Rock slope stability assessment at iron ore handling plant site in Western Australia

Srijib Chakrabarti
Neill Beattie
Jaime Tabucanon
Coffey Geotechnics Pty Ltd, Perth, Australia

Keywords: sliding, wedge, toppling, discontinuity, shear

ABSTRACT
Australia is one of the largest iron ore supplier countries in the world. As a result of recent, increased demand for iron ore on the international market, construction of new iron ore production projects and the expansion of existing handling facilities are on the rise in Western Australia, the largest state of Australia. This paper presents rock slope stability assessments undertaken for the Hope Downs Project primary crusher site in the Pilbara region of Western Australia. Geotechnical investigations included the drilling of inclined bore holes through various rocks types, such as banded iron formation, silicified mudstone etc. Rock slope stability assessments were carried out for exposed rock faces using kinematic analysis methods for planar sliding, wedge and toppling failures. The risk of such failures was assessed using DIPS software and probabilistic methods. Conceptual slope designs, developed from detailed slope stability analysis using Slope/W software, and the results of kinematic analysis, are discussed.

1 BACKGROUND

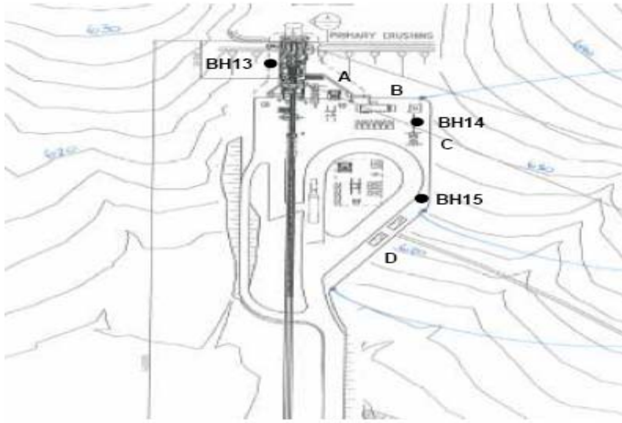
Australia is the world's largest iron ore exporter and as a producer, ranks third after China and Brazil (DoIR 2003). In 2005, Australia produced over 258 million tonnes for the domestic and export markets. Although iron ore resources occur in all the Australian States and Territories, almost 90% of identified resources (totalling 31.5 billion tonnes) occur in Western Australia, including about 80% in the Hamersley province, one of the world's major iron ore provinces. Most of the world's important iron ore resources occur in iron-rich sedimentary rocks known as banded iron formations (BIFs) that are almost exclusively Precambrian age (i.e., greater than 600 million years old). The ores from major mines in Western Australia's Hamersley Province of Pilbara region are hauled from mining faces to crushing and screening plants using trucks that can carry over 200 tonnes. The ore is then transported for further treatment and blending to port sites. Rio Tinto is currently developing Hope Downs Iron Ore Project in the Hamersley region. The associated infrastructure development for the project includes the construction of crushing, screening, stockpiling and reclaiming facilities at the mine site. This paper presents the results of geotechnical investigation and rock slope stability analysis undertaken for the primary crusher plant site of the project.

2 GEOTECHNICAL INVESTIGATION

The original site for the primary crusher plant was partially located within an area, which was later identified as an aboriginal heritage site. Two inclined boreholes (BH1 and BH2) up to 16m depth were drilled in the site outside the heritage area. Because of this constraint, the final location of the primary crushing plant was moved to about 100m south of the original plant site.
The layout of the final crusher plant site is illustrated in Figure 1. Also shown in Figure 1 is a photograph of the site taken during the drilling of three inclined boreholes BH13, BH14 and BH15 up to 25m depth. The site layout suggests that the depth of excavation within the crusher plant area would be up to 13m, and located between BH14 and BH15.

2.1 Site Condition

The ground surface at the primary crusher site is characterised by a thin mantle of fine to coarse banded ironstone (BIF) gravels and cobbles in a silty sand matrix. BIF outcrops occur locally across the site. Based on borehole data, the ground conditions within the excavation area comprise BIF, described as slightly jointed, low to very high strength, generally distinctly weathered to slightly weathered, with ironstone and siltstone interbeds.



