GEOTECHNICAL ASPECTS OF THE DESIGN AND CONSTRUCTION OF TEMPORARY WORKS FOR SUPPORT OF RAIL RECEIVAL PIT 3 AT DBCT

Sam Paterson\(^1\) and Mogana Sundaram\(^2\)
\(^1\)Worley Parsons, Albert St, Brisbane QLD, \(^2\)Connell Wagner, Spring Hill, Brisbane, QLD

ABSTRACT
Temporary works were required to support a near vertical excavation of approximately 50 m by 20 m in plan and up to 22 m depth, to allow the construction of a rail inloading pit. Significant constraints were placed on the construction by existing industrial infrastructure and site operations.

The temporary works construction comprised an upper anchored contiguous bored pile wall supporting fill, surficial soils and weathered rock and a lower passive rockbolt support system in the rock beneath the base of the piles. The design of the anchored pile wall and preliminary design of the passive rockbolt system were undertaken on the basis of information gathered during site investigations. The passive rockbolt system was amended and optimised as in situ rock data was obtained and further laboratory testing undertaken. In response to the high risk to the existing infrastructure of collapse the performance of the excavation was monitored by an array of survey monuments, inclinometers and extensometers and compared with modelled behaviour.

Data obtained from monitoring and site readings are presented.

Construcational constraints and the methods used to deal with them are discussed. Logic processes in the probabilistic design of rockbolt support in deep excavations and the interpretation of data obtained are also discussed.

1 INTRODUCTION
As part of the stage 7 expansion works at Dalrymple Bay Coal Terminal an additional rail inloading facility was required. The rail inloader is constructed alongside the two existing rail inloaders at the site and designated as Rail Receival Pit 3 (RRP3). RRP3 is of reinforced concrete construction and was built inside a temporary works excavation. This paper details the geotechnical aspects of the temporary works design with particular emphasis on the design of rockbolt support. The existing rail receival pits and inloading conveyors were to continue working without disruption at all times and any disruption to the operations caused by geotechnical failure would have had very significant financial consequences on the Coal Terminal operations. The level of analysis undertaken and factors of safety applied for the temporary works reflect this.

2 SITE LAYOUT AND INTERACTIONS
RRP3 is located on the east side of the existing receival pits and is constructed between inloading conveyors which bound RRP3 to the north and south. To the west is the first inloading pit (RRP1), ancillary plant external to RRP1 was to be maintained and be accessible for the duration of the works, further limiting the working area. Figure 1 shows a plan view of the plant layout; Figure 2 shows a section through RRP3 showing the relative location of the inloading conveyor tunnels. RRP3 is approximately 50 m in length and 20 m in width and has a depth of 12 m for majority of the structure with a deepened sump at the south end of 22 m maximum depth from surface. The proximity of the existing infrastructure required the temporary works to achieve near vertical construction on three sides of RRP3. Construction access to RRP3 was from the east, with the excavation for the conveyor tunnel becoming the haul route.

\(^1\) Formally Connell Hatch, 433 Boundary Street, Spring Hill, 4004 QLD
3 SITE INVESTIGATION INFORMATION

Site investigation comprising seven (7) auger boreholes extended by rotary coring to depth of up to 24 m was undertaken at RRP3 and showed that in the location of the north, south and western sides, significant depths of fill materials (up to 7 m) associated with open cut excavation for the existing structures were present, overlying a fractured rock mass mostly comprising granodiorite. Boreholes in the adjacent ground showed the undisturbed profile to comprise a thinly developed residual soil and weathered zone of less than 2 m overlying rock. The proximity of the walls of the temporary works to the existing structures and the layout of RRP3 meant that all sides to be supported were influenced by fill materials. In situ and laboratory testing showed the fill materials to be medium dense, clay-bound granular in nature and the rock to be generally slightly weathered to fresh, with a fracture spacing of 60 – 200 mm and with UCS in excess of 100 MPa (Is correlation with UCS of approximately 12).

