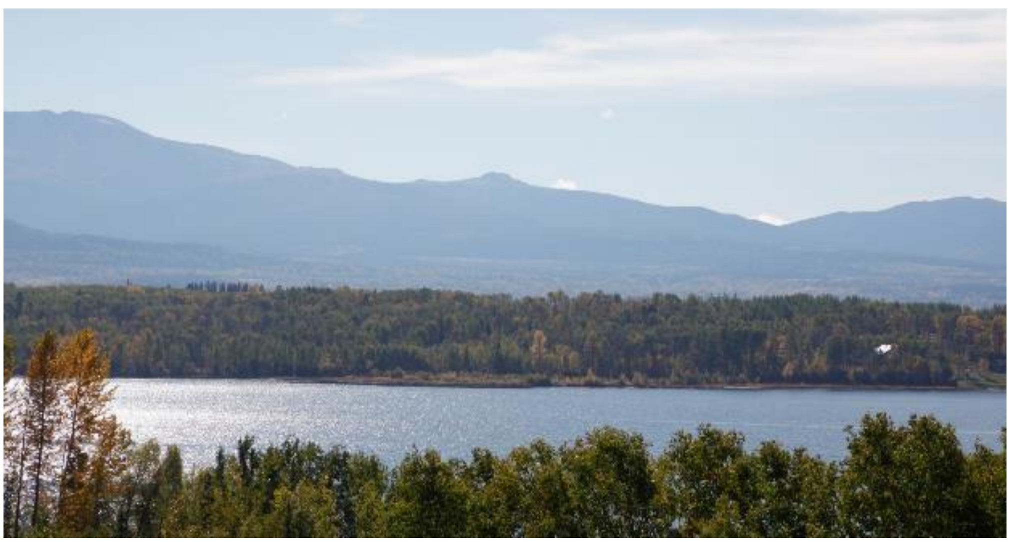
Appendix 1.0-Ia SRK, Tenas Project, Pit Backfill Stability Assessment Report

TELKWA COAL
LIMITED

Tenas Project – Pit Backfill Stability Assessment Report

Prepared for

Telkwa Coal Ltd.



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2CT025.008
April 2021

Tenas Project – Pit Backfill Stability Assessment Report

April 2021

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Appendices

Appendix A: SRK Factual Data Reports for 2018 and 2019 Geotechnical Investigations

Appendix B: SRK 2019 Overburden and Rock Pile Stability and Design Report

Appendix C: Proposed Mine Plan Presentation

Appendix D: Results of Backfill Slope Stability Analyses

1 Introduction

Telkwa Coal Limited (TCL) completed mine designs to support an Environmental Assessment (EA) application for the Tenas Project, located in central British Columbia. Telkwa retained SRK Consulting (Canada) Inc. (SRK) to conduct a stability assessment of the rock backfill designs that are to be constructed as part of the phased open pit development.

SRK had previously completed Feasibility Study (FS) level rock and overburden pile slope assessment for the Tenas Project in 2019 (SRK, 2019a) which included similar backfill piles. The FS work was based on the results of two field investigations completed in 2017 and 2018 (SRK, 2018 and SRK, 2019b), and findings from these investigations have been utilized for this assessment and are included in Appendix A and B. This report should read in conjunction with the previously completed SRK reports referenced throughout this document.

2 Proposed Mine Plan

The excavation of the Tenas Pit is to be completed in several phases, each with a subsequent backfill sequence. The proposed mine development is provided in Appendix A. The Non-Acid Generating (Non-PAG) and Potentially-Acid Generating (PAG) rock is to be segregated through strategic excavation of the sedimentary sequences. The Non-PAG rock is to be used to backfill the open pit and/or be used for constructing portions of the embankments for the Management Ponds and the PAG is to be disposed underwater in three Management Ponds comprising the East, West and North ponds (Figure 2-1). Stockpiled overburden is to be placed onto the backfill platforms, away from the slope face, following construction. The geochemical and geotechnical design of the Management Ponds is not part of this assessment report but are addressed in two separate SRK reports titled “Geochemical Baseline Report and the Metal Leaching/Acid Rock Drainage Management Plan” and “Management, Sedimentation and Control Pond Report”. The pit backfilling is to be commenced along the shallowest dipping footwall areas (Phase 1, 2 and 4 Pit) prior to rock placement on the steeper dipping southeast Phase 3 Pit footwall area (Figure 2-2). The toe of the backfills are to be buttressed against the pit highwalls. Additionally, an opposing overburden and rock cover is to be placed across the pit highwalls for reclamation purposes (Figure 2-2), providing additional buttressing in some areas. Following completion of excavation, pit lakes are to be formed.

The backfill slope design recommendation used to develop the EA mine design and plan are summarised in Table 2-1

Table 2-1: Backfill slope design recommendations

Pile	Maximum Platform Lift Height (m)	Platform Lift Face Slope Angle (°)	Maximum Overall Slope Angle (°)[1]
Pit Backfills	40	37	27 (2:1 slope gradient)

Note: [1] – if greater than single 40 m platform height, otherwise the platform slope angle face forms the overall slope.

Of significance to the open pit and backfill stability is the excavation sequence within the Phase 3 portion of the ultimate Pit. Undercutting of the west dipping sedimentary sequences is required to extract economic coal from the base of the pit (Figure 2-3). The proposed excavation is to occur from south to north in Phase 3, with an excavation of interim northern endwalls and development of a benched backfill to the south. Table 2-2 provides a summary of the Tenas soil and rock pile details. The pit slope stability assessment work is detailed in a separate report titled “Tenas Project - Pit Slope Stability Assessment” (SRK, 2020).

Of significance to the open pit and backfill stability is to be excavation sequence within the Phase 3 Pit. Undercutting of the west dipping sedimentary sequences is required to extract economic coal from the base of the pit (Figure 2-3). The proposed excavation is to occur from south to north, with an excavation of interim northern endwalls and development of a benched backfill to the south. Table 2-2 provides a summary of the Tenas soil and rock pile details.

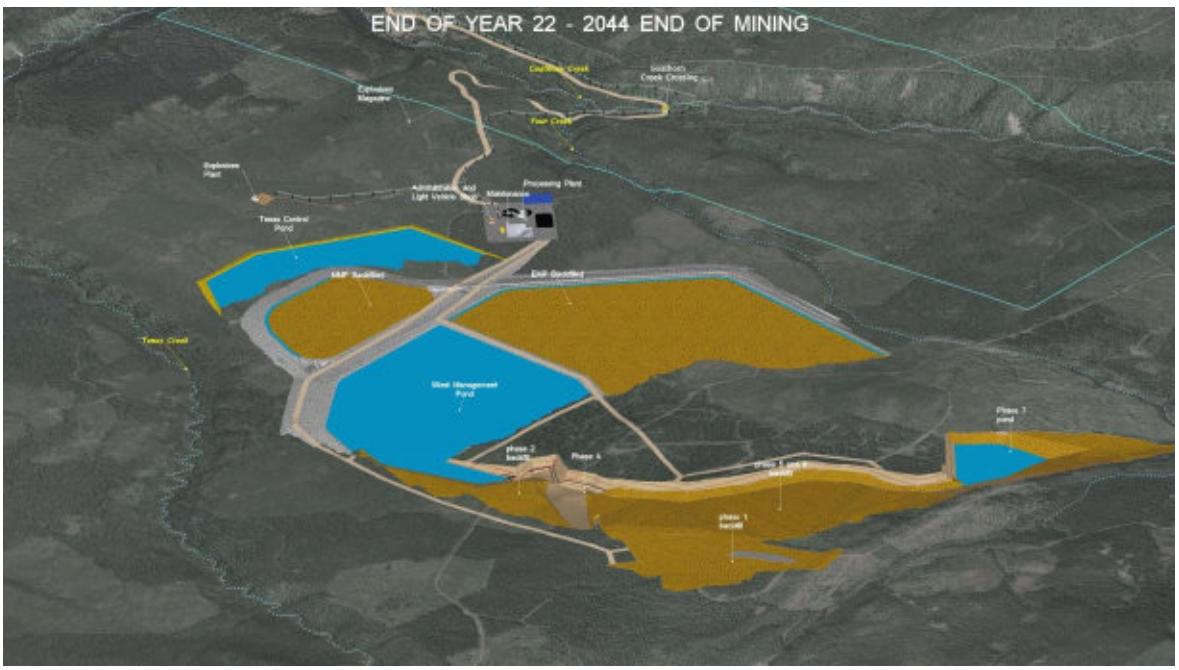

Figure 2-1: Proposed End of Year 22 Mine Plan

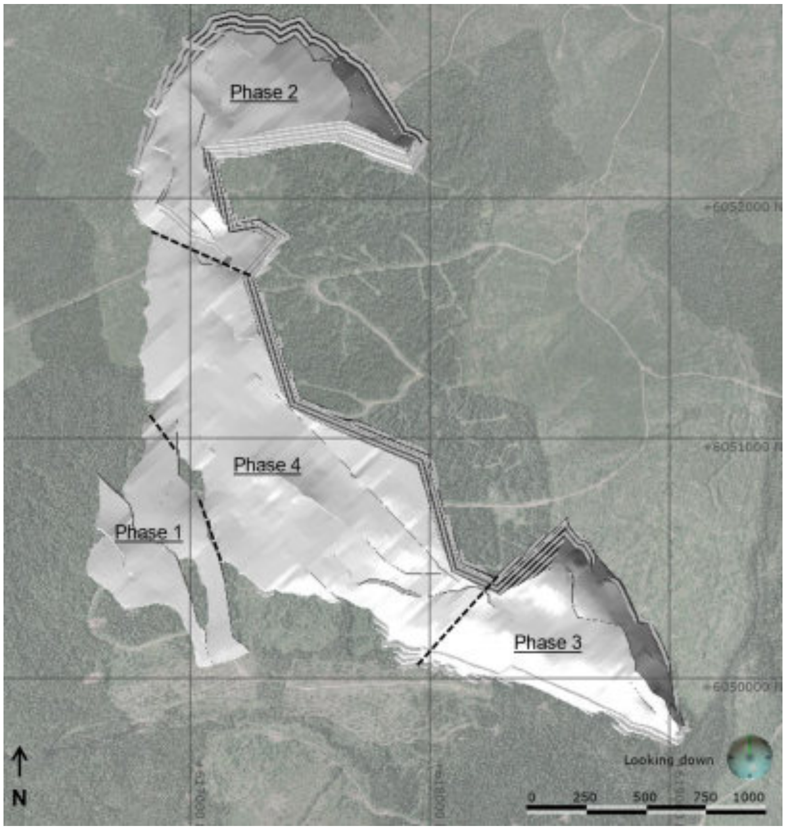

Figure 2-2: EA pit design showing excavation phases

Figure 2-3: Proposed Year 2048 mine reclamation plan

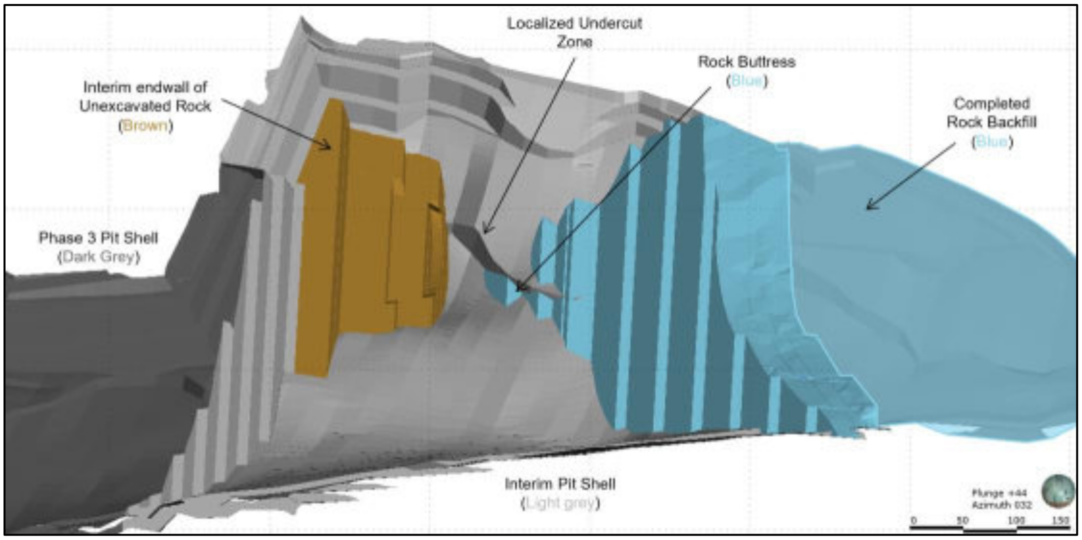

Figure 2-4: Schematic figure showing interim endwall and backfilling within the Phase 3 Pit.

