SSIM 2023 – PM Dight (ed.)

doi:10.36487/ACG_repo/2335_07

Evolution of the pit slope design process at Western Mesquite Mines

HW Newcomen *BGC Engineering, Canada*

E Schmidt *BGC Engineering, Canada*

S Sabo *Western Mesquite Mines, USA*

Abstract

Open pit mining has been ongoing at Western Mesquite Mines since the early 1980s. This low-grade gold deposit is located in a structurally complex geologic regime of metamorphic rocks beneath up to 120 m of tertiary sediments ranging from silts to conglomerates. Original slope design angles were relatively steep, did not differentiate between the surficial and bedrock geologic units, and resulted in frequent slope instability events. Through a process of slope performance evaluations in historically mined pits, back-analyses of failures, rock mass quality mapping, and geotechnical drilling and laboratory testing, slope angles have been optimised for the geotechnical units encountered in the various pits at the mine. The Brownie Pit is the most recent pit to be designed and mined. The performance of the slopes in the Brownie Pit during the first phase of mining indicated that the design slope angles were appropriate, with one multi-bench instability in the rock and manageable deformations in the tertiary sediments. This case history outlines the evolution of the investigation and design process in this unique geologic environment, and illustrates how ongoing geotechnical characterisation and documentation of slope performance can deliver optimised slope designs, improved mining productivity and safer working conditions.

Keywords: *pit slope designs, back-analysis, slope performance, case history*

1 History and introduction

The Mesquite Mine is an open pit heap leach gold mining operation in southeastern California with a long history. The area was prospected from 1957 to 1980 by Dick and Ann Singer, and produced gold from placer operations during that time. In 1981, Gold Fields Mining initiated an exploration program at the site. The first 10 drillholes from this program intercepted ore-grade oxide gold mineralisation in an area later named the Big Chief Pit. The mine was developed from 1981 until 1985, and in 1986 the first gold pour occurred, marking the start of commercial production. Initial capital costs were paid back within the first year of production from 179,000 ounces produced at a cost of USD 115 per ounce. Mining continued until 1993, when Gold Fields Mining and the Mesquite Mine were taken over by Santa Fe Pacific Gold Corporation. The mine was subsequently shut down due to low gold prices and crushing activity ceased, reverting to strictly run-of-mine ore. After a series of ownership changes (to Newmont, Western Goldfields Inc. and New Gold), the mine was renamed Western Mesquite Mines and production was resumed. The operation, now owned by Equinox Gold, is mining pushbacks of historical pits and ore-grade waste and developing new mining areas such as the Brownie Pit.

Numerous geotechnical assessments have been undertaken since mining was first initiated (JRT GeoEngineering 2011). The authors' involvement at Mesquite started when the settlement of a state highway east of the Rainbow Pit (Figure 1) occurred. A buttress was designed to stabilise the highway movements, requiring characterisation of the tertiary conglomerate gravels (TCGs) and underlying gneissic bedrock. Design work was subsequently carried out for proposed expansions of the historic open pits, including the BB1 Pit, the Vista Pit (divided into east [VE] and west [VW] areas), the VE2 Pit (BGC Engineering 2015) and the Vista-Lola-VE2 expansion (BGC Engineering 2017). Most recently, slope designs were prepared for the Brownie Pit (BGC Engineering 2020).

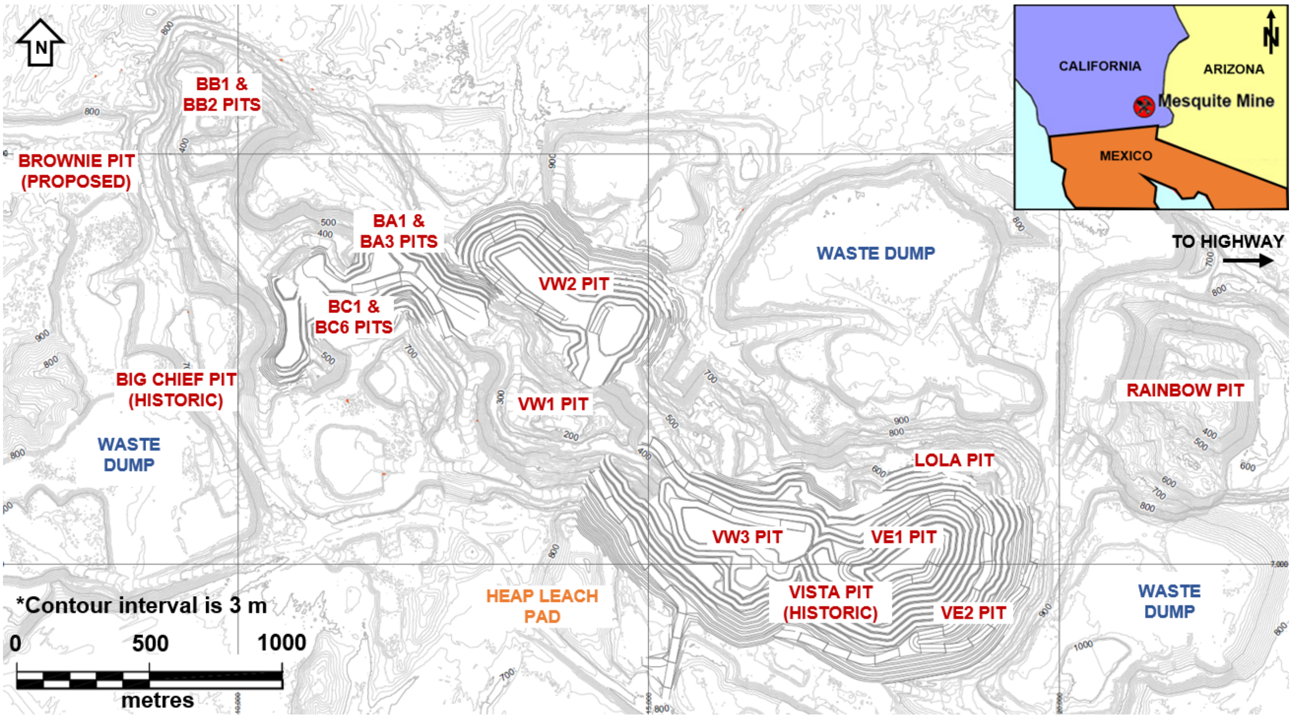


Figure 1 Overview and general arrangement of the Mesquite Mine. Topographic data is from 2017

This paper describes the evolution of the data collection, data management, analysis methods and the design process at Mesquite, with an emphasis on how the pit slope designs have changed over time (summarised in Table 1 and discussed in the following sections). The resultant pit slope performance of the Brownie Pit reflects the value of geotechnical field data collection, laboratory testing and slope design calibrated with over a decade of pit slope performance information.

