Middindi Consulting (Pty) Ltd
Rock Engineering Geotechnics Geology

Geotechnical open-pit slope design for Omitiomire - Namibia

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DOCUMENT REVIEW AND SIGN OFF

Name	Author/Reviewer	Signature
Duran Chetty	Author	
Nalika Harijhan	Principal Reviewer	

EXECUTIVE SUMMARY

Middindi Consulting (Pty) Ltd was contracted by Omico Mining Corporation to conduct the geotechnical characterisation, geotechnical analysis, and slope engineering design aspects for the Omitiomire Copper Project in Namibia. The Omitiomire project is located approximately 120km northeast of Windhoek, central Namibia.

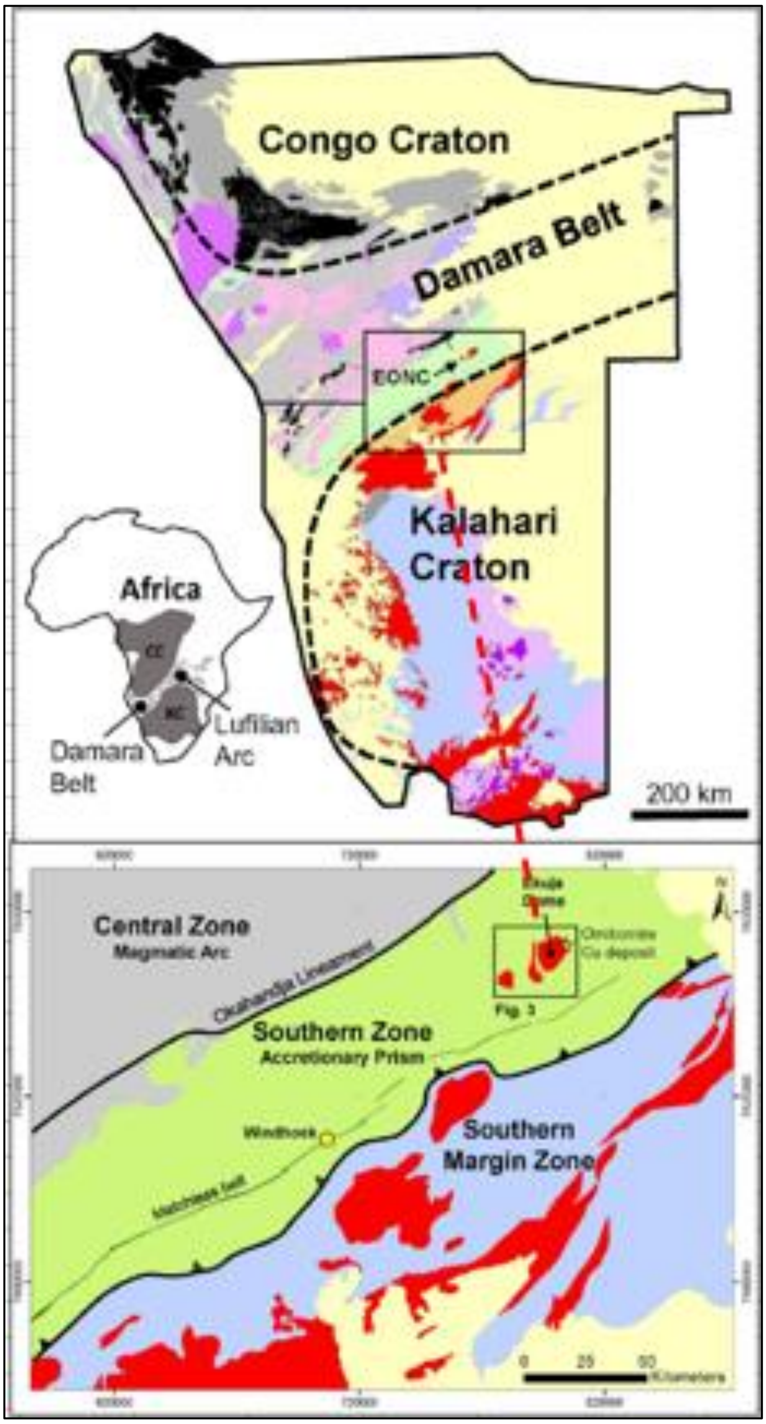

Figure 1: Location of Omitiomire, Namibia.

Limitations

The limitations for the design phase of the project were the lack of a 3-dimensional geological model or geology to confirm the validity and accuracy when performing RocScience Slide software modelling. The safety factors are contingent on the fresh and weathered materials comprising the slopes, therefore in the absence of a geological model – 2-dimensional cross-sections were used to inform the design of the slopes and lineated material boundaries for the lithological layers.

Limitation regarding the survey data. Survey data was received intermittently and therefore progression of the design and engineering was done with 5 of 8 survey data boreholes. The remaining 3 boreholes survey data was used as the proposed collar positions which may affect the accuracy of orientation data and subsequent kinematic assessments and analysis. The complete 3-D model was not completed at the time of geotechnical report compilation.

Geology

The area is hosted by the Ekuja Dome, one of three gneiss domes in the north-eastern Southern Zone accretionary prism. The Southern Zone (SZ) forms the accretionary wedge of the belt, underlain by sequences of meta turbiditic Kuiseb Formation, and amphibolite facies and is situated in Mesoproterozoic gneisses and amphibolite of the Ekuja dome. The rock types for the Omitiomire area are comprised of gneisses: white gneiss, pink gneiss, grey gneiss, mafic gneiss, banded gneiss with minor amounts of biotite schist and pegmatite.

Geotechnical Data

In the data acquisition component of the project, eight (8) orientated boreholes were drilled and geotechnically logged on site for the Omitiomire project with a total meterage of 1415m by Middindi on the 17th January to 8th February 2023. An additional, three (3) historical boreholes were combined and incorporated to form the basis of the geotechnical database. Geotechnical logging was performed with due consideration to orientation when the orientation line on the core was visible.

Rock Mass Classification – RQD, RMR$_{89}$ and GSI

The RQD recovery of the rocks varies with their degree of weathering. The package of fresh to slightly weathered gneisses exhibits good to very good recovery, while those moderately weathered show only fair recoveries. However, highly weathered and completely weathered gneisses exhibit poor to very poor recovery. The recovery data for biotite schist is consistent with that of gneisses, and the pegmatite weathering grade is fresh to slightly weathered, resulting in good recovery, which is similar to that of the gneisses.

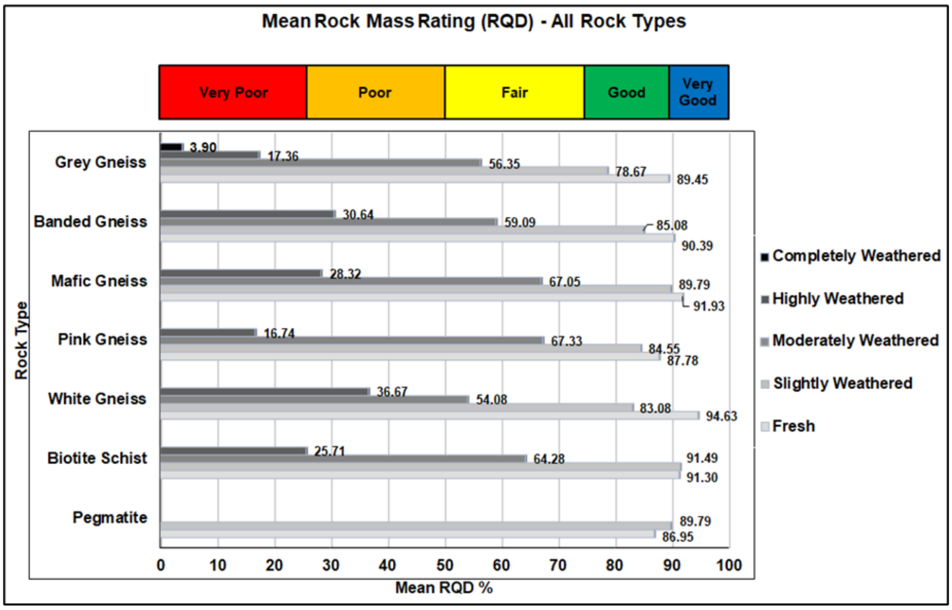

Figure 2: Average RQD values per rock type per weathering grade.

The values indicate that the fresh gneisses all range within the good rock category, slightly weathered gneisses are good while biotite schist falls within the fair rock. Moderately weathered gneisses classify as fair except for pink gneiss, which is poor rock. For highly weathered, the gneisses are fair for grey, banded, and white gneiss while mafic and pink gneiss are poor rock. Completely weathered are all poor rock for all the gneisses and biotite schist. Pegmatite classes as a good rock for fresh and slightly weathered, respectively.

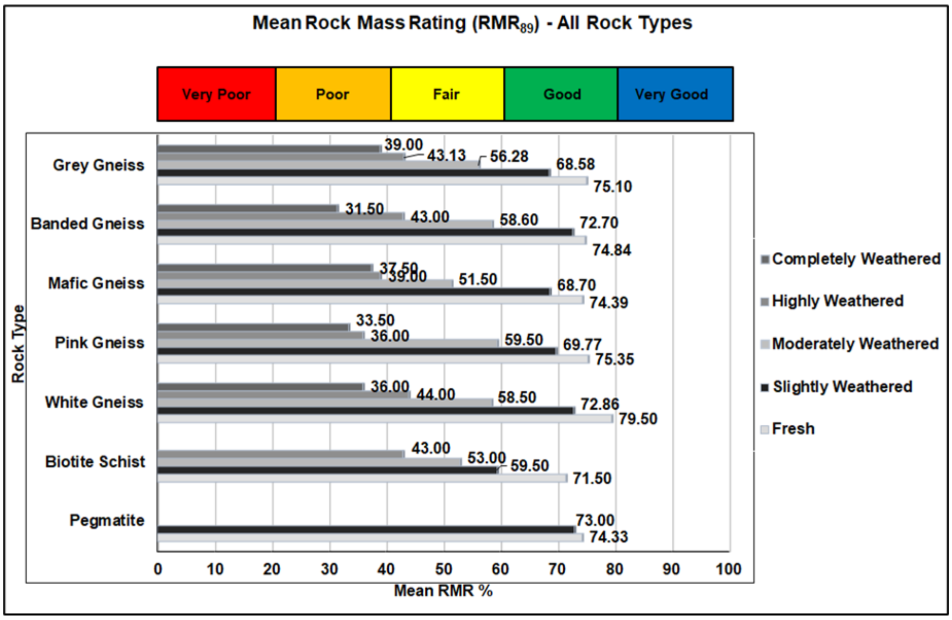

Figure 3: Average RMR_{89} values per rock type per weathering grade.

The GSI values were plotted according to the degree of geotechnical weathering in the figure below and also the GSI values were plotted according to weathering domain of fresh and weathered material.

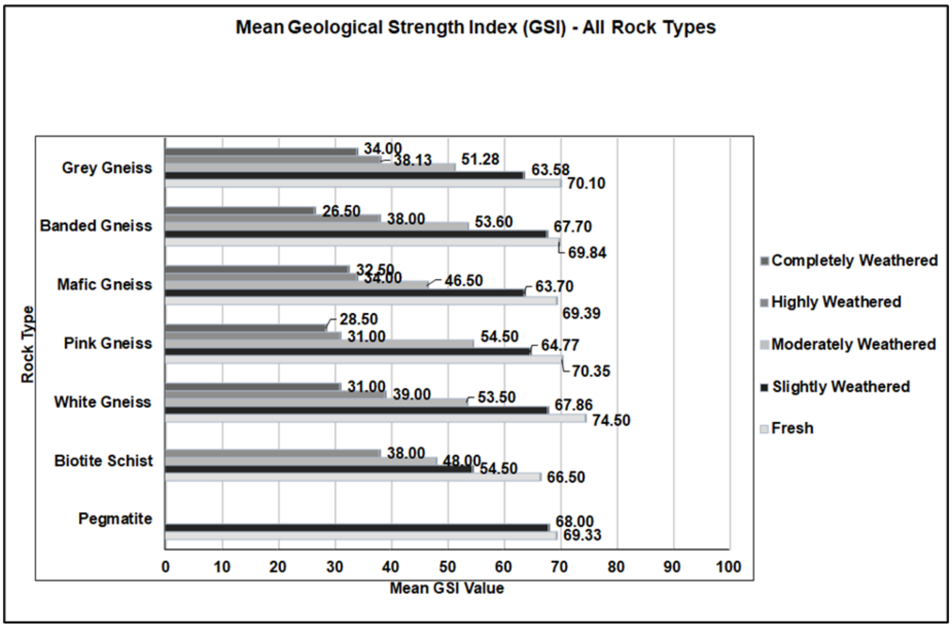

Figure 4: Average GSI values per rock type per weathering grade.

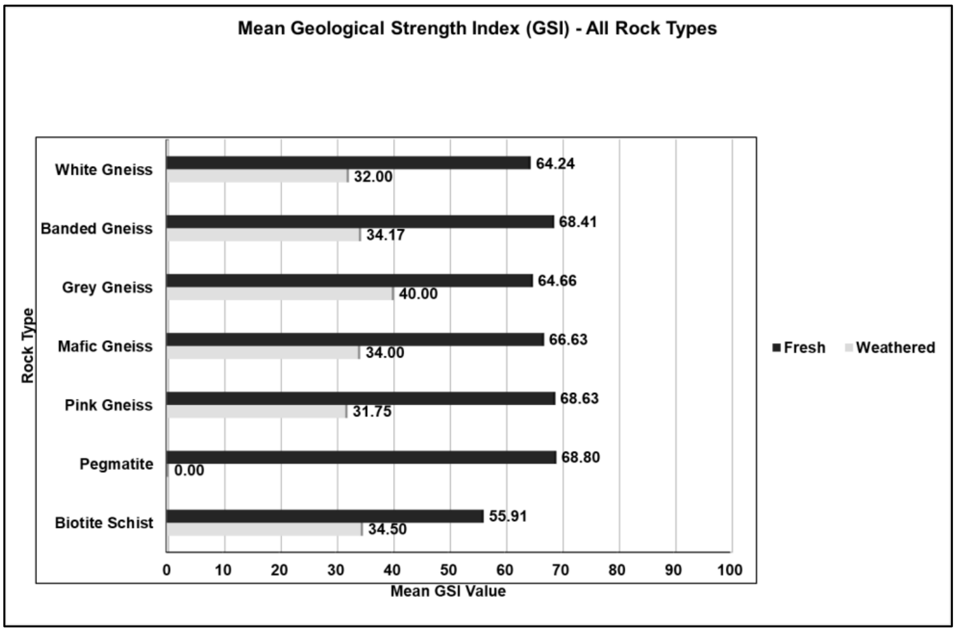

Figure 5: Average GSI values per weathering domain.

Rock Test Results

Core samples were selected from the eight (8) boreholes for rock testing. The selection was done per The International Society of Rock Mechanics (ISRM, 1983) guidelines and protocols for sample selection and "Middindi standard operating procedure for geotechnical data acquisition". Five (5) types of tests were conducted namely:

- Uniaxial Compressive Strength (UCS)
- Triaxial Compressive Strength (TCS)
- Indirect Tensile Strength/ Brazilian Tensile Strength (UTB)
- Base Friction Angle (BFA)

- Shear strength of natural joints (STJO)

A summary of the rock samples selected for the testing program is provided in Table 1. The tables below from Table 2 to Table 7, for the UCS, TCS, UTB, BFA, STJO and density respectively indicating the number of rock samples for each type of rock test conducted. The total number of samples utilised is one-hundred and thirty-eight (138).

Table 1: Rock Test Summary

Type of Test	No. of Tests		
	Middindi	SRK	Total
UCS	37	20	**57**
UTB	31	0	**31**
TCM	18	0	**18**
BFA	12	14	**26**
STJO	6	0	**6**
Total	**104**	**34**	**138**

Table 2: UCS test result summary

Rock Type	UCS Value (MPa)	
	Fresh Material	Weathered Material
White Gneiss	214.07	20.55
Banded Gneiss	91.89	44.73
Grey Gneiss	135.98	44.34
Mafic Gneiss	77.64	11.40
Pink Gneiss	237.82	76.50
Pegmatite	119.79	64.42
Biotite Schist	N/A	7.80

Table 3: TCS test result summary

Rock Type	TCS Value Summary	
	σ ci (MPa)	Mi Value
White Gneiss	195.67	35.13.
Banded Gneiss	157.14	15.06
Grey Gneiss	251.98	8.99
Mafic Gneiss	71.04	7.46
Pink Gneiss	279.09	42.11
Pegmatite	196.79	12.01
Biotite Schist	N/A	8.21

Table 4: UTB test result summary

Rock Type	UTB Value				
	Mean	Standard Deviation	25th Percentile	50th Percentile	75th Percentile
White Gneiss	12.54	1.09	11.49	13.08	13.11
Banded Gneiss	11.94	2.43	11.11	11.16	12.92
Grey Gneiss	12.26	0.92	11.47	12.20	12.84
Mafic Gneiss	7.46	0.86	6.93	7.27	7.80
Pink Gneiss	12.16	1.62	11.58	12.72	13.27
Pegmatite	12.01	2.47	10.99	13.40	13.78
Weathered Pegmatite	5.86	N/A	5.86	5.86	5.86
Weathered Biotite Schist	0.95	N/A	N/A	N/A	N/A

Table 5: BFA test result summary

Rock Type	Base Friction Angle (°)	
	Fresh	Weathered
White Gneiss	35.00	38.75
Banded Gneiss	34.90	N/A
Grey Gneiss	31.70	N/A
Mafic Gneiss	34.00	N/A
Pink Gneiss	30.88	N/A
Pegmatite	38.50	N/A
Biotite Schist	N/A	34.50

Table 6: STJO test result summary

Rock Type	STJO Value	
	Cohesion (kPa)	Friction Angle (°)
White Gneiss	55.00	31.00
Banded Gneiss	15.00	28.00
Grey Gneiss	190.00	32.50
Mafic Gneiss	55.00	34.00
Pink Gneiss	170.00	33.50
Pegmatite	175.00	27.50
Biotite Schist	No Data	No Data

Table 7: Density summary

Domain	Rock Type	Density (g/cm^3)	Density (kg/m^3)
Fresh	White Gneiss	2.62	2620.83
	Banded Gneiss	2.73	2733.09
	Grey Gneiss	2.67	2670.33
	Mafic Gneiss	2.91	2909.52
	Pink Gneiss	2.55	2552.98
	Pegmatite	2.63	2629.20
Weathered	White Gneiss	2.53	2534.44

	Banded Gneiss	2.66	2660.99
	Grey Gneiss	2.70	2703.79
	Mafic Gneiss	2.78	2782.00
	Pink Gneiss	2.67	2671.00
	Pegmatite	2.26	2260.45
	Biotite Schist	2.72	2716.74

Geotechnical Weathering Profile

Weathering data was obtained from geotechnical borehole photographs. The average thickness of weathered material from each borehole corresponds to the contour plots. The weathering depths to which point fresh rock begins in a borehole were recorded and were used to contour the spatial variation of the depths of weathering around the pit area of Omitiomire.

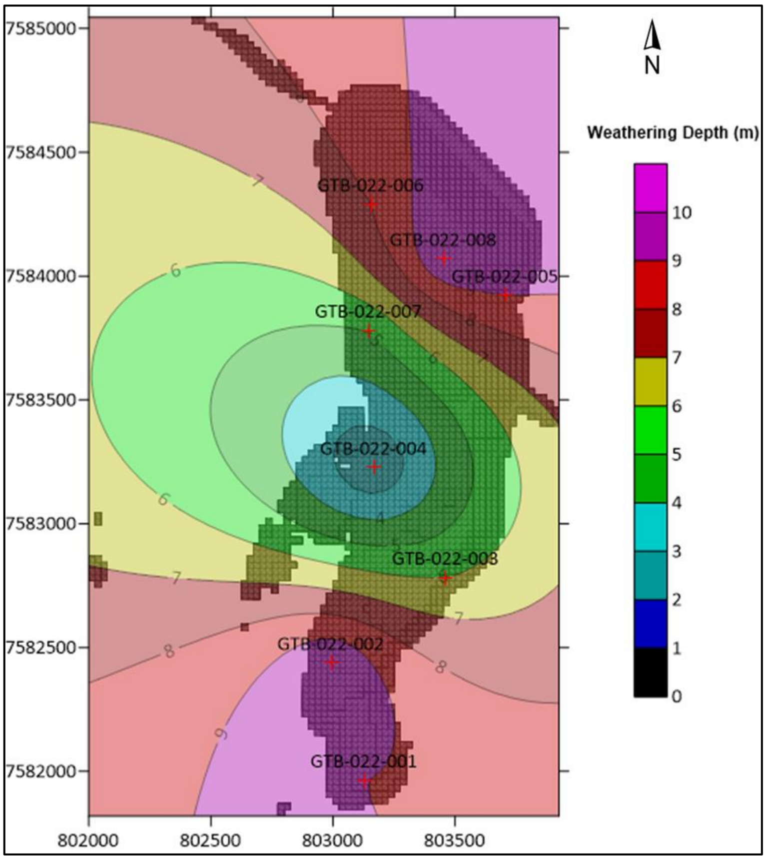

Figure 6: Omitiomire spatial distribution weathering depths (m).

Rock Mass Properties

The intact rock mass properties, field properties, Hoek-Brown constants and equivalent Mohr-Coulomb criteria for Omitiomire are presented in Table 8 to Table 12.

Table 8: Rock mass properties for fresh material. (Disturbance factor = 1)

Fresh Rock Mass Properties (D = 1) Poor Blasting/Production Blasting								
Rock properties	**Units**	**White Gneiss**	**Banded Gneiss**	**Grey Gneiss**	**Mafic Gneiss**	**Pink Gneiss**	**Pegmatite**	**Biotite Schist**
GSI	*N/A*	64.24	68.41	64.66	66.63	68.63	68.80	55.91
UCS	MPa	214.07	91.89	135.98	77.64	237.82	119.79	7.80
mi	*N/A*	35.13	15.06	8.99	7.46	20.59	12.01	8.21
D	*N/A*	1.00	1.00	1.00	1.00	1.00	1.00	1.00
mb	*N/A*	2.73	1.58	0.72	0.69	2.19	1.29	0.35
s	*N/A*	0.0026	0.0052	0.0028	0.0038	0.0054	0.0055	0.0006
a	*N/A*	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Density	kg/m^3	2.62	2.73	2.67	2.91	2.55	2.63	2.72
cohesion	kPa	1441.34	1114.30	1270.08	975.85	2189.67	1441.77	157.59
friction angle	(°)	61.51	51.57	47.68	42.87	59.47	51.31	22.61

Table 9: Rock mass properties for weathered material. (Disturbance factor = 1)

Weathered Rock Mass Properties (D = 1) Poor Blasting/Production Blasting								
Rock properties	**Units**	**White Gneiss**	**Banded Gneiss**	**Grey Gneiss**	**Mafic Gneiss**	**Pink Gneiss**	**Pegmatite**	**Biotite Schist**
GSI	N/A	32.00	34.17	40.00	34.00	31.75	N/a	34.50
UCS	MPa	20.55	44.73	44.34	11.40	76.50	N/a	7.80
mi	N/A	35.13	15.06	8.99	7.46	20.59	N/a	8.21
D	N/A	1.00	1.00	1.00	1.00	1.00	N/a	1.00
mb	N/A	0.27	0.14	0.12	0.07	0.16	N/a	0.08
s	N/A	0.00001	0.000012	0.00005	0.00002	0.00001	N/a	0.00002
a	N/A	0.52	0.52	0.51	0.52	0.52	N/a	0.52
Density	kg/m3	2.53	2.66	2.7	2.78	2.67	N/a	2.72
cohesion	kPa	171.30	195.56	201.85	82.02	254.02	N/a	71.59
friction angle	(°)	27.56	26.94	26.43	13.92	31.40	N/a	13.10

Table 10: Rock mass properties for fresh material. (Disturbance factor = 0.7)

Fresh Rock Mass Properties (D = 0.7) Good Blasting/Mechanical Excavation								
Rock properties	**Units**	**White Gneiss**	**Banded Gneiss**	**Grey Gneiss**	**Mafic Gneiss**	**Pink Gneiss**	**Pegmatite**	**Biotite Schist**
GSI	*N/A*	64.24	68.41	64.66	66.63	68.63	68.80	55.91
UCS	MPa	214.07	91.89	135.98	77.64	237.82	119.79	7.8
mi	*N/A*	35.13	15.06	8.99	7.46	20.59	12.01	8.21
D	*N/A*	0.70	0.70	0.70	0.70	0.70	0.70	0.70
mb	*N/A*	4.92	2.65	1.29	1.19	3.67	2.16	0.73
s	*N/A*	0.0056	0.0103	0.0060	0.0079	0.0106	0.0109	0.0017
a	*N/A*	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Density	kg/m^3	2.62	2.73	2.67	2.91	2.55	2.63	2.72
cohesion	kPa	1793.96	1381.77	1657.54	1251.80	2749.99	1815.42	206.07
friction angle	(°)	64.97	55.30	51.93	47.13	62.52	54.90	28.26

Table 11: Rock mass properties for weathered material. (Disturbance factor = 1)

Weathered Rock Mass Properties (D = 0.7) Good Blasting/Mechanical Excavation								
Rock properties	**Units**	**White Gneiss**	**Banded Gneiss**	**Grey Gneiss**	**Mafic Gneiss**	**Pink Gneiss**	**Pegmatite**	**Biotite Schist**
GSI	N/A	32.00	34.17	40.00	34.00	31.75	N/a	34.50
UCS	MPa	20.55	44.73	44.34	11.40	76.50	N/a	7.80
mi	N/A	35.13	15.06	8.99	7.46	20.59	N/a	8.21
D	N/A	0.70	0.70	0.70	0.70	0.70	N/a	0.70
mb	N/A	0.84	0.40	0.33	0.20	0.48	N/a	0.22
s	N/A	0.00005	0.00007	0.00017	0.00007	0.00005	N/a	0.00008
a	N/A	0.52	0.52	0.51	0.52	0.52	N/a	0.52
Density	kg/m3	2.53	2.66	2.66	2.66	2.66	N/a	2.72
cohesion	kPa	250.75	284.78	287.42	127.14	369.43	N/a	111.02
friction angle	(°)	37.34	36.29	34.77	20.93	41.38	N/a	19.78

Orientation Data

Eight (8) orientated boreholes and 42 historical boreholes were used to compose an orientation data base with a total of two-thousand-nine-hundred and fifty-nine 2959 orientation measurements. The combined data plot of all major joints for the eight (8) boreholes is attached in Figure 7.

The plot indicates that the joint set are a shallow dipping pervasive joint set, that has been identified as the foliation geological feature.

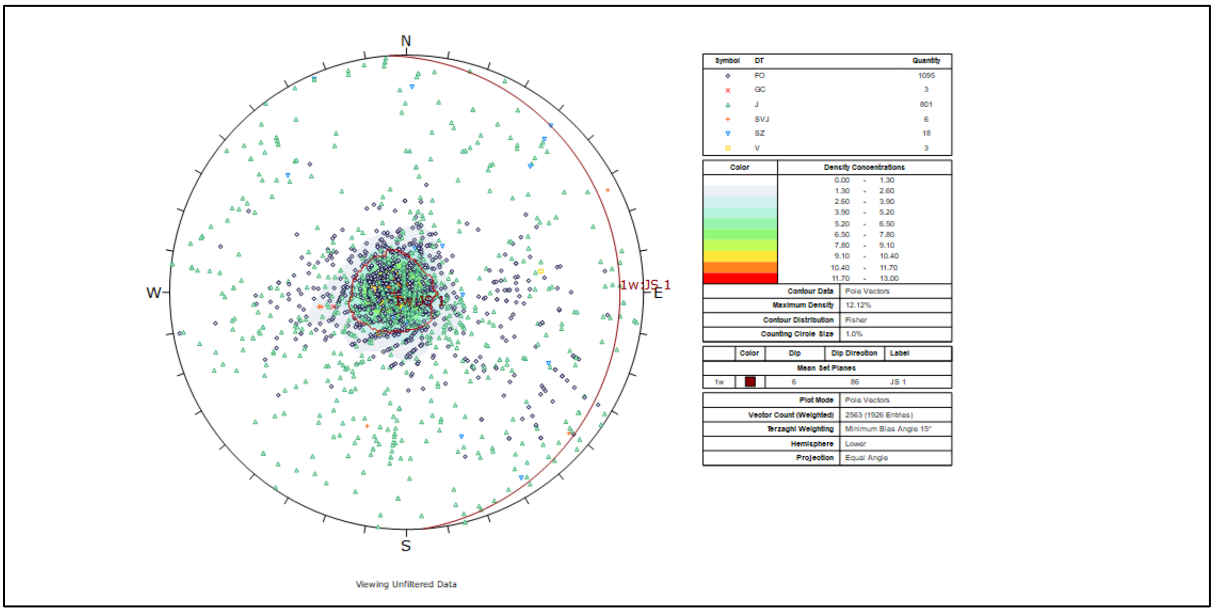

Figure 7: All major joint sets data.

Slope Configuration

The Omitiomire pit was divided into design sectors, based on pit wall directions, rudimentary fault structures and water level depths. The design sectors and their respective wall directions are listed in Table 12. The planned pit shell indicating the design sectors is illustrated in Figure 8.

Table 12: Design sectors with corresponding wall and dip direction.

Pit	Design Sector	Wall Direction (°)	Dip Direction (°)
Omitiomire	DS1	46	226
	DS2	86	266
	DS3	95	275
	DS4	120	300
	DS5	232	52
	DS6	300	120
	DS7	260	80

The design depths of the slopes with the associated design sectors are attached below in Table 13.

Table 13: Design slope depths.

Domain	Design Sector	Elevation	Depth (m)
HW	**DS1**	1685	360
HW	**DS2**	1680	360
HW	**DS3**	1680	165
HW	**DS4**	1680	195
FW	**DS5**	1680	125
FW	**DS6**	1685	105
FW	**DS7**	1685	360

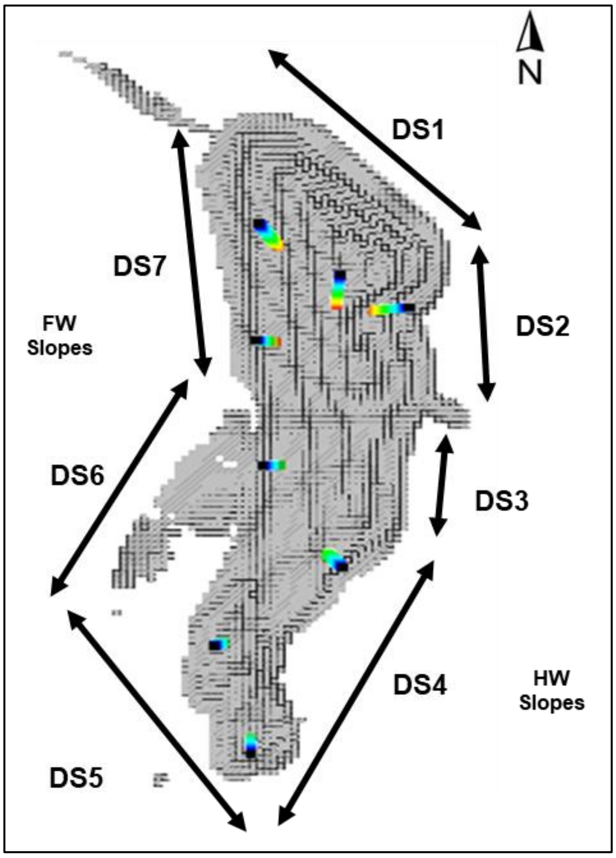

Figure 8: Design sectors for Omitiomire.

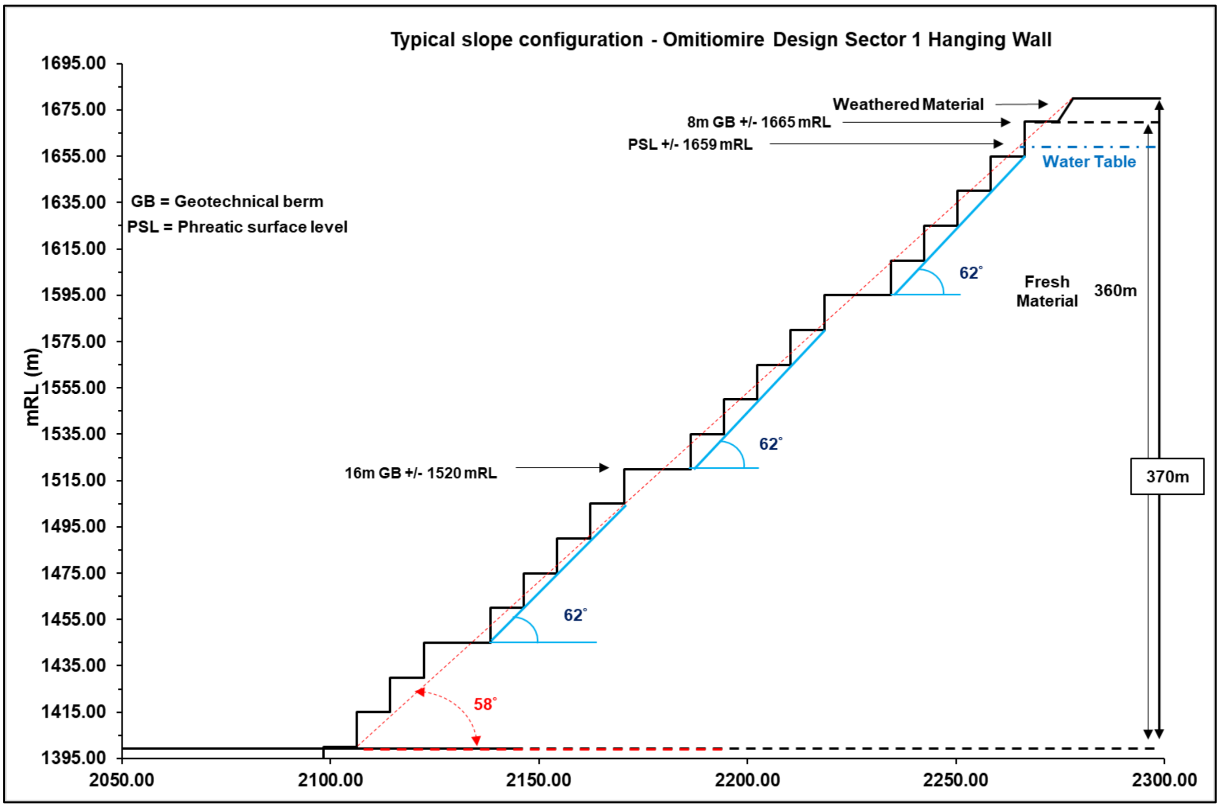

Figure 9: Slope configuration design sector 1.

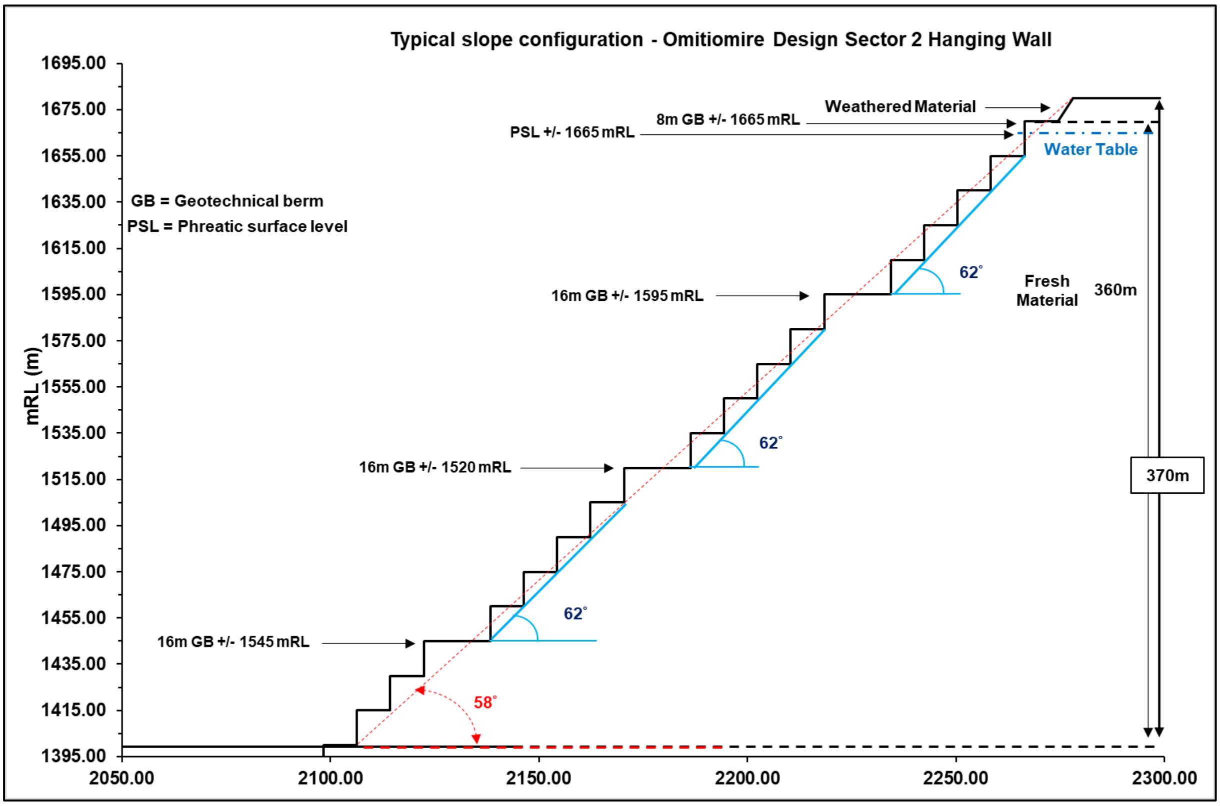

Figure 10: Slope configuration design sector 2.

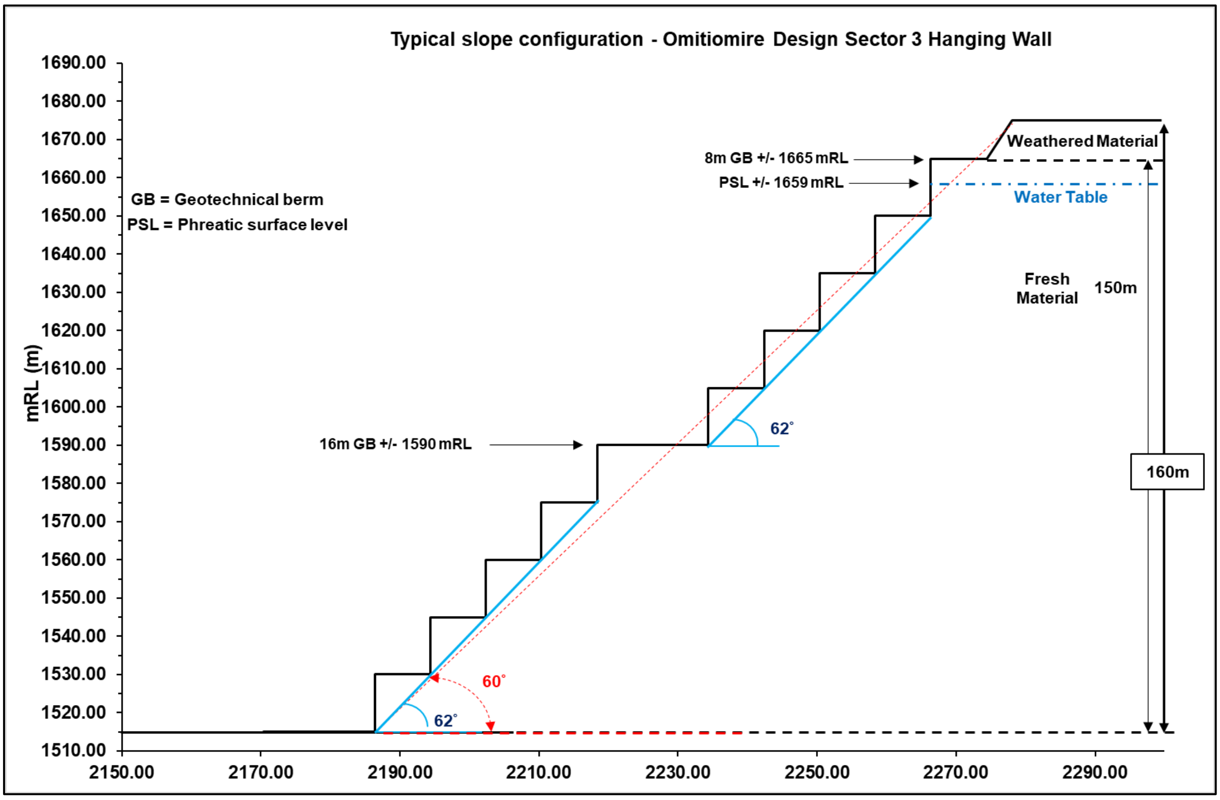

Figure 11: Slope configuration design sector 3.

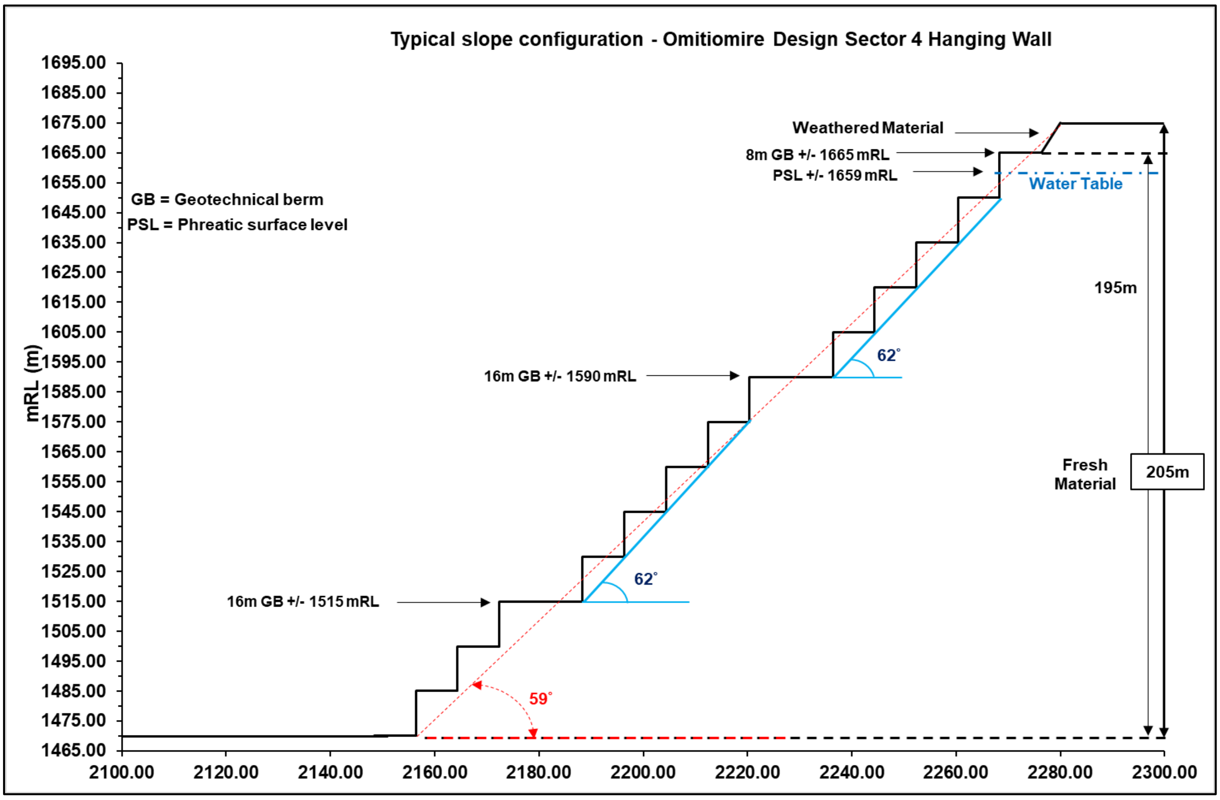

Figure 12: Slope configuration design sector 4.

