Design and construction of a multi-anchored retention system within Ashfield Shale

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Excavation for a four-storey basement alongside a busy commuter railway line required both temporary and permanent support within the Ashfield Shale formation. Rail authority requirements were such that the new building had to remain independent from the retaining wall supporting the railway line. A multi-anchored wall was therefore adopted to permanently support the excavation.

The design of the permanently anchored retaining wall was carried out using allowable or 'working' stress methods based on the worst 'credible' design wedge load, taking into account surcharge and earthquake loading. A probabilistic analysis was used to determine the worst 'credible' design wedge load acting on the wall based on a review of geological data in the area. This paper briefly presents some of the considerations involved in the design of the permanent wall with emphasis on the methods used and experience gained during construction.

INTRODUCTION

The basement excavation covered the entire area of the site to depths ranging from 13 m to 17 m below existing surface levels, as shown in Figure 1. The permanent retaining wall supported the 100 m long eastern side of the excavation, which extended to a depth of approximately 17 m below the railway line.

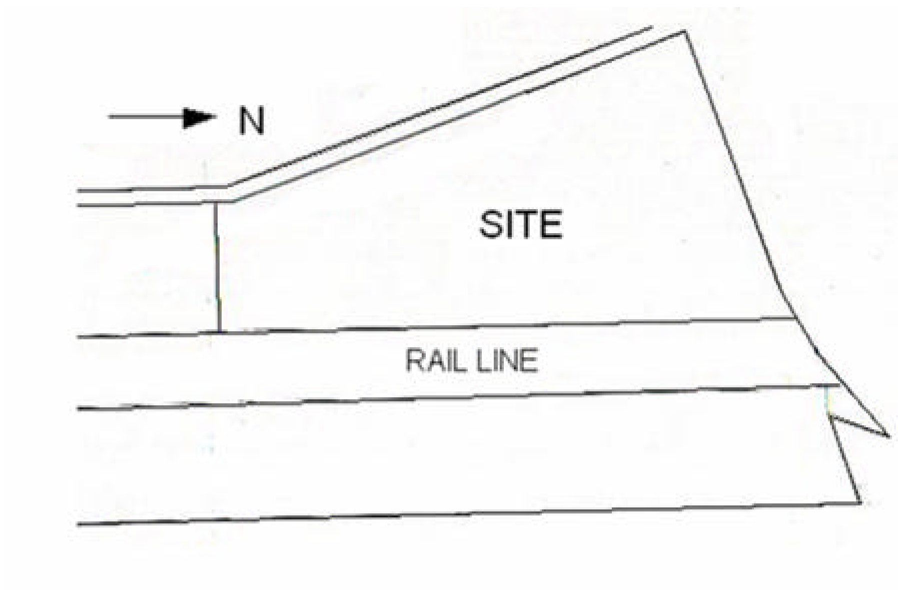

Figure 1 – Site Plan

The railway line originally ran along an open section of track on top of an earthfill embankment of 2 to 3 m height. Prior to initial excavation at the site, a Railway Enclosement Structure (RES) was built around the rail line for the length of track included in the development. The RES formed an above-ground tunnel structure of approximately 20 m width and about 10 m high over the rail corridor. The RES was constructed on beams supported by large diameter concrete filled steel piles founding at typical depths of 20 m to a level below the base of the proposed excavation. A photograph of the RES on the earthfill embankment prior to excavation is shown in Figure 2.

Site investigations revealed the following typical subsoil profile at the site:

Depth (m)	Material Description	Comments
Surface to 1.5 m	FILLING: Shale, crushed sandstone and clay fill	Original railway embankment
1.5 m – 3.5 m	CLAY: Hard, red brown clay	residual clay
3.5 m – 6 m	SHALE: Very low strength, light grey	Ashfield Shale
6 m - 14 m	LAMINITE/SHALE: Low strength, dark grey	
14 m – 18 m	SHALE: Medium to high strength, fresh	
Below 18 m	SANDSTONE: Medium strength	Mittagong Formation

Figure 2 – RES and rail embankment prior to excavation

DESIGN AND SPECIFICATION

The retaining wall system comprised reinforced concrete wall panels of about 10 m length, spanning between the large diameter shoring piles, anchored to the excavation face with horizontal ties or inclined rock anchors. The ties comprised multi strand cables running below the RES structure and tied to the concrete panels on either side of the rail corridor.

