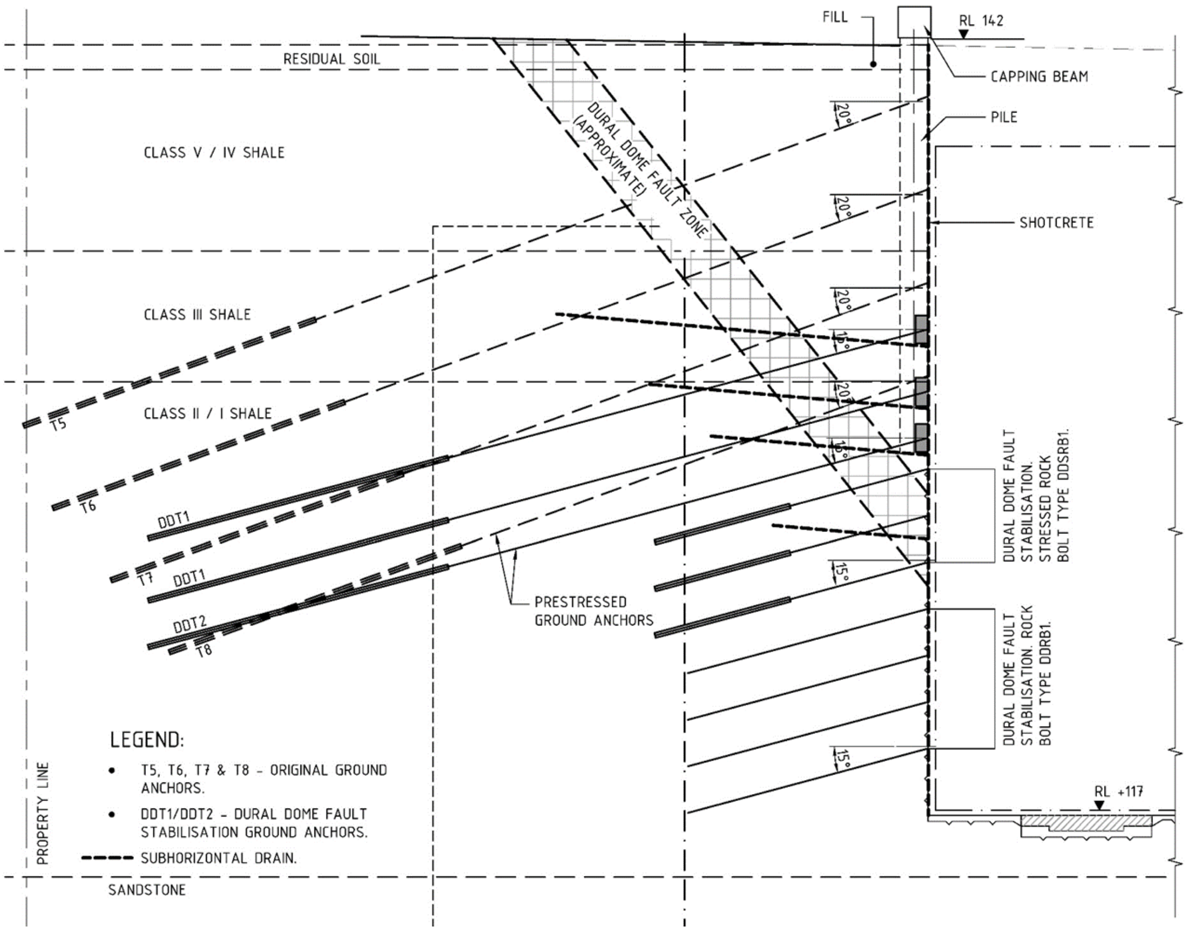

Figure 8: Typical cross-section showing the additional ground anchors and sub-horizontal drains

6.4 2D PLAXIS ANALYSIS

Two sectional models for each typical cross-section were considered in the deformation analysis using 2D Plaxis; a short pile and a long pile model. The geotechnical profile was generally based on the geological data provided by PSM, which includes delineated boundaries of various classes of rock mass, the location and orientation of the DDF and the fractured zones in the vicinity of the DDF. The 2D Plaxis modelling methodology is the same as that described in the Design Basis Report. The pile installation, excavation, installation of anchors/rock bolts/rock dowels and application of shotcrete were simulated.

Further analysis using the program Plaxis was also undertaken to assess behaviours of the retaining system, including the loads on the piles, anchor loads, lateral and vertical deformations of the anchored wall, as well as the rock bolt and rock dowel loads and deformation. A staged excavation and support was considered in the modelling and the results suggest that the rock block between the two long piles is likely to be ‘pushed out’. Accordingly, the first row of rock bolts was installed 0.5 m below the toe of short piles and tensioned to 50% of the rock bolt working load. Rock dowels were to be installed for excavation support below the base of DDF. There was no pretension load applied to these dowels, which is the same principle of the original design.

