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An Anchored Bored Pile Wall Design – A Case History

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ABSTRACT

Ipswich Motorway Upgrade – Dinmore to Goodna Project involved an upgrade of 8km of extremely constrained urban motorway from four lanes to a minimum six lanes. One of the many challenges on the project was to carry out excavations within a narrow project corridor. One of the most critical excavations was a 9m deep cut with a 25m concrete swimming pool located 7m behind the crest. The stability and serviceability of this excavation were of paramount importance due to the proximity of a swimming pool behind the crest and motorway in front of the toe. With careful design and construction considerations, an anchored bored pile wall was selected to support the cut. This paper aims to present a systematic approach to design an anchored bored pile wall in accordance with Australian Standard AS5100.3 – Foundations and Soil-supporting Structures. The wall performance and design are assessed by comparing the calculated and measured wall movements. One year after the construction, the wall performed well and the swimming pool showed no signs of distress.

Keywords: excavation, bored pile wall, passive anchors

1 INTRODUCTION

The Ipswich Motorway Upgrade – Dinmore to Goodna (IMU-D2G) Project, Queensland involved an upgrade of 8km of extremely constrained urban motorway from four lanes to a minimum of six lanes and also included two motorway to motorway interchanges. The IMU-D2G is one of the largest roads projects undertaken in Queensland. Since the middle of 2008, Origin Alliance comprising Queensland Department of Transport and Main Roads (TMR), Abigroup Contractors, SMEC Australia, Parsons Brinkerhoff (PB), Fulton Hogan, and Seymour Whyte was tasked to design, construct and provide a two-year maintenance period of the Ipswich Motorway from Dinmore to Goodna. Construction of the motorway started in early 2009 and is currently near completion.

The project route traverses densely populated areas which impose significant constraints to the design and construction work. One of the many challenges on the project was to carry out excavation within a narrow project corridor. As a result, various retained excavations were required. The stability and serviceability of the retaining structures required consideration of the existing facilities and properties in the proximity.

One of the particular challenges of the project was to excavate in front of a 25m swimming pool of a state school. The excavation was up to 9m deep and the swimming pool was 7m behind the excavation. The swimming pool has a concrete wall with a depth of 2.5m. A threshold displacement tolerance of 20mm was proposed for the excavation after consultation with the project's structural team. A bored pile wall with passive anchors was considered the most effective solution to provide support to the excavation with minimum disturbance to the existing structure. Active anchors were avoided because of potential maintenance problems in the long term.

This paper will present the geology and geotechnical conditions of the site, detailed design procedures, and site monitoring of the anchored bored pile wall.

2 GEOTECHNICAL CONDITIONS

2.1 Site Geology

The project site is dominated by the Late Triassic Raceview Formation and Aberdare Conglomerate which are mantled by Quaternary Alluvium associated with the Brisbane River Floodplain, Goodna

Creek and Six Mile Creek. The Late Triassic units are underlain by the Ipswich Coal Measures Tivoli Formation (refer to the Geological Survey of Queensland 1:100 000 series Ipswich sheet).

Regional general dip of the strata is shallow (less than 10 degrees) to the south and south – southeast, while localised variable dip directions can occur due to a number of folds with axes parallel to the general dip. Faulting is prevalent with trends mainly to the north, northwest. The faults are commonly normal, but reverse faults also occur. These faults, which affect both the rock units of the Ipswich Basin (Ipswich Coal Measures) and the Moreton Basin (Aberdare Conglomerate and Raceview Formation), are considered as being inactive since the Tertiary.

2.2 Geotechnical Model

The geotechnical model was developed primarily from boreholes IMU009, IMU212, and BH1. The subsurface materials comprise 0.3m – 0.5m of medium dense sand and silt fill, underlain by residual soil up to 3.6m thick comprising very stiff to hard clay. The residual soil overlies about 6m thick extremely weathered siltstone and sandstone layers, which grade into moderately weathered sandstone.

Based on the investigation results and local experience, the subsoil profile and geotechnical parameters assumed in the design are shown in Table 1.

Table 1: Assumed soil/rock parameters used in the design

Material	Unit Weight (kN/m^3)	Elastic Modulus (MPa)	Cohesion (kPa)	Friction Angle (degree)	Top Elevation, AHD(m)
Residual soil (CH-vst)	19	20	4	28	24.9
EW Siltstone (SIL-6)	22	100	10	25	22.7
EW Sandstone (SAN-6)	22	100	30	26	14.7
EW Sandstone (SAN-5)	23	150	50	28	13.5
HW Sandstone (SAN-4)	23	300	100	32	10.5

3 DESIGN METHODOLOGY

3.1 Design Principles

In compliance with AS 5100.3, the bored pile wall design was carried out based on the Principles of Limit State Design to satisfy the criteria for both Ultimate Limit States (ULS) and Serviceability Limit States (SLS) as follows:

- Overall stability of the soil/wall system including:
  - Depth satisfying moment and horizontal equilibrium.
  - Overturning failure of the bored pile wall socketed in rock.
  - Global failure.
- Structural strength of the wall (i.e. bending moment and shear force).
- Deflection of the wall and its impacts on adjacent structures.