Table 1 Summary of the evolution of pit slope design angles used at the Mesquite Mine

	Inter-ramp slope angle (°)					
		Soil		Rock		
Year	**Waste**	**TCG1**	**TCG2**	**Min**	**Max**	**Comments**
1988–2008	43	43	43	43	43	Historically all materials were designed to the same inter-ramp angle
2009	37–46	37–46	37–46	37–46	37–46	Per Engineering Analytics (2009) from work by Brawner (1999) and Shepherd Miller (1999)
2012–2015	29	29	29	38	38	Updated in response to the Rainbow Pit and BB1 instabilities
2016	29	29	29	38	42	Updated using structural discontinuity information from mined rock slopes
2020	27	30	34	30	38	Optimised for different TCG deposits and pit-specific structural fabric in rock

TCG = Tertiary conglomerate gravels

2 Geologic setting

The Mesquite Mine lies in a complex regional setting on the southwest flank of the Chocolate Mountains. Located in the upper plate of the Vincent-Chocolate Mountain Thrust, the deposit consists of Jurassic aged quartzofeldspathic and mafic gneisses overlain by tertiary conglomerates and volcanics formed during regional extension, uplift and erosion. Part of the Caborca orogenic gold belt, mineralisation and development of primary structures at Mesquite began during the Laramide orogeny 70–90 Ma, with mineralisation peaking around 61 million years ago. This period was followed by uplift and extension with overprinting of epithermal mineralisation during the Oligocene. Extension during the tertiary generated large-scale northwest-trending faults and reactivated some Mesozoic thrusts (Ricks 2023).

This structural setting leads to numerous potential structural domains in the bedrock geology of the deposit. North and northwest-striking faults (i.e. 'nor'westers') are often high angle (80–90°) normal faults. These faults are interpreted as reactivated structures as they follow mineralisation through the gneiss complex, reflecting the San Andreas fault system into the tertiary units. Northeast-trending faults often display left-lateral oblique slip, dipping from 40–70° across the property. Low angle thrust/reverse faulting also occurs on the property with a northwest strike and may be responsible for emplacing a muscovite schist on top of the gneiss complex (Willis & Tosdal 1992). Typical orientations of regional-scale faults at the Mesquite Mine are shown in Figure 2. The deposit itself is relatively flat lying, with foliation dipping moderately to the southwest, striking west-northwest.

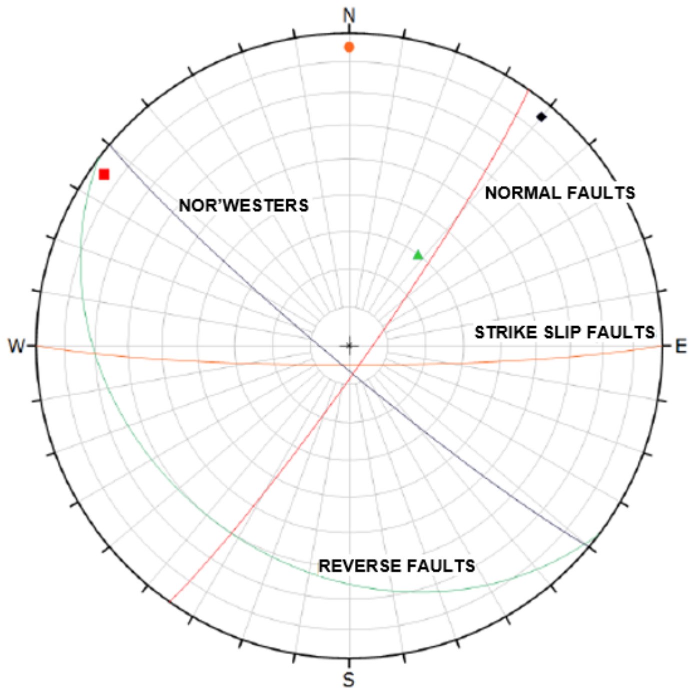


Figure 2 Orientation of regional faults found at the Mesquite Mine

The Mesozoic gneisses have been dated as Jurassic age (Tosdal et al. 1991) and consist of a muscovite schist, biotite gneiss, hornblende-biotite gneiss and mafic gneiss. The muscovite schist (hereafter referred to as MS) is described as a quartz feldspar-rich unit with abundant muscovite and a lack of mafic minerals. This unit is not laterally continuous across the property and is characteristically weak and foliation dominated. The thrust/reverse fault that emplaces this unit above the gneiss complex is low angle and contributes to geotechnical issues in the contact between the MS and biotite gneiss.

The biotite gneiss (BG) is characterised as having greater than 60% felsic minerals (quartz>feldspar) and weak foliation, and is often highly fractured. The highly fractured nature of the BG also contributes to geotechnical issues as mining activities (blasting and mucking) increase fracturing beyond designed parameters. The hornblende-biotite gneiss (HBG) is more ductile than the overlying BG and is characterised as medium-grained with equal mafic to felsic minerals. Geotechnically it tends to be more stable than the overlying units but, as with all gneiss units at Mesquite, stability is affected by the presence of intersecting faults and the orientation of foliation. The mafic gneiss (MG) is characterised by greater than 60% mafic minerals, is ductile with narrow ore zones and is folded. This is the thickest unit in the gneiss package.

Three tertiary units are identified in the package overlying the gneissic and schistose bedrock. The uppermost tertiary unit, TCG1, is a fine- to medium-grained matrix-supported conglomerate consisting of fine- to medium-grained boulders of tertiary volcanics and gneiss. TCG2 is the middle unit, consisting of fine-grained clay-rich sediments. This unit can be locally thick and, when exposed, fractures along anhydrite that forms in structural boundaries. The final tertiary unit, the TCM, is of limited distribution and consists of occasionally mineralised gravel with angular clasts of eroded gneiss. All tertiary units have been cut extensively by northeast-oriented faults related to the San Andreas fault system. These sediments originated from the Chocolate Mountains north of the property and were put in place during tertiary extension and uplift.

3 Investigation, design approaches and learnings

3.1 Historical slope designs

Brawner Engineering (1999) authored the first open pit geotechnical design report for the Mesquite Mine pits. The report summarises pit slope stability conditions in various areas of the historical pits Big Chief, Vista and Rainbow (Figure 1). Notably, it indicates that slope failures were common in the north faces of the pit walls and less frequent in the south faces. A summary of the slope performance data collected by Brawner Engineering is included in Table 2.

Table 2 Slope angle measurements and performance (after Brawner Engineering 1999)

Wall orientation	Pit	Height (m)	Slope angle (°)	Failure
North walls	Big Chief	91	33	Yes
	Big Chief	91	43	No
	Big Chief	101	30	Yes
	Big Chief	94	38	Yes
	Vista	67	39	Yes
	Vista	85	33	Yes
	Vista	70	37	No
	Rainbow	91	36	Yes
	Rainbow	76	31	Yes
		Average:	**35**	**Yes**
South walls	Big Chief	67	45	No
	Big Chief	64	49	No
	Vista	94	46	No
		Average:	**47**	**No**

In 1999 most of the pit slopes were mined at inter-ramp angles of 43°, with some mined at angles up to 49°. Instabilities were being monitored as the pits were mined, and it was noted that 'there had been no failures involving equipment or materials and that no personnel had been injured to date' (Brawner Engineering 1999). Groundwater was encountered in all pits (Big Chief, Vista, Rainbow and two other extensions) at that time; groundwater table elevations across the site vary but have been observed up to 175 m below ground surface. Due to the inefficiency of dewatering wells, sumps were developed at the base of the pits to collect groundwater for removal. Blastholes were also being pre-pumped in some areas.