Table 2-2: Summary of Tenas Soil and Rock Piles

Backfill	Material Type	Maximum Overall Slope Height (m)	Overall Slope Angle (°)[1]	Approximate Foundation Footwall Slope Angle (°)	Approximate Mine Years
Phase 1 and 2	NAG rock	15	37	5	1 to 2
Phase 3	NAG rock	135	27	41	5 to 15
Phase 4	NAG rock	80	27	13	13 to 22

Note: [1] – End of Construction Overall Slope Angle

3 Pit Slope Field Investigations

SRK conducted pit slope geotechnical investigations in 2018 and 2019 comprising of eight diamond drillholes (SRK, 2018 and SRK, 2019). The inclined drillholes were positioned to investigate the rock mass that is to be mined and the ultimate footwall rocks. The locations of the drillholes are presented in Figure 3-1.

The drillholes were geotechnically logged, sampled, field tested and supported with laboratory testing. Specific to the pit slopes that are to be backfilled, the laboratory testing comprised intact rock strength and direct shear testing along bedding discontinuities. The results of the pit slope field investigations are summarised two separate data reports (SRK, 2018 and SRK, 2019a) and are referenced throughout this report. In addition, the geotechnical logging data from four vertical diamond drillholes conducted during the Piteau field investigations were reviewed (Piteau, 1997).

Geotechnical and exploration drillholes were used to support the formation of the 3D geological and structural model that is relied on for the stability analyses presented in this report. Reviews of geological logging, Manalta basic geotechnical logging, diamond core photographs and downhole geophysical data part of the current study to evaluate confidence in the geological and structural geology models. Exploration drillholes are presented in Figure 3-1.

Table 3-1: Diamond drilling program summary

Field Program	Drillhole ID	Length[1] (mah)	Dip (°)	Azimuth (°)	Easting[2] (m)	Northing[2] (m)	Elevation[2] (masl)	Geotech Logging	Televiewer Survey (m)
SRK 2019	T19-09D	50.4	-90	000	616982	6051132	982	Yes	4.0 – 49.4
	T19-16D	73.2	-90	000	618128	6050374	973	Yes	4.0 – 71.9
	T19-17D	59.9	-90	000	617674	6050852	953	Yes	7.0 - 58.6
	T19-18D	35.5	-90	000	617292	6050679	997	Yes	0.0 - 34.7
SRK 2018	T18-01D	78.7	-90	000	617841	6050689	958	Yes	16.5 – 78.7
	T18-02D	124.9	-60	042	618412	6050732	928	Yes	12.0 – 124.9[3]
	T18-03D	102.6	-90	000	617080	6051772	904	No	10.5 – 102.6
	T18-05D	180.4	-70	064	617988	6051733	893	Yes	No[4]
	T18-07D	213.7	-90	000	617318	6052051	864	No	7.5 – 213.7
Piteau 1997	T96R-05C	105.8	-90	000	618923	6050031	948	Yes	No[5]
	T96R-28C	120.7	-90	000	618052	6051092	919	Yes	No[5]
	T96R-42C	151.6	-90	000	617377	6051844	868	Yes	No[5]
	T96R-65C	141.0	-90	000	618305	6051041	915	Yes	No[5]
Manalta 1997	T97R-95C	36.0	-90	000	616887	6050567	1019	Basic	No[5]
	T97R-100C	46.0	-90	000	617149	6051492	933	Basic	No[5]
	T97R-108C	81.0	-90	000	617379	6052531	839	Basic	No[5]

[1]: mah = metres along hole, relative to ground surface.

[2]: Datum: UTM NAD83, Zone 9 North. Coordinates and elevation surveyed after completion of program and provided by Telkwa. Values rounded to nearest metre.

[3]: Televiewer survey has poor image quality.

[4]: Televiewer survey not attempted because of drillhole instability.

[5]: Televiewer survey not conducted as part of the field program

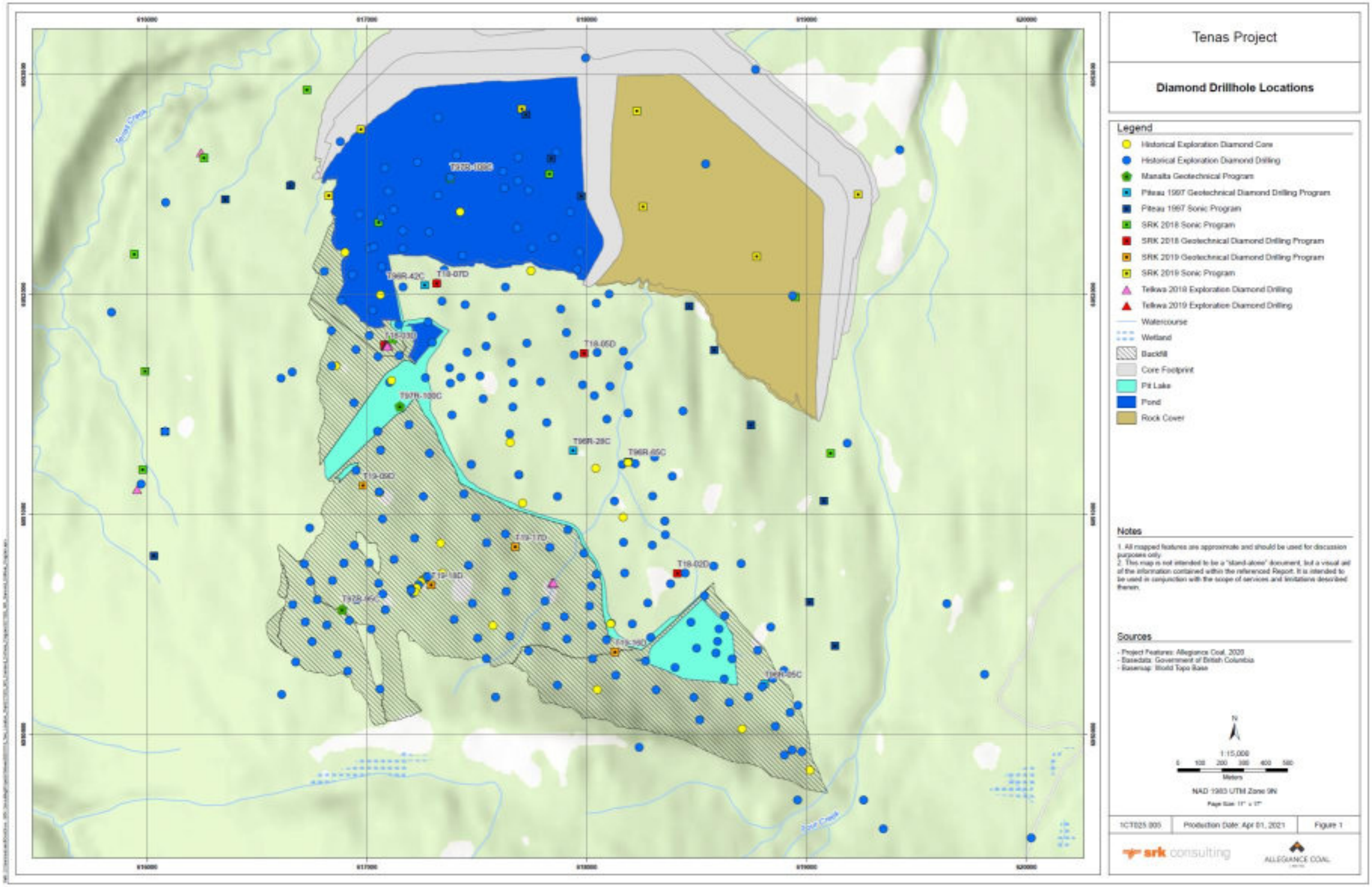

Figure 3-1: Location plan showing diamond drillholes used for geotechnical data collection

4 Foundation Conditions

The following subsections summarize the characterization of the footwall rocks that the backfills are to be construction onto. Full details of the characterization work carried out for the FS pit slope and rock/overburden soil piles are presented in the SRK 2019b and 2019c reports respectively. These two reports should be read in conjunction with the below subsections.

4.1 Footwall Forming Rock Types

Rock types encountered in the geotechnical drilling program comprised Skeena Group mudstone, siltstone, sandstone and coal. It is expected that the mudstones are located below the economic Seam-1, forming the immediate foundations below the backfills. With consideration to this possible weaker mudstone, both a bedding anisotropy and weak layer were evaluated as part of the stability analyses. The weak layer can be representative of a weak bedding structures near the backfill-footwall interface which could promote sliding.

Mudstone (MDST)

Fine grained, bedded, grey and dark grey, assessed fresh to highly weathered, generally estimated very weak to medium strong intact rock strength (R1 to R3). The mudstone is often interbedded with small coal seams and breaks on graphitic coated joints near major coal seams.

Figure 4-1: Example core of mudstone rock type

Siltstone (SLST)

Fine grained, bedded and laminated texture, dark grey, assessed fresh to moderately weathered, generally estimated very weak to medium strong intact rock strength (R2 to R3). The siltstone can be interbedded with small coal seams and breaks on graphitic coated joints near major coal seams.

Figure 4-2: Example core of siltstone rock type

Sandstone (SST)

Fine to medium grained, bedded and massive texture, light to dark grey, assessed fresh to moderately weathered, generally estimated moderate to strong intact rock strength (R3 to R4). Sandstone represents 5% of what was logged.

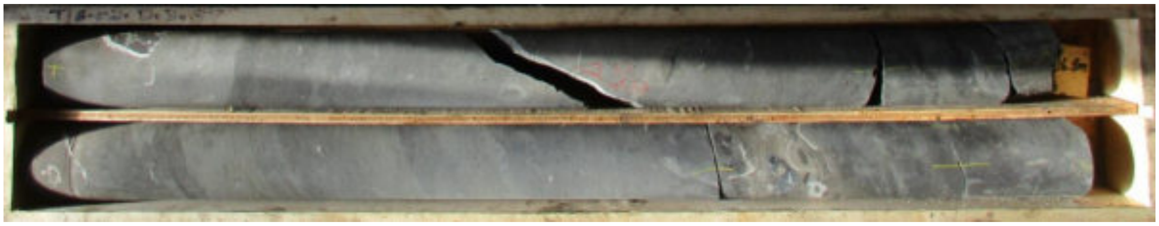

Figure 4-3: Example core of sandstone rock type

4.2 Intact Rock Strength

4.2.1 Estimated Strengths

Field estimates of intact rock strength were carried out during core logging. Logged field estimates of intact rock strength for MDST, SLST and SST are presented in Figure 4-4. For the MDST, SLST and SST rock types, the strength of the rock was noted to be weaker parallel to bedding and would be representative of the shear strength (detailed in Section 4.3).