3.2 Pile Embedment Depth

Limit equilibrium calculations were undertaken using WALLAP to determine the embedment depth of the bored pile walls. In the WALLAP analysis, characteristic values of soil parameters (i.e. unfactored parameters) were adopted. A method of applying a Factor of Safety on passive earth-pressure coefficients was then followed to design the bored pile walls. This method (CP2) is explained in detail in BS8002 – 1994: Code of Practice on Earth Retaining Structure and is also discussed in Earth

Pressure and Earth-Retaining Structures by Clayton et al (1993). A factor of safety 1.8 was adopted in the WALLAP analysis for determining the penetration depth of the piles, which was equivalent to adopt a geotechnical reduction factor φ_g=0.55 (Table 13.3.1(A), AS 5100.3) to factor passive earth pressure.

3.3 Overturning Failure for the Bored Pile Wall Socketed in Rock

As the piles were embedded into weathered rock, overturning failure of the rock socket might occur due to two possible failure modes: bearing failure of rock mass and failure by movement along preferentially oriented discontinuities (discontinuity-controlled failure) (GEO, 1993). GEO (1993) suggested idealised pressure distributions and design equations to determine the minimum rock socket depth required to prevent bearing failure. A calculation model was also presented in the GEO (1993) for the design of rock sockets against a planar discontinuity-controlled failure.

3.4 Passive Anchor Design

To control bending moment, shear force and deflections of the piles, it is sometimes necessary to use anchors. BS8002 Clause 4.4.2 recommends using anchors to support the bored pile wall when the height of the wall is generally higher than 5m.

The design of anchors involved the assessment of the pull-out resistance of the anchors embedded in the ground behind the assumed slip surface. The pull-out resistance was considered to be the lesser of the stabilising forces derived from:

- Tensile strength of the steel bar;
- Bar-grout bond resistance; and
- Grout-soil bond resistance.

Example 3 – Limit Equilibrium Method in AS 5100.3 Supp 1-2008 was followed to design the passive anchors.

The free length of the anchor was determined by a potential failure surface plus the greater of 1.5m and 0.2H (H is the wall height), in accordance with Fig. 37 FHWA Geotechnical Engineering Circular No. 4 – Ground Anchors and Anchor Systems.

3.5 Serviceability

Finite element software PLAXIS was used to estimate wall deflections and displacements of ground and nearby structures. Bending moment, shear forces of the wall and force in the anchors were also checked in the PLAXIS analyses.

3.6 Loading and Assumptions

- The loads on the bored pile wall include:
  - General live load of 10 kPa behind the wall for non-traffic areas;
  - Wind loads (horizontal force = 12kN/m and moment = 37kNm/m) from noise barriers on top of the wall;
- Groundwater tables:
  - For short term: groundwater level = 1/3 wall height
  - For long term: drawdown at 45 degrees from a peak groundwater level at 1/3 wall height dropping down to toe level just behind the wall.
- An over-excavation depth of 2.0m, which accounted for around 1.0m of pavement and drainage layers, and 1.0m unplanned excavation.

3.7 Pile Spacing

In bored pile wall design, the piles can either be contiguous, or closely spaced with infilled concrete panels or shotcrete in between. The center-to-center spacings between the bored piles should generally be less than 3 times the pile diameter. Otherwise special support and protective measures should be taken to retain the soil exposed by the excavation (GEO, 1993).

4 RESULTS

The sequence of construction assumed and modelled in the analysis is shown in Table 2 below:

Table 2 Construction Sequence

Stage No.	Description
1	Pile installation
2	Pile cap construction
3	Excavation lift 1 – 1.5 m from ground surface
4	Anchor installation – 1m from ground surface
5	Excavation lift 2 – up to 3.5m depth
6	Excavation lift 3 – up to 5.5m depth
7	Excavation lift 4 – up to the design surface
8	Excavation lift 5 – up to the excavation level for pavement and over-digging
9	Construction of pavement
10	Noise barrier built on top of pile cap

Note: High groundwater level behind the wall was modelled after the construction of the wall.

