Design of High Slopes in Transported Materials – the Pilbara Experience

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ABSTRACT: Slopes consisting of thick profiles of transported sediments are increasingly being developed in the Pilbara. In some open pit mines, thickness of these deposits can exceed 250m. The material types can be highly variable in the sediment profile. Characterizing strength and behaviour assessment of these sediments are complex considering their variability of composition, consolidation history, degree of cementation and induration. The selection of design shear strength parameters needs careful consideration as there is no avenue for conservative slope design or aggressive slopes with poor performance. The methodology used for the design of pit slopes in a thick sequence of transported sediments is discussed where peak shear strengths were used in stability assessment.

1 INTRODUCTION

The Hamersley province of the Pilbara region of North West Australia comprises the bulk of the state's iron ore resources. Three major types of iron ore occurs in the province, namely bedded, channel and detrital iron ore deposits. The bedded ores are derived by the enrichment of banded iron formations (BIF). The channel ores consisting of pisolitic transported sediments occur in Tertiary age paleochannels. The detrital ores are derived from colluvial fans adjacent to bedded deposits.

Geotechnical design studies for a large open pit where part of the orebody lies under a thick sequence of transported Tertiary sediments was conducted recently. The paper summarises the work undertaken, challenges faced with key findings.

2 GEOLOGY

The geology of the project area comprises Archaean age banded iron formations (BIF) and shales overlain by Cenozoic colluvial, fluvial and lacustrine deposits. Typically east-west trending, moderately north dipping strata (30° to 50°) outcrops and forms a low hill line at the southern boundary of the project area. The dip of strata flattens towards north (10°) below a thick sequence of Tertiary sediments deposited in a wide valley formed between the southern ridge and a much higher relief ridge line to the north. The thickness of the sediment sequence varies within the proposed pit area from a few meters next to the southern ridge and increases towards north to more than 250 m. The distribution of bedrock units, residual soil (paleo-regolith) and the transported sediments in the mid-section of the proposed pit is shown in Fig.1.

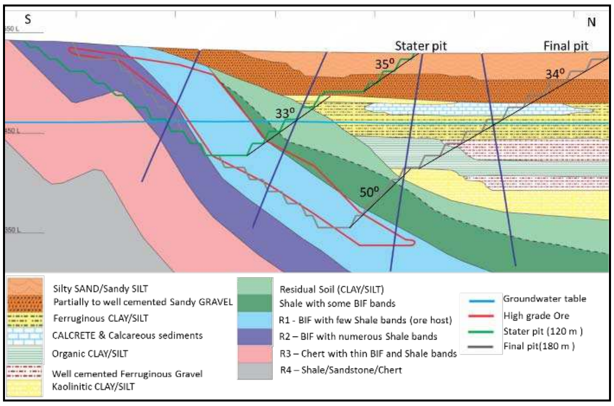

Fig. 1 Geological section through the pit area

2.1 *Cenozoic Sediments*

A varied sequence of Tertiary and Quaternary age sediments occur within the Hamersley province. Depositional environments include a broad range of fine-grained lacustrine deposits, stream and other fluvial environments of mixed grain size, and variously sorted colluvial materials that may have deposited in dry or in subaqueous environments.

2.2 *Sediment Profile in the Project Area*

The material units in the sediment profile have widely differing engineering properties. On the basis of their physical and other geomechanical attributes, the sediment units have been differentiated into geotechnical domains as shown in Table 1.

Table 1. Summary of north wall sediment domains

Sediment domain	**Colour strength and plasticity**	**Dominant material type(s) based on PSD**
Substantial gravels	Partially to well cemented	Well graded sandy gravel in a silty matrix
Mottled materials	Brown mottled grey, stiff, CL, ML & MH	Silty sand (and sandy silt)
Red brown clays	Reddish brown, stiff-very stiff, CH, CL & MH	Silty clay
Calcareous sediments	Light brown, stiff, CH, CL & MH	Silty clay (and sandy silt)
Kaolinite	Light grey, stiff - very stiff, partially indurated, CL, ML & MH	Sandy silty clay (high % of silty clay and sand). Sand layers.
Organic sediments	Dark, very stiff, lignite fragments, partially indurated, CL, ML & MH	Silty clay (high proportions of clayey silt and clayey sand)
Ferruginous gravels	Partially to well cemented	Gravels in a sandy silty matrix
Residual soil	Khaki coloured, stiff soil. ML, MH	Sandy silt (high % of clayey silt and silty clay)

2.3 *Cementation and induration*

The cementation in the coarse grained materials in the sequence is variable. The gravelly material in the upper layers proximal to the southern ridge and the ferruginous gravels at the base of the sequence are generally well cemented. The fines (especially the clay fraction) increase away from the ridge and the material becomes less cemented and weaker. The hard calcrete zones sometimes occur within a zone of weak calcareous clays.