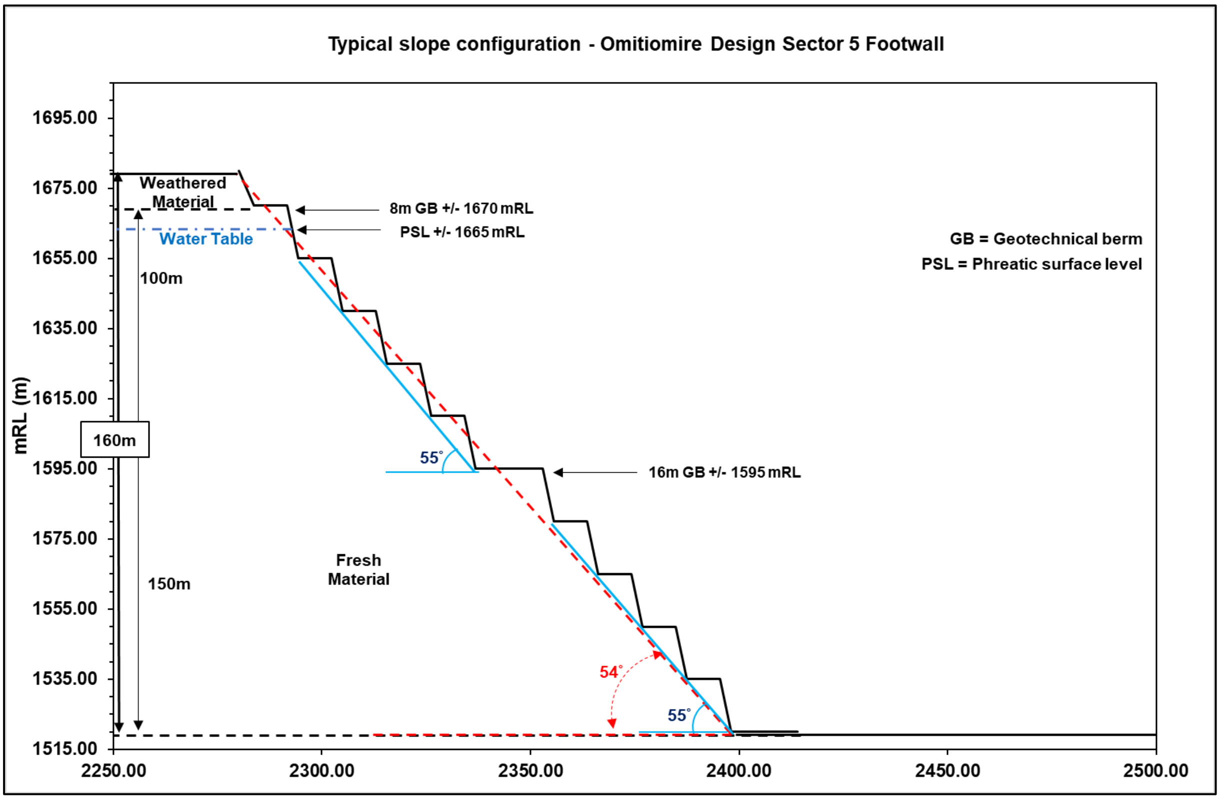

Figure 13: Slope configuration design sector 5.

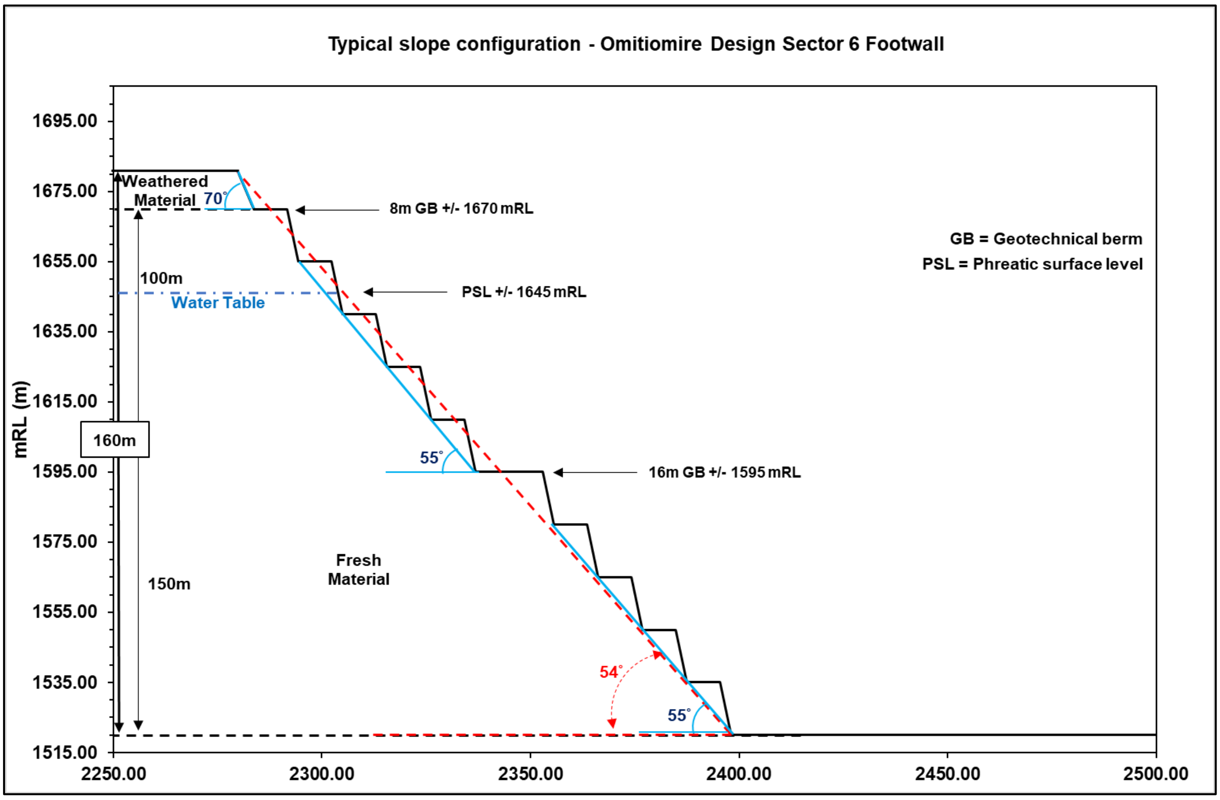

Figure 14: Slope configuration design sector 6.

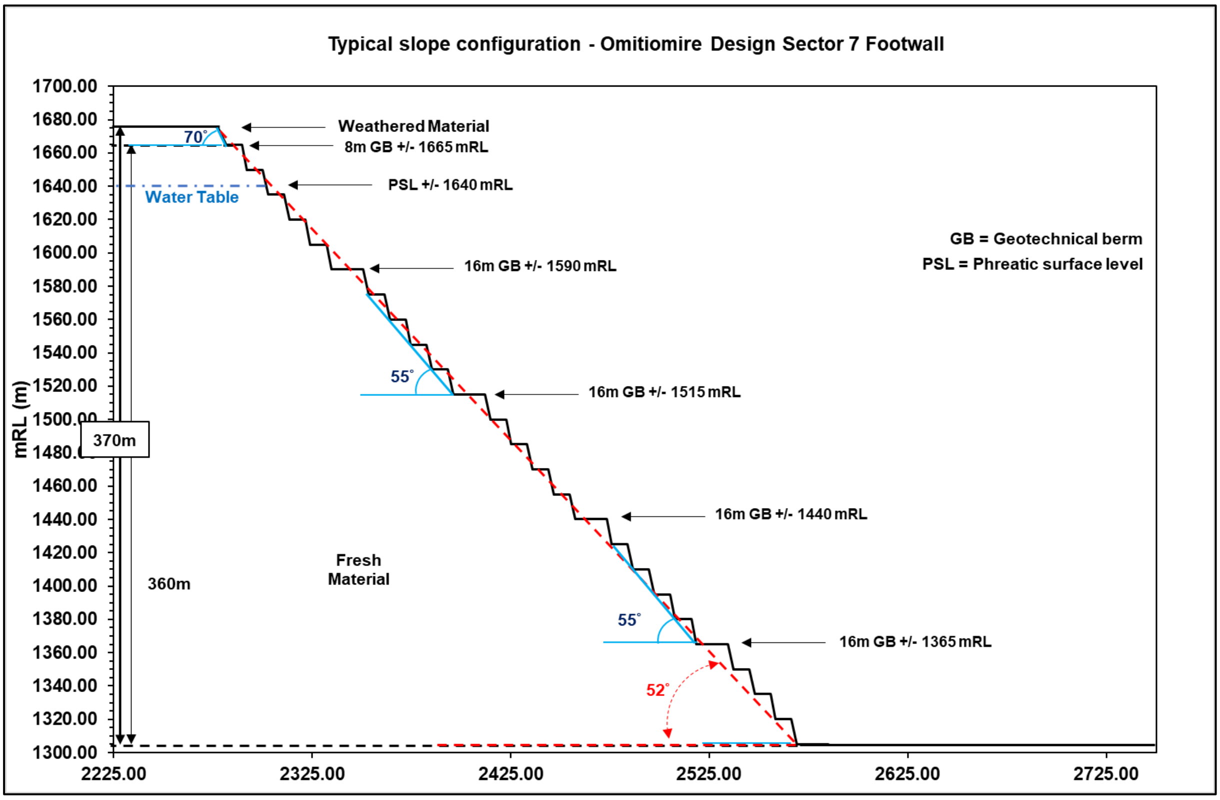

Figure 15: Slope configuration design sector 7

Conclusion

The geotechnical data made available and transformed into analysis input parameters allowed for a technically robust design to be produced at a feasibility level of accuracy. The following points summarise the geotechnical content of this submission:

- Eight (8) primary boreholes were used for the pit design, additionally with the supplementation of three (3) historical boreholes that were combined and used for validation of geotechnical parameters derived to form the basis of the geotechnical database.
- A total of 1415 metres of core was drilled and geotechnically logged for the Omitiomire project.
- RQD, RMR_{89} and GSI values were derived from geotechnical logging to form the database
- Eight (8) geotechnically logged boreholes were utilised for dip angles and dip directions and were used to derive the major discontinuity trends for the Omitiomire project area. Additionally, a total of forty-two (42) historical boreholes orientation data was used to supplement stereographic plots. A total of two-thousand-nine-hundred-and-fifty-nine (2959) orientation measurements were available.
- One-hundred and thirty-eight (138) samples were selected for various rock tests, of which one-hundred and four (104) were selected on-site for laboratory rock strength testing and the remaining twenty-four (24) were obtained from historical data.
- A detailed kinematic study was carried out and was based on orientation data and discontinuity properties derived from rock tests analysis.
- The intact rock properties derived were used either directly or indirectly to derive the following:
  - Hoek-Brown strength parameters.
  - Equivalent Mohr-Coulomb parameters.
  - Rock quality indicators.
  - Defect properties of cohesion and friction angle.
- Design sectors were devised based on pit wall directions, rudimentary fault structures and water level depth for the Omitiomire pit.

Further work: When the operation begins, geotechnical data must be continuously collected and compared with the datasets used in this design.

The geological sections used in this submission must be cross checked with the 3-D geological model to ensure all geology was correct. The geological model only became available after compilation and submission of the geotechnical slope design section.

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LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

Middindi Consulting (Pty) Ltd was contracted by Omico Mining Corporation to conduct the geotechnical characterisation, geotechnical analysis, and slope engineering design aspects for the Omitiomire Copper Project in Namibia.

2. SCOPE OF WORK

The project was divided into three (3) phases which are summarised below:

- Phase 1 consisted of a data acquisition programme which required geotechnical logging of geotechnical-orientated boreholes.
- Phase 2 entailed the transformation and characterization of data into rock mass parameters and included a rock testing programme.
- Phase 3 consisted of the geotechnical engineering design aspects for the open pit.

3. RELIANCE ON OTHER EXPERTS

The information used to populate the geotechnical section of the report, relied on certain infotamton from other experts, this includes:

- Historical geotechnical study in the form of a Pre-Feasibility Study (PFS) : R, Armstrong, 2010. Omitiomire Pre-Feasibilty Open Pit Geotechnical Slope Design. SRK Consulting, Unpublished Report.
- Knight Piesold Consutling provided information on the hyrology, in the form of hyrological boreholes, from which the water table levels were derived.
- The MSA Group provided information on the geology.

4. LIMITATIONS AND ASSUMPTIONS

The limitations encountered during the design phase of the project were:

- The lack of a 3-dimensional geological model or geology to confirm the validity and accuracy when performing RocScience Slide software modelling. The safety factors are contingent on the fresh and weathered materials comprising the slopes, therefore in the absence of a geological model – 2-dimensional cross sections were used to inform the design of the slopes and lineated material boundaries for the lithological layers. The complete 3-D model was not completed at the time of geotechnical report compilation.

- Survey data was received intermittently and therefore progression of the design and engineering was done with 5 of 8 survey data boreholes. The remaining 3 boreholes survey data was used as the proposed collar positions which may influence the accuracy of orientation data and subsequent kinematic assessments and analysis.

5. GEOLOGICAL SETTING

5.1 Regional Geology

The following section details the geology of the Omitiomire project area:

The Omitiomire project is located approximately 120km northeast of Windhoek, central Namibia. The area is hosted by the Ekuja Dome, one of three gneiss domes in the north-eastern Southern Zone accretionary prism.

The east-northeast trending Pan-African Damara Belt of central Namibia is attributed to the convergence and closure of the Khomas Sea ocean basin between the Congo and Kalahari cratons displayed in Figure 5-1. The Southern Zone (SZ) forms the accretionary wedge of the belt, underlain by sequences of meta-turbiditic Kuiseb Formation, and is situated in Mesoproterozoic gneisses and amphibolites of the Ekuja dome referenced by Figure 5-2.

5.2 Local Geology

The rock types for the Omitiomire area are composed of gneisses: white gneiss, pink gneiss, grey gneiss, mafic gneiss, banded gneiss with minor amounts of biotite schist and pegmatite.

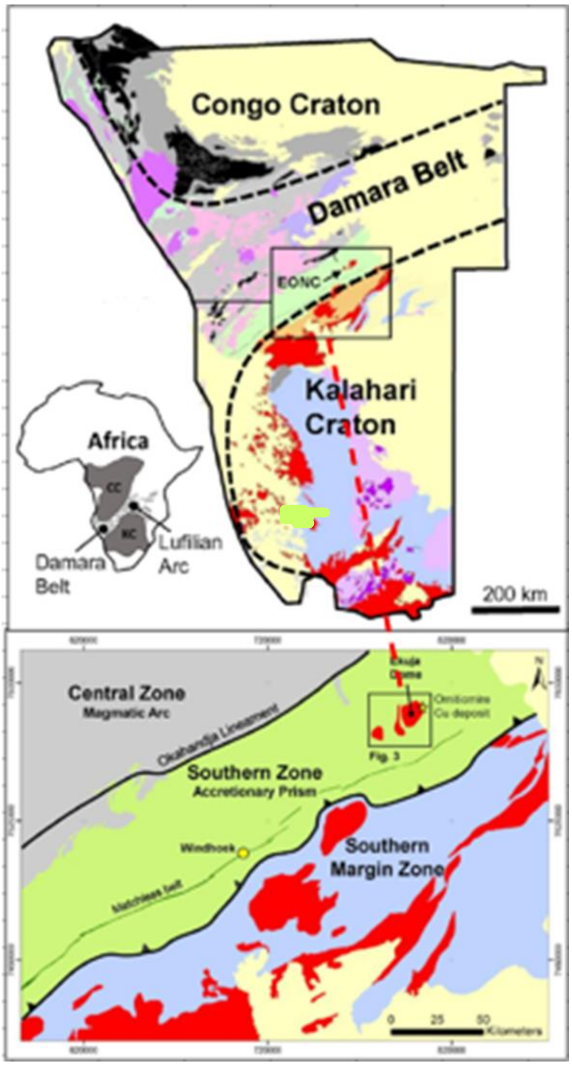

Figure 5-1: Surrounding geological environment of the Omitiomire project, (Miller, 2008).

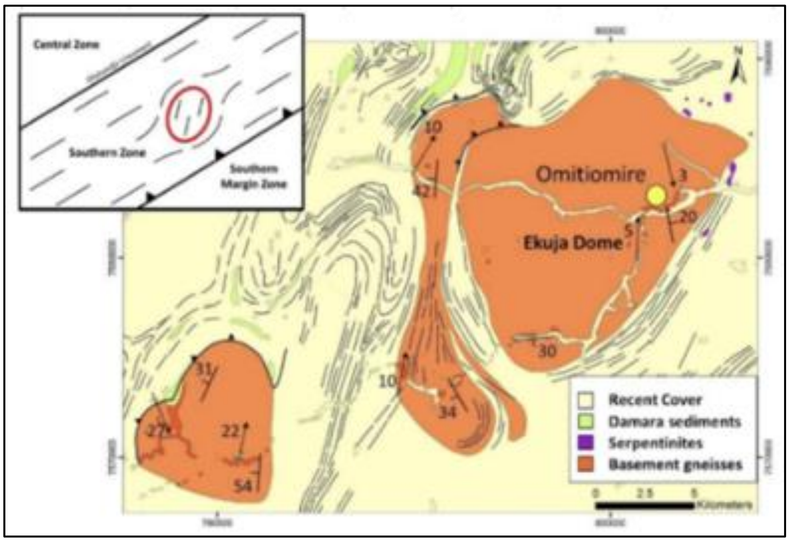

Figure 5-2: Omitiomire situated within the Ekuja Dome, (Miller, 2008).

5.3 Structural Geology

The area has undergone a complex deformation history, dominated by southeast and east-southeast directed thrusting. The mineralization dips at approximately 20° to the east (Armstrong, 2010). The rocks in the project area have very shallow to gentle dips. A basic structural model for the project area is shown in Figure 5-3 and shows the major faults structures in the project area.

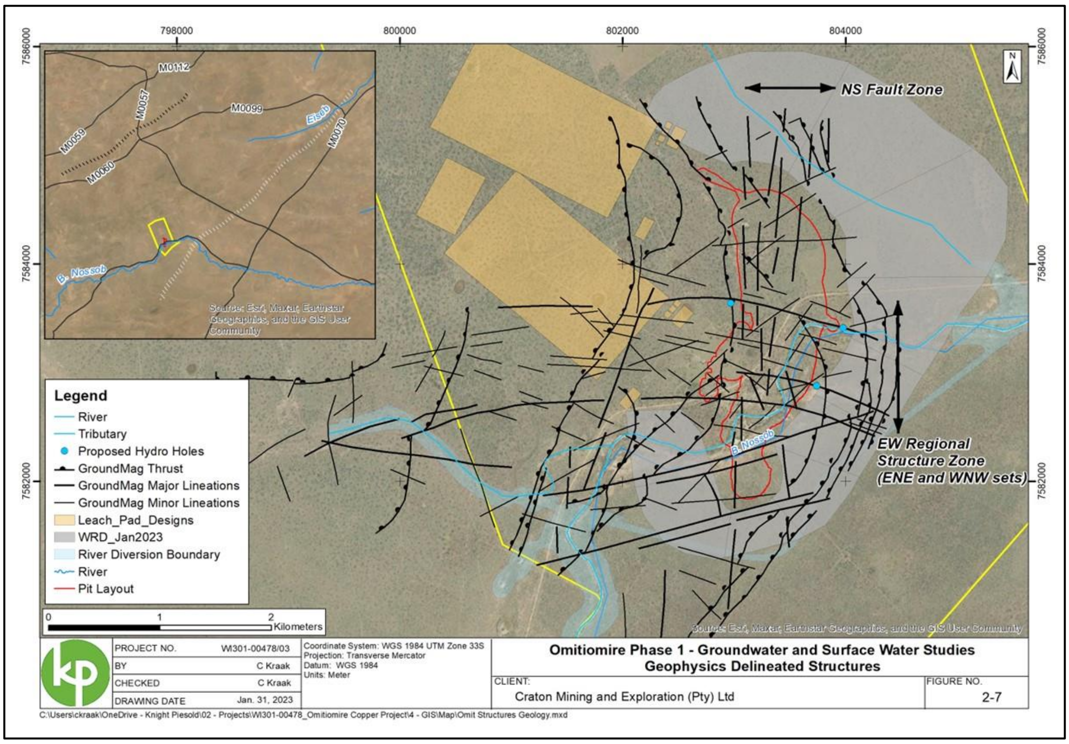

Figure 5-3: Geophysics delineated structures.

6. SEISMICITY

The seismicity of Namibia was assessed using the Earthquake hazard map of Africa Figure 6-1, which implied that the Omitiomire project area is located in a zone where peak ground acceleration ranges between 0.2 m/s^2 to 0.4 m/s^2. This range represents a low seismic hazard, suggesting that the project area lies within a region with low seismic activity. Therefore, the open pit design provided herein excludes the influence of strong ground motion.

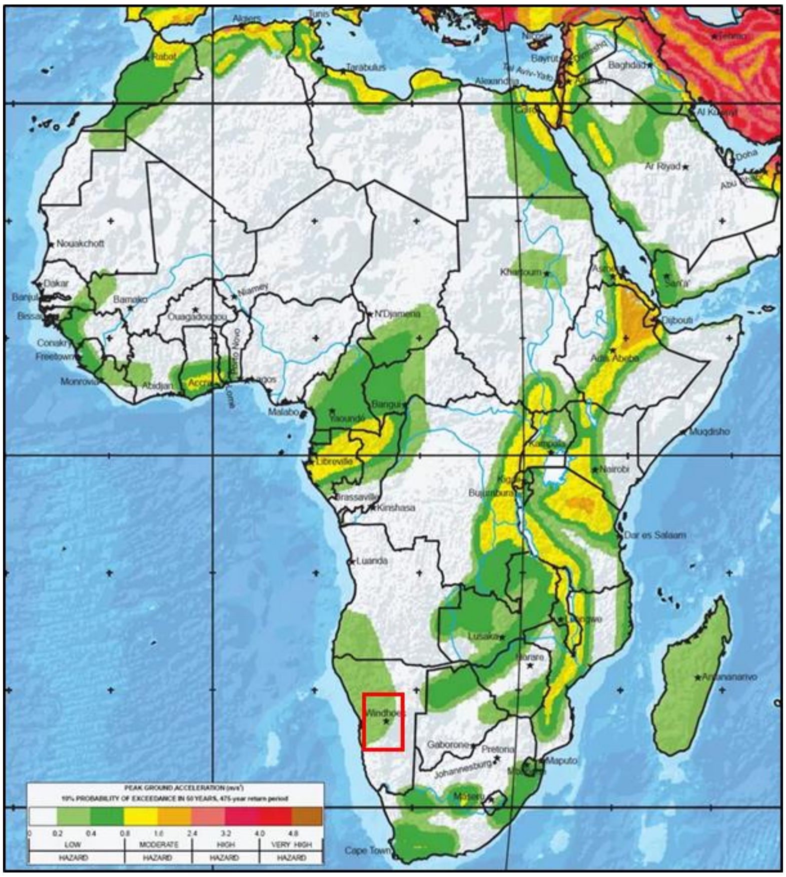

Figure 6-1: Seismic hazard map, Namibia. (Alden, 2019)

7. HYDROLOGY

The piezometric surface and depth of the water table are illustrated below in Figure 7-1. A summary for the phreatic surface was averaged based on the data provided by Knight Piésold (Pty) Ltd and subsequently assigned per designated design sector used for the limit equilibrium modelling. The averaged water level depth data is presented in Table 7-1. The raw data received is contained within the **Error! Reference source not found.**.

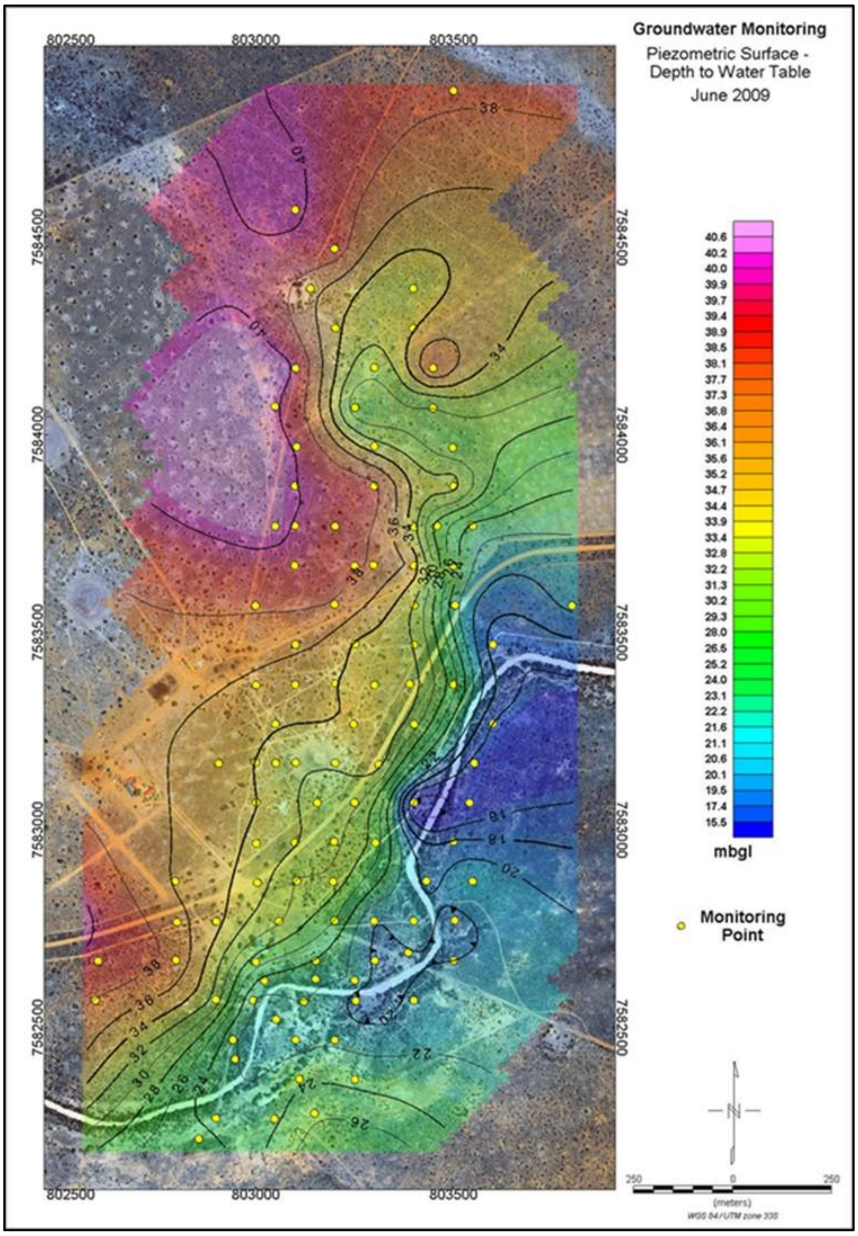

Figure 7-1: Piezometric surface, depth of the water table for Omitiomire.

Table 7-1: Water level depth per design sector.

Water level (m) in Design Sector	
DS1	22.04
DS2	13.04
DS3	15.52
DS4	15.52
DS5	15.52
DS6	33.28
DS7	33.28

8. GEOTECHNICAL DATA

As part of the data acquisition component of the project, eight (8) orientated boreholes were drilled and geotechnically logged for the Omitiomire project with a total meterage of 1415m by Middindi. An additional, three (3) historical boreholes that were logged by SRK Consulting, were combined, and incorporated with the eight (8) geotechnical boreholes to form the basis of the geotechnical database. Table 8-1 provides an overview of the drill hole population displaying the borehole ID, position, dip angle (inclination), dip direction (azimuth) and maximum depth. Figure 8-1 illustrates the borehole collar location layout, with eight (8) geotechnical holes, and one historical hole. The two (2) remaining historical holes, circled in red, were included on a diagram from the SRK report (Armstrong, 2010), as no collar positions were available.

Geotechnical logging was quantified according to Middindi standard operating procedure for geotechnical data acquisition which is based on ISRM standards and protocols (ISRM, 1983.). The geotechnical logging data was subject to Middindi's Quality Assurance and Quality Control procedures.

The complete record of geotechnical logging sheets and borehole corephotos are captured in **Error! Reference source not found.** and **Error! Reference source not found.**, respectively.

Table 8-1: Proposed collar positions of the geotechnical boreholes.

Geotechnical Boreholes						
Borehole ID	Borehole Collar Position			Inclination (°)	Azimuth (°)	Estimated Depth (m)
	Easting (UTM)	Northing (UTM)	Elevation (m)			
GTB-22-001	803129	7581964	1687.34	65	0	110
GTB-22-002	802993	7582438	1683.52	70	90	110
GTB-22-003	803462	7582782	1679.53	60	315	140
GTB-22-004	803168	7583227	1686.00	60	90	140
GTB-22-005	803706	7583922	1686.79	60	270	225
GTB-22-006	803156	7584290	1689.00	60	135	225
GTB-22-007	803147	7583780	1688.05	70	90	225
GTB-22-008	803453	7584069	1687.65	60	180	240
Estimated Length of Core to be Logged (m)						**1415**

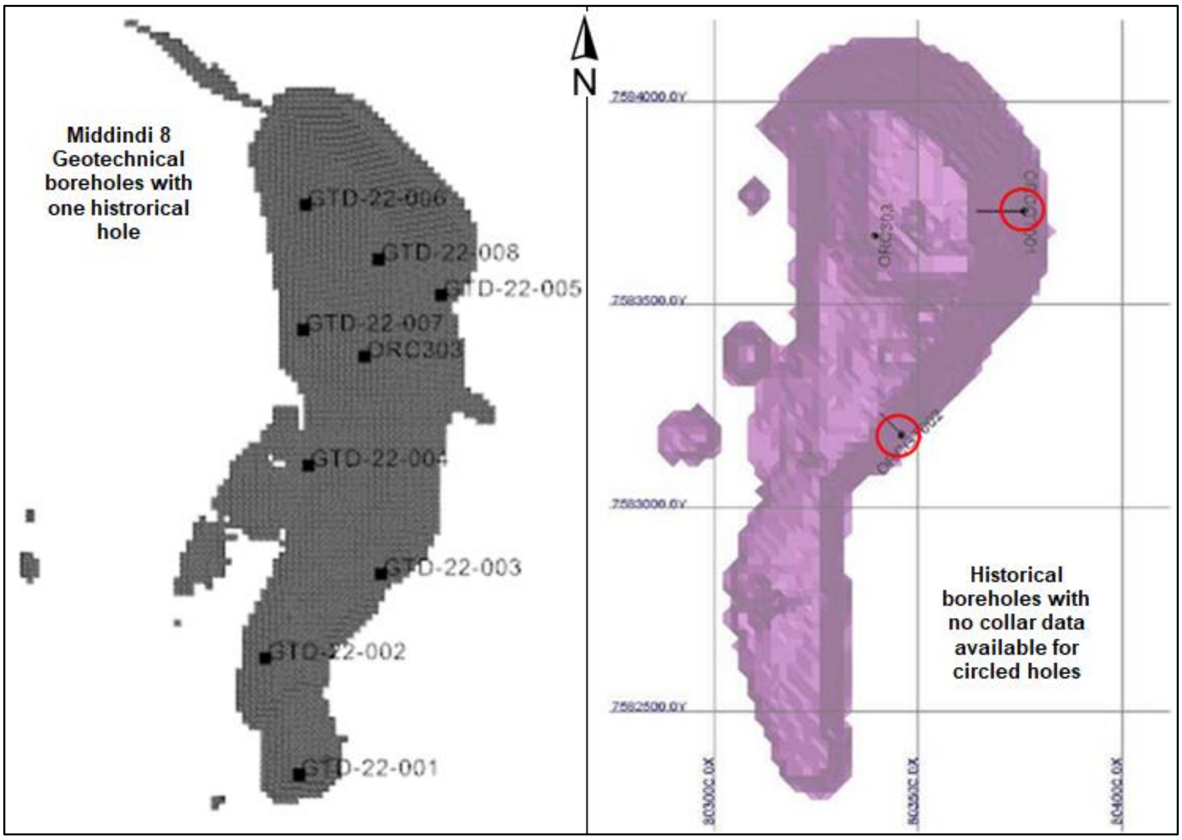

Figure 8-1: Borehole layout positions for Omitiomire.

9. ROCK MASS CLASSIFICATION

The geotechnical data captured was used to characterise the rock mass according to Deere's Rock Quality Designation (RQD%), Bieniawski's Rock Mass Rating (RMR_{89}), and Hoek and Marinos' Geological Strength Index (GSI).

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9.1 Rock Quality Designation

The Rock Quality Designation (RQD) is defined as the sum of the lengths of intact core pieces longer than 10cm (100mm) expressed as a percentage of the total drill core run length. The procedure to determine RQD is illustrated in Figure 9-1 and the RQD system range for rock quality categories is in Table 9-1.

$$Rock\ Quality\ Designation\ (RQD) = \frac{\sum Length\ of\ core\ pieces\ greater\ than\ 100mm\ (10cm)}{Total\ length\ of\ core\ run} \times 100$$

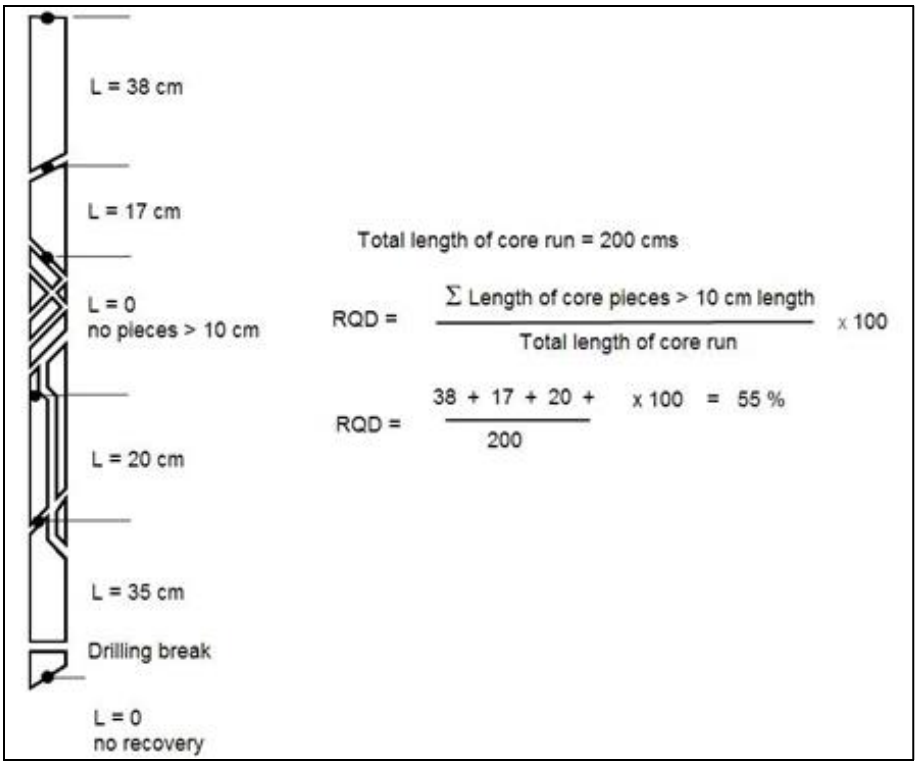


Figure 9-1: Procedure for the calculation of RQD, (Deere, 1989).

Table 9-1: RQD Classification, (Deere, 1989).

Rock Quality Designation (RQD)	Description of Rock Quality
0 – 25%	**Very poor**
25 – 50%	**Poor**
50 – 75%	**Fair**
75 – 90%	**Good**
90 – 100%	**Excellent**

The following parameters were recorded during geotechnical logging:

- Depth below the surface (From, to)
- Rock type
- Weathering of the rock mass
- The hardness of the rock mass (Field estimate)
- Total Core Recovery (TCR)
- RQD (Rock Quality Designation)
- Number of open fracture frequency per run
- Number of discontinuities per run
- Total number of cemented closed joints per run
- Depth of discontinuities intersected
- Discontinuity condition (Roughness, infill type, infill thickness and joint alteration)

- Alpha (α) and Beta (β) angles for discontinuities
- Comments about observations during logging

Table 9-2 and Figure 9-2 indicate that the RQD recovery of the rocks varies with their degree of weathering. The package of fresh to slightly weathered gneisses exhibit good to very good recovery, while those moderately weathered show only fair recoveries. However, highly weathered, and completely weathered gneisses exhibit poor to very poor recovery. The recovery data for biotite schist is consistent with that of gneisses, and the pegmatite weathering grade is fresh to slightly weathered, resulting in good recovery, which is similar to that of the gneisses.

Table 9-2: Average RQD values.

Rock Quality Designation (RQD) All Rock Types					
Rock Type	Completely Weathered	Highly Weathered	Moderately Weathered	Slightly Weathered	Fresh
Grey Gneiss	3.90	17.36	56.35	78.67	89.45
Banded Gneiss	N/A	30.64	59.09	85.08	90.39
Mafic Gneiss	N/A	28.32	67.05	89.79	91.93
Pink Gneiss	4.49	16.74	67.33	84.55	87.78
White Gneiss	0.00	36.67	54.08	83.08	94.63
Biotite Schist	17.07	25.71	64.28	91.49	91.30
Pegmatite	N/A	N/A	N/A	89.79	86.95

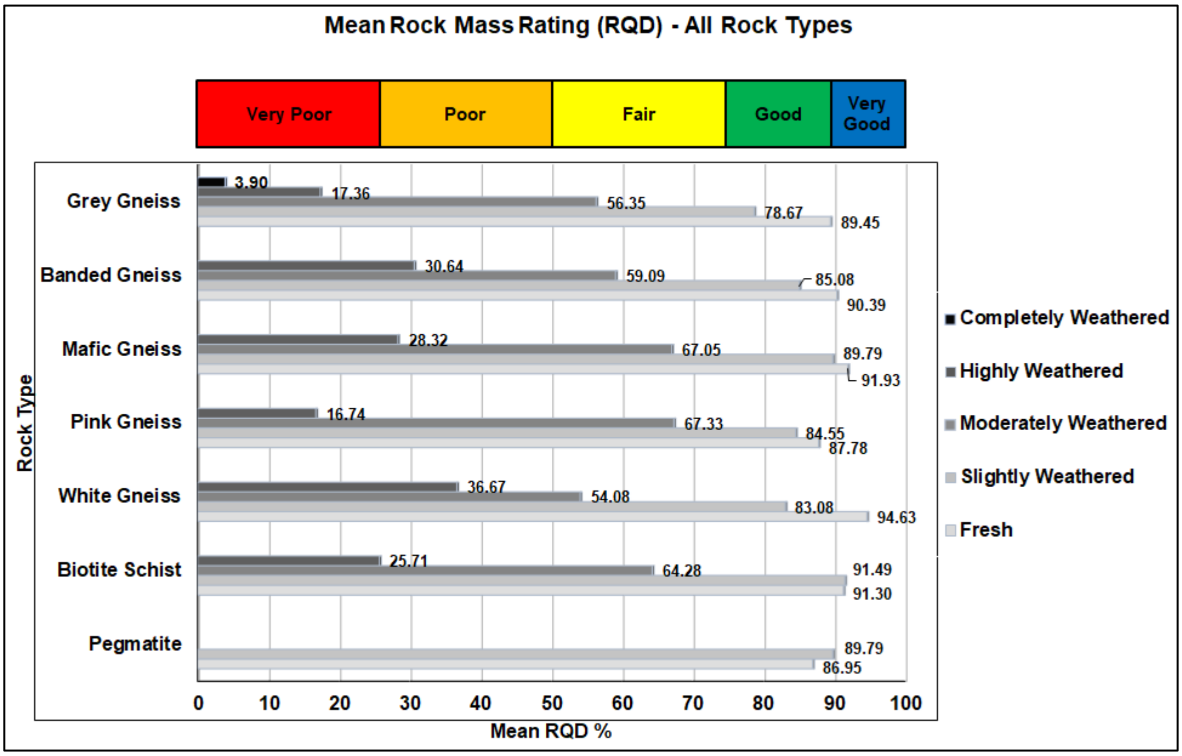


Figure 9-2: Average RQD percentages per rock type according to weathering degree.

Figure 9-3 and Figure 9-4 represent the spatial distribution of the RQD values around the pit. The recoveries are in line with those derived from logging and show that poor recoveries occur near the top of the pit and increase with depth. This pattern is consistent with weathering of the rock with depth.

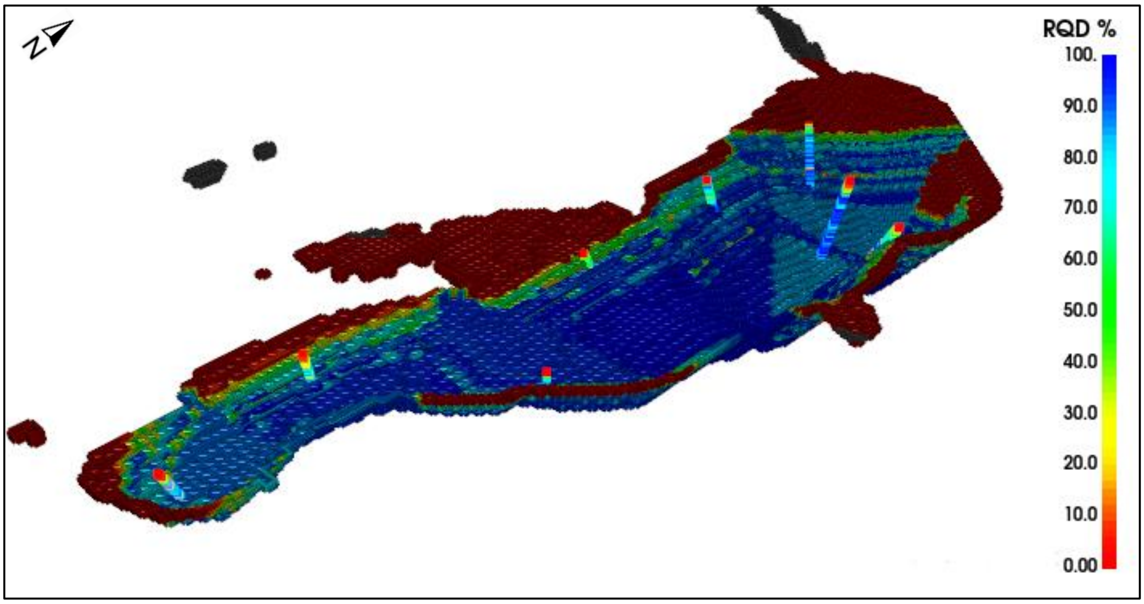


Figure 9-3: Spatial distribution of RQD % data for the Omitiomire pit, Isometric view.

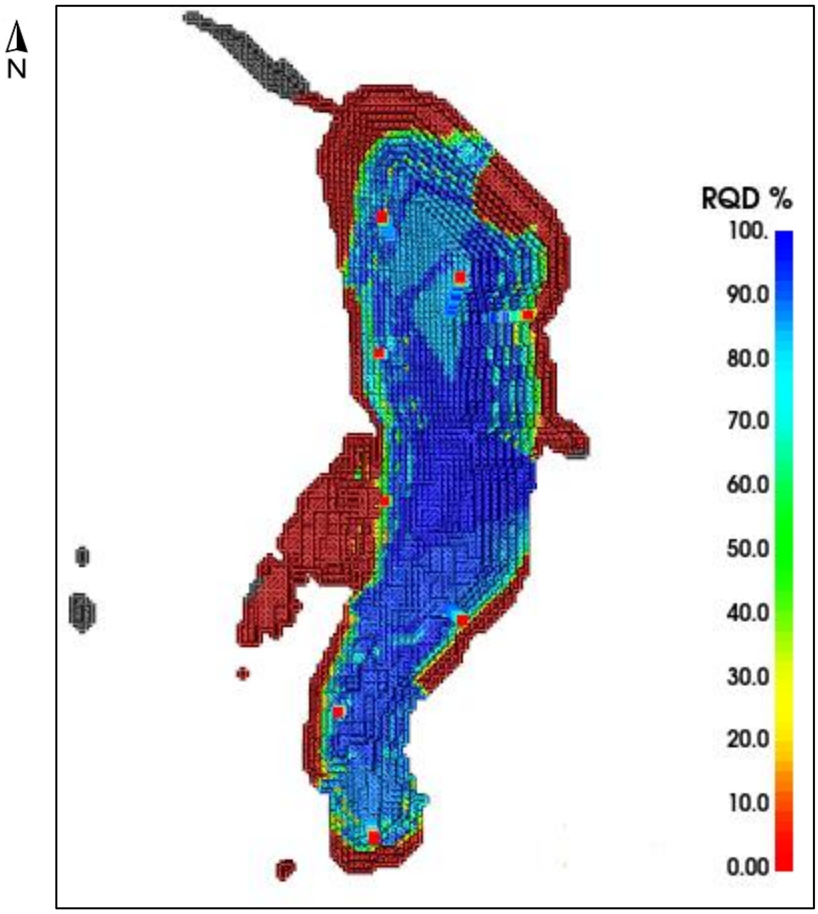


Figure 9-4: Spatial distribution of RQD % data for the Omitiomire pit, Plan view.