Brawner Engineering (1999) also stated that 'due to the variability of the rock conditions, the extensive and variable jointing, bedding and weak fault infilling, theoretical analysis indicates flatter slope angles than normal would be required for typical safety factors', negatively impacting economics. Thus the proposed slope angle of 43° was endorsed and an observational approach was recommended. This approach included visual inspections and slope monitoring along with a safety awareness program and a flexible mining scheme to mine around or stabilise instabilities.

A feasibility level pit slope design report for the Rainbow Pit (Shepherd Miller 1999) was published shortly after Brawner Engineering's report and included a stability assessment of the proposed extension of that pit. The assessments included shear strength estimates for the various material types from laboratory testing of soil samples retrieved from two geotechnical core holes. The TCGs were assigned a friction angle of $\phi = 29°$, with a peak cohesion of $c = 219$ kPa and a disturbed of $c = 0$ kPa.

The Shepherd Miller (1999) report notes that there was 'no consistent fracture dip' in the core obtained from the underlying bedrock and thus fracture orientation was 'assumed to be random', providing an early indication of the complexity of the structural geology at Mesquite but little guidance on the potential kinematic controls on the stability of the slope in the underlying rock. The HBG and MG rock units were classified as highly fractured and altered with variable thicknesses of clay gouge on fracture surfaces, however, there was no formal rock mass classification carried out. Based on previous rock testing results (Call & Nicholas 1986) and rock mass descriptions from geotechnical drillholes, all rock units were assigned Mohr–Coulomb strengths of $\phi = 37°$ and $c = 0$ kPa.

The pit slope angle of 43° proposed by Brawner Engineering (1999) – assuming fully depressurised slopes, a pit lake at the base of the Rainbow Pit east wall slope and backfilling of the Rainbow Pit – was evaluated for the feasibility study. Regarding the east wall adjacent to the highway it was concluded that 'the reduction in overburden strength with time would result in an excessively flat pit slope angle i.e. 29° in the overburden and 37° in the underlying bedrock'. As a result it was recommended that a relatively large setback of 107 m from the pit slope edge to the nearby state highway be adopted.

With regards to experience mining in other areas of the property it was noted that north wall slopes mined at 43° often slumped to an average of 35° (Table 2), with the additional observation that most of the slope failures developed as the toe of the pit wall encountered the watertable. Based on these observations, Brawner Engineering (1999) recommended that the slope angles for north slopes in rock be revised to 40° above and 37° below the watertable, with a 15 m-wide step-out of two benches above the watertable. Due to the favourable performance of the south and east slopes, the recommended slope angles were revised to 46° above the watertable and 43° below the groundwater table. However, there were no stability analyses conducted to support the updated angles.

Ten years later Mesquite planned to expand the Rainbow Pit and commissioned Engineering Analytics to assess the expansion plan. Samples of clay lenses and fault zones were collected and preserved for laboratory testing (Engineering Analytics 2009). Laboratory testing consisted of index tests, and remoulded, consolidated undrained triaxial shear tests which yielded low friction angle values of $\phi = 4°$ to 24° and cohesion values ranging from $c = 79$ to 156 kPa for the clay lenses and fault zones. It appears that, despite the results from the additional laboratory testing, the strength parameters proposed by Shepherd Miller (1999) ($\phi = 29°$ and peak $c = 219$ kPa) were used to analyse the stability of the east wall, and the pit wall angles previously recommended by Brawner Engineering (1999) (Table 3) were endorsed.

Table 3 Recommended pit slope angles in the Rainbow Pit (after Engineering Analytics 2009)

	Location of pit wall		
	North	South and east	East face adjacent to highway
Slope above watertable	40° [1,2]	46° [2]	43° [1,3]
Slope below watertable	37° [2]	43° [2]	40° [2]

[1]Per Engineering Analytics (2009); [2]Per Brawner Engineering (1999); [3]Per Shepherd Miller (1999).

There is limited discussion in any of the historical slope design reports about achievable long-term slope angles in the TCG units other than the slopes that were noted as 'excessively flat', and little attention was paid to slope angles in the bedrock. Although there was merit in the observational approach being applied at the time, the rationale behind the recommended slope angles in Table 3 was not well-documented and it appears that the potential impacts of instability in the sediments and/or underlying bedrock on the nearby highway was to be managed with a substantial setback of the pit crest. However, ongoing slope movements eventually resulted in cracks within and beyond the highway right-of-way.

3.2 Rainbow Pit east wall remediation

Slope movements in the Rainbow Pit east wall were ongoing until 2012, with surface cracking evident approximately 60 m east of the highway and settlement within the highway right-of-way. During this time there were also several small instabilities between the east wall of the pit and the highway. Due to concerns about the integrity of the highway, a waste fill buttress was proposed to limit movements and improve the long-term stability of the east wall. The Rainbow Pit east wall instability during the early stages of buttress construction is shown with the highway in the background (Figure 3).

Figure 3 Instability in the tertiary conglomerate gravels materials on the east wall of the Rainbow Pit

To design the buttress, historical geotechnical reports and laboratory testing data were reviewed and used to estimate the strength and deformability parameters of the TCGs and the HBG bedrock for stability analyses of the east wall. The stability analysis models included northwest-striking faults (nor'westers) in the TCG sediments. The HBG rock mass was assigned an unconfined compressive strength of 38 MPa and a tensile strength of 3 MPa, with discontinuities assigned a strength of $\phi = 37°$ and $c = 0$ kPa. Historical surface movements measured in the TCGs and the underlying bedrock were used to calibrate the numerical model.

Two-dimensional (Slope/W) and 3D (FLAC3D) modelling was conducted to assess the effectiveness of the proposed buttress and explore opportunities to optimise the buttress design. The modelling results were subsequently used to estimate potential displacements within the highway right-of-way and develop recommendations regarding buttress sequencing and size. Long-term displacements along the highway were also estimated.

The completed buttress is shown in Figure 4. Buttress construction successfully mitigated the displacements along the highway and, despite numerous heavy rainfall events since it was constructed, the buttress has been effective.

Figure 4 Completed buttress below the east wall of the Rainbow Pit

These assessments provided early insights into the potential range of strength of the surficial sediments and bedrock. Subsequent inspections of the other open pits at Mesquite confirmed that instabilities in the TCG materials were impacting Mesquite's ability to efficiently mine in several other areas of the operation.

3.3 Identifying structural controls in the BB2 north wall instability

In 2014 an instability of approximately 150 m high developed in the BB2 Pit north wall in heavily faulted HBG bedrock. The instability resulted in the loss of most of the catch benches in this area (Figure 5), with tension cracks observed up to approximately 150 m from the crest of the slope and with notable heave along the ramp at the toe of the slope. The failure mode was interpreted as large-scale toppling along steeply dipping east–west trending strike-slip faults (Figure 2) subparallel to the north wall of the pit.

