Design & performance of a temporary anchored soldier pile wall

3rd CAR DUMPER PIT, BHP IRON ORE FACILITY, NELSON POINT, PORT HEDLAND

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1.0 INTRODUCTION

In 1997 BHP Iron Ore commenced construction of the Capacity Expansion Project (CEP) at the iron ore handling and loading facility at Nelson Point, Port Hedland, WA. One of the key elements of the project was the construction of a new car dumper located between the existing No.1 and No.2 car dumpers. The close proximity of the adjacent structures necessitated the adoption of a 24.0 m deep, vertical excavation for the new car dumper pit. The retaining wall adopted to support the excavation was a temporary anchored soldier pile wall. The piles were 900mm in diameter, spaced at 2.5m centres and extending to a total depth of 26.0m. Three rows of temporary anchors were used to provide adequate lateral restraint.

Sinclair Knight Merz provided the Engineering, Procurement and Construction Management (EPCM) for the project and Soil & Rock Engineering were commissioned, as specialist geotechnical consultants, to review the retaining wall design, develop an appropriate monitoring system and carry out on site technical supervision of the geotechnical works.

2.0 DESIGN REVIEW

The design review by Soil & Rock Engineering was required to determine that an appropriate design approach had been adopted and that adequate factors of safety (FoS) were maintained

Following a review of Australian and international publications, assessment was made of the applicability of the soil/structure model and the method of analysis used to design the retaining wall. Detailed consideration was given to the earth pressure coefficients, and the validity of the Wallap software package, used for the analysis.

A parallel finite element analysis of the proposed retaining wall was carried out by Soil & Rock Engineering, using the Wallap programme, and included a sensitivity analysis to assess the results obtained with reduced anchor pre-stress loads and reduced values for the drained cohesion (c') for the lower soil strata.

An additional independent analysis of the wall and anchors was also carried out using a finite difference software package called FLAC.

The ground anchor design and testing regime was reviewed to ensure that appropriate systems were adopted and that acceptable FoS were maintained.

A monitoring package was defined to ensure that the optimum arrangements for installation, data collection and review/analysis were obtained. The monitoring was required to provide information on ground water draw down, wall deflections, anchor loads and settlements of the ground surface to ensure safe construction of the new car dumper while maintaining full operation capability of the existing plant.

The soil strata at the site comprised a sequence of dredged sand fill, soft clay known as Mangrove Mud, Calcarenite, a medium dense sand known as Paleosol, Upper and Lower Red Beds and a conglomerate bedrock strata. The soil structure model developed for the design and analysis of the retaining wall is presented in Sketch 1.

3.0 LITERATURE REVIEW

A literature of relevant international standards and technical indicated that there was no current Australian standard for the design of deep retaining walls or ground anchors, the Austroads Bridge Design Code was, however, found to be of some relevance.

The Australian Standard, Draft 96405, Earth Retaining Structures, 1996. was relevant only to retaining structures up to 15.0m in height.

The following publications were found to be relevant and were widely used in completing the design review, analysis and reporting work, carried out by Soil & Rock Engineering:

BS 8081:1989 - British Standard Code of Practice for Ground Anchorages.

CIRIA Report 104:1984, Design of Retaining Walls Embedded in Stiff Clays.

Clayton C.R.I., Retaining Structures UK Institution of Civil Engineers, 1993.

UK Civil Engineering Code of Practice (CP2).Earth Retaining Structures, 1951.

Contained in the UK Institution of Civil Engineers (ICE) publication Retaining Structures, edited by C.R.I. Clayton, are a number of papers on multi propped anchored retaining walls. Of particular interest were papers by Grose & Toone and Brooks & Spence in which the WALLAP and FLAC programmes were used to analyse deep retaining walls. A good correlation between recorded and WALLAP predicted wall performance is indicated, although it is noted that measured wall deflections were generally less than obtained from the WALLAP analysis.