9.2 Rock Mass Rating

The Rock Mass Rating (RMR_{89}) classification system by Bieniawski was initially designed to characterize rock masses to aid in tunnel design. However, it has since been adapted to suit both underground and open pit designs and can be used to estimate rock mass properties in situ. The system ratings can also be modified to reflect the favourable or unfavourable orientation of discontinuities concerning the excavation geometry and orientation of discontinuities. The following six parameters are used to classify a rock mass using the RMR_{89} system:

- Uniaxial compressive strength of a rock material
- Rock Quality Designation
- Spacing of discontinuity
- Condition of discontinuity
- Presence of groundwater condition
- Orientation of discontinuities

Each of the six parameters is assigned a rating value which is informed by the characteristics of the core logged during fieldwork. The RMR value lies between 0 and 100 and is obtained by summing the rating value assigned to each of the six parameters. The parameters and associated ratings for the RMR_{89} classification system are presented in Table 9-3. The rock mass rating classes are listed in Table 9-4. The results are tabulated in Table 9-5.

Table 9-3: Bieniawski's RMR 1989 Classification System. (Hoek, 2007)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence) Rating	< 1 m 6	1 - 3 m 4	3 - 10 m 2	10 - 20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.
** Modified after Wickham et al (1972).

Table 9-4: RMR$_{89}$ quality categories.

Class Rating	Class Category
0 - 20	Very poor rock
21 - 40	Poor rock
41 - 60	Fair rock
61 - 80	Good rock
81 - 100	Very good rock

Table 9-5: Average RMR values.

Rock Mass Rating (RMR89) All Rock Types					
Rock Type	Completely Weathered	Highly Weathered	Moderately Weathered	Slightly Weathered	Fresh
Grey Gneiss	39.00	43.13	56.28	68.58	75.10
Banded Gneiss	31.50	43.00	58.60	72.70	74.84
Mafic Gneiss	N/A	39.00	51.50	68.70	74.39
Pink Gneiss	37.50	36.00	59.50	69.77	75.35
White Gneiss	33.50	44.00	58.50	72.86	79.50
Biotite Schist	36.00	43.00	53.00	59.50	71.50
Pegmatite	N/A	N/A	N/A	73.00	74.33

The average RMR values per rock type with the degree of weathering are shown in Figure 9-5. The values indicate that the fresh gneisses all range within the good rock category, slightly weathered gneisses are good while biotite schist falls within the fair rock. Moderately weathered gneisses classify as fair except for pink gneiss, which is poor rock. For highly weathered, the gneisses classify as fair for grey gneiss, banded gneiss, and white gneiss while mafic and pink gneiss are poor rock. Completely weathered are all poor rock for all the gneisses and biotite schist. Pegmatite classes as a good rock for both fresh and slightly weathered. Figure 9-6 and Figure 9-7 show the spatial distribution of the RMR values across the pit.

Most of the pit contains good quality rock, with the initial surface meters ranging from 20 to 40, which is poor rock. The spatial plot shows that the rock type quality according the RMR$_{89}$ system increases in quality with depth.

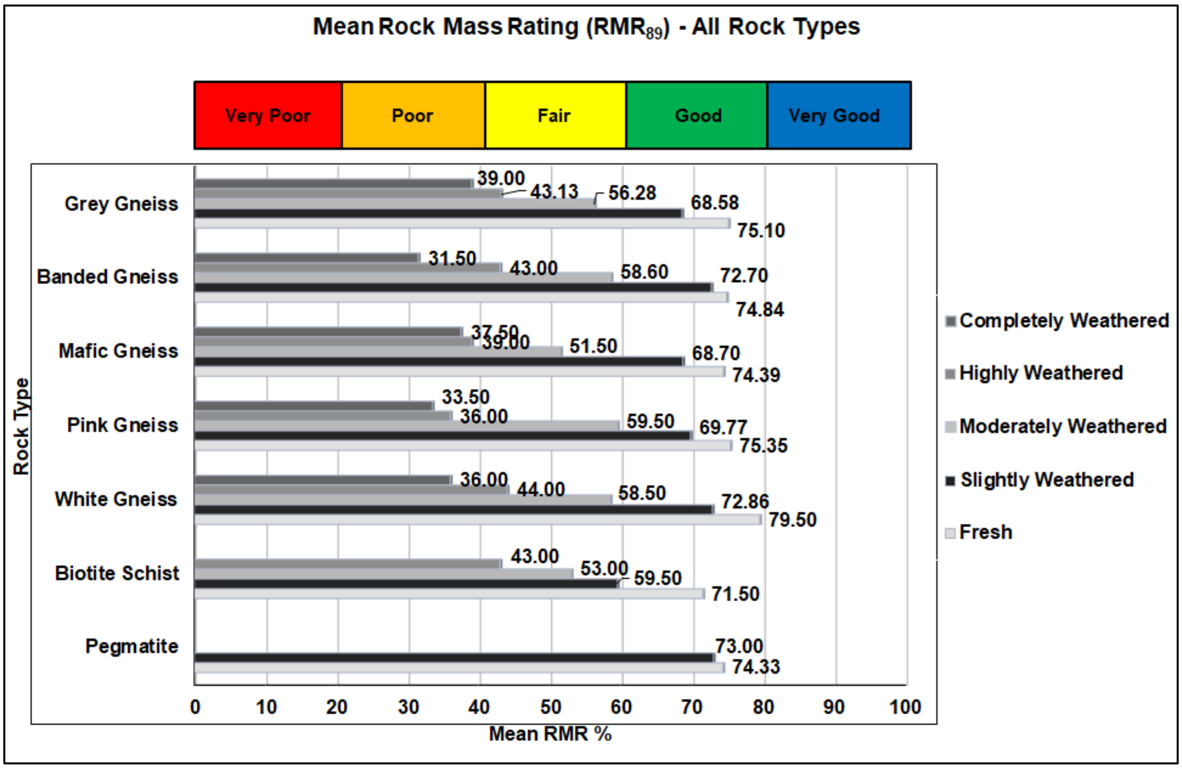


Figure 9-5: Average RMR_{89} percentages per rock type according to weathering degree.

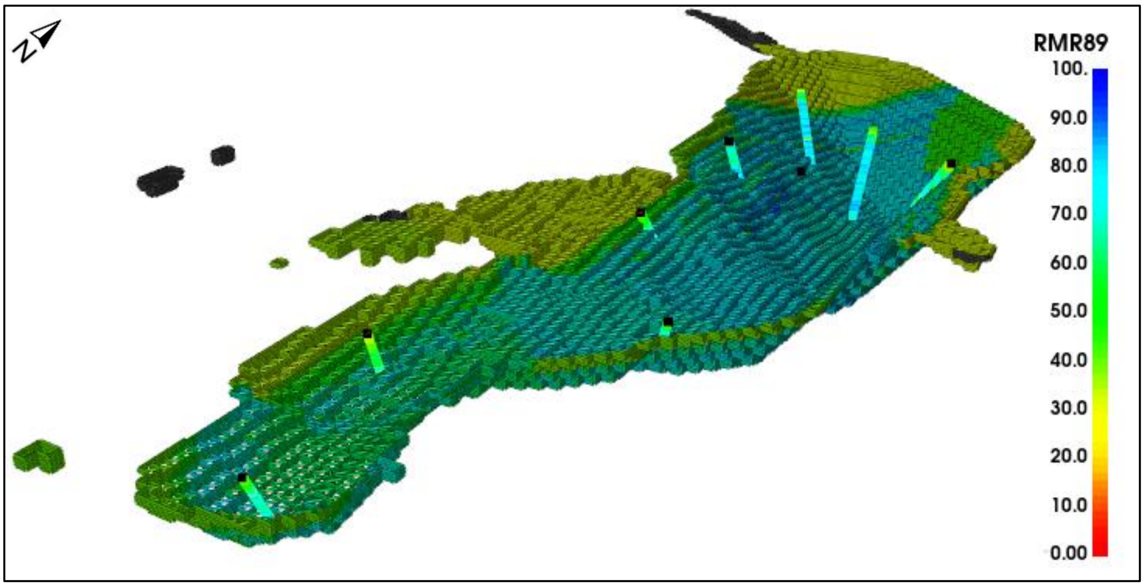


Figure 9-6: Spatial distribution of RMR_{89} data for the Omitiomire pit, Isometric view.

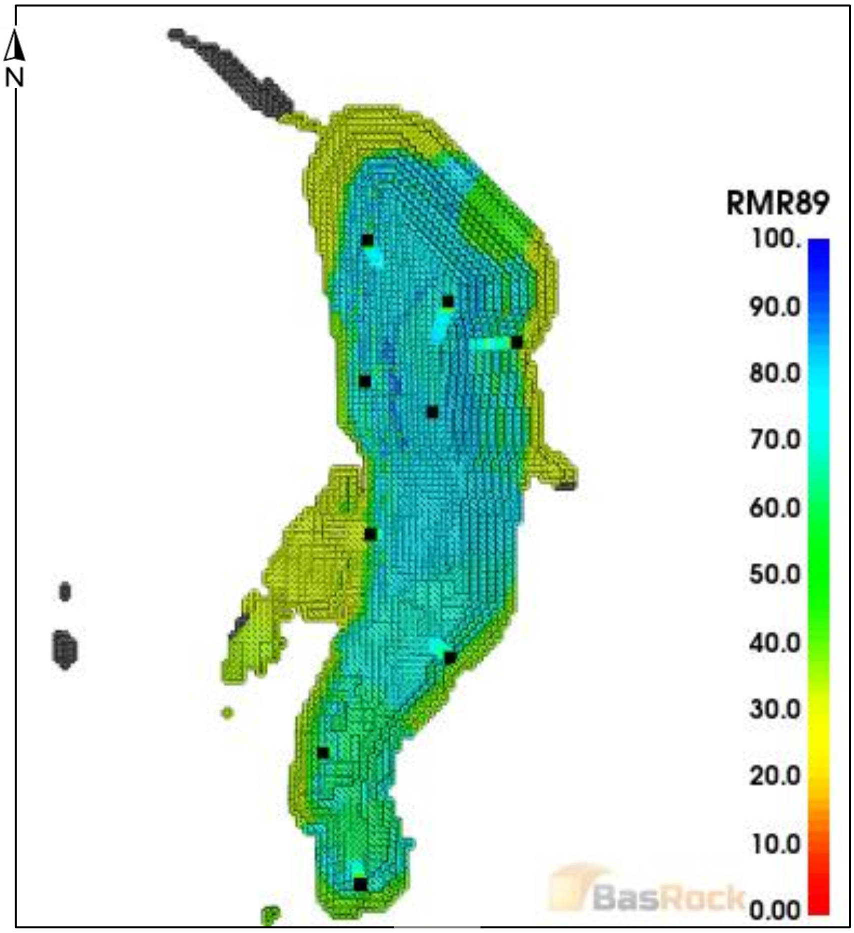


Figure 9-7: Spatial distribution of RMR$_{89}$ data for the Omitiomire pit, Plan view.

9.3 Geological Strength Index

An alternative estimation and classification of rock mass strength under different geological conditions can be made possible by using the Geological Strength Index (GSI) due to difficulties in applying Rock Mass Rating (RMR$_{89}$) to very poor rock mass. The surface conditions and geological structure of the rock mass are considered to obtain the GSI value (Marinos, et al., 2007). In 1995, Hoek, Kaiser, and Bawden (Hoek, E, et al., 1995) developed this system. The GSI chart is best used to derive a value directly from an exposed face and describe the rock structure and block surface. If a face is inaccessible or not yet exposed, the following relationship is used:

$$GSI = (RatingUCS + RatingRQD + RatingJoint\ Spacing + RatingJoint\ Condition + 15) - 5$$

A range of 0 to 100 is used to indicate the geological strength index and is calculated using the same input parameters as that applied in the previous section for RMR$_{89}$ bt assumes dry conditions at a fixed rating of 15 for the Water parameter. The chart in Figure 9-8 is used to derive the GSI values for blocky rock masses. A summary of the GSI values is reported in Table 9-6 and Table 9-7 with respect to weathering domain and presented as a plot visually in Figure 9-9 and Figure 9-10.

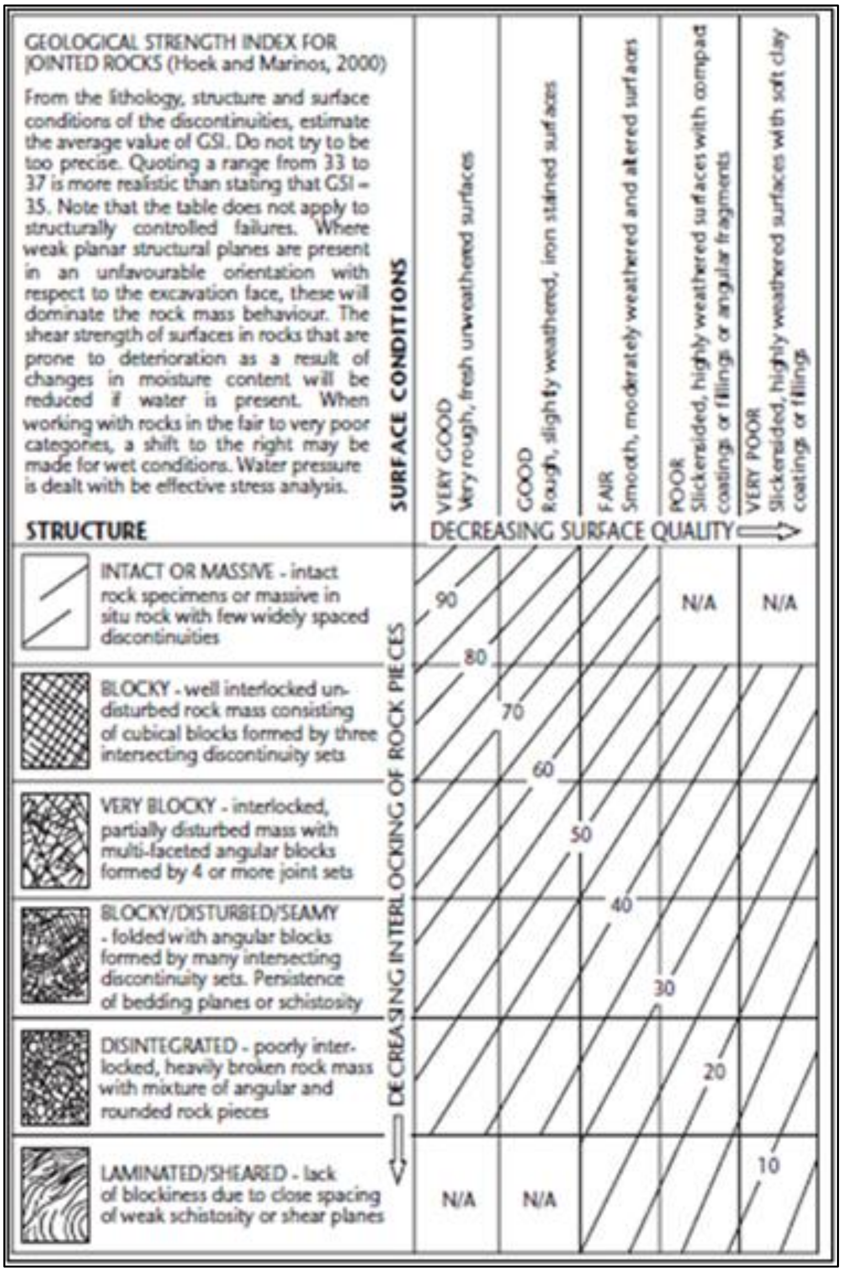


Figure 9-8:The GSI classification chart. (Marinos, et al., 2007)

Table 9-6: Average GSI values per weathering degree.

Geological Strength Index (GSI) All Rock Types					
Rock Type	**Completely Weathered**	**Highly Weathered**	**Moderately Weathered**	**Slightly Weathered**	**Fresh**
Grey Gneiss	34.00	38.13	51.28	63.58	70.10
Banded Gneiss	26.50	38.00	53.60	67.70	69.84
Mafic Gneiss	N/A	34.00	46.50	63.70	69.39
Pink Gneiss	32.50	31.00	54.50	64.77	70.35
White Gneiss	28.50	39.00	53.50	67.86	74.50
Biotite Schist	31.00	38.00	48.00	54.50	66.50
Pegmatite	N/A	N/A	N/A	68.00	69.33

Table 9-7: Average GSI values according to weathering domain

Mean GSI According to Weathering Domain							
Domain	Rock Type						
	White Gneiss	Banded Gneiss	Grey Gneiss	Mafic Gneiss	Pink Gneiss	Pegmatite	Biotite Schist
Fresh	64.24	68.41	64.66	66.63	68.63	68.80	55.91
Weathered	32.00	34.17	40.00	34.00	31.75	N/A	34.50

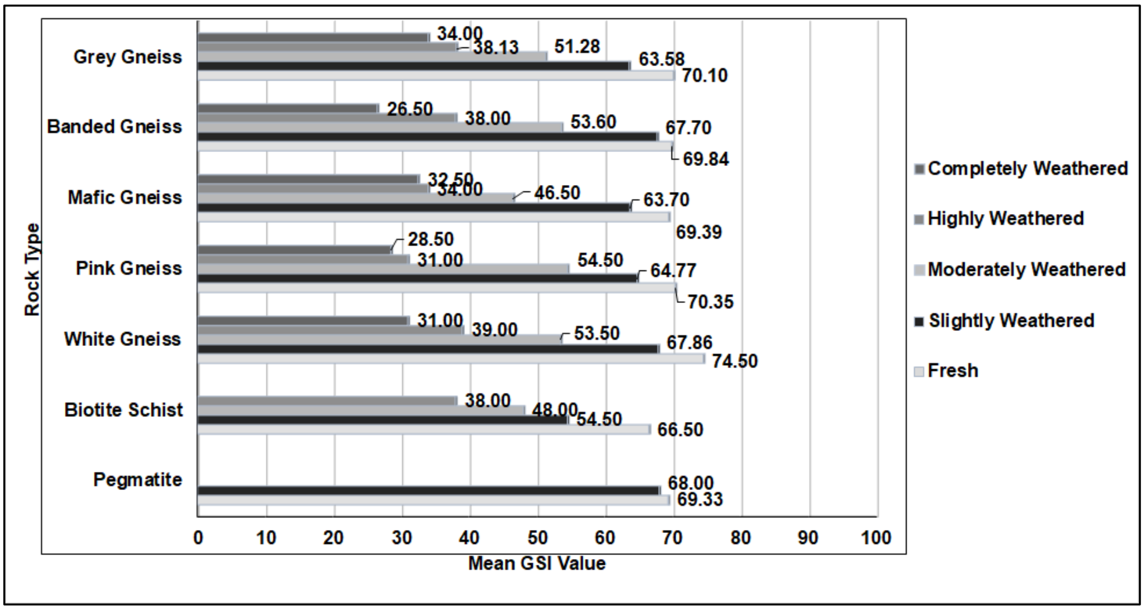


Figure 9-9: Average GSI value per rock type according to weathering degree.

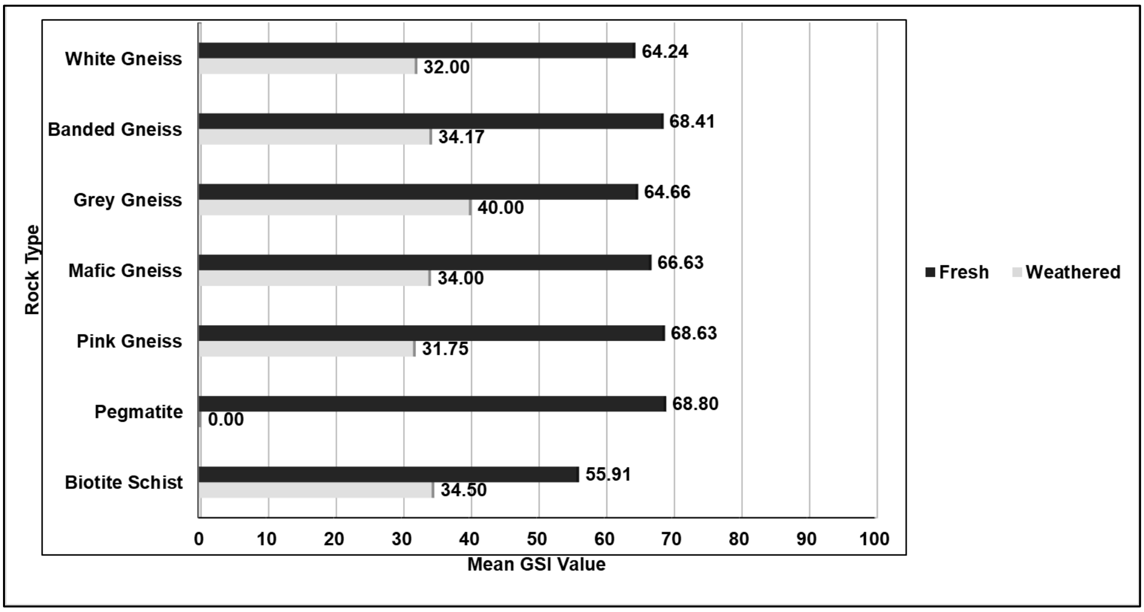


Figure 9-10: Average GSI values according to weathering domain.

The spatial distribution of the GSI values across the pit are shown in Figure 9-11 and Figure 9-12. The plot follows the similar trend of the RQD and RMR plots. However, the gradation in rock material quality is more clearly defined.

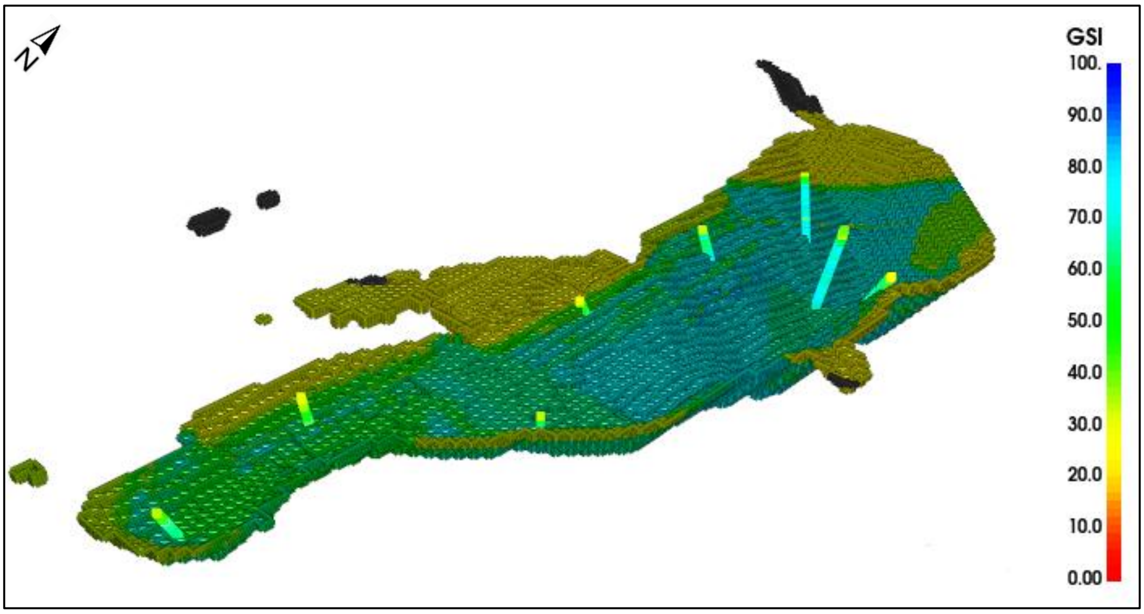


Figure 9-11: Spatial distribution of GSI data for the Omitiomire pit. (Isometric view)

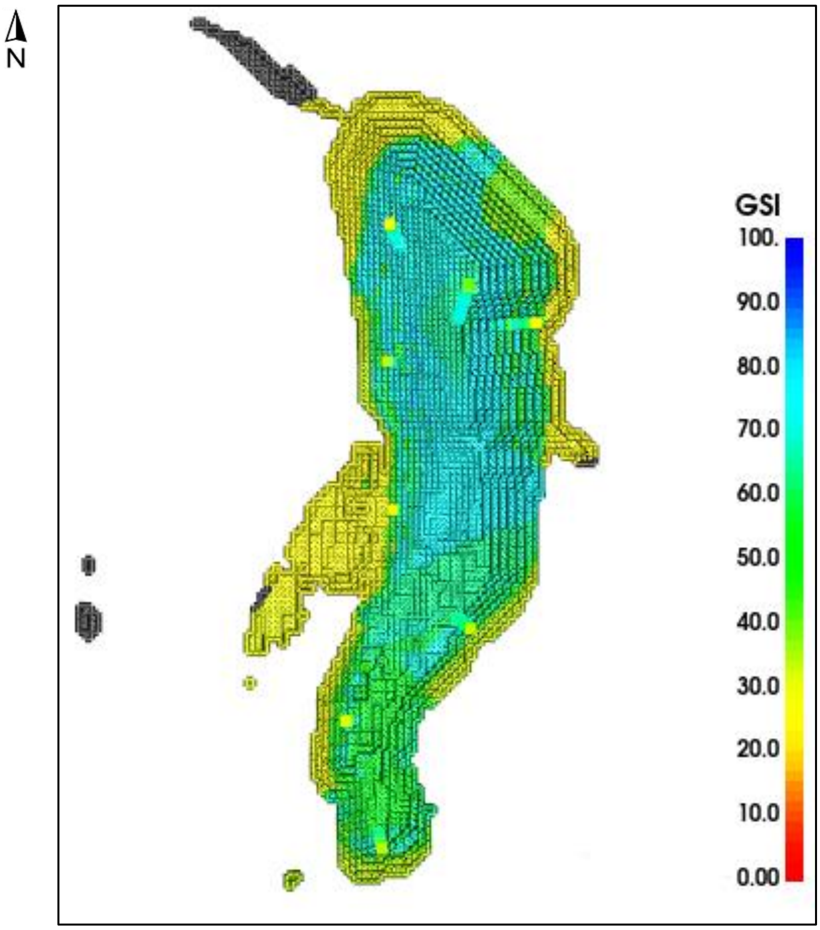


Figure 9-12: Spatial distribution of GSI data for the Omitiomire pit. (Plan view)

10. ROCK STRENGTH LABORATORY TESTING ANALYSIS

Forming part of the data acquisition process, core samples were selected from the eight (8) boreholes for rock testing. The selection was done as per The International Society of Rock Mechanics (ISRM, 1983) guidelines and protocols for sample selection and "Middindi standard operating procedure for geotechnical data acquisition". Five (5) types of tests were conducted namely:

- Uniaxial Compressive Strength (UCS)
- Triaxial Compressive Strength (TCS)
- Indirect Tensile Strength/ Brazilian Tensile Strength (UTB)
- Base Friction Angle (BFA)
- Shear strength of natural joints (STJO)

Table 10-1 indicates the number of rock samples for each type of rock test conducted. The comprehensive rock test laboratory data is inserted into **Error! Reference source not found.**.

Table 10-1: Number of rock strength tests conducted.

Type of Test	No. of Tests		
	Middindi	Historic	Total
UCS	37	20	**57**
UTB	31	0	**31**
TCM	18	0	**18**
BFA	12	14	**26**
STJO	6	0	**6**
Total	**104**	**34**	**138**

The rock strength tests were subjected to a statistical data analysis validation to determine whether the selection of results thereof informed the design as a representation of reality and is summarized in Table 10-2 to Table 10-10. The statistical analysis outlining accepted and excluded outlier values is presented in Figure 10-1 to Figure 10-3. Historical data from SRK was incorporated with that of Middindi for the UCS and BFA test results.

10.1 Uniaxial Compressive Strength Test

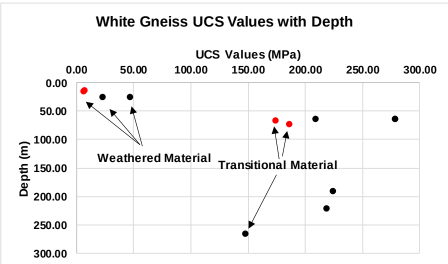
Common in rock mechanics and geotechnical engineering is the Uniaxial Compressive Strength (UCS). In this test, a cylindrical rock sample is subjected to an axial force along its axis until it fails or fractures. The force is gradually increased while measuring the corresponding deformation or strain in the sample until it reaches the maximum load capacity.

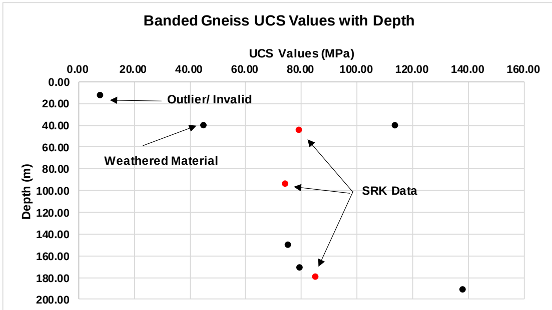
$$\sigma_{ci} = \frac{F}{A}$$

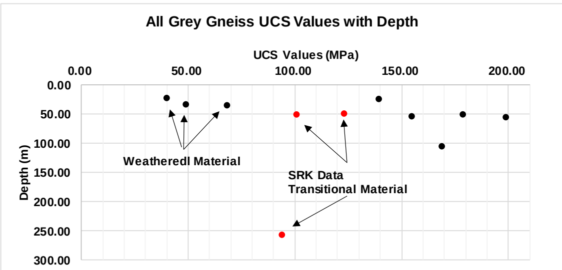
The strength of the rock sample can be determined by dividing the maximum load or force by the cross-sectional area of the sample. This results in a stress value which is a measure of the strength of the rock sample. Table 10-2 tabulates the summary of the mean UCS values per rock type and Figure 10-1 presents the statistical analysis for the rock test data.

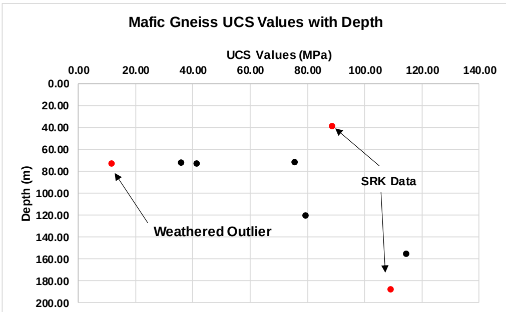
Table 10-2: UCS results summary.

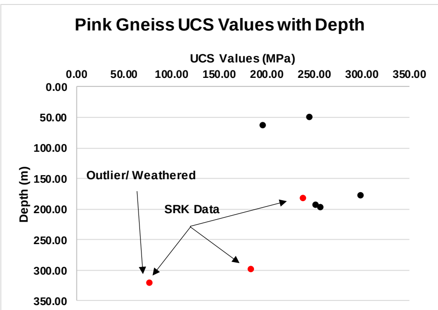
Rock Type	UCS Value (MPa)	
	Fresh Material	Weathered Material
White Gneiss	214.07	20.55
Banded Gneiss	91.89	44.73
Grey Gneiss	135.98	44.34
Mafic Gneiss	77.64	11.40
Pink Gneiss	237.82	76.50
Pegmatite	119.79	64.42
Biotite Schist	N/A	7.80

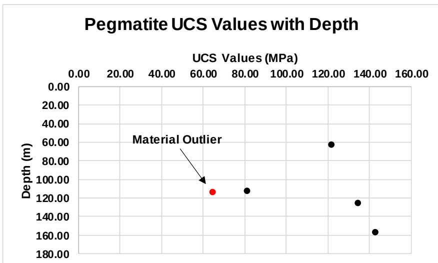












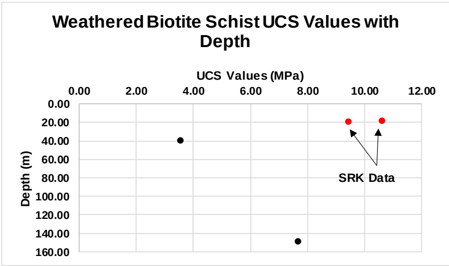


Figure 10-1: Statistical analysis of UCS values.

10.2 Triaxial Compressive Strength Test

The Triaxial Compressive Strength (TCS) test is a type of laboratory test used to determine the strength of rock. This test is commonly used in geotechnical engineering to evaluate the stability of rock formations. A cylindrical sample of the material to be tested is placed in a chamber and subjected to confining pressure. The sample is then compressed axially, or parallel to its axis, until it fails. During the test, the pressure and strain on the sample are monitored and recorded. The maximum compressive stress that the sample can withstand before failing is known as the triaxial compressive strength.

Triaxial compressive strength tests were done to determine the behaviour of rock types when specimens are axially loaded to failure while a confining pressure is constantly applied. Sigma 1 ($\sigma 1$) is the major principal stress and sigma 3 ($\sigma 3$) is the confining pressure, allowing the determination of the following:

- The principal stress state at the point of failure
- The derivation of a strengthening parameter that describes the impact of levels of confinement on the strength of the rock (slope of the principal stress graph)
- The derivation of rock material properties for both the Hoek–Brown (mi, UCS, GSI) and Mohr-Coulomb (shear strength, cohesion, and friction angle) failure criteria. These criteria are used to determine the strength of a rock mass so that failure can be estimated.

The triaxial test results were utilised as input data into the modelling software RSData (RocScience, 2023) which allowed the derivation of the rock mass properties required for the Hoek–Brown and Mohr-Coulomb failure criteria.

Table 10-3 provides the TCS summary from the statistical analysis, which includes the calculated and generated mi value. The generated mi values from RSData were used to determine the validity of each rock type. When mi values for the rock types of mafic gneiss and pink gneiss were deemed invalid, the calculated mi value was utilized, which were derived from a first-pass appraisal correlation (Cai, 2010) with the below formula and consequently substituted, highlighted in red:

$$mi = \frac{UCS}{UTB}$$

Where: UCS is the average uniaxial compressive strength (MPa) value from test results and UTB is the average Brazilian tensile strength (MPa).

Table 10-3: TCS results summary.

Rock Type	TCS Value Summary	
	σ ci (MPa)	Mi Value
White Gneiss	195.67	35.13.
Banded Gneiss	157.14	15.06
Grey Gneiss	251.98	8.99
Mafic Gneiss	71.04	7.46
Pink Gneiss	279.09	42.11
Pegmatite	196.79	12.01
Biotite Schist	N/A	8.21

The comprehensive triaxial rock test laboratory data results and statistical analysis is made available in **Error! Reference source not found.**.

10.3 Indirect Tensile Strength Test

The Brazilian Disc test known as the UTB test is designed to measure the indirect tensile strength of rock by compressing a cylindrical rock sample diametrically between two platens until it fractures or to the point of failure. This results in a vertical split along the diameter of the cylinder and the tensile strength is calculated based on the peak load and the diameter of the sample.

$$\sigma_{tB} = \frac{2\mathrm{P}}{\pi \mathrm{Dt}}$$

Where: σtB is the Brazilian tensile strength (MPa), P is the compression load (kN), D is the diameter (m), and t is the thickness (m) of the sample. (Brasil, 2008).

The statistical summary of the results is attached in Table 10-4 with corresponding statistical graphs in Figure 10-2 and the average UTB results in Table 10-5. Highlighted in red, the UTB value for weathered biotite schist was deemed invalid.

Table 10-4: UTB statistical summary.

Rock Type	UTB Value				
	Mean	Standard Deviation	25th Percentile	50th Percentile	75th Percentile
White Gneiss	12.54	1.09	11.49	13.08	13.11
Banded Gneiss	11.94	2.43	11.11	11.16	12.92
Grey Gneiss	12.26	0.92	11.47	12.20	12.84
Mafic Gneiss	7.46	0.86	6.93	7.27	7.80
Pink Gneiss	12.16	1.62	11.58	12.72	13.27
Pegmatite	12.01	2.47	10.99	13.40	13.78
Weathered Pegmatite	5.86	N/A	5.86	5.86	5.86
Weathered Biotite Schist	0.95	N/A	N/A	N/A	N/A

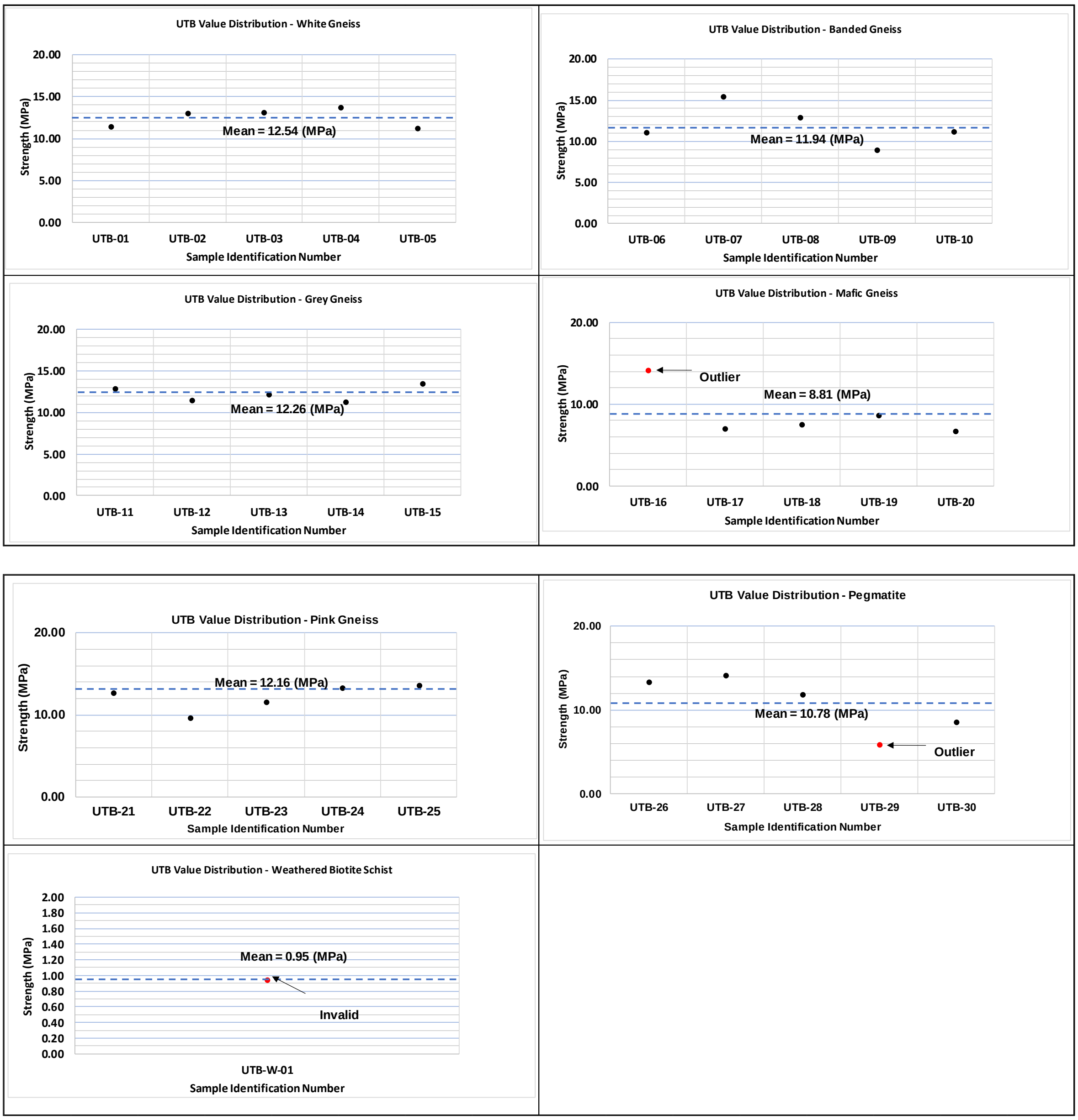


Figure 10-2: Statistical analysis of UTB values.

Table 10-5: UTB results summary.

Rock Type	UTB Value (MPa)
White Gneiss	12.54
Banded Gneiss	11.94
Grey Gneiss	12.26
Mafic Gneiss	7.46
Pink Gneiss	12.16
Pegmatite	12.01
Weathered Pegmatite	5.86
Weathered Biotite Schist	N/A

10.4 Direct Shear Test - Base Friction Angle

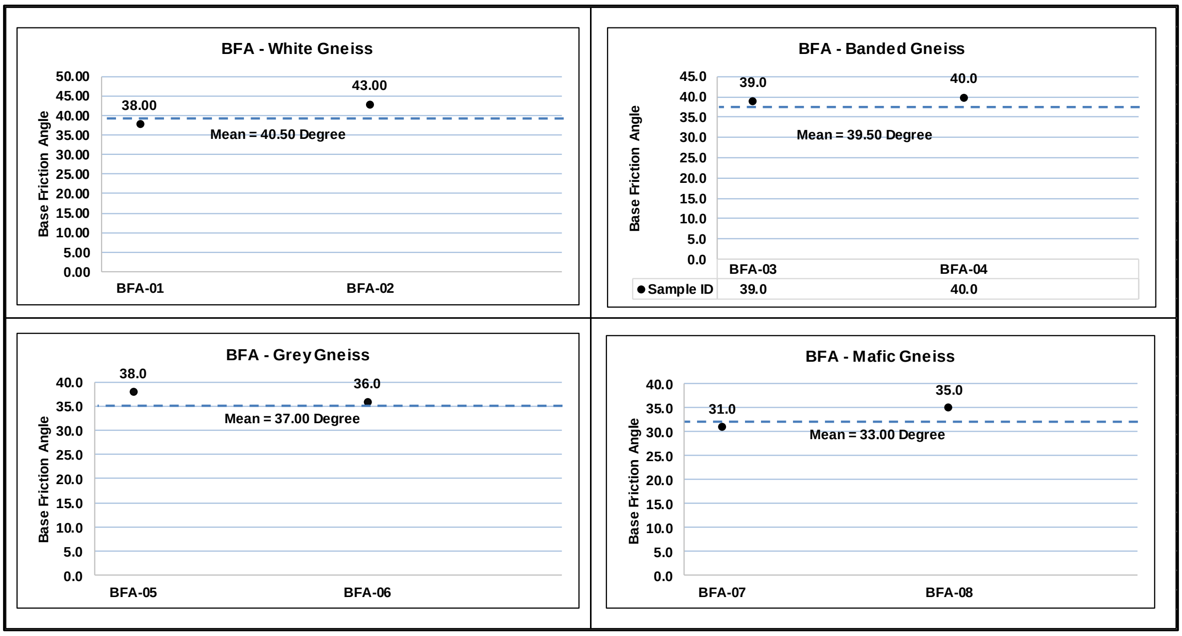
Base friction angle tests are conducted to determine the strength properties of a closed or cemented discontinuity within a rock core sample. These tests aim to measure the shear strength and deformation characteristics of the discontinuity by applying a constant load to a saw-cut rock surface. During a base friction angle test, samples of the rock core with closed joints at different angles are taken and subjected to axial loading or direct shear to measure their shear strength.

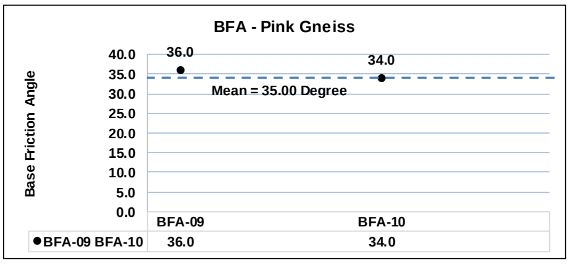
The test involves applying a gradually increasing load to the sample until it fails along the discontinuity plane. The peak and residual shear strengths of the discontinuity are recorded, and the friction angle and cohesion properties of the discontinuity can be calculated based on the test results.

The joint shear strength, namely the cohesion and friction angle can be calculated using the Barton-Bandis strength criteria by applying the joint roughness and joint wall compressive strength. (Barton, 1976) further elaborated in Rock and Joint Properties section herein this report. The received rock test laboratory data is contained within **Error! Reference source not found.**. The summary of BFA results are shown in Table 10-6, with the statistical graphs in Figure 10-3.

Table 10-6: BFA summary data.

Rock Type	Base Friction Angle (°)	
	Fresh	Weathered
White Gneiss	35.00	38.75
Banded Gneiss	34.90	N/A
Grey Gneiss	31.70	N/A
Mafic Gneiss	34.00	N/A
Pink Gneiss	30.88	N/A
Pegmatite	38.50	N/A
Biotite Schist	N/A	34.50





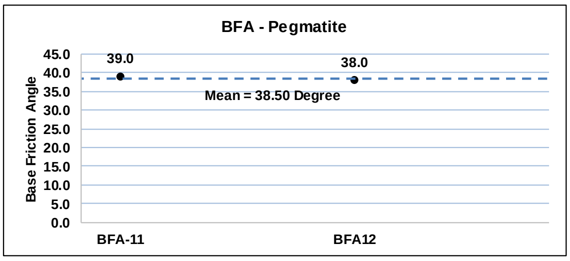


Figure 10-3: Statistical analysis of BFA values.