The calculated lateral displacement for the Section 1 and Section 2 models was about 15–30 mm for the short pile case. However, the final vertical displacement of the short pile was approximately 10 mm greater than the original design case, predominantly due to the clay seam within the DDF. The majority of this additional vertical displacement would occur once the excavation was below the toe level of the short piles. If this vertical movement could occur on site, the differential vertical displacement between the long piles and the short piles would overload the capping beam. It was noted that the primary purpose of the capping beam was to provide a vertical support to a short pile if a local rock wedge occurred immediately below the pile toe before rock support could be installed. The capping beam was expected to still fulfil this function when the shotcrete panel contribution to the load sharing was considered. It was concluded that the capping beam was not essential once the excavation was completed as the lateral support system should have reached equilibrium.

6.5 CONSTRUCTABILITY CONSIDERATIONS

At the time of designing the additional stabilisation works for the DDF the excavation within the Castle Hill Station box for Sections 1 and 2 had been carried out down to RL 130 m for the temporary ramp for the crossover cavern. It was not desirable to install additional ground anchors above the excavated level of RL130 m and shotcrete had been applied between the piles for the majority of Section 2. Additional ground anchors were installed around RL130 m or at lower levels. The excavation levels for the remaining Sections 3 and 4 were relatively higher at the western end of the Castle Hill Station box. The target level of the ramp for the construction of the crossover cavern was also taken into account in the redesign of the additional ground anchors so that minimum additional rock support would be required for the remaining excavation below the ramp level.

For Section 1 the first row of ground anchors was to be in between the existing third and fourth row of pile anchors and the second and third rows about 0.5 m and 2 m below the fourth row of pile anchors. For Section 2, the first row of ground anchors was to be in between the existing third and fourth rows of pile anchors and the second row 1.5 m below the fourth row of pile anchors. For Section 3, five rows of ground anchors were installed due to the greater daylighting depth of the DDF.

Figure 9 shows the final redesign of the south wall stabilisation works. This included additional ground anchors, horizontal drains, where required, changed DDF rock anchors and rock dowels below the inferred base of DDF.

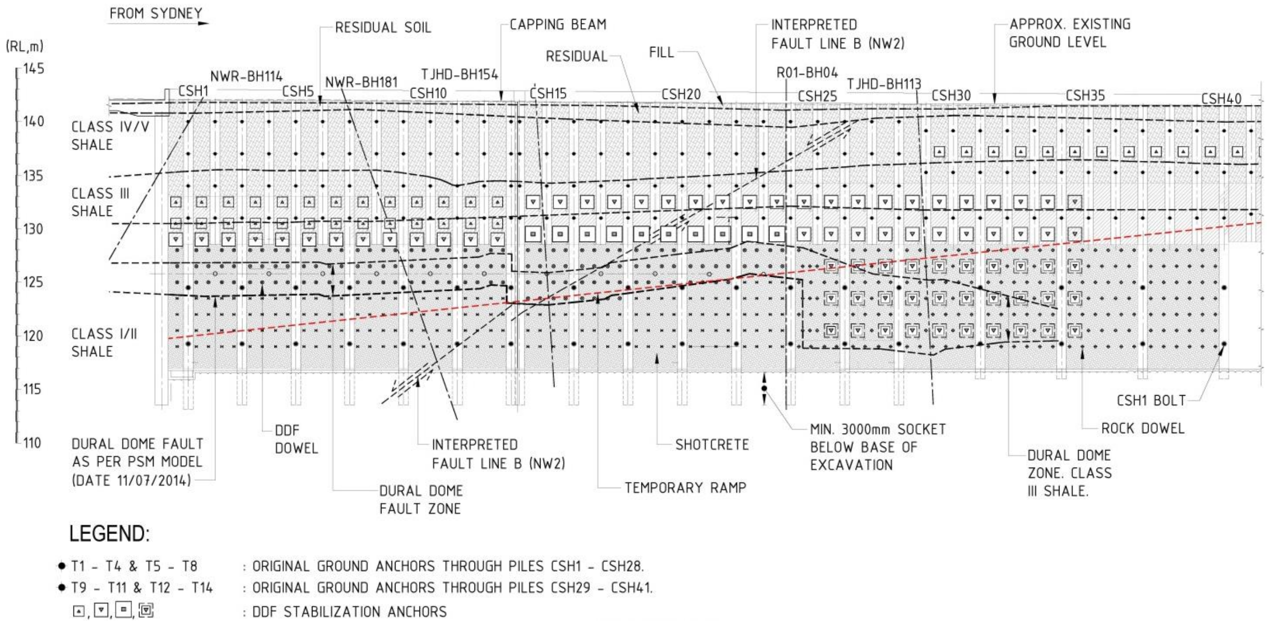

Figure 9: Additional ground anchor and existing anchors

7 CONSTRUCTION MONITORING AND CONTINGENCY PLAN

The geology of the site is complex and the alignment and extent of the geological structures were conjectured and interpreted based on the available information. Consequently, the actual site conditions could be different from the interpreted model. It was therefore extremely important that the design assumptions and lateral support design were continuously reviewed during construction, as additional information became available from face mapping and monitoring during excavation.