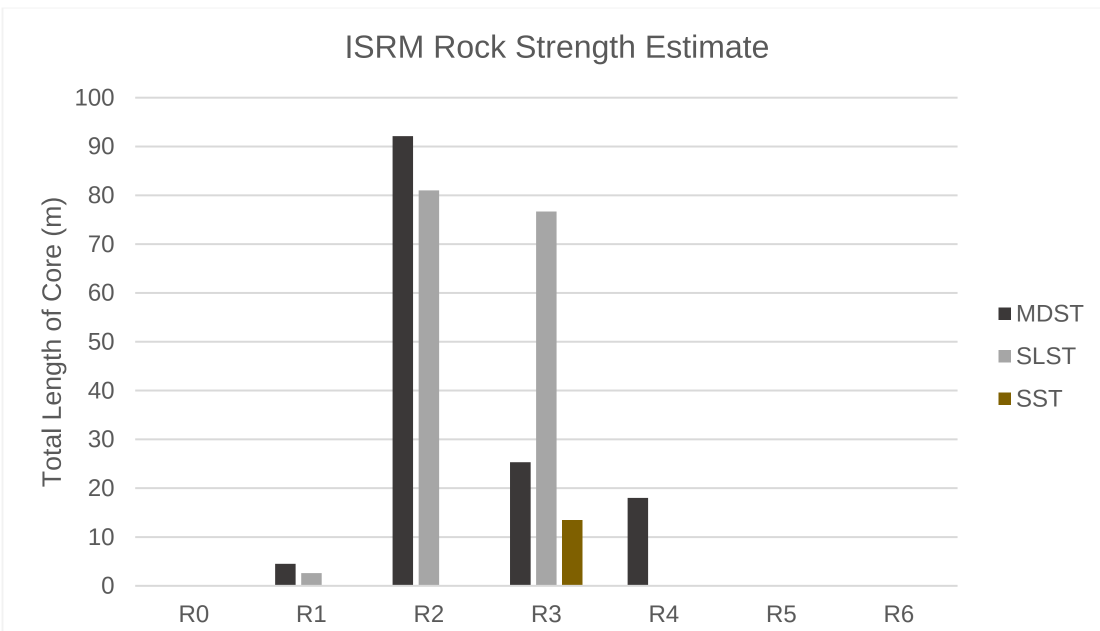

Figure 4-4: Field estimates of intact rock strength by rock type for 2018 field investigation

4.2.2 UCS Testing

A total of 17 Unconfined Compression Strength (UCS) tests were performed on rock core samples collected during the field program. Three samples were selected for additional measurement of Young's Modulus and Poison's Ratio. Stronger strengths are observed in the 2018 dataset as the samples were collected from greater depths with comparison to the 2019 dataset. Laboratory tests results are summarised in Table 4-1. The UCS test values were reviewed with consideration to final Seam-1 footwall location. The average values for the sedimentary rocks below the Seam-1 footwall is 41 MPa.

Table 4-1: Summary of 2018 UCS test results

Drillhole	Depth From (mah)[1]	Depth To (mah)[1]	Rock Type[2]	Dry Density (kg/m^3)	Tested UCS[4] (MPa)	Young's Modulus (GPa)	Poisson's Ratio
T18-02D	122.1	122.4	SLST	2486	28.8	-	-
T18-05D	127.5	127.9	MDST	2541	28.4	-	-
T18-05D	153.9	154.3	MDST	2614	46.4	-	-
T18-05D	172.5	172.9	SLST	2365	~~7.5~~	-	-
T18-02D	60.0	60.6	MDST	2471	34.1	3.2	0.06
T18-05D	54.4	54.8	MDST	2549	57.7	4.9	0.08
T18-05D	112.5	112.8	SST	2253	25.7	2.3	0.21
T18-02D	66.0	66.4	MDST	2503	48.4	-	-
T18-02D	116.7	117.0	SST	2711	75.5	-	-
19-09D	31.2	31.4	SLST	2560	19.8	-	-
19-16D	48.7	48.9	SST	2656	50.1	-	-
19-16D	57.6	57.9	SLST	2587	29.6	-	-
19-16D	61.9	62.1	MDST	2519	43.8	-	-
19-17D	72.2	73.2	SLST	2583	~~11.7~~	-	-
19-17D	45.1	45.4	SST	2609	38.5	-	-
19-18D	51.3	51.5	SLST	2778	16.2	-	-
19-18D	24.3	24.5	SLST	2593	30.5	-	-

[1]: mah = metres along hole, relative to ground surface.

[2]: Rock type logged by Telkwa during sample collection.

[3]: Lab comments and photos indicate sample had a desiccation crack prior to testing, likely compromising integrity of the sample.

4. UCS equivalent for 50 mm specimen calculated using Turk 1986 method

4.3 Discontinuity Shear Strength

A total of seven Direct Shear tests were performed on rock core samples collected during the 2017/2018 field programs. Peak friction angles were estimated from the graphs of shear displacement vs. shear stress. Laboratory tests results are summarised in Table 4-2. The friction angles are representative of the sheared of the HQ3 sized specimen and do not account for waviness across the continuous structures as observed in the field (i.e. increases to the bedding and cohesion at the inter-ramp scale).

Table 4-2: Direct shear test results – rock discontinuities

Drillhole	Depth From (mah)[1]	Rock Type[2]	Feature type	Cohesion-less Peak Friction Angle (°)	Peak Cohesion (kPa)	Residual Friction Angle (°)	Residual Apparent Cohesion (kPa)
T18-05D	128.6	MDST	Joint	31	0	15	0
T18-05D	153.4	MDST	Joint	31	0	15	67
T18-05D	155.5	SLST	Bedding	34	0	23	62
T19-09D	33.0	SST	Bedding	36	0	20	54
T19-16D	67.8	MDST	Bedding	30	0	32	0
T19-17D	42.8	SLST	Joint	45	0	38	120
T19-17D	45.4	SST	Bedding	34	0	33	17

[1]: mah = metres along hole, relative to ground surface.
[2]: Rock type logged by Telkwa during sample collection.

It should be noted that one historic direct shear test was completed (Klohn Leonoff, 1985). The results of the direct shear test conducted on a bedding surface indicated a residual friction angle of about 23º with little cohesion (Klohn Leonoff, 1985). This friction angle appears to be reasonable for weaker or residual bedding surfaces such as those observed in mudstone or carbonaceous siltstone rock types, however, greater strengths may be exhibited in the more competent rock units as indicated by the 2018 and 2019 testing results. A frictional strength of 23º is similar to the residual strength values shown in Table 4-2, and this value was used to represent a possible weak bedding layer in the stability analyses.

4.4 Rock Mass Quality

A rock mass quality assessment using RMR_{89} rock mass classification system (Bieniawski, 1989) has been prepared for the rocks logged in each drillhole. RMR_{89} values were calculated on a geotechnical interval basis. The geotechnical interval RMR_{89} values, along with other logged parameters, from the 2018 and 2019 drilling programs are presented in Table 4-3. In general, the RMR values indicate that the MDST and SLST are representative of a moderate quality rock, and the SST representative of a good quality rock.

Table 4-3: Weight Average RMR_{89} Values by Lithology

Lithology	TCR (%)	RQD (%)	IRS_ENG (MPa)	FF/m	JCR89	RMR_{89}	GSI[1]	Total Length
MDST	95	80	28	5.5	14	56	51	226
SLST	98	89	28	3.5	15	58	53	211
SST	98	97	42	1.0	17	69	64	38

Note: [1] – Geological Strength Index (GSI) equals RMR_{89} – 5

4.5 Bedding Orientation

The Tenas Pit footwalls are be excavated along shallow to moderate dipping bedding structures as shown in Figure 4-5. With consideration to the proposed pit and backfilling, the bedding orientations can be divided into the following:

- **West Footwall (Phase 1, 2 and 4 Pits):** Shallow west-dipping sedimentary sequences along the west limb of the deposit. Bedding dip angles ranged from 0° to 12° with normal and thrust faulting locally offsetting the continuous bedding structures. The coal geology model indicates bedding can be vertically displaced to approximately 15 m within the proposed pit excavation.
- **Southeast Footwalls (Phase 3 Pit):** Shallow to moderate northeast and southwest dipping sedimentary sequences within the tighter synclinal fold. Bedding dips range from 20° to 45° along the west limb of the fold and from 15° to 40° within the east limb of the fold which becoming tighter in the southern most pit area.

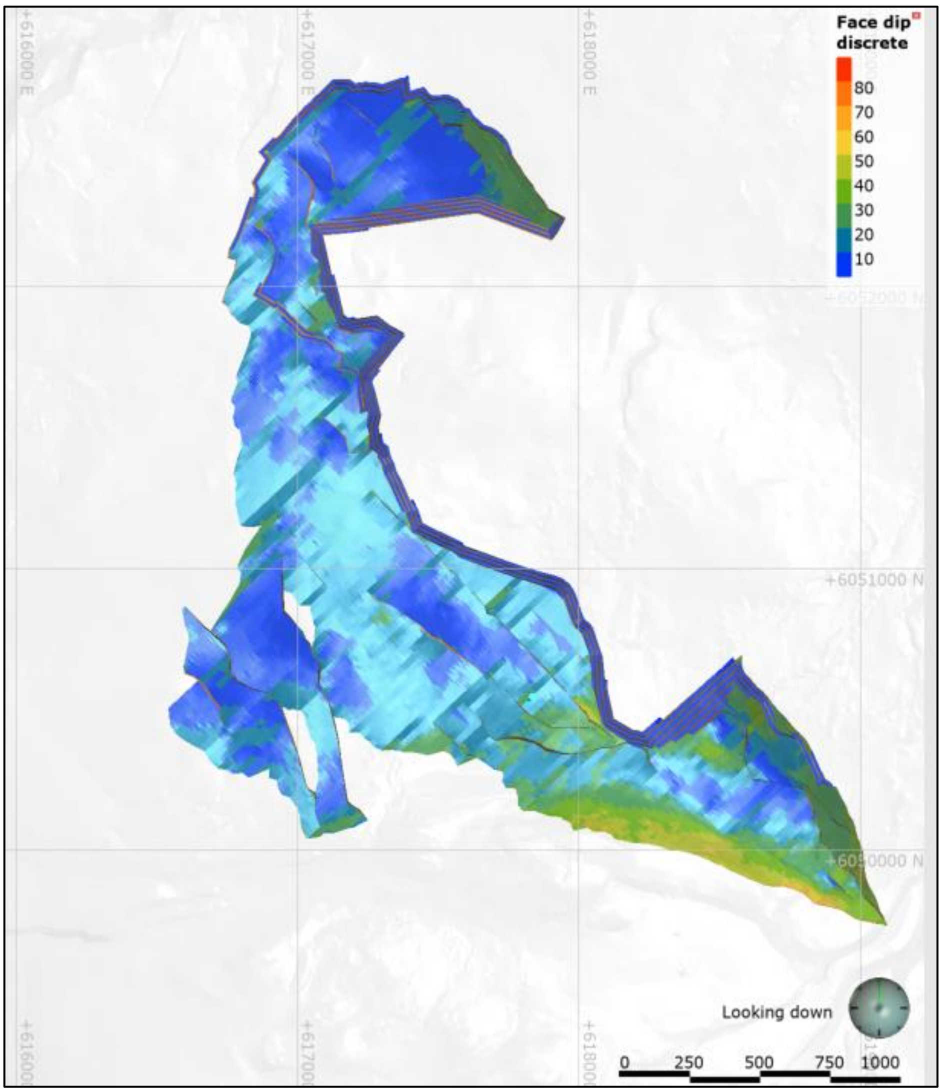

Figure 4-5: Contour of footwall slope angles

4.6 Major Structures

The 3D coal geology model and deposit structural geology has been defined by Telkwa using available exploration drillhole data (Telkwa, 2018). The major structures at Tenas are characterised by continuous north-south striking thrust and normal faults. A total of 36 fault segments have been interpreted. The sense of movement has been interpreted by Telkwa geologists, largely based on the offsets between each coal seam. Within the proposed pit area, higher fault frequencies are expected in the southeast where the folding becomes tighter and bedding inclination is steeper. With consideration to the backfilling, interim slopes should be advanced over an undercut footwall section to reduce the probability of a bi-linear sliding mechanisms through the backfill rock and along the exposed bedding surfaces.

