REPORT

GEOTECHNICAL INVESTIGATION AND DESIGN RECOMMENDATIONS

Santa Margarita Quarry

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Table of Contents

List of Tables

List of Figures

List of Appendices

1.0 INTRODUCTION

1.1 General

Golder Associates is pleased to present these pit slope design recommendations for Hanson Aggregates West's Santa Margarita Quarry. The quarry is located approximately four miles by road northeast of the town of Santa Margarita in San Luis Obispo County, California (Figure 1). Hanson Aggregates West ("Hanson") plans to expand the existing quarry to an adjacent lease to the northwest, and has requested that Golder Associates Inc. ("Golder") evaluate potential geotechnical constraints associated with the proposed expansion. This report presents the results of our evaluation.

1.2 Scope of Work

The scope of work was defined in our proposal entitled "Proposal for Geotechnical Constraints Evaluation, Hanson Aggregates, Santa Margarita Quarry, Santa Margarita, California" and dated September 6, 2007, and included:

- **Compilation and Review of Available Data** – including available published and unpublished data concerning geology, seismic setting, and hydrogeologic conditions
- **Site Mapping and Drill Core Review/Sampling** – reconnaissance of existing pit and proposed expansion area, field structural mapping, review of exploration core, and sampling of core if appropriate
- **Data Analysis and Slope Stability Evaluation** – kinematic analyses, and limit equilibrium analyses if appropriate
- **Constraints Evaluation and Preliminary Mine Plan** – evaluate the preliminary mine plan developed by Hanson
- **Report Preparation** – preparation of this report, which documents our work
- **Meetings/Permitting Support** – as required during the permitting process

In order to maximize quarry reserves, the recommended slope designs should be as steep as the geological conditions will permit, consistent with enabling safe operating practices that meet regulatory requirements for mine and quarry slope development. The quarry expansion design is based on a pit bottom elevation of 880 ft, and production bench heights of 50 ft.

1.3 Method of Work

Work was initiated with a site reconnaissance by Graeme Major, Principal Geotechnical Engineer in Golder's Reno office, and Bill Fowler, Senior Consultant in Golder's Mountain View office, on February 15, 2008. This trip included visits to the current pit and proposed expansion areas, and a review of the exploration core. On March 6 and 7, 2008, Rhonda Knupp, Project Geologist in the Reno office, completed documentation and structural mapping of the accessible upper benches in the west wall, the north end of the pit bottom, and the exploration road cut northeast of the pit. Data collected during the site reconnaissance and the structural mapping combined with information from documents provided by Hanson form the basis for the geotechnical evaluations, analyses, and recommendations presented in this report. Subsequent visits to the site to confirm site conditions and to collect supplemental information were performed by Bill Fowler in September 2011, and Denise Mason, Project Geologist, in November 2011.

2.0 EXISTING CONDITIONS

2.1 History

As described in the exploration report for the Old Anderson Tract (Huffman, 2006), quarrying at the Santa Margarita Quarry is believed to have begun in the mid-1920s. Hanson acquired the property from Kaiser Sand and Gravel circa 1994. The existing quarry is permitted by San Luis Obispo County to process 700,000 tons per year through the primary crusher. Exploration of approximately 123 acres of the Old Anderson Tract west of and adjacent to the existing lease commenced in May 2006 (Figure 1).

2.2 Topography and Site Conditions

The Santa Margarita Quarry is located among steep hills and canyons east of the Santa Margarita Valley. Surrounding hills are up to 1480 ft high, and covered with oak woodland and chaparral. The Salinas River flows northeast, parallel to and about 130 feet east of the east haul road, then turns west and runs parallel to and north of the north boundary of the property (Figure 1).

2.3 General Geologic Conditions

The geology of the Santa Margarita Quarry and surrounding area is summarized in the exploration report (Huffman, 2006). The quarry is located in Cretaceous granitic rocks at the southeast end of a ridge bordering the east side of the Santa Margarita Valley (Figure 1). Across the valley to the west is the Santa Lucia Range, which consists of Tertiary sedimentary rocks. The granitic rocks of the quarry area and the sedimentary rocks in the Santa Lucia Range are separated by the Rinconada Fault, a regional northwest-southeast trending, right-lateral fault located approximately a mile west of the quarry.

