Geotechnical Design of Retention Systems for the Chatswood to Epping Rail Link, Chatswood, NSW

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A portion of the Parramatta to Epping Rail Link involved the construction of about 500m of cut and fill retaining structures, allowing two additional rail tracks to be assembled within the existing narrow rail corridor. The cut walls were cantilever pile retaining walls up to 7m high near residential buildings. Geotechnical design including the derivation of realistic soil and rock parameters, the use of finite element analysis techniques, and comparisons of predicted versus actual displacements. Construction of the embankment walls required temporary and permanent stabilisation measures to interface with reinforced earth walls for the retention of steep operating rail embankments, including innovative designs, and monitoring of the structures and embankment.

THE PROJECT

The Parramatta to Epping Rail Link involves the construction of 13.5 km of new rail line in an urban environment. The total project value is approximately $850 million. The project principal is the Transport Infrastructure Development Corporation. The rail link is almost entirely underground (hard-rock tunnel), with the exception of the southern 1 km at Chatswood, which is above ground and utilises the existing rail corridor. Selected geotechnical design issues associated with preparatory works at this south end of the project form the basis of this paper.

The preparatory works were part of a design and construct (D&C) Contract to widened the useable area within the rail corridor for additional rail infrastructure. Transfield (now part of the John Holland Group) won the Design and Construct contract. Golder Associates was geotechnical consultant for the D&C team at tender and detailed design stages.

The bulk of the works included the construction of about 320m of cantilever pile walls up to 7m high to widen and retain an existing cutting (refer Figure 1), and about 180m of retaining wall to support and widen an operating rail embankment (Refer Figure 2). Three bridge overpasses (two road and one rail) and a short length of soil nail wall were also constructed, but are not discussed further in this paper.

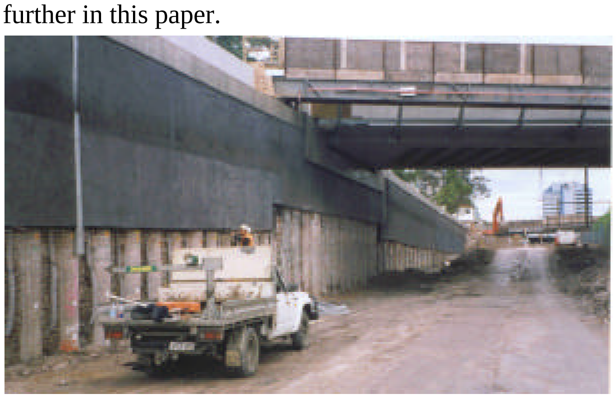

Figure 1 – Cut Retaining Wall

Figure 2 – Embankment Retaining Wall

GEOLOGY & PHYSICAL SETTING

The site is underlain by Ashfield Shale, consisting of black to dark grey shale and laminite (thinly interbedded shale and fine grained sandstone). The Australian Geomechanics Society classification[1] for shale within the Sydney region was used to describe the rock. Briefly, this classification denotes Class V Shale as extremely low strength. Class IV, III, and II Shale represent a gradually improving quality of rock up to Class I Shale, which is medium to high strength with a defect spacing greater than about 600mm.

In the cut area the surficial residual clay is generally very stiff and about 1.5m to 2m thick, deepening to about 5m near the north of the cut section. Within the depth of interest the underlying shale grades from Class V to Class II. The thickness of Class V Shale varies from about 1m to 4.5m, depending on location, as does the thickness of the underlying Class IV and III Shale, which varies from 5m to 13m. Class II Shale is penetrated at depth. The depth to groundwater below the final excavation level in the cut area was inferred to vary from about 0m to 3.5m.

The northern end of the Contract area is situated in a broad local drainage depression, and is the location of the existing rail embankment. Some firm to stiff colluvium (0.5m to 1.5m thick) is present in the lowest elevations, which overlies stiff and very stiff residual soils derived from the Ashfield Shale. Class V Shale is present at depths of about 2m to 4m below the top of the natural soil layers. Groundwater was measured in boreholes at depths varying from 0.5m to 3.6m below natural ground surface.