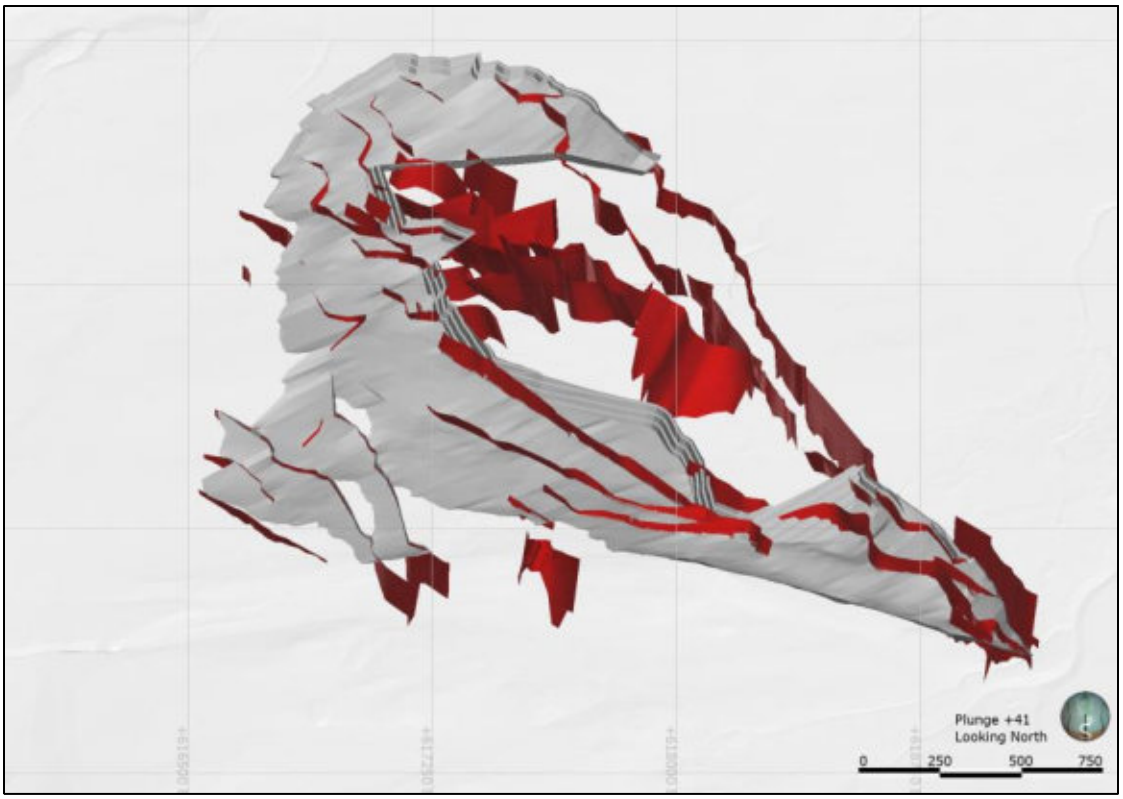

Figure 4-6: 3D fault model shown on the proposed pit (Telkwa, 2018)

5 Backfill Hazard Classification

A semi-qualitative Waste Dump and Stockpile Stability Rating and Hazard Classification (WSRHC) using guidance provided by Hawley and Cunning (2017) was completed for each backfill areas. The hazard ratings for each final pile design are summarized in Table 5-1. The results of the WSRCH indicate that the backfills are considered a low hazard class.

However, the risks associated with each backfill area may be elevated based on the location to any nearby active mining face, and the relative backfilling slope performance. A greater emphasis on managing the active work area will be required for the Phase 3 Backfill due to the proximity of the nearby pit excavations.

Table 5-1: Summary of backfill stability rating and hazard classification

Pile Name	Construction Material	Dump Stability Rating and Hazard Classification			
		Engineering Geology Index	Design and Performance Index	Dump and Stockpile Stability Rating	Hazard Rating (Class)
Phase 1 Backfill	NAG Rock	37	33.5	70.5	Low
Phase 2/3 Backfill	NAG Rock	36	29	65	Low
Phase 4 Backfill	NAG Rock	37	31	68	Low
Phase 5/6 Backfill	NAG Rock	37	31	68	Low
Phase 7 Backfill	NAG Rock	32.5	30.5	62.5	Low

6 Stability Assessment

Slope stability analyses were carried out to evaluate the backfills during construction, at the end of construction and for the mine closure configuration. The analyzed foundation conditions and groundwater conditions is based on the results of the 2019 FS geotechnical characterization hydrogeological modelling work (SRK, 2019d).

Two and three-dimensional limit equilibrium (LE) analyses were carried using the Rocscience programs Slide2 and Slide3 (Rocscience, 2020), under for both static and pseudo-static conditions (end of constriction and reclamation). Deterministic and Probabilistic analyses were carried out using the GLE/Morgenstern-Price method of slices. The GLE/Morgenstern-Price method is considered a rigorous method which satisfies both moment and force equilibrium.

6.1 Slope Instability Mechanisms

With consideration to the foundation conditions and expected slope performance, the following instability mechanisms were considered throughout the analyses and design work.

1. A deep-seated, overall slope instability that can be a combination of:
   (a) An instability through a significant volume of the rock materials; or.
   (b) An instability through a significant volume of the rock materials and sliding along the footwall contact or weaker bedding layer;
2. A shallow slope face instability through the dump materials caused either by over-steepening of the dump materials and/or concentrated weak rock layering due to end-dumping near the slope face

Shallow slope face instabilities should be expected throughout construction as material settles and should be managed in accordance with the implemented standard operating procedures.

6.2 Acceptance Criteria

The recommended criteria are based on levels of confidence in the understanding of site conditions and material parameters and the consequences of instability in accordance with Hawley and Cunning (2017) and presented in Table 6-1. The acceptance criteria were compared against the results of the analyses for a deep-seated failure mechanism. For the backfills at Tenas, a *moderate* possible failure consequence and a *moderate to high* data confidence were identified. A minimum static Factor of Safety (FOS) of 1.3 and a pseudo-static FOS of 1.05 are recommended for end of construction backfill configurations.

In addition, selected short-term, during construction designs were analyzed, and these results were compared against a minimum design FOS values ranging from 1.1 to 1.3 in accordance with British Columbia Mine Waste and Rock Pile (BCMWRP) Interim Guidelines (1991). With consideration to the Phase 3 Backfill, a minimum design FOS of 1.3 was adopted for the interim construction stages.

Table 6-1: Suggested stability design criteria (Hawley and Cunning, 2017).

Possible Consequence	Data Confidence[1]	Static Analyses		Pseudo-Static Analyses
		Minimum FOS	Maximum POF	Minimum FOS
Low	Low	1.3-1.4	10-15%	1.05-1.1
	Moderate	1.2-1.3	15-25%	1.0-1.05
	High	1.1-1.2	25-40%	1.0
Moderate	Low	1.4-1.5	2.5-5%	1.1-1.15
	Moderate	***1.3-1.4***	***5-10%***	***1.05-1.1***
	High	1.2-1.3	10-15%	1.0-1.15
High	Low	>1.5	<1%	1.15
	Moderate	1.4-1.5	1-2.5%	1.1-1.15
	High	1.3-1.4	2.5-5%	1.05-1.1

Note: [1] - Data confidence in dump material and foundation shear strength parameters, 3D overburden model

6.3 Geotechnical Parameters

Geotechnical material parameters were selected to represent the foundation and rock materials. Material parameters used in the stability analyses were based on the field investigation results, laboratory testing, and experience with materials showing similar characteristics (SRK, 2019a and SRK, 2019b). The material parameters used in the stability analyses are presented in Table 6-2.

Table 6-2: Summary of geotechnical material parameters

Material	Type	Failure Criterion	Unit Weight (kN/m^3)	Mohr-Coulomb Strength Parameters		Hoek-Brown Strength Parameters		
				Friction Angle (°)	Cohesion (kPa)	UCS (MPa)	GSI	mi
Foundation	Till	Mohr-Coulomb	18	30	5	-	-	-
	Weak Bedding Layer	Mohr-Coulomb	26	23	40	-	-	-
	Bedrock	Generalized Hoek-Brown	26	-	-	42	46	7
	Bedding	Mohr-Coulomb	26	31	0	-	-	-
Backfill	Coase Rock	Mohr-Coulomb	19	37	0	-	-	-

With consideration to possible weaker mudstone bedding planes located within the immediate footwall forming rocks (Figure 6-1), a lower shear strength material was applied to the backfill-footwall interface. Where the footwalls are steep, such as the Phase 3 West Footwall, weaker bedding structures are expected to detach from the exposed benches during excavation and were not included in the analyses. A weaker bedding layer was included in all other models, including the lower elevations of the Phase 3 sections. The weaker bedding layer is represented by a residual shear strength of 23 degrees which is consistent with the completed laboratory testing (SRK, 2019a and SRK, 2019b), and the empirical information noted by Klohn Leonoff (1985).