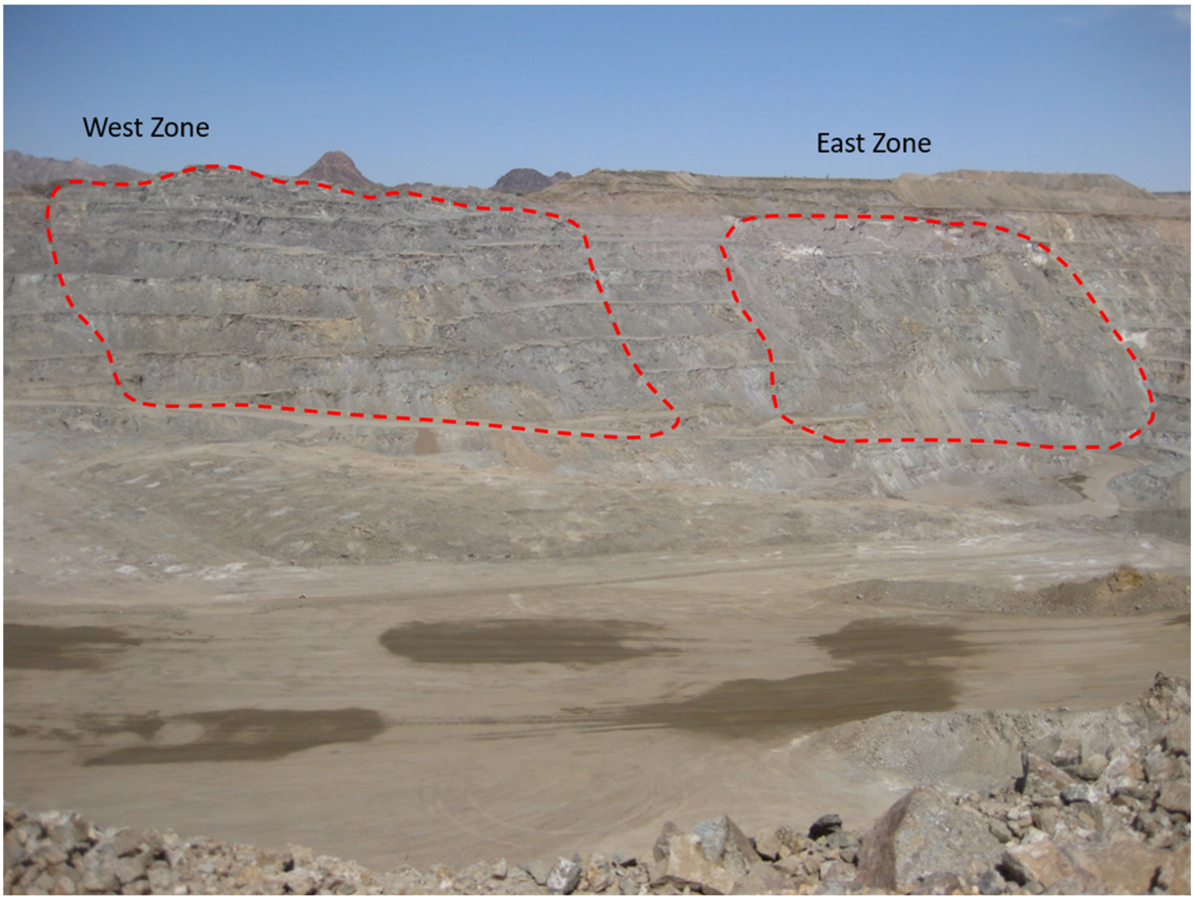


Figure 5 Toppling failure in the BB2 Pit north wall

Geologic structures mapped by Mesquite's geologists supported the toppling mechanism and became the starting point for the compilation of structural discontinuity information in the mine area, which at that time was only available on hand-drawn maps. The mapping information was subsequently digitally processed to facilitate kinematic stability analyses. This was the early stage of understanding the structural framework of the open pit area.

3.4 Application of slope performance for preliminary designs and characterisation of the rock mass for the Vista Pit expansion

After the slope instability events in the TCG sediments of the east wall of the Rainbow Pit and in the bedrock of the BB2 north wall, pit slope designs for a proposed expansion of the historic Vista Pit were initiated to estimate achievable slope angles in the proposed VW1, VW3 and VE1 Pits. A comprehensive review of historical geotechnical reports, geology data, piezometric data, annual topographic surveys and aerial photographs was carried out. Rock mass classification from pit walls adjacent to the proposed pits and core photographs from historic exploration drillholes were reviewed to estimate rock mass strength. This information was used, along with an assessment of the performance of the historic Vista Pit slopes, to provide preliminary guidance on pit slope angles for the proposed pits.

A slope failure database for the TCG unit was compiled from orthophoto and topographic survey data. Operationally disruptive failures in the TCG occurred approximately once per year, generally initiating in low (i.e. less than 40 m), double-benched slopes and becoming larger as the pit was deepened. The failures present in the pit in 2014 can be seen in Figure 6.

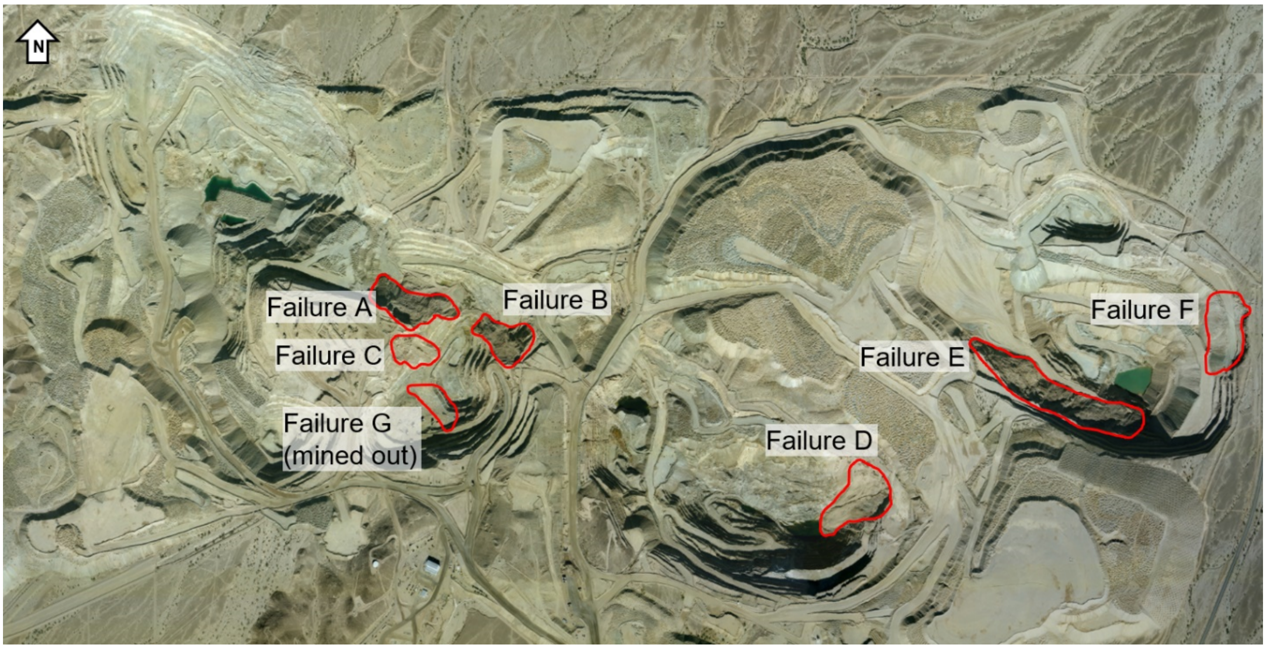


Figure 6 Outlines of failures in tertiary conglomerate gravels materials in the 2014 pit

Slope geometries for each of the failures in the TCG unit (Figure 6) are summarised in Table 4. Pre-failure slope geometries were estimated from topographic survey information.