Legend

BH13 ● Approximate Locations of Cored Boreholes
A Proposed Cut Faces (A to D)

Figure 1 Primary crusher plant location

2.2 Field and Laboratory Tests

The field testing for the primary crusher site involved drilling of one vertical borehole to a depth of 15m (BH13) and two boreholes inclined at 60^0 from the horizontal to depths of 25m (BH14) and 20m (BH15), using Hydrapower Site Investigation Drilling Rig and HQ coring techniques. A total of 54m (including core losses) of HQ3 cores (63mm diameter) was recovered for laboratory examination and testing. At the original primary crusher site, two boreholes (BH1 and BH2) inclined at 60^0 from the horizontal to depths of 16m were also undertaken. Due to the close proximity of BH1 and BH2 to the final crusher location, the data gathered from these boreholes have been considered in the analysis of rock slopes for the final crusher excavation site.

The laboratory testing of rock samples comprised 21 point load strength index, 14 uniaxial compressive strength (UCS) with strain measurements, and two direct shear tests on rock joints.

The point load strength index ($Is_{(50)}$) of the BIF core samples varied between 0.5MPa and 10.6MPa, and the unconfined uniaxial compressive strengths (UCS) varied between 1.8MPa and 103.2MPa. The interpreted Young's modulus of the BIF material varied between 260MPa and 5920MPa. The resulting correlation between point load index $Is_{(50)}$ and UCS for the BIF is summarised as follows:

$$UCS = 10.53 \times Is_{(50)} \text{ (MPa)} \qquad \text{(Coefficient of Correlation } R^2 = 0.56)$$

The results of two rock joint direct shear tests on samples from BH14 indicate peak friction angles of 40° and 52°, and residual friction angles of 30° and 43°, respectively. Direct shear test on a rock joint recovered from BH1 was previously undertaken and the result indicated a peak friction angle of 45° and a residual friction angle of 30°.

It should be noted that the joint surface tested represents a very small part of the actual joint surface. The actual joint is likely to be undulating and rough. As such, it is likely that the true shear strength of joints would be higher than the value measured in laboratory. The mobilised shear on any potential sliding plane may reach the peak strength and may progressively reduce to the residual strength over time because of environmental factors such as rain water infiltration, ground vibrations induced by operating crushing equipment and blasting at the nearby open pit mine, etc. Hence, for the purposes of analysing the stability of rock slopes at the primary crusher site, the residual strength of 30° plus 5° (to allow for potential undulation and surface roughness), was considered appropriate (rock joint friction angle of 35°).

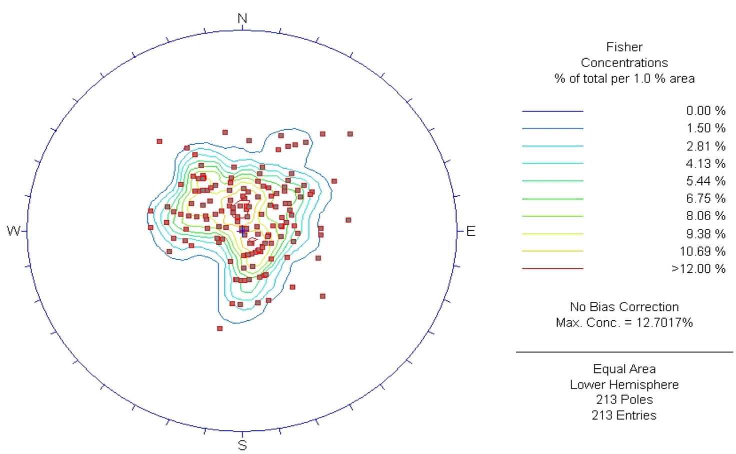


Figure 2 Contour of orientations of discontinuities

3 ANALYSIS

3.1 Stereographic Plot

A stereographic pole plot of all rock joints and partings, measured in Boreholes BH14 and BH15 drilled at the crusher site together with those from Boreholes BH1 and BH2 drilled at the original crusher site, is illustrated in Figure 2 using DIPS software version 5.1. Three slope batters of 1V:1.2H, 1V:1.5H and 1V:2H were considered in the analysis of rock slopes.
The primary objective of the slope assessment was to derive the maximum batter slope angle where mechanical slope support would not be required for permanent stability. Figure 1 shows the four slope locations analysed, which are referred to in this paper as Slopes A, B , C and D.