Both lateral earth pressures and mobilised wedge loads were considered in the design of the RES wall, in addition to surcharge and earthquake loading. The mobilised wedge loading was found to be significantly more than the lateral earth pressures for retained heights of more than about 8 m, and this was the governing failure mechanism used to design the retaining wall.

The magnitude of the design wedge loading was determined using a probabilistic assessment of joint mapping data collected from the area. The worst ‘credible’ design wedge was assessed to be formed by the intersection of joints dipping at 45^{o} with at least one joint having an incident angle to the strike of the excavated face exceeding 30^{o}. The horizontal wedge load applied to the wall was calculated to be 875 kN/m. This process is described in greater detail in Andrews & Braybrooke 2001 (Reference 1) and Douglas Partners unpublished report No. 28994G ‘Review of Geological Data for Perimeter Shoring Design’ 2002 (Reference 2).

The vertical layout of horizontal ties and inclined rock anchors over the permanent wall is presented in the typical cross section shown in Figure 3. The final design consisted of three rows of horizontal ties and three rows of inclined rock anchors, with approximately 20 ties or anchors per row. The permanent anchor lengths were designed based on the allowable working bond stresses within the low and medium strength shale shown in Table 1, and a free length extending behind a 45^{o} plane from the toe of the wall with allowance for debonding within the bond length over the life of the structure.

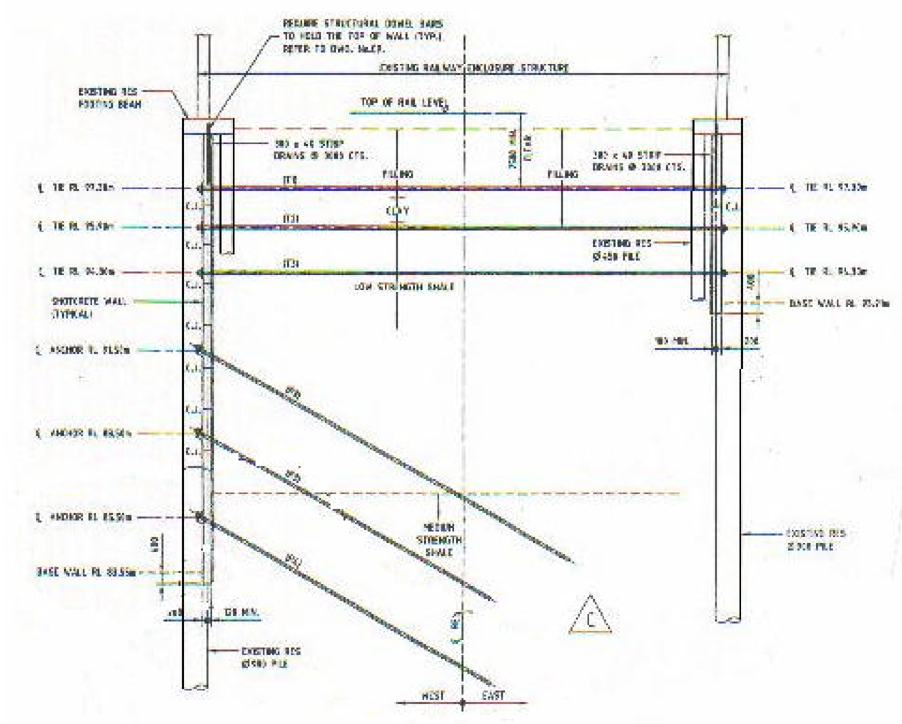

Figure 3 – Typical wall cross section

Class Shale	Strength	Design Working Bond Stress (kPa)
III	Low strength	200
II	Medium strength	500

Table 1 – Design Working Bond Stress

Most of the anchors had 7 strands (250 kN Ultimate Tensile Strength per strand) with a design working load of 875 kN. The anchor lengths ranged from 10 m to 16 m depending on the free length with the bond lengths ranging between 6 m and 8 m. A typical cross section through a permanent anchor is shown in Figure 4.