Prior to failure, measurements of overall slope angles indicated that slopes in the TCG were excavated at angles between 40° and 44° (Table 4), consistent with the historic design angles. Post-failure angles in the TCGs ranged from 21° to 30° (the median post-failure angle was 24°). The slope failures were back-analysed assuming a friction angle of 29° in the TCG, resulting in an average cohesion of 24 kPa for these materials.

Table 4 Summary of slope failure geometries in tertiary conglomerate gravels materials

Failure ID[1]	Location	Pre-failure overall slope angle (°)	Pre-failure height (m)	Post-failure angle (°)
A	BA1 south wall	42	46	22
B	BA1 southeast wall	41	37	23
C	BA3 west wall	40	52	21
D	Vista east wall	42	46	25
E	Rainbow south wall	41	55	29
F	Rainbow east wall	43	91	24
G	Bayhorse (BC1) northeast wall	44	41	30

[1]Refer to Figure 6 for failure locations

Based on the performance of the TCG materials, design curves and corresponding inter-ramp angle (IRA) guidance were developed based on slope height (Figure 7).

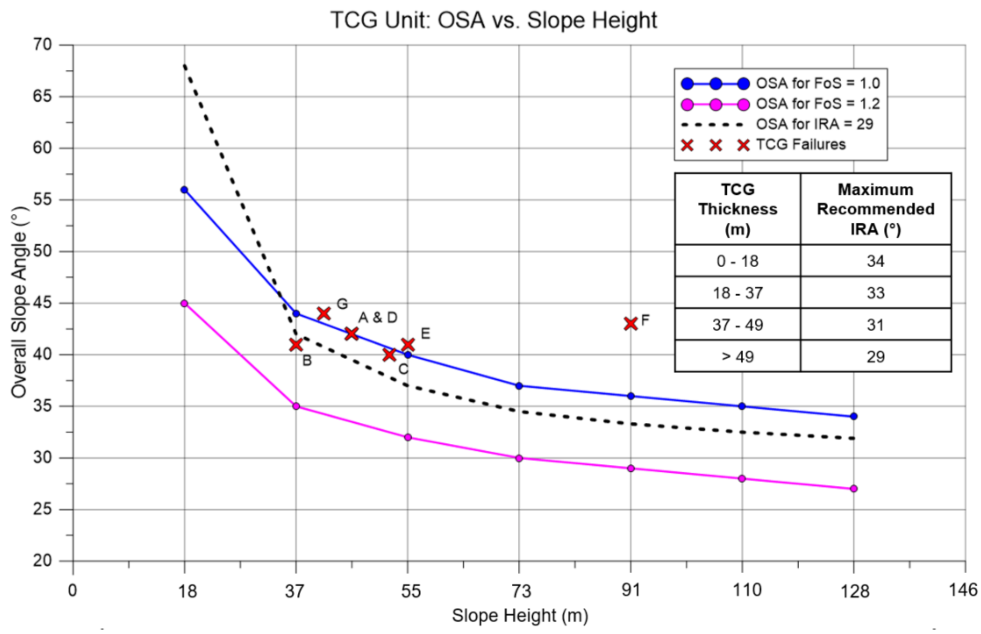


Figure 7 **Back-analysis results and overall slope angle versus slope height design curves. Table indicates the maximum recommended IRA for short-term slopes in tertiary conglomerate gravels materials**

Most of the historic open pits intersected localised areas of poor-quality rock, however, no well-documented rock mass failures at Mesquite and pit wall instabilities typically had an element of structural control. Thus the absence of rock mass failures in the existing pit slopes limited the usefulness of back-analysis to estimate rock mass properties. Nonetheless, the performance of the historic bedrock slopes provided precedence and indicated that pit walls between approximately 60 and 90 m high in the bedrock units (i.e. the MS, BG, HBG and MG units) could be mined at inter-ramp slope angles between 32° and 41°. Achievable stable slope angles were noted to be lowest on the north walls, consistent with observations made by Brawner Engineering (1999).

During the process of evaluating the performance of the pit slopes mined in rock, the rock mass in the open pit areas was classified using Bieniawski's (1976) rock mass rating (RMR) (Table 5).

Table 5 **Bedrock field rock mass rating summary**

	Rock mass rating (Bieniawski 1976)						
Pit area	RQD[1] (%)	Joint condition	Fracture spacing	Strength grade[2]	Groundwater condition	Estimated RMR	Rock quality
BC1	60–70	16–20	<0.3–1.0	R2–R3	Damp	53–63	Fair–good
BA1	30–40	12–16	<0.3	R2–R3	Damp	41–48	Fair
BA3	30–40	12	<0.3	R1–R3	Damp	41	Fair
Vista	10–50	6–12	<0.3	R1–R2	Damp	29–41	Poor–fair

[1]RQD was estimated assuming RQD = 115-3*Jv where Jv = the number of discontinuities in a cubic metre of rock. [2]Strength grade is per International Society for Rock Mechanics (1981).

Rock mass quality was also estimated from core photographs from drillholes in the proposed Vista Pit expansion. These assessments indicated that the quality of the rock anticipated in the VE1, VW1 and VW3 Pits was variable. In the absence of detailed structural data in the Vista Pit, but due to the relatively steep regional structures, a design angle of 38° was universally applied in the bedrock to manage toppling failures and potential rockfalls due to deterioration of the rock mass. Using this angle for the bedrock and 29° for the TCG units, Mesquite developed preliminary pits for the Vista expansion.

3.5 Detailed designs and consideration of the consequence of failure for the BC6 and Vista expansion pits

Structural geologic mapping and rock mass classification information from the HBG geotechnical unit exposed in the first phase of the BC6 pit was used to evaluate the potential to steepen the preliminary design angle of 38°. The consequence of instability was taken into consideration when defining the design acceptance criteria per Read & Stacey (2009), with a 'low' consequence of instability assumed for the north wall slopes where the pit crest could be unloaded during a subsequent mining phase, if necessary, and a 'medium' consequence of instability assumed for the south wall where a ramp was present.