3.2 Results of Kinematic Analysis Using DIPS

A review of the limited, measured discontinuity data from Boreholes BH1, BH2, BH14 and BH15 indicated that the potential modes of failure for excavated slopes in the BIF comprise plane failure and wedge failure. The majority of the discontinuities measured from the boreholes, which accounted for 80% of the data are bedding partings, with dips ranging from 0^0 to 40^0 and an average bedding orientation of $30^0/208^0$ (dip/dip direction). The average bedding orientation was determined empirically by the DIPS software, based on all bedding orientations measured from

Boreholes BH1, BH2, BH14 and BH15. Due to the limited number of measured joint orientations, no defined joint sets could be identified. The orientations of proposed excavated slopes A to D are presented in Table 1.

Table 1 Orientation of rock slopes

Excavation Face	Slope Direction	Slope Batters Analysed		
		1 V : 2.0 H	1 V : 1.5 H	1 V : 1.2 H
A	187^0	26^0	33^0	40^0
B	143^0	26^0	33^0	40^0
C	233^0	26^0	33^0	40^0
D	270^0	26^0	33^0	40^0

Kinematic analyses for plane and wedge failures were undertaken for Slopes A to D for slope batters 1V:1.2H, 1V:1.5H and 1V:2.0H using DIPS software. The results of these analyses are presented below. It should be noted that for long term stability, engineering design requires a Factor of Safety (FOS) of not less than 1.5.

3.2.1 1V: 1.2H Excavated Slope

Based on the results of kinematic analyses, rock excavation with a slope batter of 1V:1.2H for slopes A to D will have a low to moderate likelihood of plane failure and a low likelihood of wedge failure occurring. Typical stereographic representations of these analyses for slope A are presented in Figure 3. The estimated FOS values for the measured discontinuities at Slopes A to D are presented in Table 2.

Table 2 Results of Kinematic Analyses – 1V:1.2H Slopes

Slope Designation (1V:1.2H)	Measured Discontinuity Orientations		Range of Assessed FOS	Probabilistic Kinematic Likelihood of Instability
	Range in Dip (degrees)	Range in Dip Direction (degrees)		
A	26 - 37	139 - 237	0.9 - 1.4	Moderate (<15%)
B	26 - 40	96 -195	0.83 -1.44	Moderate (<15%)
C	27 - 41	195 - 271	0.8 - 1.37	Moderate (<15%)
D	26 - 40	218 – 290	0.83 – 1.44	Low (<5%)

It should be noted that the results of kinematic analyses are based on measured discontinuities from boreholes BH1, BH2, BH14 and BH15, which only constitute a small part of the area investigated. It is possible that discontinuity orientations outside the measured ranges indicated in Table 2 may exist. The range of dip and dip direction of rock discontinuities which, when present in the rock mass, may constitute a risk for instability (plane failure) with a FOS<1.5, were also identified.

It should be noted that the FOS in Table 2 apply for long term conditions (i.e. using the anticipated residual rock joint friction angle of 35°). For temporary slopes, where the peak joint strength is appropriate (angle of friction = 45^0), there is a low likelihood of instability for the proposed rock slopes as the angle of peak joint friction is greater than the grade of the slope. Toppling failures and wedge failures are unlikely to occur in the 1V:1.2H cut slope for the measured discontinuity orientations indicated in Table 2. Probabilistic assessment indicated in Table 2 suggests that there is a moderate probability of planar sliding failure to occur within Slopes A, B and C (<15%). For Slope D, the probability of such failure is low (<5%). Water pressure was not considered in the stability analysis because of the low water table in the area and provision of adequate surface drains for these slopes.

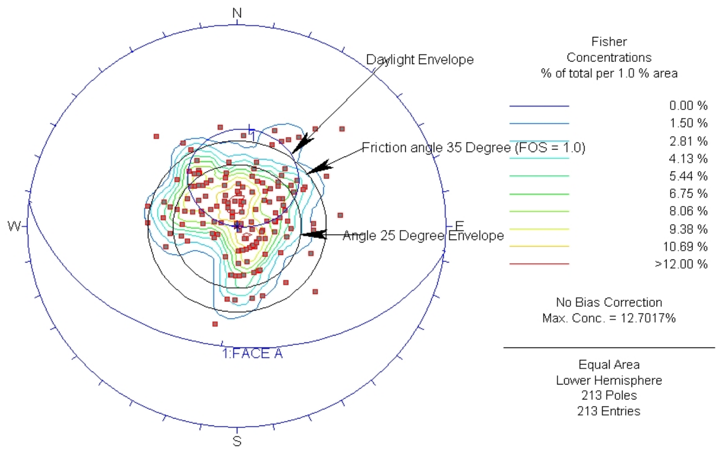


Figure 3 Cut face A (1V:1.2H): plane failure analysis

3.2.2 1V: 1.5H Excavated Slope

Table 3 below presents the results of kinematic analysis for a 1V:1.5H rock excavation in Slopes A to D. The results suggest that the likelihood of plane failure occurring in these slopes (1V:1.5H) is low to moderate. The likelihood of wedge failure to occur in these slopes is considered to be low. The estimated FOS values for the measured discontinuities for slopes A to D are presented in Table 3.