10.5 Shear Strength of Natural Joints

The shear strength of natural joints (STJO) refers to the shear strength properties of rock discontinuities or fractures that occur naturally in the rock mass. These natural joints can have a significant impact on the stability of rock masses and engineering structures built in or on the rock mass. To determine the shear strength of natural joints, samples with open joints at different angles are taken and tested in a shear testing apparatus. The output result of the STJO tests conducted produced the averaged shear strength parameters attached in Table 10-7 of cohesion and fictional angle of the joints per rock type. The statistical analysis is presented in Table 10-8 and Table 10-9 for the discontinuity friction angle and discontinuity cohesion, respectively. The laboratory data is attached within **Error! Reference source not found.**.

Table 10-7: STJO statistical parameter summary data.

Rock Type	STJO Value	
	Cohesion (kPa)	Friction Angle (°)
White Gneiss	55.00	31.00
Banded Gneiss	15.00	28.00
Grey Gneiss	190.00	32.50
Mafic Gneiss	55.00	34.00
Pink Gneiss	170.00	33.50
Pegmatite	175.00	27.50
Biotite Schist	No Data	No Data

Table 10-8: STJO friction angle statistical data.

Rock Type	STJO Friction Angle (°)				
	Mean	Standard Deviation	25th Percentile	50th Percentile	75th Percentile
White Gneiss	31.00	2.83	30.00	31.00	32.00
Banded Gneiss	28.00	1.41	27.50	28.00	28.50
Grey Gneiss	32.50	0.71	32.25	32.50	32.75
Mafic Gneiss	34.00	1.41	33.50	34.00	34.50
Pink Gneiss	33.50	9.19	30.25	33.50	36.75
Pegmatite	27.50	0.71	27.25	27.50	27.75

Table 10-9: STJO cohesion statistical data.

Rock Type	STJO Cohesion (KPa)				
	Mean	Standard Deviation	25th Percentile	50th Percentile	75th Percentile
White Gneiss	55.00	0.06	0.03	0.06	0.08
Banded Gneiss	15.00	0.01	0.01	0.02	0.02
Grey Gneiss	190.00	0.03	0.18	0.19	0.20
Mafic Gneiss	55.00	0.01	0.05	0.06	0.06
Pink Gneiss	170.00	0.08	0.14	0.17	0.20
Pegmatite	175.00	0.01	0.17	0.18	0.18

10.6 Density

To determine the densities of rocks used in the design, the UCS, UTB, TCS and historical data tests were examined employing a statistical analysis for each rock type, all of which is inserted **Error! Reference source not found.**. From the analysis for each rock type, the average value was a representative measure of density due to low standard deviation and is summarised in Table 10-10.

Table 10-10: Average density summary.

Domain	Rock Type	Density (g/cm^3)	Density (kg/m^3)
Fresh	White Gneiss	2.62	2620.83
	Banded Gneiss	2.73	2733.09
	Grey Gneiss	2.67	2670.33
	Mafic Gneiss	2.91	2909.52
	Pink Gneiss	2.55	2552.98
	Pegmatite	2.63	2629.20
Weathered	White Gneiss	2.53	2534.44
	Banded Gneiss	2.66	2660.99
	Grey Gneiss	2.70	2703.79
	Mafic Gneiss	2.78	2782.00
	Pink Gneiss	2.67	2671.00
	Pegmatite	2.26	2260.45
	Biotite Schist	2.72	2716.74

11. WEATHERING PROFILE

In the field of rock engineering, weathering is classified according to a grading system ranging from 5 to 1, with 5 representing completely weathered material and 1 representing fresh material. This system is defined in Figure 11-1 and conforms to the standards and protocols established by the ISRM International Society for Rock Mechanics in 1981 (ISRM, 1981). By using this grading system, engineers can quantitatively assess the degree of weathering in rocks, which can provide valuable insight into their strength and suitability for various engineering applications.

Term	Symbol	Description	Grade
Fresh	F	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.	I
Slightly weathered	SW	Discolouration indicates weathering of rock material and discontinuity may be somewhat weaker externally than in its fresh condition.	II
Moderately weathered	MW	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as a corestones.	III
Highly weathered	HW	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as a corestones.	IV
Completely weathered	CW	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.	V
Residual soil	RS	All rock material is converted to a soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported	VI

Figure 11-1: Rock engineering weathering description.

Derived from the geotechnical logging of the eight (8) boreholes, a weathering depth was established by observation of core photographs at which depth or elevation of the weathering transitions to fresh rock. The depths or weathering thickness is recorded in the Table 11-1 with an associated weathering depth visual plot displayed in Figure 11-2 and weathering grade spatial distribution attached in Figure 11-3 and Figure 11-4. The spatial distribution plots show that weathering generally decreases with increasing depth.

The average weathered depth is reported to be 8m subject to change based on location at Omitiomire. For conservatism to account for a worst-case scenario, a 10m weathering depth was used.

Table 11-1: Average weathering depth per borehole (m).

Borehole I.D.	Easting (UTM)	Northing (UTM)	Elevation (m)	Weathering Depth (m)
GTB-022-001	803129.00	7581964.00	1687.34	9.00
GTB-022-002	802993.00	7582438.00	1683.52	10.00
GTB-022-003	803462.00	7582782.00	1679.53	6.00
GTB-022-004	803168.00	7583227.00	1686.00	2.00
GTB-022-005	803706.00	7583922.00	1686.79	9.00
GTB-022-006	803156.00	7584290.00	1689.00	8.00
GTB-022-007	803147.00	7583780.00	1688.05	5.00
GTB-022-008	803453.00	7584069.00	1687.65	10.00
Average (m)				**7.38**
Weathering depth (m)				**8.00**

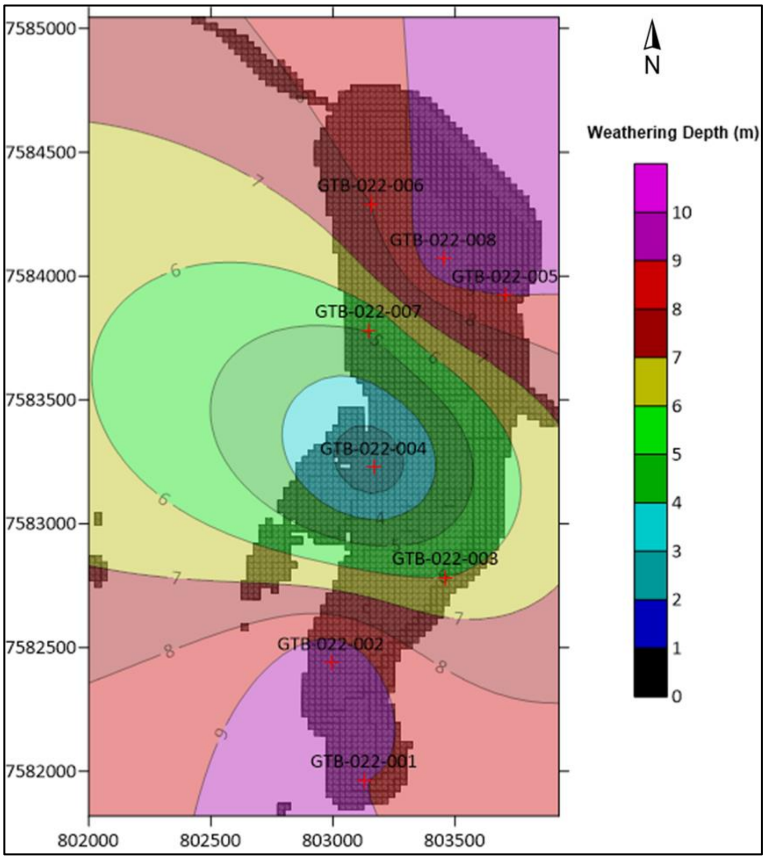


Figure 11-2: Spatial distribution of weathering depth (m).

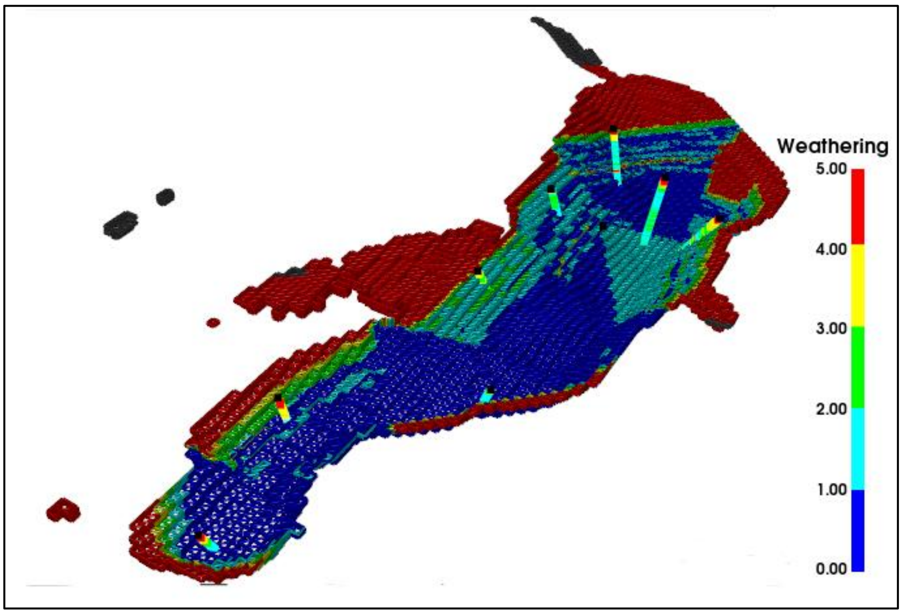


Figure 11-3: Weathering grade for the Omitiomire pit, Isometric view.

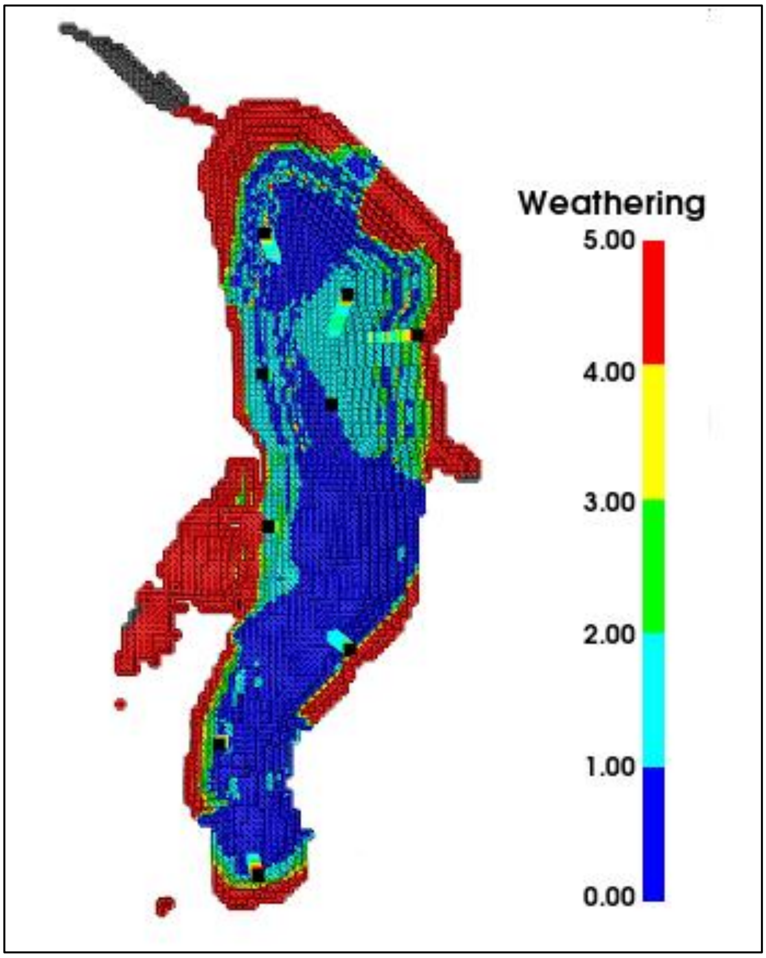


Figure 11-4: Weathering grade for the Omitiomire pit, Plan view.

12. ORIENTATION DATA

The orientation data collected from the orientated boreholes was analysed using the RocScience modelling program Dips, which utilizes spherical projection techniques for plotting, analysis, and presentation of structural data. In addition, Dips can provide a kinematic assessment to indicate the probability of different failure mechanisms, making it a valuable tool for assessing the structural integrity of rock formations.

A selection of the eight (8) Middindi boreholes and forty-two (42) historical boreholes was used as an orientation database for the kinematic assessments totalling a value of 2952 measurements of dip/dip direction to form the orientation database with 1926 values to Middindi and 1033 values to the historical data. For the Middindi source data, only highly reliable and good reliability orientation data was utilised. The raw orientation data, plots per borehole and historical stereographic net are attached in the **Error! Reference source not found.**.

The stereographic nets for major discontinuity sets are shown in Figure 12-1 to Figure 12-3.

Major Joint Sets		
Joint set	Dip (°)	Dip direction (°)
JS1	6	86

Dips plot for all data

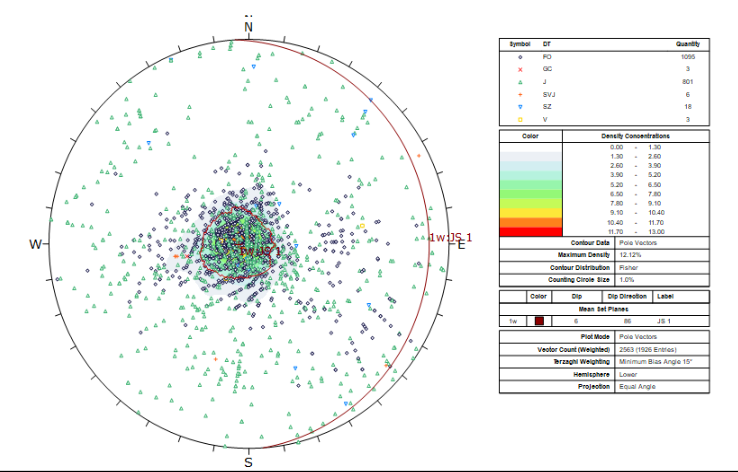


Figure 12-1: Major discontinuity sets, all data.

Major Joint Sets		
Joint set	Dip (°)	Dip direction (°)
JS1	7	96

Dips plot for Foliations

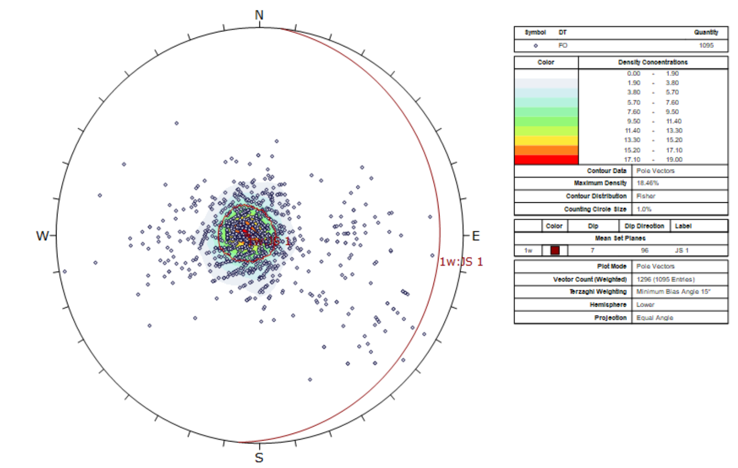


Figure 12-2: Major discontinuity sets, foliations.

Major Joint Sets		
Joint set	Dip (°)	Dip direction (°)
JS1	1	351
JS2	65	7
JS3	88	134

Dips plot for Joints

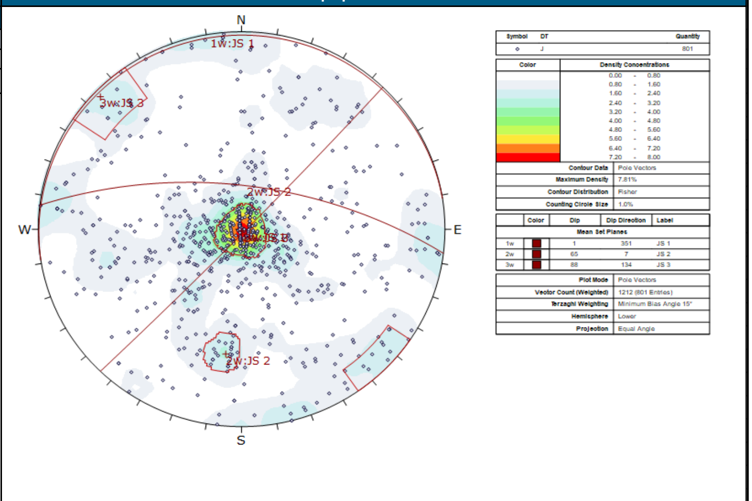


Figure 12-3: Major discontinuity sets, joints.

The plots of major discontinuity sets per rock type are illustrated in Figure 12-4 to Figure 12-10. From the analysis of the plots, a shallow dip exists according to the pervasive foliation through all rock types as the major joint set. The foliations were thus accounted for within the slope stability modelling as an anisotropic feature. Figure 12-11 shows a stereo net plot from each borehole around the pit.

Major Joint Sets

Joint set	Dip (°)	Dip direction (°)
JS1	17	169
JS2	85	271

Dips plot for White Gneiss

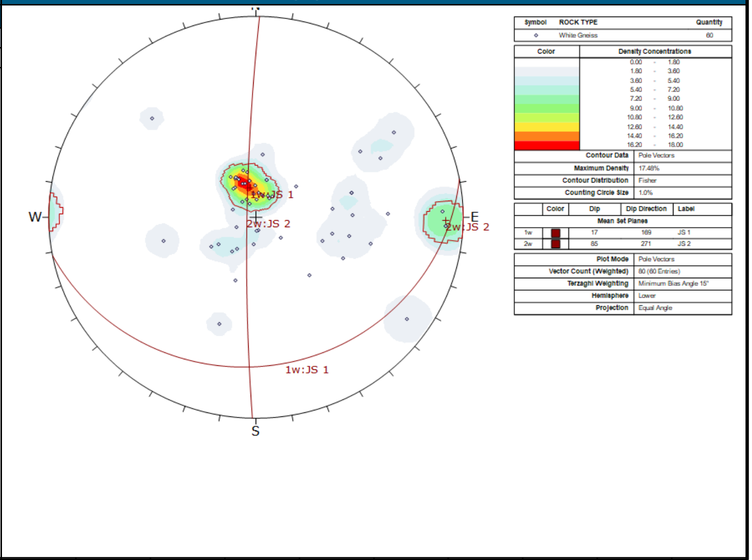


Figure 12-4: Major discontinuity sets, white gneiss.

Major Joint Sets

Joint set	Dip (°)	Dip direction (°)
JS 1	20	75
JS 2	47	268

Dips plot for Grey Gneiss

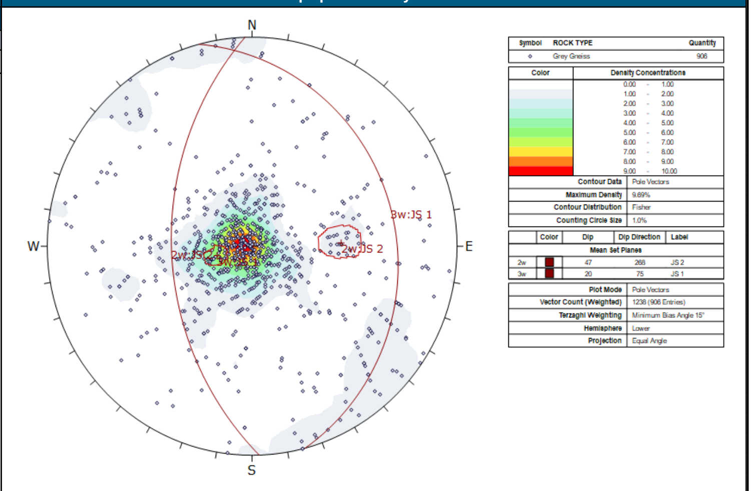


Figure 12-5: Major discontinuity sets, grey gneiss.

Major Joint Sets		
Joint set	Dip (°)	Dip direction (°)
JS1	20	86
JS2	55	79

Dips plot for Banded Gneiss

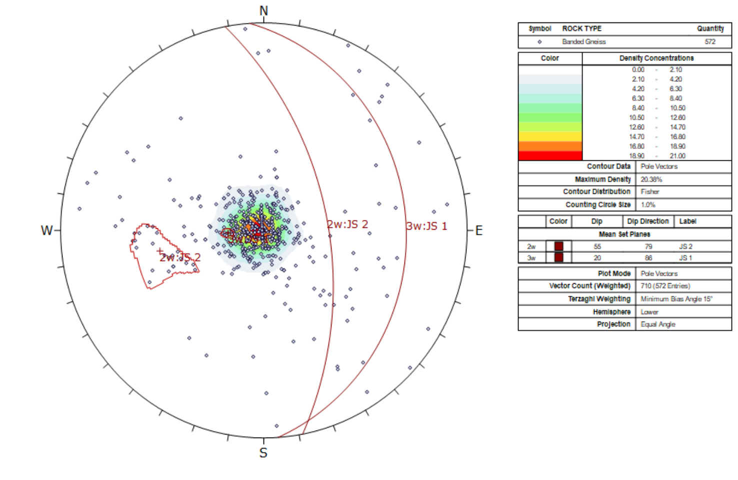


Figure 12-6: Major discontinuity sets, banded gneiss.

Major Joint Sets		
Joint set	Dip (°)	Dip direction (°)
JS1	12	40

Dips plot for Mafic Gneiss

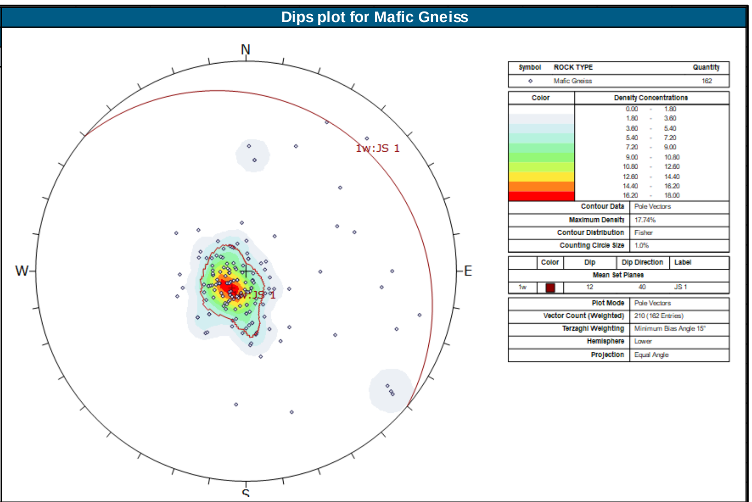


Figure 12-7: Major discontinuity sets, mafic gneiss.

Major Joint Sets		
Joint set	Dip (°)	Dip direction (°)
JS1	8	120
JS2	66	7

Dips plot for Pink Gneiss

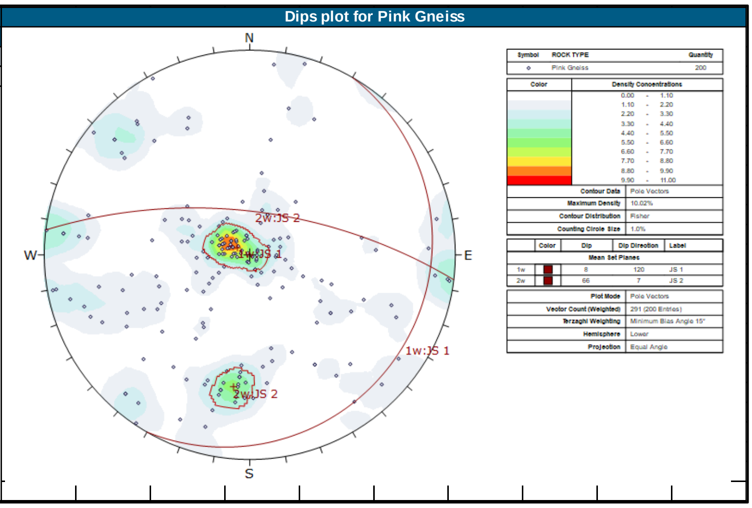


Figure 12-8: Major discontinuity sets, pink gneiss.

Major Joint Sets		
Joint set	Dip (°)	Dip direction (°)
JS1	49	70

Dips plot for Pegmatite

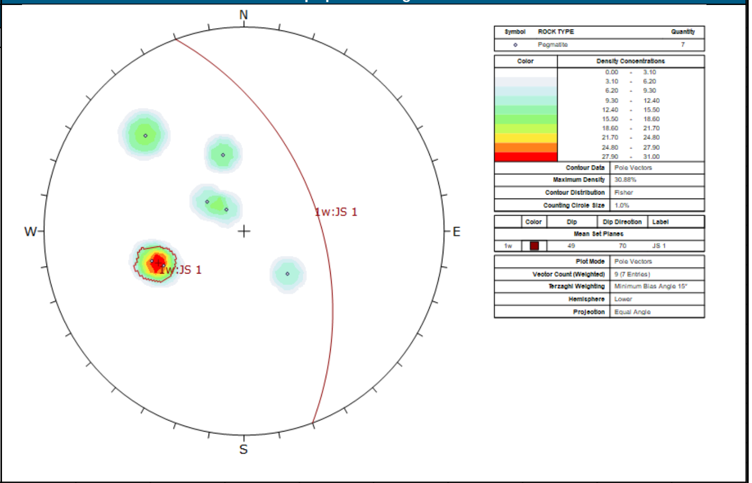


Figure 12-9: Major discontinuity sets, pegmatite.

Major Joint Sets		
Joint set	Dip (°)	Dip direction (°)
JS1	25	91
R	71	227

Dips plot for Biotite Schist

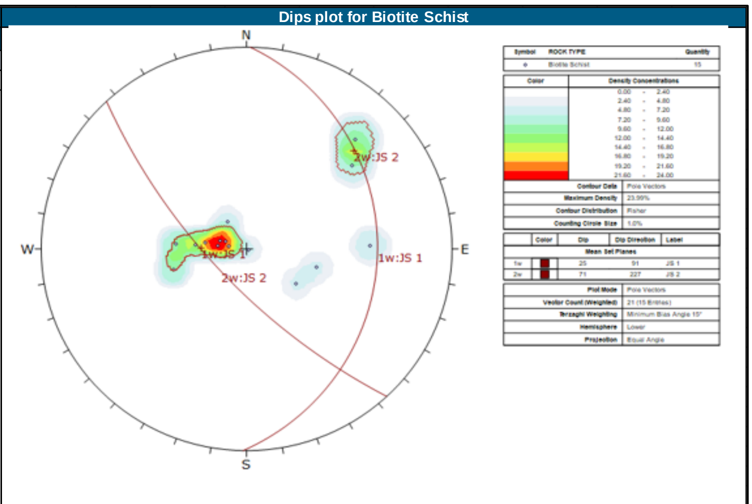


Figure 12-10: Major discontinuity sets, biotite schist.

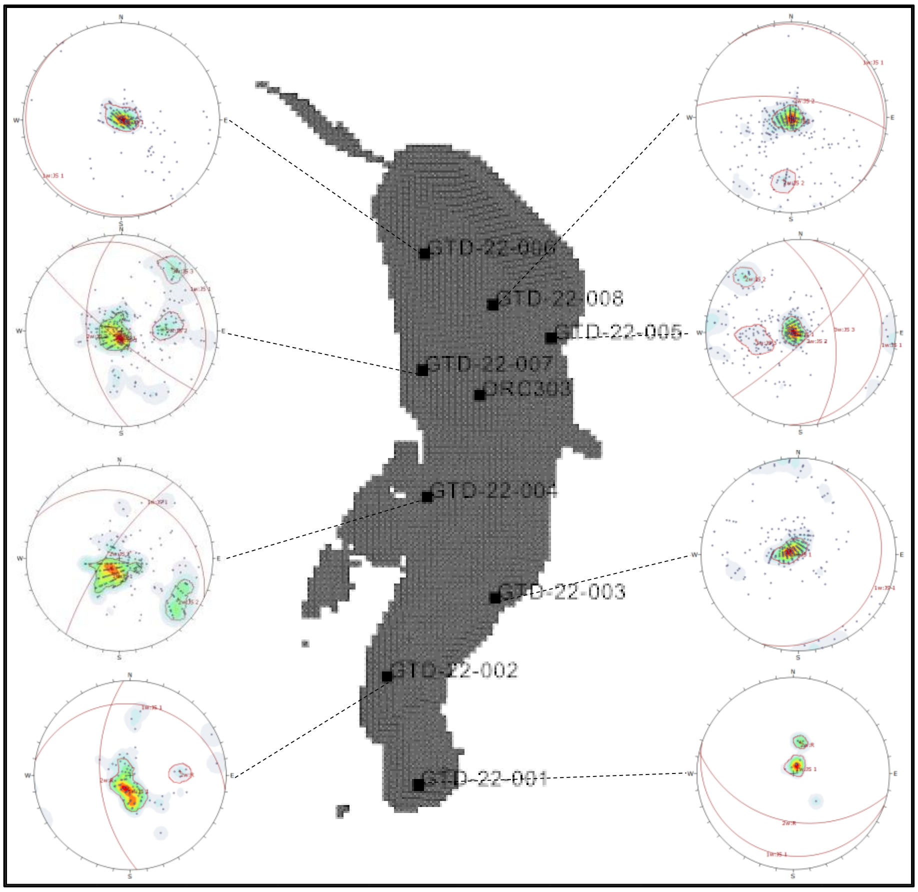


Figure 12-11: Stereographic nets plotted per borehole.

13. ROCK AND JOINT PROPERTIES

13.1 Rock Properties

The Mohr-Coulomb (MC) and Hoek–Brown (HB) failure criteria were used to analyse potential failure modes and describe the general character of the rock mass of Omitiomire. The properties for the criteria are as follows:

Input Parameters:

- The value of the mean Geological Strength Index (GSI) for the rock mass established from geotechnical logging.
- The average Uniaxial Compressive Strength (UCS) of the intact rock, is derived from laboratory rock tests.
- The disturbance factor, D.
- The mi value constants for the intact rock derived from RSData and first-pass appraisal mi value.
- The densities of the rock types were determined from laboratory rock tests.

Output Parameters:

- The value of Hoek–Brown (H-B) constants ''mb'', ''s'' and ''a''
- Mohr-Coulomb (M–C) parameters, friction angle and cohesion

The intact rock properties, field properties, joint properties, Hoek Brown constants and Mohr-Coulomb criteria for the Omitiomire pit area are presented for the fresh and weathered materials in Table 13-1 to Table 13-4, respectively with changes in disturbance factor for each.

Table 13-1: Rock mass properties for fresh material. (Disturbance factor = 1)

Fresh Rock Mass Properties (D = 1) Poor Blasting/Production Blasting								
Rock properties	**Units**	**White Gneiss**	**Banded Gneiss**	**Grey Gneiss**	**Mafic Gneiss**	**Pink Gneiss**	**Pegmatite**	**Biotite Schist**
GSI	*N/A*	64.24	68.41	64.66	66.63	68.63	68.80	55.91
UCS	MPa	214.07	91.89	135.98	77.64	237.82	119.79	7.80
mi	*N/A*	35.13	15.06	8.99	7.46	20.59	12.01	8.21
D	*N/A*	1.00	1.00	1.00	1.00	1.00	1.00	1.00
mb	*N/A*	2.73	1.58	0.72	0.69	2.19	1.29	0.35
s	*N/A*	0.0026	0.0052	0.0028	0.0038	0.0054	0.0055	0.0006
a	*N/A*	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Density	kg/m^3	2.62	2.73	2.67	2.91	2.55	2.63	2.72
cohesion	kPa	1441.34	1114.30	1270.08	975.85	2189.67	1441.77	157.59
friction angle	(°)	61.51	51.57	47.68	42.87	59.47	51.31	22.61

Table 13-2: Rock mass properties for weathered material. (Disturbance factor = 1)

Weathered Rock Mass Properties (D = 1) Poor Blasting/Production Blasting								
Rock properties	**Units**	**White Gneiss**	**Banded Gneiss**	**Grey Gneiss**	**Mafic Gneiss**	**Pink Gneiss**	**Pegmatite**	**Biotite Schist**
GSI	N/A	32.00	34.17	40.00	34.00	31.75	N/a	34.50
UCS	MPa	20.55	44.73	44.34	11.40	76.50	N/a	7.80
mi	N/A	35.13	15.06	8.99	7.46	20.59	N/a	8.21
D	N/A	1.00	1.00	1.00	1.00	1.00	N/a	1.00
mb	N/A	0.27	0.14	0.12	0.07	0.16	N/a	0.08
s	N/A	0.00001	0.000012	0.00005	0.00002	0.00001	N/a	0.00002
a	N/A	0.52	0.52	0.51	0.52	0.52	N/a	0.52
Density	kg/m^3	2.53	2.66	2.7	2.78	2.67	N/a	2.72
cohesion	kPa	171.30	195.56	201.85	82.02	254.02	N/a	71.59
friction angle	(°)	27.56	26.94	26.43	13.92	31.40	N/a	13.10

Table 13-3: Rock mass properties for fresh material. (Disturbance factor = 0.7)

Fresh Rock Mass Properties (D = 0.7) Good Blasting/Mechanical Excavation								
Rock properties	**Units**	**White Gneiss**	**Banded Gneiss**	**Grey Gneiss**	**Mafic Gneiss**	**Pink Gneiss**	**Pegmatite**	**Biotite Schist**
GSI	*N/A*	64.24	68.41	64.66	66.63	68.63	68.80	55.91
UCS	MPa	214.07	91.89	135.98	77.64	237.82	119.79	7.8
mi	*N/A*	35.13	15.06	8.99	7.46	20.59	12.01	8.21
D	*N/A*	0.70	0.70	0.70	0.70	0.70	0.70	0.70
mb	*N/A*	4.92	2.65	1.29	1.19	3.67	2.16	0.73
s	*N/A*	0.0056	0.0103	0.0060	0.0079	0.0106	0.0109	0.0017

a	*N/A*	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Density	kg/m^3	2.62	2.73	2.67	2.91	2.55	2.63	2.72
cohesion	kPa	1793.96	1381.77	1657.54	1251.80	2749.99	1815.42	206.07
friction angle	(°)	64.97	55.30	51.93	47.13	62.52	54.90	28.26

Table 13-4: Rock mass properties for weathered material. (Disturbance factor = 0.7)

Weathered Rock Mass Properties (D = 0.7) Good Blasting/Mechanical Excavation								
Rock properties	**Units**	**White Gneiss**	**Banded Gneiss**	**Grey Gneiss**	**Mafic Gneiss**	**Pink Gneiss**	**Pegmatite**	**Biotite Schist**
GSI	N/A	32.00	34.17	40.00	34.00	31.75	N/a	34.50
UCS	MPa	20.55	44.73	44.34	11.40	76.50	N/a	7.80
mi	N/A	35.13	15.06	8.99	7.46	20.59	N/a	8.21
D	N/A	0.70	0.70	0.70	0.70	0.70	N/a	0.70
mb	N/A	0.84	0.40	0.33	0.20	0.48	N/a	0.22
s	N/A	0.00005	0.00007	0.00017	0.00007	0.00005	N/a	0.00008
a	N/A	0.52	0.52	0.51	0.52	0.52	N/a	0.52
Density	kg/m^3	2.53	2.66	2.66	2.66	2.66	N/a	2.72
cohesion	kPa	250.75	284.78	287.42	127.14	369.43	N/a	111.02
friction angle	(°)	37.34	36.29	34.77	20.93	41.38	N/a	19.78

13.2 Joint Properties

The Mohr-Coulomb properties of discontinuities for each rock type were determined using the direct shear test results and the shear strength of natural joints. For each rock type, the average values of friction angle and cohesion were calculated from the test results and applied to estimate the Mohr-Coulomb parameters of each discontinuity property. The base friction angle direct shear results are presented in Table 13-5 and the shear strength of natural joints in Table 13-6.

To obtain the joint cohesion and joint friction angle of the rock types encountered, the Barton–Bandi's equation for the shear strength of rock joints was utilised (Barton & Bandis, 1990). The equation describes the relationship to model the shear strength of a joint:

$$\tau = \sigma_n \, tan\left[\emptyset_r + \mathrm{JRC}\log_{10}\left(\frac{JCS}{\sigma_n}\right)\right]$$

Joint properties were determined using the software RSData and the Barton–Bandi's analysis method in combination with the shear strength of natural joints. The base friction angle (BFA), joint roughness condition (JRC) and joint compressive strength (JCS) values were used within RSData software to determine the joint cohesion and frictional properties.

Table 13-5: Direct shear joint properties. (BFA)

Rock Type	Cohesion (kPa)	Friction Angle (°)
White Gneiss	28	40
Banded Gneiss	28	39
Grey Gneiss	25	36
Mafic Gneiss	29	37
Pink Gneiss	24	36
Pegmatite	32	42
Biotite Schist	25	36

Table 13-6: Shear strength of natural joints, joint properties. (STJO)

Rock Type	Cohesion (kPa)	Friction Angle (°)
White Gneiss	55	31
Banded Gneiss	15	28
Grey Gneiss	190	33
Mafic Gneiss	55	34
Pink Gneiss	170	34
Pegmatite	175	28
Biotite Schist	No Data	No Data

14. SLOPE NOMENCLATURE

The slope design reported herein provides recommendations for the vertical bench separation (bench or batter height), bench width or berm, bench face (or ''batter'') angle, inter-ramp angle, and overall slope angle, for different design sectors of the open pit. The descriptions below provide further information on the slope configurations listed.

- Berm or Bench Widths – bench widths are selected to facilitate the containment of potential failing material (small wedges and blocks) and to ensure that loose material does not become hazardous to personnel and equipment.
- Bench Height – Mining equipment used to drill and blast the rock determines the bench height. Currently, most large mining operations drill and blast on 12 to 15-metre intervals, with 15 metres being the most common.
- The Bench Face Angle (BFA) is controlled by the material strength, the orientation of the discontinuities in relation to the face azimuth, and/or blasting and excavation practices.
- Stack – when there are multiple benches in a slope design. A stack usually refers to several production benches between catch benches so that the vertical catch bench separation is a multiple (usually two, three, or four) of the production bench height.
- Bench toe – the bottom edge of a bench is referred to as the toe.
- Bench crest – The top edge of a bench is referred to as the crest.
- The inter-ramp angle (IRA) or stack angle is formed by a series of uninterrupted benches and corresponds to the inclination from the horizontal of a line joining the toes of the benches.
- The overall slope angle (OSA) is formed by a series of inter-ramp slopes separated by haul roads and corresponds to the angle formed by the line joining the toe of the lowest bench with

the slope crest. The incorporation of ramps onto a wall will result in a slope that has a shallower overall slope angle than the inter-ramp angle.

The slope nomenclature and geometry discussed above are illustrated in Figure 14-1.

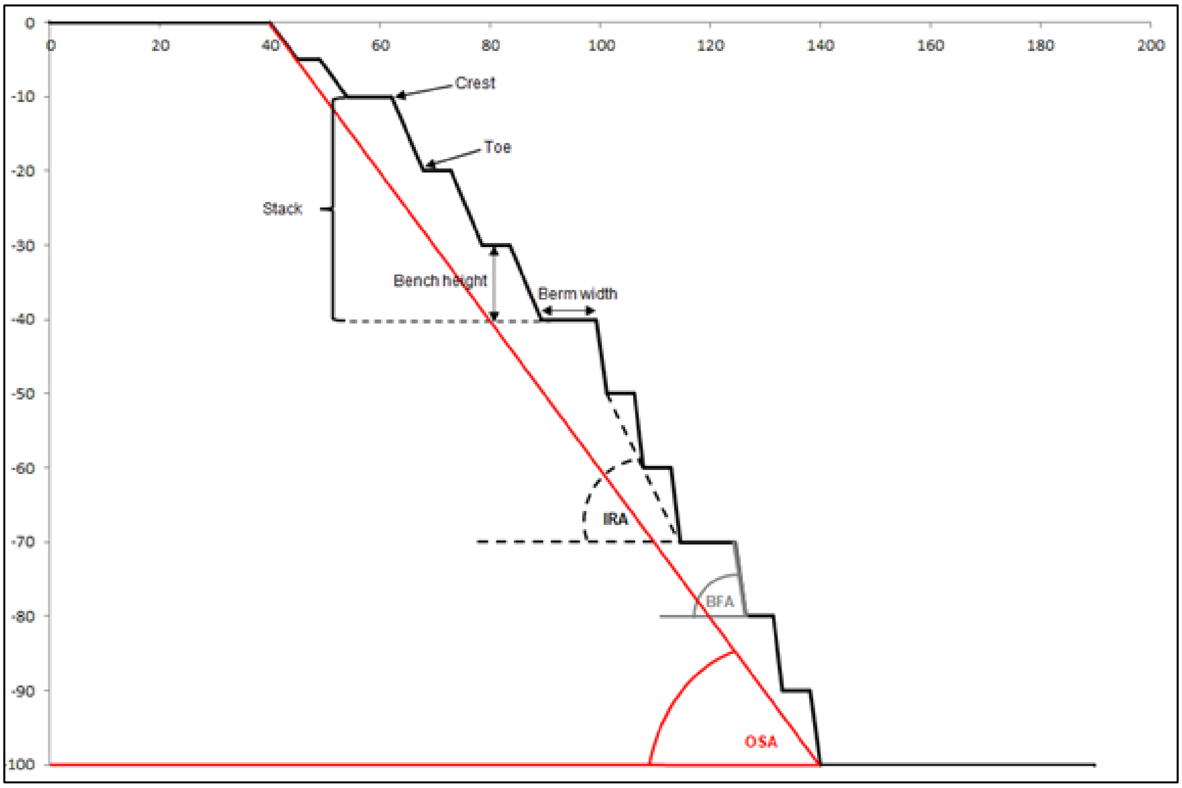


Figure 14-1: Slope nomenclature and geometry.

15. DESIGN SECTORS

The Omitiomire pit was divided into design sectors, based on pit wall directions, rudimentary fault data and hydrological considerations. The design sectors and their respective wall directions are listed in Table 15-1. The planned pit shell indicating each design sector for the pit is illustrated in Figure 15-1.

Table 15-1: Omitiomire pit wall directions and design sectors.

Pit	Design Sector	Wall Direction (°)	Dip Direction (°)
Omitiomire	DS1	46	226
	DS2	86	266
	DS3	95	275
	DS4	120	300
	DS5	232	52
	DS6	300	120
	DS7	260	80

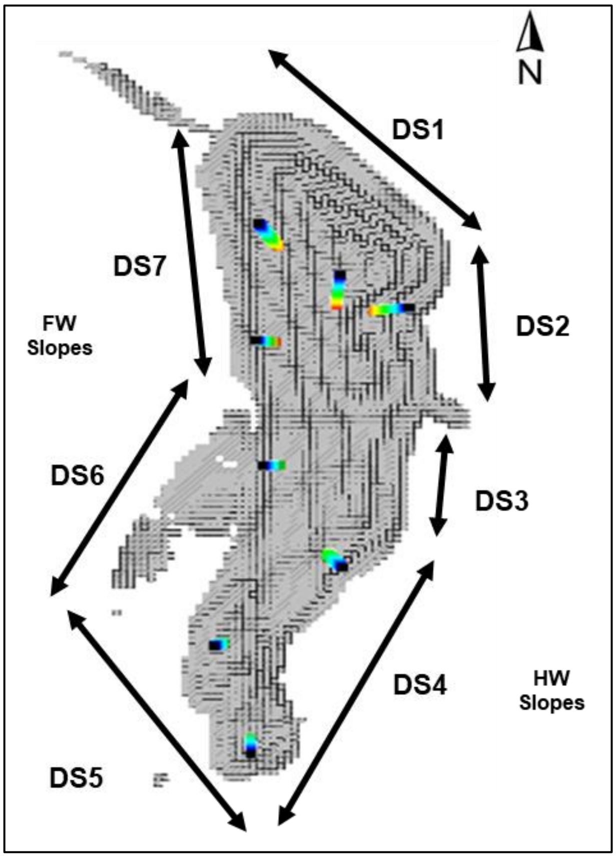


Figure 15-1: Omitiomire pit design sectors.