Some historic investigation information was available, undertaken prior to the construction of RRP1. Of most interest was indicative joint orientation information obtained from a dozer pit. No details of temporary works undertaken for either of the existing rail receival pits were available with the exception of some photographs of the construction of RRP2, which showed an open cut with battered slopes in soils and near vertical excavation in rock with strand anchors used as support.

4 DESIGN DEVELOPMENT

Initial design development centred on a full depth piled wall using contiguous 450mm diameter bored piles with multiple anchor levels. This design approach was favoured for reasons of limiting the time to undertake the excavation, when compared with installing rock bolts and for providing a more predictable behaviour model. This option was issued as the conforming design, however during the tender period it was elected by the designer to develop an alternative design comprising contiguous 900 mm diameter bored piles drilled to rockhead, with individual anchors and rockbolt support beneath the piles. The bidders were also asked to provide a cost for the second option and, given the considerable cost savings, the second option was used in construction. The final design was based on 900 mm diameter bored piles at 2 m centres with an upper and lower anchor and shotcrete infill between piles. For the purposes of tender evaluation a nominal schedule for rockbolts, shotcrete, mesh and drainage was included based on experience and comparison with empirical methods (Bieniawski, 1989; Romana, 1985).

The design included external dewatering, but drainage was included in about 75% of the works to account for malfunction in the dewatering system.

Monitoring included in the design comprised five inclinometers to the full depth of the excavation, external to the piles on the south and west walls and within the piles on the northern wall (where the piles were installed to the full depth of the excavation). In addition a system of survey monuments was installed on the periphery of the excavation and allowance was made for ten extensometers to be installed in the rockbolted section of the works. During the works, the contractor also installed survey targets on all piles which were monitored daily.
5 DESIGN ANALYSIS

On the basis of the site investigation results the soils and rock were characterised for analysis, Mohr Coulomb parameters being defined for the soils and modified Hoek-Brown parameters defined for the rock (Hoek et al., 2002). A geotechnical factor of safety was included in the analysis by applying strength reduction factors (SRF) to the parameters for analysis (Hamrah et al., 2004). Analysis was undertaken using both the most credible parameters and the SRF parameters. The in situ stress within the section of excavation in rock was unknown and this was subject to a further sensitivity analysis. In considering the results of these analyses it was noted that the use of Hoek Brown parameters includes the assumption that continuum is maintained in the rock mass.

Rock modulus parameters were calculated using the recommendations of Hoek and, based on experience and comparison with other authors’ measurements in different rocks (Bertuzzi and Pells, 2002), it was recognised that dilation would take place and the calculated Modulus values were reduced by a factor of 4.

The full excavation was modelled using Rocscience Finite Element Analysis (FEA) package Phase®. Elasto-plastic analyses using both Mohr Coulomb and Hoek Brown shear strength models were undertaken and all structural elements were included in the analysis. The FEA modelling fulfilled three design objectives: demonstrated geotechnical stability, provided structural loads for the pile and anchor design and provided deformation profiles for the excavation during construction. The envelope of movement from the analyses was used to give movement limits for the inclinometers that would trigger action by the design team. It is acknowledged that reduced rock modulus values would influence the displacements. It is considered that the resultant FEA displacement profiles were more credible than those generated using the higher modulus value. It was also recognised that the modulus value does not preclude the instrumentation from identifying free body movement in the monitoring results.

The FEA modelling also calculated the relative displacements that the existing infrastructure would experience, showing that the works would have little influence on the existing structures.

The bored pile wall design was also analysed using Geoslope Wallap v5, (referred to as Beam on Springs analysis) to provide verification of the FEA design. In all cases the structural design was undertaken on the basis of the most credible parameters to avoid compounding factors of safety and the ultimate capacity checked against the SRF results.