Table 3 Results of kinematic analyses – 1V:1.5H slopes

Slope Designation (1V:1.5H)	Measured Discontinuity Orientations		Range of Assessed FOS	Probabilistic Kinematic Likelihood of Instability
	Range in Dip (degrees)	Range in Dip Direction (degrees)		
A	26 - 30	146 - 218	1.21 - 1.44	Low (<5%)
B	26-31	129 - 174	1.17 – 1.44	Low (<5%)
C	26 - 31	194 - 239	1.17 – 1.47	Moderate (<15%)
D	26 - 30	231 -290	1.21 – 1.44	Low (<5%)

Toppling failures were assessed to be unlikely to occur in the 1V:1.5H cut slope for the measured discontinuity orientations indicated in Table 3. The results of the probabilistic assessment presented in Table 3 suggested that there is a low probability of planar sliding failure occurring within Slope A, B and D (<5%). There is moderate probability for planar sliding failure to occur within Slope C (<15%).

3.2.3 1V: 2.0H Excavated Slope

Typical stereographic representations of the kinematic analysis for Slopes A at a slope batter of 1V:2H is presented in Figure 4. The results of the kinematic analysis for Slopes A to D, based on measured discontinuities from Boreholes BH1, BH2, BH14 and BH15, gave FOS greater than 1.5. The kinematic analyses further suggest that the likelihood of plane failure, toppling failure and wedge failure to occur in Slopes A to D is negligible.

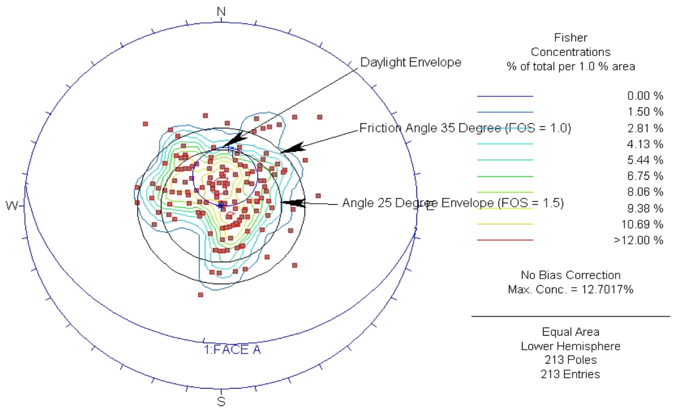


Figure 4 Cut face A (1V:2.0H): plane failure analysis

Global stability along discontinuity planes were also undertaken using SLOPEW software. The 1V:2H cut slope were calculated to have FOS greater than 1.5 and 1.2 for normal and seismic (horizontal acceleration = 0.1g) conditions respectively, which were considered adequate.

4 DISCUSSION AND CONCLUDING REMARKS

The rock cut slope kinematic analyses undertaken for the primary crusher site suggested that a slope batter 1V:2H for Slopes A to D would not require the installation of slope support measures to stabilise the slope for the life of the project (estimated at 20 years) for permanent stability, whereas installation of pattern rock bolts and steel reinforced shotcrete would be required for a slope batters 1V:1.2H and 1V:1.5H. A number of recommendations were presented including:

- The excavated faces should be inspected by an experienced engineering geologist to check for occurrence of unfavourable joints which might contribute to instability or might impact on the integrity of the slope.
- Rainfall runoff drain at the top of excavation should be provided to prevent rain water seeping into the cut slope.
- Collector drains at the excavation toe should also be provided in the design.
- Benches for slopes greater than 10m in height may not be required unless required for drainage purposes.

The rock excavation with a slope batter of 1V:2H has been adopted by the stakeholder.

REFERENCES

http:// www.industry.gov.au/content accessesed on January 2007

http://www.nationalminesatlas.gov.au/info/factsheets/iron.jsp accessesed on January 2007

Department of Industry and Resources (DoIR), WA (2003), *Western Australian Iron Ore Industry 2003*

DIPS Software Version 5.1 manual

GEO-SLOPE International 2004). Stability modelling with SLOPE/W