4.0 APPROPRIATE METHOD OF ANALYSIS

For cantilever or single propped walls, a simple consideration of the limiting equilibrium condition, such as the active/passive approach, produces adequate information on wall stability. The results obtained can be used for the structural design of the wall element, and anchors/props, provided adequate precautionary measures are adopted to allow for the simplification inherent in this design approach. CIRIA report 104 recommends that strut or anchor loads obtained in this manner be increased by 25% to allow for these limitations.

Where more than one level of propping is provided, a degree of redundancy is introduced and a limiting equilibrium analysis is not appropriate. The design pressures applied to the back of a multi level propped retaining wall would lie between the upper limit of *at rest* and lower limit of *active* earth pressures. The actual magnitude of the applied pressure is influenced by the initial at rest (Ko) condition for the soil strata, the method of wall installation, the wall and soil stiffness properties, wall deflections, the location, stiffness and pre-stressing of the anchors and the sequence of excavation. Due to the complexity of the analysis, finite element analysis programmes have been developed to enable the wall and soil elastic properties to be modeled thereby producing predictions of wall movements, anchor loads and design pressures for multi-propped walls.

The retaining wall design carried out for the Port Hedland project utilised the 2 - D finite element analysis available within the software package WALLAP. The analysis is carried out with staged development of forces and wall movements as the sequential construction process proceeds. The software analyses pressures and deflections as the excavation is advanced, anchors are installed and stressed and ground water levels are drawn down.

The UK Construction Industry Research and Information (CIRIA) Report 104, The Design of Retaining Walls Embedded in Stiff Clays, is written primarily for walls in clay but contains many widely adopted practices and recommendations. The following relevant observations were found in CIRIA 104:

The pressure distribution used for design of the wall should generally be the same as that used for calculation of the overall stability, i.e. based on active and passive limits of earth pressure. Thus the use of results from methods of analysis such as WALLAP are valid for consideration of the piled retaining wall and anchor structural capacity at the limiting equilibrium conditions.

It is possible that the earth pressures applied behind the wall are greater than active, particularly for stiff buried structures, where the soil strata is over consolidated giving high in-situ horizontal stresses and/or where a flexible wall allows soil arching to occur.

Due to deflections during the conventional excavation process, it is likely that the pressures behind the wall will reduce to approach the lower limit of the active pressures. The actual design pressure on the back of the wall will lie between at rest (Ko) and active (Ka) earth pressures. The actual magnitude of the applied pressure depends on the restraint-deformation characteristics of the anchor / strut system.

A factor of safety of 2.0 should be applied to the prop or anchor system and redundancy should be built in such that overall wall failure does not directly result from the failure of any one prop or anchor.

The retaining wall analysis was based on specified groundwater profiles which assumed that the dewatering system was effective in lowering the groundwater to at least 1.0m below each excavation level.

5.0 EARTH PRESSURE COEFFICIENTS

5.1 At Rest Earth Pressure Coefficient

Analysis to determine retaining wall stability and structural loads requires definition of the initial *at rest* (Ko) earth pressures as well as the active and passive earth pressure limits. The value of Ko is not a fundamental soil property but represents the ratio of insitu horizontal effective stress to vertical effective stress. For normally consolidated soils the value of Ko is accepted as being reasonably well defined by the following equation:

$$Ko = 1 - \sin(\Phi)$$

For soils, which are over consolidated, due to a previous reduction in vertical stress, the value of Ko is higher and can exceed unity, i.e. the horizontal stresses locked into the soil strata can be higher than the current vertical stress.

It was apparent from the project geotechnical investigation report that significant difficulty was encountered in determining the insitu value of horizontal stresses in the Red Beds and accordingly, definition of Ko was problematical. Both the Upper and Lower Red Beds are described as being cemented fluvial and alluvial sediments. Due to the effects of cementation and ageing, these strata are described as quasi-over consolidated and have consolidation characteristics compatible with an overconsolidated material. Since the quasi-overconsolidation results from effects such as cementation, rather than any historical reduction in vertical effective stress, that there is no direct correlation between Ko and the degree of quasi-overconsolidation.