Direct comparison between the FEA and the Beam on Springs analysis is difficult as the parameters used are different. Of particular note was the significantly higher shears at the pile toe embedment prior to the installation of the second anchor in the Beam on Springs analysis. The decision was taken to use the more conservative results due to the greater body of experience with the Beam on Springs model.

A feature of the FEA package was the ability to model fully-bonded rock bolts (as proposed in the design). These were included in the model for the calibration of expected movements. The FEA continuum model is considered by the authors to preclude the use of this information in anything other than indicative rockbolt design and deflection assessment for such works.

6 PILE INSTALLATION AND MONITORING

All piles were installed to design depth and prior to the commencement of excavation inclinometers were installed and dewatering commenced. As a result of issues relating to the training of monitoring staff and the change of personnel, measurements taken at the early stages of the work were inconsistent and a definitive datum was not established before excavation started. Further issues occurred when surface site operations caused disturbance. For the purposes of monitoring the works, the absence of a datum was of limited influence as the action limits were easily set to incremental movements between excavation lifts and presentation of the full result set has little meaning. A rigorous analysis of the data indicated that pile movements were significantly less than had been demonstrated by modelling. The range of movements being generally less than 5 mm and, in the instance of the piles on the northern wall which extended beyond the base of the excavation, less than 2 mm. This was generally confirmed by the survey target monitoring, but given the limited accuracy of this method (approx +/-2 mm), direct correlation was difficult and an averaging technique was used to assess the data. Of particular note during this phase of the works was that survey target monitoring indicated that a number of piles moved approximately 4 mm when the toe was exposed (having only moved 2-3 mm prior to this). On the basis of the analyses undertaken this would indicate that the toe was having a much greater effect than was anticipated in design.
7 ROCKBOLT DESIGN

7.1 DISCONTINUITY DATA COLLECTION

Site activities and excavations to provide access to the RRP3 provided exposures of rock close to the RRP3 site from which Geotechnical site staff collected discontinuity data prior to the excavation extending beyond the base of the piles.

A total of 843 readings were taken prior to the commencement of rock bolting, increasing to more than 1000 readings in the course of the temporary work (Figure 3). Discontinuity data was managed using Rocscience program DIPS, Ver 5 which provides facilities for stereographic projection and contouring of the data. Using DIPS, data sets can be identified and the statistical distribution of the data sets can be viewed, including joint condition data.

7.2 SHEAR STRENGTH OF DISCONTINUITIES

The shear strength of the discontinuities was modelled using the Barton and Bandis strength model (Barton and Bandis, 1990), which requires the following parameters; basic friction angle ($\phi'_b$ equivalent to a smooth surface), joint compressive strength (JCS) and joint roughness coefficient (JRC).

Block samples of discontinuities were taken from site and dispatched to AG labs in Milton, Brisbane for shearbox testing. The first tests were undertaken to obtain $\phi'_b$, and two samples were prepared by saw cutting and grinding flat. The results recorded $\phi'_b$ values of 45.5° and 46° which were much higher than anticipated. A further six tests were undertaken on natural joint surfaces, the results showed significantly lower $\phi'$ values (22° – 39.5°) than the $\phi'_b$ values. A review of the testing was undertaken with the laboratory manager and the following issues were identified;

- All tests were undertaken in a submerged condition and it was thought that in the instance of the $\phi'_b$ tests that a thin film of water on the shear surface may have caused suction analogous to that which occurs when two plates of glass are placed together in the presence of water.
- A number of the samples had fractured into small pieces during preparation and had been re-set by the lab technicians. The resulting contact surfaces were not always a good match with the complementary sample.

From this review it was elected to reject the $\phi'_b$ tests, as the results could not be justified when compared with other results. Results from tests where low $\phi'$ values were obtained corresponded with the fractured and re-set samples with poorly matching contact surfaces and these were also rejected.