The design sectors for the Omitiomire pit were designed to varying depths for the final or endwall positions and are summarized in Table 15-2. The elevations for the pit from which the slope depths were derived are depicted in Figure 15-2. The water table levels in and arpunf the pit that were considered in design sector allocation, are depicted in Figure 15-3.

Table 15-2: Final wall depths per design sector.

Domain	Design Sector	Elevation	Depth (m)
HW	**DS1**	1685	360
HW	**DS2**	1680	360
HW	**DS3**	1680	165
HW	**DS4**	1680	195
FW	**DS5**	1680	125
FW	**DS6**	1685	105
FW	**DS7**	1685	360

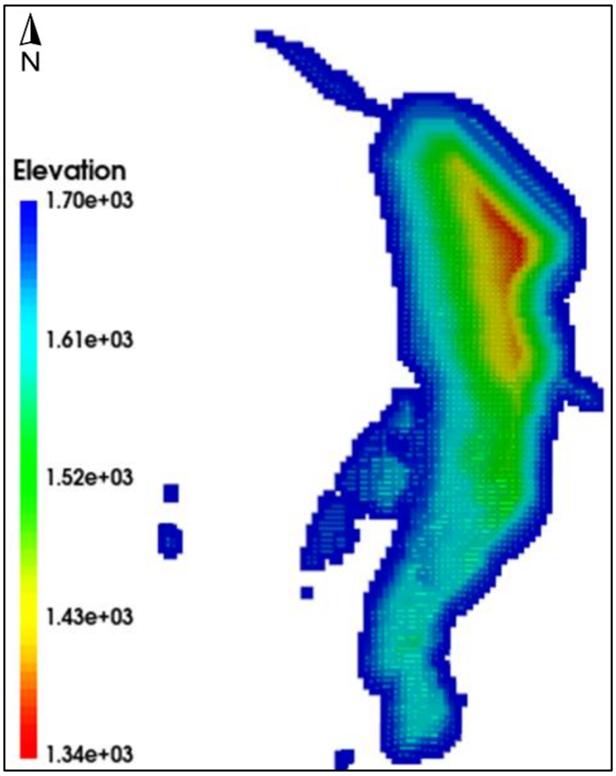


Figure 15-2: Omitiomire pit elevations.

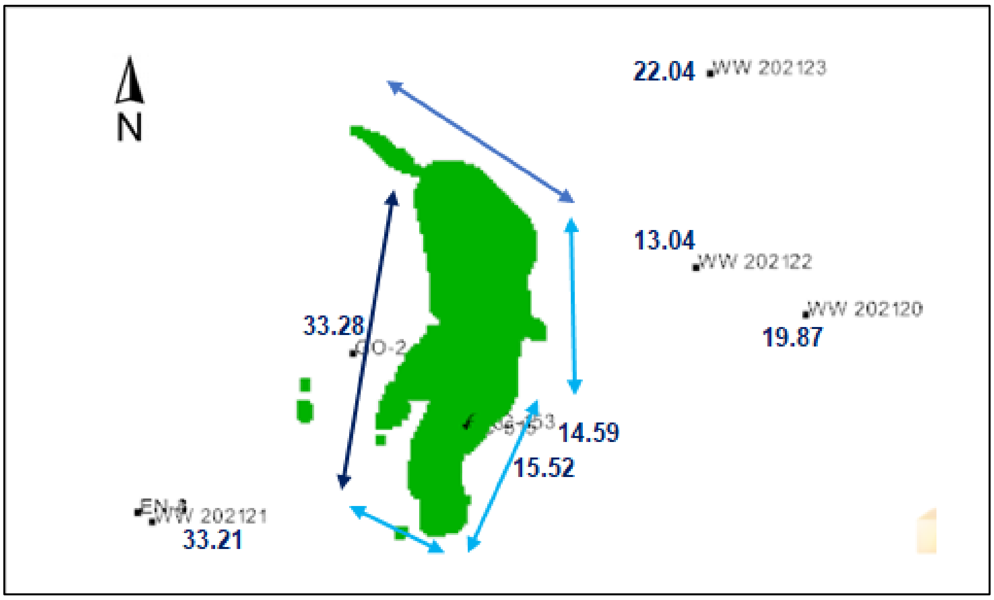


Figure 15-3: Water level depths around the Omitiomore pit.

16. THRESHOLD SAFETY FACTORS

To ensure the risk associated with slope failure was correctly accounted for in the design, an overall slope safety factor (SF) of 1.3 was applied for all slopes at the Omitiomire open pit. This was based on an overall slope scale failure resulting in a high consequence. The SF of 1.5 was selected for the weathered bench stack, which poses a high risk of inter-ramp failure.

The limiting probability of failure (PoF), applied to the bench scale designs, was selected at 10 percent (%). This was applicable for the slope probabilistic assessments and based on bench scale failure posing a moderate to high consequence. The threshold safety factors are shown in Figure 16-1.

		Acceptance criteria[a]		
Slope scale	Consequences of failure	FoS (min) (static)	FoS (min) (dynamic)	PoF (max) P[FoS ≤ 1]
Bench	Low–high[b]	1.1	NA	25–50%
Inter-ramp	Low	1.15–1.2	1.0	25%
	Moderate	1.2	1.0	20%
	High	1.2–1.3	1.1	10%
Overall	Low	1.2–1.3	1.0	15–20%
	Moderate	1.3	1.05	10%
	High	1.3–1.5	1.1	5%

a: Needs to meet all acceptance criteria
b: Semi-quantitatively evaluated, see Figure 13.9

Figure 16-1: Suggested limiting safety factors and probability of failure, (Stacey, 2009).

17. KINEMATIC ASSESSMENT

The design of open-pit mines requires consideration of various failure modes that may occur in both weathered and fresh materials. For weathered or soft material, homogeneous soft rocks or soils are prone to rotational or circular slips. Such failures involve movement along a curved shear surface, leading to slumping of the slipping mass near the crest of the slope and bulging near the toe.

For fresh or hard rock excavations, the stability is often dictated by the presence and orientation of geological discontinuities within the rock mass. Structural failures in such cases can result from slip or failure along pre-existing discontinuities and are primarily observed in fresh material within the pit. The three principal failure mechanisms that may manifest in hard rock excavations are plane failure, wedge failure, and toppling failure. The various failure modes are illustrated in Figure 17-1.

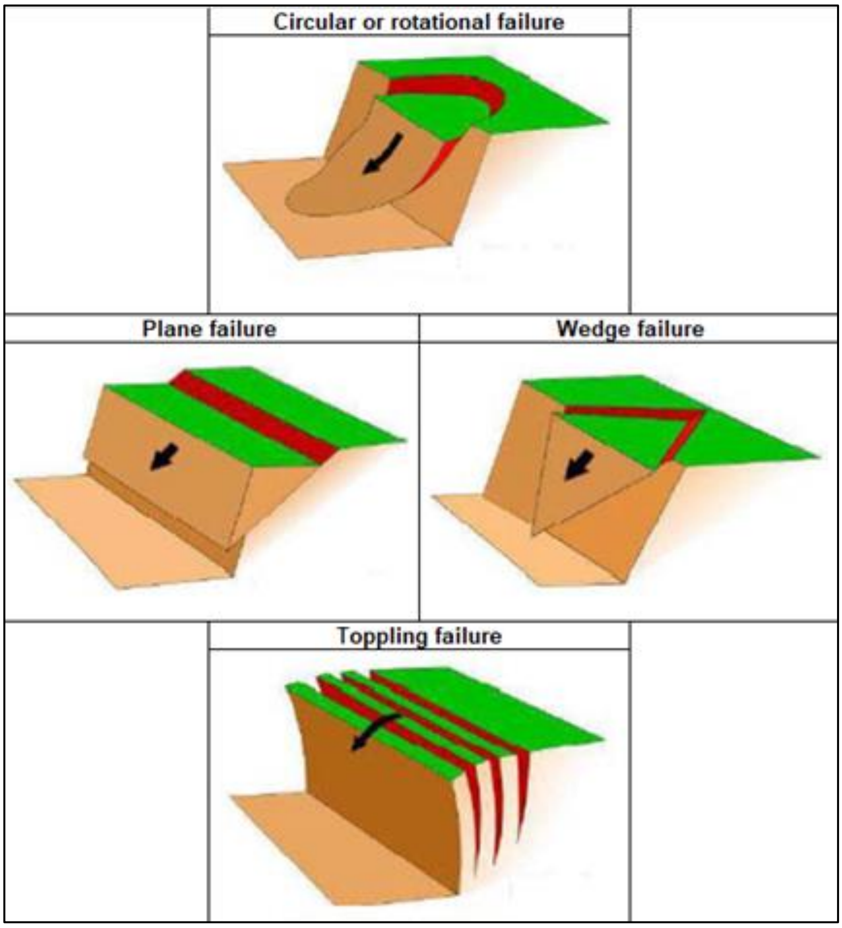


Figure 17-1: Failure mechanisms in slopes.

17.1 Circular/Rotational Failure

The circular failure analysis was carried out using RocScience's software Slide. Slide is a 2-dimensional slope stability programme for evaluating the stability of circular or non-circular failure surfaces in soil or rock slopes.

The weathered material analysed in Slide was a single bench of soil with a 10m bench height. The bench face angle (BFA) was varied between 40, 50, 60, and 70 degrees. Each slope configuration provided a safety factor, which was graphed against the bench face angle. Using these results, a curve could be plotted and the optimal BFA derived for the weathered material.

The analysis provided guidelines for the best-suited bench face angles for the weathered material benches. The Slide analyses are displayed in Figure 17-2 to Figure 17-5. The Slide results are listed in Table 17-1. The graph used for the selection of the optimum bench face angle for the 10m weathered material bench is provided in Figure 17-6.

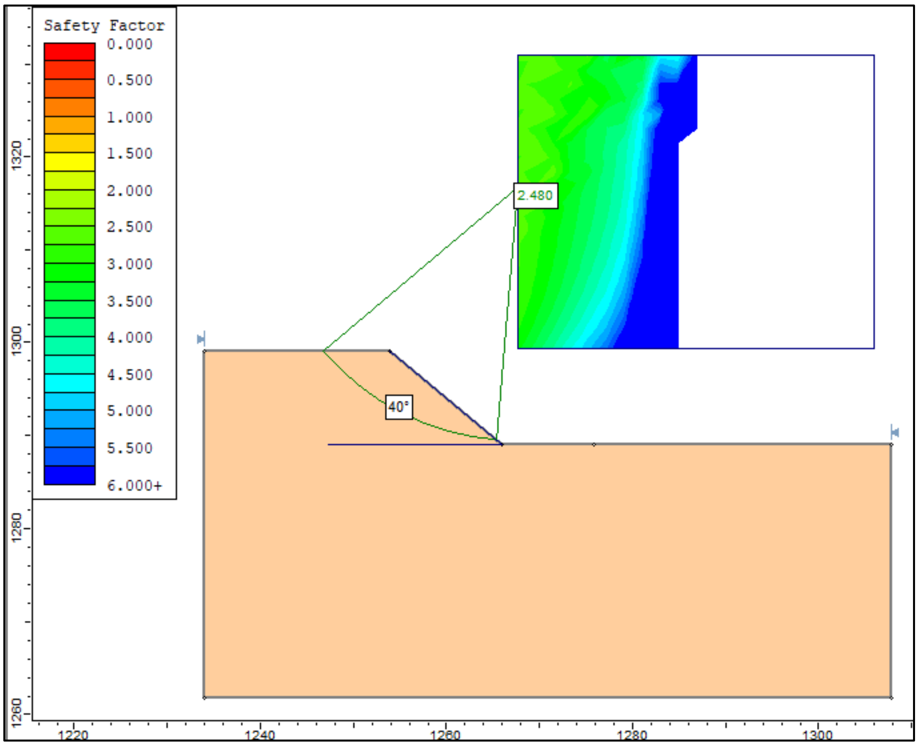


Figure 17-2: Slide analysis for a weathered bench, 40-degree BFA.

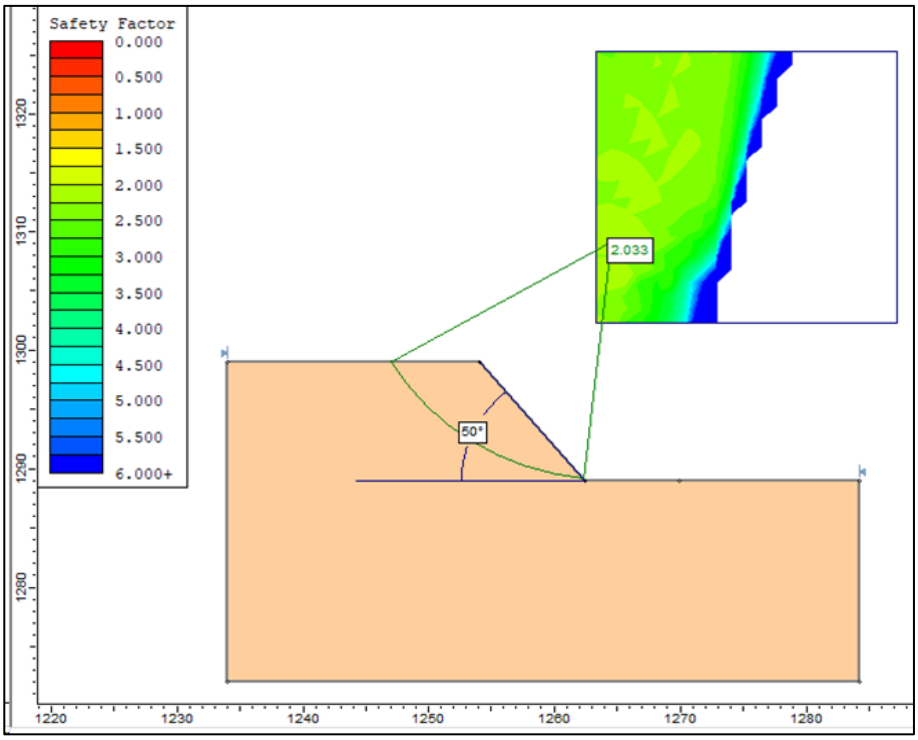


Figure 17-3: Slide analysis for a weathered bench, 50-degree BFA.

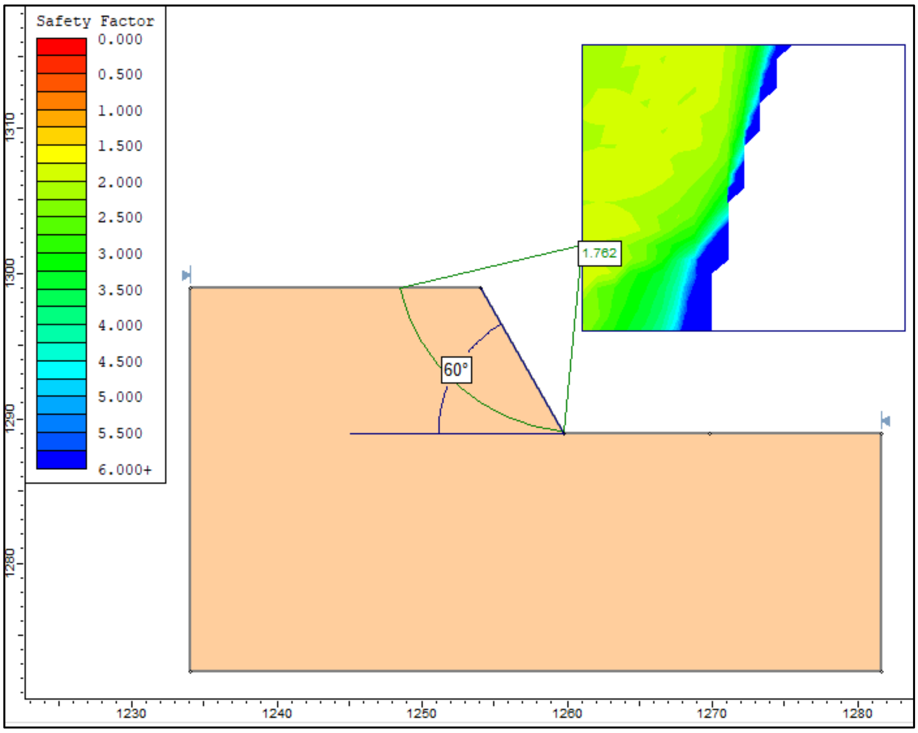


Figure 17-4: Slide analysis for a weathered bench, 60-degree BFA.

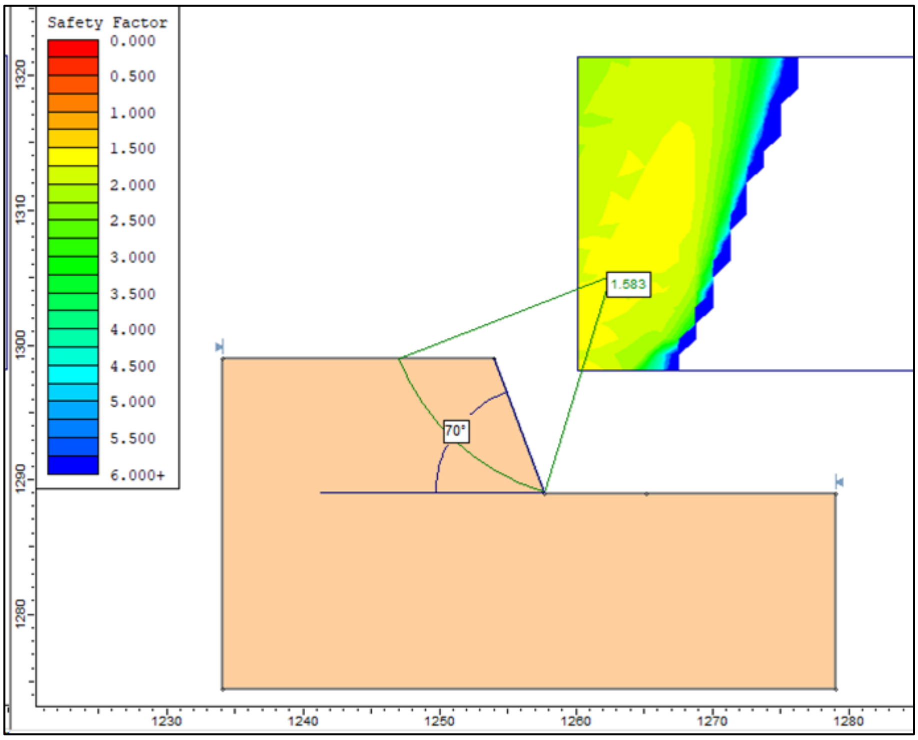


Figure 17-5: Slide analysis for a weathered bench, 70-degree BFA.

Table 17-1: Slide safety factor results.

Weathered 10m Bench	
Bench Face Angle (°)	**Safety Factor**
40	2.19
50	1.96
60	1.75
70	1.50

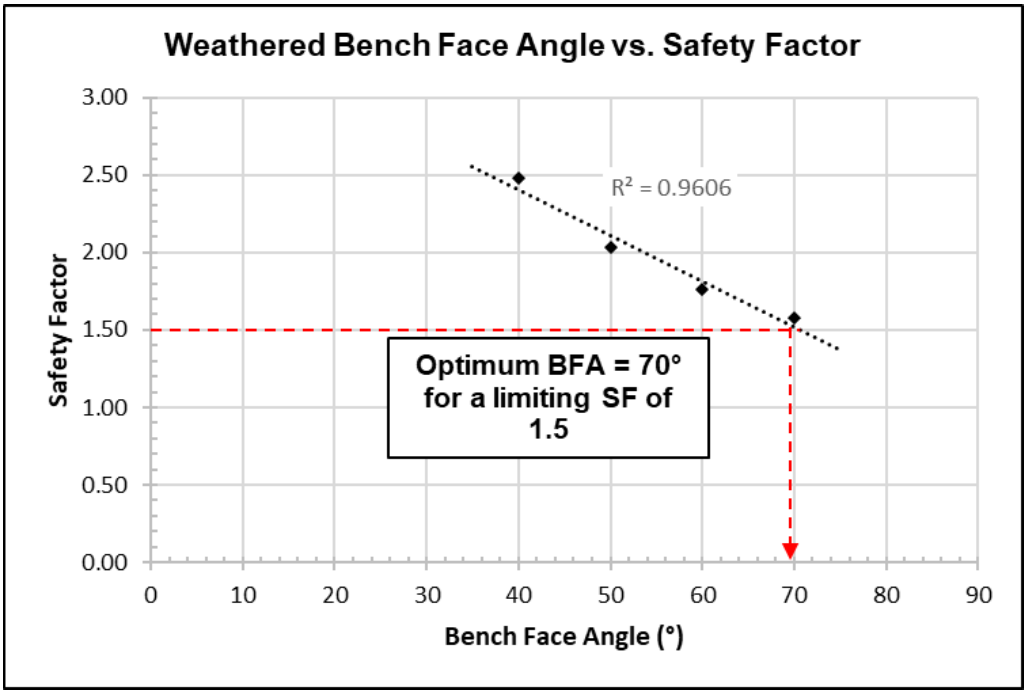


Figure 17-6: Optimum BFA graph for weathered material.

17.2 Plane Failure

The probability analysis of planar failure assessment was performed for the Omitiomire site by utilising RocScience's Dips software. The primary focus of this analysis involved creating models for various bench face angles, ranging from 60°, 70°, 80° and 90° for each design sector in the pit, and estimating the likelihood of planar failure across all rock types.

The critical zone considered in the analysis refers to the area within the daylight envelope of the slope where planar sliding can occur, but outside the friction cone where frictional forces can prevent the failure. The daylight envelope of the slope represents the region where a rock slab can slide if it becomes frictionally unstable. Conversely, any pole that falls outside the friction cone, but within the daylight envelope represents a kinematically and frictionally unstable plane.

Based on the assessment, the probability of planar failure in the example below in Figure 17-7 appears to be very low, producing a probability of failure of 0.00%, as it does not surpass the established threshold of 10%. Detailed Dips plane failure analyses plots for all rock types and corresponding design sectors are provided in the **Error! Reference source not found.**.

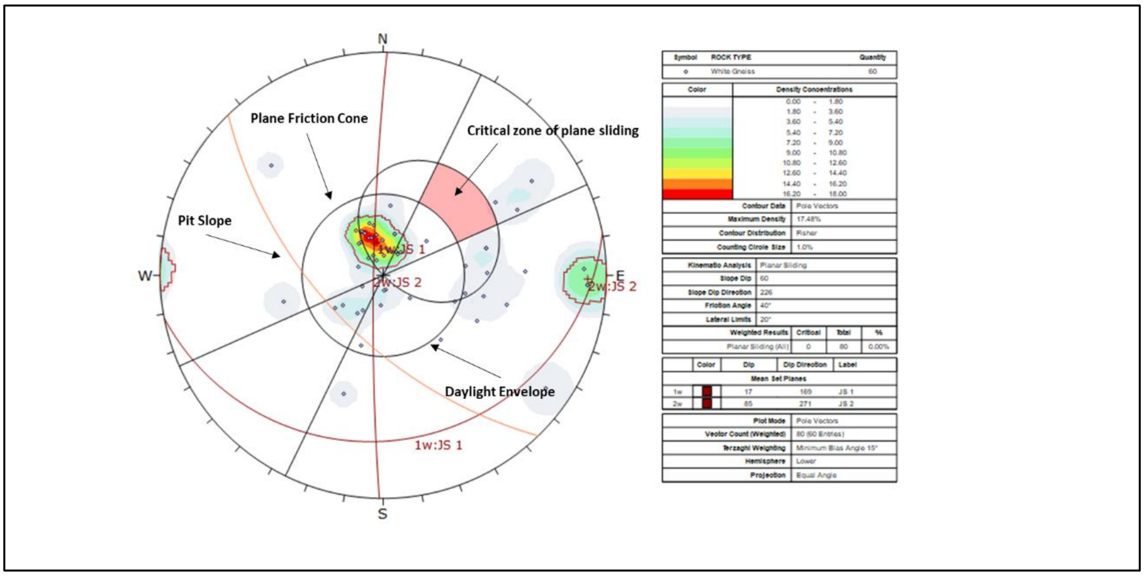


Figure 17-7: Plane failure probability, Omitiomire, DS1, 60˚ BFA.

17.3 Wedge Failure

The wedge stability analysis was conducted at the Omitiomire site using Dips software to model the probability of wedge failure for each rock type, considering different bench face angles ranging from 60°, 70°, 80° and 90° for each corresponding pit design sector. An example of the wedge failure analysis using Dips is depicted in Figure 17-8.

In the analysis, the red and orange crescent areas depict the failure envelope of the slope. The red area is considered the primary critical zone for potential wedge failure, while the orange area is regarded as the secondary critical zone. Specifically, the primary critical zone lies within the plane friction cone but outside of the sloping plane. Any intersection planes within this zone indicate wedges that could potentially slide. The intersections that fall within the secondary critical zone, represent wedges which slide on one joint plane (or planar failure).

The results of the wedge failure analysis below indicate that in design sector 5 of the Omitiomire pit, with a bench face angle of 90º, the probability of wedge failure is 24.99%, which indicates a likely possibility that wedge failure will occur as it exceeds the design threshold of 10%. The complete Dips wedge failure analysis plots are attached in **Error! Reference source not found.**.

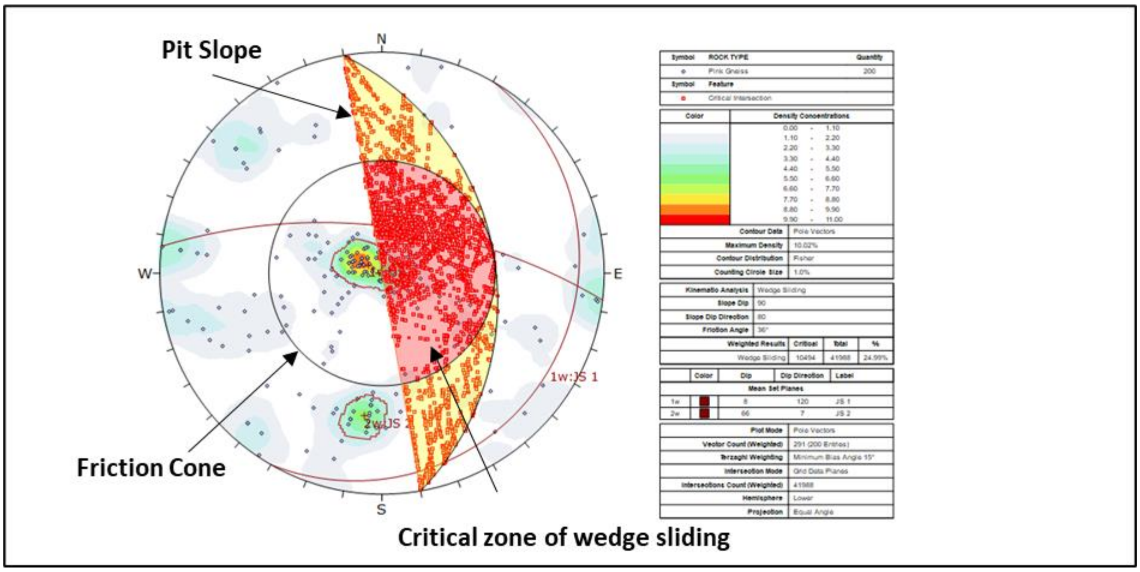


Figure 17-8: Wedge failure probability, Omitiomire, DS5, 90˚ BFA.

17.4 Flexural and Direct Toppling Failure

Toppling failure, according to (Hoek, E & Bray, J, 1981) is a type of instability where columns or blocks rotate about a fixed base. There are two classifications of toppling failures: block (direct) toppling and flexural toppling.

Block toppling transpires when closely spaced joints dip steeply at an angle between 65-85° into the bench and form distinct columns. Additionally, when another joint set that is more widely spaced undercuts the bench's toe. (Lorig, L, et al., 2009)

In contrast, flexural toppling happens when inward dipping columns are more consistent and maintain face-to-face contact while bending over in flexure. This type of toppling failure is typically linked with thinly bedded or slightly metamorphosed rocks instead of jointed sedimentary or igneous rocks (Lorig, L, et al., 2009)

The potential for flexural toppling was assessed in Dips, with an example of the analysis for design sector 3 shown in Figure 17-9. Attached in Figure 17-10 is an example of direct toppling assessment analysis in design sector 5.

The critical zone (shaded in light red) for flexural toppling is bounded by the stereo-net perimeter, lateral limits, and the slip limit plane. Any poles that fall within this region, represent a risk of flexural toppling.

The critical zone for direct toppling falls within the area shaded in red. The area shaded in orange represents the zone where a risk of oblique toppling exists. The lateral limits define the extent of the primary critical zone, relative to the dip direction of the slope. The complete analysis of direct toppling failure Dips plots and flexural toppling failure Dips plots are available in **Error! Reference source not found.**.

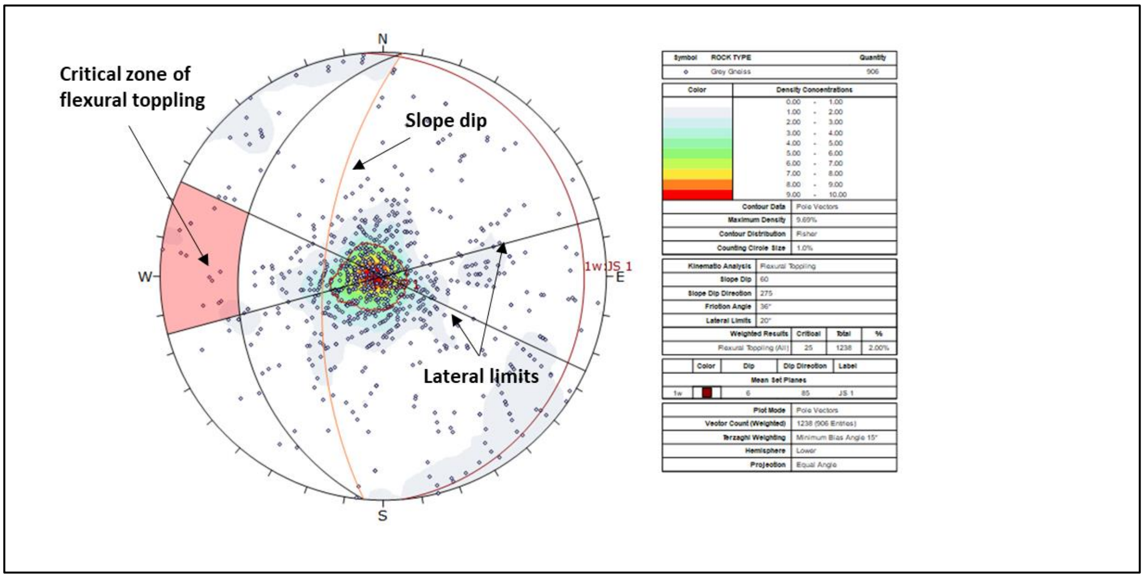


Figure 17-9: Flexural failure probability, Omitiomire, DS3, 60˚ BFA.

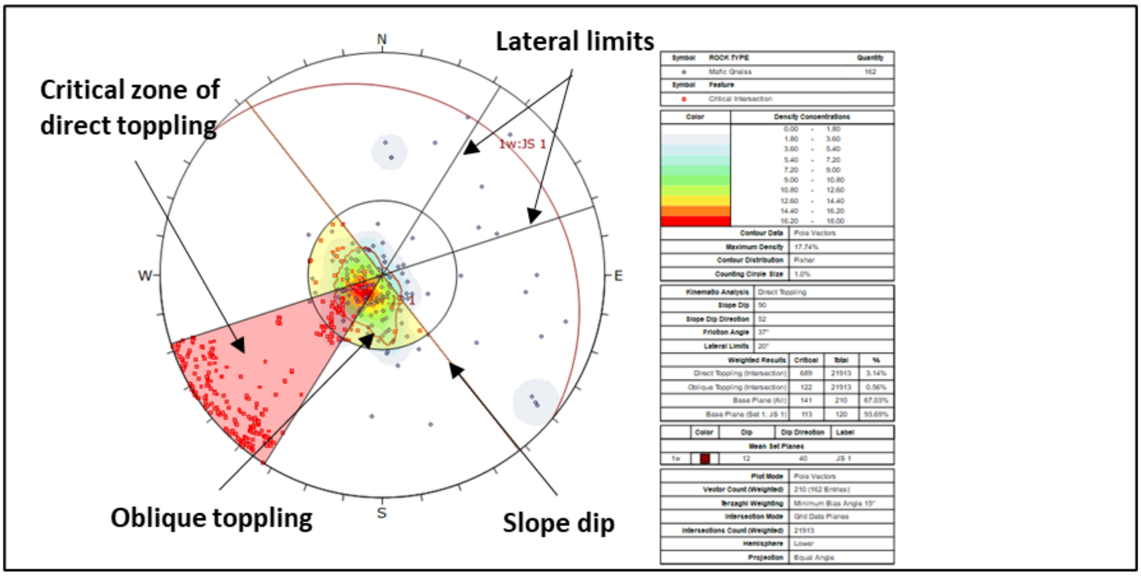


Figure 17-10: Direct failure probability, Omitiomire, DS5, 90˚ BFA.

17.5 Kinematic Results

A kinematic assessment was conducted in Dips to determine the probability or chance of plane, wedge, or toppling instability taking place within the Omitiomire pit. The likelihood of each type of failure was recorded for the different pit wall sectors, utilizing varying bench face angles of 60°, 70°, 80° and 90°. The friction angle utilized in Dips was obtained from the properties listed in Table 13-5 and Table 13-6.

Table 17-2 to Table 17-8 display the results of the assessment. Any scenario where the probability percentage exceeded 10% was shaded red. The Dips kinematic assessment produced the following outcomes:

- Generally, direct or flexural toppling occurs for all rock types, with different probabilities for direct or flexural toppling in the different design sectors.
- Prevalent in most of the design sectors and rock types at most bench face angles is a high likelihood that planar and wedge failure will occur.

- The highest levels of failure occur for the planar and wedge failures in most of the design sectors at most bench face angles.

Table 17-2: Dips kinematic results for banded gneiss.

Banded Gneiss											
Design Sector 1 (226)						Design Sector 2 (266)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	0.00	0.60	0.42	0.00	0.17	60	0.50	0.40	0.54	1.11	0.47
70	0.21	1.26	0.42	0.22	0.38	70	1.41	0.91	0.54	2.65	0.98
80	0.84	5.11	0.42	0.22	1.14	80	1.41	4.20	0.54	4.21	1.31
90	2.92	14.67	0.42	0.52	2.60	90	2.04	11.30	0.54	4.95	2.22
Design Sector 3 (275)						Design Sector 4 (300)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	0.67	0.38	0.56	1.35	0.52	60	1.67	0.39	0.54	1.30	0.72
70	1.78	0.89	0.56	2.55	1.13	70	2.97	1.09	0.54	1.30	1.49
80	1.78	4.32	0.56	3.49	1.45	80	2.97	4.70	0.54	1.62	1.93
90	2.40	11.46	0.56	4.36	1.98	90	3.74	13.24	0.54	2.47	3.06
Design Sector 5 (052)						Design Sector 6 (120)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	1.07	0.20	0.74	2.20	0.75	60	1.17	0.56	0.50	0.77	0.99
70	1.64	0.41	0.74	2.41	1.24	70	1.38	0.76	0.50	2.08	1.53
80	1.64	1.80	0.74	2.41	1.57	80	1.99	1.47	0.50	2.81	2.44
90	1.64	7.86	0.74	2.41	1.97	90	2.47	6.56	0.50	3.74	3.10
Design Sector 7 (080)											
Slope Angle	Failure Mode										
	Planar %	Direct Toppling		Flexural Toppling	Wedge %						
		Direct %	Oblique %	Flexural %							
60	2.60	0.37	0.59	0.96	1.37						
70	4.14	0.50	0.59	1.56	2.51						
80	4.57	1.53	0.59	1.73	3.21						
90	4.81	7.13	0.59	1.91	3.80						

Table 17-3: Dips kinematic results for white gneiss.

White Gneiss											
Design Sector 1 (226)						Design Sector 2 (266)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	0.00	0.50	0.98	0.00	1.40	60	8.52	0.23	0.14	0.00	6.76
70	3.75	1.34	0.98	0.00	4.42	70	11.92	0.37	0.14	0.00	11.34
80	6.25	1.93	0.98	0.00	7.77	80	11.92	4.03	0.14	0.00	14.39
90	6.25	3.29	0.98	0.00	9.94	90	20.93	9.55	0.14	3.49	25.95
Design Sector 3 (275)						Design Sector 4 (300)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	10.15	0.00	0.24	0.00	6.12	60	5.73	0.00	2.03	0.00	3.70
70	11.92	0.08	0.24	0.00	9.45	70	7.37	0.56	2.03	1.99	5.76
80	11.92	5.15	0.24	0.00	12.30	80	7.37	4.43	2.03	1.99	7.21
90	20.93	11.11	0.24	3.49	21.50	90	10.57	10.22	2.03	1.99	12.48
Design Sector 5 (052)						Design Sector 6 (120)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	0.00	1.64	3.05	2.50	0.61	60	0.00	3.37	1.25	3.20	0.07
70	0.00	2.99	3.05	6.25	1.01	70	0.00	3.45	1.25	3.20	0.45
80	0.00	10.34	3.05	6.25	1.19	80	1.99	3.81	1.25	6.33	2.11
90	0.00	15.22	3.05	6.25	1.90	90	1.99	5.86	1.25	9.24	3.13
Design Sector 7 (080)											
Slope Angle	Failure Mode										
	Planar %	Direct Toppling		Flexural Toppling	Wedge %						
		Direct %	Oblique %	Flexural %							
60	3.49	0.64	3.48	9.00	3.77						
70	3.49	0.71	3.48	12.72	4.73						
80	3.49	1.37	3.48	15.68	5.16						
90	3.49	3.77	3.48	18.46	5.42						

Table 17-4: Dips kinematic results for grey gneiss.

Grey Gneiss											
Design Sector 1 (226)						Design Sector 2 (266)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	0.74	2.87	1.82	1.25	2.65	60	3.27	2.20	1.71	2.64	4.50
70	1.65	5.04	1.82	2.06	4.13	70	3.89	3.92	1.71	3.73	6.35
80	2.49	8.91	1.82	3.30	5.91	80	4.42	7.59	1.71	4.92	8.14
90	2.73	15.12	1.82	4.81	7.42	90	5.58	13.61	1.71	6.76	10.11
Design Sector 3 (275)						Design Sector 4 (300)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	2.61	1.75	1.82	2.00	4.03	60	1.53	0.96	1.80	2.79	2.50
70	3.54	3.44	1.82	2.80	5.80	70	2.24	2.53	1.80	2.79	4.50
80	4.63	7.58	1.82	3.57	7.85	80	4.76	7.05	1.80	2.87	7.89
90	5.63	13.40	1.82	5.26	10.30	90	7.06	12.08	1.80	4.02	11.48
Design Sector 5 (052)						Design Sector 6 (120)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	2.80	2.19	1.80	1.24	4.39	60	1.36	1.16	3.05	5.25	2.63
70	3.74	3.12	1.80	2.31	6.88	70	1.36	1.85	3.05	5.64	3.74
80	4.06	5.09	1.80	2.70	8.56	80	2.57	3.10	3.05	6.31	5.86
90	4.39	9.84	1.80	3.15	10.15	90	4.15	6.40	3.05	7.06	8.31
Design Sector 7 (080)											
Slope Angle	Failure Mode										
	Planar %	Direct Toppling		Flexural Toppling	Wedge %						
		Direct %	Oblique %	Flexural %							
60	3.35	2.64	1.51	1.91	5.29						
70	4.75	3.46	1.51	3.11	7.42						
80	5.92	5.02	1.51	4.77	10.03						
90	6.54	8.83	1.51	6.26	11.76						

Table 17-5: Dips kinematic results for pegmatite.

Pegmatite											
Design Sector 1 (226)						Design Sector 2 (266)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	0.00	0.00	6.69	0.00	0.00	60	0.00	12.70	0.00	0.00	0.00
70	0.00	0.00	6.69	0.00	0.00	70	0.00	16.84	0.00	0.00	0.00
80	0.00	0.00	6.69	0.00	0.00	80	0.00	16.84	0.00	0.00	0.00
90	0.00	12.74	6.69	0.00	0.00	90	0.00	21.12	0.00	32.07	0.00
Design Sector 3 (275)						Design Sector 4 (300)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	0.00	0.00	6.69	0.00	0.00	60	0.00	11.39	6.69	0.00	0.00
70	0.00	4.14	6.69	0.00	0.00	70	0.00	24.30	6.69	16.91	0.00
80	0.00	4.14	6.69	0.00	0.00	80	0.00	24.30	6.69	16.91	0.00
90	0.00	8.42	6.69	0.00	0.00	90	0.00	24.30	6.69	16.91	0.00
Design Sector 5 (052)						Design Sector 6 (120)					
Slope Angle	Failure Mode					Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %		Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %				Direct %	Oblique %	Flexural %	
60	15.17	0.00	0.00	0.00	12.70	60	0.00	0.00	0.00	0.00	6.01
70	15.17	0.00	0.00	0.00	22.23	70	0.00	0.00	0.00	0.00	12.70
80	15.17	4.31	0.00	0.00	28.23	80	0.00	0.00	0.00	0.00	12.70
90	15.17	12.51	0.00	0.00	32.85	90	0.00	0.00	0.00	0.00	18.70
Design Sector 7 (080)											
Slope Angle	Failure Mode										
	Planar %	Direct Toppling		Flexural Toppling	Wedge %						
		Direct %	Oblique %	Flexural %							
60	32.07	0.00	0.00	0.00	29.33						
70	32.07	0.00	0.00	0.00	38.45						
80	32.07	4.31	0.00	0.00	38.45						
90	32.07	4.31	0.00	0.00	38.45						

Table 17-6: Dips kinematic results for mafic gneiss.

Mafic Gneiss

Design Sector 1 (226)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	0.55	0.95	0.14	0.00	0.73
70	1.26	3.80	0.14	0.00	2.08
80	2.59	15.25	0.14	0.00	2.94
90	3.86	21.88	0.14	1.77	4.52

Design Sector 2 (266)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %	
60	1.09	0.27	0.23	0.00	1.34
70	2.29	1.38	0.23	0.00	2.39
80	2.29	8.69	0.23	0.00	3.06
90	3.03	20.55	0.23	0.49	3.74

Design Sector 3 (275)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	0.48	0.21	0.23	0.00	0.87
70	1.67	1.38	0.23	0.00	2.71
80	2.35	8.31	0.23	0.00	3.67
90	3.09	18.68	0.23	0.00	4.12

Design Sector 4 (300)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %	
60	0.62	0.36	0.20	0.00	0.83
70	0.62	1.11	0.20	0.00	1.66
80	1.29	7.45	0.20	0.00	3.35
90	4.71	14.81	0.20	0.51	4.93

Design Sector 5 (052)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	2.26	0.95	0.56	1.27	0.76
70	2.26	1.02	0.56	1.98	1.20
80	2.26	1.24	0.56	3.14	1.50
90	2.26	3.14	0.56	3.14	1.90

Design Sector 6 (120)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %	
60	0.51	0.68	0.95	4.09	1.15
70	0.51	1.35	0.95	4.09	1.22
80	0.51	3.53	0.95	4.71	1.33
90	0.51	8.69	0.95	4.71	1.74

Design Sector 7 (080)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	0.49	0.47	0.80	0.61	0.64
70	0.49	0.67	0.80	1.81	1.05
80	0.49	1.48	0.80	2.35	1.40
90	0.49	5.84	0.80	2.84	1.77

Table 17-7: Dips kinematic results for biotite schist.

Biotite Schist

Design Sector 1 (226)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	0.00	0.00	0.00	0.00	0.00
70	7.75	0.00	0.00	0.00	11.20
80	17.59	0.00	0.00	0.00	26.08
90	17.59	3.85	0.00	0.00	30.91

Design Sector 2 (266)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %	
60	5.08	1.85	0.00	0.00	4.12
70	11.49	1.85	0.00	0.00	12.47
80	11.49	2.54	0.00	0.00	15.70
90	11.49	3.23	0.00	18.54	15.70

Design Sector 3 (275)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	5.08	1.85	0.00	0.00	4.82
70	11.49	1.85	0.00	0.00	10.31
80	11.49	4.73	0.00	0.00	12.23
90	11.49	5.42	0.00	9.00	13.02

Design Sector 4 (300)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %	
60	3.56	0.00	0.00	0.00	3.56
70	5.99	1.46	0.00	0.00	5.99
80	5.99	12.75	0.00	0.00	5.99
90	5.99	13.92	0.00	0.00	5.99

Design Sector 5 (052)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	0.00	0.00	2.43	17.59	1.85
70	0.00	0.00	2.43	17.59	1.85
80	0.00	0.00	2.43	17.59	3.31
90	0.00	1.80	2.43	17.59	6.51

Design Sector 6 (120)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %	
60	0.00	6.84	1.07	0.00	1.85
70	0.00	6.84	1.07	0.00	1.85
80	0.00	6.84	1.07	0.00	1.85
90	0.00	7.91	1.07	5.08	1.85

Design Sector 7 (080)

Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	18.54	1.07	1.36	0.00	7.34
70	18.54	1.07	1.36	6.42	9.77
80	18.54	1.07	1.36	6.42	9.77
90	18.54	1.07	1.36	6.42	9.77

Table 17-8: Dips kinematic results for pink gneiss.