The geotechnical investigation report stated that due to the difficulty in actually determining the ratio of horizontal to vertical stresses that a value of unity be adopted for Ko in the Red Beds. This approach, with Ko = 1.0, was adopted for the wall design and gave a conservative prediction of the initial insitu stresses in the ground and so produced *safe* results from the analysis.

5.2 Active and Passive Earth Pressure Coefficients

The WALLAP program requires definition of the limiting Active and Passive earth pressures. The limiting pressures applied to the wall, for design, are determined by the following input parameters :

Φ the angle of soil friction.
c' the drained shear strength of the soil.
K_a and K_{ac} the active earth pressure coefficients.
K_p and K_{pc} the passive earth pressure coefficients.

The value of Φ for each soil strata was documented in the geotechnical investigation report. The accepted approach for retaining wall design is to adopt moderately conservative values for strength parameters and apply an appropriate Factor of Safety (FoS) to the design for stability (typically 1.2 to 1.5).

Values of c' for the various soil strata were given in the geotechnical investigation report and it was concluded that c' = 50kPa was reasonable for the Red Beds. This value was primarily based on back analysis of the excavation for Car Dumper Pit No.2, inspection of photographs, and earlier stability assessment reports, confirmed that a stable open cut excavation had been achieved at the design batters.

It was noted that reduced values of c' should be considered as representative of localised, less cemented, layers within the lower strata. The value of c' adopted has a significant effect on the pressures applied to the wall and it is recommended that high values be adopted with caution. It was considered appropriate to investigate the influence on the design of lower values of the drained cohesion, c', to assess the sensitivity of the solution to the value likely to be available in the field.

Additional data, specifically on the drained strength c' for the Red Beds was available from a second geotechnical report. Of particular relevance was the conclusion, after detailed additional triaxial testing, that c'= 50kPa and Φ =35° were appropriate parameters for design. It was noted that tests carried out on remolded samples indicate values of c' of 75kPa and 88kPa. These results suggest that the Red Beds are not a particularly brittle material and that the drained strength is not directly associated with cementation.

The values of Ka, Kac, Kp and Kpc define the limiting active and passive earth pressures in a given strata. For a horizontal ground strata, the values of Ka and Kp are influenced by the soil friction angle Φ and the angle of wall friction δ_w The active and passive pressure coefficients for cohesion, Kac and Kpc, are determined from the value of Ka or Kp and the ratio of wall adhesion cw to the drained shear strength c'. This approach, incorporating soil friction and cohesion with appropriate levels of wall friction and wall cohesion, has been utilised for some time and the recommended design methods in both CP2 and the British Steel Piling Handbook incorporate the adoption of soil cohesion, wall friction and wall adhesion.

Provided downward wall movement due to high vertical pressure is not significant, it is recommended that δ_w = 2/3 Φ be adopted in the determination of Ka. One convenient, and widely adopted, set of values are those derived by Caquot and Kerisel which are reproduced in CIRIA Report 104.

The values adopted in the original design had been derived from Eurocode 7 and were in reasonable agreement with the values obtained from Caquot and Kerisel with $\delta_w = 2/3\ \Phi$.

The values of Kac to be used are derived from the following relationship:

$$Kac = 2((Ka \times [1+c_w/c'].$$

It is generally recommended that the wall adhesion cw, be ignored and so Kac = 2(Ka.

The values adopted in the original design were derived from Eurocode 7 and were higher than those obtained from Caquot and Kerisel with $\delta_w = 1/3\ \Phi$. If, as is recommended in CIRIA report 104, cw is ignored, the value of Kpc to be used is given by the relationship Kpc = 2 (Kp. (?)