4.1 WALLAP Analyses

The results of WALLAP analysis are summarized in Tables 3 and shown in Figure 1 below:

Table 3 Summary of WALLAP Analysis Results

Pile Dia. (mm)	Pile Spacing (mm)	Wall Height including over-digging (m)	Pile Embedment (m)	Max. Unfactored Bending Moment (kNm)	Max. Unfactored Shear Force (kN)	Max. Wall Deflection (mm)	Unfactored Anchor Force (kN,)
900	1000	9.2	6.0	537	170	14	210

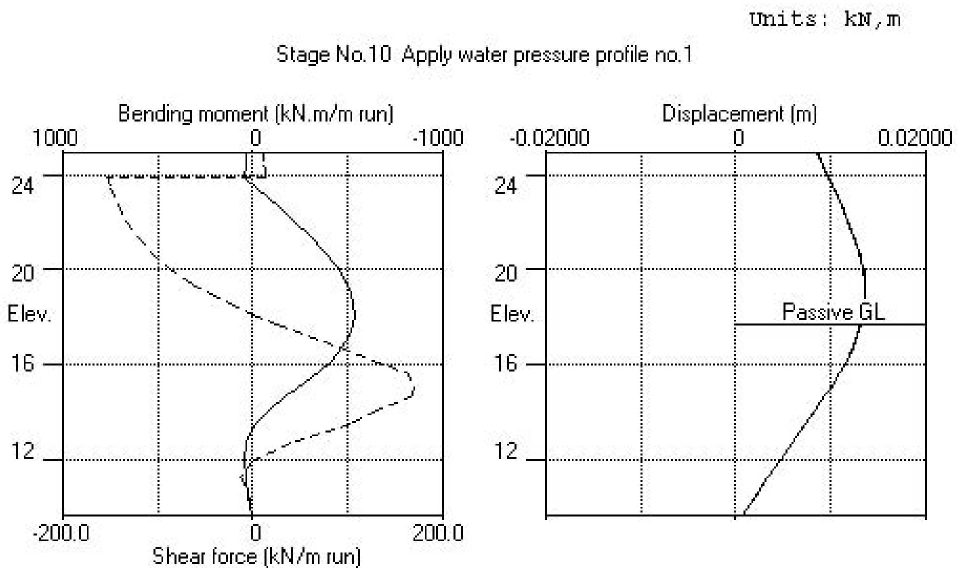

Figure 1 WALLAP outputs

To achieve the equilibrium and factor of safety required, the embedment depth of the bored piles was determined to be 6m into extremely weathered to moderately weathered rock, and the axial force in the passive anchor was 210kN.

A load factor of 1.5 is applied to obtain the design anchor load:
S* = 1.5x210 = 315 kN

4.2 Anchor Design

A diameter of 36mm and an ultimate tensile strength of 500MPa were proposed for the anchors. The design tensile strength of the bars was 321kN which is greater than the design anchor load.

As 7m of the anchor length was within the active wedge behind the bored pile wall, bond stress contributed by this interval was not included in the pull-out resistance of the bars.

The bond stress between the bar and the grout is assumed to be 1MPa according to BS8089. The minimum required bond length for the interface between the bar and grout is 5.3m.

The bond section of the anchor was in extremely weathered siltstone and sandstone. Field pull-out test results showed an ultimate bond stress of 200kPa for these materials. The minimum required bond length for the interface between the grout and the weather rock was 7.9m. Therefore 8m of bond length was selected.

A summary of the anchor design is given in the table below:

Table 4 A Summary of Bored Pile Wall Design Using WALLAP

Bar Size (mm)	Spacing (m)	Inclination (deg)	Depth (m)	Free Length (m)	Bond Length (m)	Total Length (m)	Hole Dia. (mm)	Working Load (kN/anchor)
36	1	30	1	7	8	15	150	191

4.3 Plaxis Analyses

The staged excavation and construction sequence as well as the interaction between bored piles and the soil/rock were modelled using the finite element program PLAXIS 2D.

The PLAXIS results show that the predicted bored pile wall deflection is about 11mm and ground displacements at the swimming pool are within 5mm. The calculated forces in the piles and anchors are given in Table 5.

Table 5 Response of Bored Pile and Anchor after Full Excavation

Maximum unfactored bending moment (kNm)	Maximum unfactored shear force (kN)	Maximum axial force in anchors (kN)
239	143	95

Different results were obtained from WALLAP and PLAXIS 2D as both programs used different methodology The highest forces from these two programs were adopted for the structural design of the piles and anchors. WALLAP predicted higher shear force, bending moment and anchor force than PLAXIS. Reasons for this may include that PLAXIS considers redistribution of forces and moment due to soil-structure interaction whilst WALLAP considers full mobilisation of earth pressures.