In this northern end of the site the existing rail line is positioned on a fill embankment up to about 6m high. The embankment was constructed during the early 1900's from clay and shale fill, inferred to have been won from the rail cuts to the north and south.

REFERENCE DESIGN AND DISPLACEMENT CRITERIA

The Reference Design for the project detailed a cantilevered contiguous bored pile wall in the cut area. The wall is located on the property boundary, and ground anchors or other restraints were not permitted to extend outside the property boundaries. The adjacent property was mostly public roads, with the exception of one two-story residence near the south end of the project, and a brick commercial building, also at the south end of the project. During the tender stage it was identified that design efficiencies would likely be available with careful selection of geotechnical design parameters, and use of finite element methods in design of the pile retaining wall.

The reference design for the rail embankment widening called for a gravity retaining wall. Access for construction was restricted by the existing (operating) 1.75H to 1V rail embankment, a line of significant deciduous trees along the property boundary, and a deep swale drain at the toe of the embankment (Refer Figure 2). The tender design of a piled counterfort wall was modified during detailed design to a reinforced earth wall, as construction limitations and economies were recognised. This is discussed in more detail later in this paper.

Primary displacement criteria in the project Deed were 5mm settlement and 1/500 angular distortion for sensitive structures (i.e. homes and masonry structures), and 30mm settlement and 1/250 angular distortion for roads.

GEOTECHNICAL DESIGN PARAMETERS

Selected geotechnical parameters adopted for design of the cantilevered bored pile wall are presented in the following table. The elastic modulus values adopted are somewhat higher than those normally used for foundation design. However, it is important to make the distinction between the loading modulus used for foundation design and the unloading modulus that is considered more appropriate for excavations. The unloading modulus is higher, as is evident from the results of field pressuremeter tests. The use of higher unloading moduli was supported by personal communications with Dr Chris Haberfield, who has been involved with recent developments in the understanding of the behaviour of fractured rock, which tends to be stiffer in unloading than previously thought. This is inferred to be a result of partial relaxation of the rock mass occurring naturally along the fractures in the rock, hence some displacement due to unloading will have occurred prior to excavation as a consequence of geological phenomena.

The rock mass strengths are based on the expected Geological Strength Index parameter (GSI[2]) for the relevant rock types.

Soil/Rock Unit	Cohesion (kPa)	Friction Angle (degrees)	Horizontal Unloading Modulus (MPa)
Residual Soil	5	30	100
Class V Shale	20	33	390
Class IV/III Shale	30	40	600
Class II Shale	50	44	2000

Selected geotechnical parameters adopted for design of the embankment widening are presented in the table below. The Select Fill was used in the reinforced earth block, while the New Fill is the material above the reinforced soil block and existing embankment.

The moduli used for design are lower than those used for the soldier pile wall, as the stress conditions were primarily compressive. The properties of the existing embankment fill were assessed from back-analysis of the existing 1.75H to 1V slopes, assuming that the existing embankment had a slope factor of safety of at least 1.2. This was inferred from the lack of evidence of slope creep or past slope instability.

Soil/Rock Unit	Cohesion (kPa)	Friction Angle (degrees)	Modulus (MPa)
Select Fill	0	36	50
New Fill	5	30	35
Existing Fill	2	30	15
Alluvium	2	28	15
Residual	5	30	35

GEOTECHNICAL DESIGN OF SOLDIER PILE RETAINING WALL

During tender, preliminary design was carried out using program *WALLAP* (Geosolve), using serviceability design parameters. This analysis indicated that the reference design of a contiguous pile wall could be economised (as the structural capability of the piles was not being fully utilised) and a soldier pile wall with shotcrete panel infill was proposed. Depending on the wall height, expected ground conditions, and restrictions on displacement, the piles varied from 0.6m to 0.9m diameter, at spacings up to 2.4m (Refer Figure 3).