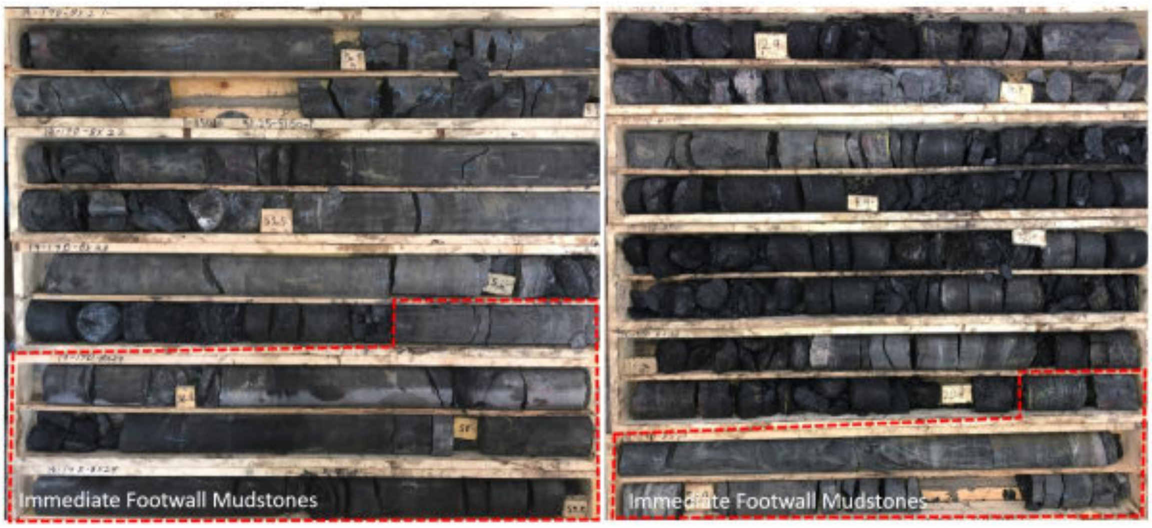

Figure 6-1: Example of Mudstones immediately below the design footwall (19-17D, left and 19-09D, right)

6.4 Groundwater and Pit Lake

The groundwater influence on the foundation and dump material characteristics is critical to understanding the expected slope stability. The coarse-grained materials in the pit backfills are expected to be free-draining, and it has been assumed that elevated groundwater pressures will not develop during and post-construction. At the end of the pit excavation, groundwater levels due to pit inflows are expected to contribute to the formation of pit lakes. At that time, the backfills will be buttressed against the opposing bedrock pit and the lower slopes saturated. Predicted pit lake elevations are shown in Figure 6-1 and are included in the analyses.

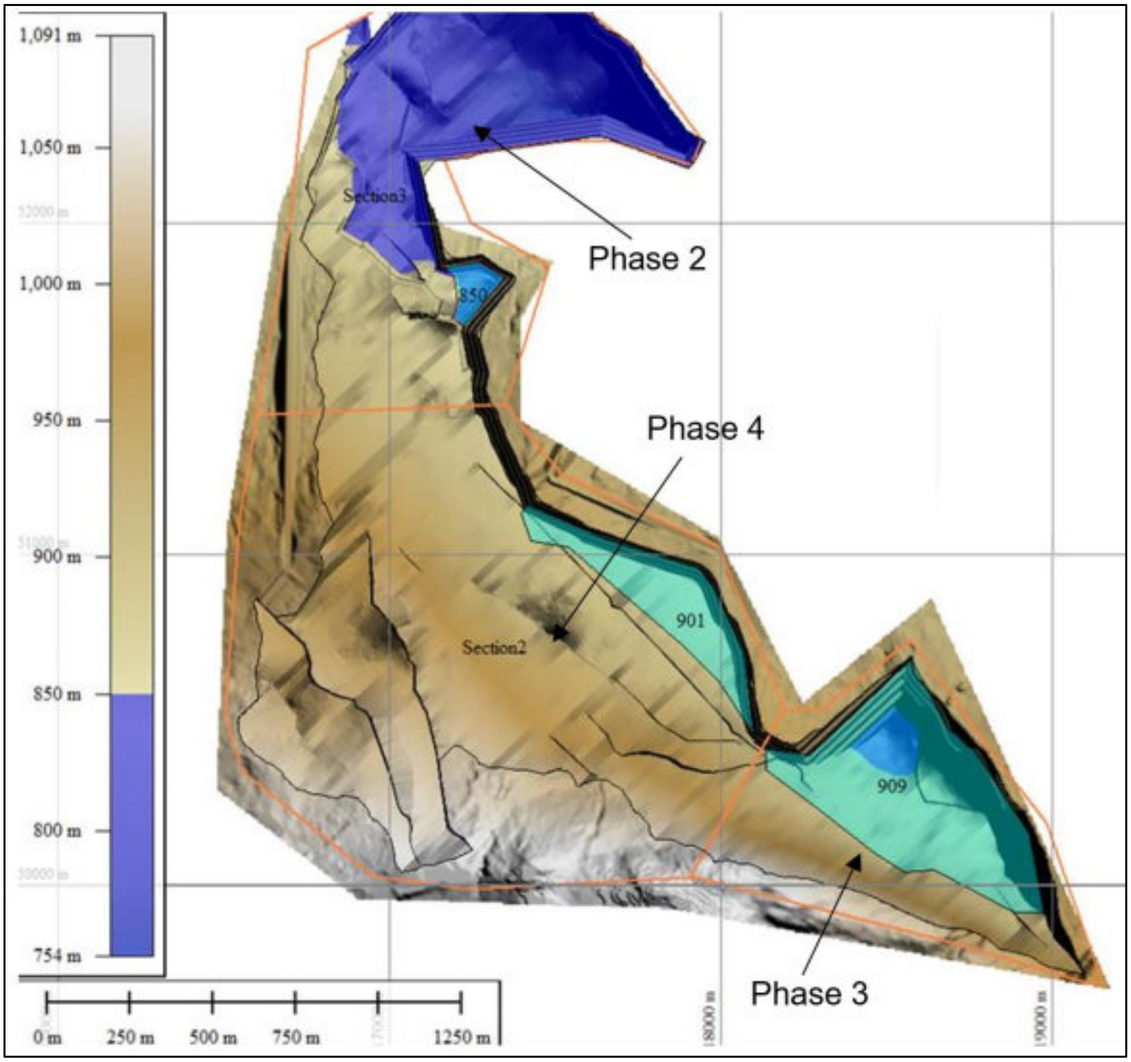

Figure 6-2: Predicted pit lake elevations at end of mining

6.5 Seismicity

Pseudo-static analyses have been performed to assess the end of construction and reclamation configurations consideration to an earthquake event. Probabilistic peak ground acceleration (PGA) values determined by the 2015 National Building Code of Canada (NBCC) were reviewed. The PGA values for 2% probability of exceedance in 50 years; i.e., the 2,475-year return period earthquake event is 0.049 g (NRC, 2019). For these analyses, it is common practice to use the 50 percent of the peak ground acceleration (PGA) where there is no threat of liquefication or severe loss of strength under seismic shaking. The rock is not expected to be susceptible to liquefication of a severe loss of shear strength through the bedrock foundations. A PGA of 0.025 (one half of 0.049g) was used in the pseudo-static analyses.

Given the relatively low seismic hazard and 20-year mine operating life, the 2,475 years is considered appropriate time period for the end of construction/reclamation conditions. The 2,475 return period was also selected due to the reduced consequence of failure within the pit boundaries and that backfills are buttressed following completion of mining. The return period for the operational design earthquake is recommended by the BCMWPRC (1991) is an earthquake event with a 10% probability of exceedance in 50 years (with a corresponding return period of 475 years).

6.6 Stability Sections

A total of five 2D sections were selected for stability analysis and represented the steepest footwall slope foundations and where the backfill slopes are located closer to the active mining areas. The section locations are shown in Figure 6-3 and include:

- Section 1: Phase 2 Backfill. Although this section was not cut perpendicular to the footwall slope, the purpose was to evaluate the backfill set-back from a locally undercut footwall bench located above the West Management Pond.
- Section 2: Phase 4 Backfill (Figure 6-4).
- Section 3 to 5: Phase 3 Backfill.

Two 3D models were also developed to analyze the interim backfilling sequence through the Phase 3 Pit area (Figure 2-2). An interim excavation and backfilling stage is shown in Figure 6-5.