Care needs to be taken in the evaluation of passive pressure coefficients, particularly where the value adopted exceeds 3.0, due to vertical equilibrium considerations. In addition passive reaction deformations are often relatively large and the wall deflection required to fully mobilise these forces may not be acceptable from structural considerations. On the basis of the above factors reduced values of Kp were adopted for the final wall design.

6.0 DESIGN EARTH PRESSURES ON PILE WALL

It has been mentioned that the actual earth pressures acting on the back of the wall will lie between at rest pressures (Ko) and active pressures (Ka). Retaining walls, particularly those which rely on passive resistance in front of the wall toe, will rotate and move forward as excavation is carried out. This forward displacement will reduce the pressures on the wall from at rest down towards active pressures.

Measured field data, model tests and consideration of the definition of active failure indicate that relatively small wall displacements reduce the pressures on the wall to the active condition. Published test data indicates that the active earth pressure condition is reached at displacements in the order of S/H = 0.0015, where S = wall displacement and H = wall height. For the wall at Port Hedland, the final wall height H_{max} = 23.8m. On this basis, horizontal wall deflections of up to 36mm would be compatible with reaching the active condition behind the wall. The wall displacements predicted by the design were in the order of 20mm to 26mm. It is relevant to note that the predicted and actual horizontal deflections would be directly influenced by the level, stiffness and pre-stress load of the anchors used to restrain the pile wall.

7.0 ANCHOR DESIGN LOADS

The waler load obtained from the WALLAP 2 - D Finite Element analysis was used to define the anchor design loads. This analysis imposes pressures on the wall between at rest and active with these pressures being derived from predicted wall and soil deflections and assumed elastic properties.

It is recommended in CIRIA Report 104, and other publications, that where limiting equilibrium analysis has been used to determine waler loads, with assumed active earth pressures on the back of the wall, the design load for the anchors should be increased by 25% to allow for soil arching and stress redistribution effects etc. This approach can result in a general expectation that the anchor loads derived from analysis are lower than those which should be adopted for design purposes. It must be noted however that this approach does not apply when finite element analysis, such as that used in WALLAP, has been used, to determine the design pressures applied to the wall.

8.0 SENSITIVITY ANALYSIS USING WALLAP

In order to investigate the sensitivity of the wall design to the method of analysis and assumed c' for the soil strata, a number of different cases were analysed using the WALLAP program.

The magnitude and reliability of the assumed soil cohesion c' was raised as a specific point to be considered in the design appraisal and was addressed in the sensitivity analysis. In order to investigate the available FoS against failure with reduced drained strength in the Calcarenite and Red Beds, additional design cases were investigated. In each of these cases, the Calcarenite strength was reduced to zero to model the situation where the Calcarenite is assumed to have fractured through the full depth of the strata or to be locally uncemented. In addition, the drained strength for the Red Beds and Conglomerate was varied between zero and the values recommended in the geotechnical investigation report to model the range in Red Bed materials in particular where the clay contents and degree of iron and carbonate cementing varies.

Some relevant observations resulting from this sensitivity analysis are presented below :

Analysis with the original soil parameters, utilising reduced anchor pre-stress loads of 250kN for row 1 and 300kN for rows 2 & 3, produced an approximate 38% increase in wall deflections, up to a maximum of 33mm, and anchor loads between 72% and 85% of those obtained in the base model.

Analysis with all parameters as for the base model, with the exception that the shear strength of the Calcarenite was reduced to zero,. showed an increase in the maximum lateral deflection of the wall to 43mm while the anchor loads were observed to increase by between 0% and 18%.

Further design cases had the shear strength of the Calcarenite reduced to zero and adopted 50% of the recommended values of drained strength for the Red Beds and. The results indicated an increase in maximum lateral deflection to 53mm and the anchor loads were seen to increase by between 20% and 38%.