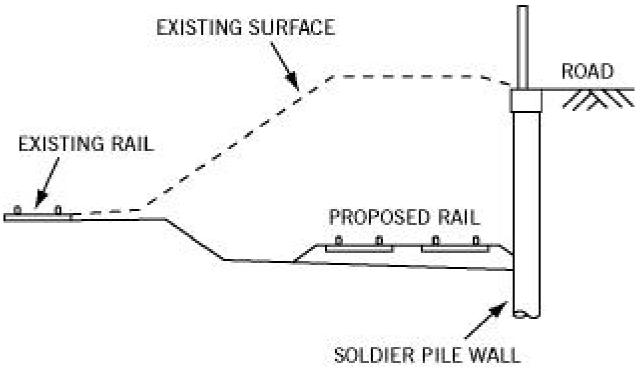

Figure 3 – Typical Section of Cut Retaining Wall

Detailed design of the solider pile wall was trialed using more than one numerical analysis program. Sensibility checks indicated that the soil/rock pressures behind the piles were well modelled by program *FLAC* (Itasca), which was then used to complete the analysis. The variance in results demonstrated the importance of not placing blind faith in the results of numerical analysis programs.

The analysis results were presented for two load conditions. Condition 1 represented the calculated serviceability displacements, without other external loads. Condition 2 included other loads such as rock wedge forces, and external loads such as wind loads on noise walls. For Condition 1, the analysis predicted displacements at the top of the soldier pile wall from 5mm to 25mm, depending on the spacing and size of the piles. Similarly, for Condition 2 predicted displacements ranged from 10mm to 35mm.

Detailed design led to other considerations, several of which are discussed below.

The project Deed specified a list of precedence for Design Codes. For piled retaining walls the Austroads(3) design Code took precedence. The Austroads ULS design methodology required the geotechnical design parameters to be factored down for use in analysis, then the resulting structural forces factored up. The overall factor on design forces was a value of approximately 1.8. The reduced input parameters resulted in the peak bending moment occurring lower in the pile than for the case where unfactored geotechnical design parameters were used. This potential variability in the depth at which the peak bending moment could occur had to be clearly communicated to the structural engineer, particularly as the Contractor requested an economic reinforcement design.

Concrete piles subject to bending can develop minor cracks, effectively lowering the pile stiffness. Hence lateral design of the concrete piles utilised a cracked-section modulus. The structural engineer advised us to use a cubic power formula to calculate an average cracked stiffness. The implication for design was that some iteration was required, as a more flexible pile attracted less load, which then changed the effective stiffness. This also had implications for displacements of the wall and supported ground.

The design pressures on the wall included forces from potential rock wedges. Whilst the force from a characteristic three-dimensional wedges could be resisted by the piles, consideration also had to be given to larger two-dimensional wedges arising from relatively continuous defects dipping out of the face at about 45 degrees. The risk of encountering such defects striking in this unfavourable orientation was considered to be low, but could not be discounted. There are some documented cases of encountering such wedges in Ashfield Shale, including one by Golder Associates in a recent basement excavation in Surry Hills. The impact on design meant that contiguous piles were designed to cope with the potential forces from such wedges. Where the spacing between soldier piles was greater than about 0.5m, geological mapping was required to confirm that no such defects were present. If a wedge was observed, it may have been necessary to install anchors. Such anchors would have extended outside the property boundary, which was not permitted. However, as the risk was considered low, and could only potentially impact public property, the Contractor and Principal accepted this risk management procedure.

Late in the detailed design stage the solider pile retaining wall was affected by an issue regarding drainage provisions. The cut area had to be drained to the north, against the final grade, which meant a trench between about 0.5m and 1.8m deep had to be excavated near the toe of the solider pile wall (refer Figure 4).