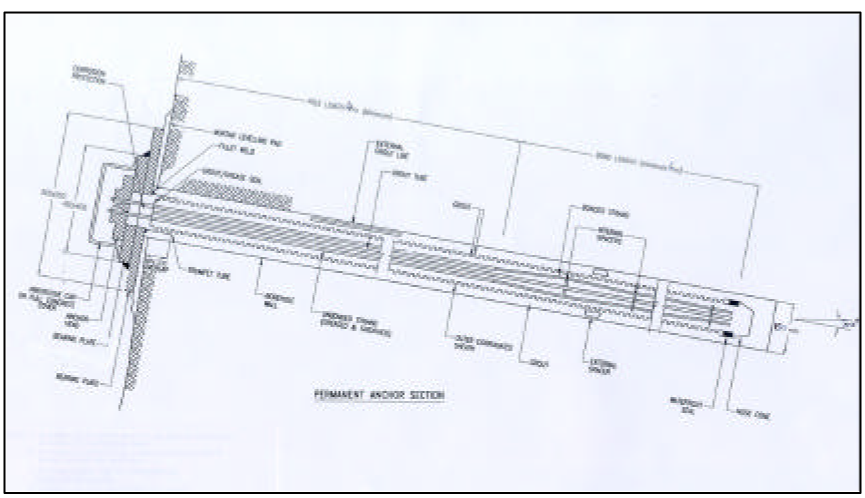

Figure 4 – Typical cross section through permanent anchor

The design specification called upon a design life of 100 years for the anchors supporting the permanent wall. Corrosion protection measures were incorporated for the anchors including an outer plastic sheathing to encapsulate the anchor strands and provide an additional barrier to corrosion through hairline cracking in the anchor grout. The anchors were also electrically isolated from the wall with isolation bearing plates beneath each anchor head to reduce stray currents.

CONSTRUCTION

Douglas Partners Pty. Ltd. (DP) was engaged to supervise the construction of the permanent retaining wall. This included face mapping and recommending lift heights through the soil/rock profile and supervision of the installation of the permanent anchors. The author was involved in this project from the beginning of the installation of the inclined rock anchors, discussed below.

Face Mapping

DP were required to be present during excavation for each panel of the RES wall to confirm that the conditions exposed were consistent with the design parameters and to advise on temporary stabilisation measures, where required.

Through the filling and upper clays this generally involved limiting the height of the lifts to around 1.5 m to reduce the risk of collapse of the temporarily unsupported face. Steel reinforcement and strip drains were placed prior to shotcreting each panel.

The faulting and jointing was mapped and measured for every panel exposed within the shale, allowing larger features to be identified and tracked to lower panels on the wall. A typical joint exposed within the shale is shown in Figure 5.

Figure 5 – Typical exposed joint

Figure 6 – Drilling of the anchor holes

Construction Methods

Lift heights of between 2.5 m and 3.5 m were adopted through the laminate/shale. Reinforcement was placed over the wall panel following final trimming of the face to ensure adequate concrete cover to the steel. Anchor pads were constructed at each anchor location prior to shotcreting.