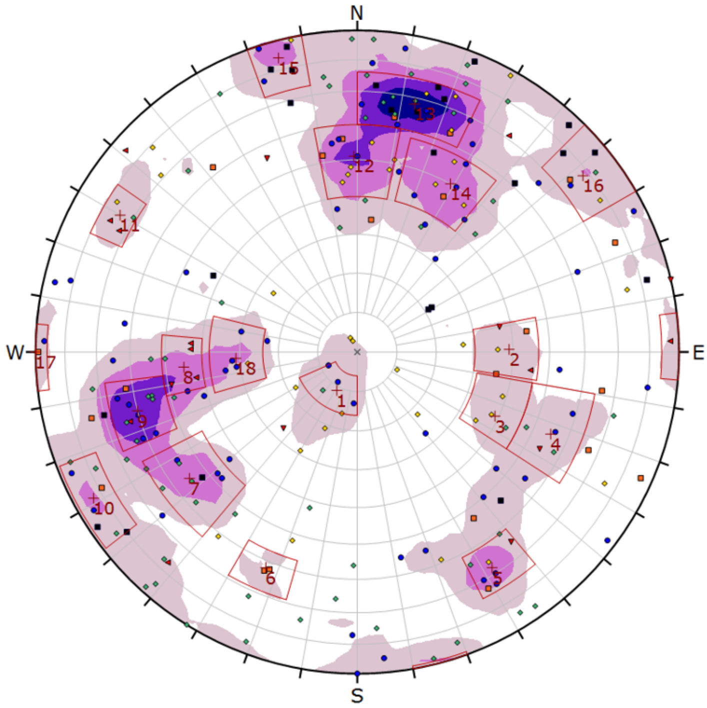
Strength anisotropy estimates due to foliation were estimated and incorporated into limit equilibrium models. Along with the favourable performance of the first phase of the BC6 pit, stability analyses indicated that the preliminary 38° IRAs mined could be steepened. Breakback of benches on the north wall along out-dipping foliation was taken into consideration, indicating that the IRA could be increased to 40°. Limit equilibrium stability assessments using rock mass strength estimates demonstrated that the south and southeast walls, which had no structural control, could be steepened to 42° and achieve a Factor of Safety (FoS) of greater than 1.2, which was considered appropriate for inter-ramp slopes.

The proposed expansion of the Vista Pit, comprising a shallow pit (Lola) to the northeast and a pushback pit (VE2) to the east and south, was anticipated to intersect the groundwater table. Geotechnical and hydrogeological evaluations consisting of geotechnical logging of split exploration core, characterisation of the rock mass (primarily HBG) in the pit bottom, collection of structural data using photogrammetry and a pumping test to assist in estimating pore pressures were undertaken to optimise pit slope angles. As with previous evaluations, the performance of the various historic pit walls in the Vista Pit was further evaluated with the following results:

- North wall – poor performance due to pervasive bench scale instability (Figure 8) and localised inter-ramp instability along foliation and through poor-quality rock resulting in achievable IRAs between 32° and 38°.
- East wall – moderately good performance with limited bench scale instability resulting in an achievable IRA of 38°.
- South wall – good performance with a measured IRA of 42°.

Figure 8 Bench and inter-ramp instabilities on the north wall of the proposed Lola Pit

Structural geology data for the Lola-VE2 pit designs collected from photogrammetric mapping of the north and south walls of the historic Vista Pit (Figure 9) confirmed the presence of discontinuities parallel to the strike-slip, nor'westers and normal faults (Figure 2). However, structural discontinuities consistent with the reverse faults were less apparent.



Symbol	PERSISTENCE (M)	Quantity
■	0.64 - 1.39	24
◆	1.39 - 3.04	66
●	3.04 - 6.66	68
◆	6.66 - 14.57	39
■	14.57 - 31.86	22
◄	31.86 - 69.67	11
▼	69.67 - 152.35	7

Color	Density Concentrations
	0.00 - 1.00
	1.00 - 2.00
	2.00 - 3.00
	3.00 - 4.00
	4.00 <

Contour Data	Pole Vectors
Maximum Density	4.76%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Plot Mode	Pole Vectors
Vector Count	237 (237 Entries)
Hemisphere	Lower
Projection	Equal Area

Figure 9 Stereographic projection of inter-ramp scale structural discontinuities from photogrammetric mapping in the historic Vista Pit

The inter-ramp and overall slopes in the western portion of the proposed Lola-VE2 were not assessed because the west wall slopes only comprise a limited portion of the proposed pit and the IRA for the west wall would be dictated by blending of the north and south wall inter-ramp angles.

Kinematic stability analyses of potential structurally controlled failure mechanisms and limit equilibrium stability analyses to assess the potential for rock mass failure were carried out. Pit slope stability design acceptance criteria (i.e. FoS) were consistent with those recommended by Read & Stacey (2009). For kinematic stability assessments, the Lola-VE2 pit area was treated as a single structural domain. Inter-ramp scale structural discontinuity sets were identified from the mapping data (Figure 9) and assigned a shear strength of $\phi = 30°$, $c = 0$ kPa, based on empirical estimates (Barton & Choubey 1977). Potential anisotropies were incorporated into the geotechnical models for further assessment on the impact of large-scale geologic structures.

Two-dimensional stability analyses were performed to assess the stability of the proposed pit slope designs using the predicted groundwater total head distributions. Modified slope geometries were evaluated to assess the feasibility of steepening the inter-ramp slopes. Material properties (Table 6) were modified to account for blast disturbance and anisotropy in the strength of the bedrock due to faulting and pervasive geologic structures. Pore pressures were also incorporated into the models.

Table 6 Design rock mass properties for the Lola-VE2 pit design

Unit	Unit weight (kN/m^3)	Friction angle (°)	Peak cohesion (kPa)	Residual cohesion (kPa)	Generalised Hoek–Brown criteria			
					UCS (MPa)	GSI	m_i	Disturbance factor[2]
TCG	19.6	29	24	0	–	–	–	–
Bedrock	23.1	–	–	–	37	48	15	0–1
Waste	17.9	32	0	0	–	–	–	–
Fault	23.1	12[1]	0	0	–	–	–	–
Bedrock anisotropy	23.1	30	0	0	–	–	–	–

[1]Lower-bound fault strength estimated from back-analysis of failure in the BC6 pit. [2]Fully disturbed properties were applied to bedrock 30 m behind the slope face; undisturbed properties were applied to all other bedrock. TCG = tertiary conglomerate gravels

Stability analyses indicated that the inter-ramp angles were not limited by the stability of the overall slopes. Previously mined north wall benches were failing along well-defined, south-southwest dipping foliation and thus the inter-ramp design was limited by the bench width required to retain bench failure. The existing benches on the historic east and south walls were noted to be sufficiently wide to collect rockfall debris and contain typical bench failures, and had experienced minimal breakback, indicating that the berm width was appropriate for the structural geologic controls assumed. Based on the bench performance and the kinematic analysis, the inter-ramp slope angles in the east and south walls were increased from 38° to 40° and 42°, respectively. The north wall slope angle was kept at 38° due to the poor performance of the historic slopes.