On the basis of the sensitivity analysis, it was concluded that, even based on worst credible soil parameters, the minimum FoS obtained on the wall and anchors would be in excess of approximately 1.15. Based on the high level of monitoring adopted on site, it was agreed that this represented an acceptable design basis for the scheme.

9.0 anchor redundancy

Established practice with temporary works, and specific recommendations in publications such as CIRIA 104, suggest that sufficient structural robustness should be provided such that any system will not fail catastrophically with the loss of any one anchor or prop. One commonly adopted way to achieve this condition is to provide horizontal continuity, generally by means of a waler, such that design loads can be transferred to adjacent anchors in the event of failure of one anchor unit.

The adopted retaining system had anchors located through individual pile shafts and therefore there was no provision for horizontal continuity or load transfer. In order to address the anchor redundancy issue, provision was made for the construction of a reinforced concrete capping beam along the top of the piled retaining wall.

10.0 GROUND ANCHOR DESIGN AND TESTING

The anchor design load was in the order of 900kN and so it was necessary to load the trial anchors up to between 1,800kN and 2,250kN to demonstrate Factors of Safety (FoS) of 2.0 to 2.5. It was appropriate to over reinforce anchors and/or construct anchors with a shortened fixed anchor length (FAL) to ensure adequate test capacity to fully investigate the available soil to ground bond stress.

A value of 2.0 for the FoS at the ground to grout interface was adopted for the anchor design. This is lower than the value of 2.5 recommended BS 8081, however, for temporary anchorages used as a retaining wall tie back where full scale field tests are carried out FoS = 2.0 can be adopted. The results obtained from three full scale trial anchors, installed and tested prior to installation of production anchors, were used as the basis of the design and accordingly, the FoS = 2.0 was considered to be acceptable.

The preliminary trial anchors were installed with reduced FAL in the Calcarenite and with additional strand reinforcement in the Red Beds trial anchors. These arrangements were adopted specifically to enable the trail anchors to be safely loaded beyond twice the design working bond stress. The production anchor design was revised on the basis of the trial anchor tests and to be in accordance with the requirements of BS 8081 in lieu of a relevant Australian Standard.

A number of production anchors in the middle row failed where the FAL was formed in an inappropriate manner in unstable ground below the ground water level. All other anchors performed as predicted with fully elastic behavior up to 150% proof loads and with minimal creep losses over the project period. Remedial middle row anchors, installed with revised techniques, also performed adequately.

11.0 comparison between wallap & flac analysis

In order to check the validity of WALLAP, a separate analysis of the wall was carried out using FLAC. The option to introduce cable, pile and beam structural elements is available in FLAC and was used in the analysis to model the retaining wall.

FLAC provides a more sophisticated analysis of the soil/structure interaction and required considerably more detailed input data. The time required to develop the FLAC analysis was considerable, and it is unlikely that this programme would be used for the design of smaller or more conventional retaining wall structures.

The anchor load and wall deflections results obtained from FLAC compared well with the Wallap analysis, however FLAC predicted greater wall deflections at the final stage of excavation. This behaviour was not observed on site and suggests that the parameters adopted for the lower strata in the FLAC analysis were overly conservative in terms of small strain stiffness. Good agreement was obtained between both software packages for the predicted anchor loads. As noted above, the anchor loads recorded on site were not as high as predicted and, in particular, did not increase as anticipated, as the depth of excavation was increased.

The ability to carry out an independent analysis, with a programme operating on a totally different basis, was extremely valuable in providing conformation on anticipated performance and the overall factors of safety likely to be available during the project. This approach was essential to enable sufficient confidence in the design to be available to commence the excavation, send the workforce into the car dumper pit and ensure continuing safe operation of the adjacent facilities.

12.0 WALL AND ANCHOR MONITORING

12.1 Requirements for Monitoring

The retaining wall design was based on recommended soil parameters with an acceptable analytical model being utilised. It was recognised that there are certain inherent limitations and uncertainties in the analysis and design of retaining structures of this nature. Detailed monitoring of the ground and structure during construction, and performance after excavation, was carried out as follows:

Monitoring of in-service anchor loads to confirm the validity of the design assumptions and give immediate data on the performance of the anchors.