The clayey and silty material domains have been classified as stiff or very stiff and occasionally lightly fissured. Some sections of these materials are in a partially indurated state; i.e. in a physical state between very weak rock and soil.

3 SHEAR STRENGTH

3.1 *Consolidation*

The very stiff nature of clayey material units in the transported sequence suggested that they were possibly overconsolidated. The results of several odometer tests for the upper horizon of the sediment profile were not conclusive but indicated normally consolidated material. Due to limitations of the applicable load in the odometer used (max 1600 kPa), the more important clay units in the lower part of the sediment profile (exceeding 80 m depth) could not be tested.

The pre-consolidation stress for different clay materials was derived using a large database of triaxial (CU) test results. As shown in Fig. 3 the results indicated OCRs of 0.34 for the material from the upper horizon and 2.0 to 2.2 for the lower horizons. Therefore the materials were classified as lightly overconsolidated.

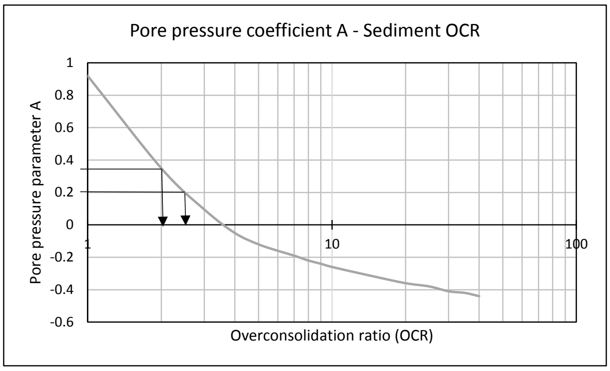

Fig. 2 Sediment OCR determined from CU- traxial test data

3.2 *Choice of shear strength parameters*

Back analysis of slope failures in overconsolidated clays has led to suggestions that the overall shear strength of stiff fissured clay at first failure corresponds to the fully softened shear strength of the material which can be obtained from a shear strength test on normally consolidated, remolded material (Skempton 1970 and Moon 1984). Moon 1984 suggests that the effective shear strength parameters for first time slides in overconsolidated, fissured clays were the fully softened friction angle.

A key concern in the selection of shear strength parameters for the lightly overconsolidated sediments in the north wall of the proposed pit was that these parameters could change over time when subjected to mining induced stress changes.

If degradation of the material is expected, as may be the case for long term slopes or where significant movement is expected, then the design might be based on the fully softened shear strength. Consideration of the residual shear strength of materials is generally afforded to structures that are required to have long service lives (Smith et al, 2006).

Typically, open pit mines have relatively short life spans and mining takes place rapidly with continued expansion via sequential cutbacks. Therefore it was considered that undrained peak shear strengths will be applicable for slope design.

3.3 *A case history*

A relatively small pit with a similar geology is sighted as an example. It is located approximately 50 km north west of the project area at the edge of the sedimentary basin. The thickness of the transported sequence is relatively lower, approximately 50 to 80m in the pit area. It consisted of partially cemented gravels, calcrete and calcareous clay and a silty clay unit at the bottom, overlying a weathered shale. The pit wall was designed using undrained peak shear strength parameters, obtained from laboratory multistage triaxial (CU) tests.

The wall experienced a moderate scale failure; about 30 to 50 thousand tons involving the basal sediment units, namely the silty clay, calcareous clay and hard calcrete and extended party into the overlying partially cemented gravelly zone.

The main reason for the failure development was concluded as considerable reduction in the effective shear strength in the basal sediment units due to pore pressure rise (20 m rise of piezometric surface) as a result of the malfunction of a dewatering bore located behind the failed section of the wall.

It was apparent that the peak shear strength parameters used in the design were valid under the perceived groundwater drawdown conditions.

3.4 *Design shear strength parameters*

A comprehensive laboratory testing program was conducted to characterize different sediment types and obtain their shear strength parameters.

3.5 *Multi-stage triaxial testing*

Multi stage, consolidated-undrained triaxial tests with pore pressure measurements (MST/CU) were conducted on large number of representative samples to obtain design shear strength parameters applicable to each material domain. Fig. 3 presents CU-triaxial results for one such material domain.

The geotechnical design recognised that the approach used for the project may be slightly conservative but appropriate for the design due to the limited experience of the deformational behavior of the sediments.