4 Applying historical learnings to the Brownie Pit design

In 2019, Mesquite proposed mining the Brownie Pit and required slope designs for this new area of mining. The Brownie Pit is located west of the BB2 Pit and north of the backfilled Big Chief Pit (Figure 1). The proposed pit would encounter thick TCG deposits over the gneissic (HBG and BG) bedrock, a limited amount of schist (MS) and a small amount of previously handled waste material (Figure 10).

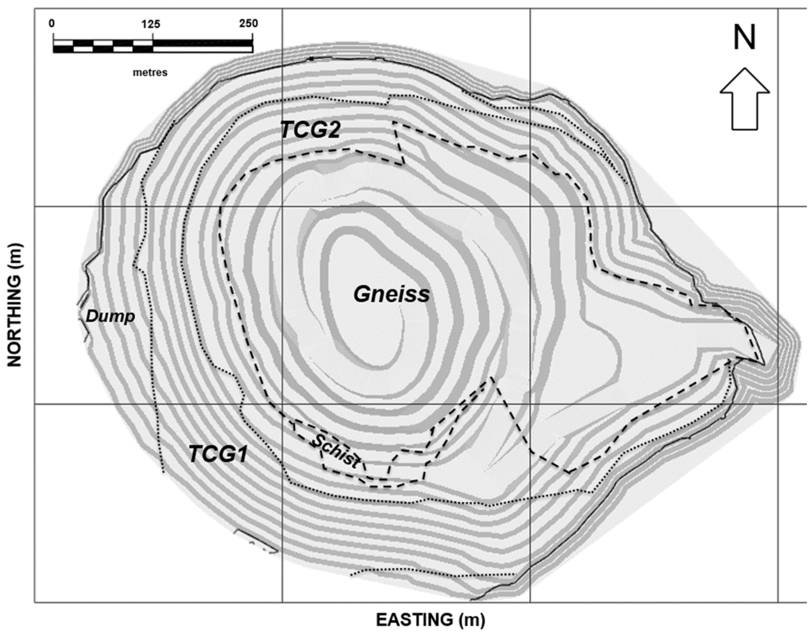


Figure 10 Proposed Brownie Pit wall geometry and geotechnical units

Locally the TCG unit was expected to reach a thickness of approximately 120 m, representing the thickest exposure of this material encountered to date at Mesquite. Because of this it was important to optimise the design angles for the soil slopes due to the potentially high strip ratio in the soil. With the learnings from the previous pit design iterations at Mesquite in mind, the design process for the Brownie Pit consisted of the following:

- Collecting detailed site investigation data from the proposed Brownie Pit area to create a site-specific model of geotechnical units in the proposed pit footprint.
- Leveraging regional structural knowledge and previously collected data to develop an improved structural model.
- Referring to the performance of previously mined slopes to confirm that the geotechnical properties used to develop the current slope design agree with the behaviour observed to date.

4.1 Site investigation

A geotechnical investigation was carried out to characterise the materials in the Brownie Pit footprint. Six drillholes and four test trenches were geotechnically logged, and samples were collected for laboratory strength testing of the various soil and rock units encountered. Televiewer surveys were also conducted to collect structural data in the bedrock and competent portions of the TCG encountered in each hole.

Geotechnical drilling data collected during the site investigation assisted with spatial delineation of the soil and rock units in the Brownie Pit area added valuable structural and laboratory strength data to the geotechnical database, and identified important differences between the TCG sub-units TCG1, TCG2 and TCM. Specifically, the TCG1 sediments were coarser-grained, ranging from silty sands with trace clay to sandy gravels with various levels of calcareous cementation, often containing sub-angular to angular cobbles and boulders, while the TCG2 sediments ranged from stiff to hard sandy clays to clayey sands with weak to strong calcareous cementation. Due to its limited distribution and similar expected properties, the TCM unit was grouped with the TCG2. With the additional drilling and laboratory data available from the Brownie site investigation, the TCG1 and TCG2 units could now be separated for better characterising of their strength differences and assigning of different slope design angles to each.

4.2 Building the structural model

By this point a wealth of structural data had been collected over the years at Mesquite, including structural models developed for the previous pits, a 3D fault model built by the Mesquite geology team, structural measurements collected from the site investigation program and a variety of publications (Manske 1991; Tosdal et al. 1991; Willis & Tosdal 1992) which were reviewed in the context of the Brownie design. Together these data sources were used to inform an overall structural model for the Brownie Pit area. The following structural controls were observed:

- Faulting within the bedrock, which impacts both the rock mass quality of these units and creates preferential orientations of weakness within the proposed pit walls. The MS unit present in the southwest walls of the proposed Brownie Pit is interpreted to be heavily sheared due to thrusting of the unit over the underlying gneissic rocks. Fault set orientations were interpreted based on historical mapping data conducted east of the Brownie Pit area and from 3D fault surfaces provided by Mesquite's geology team.
- Foliation and joints within the bedrock, which create preferential orientations of weakness within the proposed pit walls. Based on mapping conducted in other areas of the mine, foliation ranges from planar and continuous to irregular and contorted due to disruption from faulting.
- Structural zones of weakness within the TCG units. Slickensides and weak bedding planes were identified within the TCG1 and TCG2 units during the Brownie Pit site investigation, indicating that

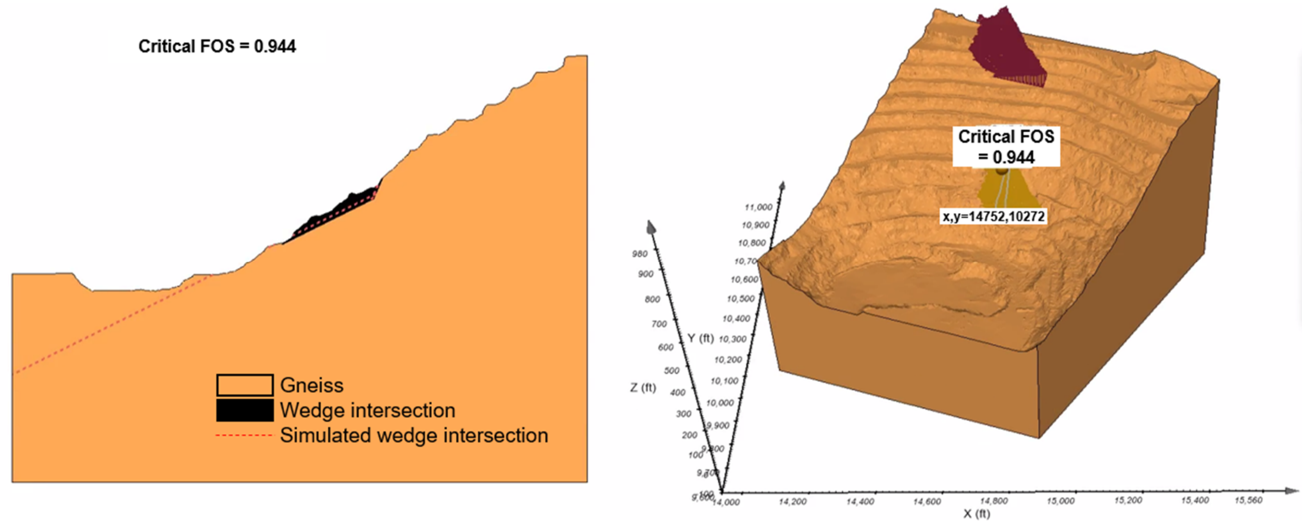
structure is present within these soils, which may act as preferential planes of weakness when the slopes are excavated.