High angle faults that appear to be reverse faults crosscut the pit parallel to the regional structural trend (Figures 1 and 2), and one of these appears to be a splay of Rinconada Fault. The faults generally dip southwest. The pit area is believed to consist of two blocks of granite displaced upwards relative to the surrounding rock.

2.4 Pit Geology

Fresh rocks exposed in the pit consist of medium-grained granite that is strong but moderately fractured. Numerous thin, steep faults and shears were observed in the pit walls during structural mapping. The faults are generally about 2-3 inches thick and filled with clay or strongly clay-altered breccia and fault gouge. A few shallow-dipping faults with similar infill characteristics were also observed. This strong fresh granite is capped by an approximately 50-ft thick zone of weathered granite, and 6-25 ft of decomposed granite at the surface.

A series of steep, west-northwest-striking faults that crosscut the pit were mapped (Figure 2). Several of these faults contain dikes up to eight feet thick, and some of the dikes are strongly clay altered. Most likely these are the high-angle faults described in the exploration report.

A large north-northwest-striking, moderately west-dipping fault cuts across the north end of pit (Figure 2). There are also several near-vertical, northeast-striking faults in the upper benches of the west wall. These larger-scale faults are generally about 2 to 3 feet wide but can be wider, and are filled with clay and strongly clay-altered fault breccia and gouge.

2.5 Geology Expansion Area

As described in the exploration report, exploration core from the proposed expansion area encountered fine- to coarse-grained porphyritic granite cut by faults. The faults range in thickness from 3-25 ft, and contain soft gouge and breccia. The thickness of surficial weathered granite encountered ranged from 17-80 ft, but averaged about 50 ft. This material is generally granular with an estimated clay content of less than 6-8%. The granitic body targeted by the expansion is bounded to the southwest by a large fault sub-parallel to the one that is believed to be a splay of the Rinconada fault (Figure 1).

2.6 Existing Pit

The Santa Margarita quarry is mined using Caterpillar 988 wheel loaders, which can reach to a height of approximately 25 feet for scaling bench faces. Blasting is performed under contract using 5-inch diameter blastholes drilled on a 12-ft by 12-ft pattern for regular production blasting. The pattern dimensions are increased when blasting for rip rap, and decreased to produce increased fines or to reduce crushing. The production blast pattern is drilled at the final pit walls with no special perimeter blasting.

The current pit bottom elevation is 882 ft. Crest elevations vary from 1318 ft on the west side of the pit, to 1010 ft on the east side of the pit. Due to several wide benches in the west wall, the crest of the uninterrupted west slope is 1198 ft. The bench design in the existing pit generally consists of 12-15 foot wide catch benches (“berms”) at vertical intervals (“bench heights”) of 30 feet, with vertical bench faces. Existing catch benches tend to be narrow or inaccessible over the lower west wall. Bench face angles range from 38° to 77°, but are generally around 50°-60°.

The maximum slope height in the pit is approximately 360 feet in the west wall. Overall (toe to crest) slope angles in the west wall range from 35° over 360 vertical feet to 44° over 260 vertical feet. In the west wall, uninterrupted inter-ramp slope angles are as steep as 59° over 138 vertical feet. Overall slope angles in the east wall range from 39° over 74 vertical feet to 52° over 130 vertical feet, the maximum slope height in that wall. Uninterrupted east wall slopes are as steep as 64° over 112 vertical feet.

2.7 Slope Performance

Overall slope performance at the Santa Margarita is good with no indication of large-scale, deep-seated slope failures involving the rock mass. This is related to the relatively high strength and competency of the fresh granitic rocks and generally favorable structural conditions. Localized instabilities are observed in certain areas of the quarry. The west wall, in particular, features a number of localized wedge failures that have deposited loose rock on slope benches. In addition, a planar failure approximately 90 feet high is visible in north end of the east wall, with smaller-scale planar failures occurring in the south part of the wall. However, overall the rock characteristics are considered favorable for quarry development provided that appropriate designs are prepared, and sound operational procedures are implemented.