An additional $\phi'_b$ test undertaken in a dry shearbox recorded a $\phi'_b$ value of 35°, again in excess of the values indicated by other tests. A Barton and Bandis model curve fitting exercise was undertaken on all of the test data to obtain design parameters. The exercise showed that JCS had only limited influence on the curve fit within the range of likely strengths on site. From the curve fitting exercise the following were considered to be site constants for analysis of discontinuities at the site; $\phi'_b = 30°$, JCS= 50 MPa. Joint roughness was recorded in the field by profile gauge and calibrated against the standard profiles (Barton and Choubey, 1977).

7.3 EXTERNAL FORCES

For the purposes of rockbolt design the external forces acting on a given free body were surmised to come from the following sources.

- Point load forces at the toe of piles from the vertical component of the anchors and the weight of the pile.
- Surcharge from overburden.
- Live load surcharge.
- A nominal seismic load of 0.1 g horizontal force (included to account for the effects of low velocity accelerant blasting and vibrations from existing plant).
- A groundwater force equivalent to a 75% discontinuity height water filled crack with a mid point peak force (included to account for de-watering failure and the untested status of the passive drainage).
7.4 STABILITY ANALYSES

From the stereographic projection the discontinuity sets that could influence stability for each of the excavation faces were identified. Two modes of failure were considered for analysis: Planar Failure for parallel and sub parallel striking joint sets and Wedge Failure where the intersection of up to three joint sets (including a release plane) could cause failure.

Analysis was undertaken using Rocscience SWEDGE Ver 4.0 to analyse wedge failure and Rocscience ROCPLANE Ver 2.0 to analyse planar failure. Both programs allow the input of statistical distributions of data and undertake stability analyses that outputs both a Factor of Safety (FOS) of the mean value and probability of failure expressed as a percentage of the cases analysed (PF%).

The planar failure analysis program allowed the use of a Barton and Bandis shear strength model and also calculated the surcharge load from overburden in relation to geometry.

The wedge analysis program allowed only a point load surcharge to be input and allowed only the use of Mohr Coulomb shear strength parameters. This limitation was overcome, to an extent, by undertaking preliminary analyses to find a mean wedge size and then interrogating the model to get a normal force on each discontinuity face of the wedge followed by calculation of the equivalent Mohr Coulomb parameters from the Barton and Bandis model. The mean wedge surface size was also used to calculate a surcharge load which was added to the point load to give a vertical load.

For both planar and wedge failure analysis a number of face heights were analysed to determine the worst case.

Both programs have the limitation that any bolt that penetrates the failure surface is included in the analysis at full capacity. To accommodate this limitation only a horizontal force was included in the analysis. A spreadsheet was written to calculate the bolt force that could be expected from a given bolt pattern and then the bolt patterns were reviewed to meet the requirements of the stability analyses. The spreadsheet formulation included recognition of the following for a given pattern bolt system:

- The upper bolts may not intersect failure planes at greater wall heights;
- Where bolts have a limited embedment beyond the failure plane the bond capacity is likely to determine the contribution to support;
- Where bolts have a significant embedment tendon capacity is the limiting factor;
- Where bolts extend significantly beyond the worst case failure plane they can be shortened (this is mainly relevant towards the base of the excavation).
Shotcrete was included in the works; however as it was an option to use only bolts and mesh for support it was not included in the analysis as a stabilising force. It was recognised that that this was a conservative assumption particularly for smaller wedge-formed free bodies.

Sections were analysed for each face and the most onerous support requirement from either planar failure or wedge failure analysis was considered and a pattern bolt system assessed to meet the loading requirements.

With the bolt force applied the following FOS were sought:

- Planar Failure; Mean Dip FOS>=1.5 PF%=0; Worst Case Dip FOS>1 PF%<10%
- Wedge Failure (constant shear strength parameters); FOS>1.5, PF<1%

Upon review of the stability analysis results it was decided that in a number of instances relating to the upper parts of the excavation (smaller wedge defined free bodies), the surcharge load may have been too onerous and so the criteria for probability of failure was relaxed in some instances. The analysis showed that where the required probability of failure criteria (PF%) were achieved, the FOS was significantly in excess of the required value, reflecting the deviation in parameters.