Comprehensive monitoring of wall deflections to confirm the validity of analysis and wall performance as the excavation was progressed.

On site monitoring of the nature and uniformity of the soil strata encountered as the excavation proceeded to facilitate comparison between the as built structure and the design model.

Routine measurement of the ground water drawdown in standpipe piezometers in the area around the car dumper pit to enable comparison with the design ground water levels.

12.2 Nature of Monitoring

Vibrating wire load cells were provided to 10% of all ground anchors, distributed around the perimeter of the pit at each of the three anchor levels. Data was obtained on a daily basis during and immediately after each stage of the excavation. The anchor loads were then monitored for a further period of approximately two months at which stage the base slab was complete.

The adopted method of monitoring wall deflections was to install inclinometer casing, in which a torpedo was lowered to establish base line data prior to commencement of excavation. As excavation, anchoring and construction proceed, repeat surveys with the inclinometer were carried out and wall deflection characteristics obtained. The inclinometer casing was extended beyond the toe of the wall to sufficient depth to be founded in stable strata.

13.0 actual wall performance

The excavation commenced in August 1997 and was successfully completed by October 1997. The anchored pile retaining wall performed largely as predicted with acceptable wall deflections and anchor loads being recorded at all stages of the excavation. Sketches 2 and 3 present typical details of the anchor loads and wall deflections measured during the project.

In general, the anchor loads were less than predicted by the Wallap analysis and, in particular, did not increase by the predicted magnitude as the depth of excavation was increased. Wall deflections were also less than predicted, particularly in the upper strata. This is consistent with other published observations of actual wall performance compared with predicted deflections obtained from Wallap. It is likely that the stiffness of the retained soils, at the relatively small strains associated with the anchored pile wall deflections, is far higher than assumed based on the available soil strata design parameters.

A back analysis of the wall performance was carried out using Wallap to obtain a better correlation between observed and predicted values. The final soil model developed to give reasonable comparison between measured and predicted wall deflections and anchor loads was based on higher values of soil stiffness, reduced values of Ko and increased values for the drained cohesion c' particularly for the upper granular strata.

One particular influence, which appeared to significantly reduce the wall deflections over the upper wall area, was an apparent restraining action provided by the Calcarenite. The excavation was trimmed flush to the inside face of the pile wall over the depth of the Calcarenite and so the rock remained keyed in around the piles. The strata was generally observed to be continuous and of high strength and appeared to act in tension to restrain the wall from moving forward. This effect was modeled in the Wallap back analysis by adding a theoretical external horizontal force applied to the wall at the level of the Calcarenite. This analysis produced the best comparison between predicted and measured wall deflections and anchor loads. Tables 1 and 2 present a summary of the original design parameters and the optimum values determined from the back analysis.

Table 1 ORIGINAL SOIL STRATA PARAMETERS
Table 2 OPTIMUM BACK ANALYSIS SOIL STRATA PARAMETERS

14.0 Conclusions

The analysis, design, monitoring and back analysis of this deep excavation, with the multi level anchored pile retaining wall, has demonstrated that the use of programmes such as Wallap is valid provided appropriate geotechnical parameters and factors of safety are adopted. In particular, it has been demonstrated that the use of an appropriate value for the drained cohesion c' can safely be adopted in the soil types encountered at Port Hedland.

As reported in other published information, it appears that the use of Wallap with conventional soil stiffness parameters over predicts the wall deflections and anchor loads. This produces a safe solution and can generally be adopted where limited information or analysis time is available.

Further monitoring, detailed insitu and laboratory testing of the soils and back analysis of actual wall performance would produce sufficient data to enable more accurate, and therefor less conservative, soil parameters to be used for future designs in comparable soil strata.