The shotcrete was applied using a hand held nozzle fed by a concrete pump truck located at surface level. Drilling of the anchor holes commenced 1 day after shotcreting of the panels (Figure 6). Typically 3 anchors holes were drilled into each panel between the large diameter piles. The holes were inclined at 30^o to the horizontal and extended to between 12 m and 16 m depending on the free length of the anchor to be installed. 150 mm diameter holes were used to allow for placement of the 125 mm diameter anchor assembly. Drilling resistance and cuttings returned by the compressed air from the drill bit were used to confirm the strength of the anchor bond material.

Following drilling, the holes were filled with water and the rate of water loss from the hole monitored. The rate of water loss was used to indicate the potential for grout leakage from the holes into the joints within the shale during the grouting stage and the need for the hole to be sealed with grout and re-drilled prior to the installation of the anchors. The water leakage observed in the drilled holes was relatively minor and grouting of the holes was only required for a few anchors.

Following the water test the water was flushed out of the holes with compressed air pumped into the bottom of the holes using a long tube. The permanent anchors were then inserted into the holes in preparation of the outer sheath integrity test. The outer sheath (a corrugated plastic tube) of each anchor was filled with water to detect leakage which would indicate defects within the corrosion protection sheath around the anchor. One anchor failed this test and was removed to reveal a crack in the outer sheath which was repaired, re-inserted and re-tested. Grouting of the anchors commenced once each anchor had passed the outer sheath test.

The grout mixture was prepared on-site to a predetermined water:cement ratio with a electric grout mixer. The grout was injected into the 15 mm diameter grout tubes extending out of the anchors to allow a grout column to be poured from the bottom of the anchor to the top. The grout was then injected into the outer tube to grout the void between the outer sheathing and the rock. The grout levels were inspected at regular intervals after grouting to monitor grout loss from the outer section of the anchors. Grout cubes were cast to monitor the strength gain of the grout with time and select a suitable time to commence stressing. An admixture was used halfway through the construction of the wall to accelerate the strength gain of the grout and allow for a quicker turnaround between constructed anchor rows.

Anchor Stressing & Monitoring

The anchors were stressed using the jack shown in Figure 7, which was lifted using a 7 tonne excavator. The anchors were proof loaded to 125 % of the design working load in 5 equal increments with the load held at each increment for 30 seconds and the final proof load held for 10 minutes to allow time for anchor-wall movements and some movement of the wall within the extremely low strength shale. Once the anchors had demonstrated that they could successfully hold the proof load they were locked-off at 5% above the design working loads. Lift-Off testing was carried out on each anchor to provide a reference force in each anchor to compare to future lift-off test results.

Figure 7 – Stressing of Anchor

Following the immediate lift-off test the next panel below was excavated with the above procedure repeated until the full depth of excavation had been reached. A photograph of the completed wall is shown in Figure 8.

Figure 8 – Completed Permanent Multi-Anchored Retaining Wall

Figure 9 – Grease capped RES anchor

Lift-off testing was carried out on all anchors at 24 hours and 7 days after stressing with 15% of anchors tested after 28 days of stressing (i.e. 3 per row). This allowed the monitoring of anchor load with time to confirm that the anchors were holding the design loads.

Anchor Head Protection

During the construction of the building the final corrosion protection was applied to the anchors. This included secondary grouting and a grease filled cap. The secondary grouting was used to fill the void which is left behind the anchor plate during initial grouting of the anchors. The placement of a protective cap filled with grease protects the strands extending from the anchor head and allows for removal of the cap in the future for long-term lift-off testing over the life of the structure. A photograph of a capped anchor is shown in Figure 9.

CONCLUSIONS

The design of the permanent retaining wall was carried out using a probabilistic quantitative method using available jointing data within Ashfield Shale. Geotechnical supervision was maintained throughout the construction process with inspections on each individual anchor during the various stages of their installation. Corrosion protection measures were applied to the anchors to satisfy the 100 year design life of the anchors.

The experiences gained during the construction of this wall have increased the authors understanding of the factors involved in the design and construction of both temporary and permanent multi-anchored retaining walls.

ACKNOWLEDGEMENTS

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REFERENCES

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