2.8 Ultimate Pit

The planned ultimate pit configuration is shown in Figure 3, and will push back the west wall up to 1500 ft to the northwest, forming a triangle-shaped pit. The pit bottom elevation is 880 ft which is the current depth. Maximum slope heights are 460 feet in the west wall, 240 ft in the north wall, and 150 ft in the east wall. Design inter-ramp slope angles conform to the recommendations provided in this report and are approximately 49° in fresh and weathered rock in the west and northwest walls, and 43° in fresh and weathered rock in the north, northeast, and east walls. Weathered rock slopes are generally designed at 1.5H to 1V (34°) with local maximums of 1.25H to 1V (39°). Slope and bench designs are discussed in more detail in Section 6.0 of this report.

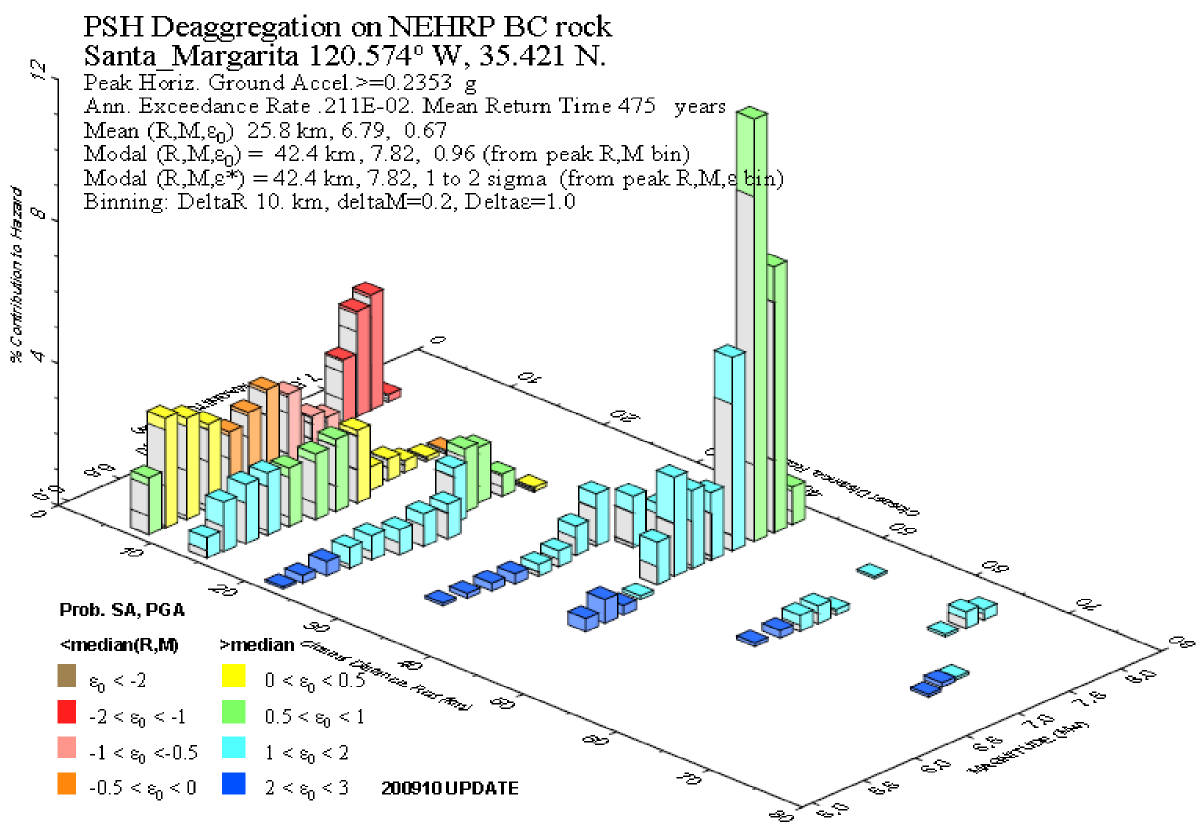
The north wall of the expansion pit will run parallel to the Salinas River, but will be separated from the river by a ridge approximately 285 ft high. The river elevation in the vicinity of the north wall is 915 ft, 35 ft above the pit bottom. As with the existing pit, the east wall of the expansion pit is parallel to the Salinas River, as described below.