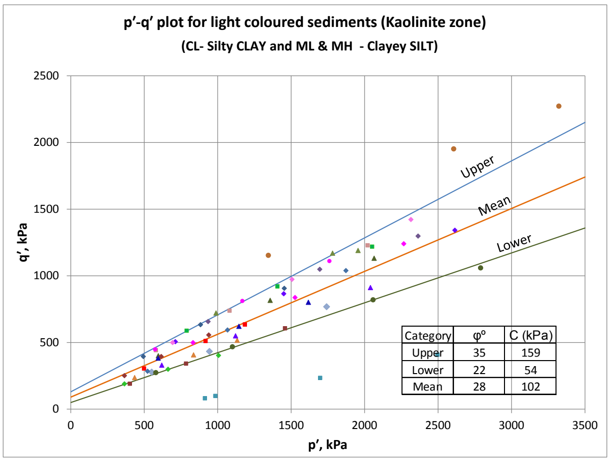

Fig. 3 CU triaxial test results for Kaolinite material domain

3.6 *Assignment of design shear strengths*

It was expected that the response to slope excavation would be influenced by the dominant weakest material types (within domains) in a particular area. Because of this reason the shear strength parameters estimated from triaxial test results were grouped based on classified material domains as well as material types. In the absence of clearly defined sediment domain boundaries for a particular stability section, strength parameters for the sediment material types have been assigned. Both the geotechnical logging information and particle size distribution data were used to determine the sedi ment type in this process.

4 HYDROGELOGY

4.1 *Groundwater regime*

It was inferred that a cavernous dolomite hosted regional aquifer with a large volume of water controls the groundwater regime in the project area.

The regional groundwater table lies at a depth of about 70 m within the sediment profile. The presence of very low permeable clay units, the underlying highly to extremely weathered shales and the deposit wide estimated weathering profile were the factors impact on achievable groundwater depressurisation within the final pit slopes.

4.2 *Groundwater modelling*

Pore pressure modelling was conducted for the starter pit, based on very limited pumping test data. The gravelly units were generally permeable material where effective depressurization can be achieved. However with permeability in the range of 10-7 m/s or lower, significant pore pressure expect to remain during mining in the silty clay units including the residual soil zone.

Modelling showed that substantial drawdown occurs in the bed rock and gravelly units. No drawdown was indicated in the clayey domains.

It is expected that depressurization aspects need be properly ascertained during the Stage 1 development and applied to the next stage design.

5 STABILITY ASSESSMENT

Limit equilibrium analyses were conducted for the pit wall stability assessment. The sediments were represented by Mohr-Coulomb shear strength parameters while rock mass and bedding shear strength were modelled using shear normal strength functions. The path-search with surface optimisation technique was used in the analyses.

5.1 *Design factor of safety*

The target FoS values adopted for the slopes design were:

- Slopes consisting of a single access ramp were designed for a target FoS of 1.3
- Slopes that do not carry infrastructure or have more than one ramp access were designed for a target FoS of 1.2.
- All slope designs were required to satisfy a FoS ≥ 1.0, using reduced shear strength parameters.

5.2 *Reduced shear strength for design validation*

The client's pit design criteria required validation of slope designs that were made using best-estimate (i.e. design) shear strength parameters to meet a minimum factor of safety of 1.0, using reduced shear strength parameters, which were the lower quartile values determined after removal of outliers.

5.3 *Results of the stability assessment*

Critical surfaces showed that potential for deep seated failure exist in the north wall, either by circular failure or by failure path following the bedding in the rock mass at the base. In the latter case typically the critical surface progresses through the sediment profile into the weaker shale rock mass and follows the bedding or contact with BIF. An example is shown in Fig. 4. In this stability section, the base of the critical surface follows the shale/BIF contact for a short distance. This is due to the anisotropic shear strength of the rock mass where the shear strength along the bedding/contact is relatively very low. The failure surface can extend into the BIF rock unit as well (underlying the shale), depending upon the orientation of the bedding/weaker shale bands. However, the risk of such a structure closely align with a critical failure path is low.

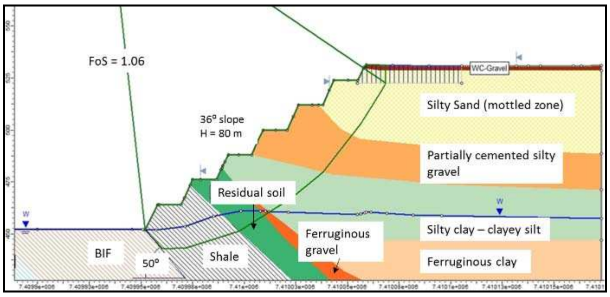

Fig. 4 Stability section from North West area

5.4 *Slope design*

Based on the results of the stability assessment and the slope design criteria adopted, slope design angles were recommended for different sectors of the north wall. The recommended maximum inter ramp angles ranged from 28^0 to 43^0.

6 CONCLUSIONS

The sediment profile is sensitive to the location and shear strength of the weaker clay domains and pore pressure distribution.

Based on this investigation the pit development was recommended to be carried out in two stages; an initial starter pit with a maximum slope height of 120 m and final cutback to a depth of 180 m. This strategy will provide the opportunity to evaluate depressurisation and pore pressure responses in the low permeability sediment units and slope performance where prior experience is very limited.

The optimisation of the next stage designs will benefit from the current pit development experience and develop strategies for risk management.

7 REFERENCES

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