Pink Gneiss

Design Sector 1 (226)					
Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	0.71	5.61	9.51	5.58	2.40
70	0.71	6.92	9.51	7.03	3.70
80	0.71	8.69	9.51	8.31	5.95
90	0.71	14.92	9.51	9.58	8.84

Design Sector 2 (266)					
Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %	
60	0.44	4.08	9.09	7.49	2.77
70	1.35	5.57	9.09	8.61	6.04
80	1.35	8.87	9.09	10.02	7.87
90	5.26	12.06	9.09	10.65	14.13

Design Sector 3 (275)					
Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	1.25	3.86	7.70	6.37	3.16
70	2.77	5.47	7.70	7.06	5.87
80	4.43	9.94	7.70	8.85	8.83
90	8.83	13.18	7.70	9.48	15.85

Design Sector 4 (300)					
Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %	
60	3.03	3.67	6.40	5.38	5.62
70	3.63	5.07	6.40	5.38	8.57
80	5.74	9.75	6.40	6.26	13.33
90	7.41	13.21	6.40	7.69	18.81

Design Sector 5 (052)					
Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	3.87	1.88	3.84	0.00	7.44
70	5.83	3.22	3.84	0.00	12.88
80	7.50	5.04	3.84	0.00	18.86
90	10.54	8.39	3.84	0.71	24.79

Design Sector 6 (120)					
Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge
		Direct %	Oblique %	Flexural %	
60	2.94	3.27	7.84	3.78	5.55
70	2.94	3.88	7.84	4.90	7.60
80	4.64	5.50	7.84	6.17	12.35
90	8.32	8.47	7.84	7.41	17.95

Design Sector 7 (080)					
Slope Angle	Failure Mode				
	Planar %	Direct Toppling		Flexural Toppling	Wedge %
		Direct %	Oblique %	Flexural %	
60	2.30	1.12	4.98	3.90	5.91
70	4.43	1.72	4.98	5.26	10.40
80	7.03	3.15	4.98	5.26	16.58
90	11.93	5.81	4.98	5.26	24.99

The probability of wedge, plane, and toppling failure occurring was determined as a pseudo-probabilistic method using the Dips analysis assessment. If the probability of wedge or plane failure exceeded 10%, further analysis was conducted using RocScience's Swedge or RocPlane programs, respectively. Swedge calculates the factor of safety for a bench face of the wedge failure by considering the discontinuity properties, bench face angle inclination, and pit wall orientation. RocPlane calculates the safety factor for each plane failure scenario, providing more certainty on whether a failure is expected. However, the toppling failures were not assessed in detail as large-scale failure volume for toppling is not expected. Instead, the berm widths of each bench were designed to account for any small-scale failures.

17.5.1 Plane and Wedge Failure Results

A detailed assessment of plane and wedge failure was performed using RocPlane and Swedge to determine the potential for planes or wedges to mobilize after exposure. A safety factor of 1.3 was applied to the analysis, and all planes and wedges with a safety factor higher than 1.3 were considered stable. The results for RocPlane for Omitiomire are presented in Table 17-9, and the results for Swedge are shown in

Table 17-10 to

Table 17-15. The result table indicates "np" where no plane has formed, and "nw" where no wedge is formed between the discontinuities. All RocPlane and Swedge results are available in **Error! Reference source not found.**. Plane failures were mainly observed at a bench face angle of 90 degrees and wedge failure only in design sector 7, just below the safety factor at 90 degrees. Based on the kinematic results, a bench face angle of 90 degrees was selected for use on all fresh material benches for the Omitiomire pit.

Table 17-9: RocPlane kinematic results, Omitiomire.

15m Bench											
Pink Gneiss						White Gneiss					
15m bench		Bench Face Angles (°)				15m bench		Bench Face Angles (°)			
Joint set	Joint set dip (°)	60	70	80	90	Joint set	Joint set dip (°)	60	70	80	90
JS1	8	5.87	5.85	5.83	5.81	JS1	17	3.20	3.17	3.17	3.12
JS2	66	np	np	0.72	0.56	JS2	86	np	np	np	0.00
Banded Gneiss						Biotite Schist					
15m bench		Bench Face Angles (°)				15m bench		Bench Face Angles (°)			
Joint set	Joint set dip (°)	60	70	80	90	Joint set	Joint set dip (°)	60	70	80	90
JS1	20	1.23	1.18	1.15	1.12	JS1	25	1.89	1.85	1.82	1.80
JS2	55	1.70	0.74	0.54	0.45	JS2	71	np	np	0.86	0.55
Grey Gneiss											
15m bench		Bench Face Angles (°)				Pegmatite					
Joint set	Joint set dip (°)	60	70	80	90	15m bench		Bench Face Angles (°)			
JS1	20	2.63	2.33	2.31	2.29	Joint set	Joint set dip (°)	60	70	80	90
JS2	47	1.17	0.98	0.91	0.86	JS1	49	1.52	1.21	1.09	1.03

Table 17-10: Swedge kinematic results white gneiss.

DS 1, Wall direction 226, Bench height 15m					DS 2, Wall direction 266, Bench height 15m				
White Gneiss					White Gneiss				
Joint set	60°	70°	80°	90°	Joint set	60°	70°	80°	90°
JS1+JS2	2.69	2.66	2.63	2.62	JS1+JS2	nw	nw	2.09	2.22
Design Sector 3 (275)					Design Sector 4 (300)				
White Gneiss					White Gneiss				
Joint set	60°	70°	80°	90°	Joint set	60°	70°	80°	90°
JS1+JS2	nw	nw	nw	nw	JS1+JS2	nw	nw	nw	nw
Design Sector 5 (052)					Design Sector 6 (120)				
White Gneiss					White Gneiss				
Joint set	60°	70°	80°	90°	Joint set	60°	70°	80°	90°
JS1+JS2	nw	nw	nw	nw	JS1+JS2	2.03	2.07	2.09	2.11
Design Sector 7 (080)									
White Gneiss									
Joint set	60°	70°	80°	90°					
JS1+JS2	1.08	1.21	1.25	1.28					

Table 17-11: Swedge kinematic results mafic gneiss.

DS 1, Wall direction 226, Bench height 15m					DS 1, Wall direction 266, Bench height 15m				
Mafic Gneiss					Mafic Gneiss				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1	nw	nw	nw	nw	JS1	nw	nw	nw	nw
Design Sector 3 (275)					**Design Sector 4 (300)**				
Mafic Gneiss					**Mafic Gneiss**				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1	nw	nw	nw	nw	JS1	nw	nw	nw	nw
Design Sector 5 (052)					**Design Sector 6 (120)**				
Mafic Gneiss					**Mafic Gneiss**				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1	nw	nw	nw	nw	JS1	nw	nw	nw	nw
Design Sector 7 (080)									
Mafic Gneiss									
Joint set	**60°**	**70°**	**80°**	**90°**					
JS1	nw	nw	nw	nw					

Table 17-12: Swedge kinematic results grey gneiss.

DS 1, Wall direction 226, Bench height 15m					DS 1, Wall direction 266, Bench height 15m				
Grey Gneiss					**Grey Gneiss**				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1+JS2	nw	nw	nw	nw	JS1+JS2	nw	nw	nw	nw
Design Sector 3 Wall direction 275, Bench height 15m					**Design Sector 4 (300)**				
Grey Gneiss					**Grey Gneiss**				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1+JS2	54.62	54.66	54.69	54.72	JS1+JS2	56.36	56.36	56.37	56.38
Design Sector 5 (052)					**Design Sector 6 (120)**				
Grey Gneiss					**Grey Gneiss**				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1+JS2	59.38	59.36	52.34	59.32	JS1+JS2	nw	nw	nw	nw
Design Sector 7 (080)									
Grey Gneiss									
Joint set	**60°**	**70°**	**80°**	**90°**					
JS1+JS2	58.32	58.25	58.20	58.15					

Table 17-13: Swedge kinematic results pink gneiss.

DS 1, Wall direction 226, Bench height 15m					DS 1, Wall direction 266, Bench height 15m				
Pink Gneiss					Pink Gneiss				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1+JS2	nw	nw	nw	nw	JS1+JS2	nw	nw	nw	nw
Design Sector 3 (275)					**Design Sector 4 (300)**				
Pink Gneiss					**Pink Gneiss**				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1+JS2	nw	nw	nw	nw	JS1+JS2	nw	nw	nw	nw
Design Sector 5 (052)					**Design Sector 6 (120)**				
Pink Gneiss					**Pink Gneiss**				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1+JS2	5.82	5.83	5.83	5.84	JS1+JS2	nw	nw	nw	nw
Design Sector 7 (080)									
Pink Gneiss									
Joint set	**60°**	**70°**	**80°**	**90°**					
JS1+JS2	5.83	5.84	5.84	5.84					

Table 17-14: Swedge kinematic results banded gneiss.

DS 1, Wall direction 226, Bench height 15m					DS 1, Wall direction 266, Bench height 15m				
Banded Gneiss					Banded Gneiss				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1+JS2	8.27	8.29	8.31	8.32	JS1+JS2	nw	nw	nw	nw
Design Sector 3 (275)					**Design Sector 4 (300)**				
Banded Gneiss					**Banded Gneiss**				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1+JS2	nw	nw	nw	nw	JS1+JS2	nw	nw	nw	nw
Design Sector 5 (052)					**Design Sector 6 (120)**				
Banded Gneiss					**Banded Gneiss**				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1+JS2	nw	nw	nw	nw	JS1+JS2	8.39	8.40	8.41	8.41
Design Sector 7 (080)									
Banded Gneiss									
Joint set	**60°**	**70°**	**80°**	**90°**					
JS1+JS2	1.08	1.21	1.25	1.28					

Table 17-15: Swedge kinematic results pegmatite.

Pegmatite					Pegmatite				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1	nw	nw	nw	nw	JS1	nw	nw	nw	nw
Design Sector 3 (275)					**Design Sector 4 (300)**				
Pegmatite					**Pegmatite**				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1	nw	nw	nw	nw	JS1	nw	nw	nw	nw
Design Sector 5 (052)					**Design Sector 6 (120)**				
Pegmatite					**Pegmatite**				
Joint set	**60°**	**70°**	**80°**	**90°**	**Joint set**	**60°**	**70°**	**80°**	**90°**
JS1	nw	nw	nw	nw	JS1	nw	nw	nw	nw
Design Sector 7 (080)									
Pegmatite									
Joint set	**60°**	**70°**	**80°**	**90°**					
JS1	nw	nw	nw	nw					

18. BENCH HEIGHTS

The bench heights used for the slope design for the Omitiomire pit were 10m, for weathered and 15m for the fresh material.

19. BERM WIDTH

The evaluation of bench widths required for the Omitiomire pit was performed through the analysis of failure volume containment within a bench using the Slide, plane, and wedge failure assessments. The weight of the failed material, as determined by the Slide, RocPlane, and Swedge analyses, was utilized to establish the minimum necessary berm width. The failure volume was calculated by dividing the weight by the density and multiplying it by an appropriate bulking factor. The calculation of the required berm width was accomplished by obtaining the cube root of the failure volume, as shown in the equation below:

$$\boldsymbol{Berm\ width} = \sqrt[3]{(\boldsymbol{failure\ volume\ x\ bulking\ factor}}$$

For the berm width analysis, the bulking factor selected for the Omitiomire weathered rock mass was determined by averaging the factors of clay and gravel, and sand, which was 1.20, highlighted in orange in the table below. The fresh material bulking factor was 1.68, which comprised the average values of basalt and granite in red shading in Table 19-1. This selection is based on the combination of material types best suited to depict the metamorphic nature of the host rock mass.

Table 19-1: Bulking factor selected for failure volume calculations.

Bulking factors				
Material	Bulk Density Mg/m^3	Bulking Factor	Shrinkage Factor	Diggability
Clay (Low PI)	1.65	1.30	-	**M**
Clay (High PI)	2.10	1.40	0.90	**M-H**
Clay and Gravel	1.80	1.35	-	**M-H**
Sand	2.00	1.05	0.89	**E**
Sand & Gravel	1.95	1.15	-	**E**

Gravel	2.10	1.05	0.97	**E**
Chalk	1.85	1.50	0.97	**E**
Shales	2.35	1.50	1.33	**M-H**
Limestone	2.60	1.63	1.36	**M-H**
Sandstone (Porous)	2.50	1.60	-	**M**
Sandstone (cemented)	2.65	1.61	1.34	**M-H**
Basalt	2.95	1.64	1.36	**H**
Granite	2.41	1.72	1.33	**H**
E - Easy digging, M - Medium diggability, H - Hard diggability				

19.1 Berm Width from Circular Failure Volume

Table 19-2 presents the results of the berm width calculation from the Slide analyses. The failure volume is divided into several slices, where the weight of each slice is obtained and used to determine the total weight of the failure for berm width calculations.

The analysis determined a limiting berm width of 3.66m for a 10.00m bench height in the weathered material, which was increased to 4.00m in the slope model, taking into consideration practicality for mining.

Table 19-2: Slide weathered material failure volume analysis, 10m bench.

Slice	70° BFA
	Slice weight (kN)
1	26.79
2	37.45
3	42.76
4	46.87
5	50.27
6	53.20
7	55.77
8	58.04
9	60.08
10	61.90
11	63.55
12	65.04
13	65.75
14	61.89

15	57.04
16	52.07
17	47.00
18	41.82
19	36.54
20	31.16
21	25.69
22	20.13
23	14.49
24	8.75
25	2.93
Sum	1086.98
Mass	110724.11
Volume	48.85
Bulking factor	1.20
Berm width	3.66
Design berm width	4.00

19.2 Berm Width from Plane Failure Volume

Table 19-3 to Table 19-10 summarize the failed volumes obtained from the plane failure analysis for the Omitiomire pit. Based on the results of the limiting berm width assessment presented in Table 19-8 and Table 19-9, a maximum berm width of 8.0m was determined for a 90-degree bench face angle in anticipation of potential failure in the rock types banded gneiss and grey gneiss. This outcome is consistent with the shallow-dipping orientation of the major joint sets in these rock types. The limiting berm width of 8.00m was applied to all design sectors of the Omitiomire pit.

Table 19-3: Calculated berm widths from plane failure – pink gneiss.

All design sectors (66° joint dip) Pink Gneiss				
BFA (°)	Failure volume (m^3)	Bulking factor	Final volume (m^3)	Berm width (m)
80	30.25	1.68	50.82	3.70
90	50.09	1.68	84.15	4.38

Table 19-4: Calculated berm widths from plane failure – white gneiss

All design sectors (86° joint dip) White Gneiss				
BFA (°)	Failure volume (m^3)	Bulking factor	Final volume (m^3)	Berm width (m)
90	7.87	1.68	13.22	2.36

Table 19-5: Calculated berm widths from plane failure - pegmatite

All design sectors (49° joint dip) Pegmatite				
BFA (°)	Failure volume (m^3)	Bulking factor	Final volume (m^3)	Berm width (m)
70	56.85	1.68	95.50	4.57
80	77.96	1.68	130.97	5.08

90	97.79	1.68	164.30	5.48

Table 19-6: Calculated berm widths from plane failure – biotite schist

All design sectors (71° joint dip) Biotite Schist				
BFA (°)	Failure volume (m^3)	Bulking factor	Final volume (m^3)	Berm width (m)
80	18.90	1.68	31.75	3.17
90	38.74	1.68	65.08	4.02

Table 19-7: Calculated berm widths from plane failure banded gneiss

All design sectors (55° joint dip) Banded Gneiss				
BFA (°)	Failure volume (m^3)	Bulking factor	Final volume (m^3)	Berm width (m)
70	37.83	1.68	63.55	3.99
80	58.94	1.68	99.02	4.63
90	78.77	1.68	132.34	5.10

Table 19-8: Calculated berm widths from plane failure – banded gneiss

All design sectors (20° joint dip) Banded Gneiss				
BFA (°)	Failure volume (m^3)	Bulking factor	Final volume (m^3)	Berm width (m)
70	37.83	1.68	63.55	3.99
80	58.94	1.68	99.02	4.63
90	309.09	1.68	519.27	8.04

Table 19-9: Calculated berm widths from plane failure – grey gneiss

All design sectors (20° joint dip) Grey Gneiss				
BFA (°)	Failure volume (m^3)	Bulking factor	Final volume (m^3)	Berm width (m)
60	244.14	1.68	410.15	7.43
70	268.15	1.68	450.48	7.67
80	289.25	1.68	485.95	7.86
90	309.09	1.68	519.27	8.04

Table 19-10: Calculated berm widths from plane failure – grey gneiss

All design sectors (47° joint dip) Grey Gneiss				
BFA (°)	Failure volume (m^3)	Bulking factor	Final volume (m^3)	Berm width (m)
60	39.96	1.68	67.13	4.06
70	63.96	1.68	107.45	4.75
80	85.07	1.68	142.92	5.23
90	104.91	1.68	176.25	5.61

19.3 Berm Width from Wedge Failure Volume

The wedge failure volume analysis was performed on the fresh rock in the Omitiomire pit, and the results are presented in Table 19-11, where the volumes of failed wedges are summarized in design sector 7, which had safety factors of less than 1.3. The analysis showed that wedge failure volumes required very large berm widths of 34.48 m at a 90-degree bench face angle, However, the safety factor for this scenario was 1.28, which is just below the threshold safety factor of 1.3 and can be deemed as stable.

The results for the 80-degree bench face indicated a 29.46 m berm width. However, the safety factor was stable, but the 80-degree bench face angle was applied to design sectors 5, 6 and 7, which are the footwall sectors of the Omitiomire pit. The limiting berm width of 8.00m suggested by the plane failure analysis was applied.

Table 19-11: Calculated berm widths from wedge failure – white gneiss design sector 7

White Gneiss				
DS 7 (080), Bench height 15m				
Joint Set	BFA (°)	Failure volume (m^3)	Bulking factor	Berm width (m)
JS1+JS2	60	2373.36	1.68	15.86
JS1+JS2	70	7822.29	1.68	23.60
JS1+JS2	80	15222.30	1.68	29.46
JS1+JS2	90	24400.56	1.68	34.48

19.4 Berm Summary

Table 19-12 below presents a summary of the berm widths applied to the Omitiomire open pit slope design. To ensure stability, a geotechnical berm, typically twice the berm width was placed at the base of every stack or the base of a change in material type. For instance, a geotechnical berm was placed at the base of the weathered material before the fresh material benches. A stack is generally made up of 4 to 6 benches, and the stack height used for the Omitiomire pit was 5 benches, equivalent to every 75.00m in vertical height.

Table 19-12: Omitiomire pit berm width summary.

Berm widths based on bench material		
Rock type	Berm width (m)	Geotech Berm (m)
Weathered 10m bench	4.0	8.0
Fresh 15m benches	8.0	16.0
*Geotechnical berms placed every 5 benches in fresh material, and at the base of weathered and fresh material.		

20. SLOPE STABILITY

The bench face angles and berm widths obtained from the pit design analysis were used to create an overall slope configuration for each design sector within the Omitiomire pit. To ensure slope stability, the overall slope was tested using the software Slide. A safety factor of 1.3 is considered the minimum threshold safety factor. The slope stability was mainly influenced by the kinematic interaction of the anisotropy of the foliations, jointing and wall orientations in the fresh material. Therefore, the safety factors derived from Slide were not considered as an underestimated slope optimization, but merely a check of the overall slope stability.

The kinematic assessment conducted for the Omitiomire pit indicated that a distinct difference exists between different design sectors namely the footwall and hanging wall design sectors. This was because the shallow dipping joint sets are unfavourably oriented in the footwall slopes. The Slide analyses per design sector differed in overall geology, pit depth and water table depth. The results from the Slide analysis for the Omitiomire pit are displayed in Figure 20-1 to Figure 20-7, with the safety factors listed in Table 20-1.

Safety factors in four (4) out of the seven (7) design sectors are well above the threshold of 1.3, as they range from 2.35 to 2.72. Design sectors 1, 2 and 7 have safety factors of 1.44, 1.29 and 1.28 respectively. The overall depth of 370m in these sectors has a greater influence on the slope stability than the geology, as design sectors 1 and 2 are in the hanging wall of the pit, while design sector 7 is in the foot wall, where anisotropy was applied.

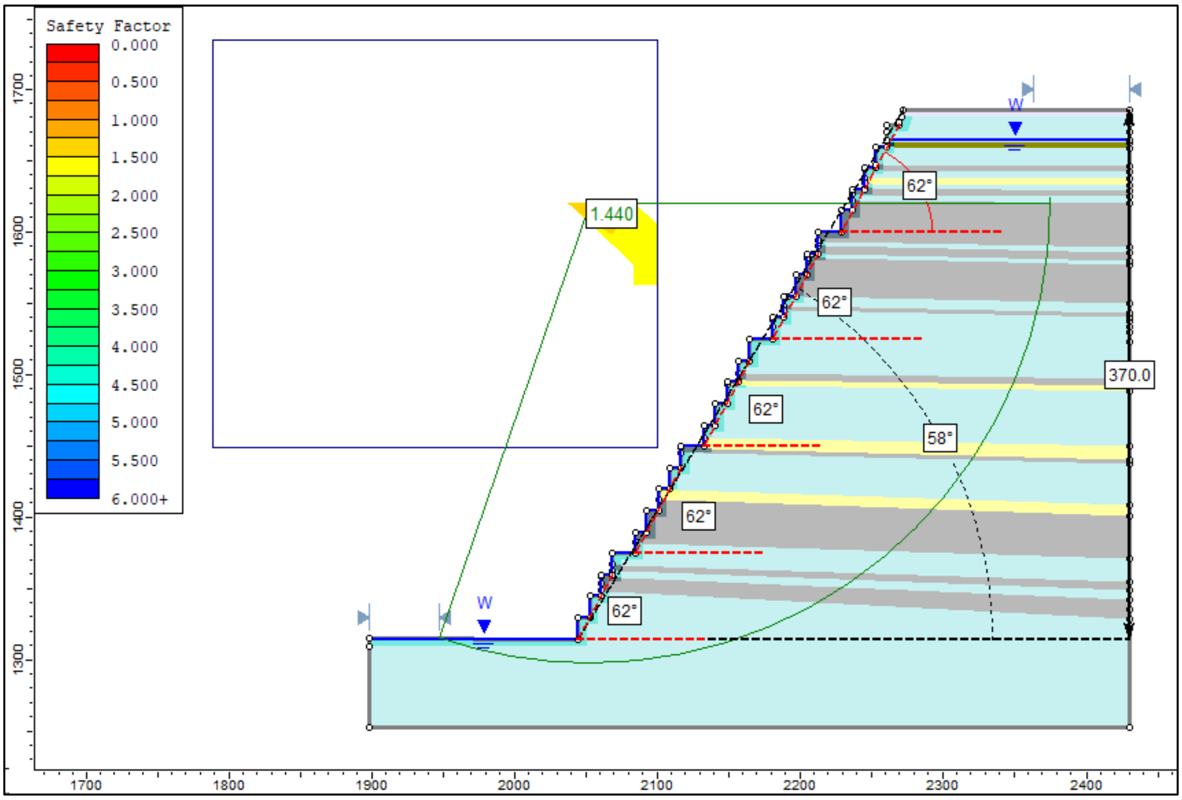


Figure 20-1: Slide analysis – Omitiomire design sector 1, HW.

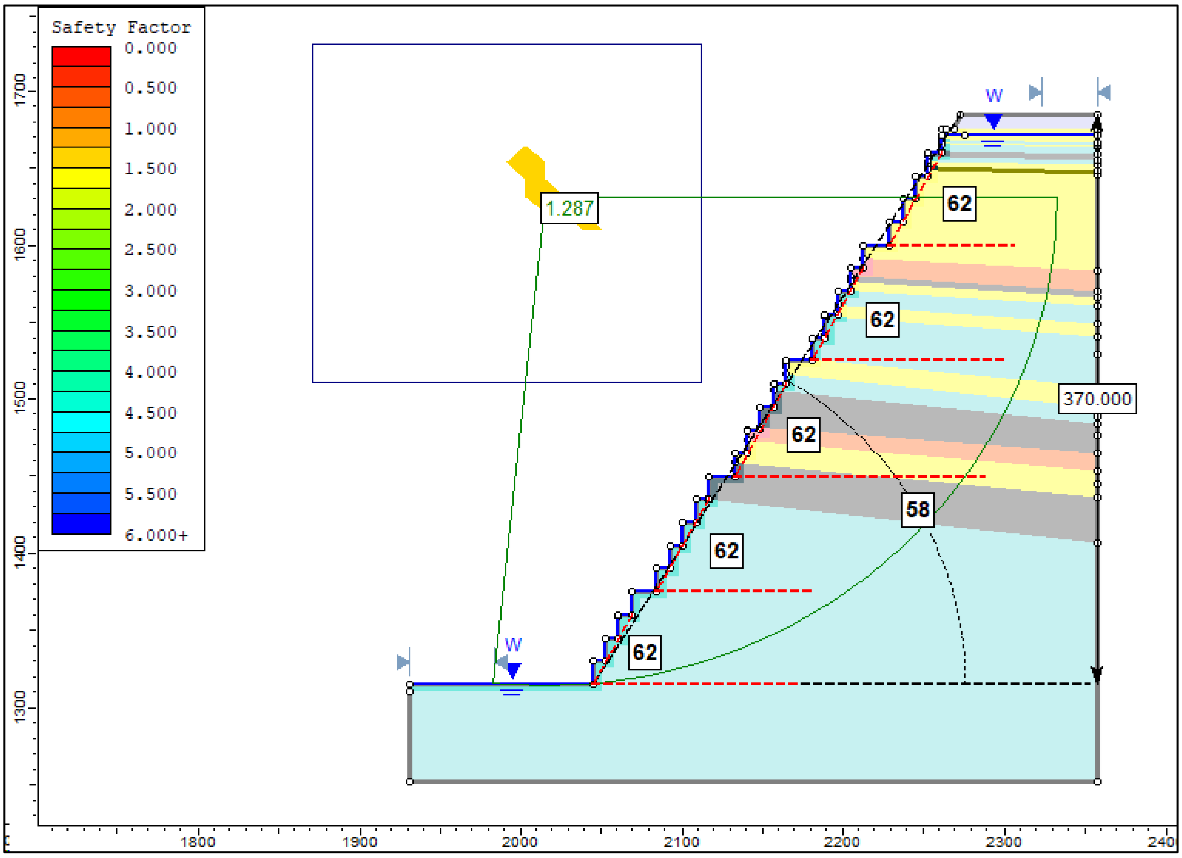


Figure 20-2: Slide analysis – Omitiomire design sector 2, HW.

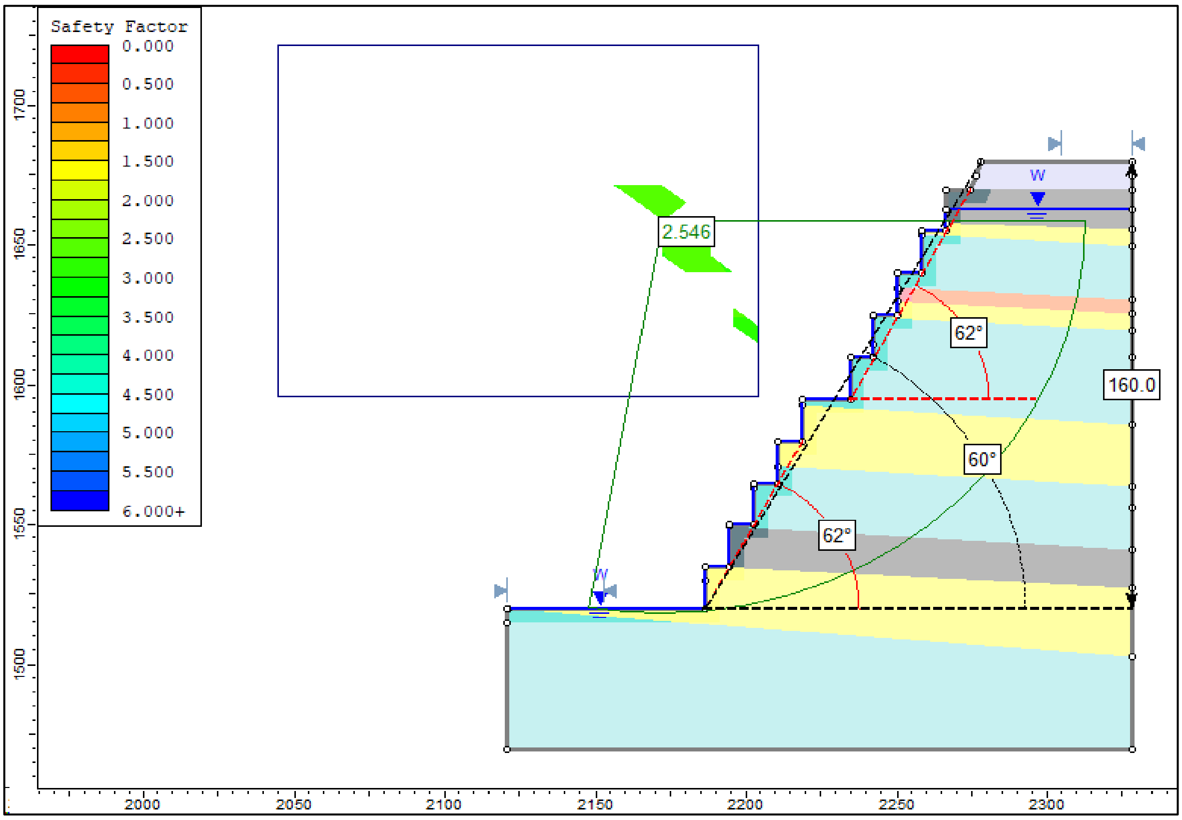


Figure 20-3 Slide analysis – Omitiomire design sector 3, HW.

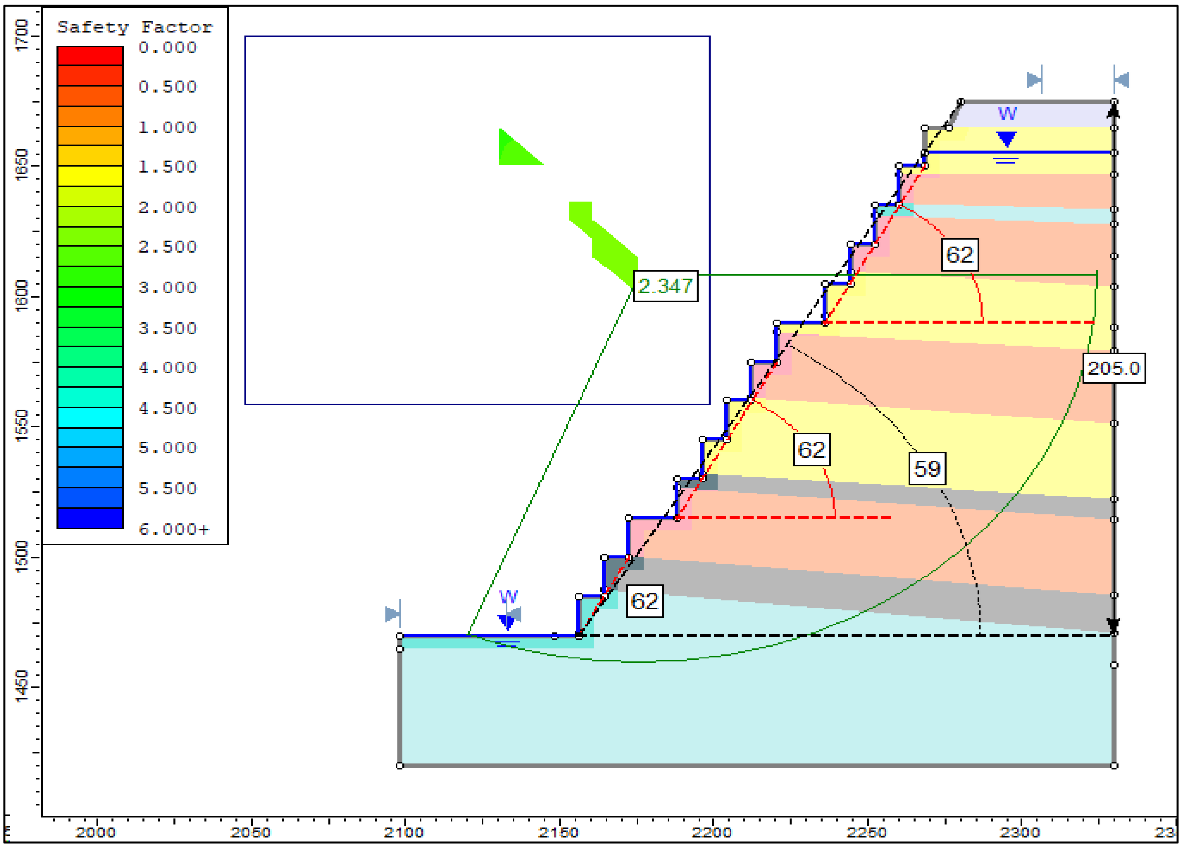


Figure 20-4: Slide analysis - Omitiomire design sector 4, HW.

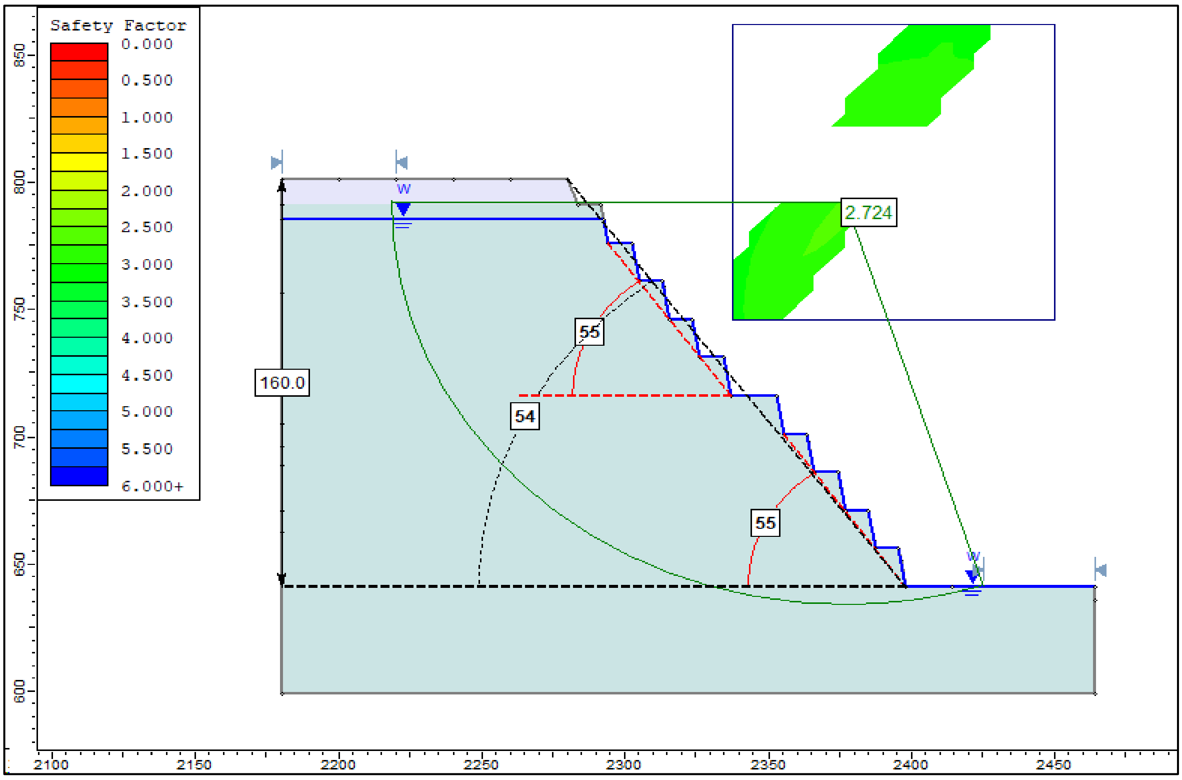


Figure 20-5: Slide analysis – Omitiomire design sector 5, FW.

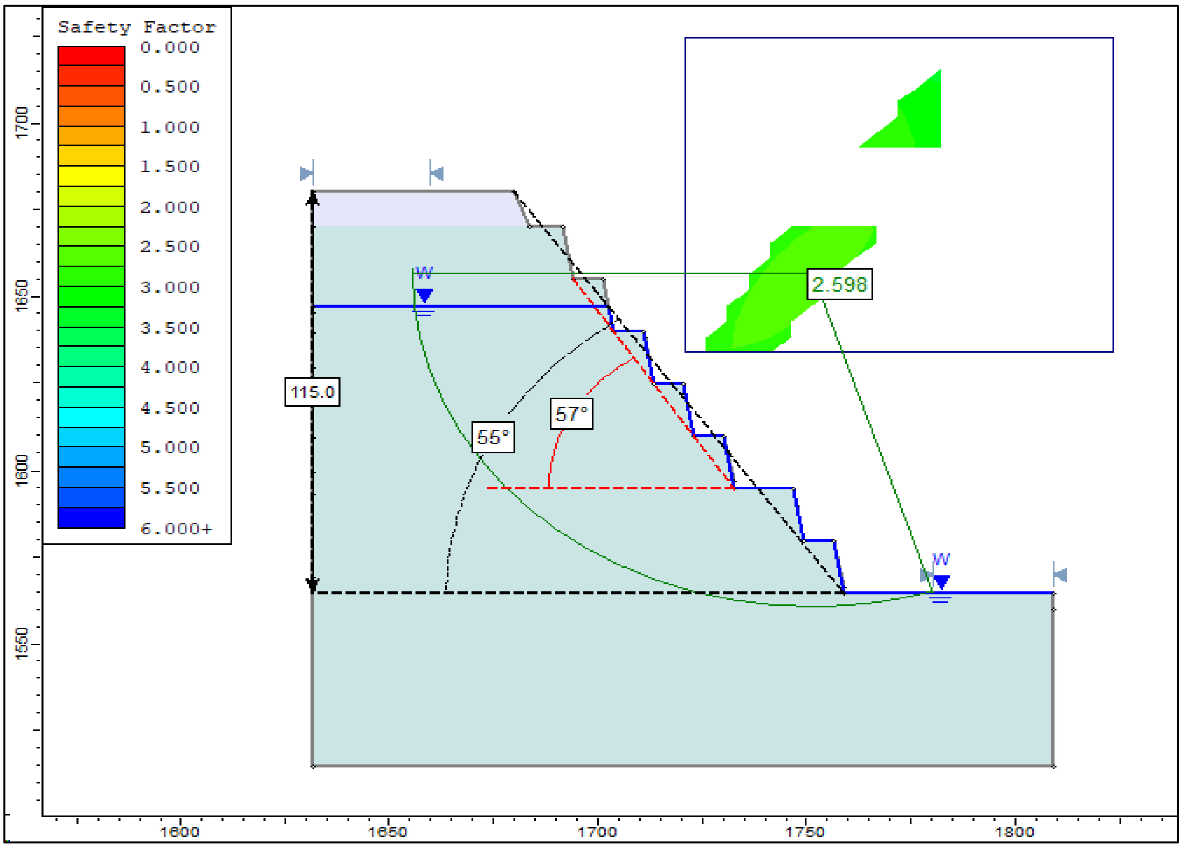


Figure 20-6: Slide analysis – Omitiomire design sector 6, FW.

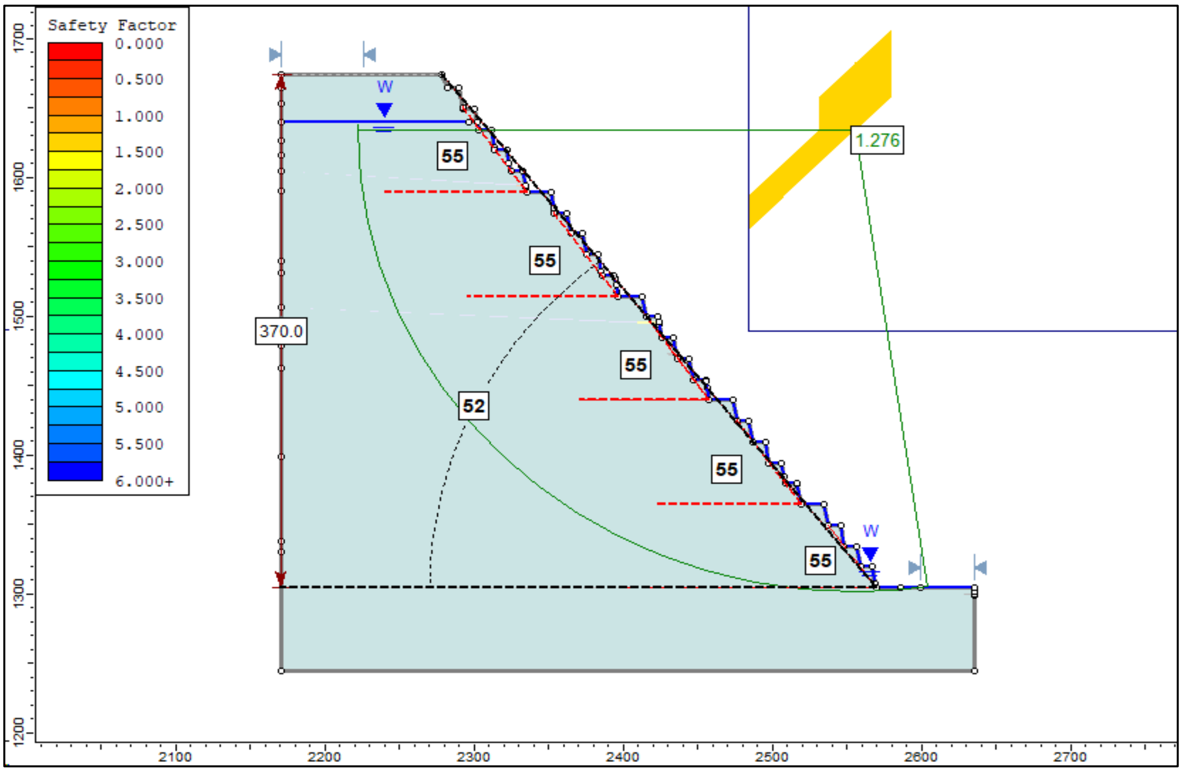


Figure 20-7: Slide analysis – Omitiomire design sector 7, FW.

Table 20-1: Slide safety factor results for the Omitiomire.

Slope Stability	
Design Sector	**Safety Factor**
DS 1	1.44
DS 2	1.29
DS 3	2.56
DS 4	2.35
DS 5	2.72
DS 6	2.60
DS 7	1.28

The slope stability results were presented using the deterministic analysis. The components of the Slide models have been outlined in Figure 20-8. The materials in each slope were separated into the following sections:

- Weathered material,
- Fresh material, with a D factor for blast damage in the first 5 m of the slope,
- Fresh material, with no effect from blasting

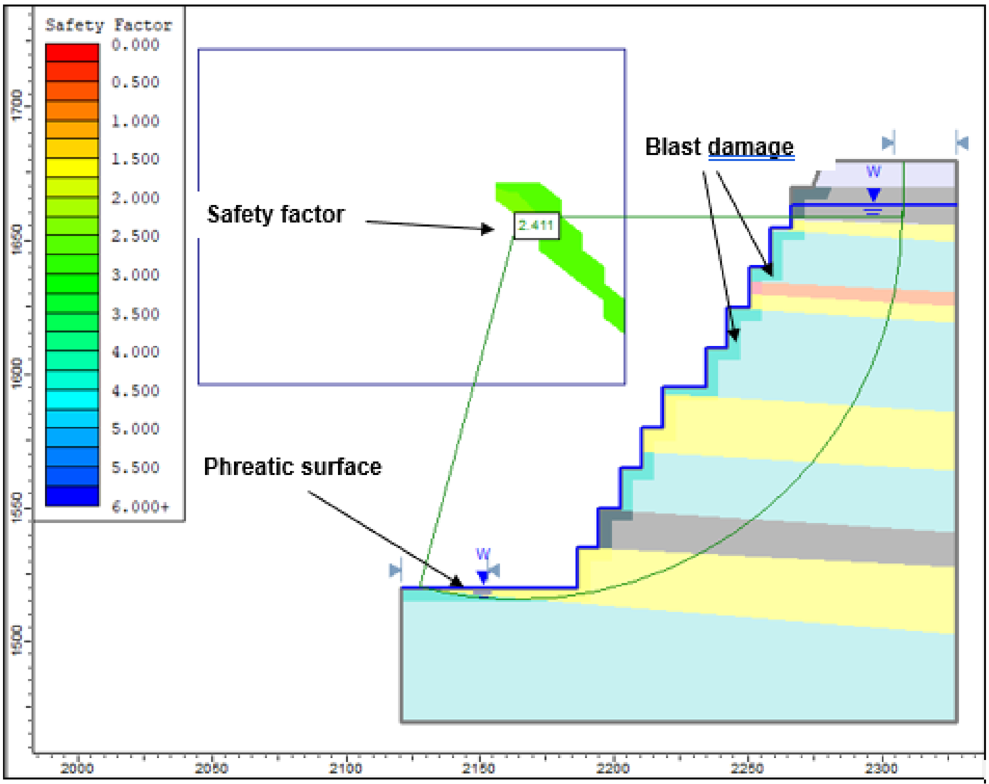


Figure 20-8: Materials and components in the Slide models.