Figure 4 – Drainage Trench in front of Cut Wall

Relief of the passive restraint in front of the piles had not been designed for. Maintenance of the final track configuration meant that the drainage trenches were positioned about 3.7m from the wall. Our assessment considered this acceptable, provided the trench was constructed in sections no longer than 6m, backfilled with 2 MPa sand-cement mix (a similar strength as the surrounding rock), and geological mapping of the excavated shale was carried out to check for unfavorable joint orientations and potential failure blocks. Several response measures were available if potential failure blocks were exposed, including a reduction in the length of excavation, and temporary bracing.

RETENTION OF RAIL EMBANKMENTS AND GLOBAL STABILITY

The tender design of a pile counterfort wall was considered difficult to achieve in practice due to access constraints, and the Contractor sought to further assess the feasibility of a reinforced earth (RE) wall solution.

The RE wall solution required minimum levels of embedment (and strap length) to maintain global stability, potentially resulting in up to approximately 4.5m of excavation and temporary retention of the operating rail embankment. We suggested that it may be possible to satisfy global stability requirements by installing a row of soldier piles below the RE wall, thereby reducing the depth of excavation and height of temporary retention (Refer Figure 5). This solution was later adopted following a period of intensive design assessment, as discussed below.

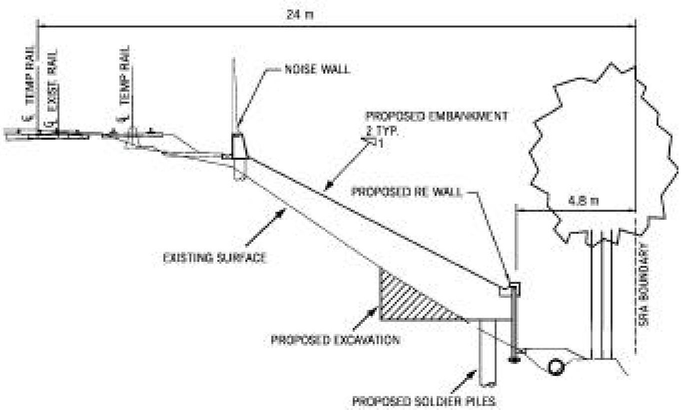

Figure 5 – Typical Section of Embankment Widening

To construct the proposed design, a temporary working platform 5 m to 6 m wide was required for machinery, access, and as a formation level for the reinforced soil wall. To achieve these widths, partial excavation and temporary retention of the existing embankment using sheet piles was proposed. To allow the sheet piles to be cantilevered without anchors or props, the level of the working platform for the reinforced earth wall was set to limit the maximum height of sheet pile retention to 2.2 m.

The Ultimate Limit State (ULS) design methodologies adopted by the Design Codes specified in the Works Brief were incompatible with the design of the temporary sheet pile retention, as the factored (reduced) soil input parameters were less than the slope angle of the existing embankment (1.75H to 1V). As a result a working stress approach using engineering principles was adopted. A factor of 1.5 was applied to the forces calculated in the sheet piles for structural design.

The design of the sheet piles was carried out using program *WALLAP*. The results were supported by analysis using program *FLAC*, where the sheet piles formed a construction stage in the design of the permanent works. Larrsen 6W sections ("back-to-back") provided the required design capacities and were chosen by the Contractor for construction.

For the design of the permanent works, global stability of embankments and the use of solider piles to improve the stability of embankments are also not covered by the Design Codes specified in the Works Brief. The stability analysis and design was thus carried out using engineering principles.

The stability of the proposed embankment was assessed using limit equilibrium methods and program *SLOPE/W* (Geoslope). The results of the analysis indicated that the factor of safety (FOS) against global slope instability (potentially undermining the reinforced earth wall) varied between about 1.2 and 1.4, if no measures were adopted to improve stability for the proposed depth of excavation. A design FOS of 1.5 was chosen for the permanent (operating) rail embankment.