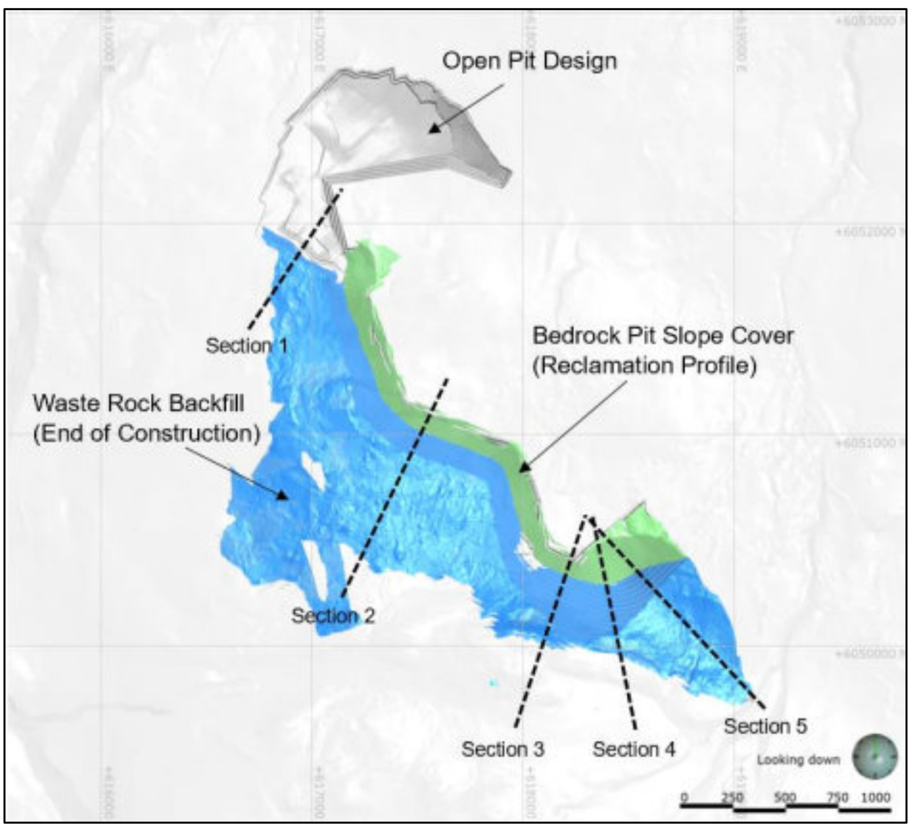

Figure 6-3: Location of overburden and rock stability sections

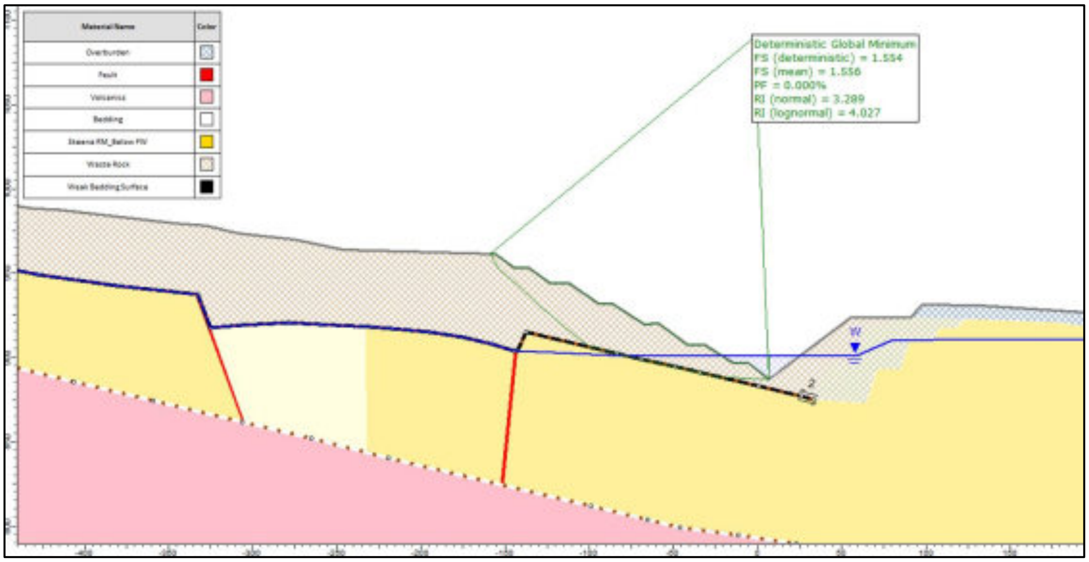

Figure 6-4: Example backfill section through Phase 4, Section 2

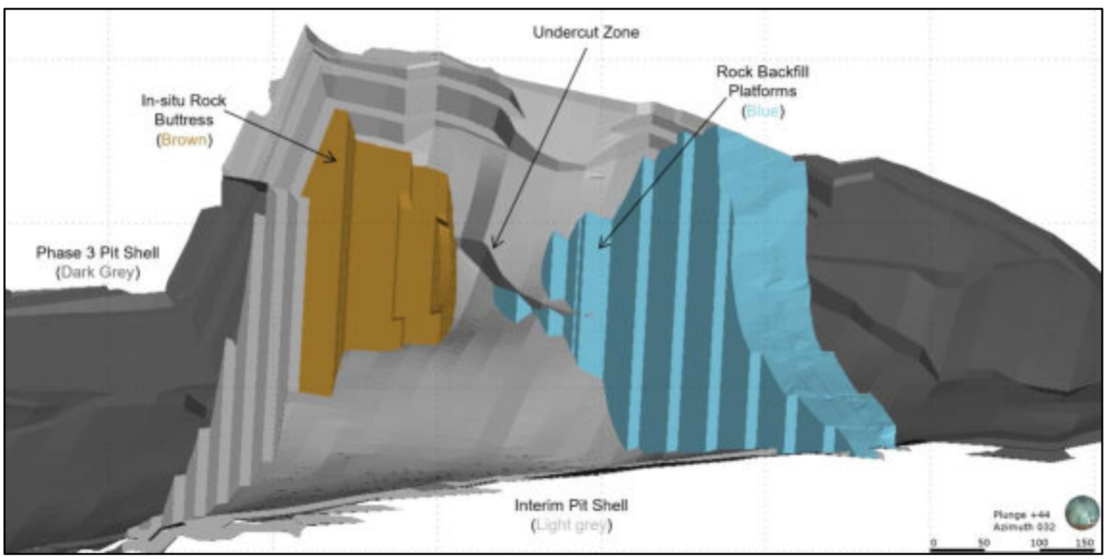

Figure 6-5: Schematic figure showing interim endwall and backfilling within the Phase 3 Pit.

6.7 Results

6.7.1 2D Limit-Equilibrium Stability Analyses

The results of the 2D stability analyses indicate the proposed rock backfill slopes are expected to achieve the design FOS and POF acceptance criteria, as outlined in Table 6-1. The analyses evaluated a deeper-seated failure mechanism through the backfill and/or sliding along the footwall foundations that could be considered design constraining and pose a significant risk to people and equipment. The results for are summarized in Table 6-3. Furthermore, pseudo-static analyses were carried for the *end of construction* and *reclamation* configuration, and the results indicate that acceptance design stability conditions are expected under the 2,475 return period event.

Several during construction analyses were carried out to evaluate backfill performance while they are being built also satisfied the minimum adopted FOS of 1.3. Although shallow slope instabilities can exhibit a lower FOS or POF than those indicated in Table 6-3, these lower consequence instabilities should be managed by the operational teams as the backfills are constructed.

Table 6-3: Summary of 2D stability analyses for pit backfills

Backfill	Stability Section	Modelled slope Height (m)	Modelled Footwall Slope Angle (°)	Construction Stage	Groundwater Conditions	Static Conditions				Pseudo-static Conditions	
						Minimum Acceptable Design FOS	Minimum Modelled FOS	Maximum Acceptable Design PoF	Maximum Modelled PoF	Minimum Acceptable Design FOS	Minimum Modelled FOS
Phase 2	1	20	5	End of Construction	Footwall elevation	1.3	1.3	10%	3%	1.1	1.3
Phase 4	2	65	10	During Construction	Footwall elevation	1.3	1.8	10%	<1%	n/a	-
		80	13	End of Construction	Footwall elevation	1.3	1.6	10%	<1%	1.1	1.5
		80	13	Mine Reclamation	Pit Lake (901 m)	1.3	1.6	10%	<1%	1.1	1.5
Phase 3	3	95	41	End of Construction	Footwall elevation	1.3	1.4	10%	2%	1.1	1.3
		120	41	Mine Reclamation	Footwall elevation	1.3	1.4	10%	<1%	1.1	1.3
		120	41	Mine Reclamation	Pit Lake (910 m)	1.3	1.3	10%	5%	1.1	1.2
	4	115	39	End of Construction	Footwall elevation	1.3	1.4	10%	4%	1.1	1.3
		135	39	Mine Reclamation	Footwall elevation	1.3	1.4	10%	4%	1.1	1.3
		135	39	Mine Reclamation	Pit Lake (910 m)	1.3	1.4	10%	<1%	1.1	1.3
	5	75	5	During Construction – Stage 3	Footwall elevation	1.3	1.5	10%	<1%	n/a	-
		80	5	During Construction – Stage 6	Footwall elevation	1.3	1.5	10%	<1%	n/a	-
		90	12	During Construction – Stage 8	Footwall elevation	1.3	1.4	10%	<1%	n/a	-
		100	12	End of Construction – Stage 9	Pit Lake (910 m)	1.3	1.3	10%	<1%	1.1	1.2
		100	12	End of Construction – Stage 9	Mine reclamation	1.3	1.3	10%	6%	1.1	1.2

Note: [1] – measured long analyzed slope

6.7.2 3D Limit-Equilibrium Stability Analyses

The results of the 3D stability analyses indicate the proposed interim Phase 3 backfill slopes are expected to achieve the minimum FOS of 1.3 which was adopted for the construction conditions (Table 6-4). The modelled failures are representative of a consequential of a sliding surface through the backfill rock slopes and/or along the underlying footwall contact that could adversely impact the downslope mining activities, as shown in Figure 6-3. The modelled failures surfaces are similar in extent and resulting FOS to those evaluated using the 2D LE methods (Table 6-3).

Table 6-4: Summary of 3D stability analyses for the Phase 3 pit backfills

Backfill	Interim Construction Stage	Modelled slope Height (m)	Construction Stage	Modelled Groundwater Conditions	Static Conditions	
					Minimum Acceptable Design FOS	Critical Modelled FOS
Phase 3	3	75	During Construction	Footwall elevation	1.3	1.5
	6	80	During Construction	Footwall elevation	1.3	1.5

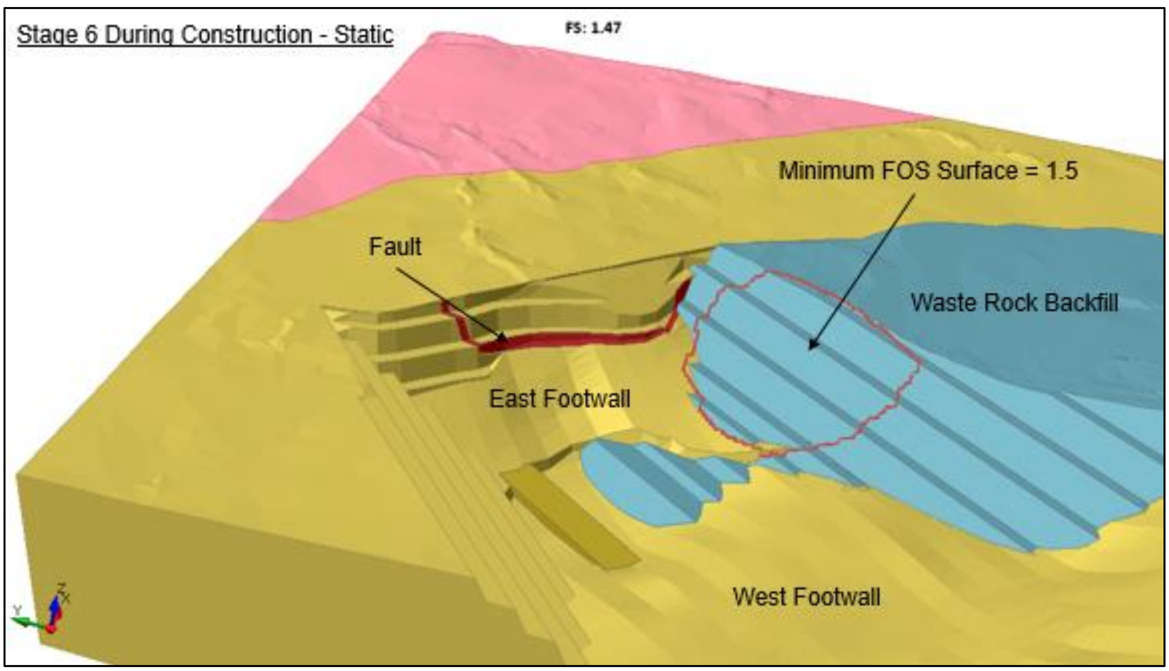

Figure 6-6: 3D Slide analyses results for the Phase 3, Stage 6 Backfill slope.

6.8 Discussion of Results

6.8.1 Backfilling over locally undercut footwalls

The in-pit dumping of rock will be advanced down the footwall slopes to buttress the previously excavated pit walls. Analyses were performed to assess the during construction in Sections 2 and 5. These analyses indicate acceptable results for the design conditions assessed (Table 6-3 and Table 6-4) and it is recommended that the risks associated with short-term stability conditions be managed with appropriate operational procedures, including inspections, loading rate guidelines, material quality evaluation and monitoring.

An important operational procedure needs to include the advance over and the local buttressing of any previously undercut footwall sections within Phase 1, 2 or 5 pits. The fault model shows where possible undercutting and/or offset may be located across the ultimate pit footwalls (Figure 4-6). Loading of any locally undercut benches could contribute to a foundation planar sliding mechanism downslope below the backfill pile. A conceptual example of the backfill rock loading on a locally undercut footwall slope is shown in Figure 6-7.

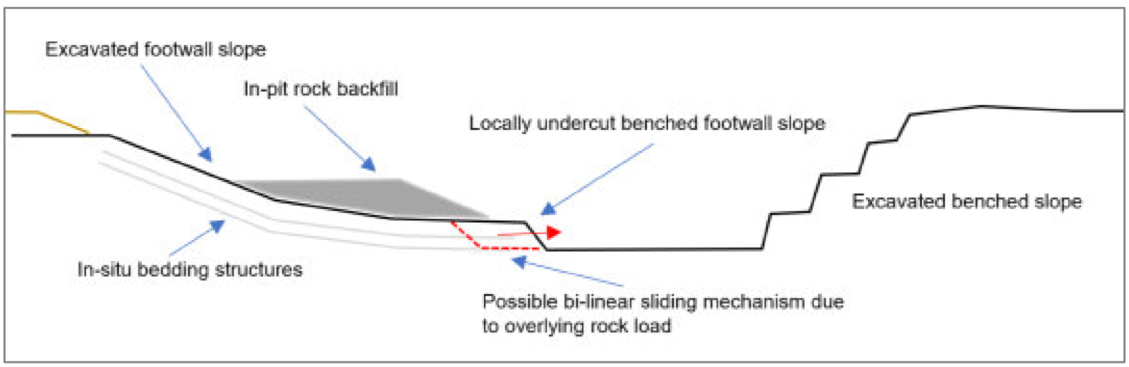

Figure 6-7: Conceptual example of the backfill rock loading on a locally undercut footwall slope

6.8.2 Backfilling near active mining areas

Stability analyses for the construction phase indicate that acceptable design conditions are expected. The results summarized in Table 6-3 and Table 6-4 are consistent with previous construction phase backfill analyses completed for the Tenas FS work (SRL, 2019c). However, there still remains a risk to workers or equipment within the open pit due to a slope face slumping, rock roll out or an unexpected instability event where backfills are constructed near access ramps and excavation areas. The mine plan has incorporated the separation of active backfilling operation from excavation blocks to reduce such hazards. The need to submerge PAG rock into the Management Ponds location in the north, naturally contributes the separation of mining areas.