7.5 PASSIVE VERSUS ACTIVE BOLTING

Design was undertaken to consider both active and passive bolt configurations and site tests were undertaken on both to determine bond strengths. During the testing a small number of test anchors failed at very low loads. This was attributed to difficulty experienced by the contractor in getting the resin capsules to mix correctly. Given the uncertainty, the contractor elected to use fully grouted passive anchors. The influence on the design was to necessitate an increase in anchor length of approximately 20% of the active anchor length. During the course of the works, 5% of the installed bolts were tested and met the requirements.

8 MONITORING

Having identified the potential failure plane locations for each face an array of 10No extensometers were installed to monitor dilation. The monitoring data from the inclinometers had stabilised by the time that excavation commenced beneath the toe of the piles and had recorded no discernable movement below the piles. Therefore the inclinometers were operating as intended at the rock bolting level.

Figure 4: Typical Extensometer Displacement vs time (depth of excavation). Minor variations in movement between excavations was due to external factors (diurnal variations).
As would be expected all displacements showed convergence of the face towards the pit and that rock mass dilation was maximum nearest to the excavation face, diminishing with depth. The trend was similar in all extensometers but with varying magnitude. The maximum recorded rock mass dilation was 4 mm at 1 m from wall (Figure 4). The dilation diminished at a distance of 4 m from the face. Dilation occurred almost immediately following each stage of excavation and stabilised gradually once the excavation was complete. The successful monitoring was integral to proving that support works were performing as designed and when considered with favourable site observations gave site staff confidence to amend the design as the excavation proceeded.

Figure 5: Deepest section of RRP3 is being excavated. Existing coal loading facilities are in the background. View towards west, facing conveyor tunnel. Location of extensometers are shown with circles.

9 CONCLUSIONS
The site constraints and ongoing site operations placed significant restrictions on construction techniques and also placed the temporary works in a risk context more usually associated with a congested urban environment. The upper part of the temporary works was controlled by the use of structural elements (piles and anchors) within the design, for which safety factors could be readily incorporated. The design of the rockbolt support system required an observational approach such that the support measures could only be accurately defined once excavation had commenced and site measurements made to verify the relevant assumptions. Much of this risk was mitigated by the data gathering undertaken in areas adjacent to the RRP3 excavation. It is considered that a rigorous and defensible design exercise was undertaken which was verified, and controlled, on site using the installed monitoring as the works proceeded.

In the design phase the excavation was modelled using FEA (where it is considered as a continuum) to determine global stability of the retaining system by the use of SRF and to model movement and the influence on nearby structures. The
parameters for the analysis were readily derived by study and analysis of the site investigation data with the notable exception of \textit{in situ} stress which was subject to sensitivity analysis. The analysis and design phases demonstrated that, based on the available information, the proposed temporary works were adequate for the purpose provided that the rock mass was maintained in continuum.

Following the commencement of site works a rockbolt support system was designed using a probabilistic approach based on the data gathered during the site works prior to the excavation reaching the depths at which rock bolts were to be installed. Although in the instance of the wedge stability a number of compromises were required that arguably mean that it was semi-probabilistic (software updates are likely to improve this issue). The overriding assumption governing the rock bolt design is that any given joint would be considered to be sufficiently continuous to define the free body whilst still obeying the prescribed shear strength parameters. This is arguably an over-conservative assumption for a fractured rock mass, however in the context of the consequence of failure it is considered to be a reasonable argument.

The installation and monitoring of instrumentation provided the site engineers with a means of confirming that the temporary works were performing as designed and also provided a rationale for undertaking various actions (including emergency evacuation if necessary) in response to the readings obtained hence providing further risk control.

The authors consider that given the inherent variability in discontinuity orientation the only true measure of stability is that given by a probabilistic approach. It is also the authors’ experience that true normal distributions with a low order standard deviation are rarely recorded in practice.

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