Design of the stabilising soldier piles was carried out using two methods. The first assessed the additional restoring force (provided by the piles) required to raise the global FOS to 1.5. The magnitude of the additional force was assessed using *SLOPE/W*, for both circular mechanisms and wedge mechanisms. The bending moment and shear force profile in the pile was assessed by analysing the embedded pile below the point where it is cut by the critical failure surface, using the program *DEFPIG* (Sydney University).

The second design method used program *FLAC* to model the soil-structure interaction. The sequence of construction including installation of sheet piles, excavation, installation of soldier piles, construction of the RE block and construction of the final embankment was modelled in the *FLAC* analysis. In order to derive ultimate loads from the results of the working load *FLAC* analysis, the calculated working bending moments and shear forces were multiplied by a factor of 2, to approximate the effect of an average load factor of 1.4, and an average material factor of 0.7.

Results of the *SLOPE/W* analysis indicated that the soldier piles would be required to provide an additional lateral force of up to 140 kN/m of wall in order to increase the global FOS to 1.5. Figure 6 presents a comparison of the factored bending moment and shear force profiles calculated using *FLAC*, and the *SLOPE/W* and *DEFPIG* combination. The results provided by the two methods were comparable, with similar peak bending moments and peak shear forces.

The design of the reinforced soil wall itself (including internal stability analysis) was carried out by Reinforced Earth Company. The design was based on AS4678[4].

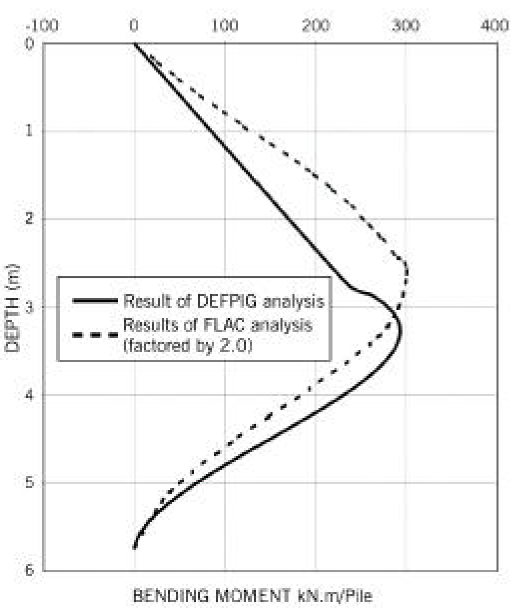

Figure 6 – Comparison of Predicted Pile Forces

The final design sought a solution that reduced the height of temporary retention, while not over-stressing the solider piles, and adequately interfaced with the RE wall. The design specified 0.6m diameter bored cast in place concrete piles, at 1.8m centres. In most areas, to limit the height of the temporary cantilevered sheet piles to 2.2m, the level of the working platform was above the toe of the final RE wall configuration (see Figure 5 and 7). To maintain the continuity of urban design, the soldier piles were faced by the reinforced earth wall panels. The area between rear of the piles and the RE panels was backfilled with lean-mix concrete, to reduce the potential for plastic deformations associated with the soil squeezing between the piles. The select fill layer used to construct the RE wall was also placed behind the top 0.5m of the piles, to reduce the risk of sliding failure between the piles and RE block.

Figure 7 – Proposed base of RE block (left), and base of RE facing panels in front of piles (right)

CONSTRUCTION OBSERVATIONS

During the construction stage, geotechnical observations were carried out to confirm the design assumptions regarding ground conditions. For the cut areas and soldier pile wall these included:

- Logging cuttings of bored piers during drilling.
- Geological mapping of material exposed between piles. This included consideration of the design issue regarding potential earth pressures from relatively large two-dimensional wedges associated with continuous defects dipping at about 45 degrees. One such defect was noted, but the strike of the defect was approximately perpendicular to the cut face, and did not impact the design.
- Geological mapping of rock defects during excavation of the drainage trench in front of the pile retaining wall. Within the 6m long sections excavated, we did not observe joint defects continuous enough and of the correct orientations to form large passive failures.