From the above model it was apparent that structure in both the soil and bedrock would have the potential to impact the stability of the Brownie Pit walls, and the updated structural model could now be used to incorporate both into the design.

4.3 Applying past slope performance to design

Observations of the performance of previously mined slopes at Mesquite were used to validate the design strengths of both the soil and bedrock units in the Brownie Pit design. In the soil slopes, historical performance indicated that the TCG materials stood up at inter-ramp angles in excess of 40° (Table 2). However, long-term stability was a concern as post-failure angles observed in these same slopes were considerably lower, suggesting that strength loss occurs in the TCG materials when they are disturbed from their in situ state. The back-analyses performed as part of the Vista Pit expansion design considered this effect, however, they did not consider the strength difference between the TCG1 and TCG2. Nor did the analyses have detailed laboratory data to inform the friction angles used in the analyses (this was since collected as part of the Brownie site investigation). The back-analyses were ultimately re-run with friction angles of 37° and 17° in the TCG1 and TCG2, respectively, which were calibrated against reconstituted triaxial tests and residual direct shear testing. The results demonstrated that peak values of up to 14 kPa of cohesion could be expected in the TCG1 unit, while up to 101 kPa could be expected in the TCG2 unit. However, this cohesion could be lost if the units are disturbed, highlighting the importance of maintaining the peak strength in these materials.

Historical failure mechanisms in the rock slopes of the existing open pits were often complex, involving sliding along faults combined with failure through a weak rock mass. This made it difficult to use back-analyses to estimate the shear strength of the faults without assuming a rock mass strength, or vice versa. However, one rock slope failure in the VW2 pit was entirely bounded by faults on three sides and considered suitable for back-analysis. This wedge was modelled in the three-dimensional limit equilibrium software SVSLOPE® with an assumed FoS of 1.0. The back-analysis produced an estimated frictional strength of 29° on the fault surfaces (Figure 11).



(a) (b)

Figure 11 Results of VW2 wedge failure back-analysis. (a) Cross-section parallel to wedge intersection; (b) 3D view of simulated wedge (burgundy colour), sliding surface (yellow) and critical Factor of Safety of 0.94 for ϕ_r = 29° and c = 0 kPa

4.4 Results

Using the updated design strengths derived for the TCG1, TCG2 and bedrock geotechnical units, historical design strengths for the waste material, an updated structural model to inform the orientation of structural weaknesses and a design FoS of 1.3 (overall slopes) or 1.2 (inter-ramp slopes), geometries were developed for all wall orientations present in the proposed Brownie Pit. Design inter-ramp angles in the TCG materials varied from 30° in the TCG2 to between 32° and 34° in the TCG1. Design inter-ramp angles in the rock varied from 30°, where moderately dipping foliation acted as a control on stability, to 33° in the weak, faulted MS, with a maximum angle of 38° for slope orientations with favourably oriented discontinuities.

It should be noted that although the Brownie design slope angles range from being steeper to shallower than those previously applied to pit slopes at Mesquite, the designs are now better fit for purpose than in previous iterations. Weak units, including the TCG2 and MS, have been delineated and treated as their own units with unique slope angles, and the stronger TCG1 and favourably oriented bedrock have accordingly been assigned higher inter-ramp angles. The benefits of this approach are illustrated in Figure 12, which shows the mined slope configuration of the first phase of the Brownie Pit. No inter-ramp or overall instabilities were observed in the Phase 1 Brownie Pit walls, and bench scale instabilities were sparse and operationally manageable.

Figure 12 View of the mined Phase 1 Brownie Pit walls, looking southwest

5 Conclusion

Over the past 25 years, Mesquite's slope design process has evolved from being a primarily performance-based approach to a more comprehensive approach incorporating structural geologic and rock mass mapping, geotechnical core logging data, laboratory testing data of soil and rock units, and performance data supported by back and forward stability analyses. This has resulted in more stable pit slopes, improved mining productivity and safer working conditions.

Acknowledgement

Thanks are extended to the various Mesquite Mine staff over the years and Equinox Gold for permission to publish this paper. In particular, the authors would like to thank Bill Martinich for encouraging them to 'take a harder look at the pit slope angles', and Paul Martin for his prowess in monitoring the pit slopes and upgrading the slope monitoring systems to provide accurate and timely information on slope behaviour. Finally, the authors would like to thank Don Wagstaff (retired) for his curiosity about open pit geotechnics and for giving them the opportunity to collect geotechnical and slope performance data in the early days of their tenure at the site; allowing the team to make solid, risk-informed decisions regarding pit slope designs at Mesquite.

References

Barton, N & Choubey, V 1977, 'The shear strength of rock joints in theory and practice', *Rock Mechanics*, vol. 10, pp. 1–65.
BGC Engineering Inc 2015, *Proposed Vista Pit Expansion Slope Stability Assessments*.
BGC Engineering Inc 2017, *Proposed Lola-VE2 Expansion Slope Stability Assessment*.
BGC Engineering Inc 2020, *Western Mesquite Mines Brownie Pit (Phase 1) Open Pit Slope Designs*.

Bieniawski, ZT 1976, 'Rock mass classification in rock engineering', in ZT Bieniawski (ed.), *Proceedings of the Symposium on Exploration for Rock Engineering*, pp. 92–106.

Brawner Engineering Ltd 1999, *Mesquite Mine Pit Slope Design and Development*.

Call & Nicholas 1986, *Mesquite Mine Pit Slope Design*.

Engineering Analytics 2009, *Slope Stability Analyses of the East Rainbow Pit Expansion*.

International Society for Rock Mechanics 1981, 'Suggested methods for determining hardness and abrasiveness of bocks', in Brown (ed.), *Rock Characterization, Testing and Monitoring: ISRM Suggested Methods*, pp. 95–96.

JRT GeoEngineering 2011, *Geotechnical Site Inspection and Review of NGI's Mesquite Mine*.

Manske, SL 1991, *Epithermal gold in gneissic rocks of the Mesquite District, Imperial County, California*, PhD thesis, Stanford University, Stanford.

Read, J & Stacey, P 2009, *Guidelines for Open Pit Slope Design*, CRC Press, Boca Raton.

Ricks 2023, *Mesquite: Geology and Mineralization*, internal report.

Shepherd Miller 1999, *Geotechnical Stability Evaluation of the Rainbow Pit Extension Newmont's Mesquite Mine*.

Tosdal, RM, Willis, GF, Manske, SL, Lang, D & Lusk, M 1991, 'Day 1: Mesquite Mining District, southeastern California', *Society of Economic Geologists Guidebook Series*, vol. 12, pp. 87–103.

Willis, GF & Tosdal, RM 1992, 'Formation of gold veins and breccias during dextral strike-slip faulting in the Mesquite Mining District, Southeastern California', *Economic Geology*, vol. 87, no. 8, pp. 2002–2022.