The backfill platforms have been designed to be 20 m high with 10 m width catch benches to an overall 2:1 (horizontal:vertical) slope profile. Where bottom-up construction methods are adopted, the catch benches will also include 2 m berm near the platform crest to mitigate any rock roll from future upslope backfill construction. The use of the shallower overall slope profile and benched configuration is to improve the stability conditions, and provide containment for any rock roll out.

With consideration to the Phase 3 mining sequence, the mine plan is designed to limit the height of the platforms and position ramps to reduce the potential for slope hazards, including possible run-out from a multiple-bench slump and rock roll out. The staged construction is intended to achieve the following geotechnical objectives:

- Limit the height of backfill and access lifts to mitigate against instability or run-out.
- Control ramp access to prevent exposure to boulder roll-out hazards.
- Construct 2 m berm near the platform crest to mitigate any rock roll from future upslope platforms.
- Manage the backfill advance direction so that active faces are directed away from mine operations.
- Sequence backfill configuration to maintain offsets (both vertical and horizontal) and/or barriers between the mining benches and dumping areas.
- Manage the backfill material quality with competent coarse rock being used to construct the lower platforms and exposed slope faces.
- Conduct tactical backfilling operations at times where there is no nearby excavation activities.

6.9 Run-Out Risks

6.9.1 Summary

The results of the stability analyses indicate that acceptable design conditions for a deeper-seated failure event are expected during construction and in the longer-term for the proposed backfills. This is primarily due to the shallow dipping bedrock foundations and lower overall backfill slope heights. Given the above considerations, a failure with a significant volume is not expected to contribute to a consequential run-out event that can adversely impact people and equipment.

However, it is acknowledged that there is considerable historical waste rock runout record from coal mines in British Columbia. The majority of the recorded British Columbia run-outs have occurred in an ex-pit setting where materials have been channelized, fluidized by substrate and can flow to significant distances (Hungr, 2002). At Tenas, the in-pit backfill platforms are relatively wide and placed onto uniformly shallower sloping footwalls which would encourages spreading of overland debris from an early stage (Hungr, 2002), should an unlikely event occur. The maximum recommended platform slope heights is 40 m which can further reduce failure volumes of a shallower slumping event.

A primary geotechnical goal for the Phase 3 excavation/backfill strategy is to reduce the mining interaction with potential stability/run-out risks as discussed in the previous sections. The backfill platform heights are limited to 20 m and an overall 2:1 slope profile has been incorporated in the Phase 3 mine plane. In the most part, the excavation working benches are to be located above the pit floor and provides an additional run-out buffer (Figure 6-8).

Given the need to haul PAG materials from the Phase 3 Pit to the Management Ponds, there is reduced production pressures to continually short-haul rock between the active excavation face and backfill. Therefore, TCL have the opportunity to tactically place rock (i.e. during night-shift) to minimise mining interactions.

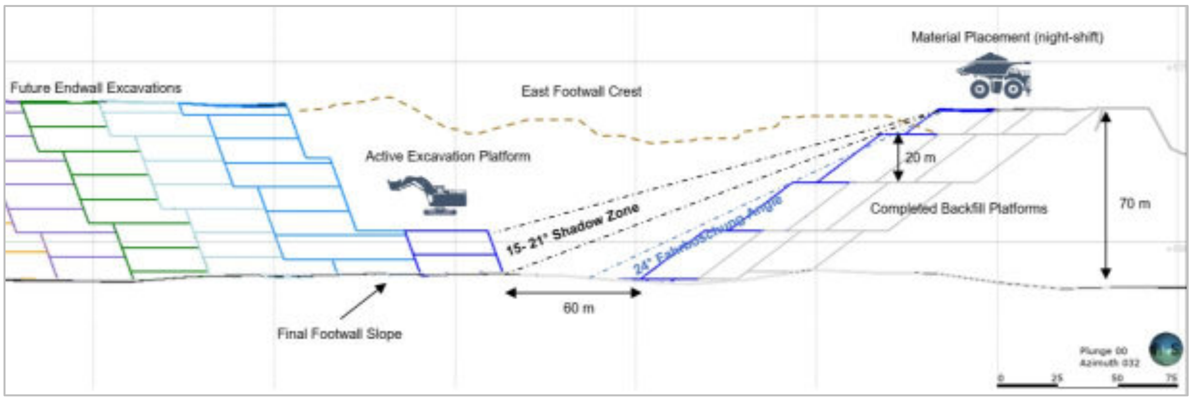

Figure 6-8: Schematic showing the Phase 3 Pit excavation and backfilling sequence

6.9.2 Analyses and Results

With respect to a multi-bench instability event, a 2D run-out analyses was carried out using the software package Dan-W (Hungr, 2010). A multi-bench instability assessment is expected to exhibit acceptable design stability conditions, however, a scenario was modelled to evaluate the sensitivity of an event such as a larger slump associated with layers of poor quality rock, end-dumped close to an active face. The lower-platforms are to be constructed with bottom-up construction and the upper materials end-dumping during times where there is no downslope mining activities. A failure volume based on multi-platform event from the upper platforms was modelled using Dan-W, as shown in Figure 6-9.

The run-out was modelled using frictional behaviour parameters due to the unconfined sliding surface and expected characteristics of the coarse, granular, free draining backfill rock. The backfill rock parameters are based on empirical work carried out by Golder and Hungr (1994) as presented in Table 6-5. The results are representative of a slumping event with a resulting Fahrboschung of 24 degrees representing the angle from failure scarp to run-out toe (Figure 6-9). The analyses results show the multi-bench event to slump and sit above the lower backfill platforms with some rock roll-out down the slope face. The Fahrboschung is steeper than the shadow angle which suggests that a multi-bench run-out event from the upper backfill platforms would lie outside of potential endwall excavation work areas (Figure 6-8).

Table 6-5: Summary of modelled run-out parameters

Type	Mobilization/ Substrate Criterion	Unit Weight (kN/m³)	Frictional Parameters		Pore Pressure Coefficient
			Dynamic Friction Angle (°)	Internal Friction Angle (°)	
Coarse Backfill Rock	Frictional	19	32	37	0.1
Footwall Rock	Frictional	25	31	31	0.1
Endwall Rock	Frictional	25	45	45	n/a

Note: [1] – pore pressure coefficient of 0.1 applied to the coarse backfill road, however, expected to be free-draining

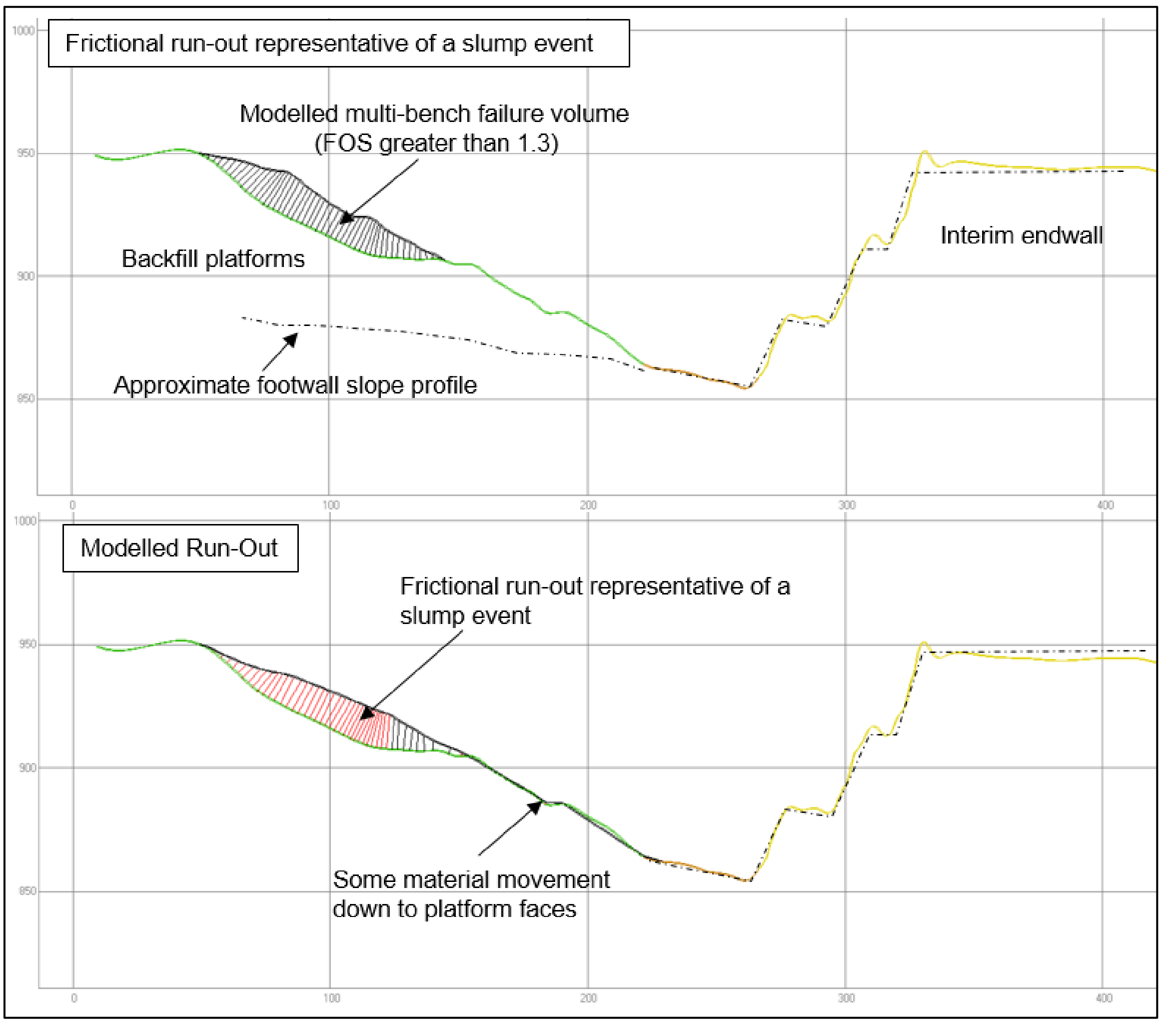

Figure 6-9: Dan-W run-out model and results from a multi-bench backfill slope failure in Phase 3.

7 Operational Considerations

7.1 Construction

The following subsection details operational considerations related to the construction and monitoring of the backfill slopes. These aspects are to be expanded as part of the TCL Ground Control Management Plan (GCMP).

Standard Operating Procedures

Standard operating procedures for the backfill construction should be implemented to improve the safety for operating staff who are tasked with building the backfills in the pit. The standard operating procedures should be implemented prior to construction and approved by a qualified geotechnical engineer.