2.9 Groundwater Conditions

Based on Golder's site observations, water collects in the floor of the pit on a seasonal basis to an elevation of approximately 890 ft, and a depth of approximately eight feet. Seeps were visible in the west wall at elevations as high as 1100 feet, and in the east wall at approximately 1010 ft, but the water appeared to be emanating from fault zones. The elevation of the Salinas River immediately east of the pit is approximately 940 feet, some 60 feet above the current and planned ultimate quarry bottom elevation. The exploration report indicates that no significant groundwater was encountered in the planned expansion area during core drilling. Since preparation of this report, Golder has conducted a supplemental study of groundwater conditions and hydrology of the site. This work is described in our report entitled: "Hydrogeologic Evaluation – Santa Margarita Quarry Extension" prepared for Lehigh Hanson dated February 2012.

2.10 Seismic Setting

The Santa Margarita Quarry is located on the central California Coast which is a region characterized by moderately high seismicity. The Surface Mining and Reclamation Act (SMARA) does not specify a minimum seismic design event that should be used for slope stability analyses. However, SMARA does specify that final slopes shall be flatter than the critical gradient, which is defined as the maximum stable slope inclination of an unsupported slope under the most adverse conditions (i.e. seismic loading) that it will likely experience, as determined by current engineering technology. Accordingly, Golder evaluated potential seismic impacts for the project resulting from an earthquake event associated with 10 percent probability of exceedance (POE) in a 50-year period. Golder has used the 10 percent POE in a 50-year event to evaluate seismic impacts for other Quarry reclamation projects in California, and considers this an appropriately conservative criterion for mine reclamation projects where there is little to no risk to public safety or critical structures. This criteria has previously been accepted by regulatory agencies on similar projects. Using the 2008 Update of the United States National Seismic Hazard Maps (Peterson, et.al., 2008), which incorporates the findings of the Next Generation Attenuation Relation Project, Golder estimates that design peak ground acceleration is approximately 0.24g for the site (see Figure below). This design peak ground acceleration is associated with a Magnitude 7.8 earthquake along the San Andreas fault located at a distance of approximately 42 kilometers.



3.0 AVAILABLE DATA

3.1 Previous Studies

A slope stability analysis was completed in support of the pit design in the 2004 Reclamation Plan by Earth Systems Pacific of San Luis Obispo in May 2004 (Earth Systems Pacific, 2004). The bench design specified 10-foot wide catch benches at intervals of 30 vertical feet and a bench face angle of 76°, for a slope angle of 60° between ramps (Earth Systems Pacific, 2004). The pit slope design in the 2004 Reclamation Plan is not the same pit slope design under consideration in this report.

The field investigation performed by Earth Systems Pacific consisted of structural mapping of eight windows, with each window being approximately 5-7 feet tall and 100 feet long. Due to a lack of access to the benches, all data was collected at the pit bottom. Shears and fractures were found to be nearly vertical, generally minor and discontinuous, and predominantly northwest-trending. Shears were infilled with clay. Pocket penetrometer and Torvane readings were taken of the infill, and ranged from 0.2 to 3.0 tsf. A classification test was performed on a sample of the infill, which indicated the sample consisted of sand with silt or clay (SC-SM according to the Unified Soil Classification System), and contained 47% fines.

At the time of the structural mapping, three well-defined wedge failures were observed in the pit wall. Wedge analyses were performed on two slope configurations, the planned overall slope with a height of 300 feet and a slope angle of 60°, and the individual bench configuration of 30-foot high benches with 76° bench face angles and 10-foot catch benches.

Earth Systems Pacific concluded that for the 2004 Reclamation Plan pit there was a low potential for bench-scale wedge failures, and that any failure debris would be caught by the 10-foot catch bench. They also concluded that no wedges formed in the overall slope, as the structures dipped steeper than 60° and did not daylight. Additionally, they stated that after the pit was deepened below the level of the Salinas River, water flow into the pit was unlikely because of the massive nature of the granite and the discontinuous nature of the joints, and that any water entering the pit would be in the form of a slow, intermittent spring.

Earth Systems Pacific recommended scaling the bench faces, periodic inspection of the slopes by an engineering geologist during operations, and re-vegetating the slopes after the completion of mining to reduce erosion and surficial failures.