The Slide program displays the material as solid colours when applying different material layers, however, to account for the foliations feature of the rock mass, the dip and direction of the foliations have been modelled by adding "anisotropy" to design sectors 5, 6 and 7 models. This implies that in reality, the monotone colour appearance of the weathered and fresh material has included the pervasive foliation feature characteristic of the rock mass, which is demonstrated in Figure 20-9 and Figure 20-10.

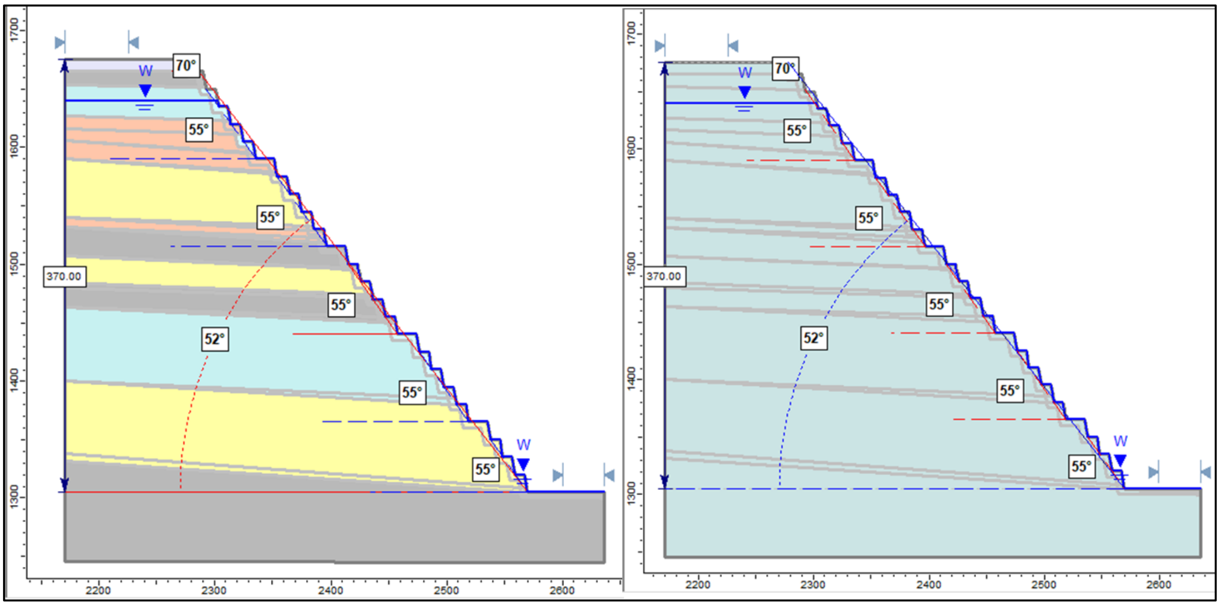


Figure 20-9: Illustration of anisotropy application in the model.

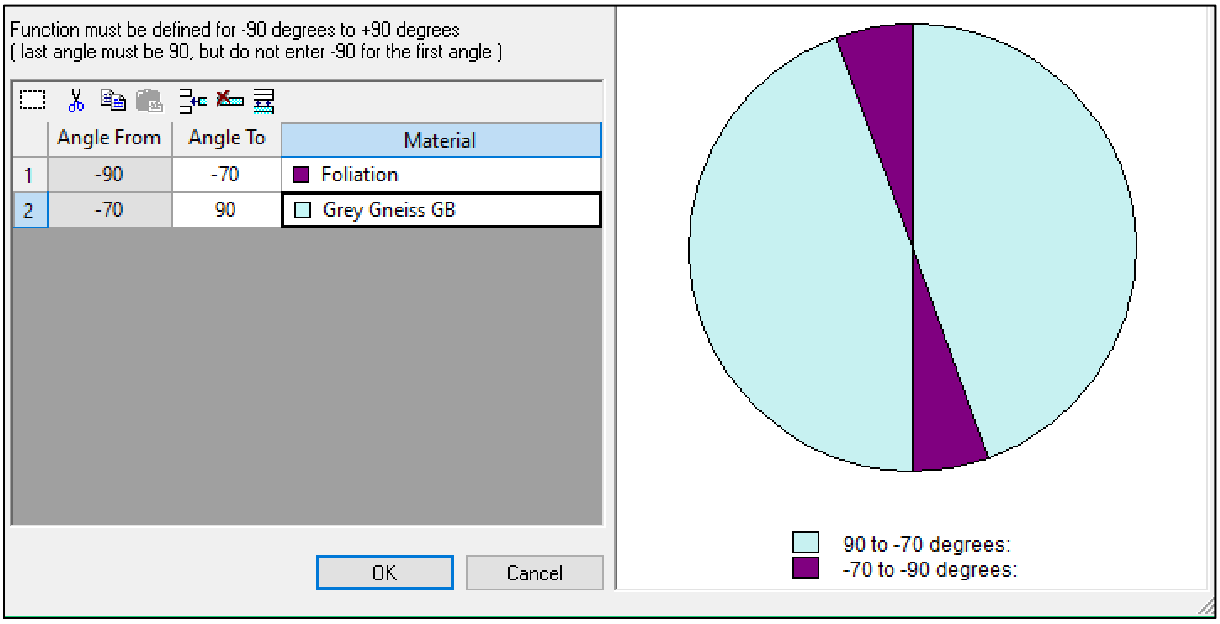


Figure 20-10: Example of anisotropy feature in Slide.

21. SLOPE CONFIGURATIONS

The final overall slope configurations for the Omitiomire pit for all design sectors are presented in Table 21-1 to

Table 21-7 and are illustrated in Figure 21-1 to Figure 21-7.

The shallowest overall slope angle (OSA) was 52° in design sector 7. The footwall design sectors have the shallowest overall slope angle. These slopes were all designed to a depth of 160m, 115m, and 370m for design sectors 5, design sector 6 and design sector 7 respectively. The steepest OSA is 60° in design sector 3 of Omitiomire. The end walls have been designed to extend to a maximum depth of 370m for all design sectors

Table 21-1: Slope configuration design sector 1.

HANGING WALL SLOPES								
Design Sector 1								
Highwall Material	**Bench height (m)**	**Berm width (m)**	**Geotechnical berm width (m)**	**Position of geotech berm elevation**	**Number of benches**	**Bench Face Angle**	**Stack Angle**	**Overall slope angle**
Weathered	10.00	4.00	8.00	1675	1.00	70	**na**	58
Fresh	15.00	8.00	16.00	1600, 1525, 1450, 1375	24.00	90	**62**	

Table 21-2: Slope configuration design sector 2.

Design Sector 2								
Design Sector Material	**Bench height (m)**	**Berm width (m)**	**Geotechnical berm width (m)**	**Position of geotech berm**	**Number of benches**	**Bench Face Angle**	**Stack Angle**	**Overall slope angle**
Weathered	10.00	4.00	8.00	1670	1.00	70	**na**	58
Fresh	15.00	8.00	16.00	1595, 1520, 1445, 1370	24.00	90	**62**	

Table 21-3: Slope configuration design sector 3.

Design Sector 3								
Highwall Material	**Bench height (m)**	**Berm width (m)**	**Geotechnical berm width (m)**	**Position of geotech berm**	**Number of benches**	**Bench Face Angle**	**Stack Angle**	**Overall slope angle**
Weathered	10.00	4.00	8.00	1670	1.00	70	**na**	60
Fresh	15.00	8.00	16.00	1595	10.00	90	**62**	

Table 21-4: Slope configuration design sector 4.

Design Sector 4								
Design Sector Material	**Bench height (m)**	**Berm width (m)**	**Geotechnical berm width (m)**	**Position of geotech berm**	**Number of benches**	**Bench Face Angle**	**Stack Angle**	**Overall slope angle**
Weathered	10.00	4.00	8.00	1670	1.00	70	**na**	59
Fresh	15.00	8.00	16.00	1595, 1520	13.00	90	**62**	

Table 21-5: Slope configuration design sector 5.

FOOT WALL SLOPES								
Design Sector 5								
Highwall Material	**Bench height (m)**	**Berm width (m)**	**Geotechnical berm width (m)**	**Position of geotech berm**	**Number of benches**	**Bench Face Angle**	**Stack Angle**	**Overall slope angle**
Weathered	10.00	4.00	8.00	1670	1.00	70	**na**	54
Fresh	15.00	8.00	16.00	1580	10.00	80	**55**	

Table 21-6: Slope configuration design sector 6.

Design Sector 6								
Highwall Material	**Bench height (m)**	**Berm width (m)**	**Geotechnical berm width (m)**	**Position of geotech berm**	**Number of benches**	**Bench Face Angle**	**Stack Angle**	**Overall slope angle**
Weathered	10.00	4.00	8.00	1675	1.00	70	**na**	54
Fresh	15.00	8.00	16.00	1600.00	10.00	80	**55**	

Table 21-7: Slope configuration design sector 7.

Design Sector 7								
Highwall Material	**Bench height (m)**	**Berm width (m)**	**Geotechnical berm width (m)**	**Position of geotech berm**	**Number of benches**	**Bench Face Angle**	**Stack Angle**	**Overall slope angle**
Weathered	10.00	4.00	8.00	1675	1.00	70	**na**	52
Fresh	15.00	8.00	16.00	1600, 1525, 1450, 1375	24.00	80	**55**	

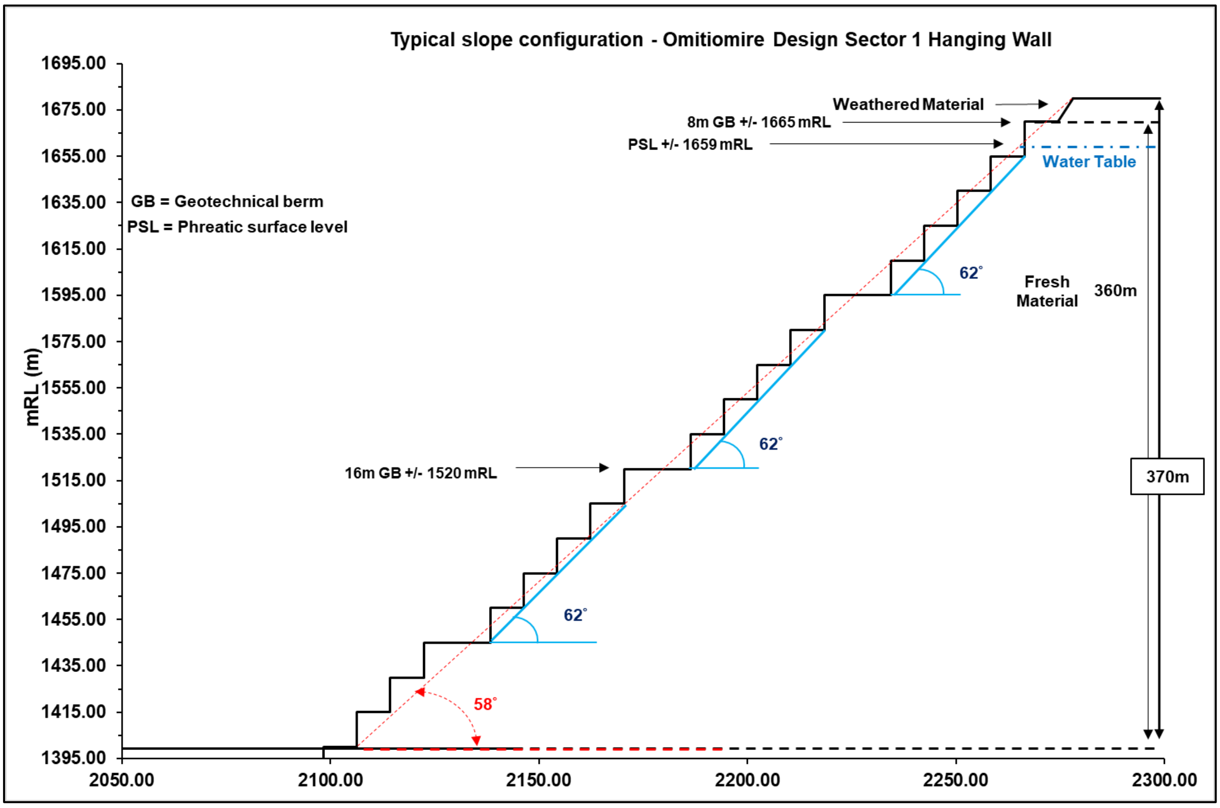


Figure 21-1: Slope configuration design sector 1.

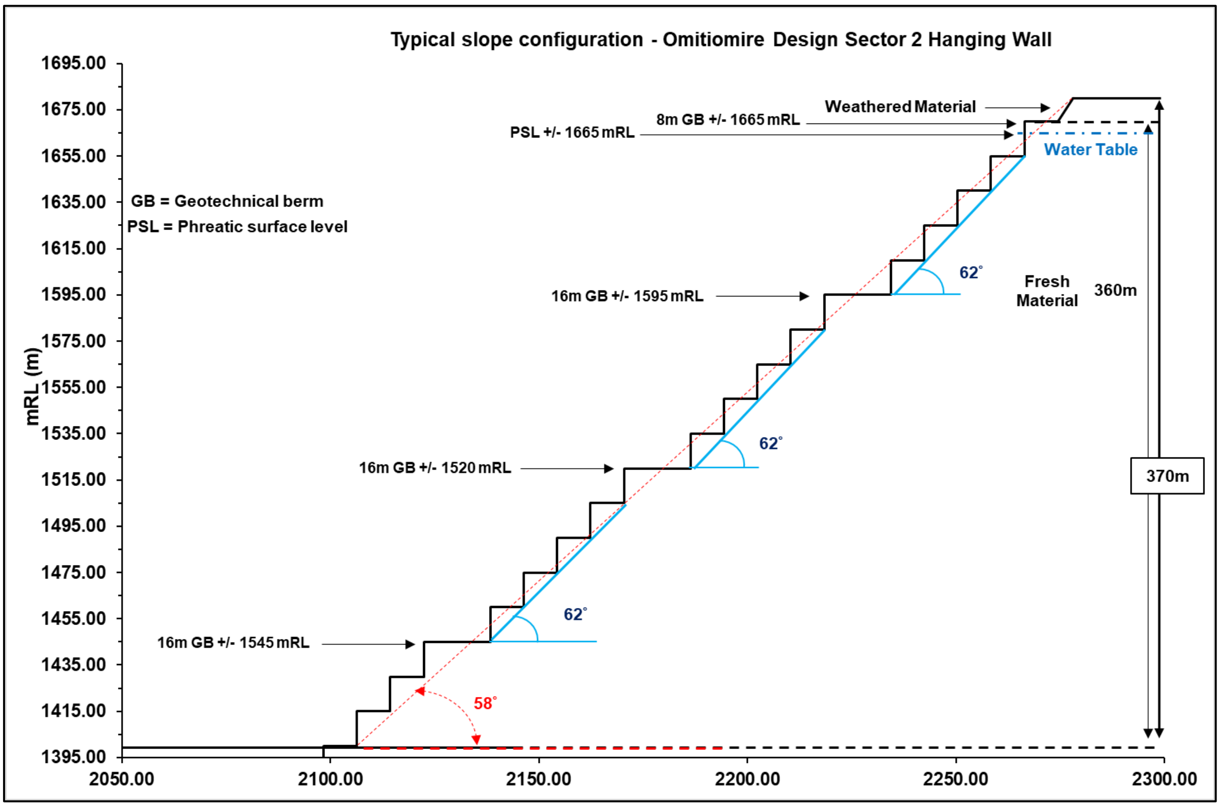


Figure 21-2: Slope configuration design sector 2.

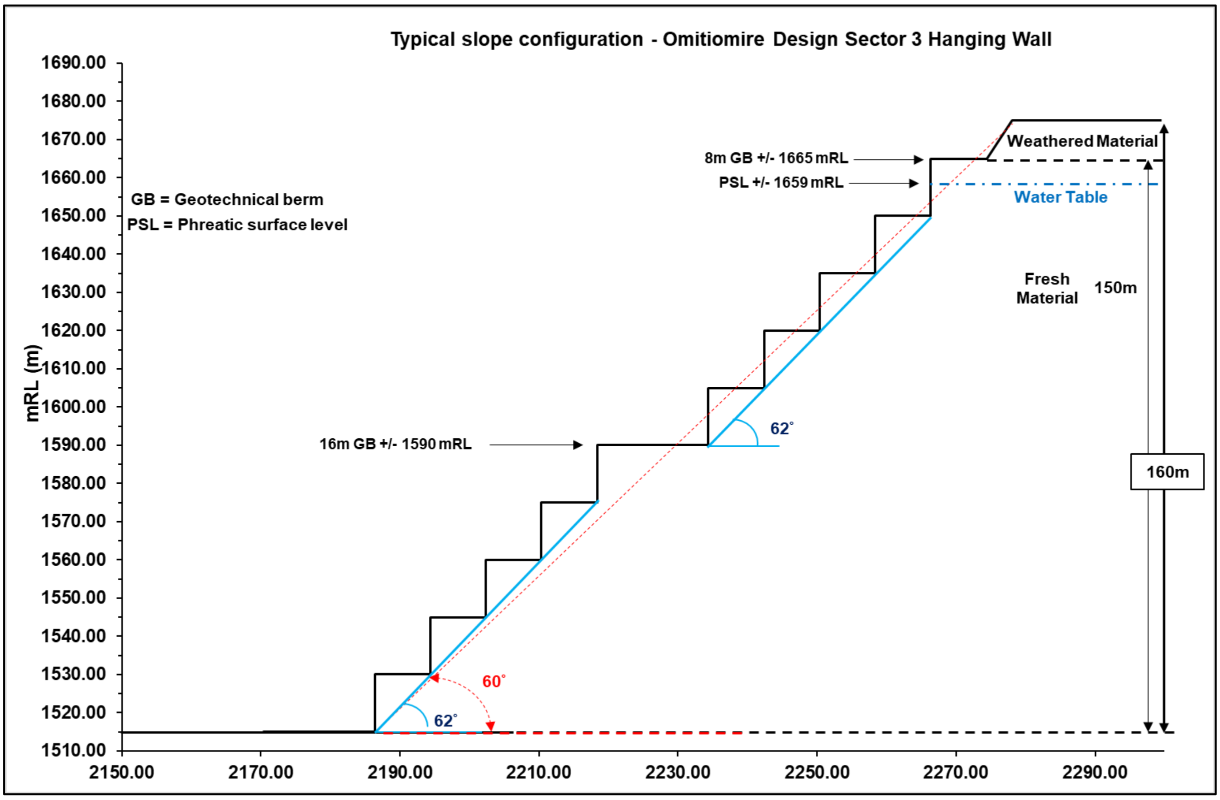


Figure 21-3: Slope configuration design sector 3.

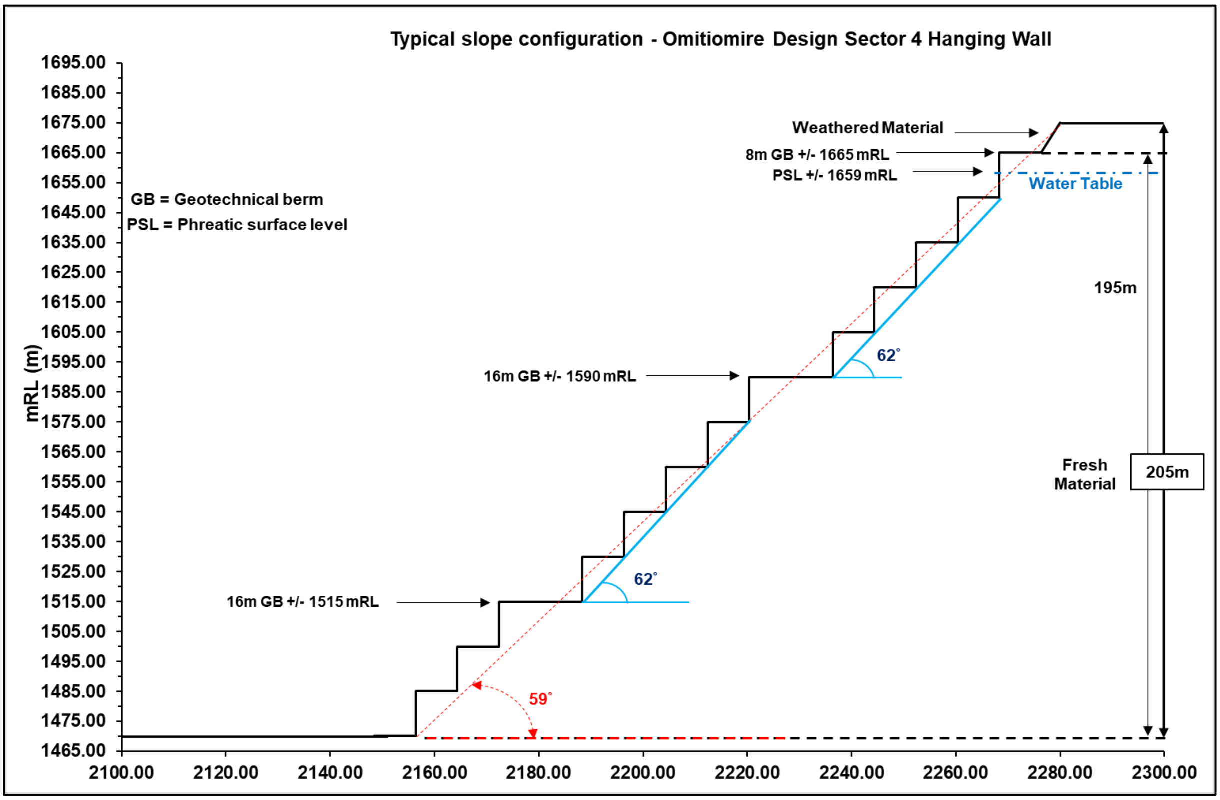


Figure 21-4: Slope configuration design sector 4.

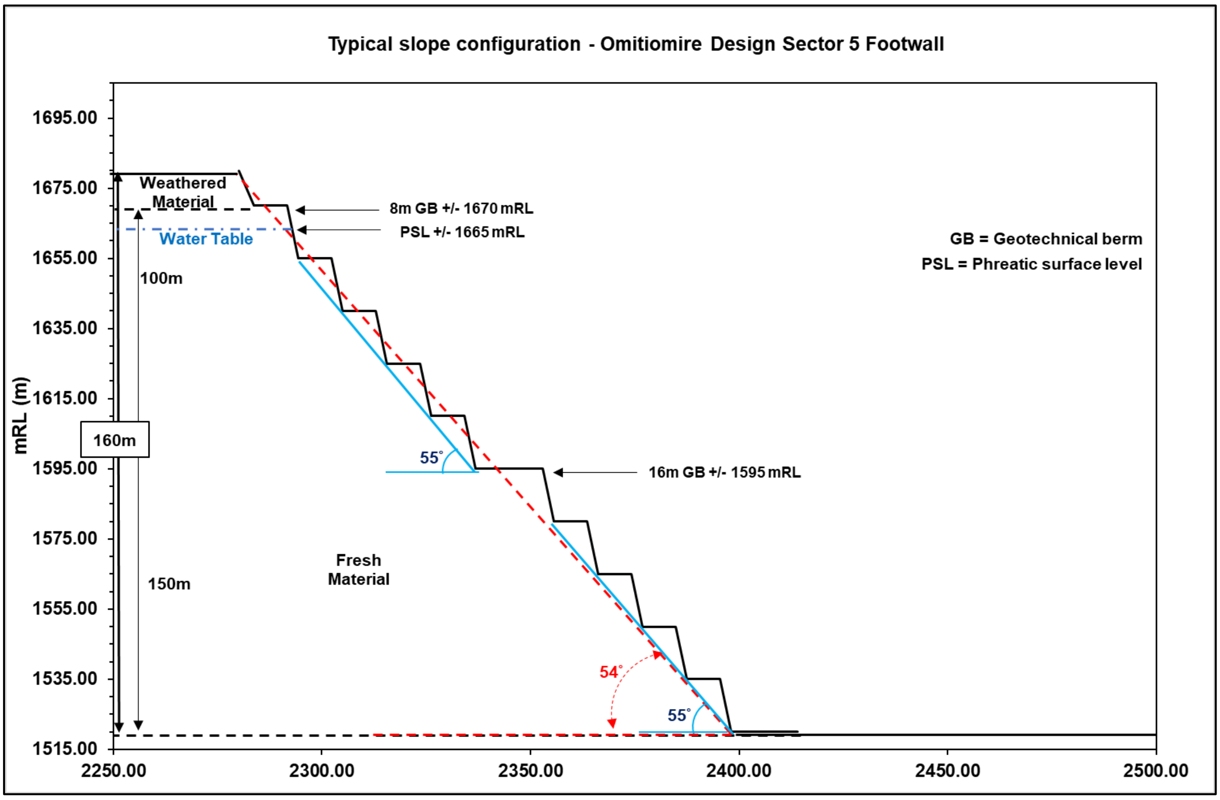


Figure 21-5: Slope configuration design sector 5.

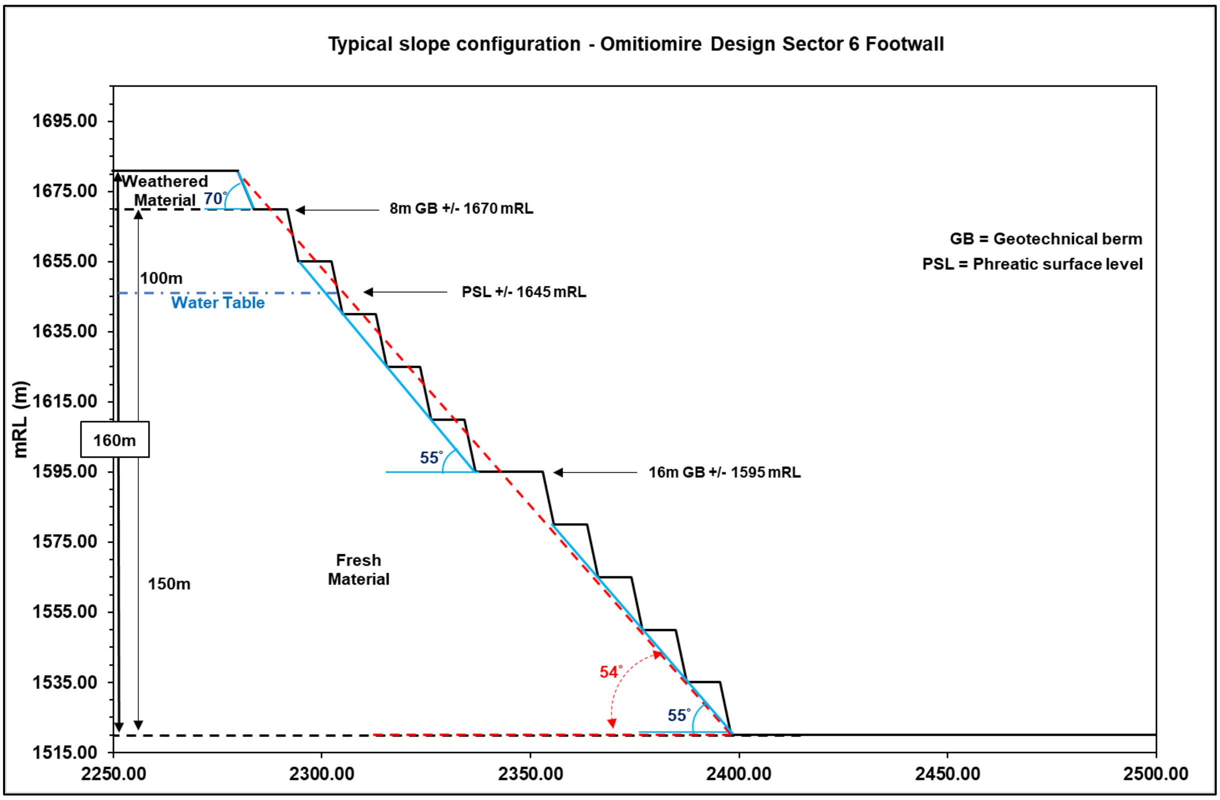


Figure 21-6: Slope configuration design sector 6.

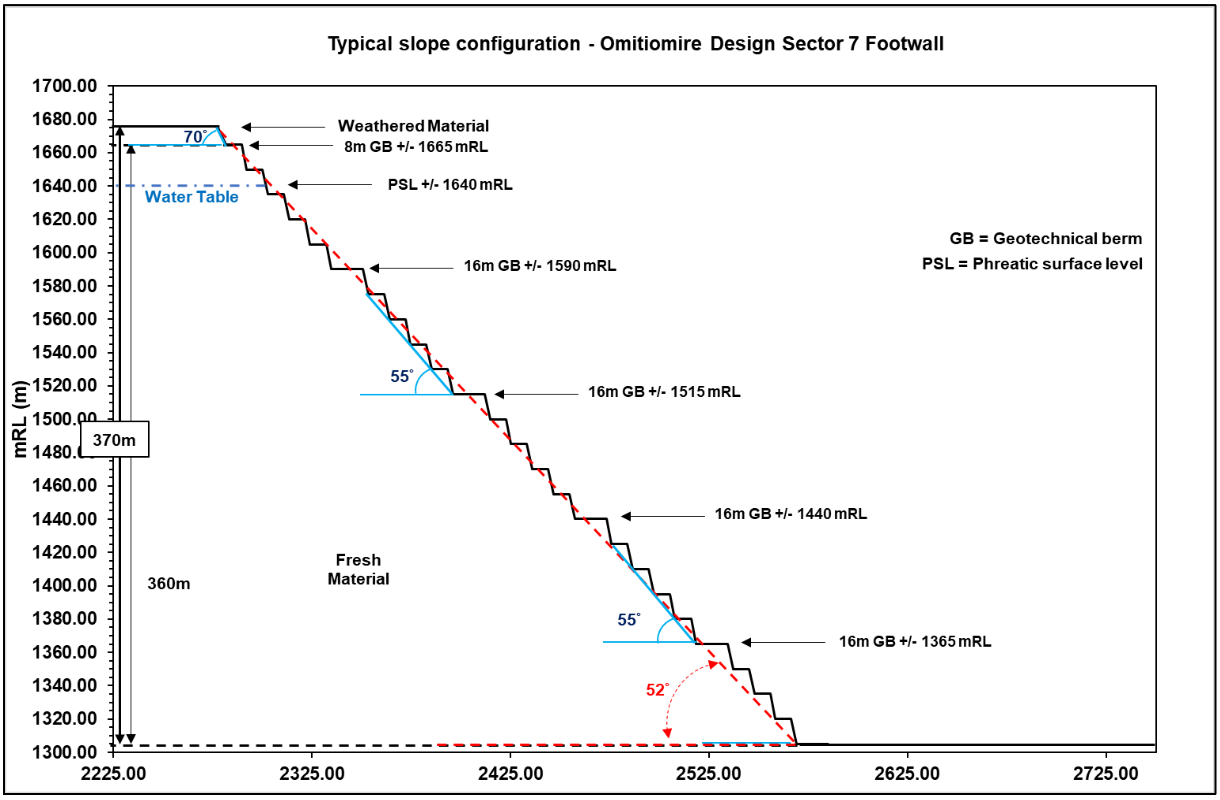


Figure 21-7: Slope configuration design sector 7.

22. WASTE ROCK DUMP DESIGN

Surface waste rock dumps were designed for the Omitiomire pit to ensure that waste material is stable and disposed of in a controlled manner. The terminology used to describe the various components of the waste rock dump is depicted in Figure 22-1. The components include step width (SW), lift height (LH), batter face angle (BFA) and overall slope angle (OSA).

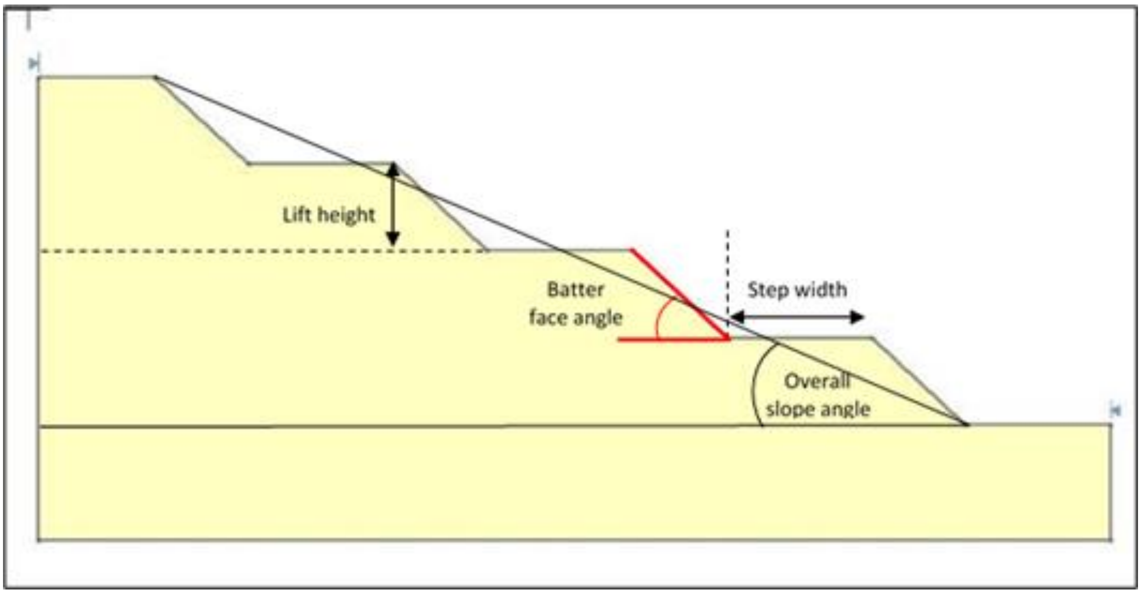


Figure 22-1: WRD layout and terminology.

The assumption was made that there will be two (2) surface waste rock dumps at Omtiomire, and the design described in this section applies to the two (2) WRDs. The assumption was based on a plan in the report document *301-00478-06 – FIG5 – rev A*. The planned layout of the WRDs is shown in Figure 22-2.

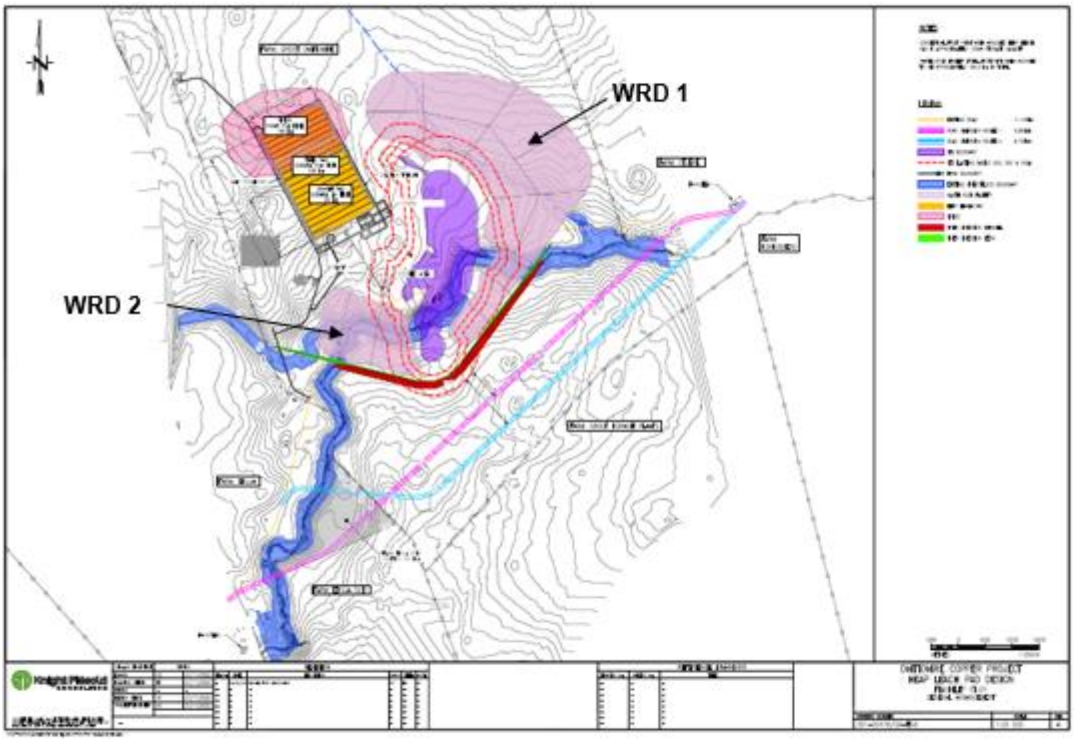


Figure 22-2: Preliminary WRD layout.

The waste rock dumps will consist of the rock types encountered through geological logs from the geotechnical logging. The selection of material properties used in the design could not be based on the properties of only a single rock type but rather an amalgamation. Using the most conservative set of material properties could lead to the stability of the dump being grossly underestimated. Therefore, the overall average between the various rock types was used on the assumption that this would most realistically describe the mixed nature of the dump.

The properties used in the analysis are outlined in below Table 22-1. The bulking factor of 1.44 was applied which is the combined average for the weathered and fresh material bulking factor applied to the rock types used in the pit design in earlier sections of the report; also applied to the combined densities of the material that comprise the WRD, to determine the bulk density of the dump as a whole.

Table 22-1: Material properties used for the waste rock dump design.

Waste Rock Properties (80m Dump Height)												
WASTE TYPE	**ANG**	**FINES**	**cc**	**UCS (MPa)**	**Density (kg/m³)**	**g (m/s²)**	**H (m)**	**Normal stress (kPa)**	**a**	**b**	**c**	**f**
White Gneiss	5	20.00	0	214.07	2620	9.817	80	2057.64	33.08	115.50	-0.40	38.65
Banded Gneiss	5.00	20.00	0	91.89	2730	9.817	80	2144.03	27.48	165.35	-0.40	35.32
Grey Gneiss	5.00	20.00	0	135.98	2670	9.817	80	2096.91	29.50	147.36	-0.40	36.55
Mafic Gneiss	5.00	20.00	0	77.64	2910	9.817	80	2285.40	26.82	171.16	-0.40	34.74
Pink Gneiss	5.00	20.00	0	237.82	2550	9.817	80	2002.67	34.17	105.81	-0.40	39.33
Pegmatite	6.00	20.00	0	119.79	2630	9.817	80	2065.50	28.49	164.24	-0.40	36.40
Biotite Schist	4.00	25.00	0	7.8	2720	9.817	80	2136.18	23.02	192.13	-0.40	32.15
Weathered White Gneiss	4.50	35.00	0	20.55	2530	9.817	80	1986.96	21.75	197.56	-0.40	31.42
Weathered Banded Gneiss	4.50	35.00	0	44.73	2660	9.817	80	2089.06	22.86	187.69	-0.40	31.86
Weathered Grey Gneiss	4.50	35.00	0	44.34	2700	9.817	80	2120.47	22.85	187.85	-0.40	31.80
Weathered Mafic Gneiss	4.50	35.00	0	11.4	2780	9.817	80	2183.30	21.33	201.29	-0.40	30.82
Weathered Pink Gneiss	4.50	35.00	0	76.5	2670	9.817	80	2096.91	24.32	174.73	-0.40	32.69
Weathered Pegmatite	5.50	35.00	0	119.79	2630	9.817	80	2065.50	26.04	167.34	-0.40	34.10
Weathered Biotite Schist	4.00	30.00	0	7.8	2720	9.817	80	2136.18	22.16	194.88	-0.40	31.42
Average properties	**4.79**	**27.50**	**0**	**86.44**	**2680**	**9.82**	**80.00**	**2104.76**	**25.99**	**169.49**	**-0.40**	**34.09**
Average density					**1861.11**							
Unit Weight (MN/m3)					0.01827							
Unit Weight (kN/m3)					**18.27**							

The properties of the different types of waste material were derived using statistical methods that incorporate the following parameters into the accompanying set of equations to determine the waste secant friction angle (Rangasamy, 2009).

- Angularity is measured on a scale of 1-8 with 8 being extreme angularity and 1 being low angularity
- Fines – the percentage of fines passing 0.075 mm (%)
- UCS – Unconfined compressive strength of the rock (MPa)

$$\phi = \boldsymbol{a} + \boldsymbol{b}\boldsymbol{\sigma}_{\boldsymbol{n}}^{\boldsymbol{c}}$$

Where ϕ is the friction angle, and the variables a, b and c are defined as:

$a = 36.43 - 0.267\,ANG - 0.172\,FINES + 0.756\,(C - 2) + 0.0459\,(UCS - 150)$

$b = 69.51 + 10.27\,ANG + 0.549\,FINES - 5.105\,(C - 2) - 0.408\,(UCS - 150) - 0.408$

$c = -0.3974$

Based on the mixed nature of the material of the dump, the average friction angle was used for the overall design (34°). Due to the broken and angular nature of the rock, the design of the waste rock dump did not account for any saturation because the material will be unable to maintain pore water, allowing for free draining conditions.

22.1 Waste Rock Dump Height

To determine the maximum height at which the dump will remain stable, the Rocsience Software Slide was used. A slope was constructed at the angle of 34° and the height was increased incrementally from 20m, 40m, 60m, 80m to 100m. The Slide analysis for each height is illustrated in Figure 22-3 to Figure 22-7.

The safety factors obtained from each slope were plotted against the height of that slope to determine the best possible height for the waste rock dump (Figure 22-8). A limiting safety factor of 1.3 for the waste rock dump was used. The analysis indicated that a maximum dump height of 60m can safely be achieved.

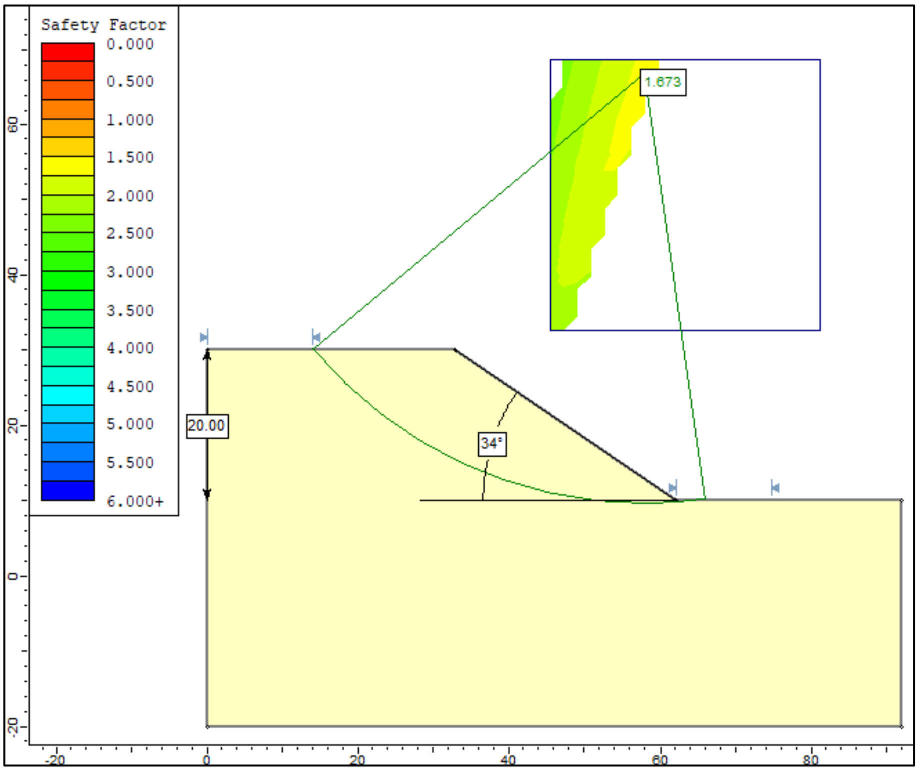


Figure 22-3: Slide analysis for a 20m waste rock dump.