For the embankment retaining wall, construction observations included:

- Confirmation of design bearing pressures for shallow footings and at the base of the reinforced earth wall.
- Proof rolling of subgrade.
- Assessment of unsuitable material.
- Observations of temporary sheet pile installation.

FIELD PERFORMANCE & COMPARISON WITH DESIGN

Monitoring plans were developed for both the pile retaining wall (cut) and embankment retaining wall (fill) were based on the “traffic light” reporting system, where “green” represents acceptable movements, “orange” indicates that further assessment of movements and a reduction in the rate of excavation/construction is required, and “red” means halt construction and implement remedial response.

Monitoring of the pile retaining wall indicated that measurable displacements continued for a period of about 3 months after the completion of excavation, which is inferred to be the period required to effectively reach drained conditions and overall stress-strain equilibrium.

In summary, lateral displacements recorded at the top of the pile wall were:

- 7mm to 11mm in inclinometers.
- 6mm to 17mm from survey (repeatability was found to be typically ±5mm).

Comparisons with the predicted lateral displacements at the main locations chosen for *FLAC* analysis during design are presented in the following table.

Pile Config.	Predicted Displ. (mm)	Measured Inclinometer Displ. (mm)	Measured Survey Displ. (mm)
0.9m dia. @ 1.2m c/c	5 to 10[a] 10 to 15[b]	7	7 to 10[c]
0.75m dia. @ 2.4m c/c	5 to 10[a] 25 to 30[b]	N/A	17
0.85m dia. @ 1.2m c/c	20 to 25[a] 30 to 35[b]	11	16
0.6m dia. @ 1.2m c/c	10 to 15[a] 15 to 20[b]	N/A	12

(a) Load Condition 1 - serviceability displacements, without other external loads
(b) Load Condition 2 - includes other loads such as rock wedge forces, and external loads
(c) Two monitoring points in close proximity

As indicated above, the results of displacement monitoring were in good agreement with the predicted lateral displacements, justifying the use of higher unloading moduli for the fractured shale.

Monitoring of the embankment retaining wall included survey displacement monitoring of the temporary sheet pile wall during construction. Lateral displacements recorded at the top of the sheet piles varied between about 10mm and 25mm, which were within the "green" region of the surveillance criteria.

After construction, a program of precise survey monitoring of the completed wall was implemented. The survey targets were installed at the base and top of the RE wall, and on a concrete barrier at the top of the new embankment. As the predicted displacements were relatively low (the "green" region called for displacements less than 10mm in some areas), an accurate survey methodology was called for (± 2 mm to 3 mm) to prevent unwarranted alarm over potential displacements in the "orange" region. The surveyor decided to install dedicated lugs in the concrete panels, to which a prism could be connected for survey.

Since the new rail lines became operational at the end of December 2003, displacements have been less than 8mm settlement at the top of the embankment, and less than 4mm lateral displacement at the face of the RE wall. These displacements are within "green" region of the surveillance criteria, and compare with predicted settlements at the top of the embankment of about 10mm to 15mm, and lateral displacements of the wall of about 5mm to 10mm.

CONCLUSIONS

Detailed design of the cantilevered solider pile retaining wall incorporated several critical design issues, including the impact of applying Limit States approaches to geotechnical design, cracked section moduli for laterally loaded piles, and the use of higher moduli for fractured rock masses when applied to the unloading conditions in excavations. The validity of using realistic unloading moduli was demonstrated by displacement monitoring during construction. Management of geotechnical design risks associated with the fractured rock was achieved by geological mapping during construction.

The design of the embankment retaining wall presented significant design complications, including an operating rail line at the top of the embankment and limited construction access. To overcome these constraints, original designs were developed using solider piles below a reinforced earth wall to improve the global stability of the final embankment, and to reduce the temporary excavation and retention required for construction of reinforced earth walls.

The successful design outcome also depended on cooperation between the Contractor, geotechnical engineer, and structural engineer.

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