A number of inclinometers, survey prisms and extensometers, as shown in Figure 10, were installed for the entire station box excavation. The frequency of inclinometer CSH_SRF_33584_INC_02 along the south wall was set to enable 24/7 monitoring to ensure the safety of people working within the station box as well as users of Old

Northern Road, adjacent to the site. This was considered to provide the most reliable information should any potential development of excessive horizontal displacement occur.

Where worse than originally assumed geological conditions were identified, additional rock bolts and/or ground anchors were employed to ensure excavation stability. Particular attention was paid to the potentially unstable wedges immediately below the toe of short piles.

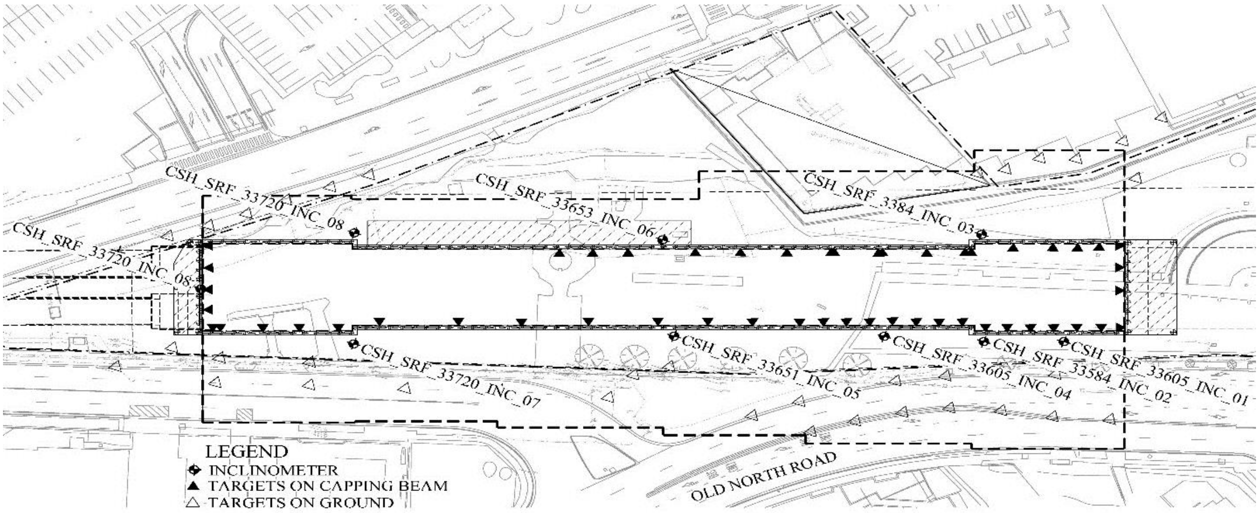

Figure 10: Plan of monitoring locations on wall, capping beam and pavements

The maximum monitored lateral movement of the retaining wall ranged between about 7 mm to 25 mm which was within predictions. The inclinometer reading was about 5 mm for the section concerned while the ground surface settlement was approximately 5 mm. This was considered to be due to the effect of the additional ground anchors and the location of the inclinometer about 2 m behind the piles.

8 CONCLUSIONS

The design and construction team worked closely together throughout the planning, tender design, detailed design and construction stages of the project. The identified Dural Dome Fault (DDF) presented a great deal of challenges to the team during the tender, detailed design and construction phases. It has demonstrated that geological information was of vital importance in the development of a reliable geotechnical model for the design and analysis of the deep excavation at the Castle Hill Station site. The orientation of the DDF plane, which day-lighted above the base of excavation was only ascertained during the construction stage when adequate geological information was available. The stabilisation work for the potential large wedge instability by means of additional ground anchors proved to be successful; based on the monitored lateral and vertical settlements on and behind the retaining structures. The live monitoring instrument and contingency plan enabled the team to have full confidence in safety in design for the remedial design. The cooperative approach among the team contributed to the successful completion of the project about seven months ahead of schedule.

9 DISCLAIMER

10 ACKNOWLEDGEMENTS

A large number of CPBJHD and Arcadis staff worked cooperatively on the Sydney Metro Northwest project, both directly and indirectly, contributing to the successful completion of the project. Transport for New South Wales (TfNSW) and the independent certifier also assisted in timely delivery in providing the necessary approvals without delay. The author is grateful to everyone who contributed to this project. Particular gratitude is owed to CPBJHD Design Manager David Polkinghorne and Doug Tomic, who was the Arcadis design manager during the construction phase. Some of the analyses presented in this paper were carried out by Raymond Hui and Zakir Hossain. Their contributions are greatly appreciated.

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