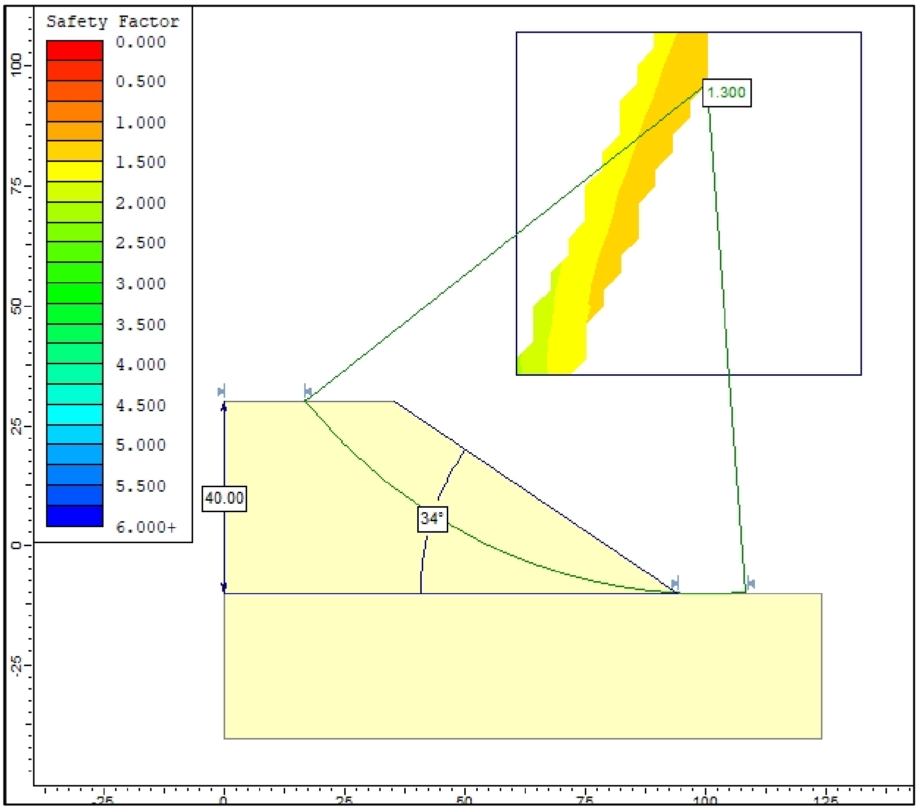


Figure 22-4: Slide analysis for a 40m waste rock dump.

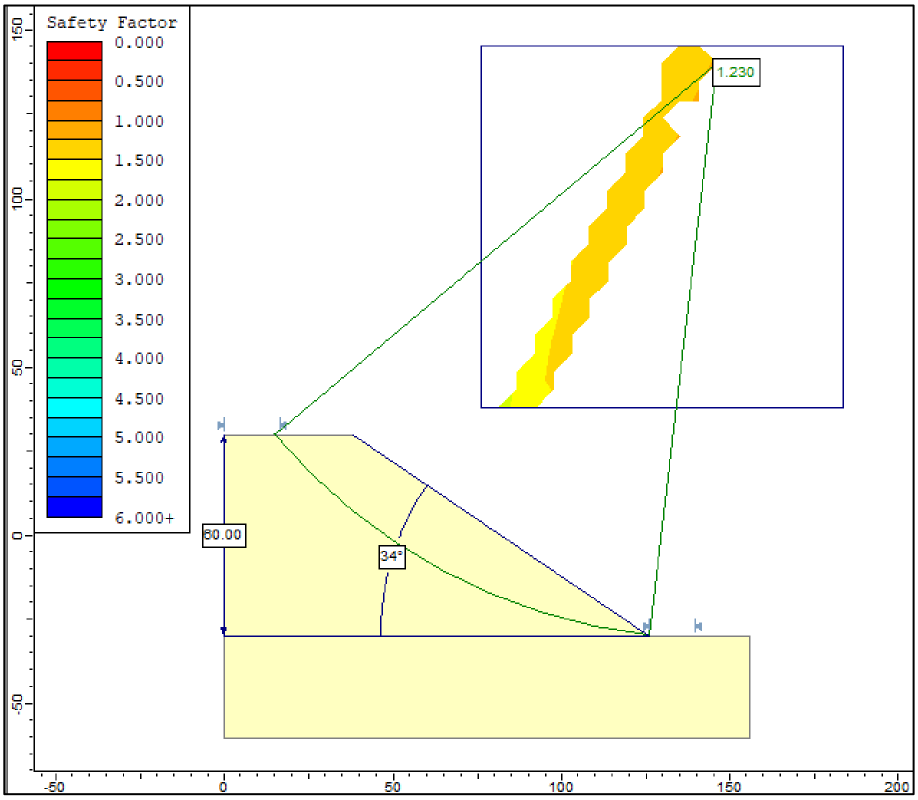


Figure 22-5: Slide analysis for a 60m waste rock dump.

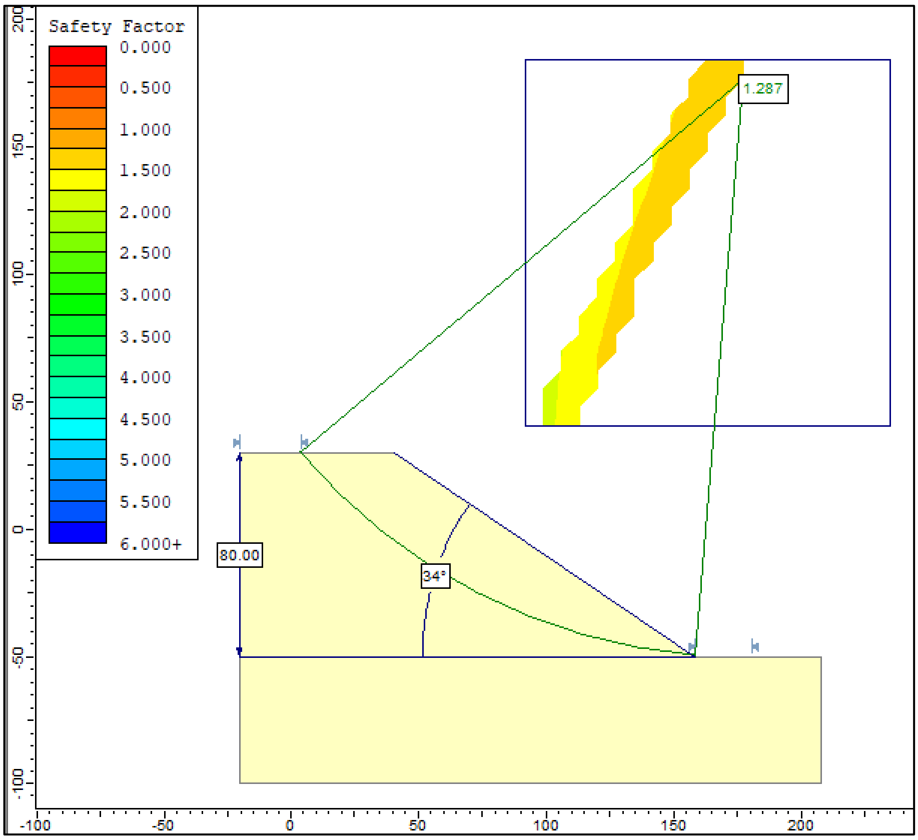


Figure 22-6: Slide analysis for a 80m waste rock dump.

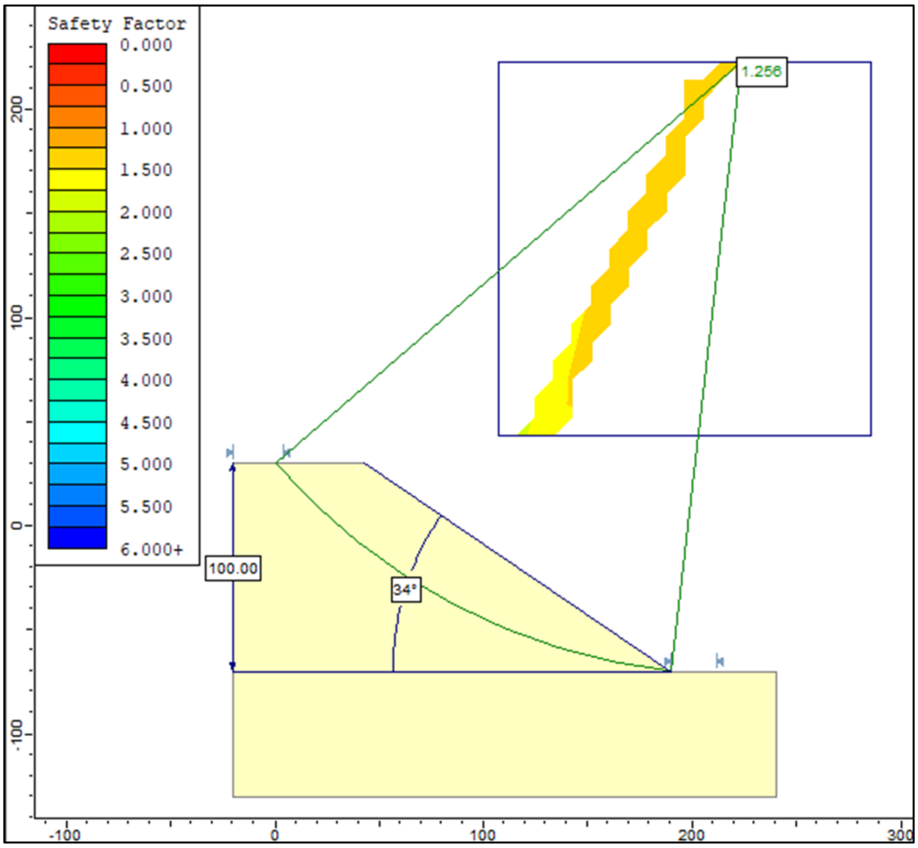


Figure 22-7: Slide analysis for a 100m waste rock dump.

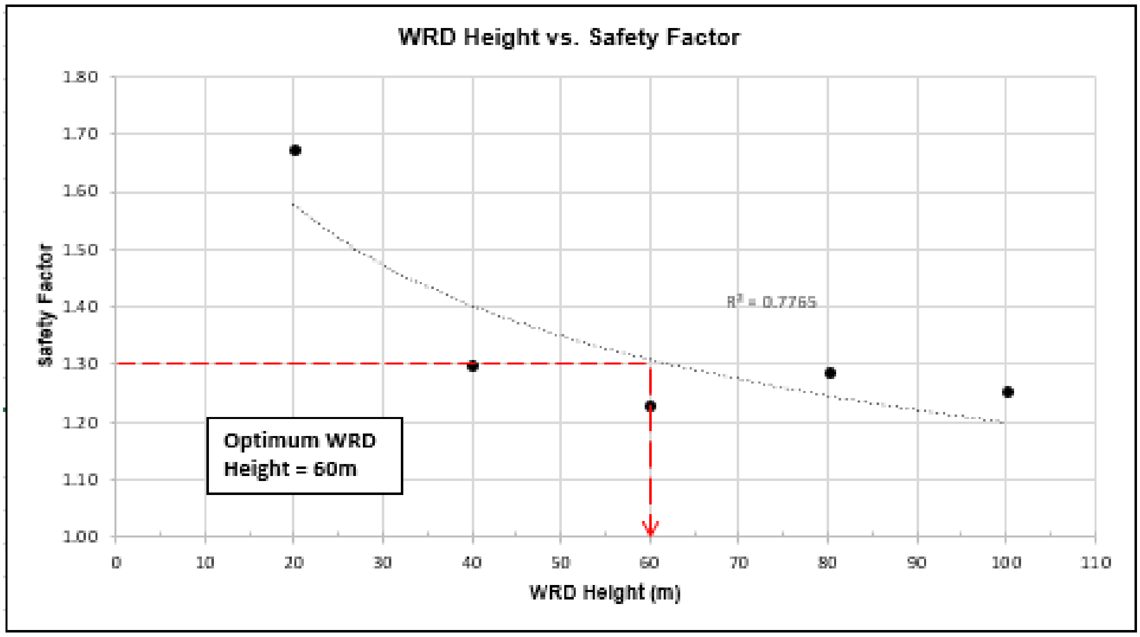


Figure 22-8: Optimum WRD height.

22.2 Distance Required between Waste Rock Dump and Highwalls

The minimum distance that the WRD can be placed away from the pit edge, was calculated by analysing the zone of influence of the WRD on the pit wall in Slide. The WRD's influence was calculated by determining the load that the WRD places on the pit wall.

The properties that were kept constant for the WRD load calculation were the angle of repose, average density, and planned dump width shown in Table 22-2 for both waste rock dumps. The maximum height, derived from the section above, was applied for the calculation of the loads for each WRD.

Table 22-2: Constant properties for WRD load calculations.

Summary		WRD 1	WRD 2
Angle of repose	**Average Density**	**Planned dump width**	**Planned dump width**
34	1861.11	1064.93	925.99

The load was calculated by obtaining the total area of the WRD, to calculate the mass or volume of the WRD, and thereby the force or load. A schematic diagram is shown in Figure 22-9 to depict the shape of the WRD, and the areas a, b, and c. The load calculated for WRD1 is listed in Table 21-3 and Table 21-4 for WRD2.

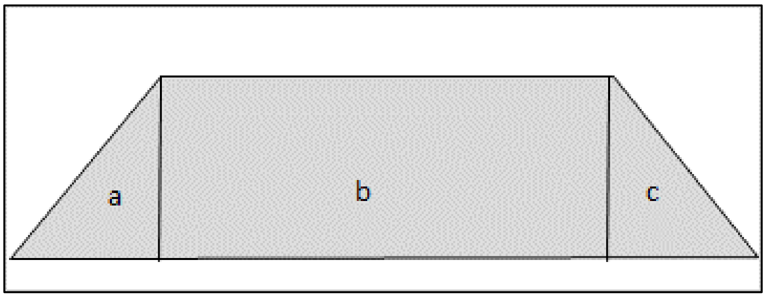


Figure 22-9: Schematic geometry of WRD.

Table 22-3: Calculation of the force generated by the waste rock dump 1.

WRD 1								
Height of dump	**Base (a)**	**Area (a)**	**Area (c)**	**Area (b)**	**Total area**	**Mass (kg)**	**Force (kN)**	**Force per m (kN/m)**
60	**88.95**	**2668.61**	**2668.61**	**63896.08**	**36787.54**	**68465706**	**672127.8**	**540.8**

Table 22-4: Calculation of the force generated by the waste rock dump 2.

WRD 2								
Height of dump	**Base (a)**	**Area (a)**	**Area (c)**	**Area (b)**	**Total area**	**Mass (kg)**	**Force (kN)**	**Force per m (kN/m)**
60	**88.95**	**2668.61**	**2668.61**	**55559.64**	**60896.86**	**1.13E+08**	**1112618**	**1007.9**

The force per metre or load of 540.80 kN/m was applied in Slide for WRD1, and 1007.90 kN/m for WRD2, to model the load of the waste rock dumps on the pit walls.

The WRDs were modelled at different distances away from the pit crest edge. The distance where the WRD no longer affected the safety factor of the pit, became the limiting or minimum distance that the WRD must be placed away from the pit crest edge. The change in safety factor with the distance away from the pit edge was recorded for each WRD. The safety factors and distances are listed for WRD1 and WRD2 in Table 22-5 and Table 22-6 respectively. The distance hgiglited in red was the limitimg distance from the pit edge that the dump can be placed.

Table 22-5: Distance from pit edge versus change in Safety Factors for WRD1.

WRD 1 distance away from edge (m)	Safety Factor
0	1.162
20	1.177
40	1.183
60	1.183
80	1.183
100	1.183

Table 22-6: Distance from pit edge versus change in Safety Factors for WRD2.

WRD 2 distance away from edge (m)	Safety Factor
0	1.963
20	2.084
40	2.193
60	2.193
80	2.193
100	2.193

The distance away from the pit edge was graphed against the change in safety factor in Figure 22-10 and Figure 22-10. The minimum distance that the WRD must be placed away from the pit edge was indicated where there was no change in safety factor as there is no influence on the stability of the pit wall at that distance. The WRD must be placed at least 60 m away from the edge of the pit for both WRD1 and WRD2.

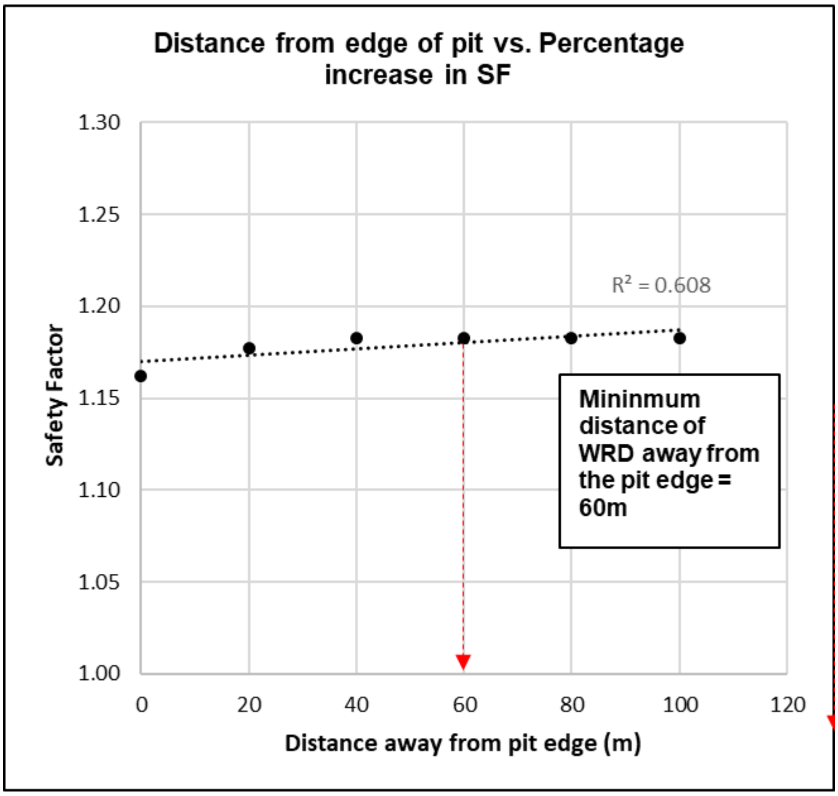


Figure 22-10: The minimum WRD distance away from the pit edge, WRD1.

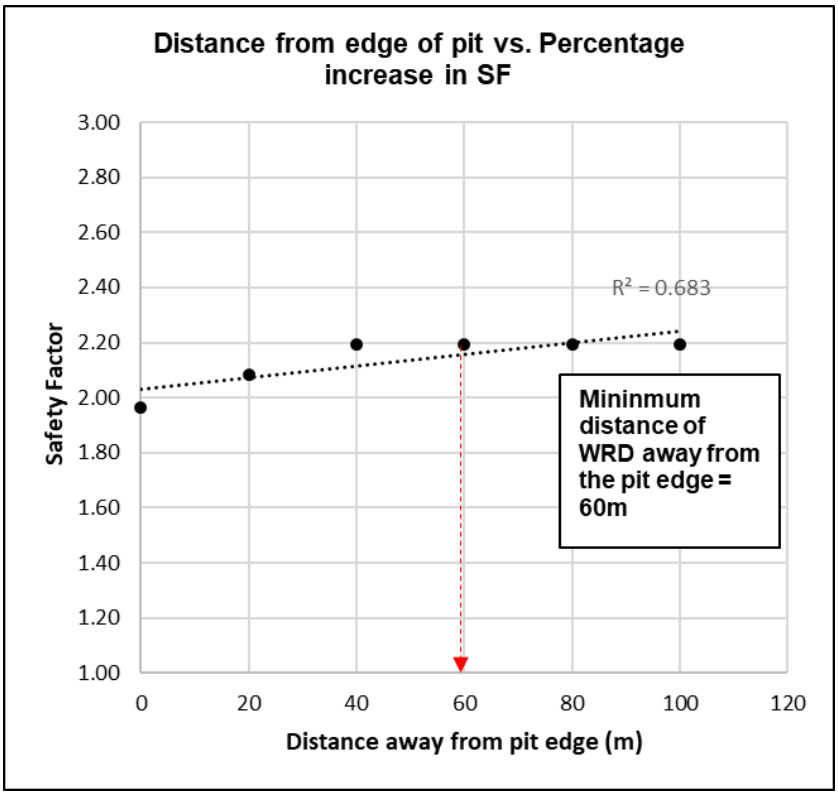


Figure 22-11: The minimum WRD distance away from the pit edge, WRD1.

22.3 Waste Rock Dump Summary

To summarise the waste rock dump analysis, the suggested geometry of each dump is listed in Table 22-7, and is illustrated in Figure 22-12.

Table 22-7: Summarised geometry of the surface WRDs.

Lift height (m)	Lift face angle (°)	Step out distance (m)	OSA (°)
20	34	16	26
Maximum Waste Rock Dump Height (m)			
60			
Minimum Waste Rock Dump Stand-off distance from pit crest edge (m)			
60			

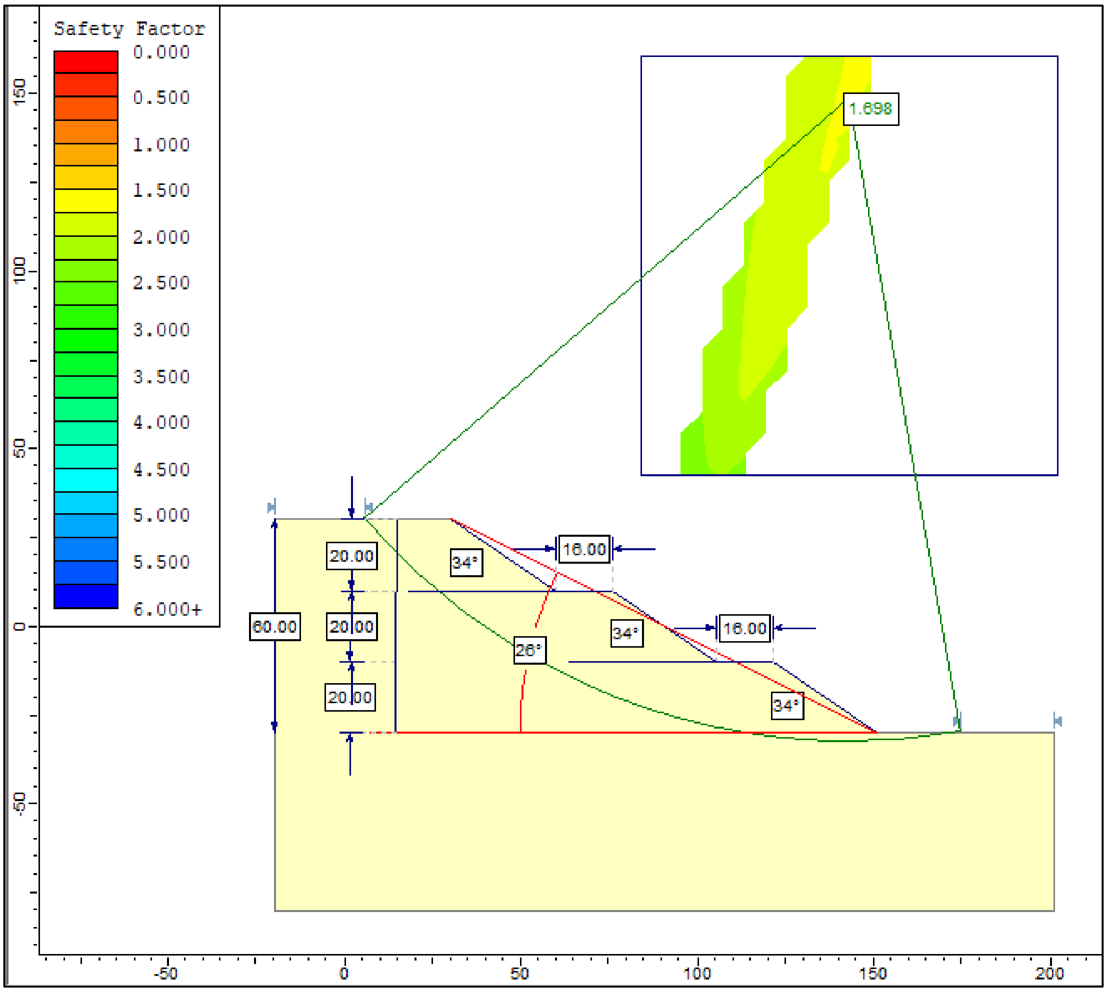


Figure 22-12: Overall safety factor for 60m WRD and geometry.

23. CONCLUSION

The geotechnical data made available and transformed into analysis input parameters allowed for a technically robust design to be produced at a feasibility level of accuracy. The following points summarise the geotechnical content of this submission:

- Eight (8) primary boreholes were used for the pit design, additionally with the supplementation of three (3) historical boreholes that were combined and used for validation of geotechnical parameters derived to form the basis of the geotechnical database.
- A total of 1415 metres of core was drilled and geotechnically logged for the Omitiomire project.
- RQD, RMR_{89} and GSI values were derived from geotechnical logging to form the database
- Eight (8) geotechnically logged boreholes were utilised for dip angles and dip directions and were used to derive the major discontinuity trends for the Omitiomire project area. Additionally, a total of forty-two (42) historical boreholes orientation data was used to supplement stereographic plots. A total of two-thousand-nine-hundred-and-fifty-nine (2959) orientation measurements were available.
- One-hundred and thirty-eight (138) samples were selected for various rock tests, of which one-hundred and four (104) were selected on-site for laboratory rock strength testing and the remaining twenty-four (24) were obtained from historical data.
- A detailed kinematic study was carried out and was based on orientation data and discontinuity properties derived from rock tests analysis.
- The intact rock properties derived were used either directly or indirectly to derive the following:
    - Hoek-Brown strength parameters.
    - Equivalent Mohr-Coulomb parameters.
    - Rock quality indicators.
    - Defect properties of cohesion and friction angle.
- Design sectors were devised based on pit wall directions, rudimentary fault structures and water level depth for the Omitiomire pit.

Further work: When the operation begins, geotechnical data must be continuously collected and compared with the datasets used in this design.

The geological sections used in this submission must be cross checked with the 3-D geological model to ensure all geology was correct. The geological model only became available after compilation and submission of the geotechnical slope design section.

24. RISK ASSESSMENT

24.1 Geotechnical Risk Assessment Process

Geotechnical risks arise from the movement of the ground during and following the creation of an excavation. Risks may relate to slope failures, to changes in flow rates of watercourses and surface water bodies or they may relate to movements of structures and infrastructure adjacent to or within the mine.

Legislation requires that a rock engineering risk assessment be carried out to identify hazards and assess the health and safety risks to which employees may be exposed whilst they are at work. The significant hazards identified should be recorded, the risks assessed, and significant risks are to be mitigated to create a safe work environment. This risk assessment was focused on the risks and hazards associated with the geotechnical data acquisition process for the Omitiomire open pit.

A geotechnical risk assessment is never a static activity and as the site develops, more geological knowledge may be available and a greater understanding of the behaviour of the ground may develop.

The areas being mined will be altered and slopes will continually be renewed. In all cases, risks may change. It is thus suggested that the assessment of geotechnical risk be an iterative and ongoing process throughout the life of a site.

24.2 Risk Assessment Methodology

The impact and risk induced by the planned mining on the surrounding environment and infrastructure were assessed using a risk matrix. To convey the full impact of the risk, the matrix expands risk into its three parts, the hazard, the likelihood of the hazard becoming an event and the magnitude of the consequences of the event.

Risk emanating from a single hazard is then computed by multiplying the probability of occurrence by the magnitude of the consequence, with the total risk given by the set of all possible hazards and their risks. Standard risk assessment tables (RAMP, 1998) were used in Table 24-1 and Table 24-2 to assess the hazards that collectively influence the level of risk for a particular risk parameter.

Table 24-1: Risk rating likelihood.

Likelihood			
Description	**Scenario**	**Probability**	**Scale Value**
Highly likely	Very frequent occurrence	Over 85%	16
Likely	More than an evens chance	50-85%	12
Fairly likely	Quite often occurs	21-49%	8
Unlikely	Small likelihood but could well happen	1-20%	4
Very unlikely	Not expected to happen	0.01-1%	2
Extremely unlikely	Just possible but very surprising	Less than 0.01%	1

Table 24-2: Consequence rating likelihood

Consequence		
Description	**Scenario**	**Scale value**
Disastrous	Business investment can not be sustained	1000
Severe	Serious threat to business or investment	100
Substantial	Reduces profit significantly	20
Marginal	Small effect on profit	3
Negligible	Trivial effect on profit	1

24.3 Risk Acceptance Levels

The risk matrix adopted provided acceptance levels as shown in Table 24-3 and Table 24-4. The risk ratings were done initially without controls in place (inherent risk) and then with controls in place (residual risks).

Table 24-3: Risk rating matrix.

Likelihood	Consequence					
	Ratings	Disastrous	Severe	Substantial	Marginal	Negligible
		1000	100	20	3	1
Highly likely	16	16000	1600	320	48	16
Likely	12	12000	1200	240	36	12
Fairly likely	8	8000	800	160	24	8
Unlikely	4	4000	400	80	12	4
Very unlikely	2	2000	200	40	6	2
Extremely unlikely	1	1000	100	20	3	1

Table 24-4: Risk rating threshold.

Acceptance thresholds		
Points	Category	Action required
Over 1000	Intolerable	Must eliminate or transfer risk
101-1000	Undesirable	Attempt to avoid or transfer risk
21-100	Acceptable	Retain and manage risk
Up to 20	Negligible	Can be ignored (monitor)

24.4 Risk Rating Register

The complete register is shown in Table 24-5. The results with and without controls are graphically depicted in Figure 24-1 and Figure 24-2.

Table 24-5: Risk rating register.

Process	Ref	Risks and hazards (L=Likelihood, C=Consequence)		Rating - No Controls			Design controls	Rating - With controls		
		Hazard	**Risk**	**L**	**C**	**Overall Risk**	**Controls to mitigate risk**	**L**	**C**	**Overall Risk**
Geological aspects influencing geotechnical design	**A1**	*The lithological model for the operations is lacking (3-Dimensional)*	The geological model informs the design aspect and holistic understanding of the environment.	4	100	400	A complete geological model must be administered to supplement the geotechnical data to increase the validity and accuracy of design and modelling	8	3	24
	A2	*Major geological structural intersections are unknown (disposition and spatial)*	Fault and shear structures initiate weakness in the rock mass quality	4	100	400	A very rough indication of the major structures was included in the design as limited information regarding these structures was available. More detail should be included at a later stage.	8	20	160
	A3	*Higher than normal degree of weathered material*	Lower slope angles than would typically be the case in slopes with shallow weathered profiles	4	20	80	The weathering based on boreholes used for the study reached an average depth of 10m. Weathering modelling analysis was conducted to determine appropriate berm widths and bench face angles for weathered material catchment	4	3	12
	A4	*Strata dips at unfavourable angles to the high wall*	Day-lighting anisotropic geological structures within the high wall may initiate slope failure	12	100	1200	Anisotropic features are incorporated into modelling to accommodate prominent structures. Berms have been designed to incorporate any bench scale failures	8	3	24
	A5	*Lack of geotechnical sampling of core from a historical database*	Limited historical data from previous study data acquisition	2	100	200	The combined geotechnical database included to supplement design and engineering	1	3	3
	A6	*No survey data provided*	Incorrect traverses incorporated into survey data for the kinematic assessment	4	100	400	Design proceeded with 5 of 8 survey data boreholes. The remaining 3 boreholes survey data was used as the proposed collar positions.	4	100	400
	A7	*Standard logging procedures were not followed*	No geological logs and identification of rock types for the geotechnical boreholes	12	100	1200	ISRM logging procedures were used for all holes. Quality assurance and quality control conducted on geotechnical logging. Geological logging must be done to confirm rock types for geotechnical boreholes	8	3	24

Geotechnical data acquisition and incorporation	**B1**	*Geotechnical drilling and logging are unrepresentative of mining tenure*	Limited understanding of the geotechnical aspects affecting the environment	12	100	1200	More geotechnical drilling and data must be acquired as operations expand to deeper depths to increase confidence in spatial representativity	8	3	24
	B2	*Defect orientations within poor rock mass conditions or subpar drilling practice*	Limited orientation data to inform the design	4	100	400	Highly reliable and good reliability logging data and historical ATV orientations used for kinematics	4	20	80
	B3	*Lab rock test results were improperly interpreted and used. Outliers included.*	Incorrect data input parameters for domains and the subsequent design output were overestimated or underestimated.	8	100	800	Outliers/ anomalies excluded. Statistical data analysis was completed to represent the rock mass	4	20	80
	B4	*Geotechnical test work is inadequate*	Design based on unquantifiable parameters	4	100	400	All major rock types were laboratory tested and historical data incorporated	2	3	6
	B5	*Field estimates of strengths have not been derived*	Overly optimistic design created	4	100	400	The Hoek-Brown and Mohr-Coloumb strength criterion was used to derive strength estimates for geotechnical domains	2	20	40
	B6	*A groundwater study was not completed*	Geotechnical design not based on the complete suite of input parameters	4	100	400	The groundwater study was completed and was incorporated into the design modelling the phreatic surface	2	20	40
Design elements	**C1**	*A slope design is unavailable*	Risk of a bench and overall scale instability, rudimentary and indicative design conducted	12	100	1200	Slope design using limit equilibrium, empirical, numerical and analytical methods have been completed. Areas of variation are still expected and should be assessed using the mine's operational standards and monitoring	1	100	100
	C2	*The slope design is not reviewed*	Major risks overlooked or not engineered around	4	20	80	Internal review processes and qualified person competency sign-off. Geotechnical aspects of design are to be reviewed externally to validate and ensure validity and quality of work. Independent peer review of the design to be conducted	2	20	40
	C3	*The slope design does not conform to internationally accepted thresholds*	A high probability of slope failures is inherent in the design	4	100	400	Acceptance thresholds and safety factors typical for mining operations have been used and adhered to	1	20	20
	C4	Design sectors not defined	Specific characteristics relating to specific regions of the pit might be overlooked resulting in a "one size fits all "design that does not necessarily work for the entire pit.	8	100	800	Appropriate design sectors have been formulated based on geotechnical conditions. Design is tailored to conform to the geotechnical conditions existing in the respective sectors.	2	6	12

	C5	*Slope design does not subscribe to local regulations*	The design does not conform to local MHSA practice	12	100	1200	Designs are based on proven industry best practices and literature and are vetted by industry expertise and experience	2	20	40
Waste Rock Dumps	**D1**	*The insufficient surface footprint for expansion*	Excessive height of waste rock dump	12	100	1200	Height restricted to 60m, operational controls	4	20	80
	D2		Encroachment onto surface infrastructure	12	100	1200	Ground control monitoring via instruments, visual inspections & planned task observations	4	20	80
	D3		Too close to the mine boundaries	12	100	1200	Ground control monitoring via instruments, visual inspections & planned task observations	1	20	20
	D4		Increase in lift heights to accommodate insufficient foot space	8	100	800	Height restricted to 20m, operational controls	4	20	80
	D5		Insufficient berm widths	4	20	80	Monitoring and reporting of actual against designs, geotechnical berm widths restricted to 16m	4	3	12
	D6	*Presence of natural hazards*	Presence of liquefiable layers	1	100	100	No history was reported, test work must be conducted to investigate if present	1	3	3
	D7		Presence of old landslides	1	3	3	No history reported	1	3	3
	D8		Susceptibility to strong ground motions	1	3	3	No seismicity was reported. Project in a low seismic area.	1	3	3
	D9	*Restrictive geohydrological characteristics*	High mean annual precipitation	4	100	400	Semi-arid savannah-type land status confirmed through hydrological analyses. The shallow groundwater level in the vicinity of the Black Nossob River	4	20	80
	D10		Elevated groundwater table	4	20	80	WRDs must be placed away from ponds and pans. Diversion of Black Nossob River essential for inflow and water recharge in the immediate mining environment	4	3	12
	D11 **D12**		A complex geological structure that acts as water conduits into the foundation, schistosity of dominant gneiss rock type may act as such	2	20	40	Geological model to confirm. Monitoring and water-draining practices must be adhered to if geohydrological influences are present	2	3	6
	D13	*Unspecified waste material characteristics*	Particle size distribution is highly variable	8	20	160	Crushing protocols need to be developed and implemented	4	20	80
	D14		Uncontrolled tipping of clay materials	8	100	800	Planning and scheduling protocols need to be developed and implemented	4	20	80
	D15		Excessive lift heights that influence crest settlement	8	100	800	Height restricted to 20m, operational controls	4	20	80
	D16		Segregation of waste materials	8	20	160	Planning and scheduling protocols need to be developed and implemented	4	3	12

	D17		Exposure of clay materials to weathering	8	100	800	End-tipping sequence protocols need to be implemented if clay materials present	4	3	12
	D18									
	D19	*Inappropriate design of waste rock dump*	Failure to comply with legislative practices	2	20	40	The final grading plan as required by legislation needs to be adopted	2	10	20
	D20		Failure to design the final grading plan	2	20	40	The final grading plan as required by legislation needs to be adopted	2	10	20
	D21		Inappropriate derivation of input parameters	4	100	400	Industry-adopted practice implemented and validated through modelling	4	10	40
	D22		Failure to conduct limit equilibrium analyses	4	100	400	Numerical modelling assessments have been conducted	2	10	20
	D23		Excessive lift heights that influence crest settlement	8	100	800	Height restricted to 60m, operational controls	2	10	20
	D24	*Inadequate operational ground control protocols*	Visual indicators for potential failures not recorded routinely	4	100	400	Implementation of policies and procedures together with safe working practice	4	20	80
	D25		Inadequate dump movement monitoring	4	100	400	Implementation of policies and procedures together with safe working practice	4	20	80
	D26		Routine dump inspections not conducted	4	100	400	Implementation of policies and procedures together with safe working practice	4	20	80
	D27		Exposure to restricted areas	4	100	400	Implementation of policies and procedures together with safe working practice	4	20	80
	D28		Inadequate back analysis of failures	4	100	400	Implementation of policies and procedures together with safe working practice	4	20	80
	D29		Failure to report failures	4	100	400	Implementation of policies and procedures together with safe working practice	4	20	80

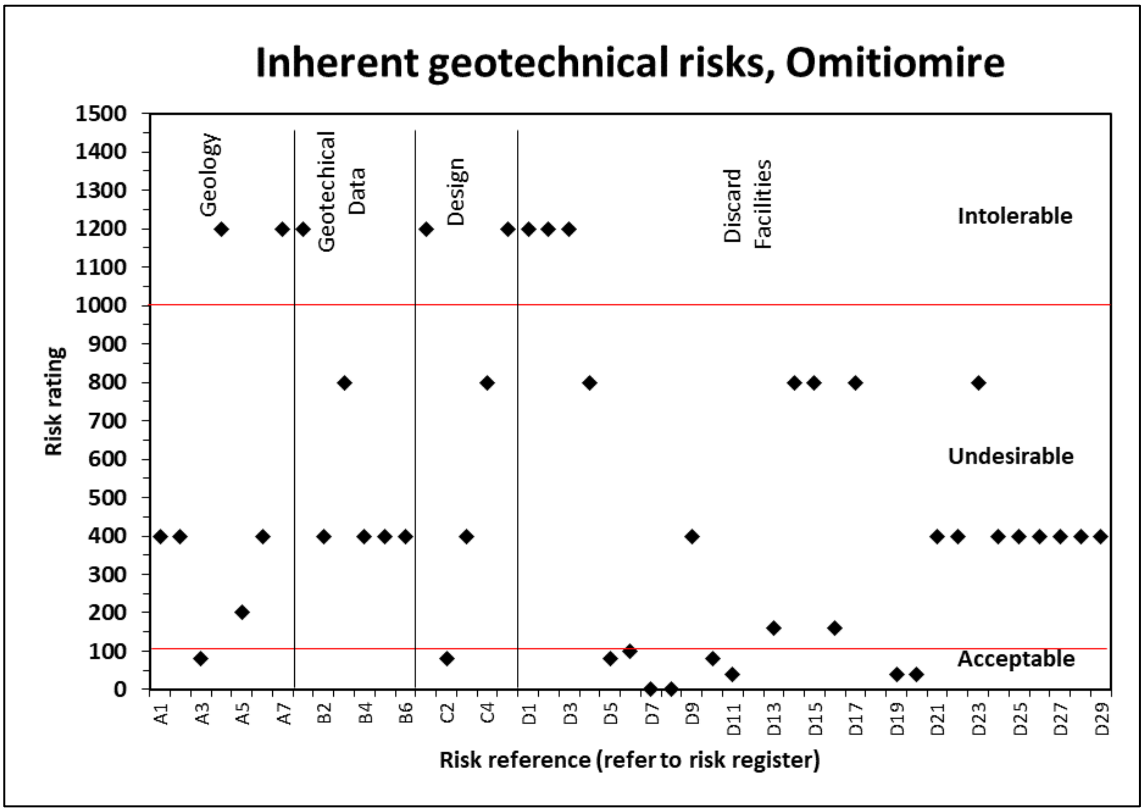


Figure 24-1: Geotechnical risk rating (no controls).

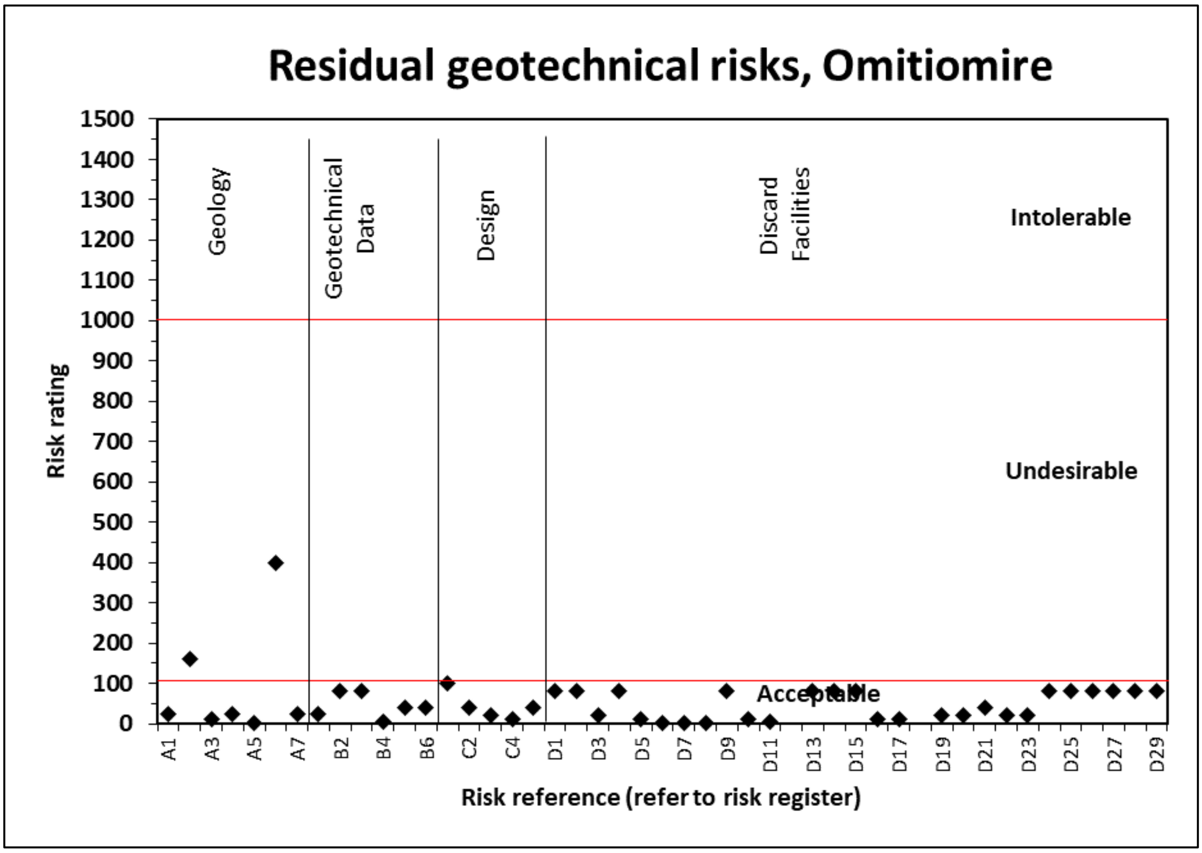


Figure 24-2: Geotechnical risk rating (controls).

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