5 GLOBAL STABILITY OF THE BORED PILE WALL

Global stability analyses have been carried out for the bored pile wall using SLOPE/W. The potential failure slip surfaces were assumed to pass beneath the pile toe. Two cases were investigated. In one case, parameters given in Table 1 were adopted in the Slope/W analysis. In the other case, as the piles are embedded in weathered rock, global failure slip surfaces passing through anticipated defects have also been investigated. In this case, assumed shear strength parameters for the defects in SAN-5 or better rock include cohesion = 10 kPa and friction angle = 35^{o}. Factor of Safety greater than 1.5 were achieved using this approach.

6 SITE MONITORING

Inclinometers were installed inside two bored piles with one (Inclinometer 46) located along the middle of the wall and the other (Inclinometer 51) at the end of the wall, in front of the swimming pool. Raw data were submitted to the design team by TMR for monitoring, analysis and verification of the design. The measured results and predicted results during design are shown in Figure 2. A threshold of pile head movement advised by the structural team is also given in the figure.

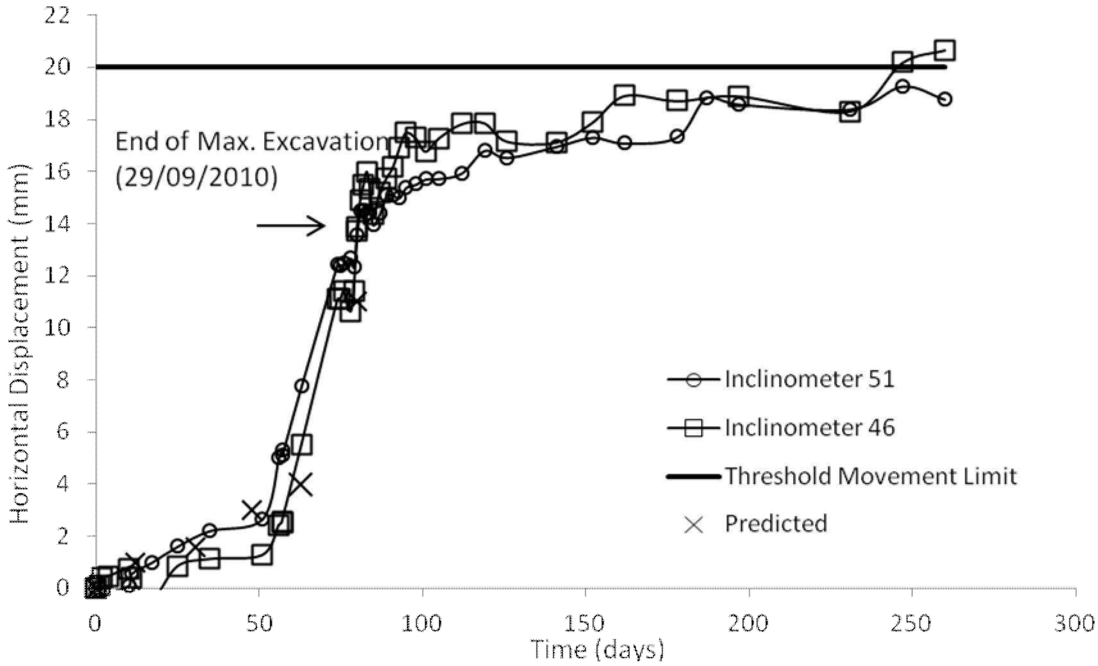

Figure 2 Horizontal wall movements

The maximum wall horizontal movements were in the order of 20mm, which was about 0.2% of the maximum excavation (i.e. 9m). Around 70% of the total movements (i.e. 13.5mm) were completed at the end of the maximum excavation. However the piles continued to move after the excavation was completed. The latter part of pile movements might be caused by creep between the ground anchors and surrounding soil.

The predicted pile head movements matched reasonably well with the site measurements until the end of the maximum excavation. After the excavation was completed, the modelling could not predict further creep like movements due to the limitation of the program. However, site inspections of the wall and the swimming pool were undertaken, and both structures performed well.

Readings currently in processing indicate the rate of wall movement beyond the dates shown in Figure 2 have levelled off.

7 CONCLUSIONS

Anchored bored piles have been used on the IMU-D2G project to provide support to a 9m high excavation. This wall presented a particular challenge to the designer and the contractor as a 25m concrete swimming pool was located behind the wall and was sensitive to ground movements. Limit state principles were followed to design the bored pile wall involving checking embedment depth using WALLAP, overturning of rock socket, global stability using SLOPW/W, passive anchor design, and serviceability of the wall based on a finite element approach. The ground movements measured during the excavation and construction of the anchored bored pile wall compared favourably with the results from the finite element modelling. However, the retaining wall continued with creep like movements after the excavation. The total wall movements were still within the limiting values and the anchored bored pile wall performed well for the project.

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