Methodology

The backfills are initially constructed over the shallow to moderately sloping footwalls. There may be some shallow slumping as the backfill rock materials are initially placed and loaded; this can be considered a hazard to mine operations during the construction. To reduce the risk of instability, bottom-up construction methods are recommended where footwalls are steeper and/or located closer to the active mining areas. Bottom-up construction is to be implemented within the lower Phase 3 Pit backfill platforms.

Material Quality

The quality of materials should be managed during the backfill construction and especially during the placement within Phase3 Pit. Geotechnically-trained personnel should regularly characterize the materials that are being excavated from the pit and hauled to the backfill slopes. This includes any concentration and rapid advancement with weaker rock. Any overburden should be stockpiled and placed above the earlier completed backfill platforms or placed on completed backfill lifts. Strict controls should be implemented to ensure that overburden or weak rock (i.e. carbonaceous mudstones) are located away from the backfill foundations and slope faces.

Advance Rates

The backfill headings are generally wide and advance rates should be more-easily managed with the spreading of material across the crest widths. However, there may be circumstances where concentrated material placement occurs over a short duration. The advance rates will need to be managed and inspected by mine operations regularly.

A case study stability chart for pile height versus advance rate is provided in Figure 7-1 for short-term mine planning guidance (Hawley and Cunning, 2017). The actual stability relationship between advance rate and pile height is site-specific and can be determined from a site geotechnical inspections and monitoring data.

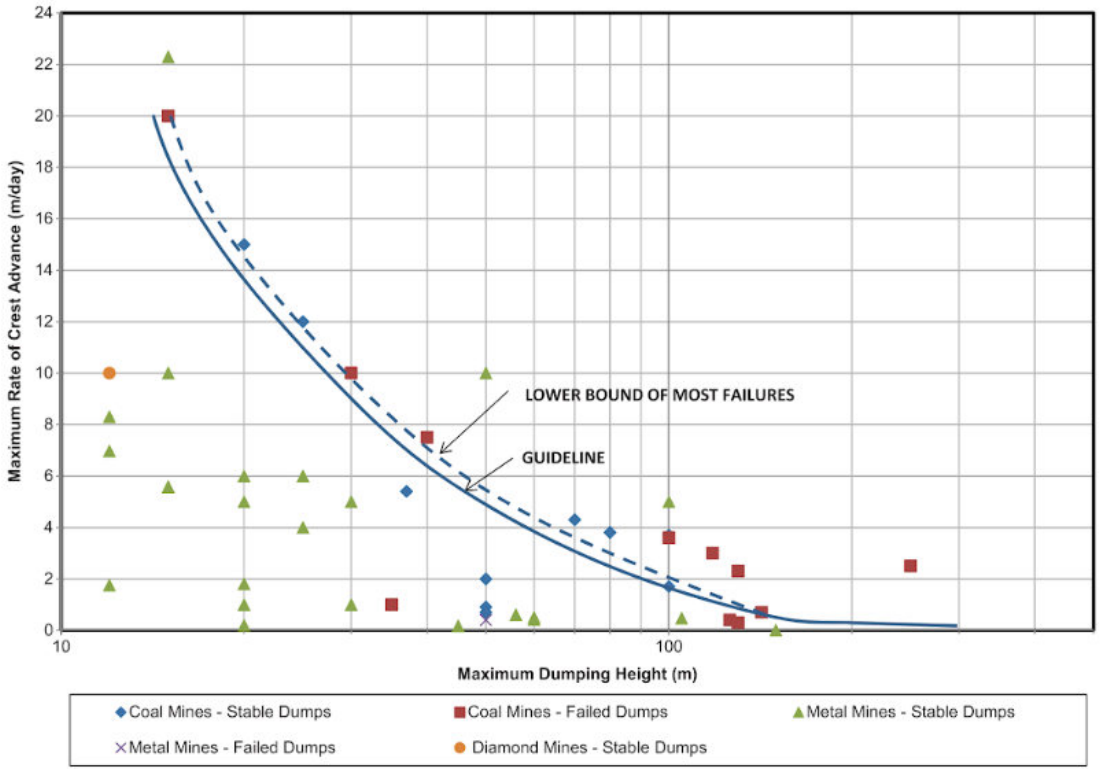

Figure 7-1: Plot of dump crest advance rate versus dump height (Hawley and Cunning, 2017)

<u>Seasonality</u>

During the winter seasons, the shallow foundation conditions can become frozen. At some mine sites within British Columbia, instabilities have occurred where foundations have been quickly loaded during winter and the shallow frost zone has limited time to dissipate any elevated pore pressure, resulting in a loss of shear strength. Although unlikely within the exposed footwall rock, this condition can not be discounted where rock is less durable, and saturation prior to winter is observed.

7.2 Monitoring

Implementation of a tactical monitoring program will be needed to validate the design stability conditions detailed in this report and provide data to reliably predict stability risks. For smaller operations like Tenas, visual inspections by geotechnical-trained mine personal are critical to identifying any potential slope hazards. The backfill platforms should be inspected for signs of deformation, including for example tension cracks (i.e. settlement), over-steepening, and bulging Third-party inspections should also be carried out by a qualified geotechnical engineer.

<u>Wireline Slope Extensometers</u>

Wireline extensometers can be used to monitor active backfills which are considered a higher risk, as determined by the visual inspections and observed slope performance. Wireline extensometers should be installed across the front of the higher backfill crests to monitor the development of cracks and displacement rates (Figure 7-2). If settlement rates show an increasing trend, consideration would be given to the following:

- Reducing the dumping rates.
- Prioritising the addition of higher quality rock material to the dump.
- Spreading the rock over a wider strike length of crest.
- Temporary closure of the dump.

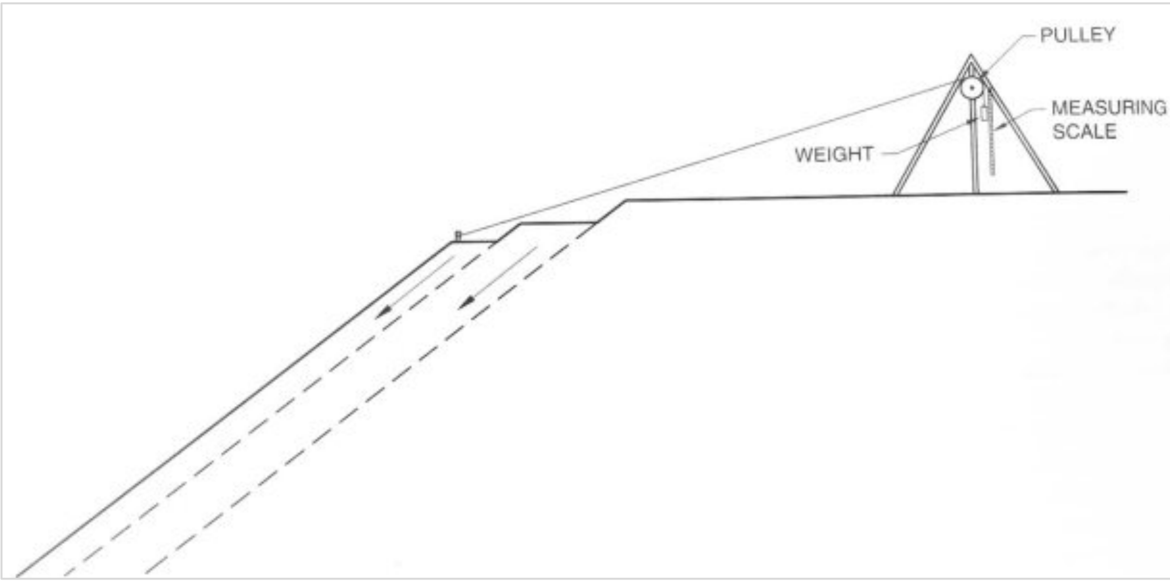

Figure 7-2: Schematic of a manual wireline extensometer (Hawley and Cunning, 2017)

<u>Global Navigation Satellite System (GNSS) Stations</u>

Similar to wireline extensometers, GNSS stations can be used to monitor displacements near the crest of the backfill during construction. The GNSS system should be considered where there are risks of deeper failure mechanisms within completed backfill platforms. This is mainly due to the risk of losing a GNSS systems that is located near a dumping crest that is exhibiting settlement or slumping.

<u>Prism Monitoring</u>

The prims should be placed along the crests of the pit slopes and spread evenly across benched slopes that will be exposed in the medium to long-term. This includes the crests and/or benches along the steeper and higher footwall slopes, such as the benches incorporated into Phase 3. The prisms should be reviewed during the active backfilling operations. For any closed backfill that is considered a higher risk, prisms should be considered as an appropriate monitoring tool.

Prisms should be surveyed from a base station set-up by an experienced survey engineer. The base stations need to be on stable ground and should be made of concrete. The base stations need to be located so that they are largely looking perpendicularly at the higher risk slope areas to reduce survey error. Regular “shooting” of prisms should be done, factoring in dust, time of day, angle of prism relative to total station etc to ensure accurate data is collected.

Groundwater Monitoring

A groundwater monitoring network has been established at Tenas, and will need to be increased to tactically monitor the pit walls. The existing standpipe piezometers and new vibrating wire piezometers should be regularly monitored to validate the groundwater assumptions used in the stability analyses, including the required slope depressurization of pit walls. Seasonality trends observed within the monitoring data should be tested with ongoing stability analyses as mining progresses. This is important for the evaluation of pore pressures within the steeper footwalls prior to and during placement of rock. A future evaluation of the steeper footwall stability conditions should be carried out prior to commencement of backfilling operations within the Phase 3 Pit.

8 Conclusions

A slope stability assessment for the pit backfills has been carried out utilizing the guidance provided by the BCMWRP (1991) and the more recent *Guidelines to Waste Rock and Stockpile Design* publication (Hawley and Cunning, 2017). The results of 2018 and 2019 field investigations have been used to characterize the footwall foundation conditions underlying the backfills. Representative stability sections and the 3D Phase 3 Pit mine plan were analyzed, and the results indicated acceptable design conditions during construction, at the end of construction and for reclamation configurations.

Two important aspects related to the operational stability of the backfills have been discussed in this report, including:

- The construction of the backfills are to be advanced over previously mined shallow footwall slopes that were excavated during the earlier operations. Where there is local undercutting of the footwall, the loading from the rock could contribute to foundation planar sliding mechanisms. To reduce the potential for planar sliding risks, backfills should be advanced across any undercut footwall to provide buttressing of the undercut portions. The geological model indicates that the footwall will be locally undercut by approximately 10 m to 15 m within the Phase 1 through 6 pit development.
- The dump platforms have been designed to be 20 m high with 10 m width catch benches to an overall 2:1 (horizontal:vertical) slope profile. Where bottom-up construction methods are adopted, the catch benches will also include 2 m berm near the platform crest to mitigate any rock roll from future upslope backfill construction, such as the Phase 3 Pit. The use of the shallower overall slope profile and benched configuration is to improve the stability conditions, and provide containment for any rock roll out.
- The Phase 3 Pit backfilling requires strict material quality management and monitoring to achieve the mine plan. Tactical backfilling using good quality, coarse rock is required to reduce the potential for slope face instability events that could have an impact on excavation activities in the south.

Critical to safe backfill construction is the implementation of operational procedures typically included in a GCMP. Operational procedures, expanding on those included in Section 7, will need to be included and adjusted as slope performance knowledge increases during the backfilling operations.

9 Closure

This report, Pit Backfill Slope Design Report, was prepared by

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All data used as source material plus the text, tables, figures, and attachments of this document have been reviewed and prepared in accordance with generally accepted professional engineering and environmental practices.

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