3.2 Review of Available Core and Rock Quality Data

Hanson drilled three exploration coreholes in the expansion area in 2006 (Figure 3). Recovery and RQD (Rock Quality Designation – a modified core recovery index in which only sound core recovered in lengths of four inches or greater is counted as recovery) were recorded during geological logging of the core. Two of the drillholes were vertical, and one (SM06-1 (B1)) was inclined. No azimuth information was available for the inclined drillhole. Average RQD and recovery were calculated for each corehole and are presented in Table 1 below, along with drilling details.

Table 1 -Summary of Exploration Corehole Data

Drillhole ID	Inclination	Length (ft)	Average RQD* (%)	Average Recovery* (%)
SM06-1 (B1)	-56°	243.5	**63**	90
SM06-2 (B2)	-90°	430.0	78	93
SM06-3 (B3)	-90°	271.0	90	96

*Weighted Average

Average RQD values indicate that the rock recovered in SM06-1 classifies as Fair quality rock, while the rock in SM06-2 classifies as Good quality rock and in SM06-3 classifies as Excellent quality rock (Table 2). Plots of RQD vs. Depth are included in Appendix B.

Table 2 – Rock Quality Designation

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

SM06-01, which is located approximately 300 ft behind the expansion pit crest, was drilled to investigate the quality of rock in the vicinity of the fault that is sub-parallel to the Rinconada splay and bounds the granite block to the southwest. Core from this drillhole is more fractured than that from the other drillholes, and the moderate fracturing is spread throughout the entire drillhole. Typical core is shown in Appendix B. Fault zones occur from 87-98 ft, with the geology log indicating 10% gouge present from 90-96 ft, and from 135-146 ft, where logging indicates 30% gouge.

In SM06-02, drilled in the Northwest wall and in the vicinity of the Rinconada splay, rock quality is lower in the first 175 ft than in the rest of the drillhole, averaging 57% (Fair quality rock). The weathered granite zone is deeper than in the other drillholes, extending to approximately 93 ft. A zone of brittle fracturing was encountered from 119.5-152 ft, with 10% gouge present from 120.5-130 ft. This zone may represent the Rinconada splay. The rest of the drillhole intersected Good to Excellent quality rock, with RQD averaging 90%, except for 385-395 ft, where the core is more highly fractured, altered, and relatively soft, presumably the result of faulting. Photographs showing typical core from the upper 175 feet and from the lower 255 ft of SM06-2 are included in Appendix B.

SM06-03 was drilled near the crest of the north wall. Rock quality is Good to Excellent except for the upper weathered zone to 41 ft depth, and the interval from 252-257 ft, which appears to be a brittle fracture zone with no gouge. Typical core from this drillhole is shown in Appendix B.

Because the core had been split, and because of the evident high strength of the fresh granitic rocks relative to the slope height proposed for the quarry expansion, the split core was not sampled for rock strength testing.

3.3 Structural Data

3.3.1 Data Set

Structural orientation data was collected on faults, prominent joints, and joint sets from the 1140, 1170, 1230, and 1250 benches; the north end of the pit bottom; and the roadcut along the drill road northwest of the pit. Mapping locations are shown in Figures 2 and 3. Prominent joints and faults visible in the pit walls were also mapped. A total of 183 orientations were recorded, as summarized below (Table 3). The mapping data is included in Appendix A.

Table 3 - Structural Data Collected at Santa Margarita Quarry

Location	Faults	Shears	Dikes	Joints	Joint Sets	Total
Upper Benches	37	**8**	4	17	29	95
Pit Bottom, N. End	9	1	0	9	17	36
Roadcut	6	3	2	4	37	52
Total	52	12	6	30	83	183

The majority of discontinuities mapped were faults or joint sets. Most structures were undulating or planar. Faults and shears were generally slickensided or polished, while most of the joints were rough. The characteristics of the discontinuities mapped are summarized below in Table 4.

Table 4 -Summary of Discontinuity Properties

Type	% of Features	Shape	% of Features	Roughness	% of Features	JCR	% of Features
Faults	28	Planar	38	Very Rough	11	0-5	33
Shears	7	Stepped	4	Rough	42	6-11	4
Dikes	3	Curved	13	Smooth	16	12-16	36
Joint Sets	45	Undulating	45	Slickensided or Polished	31	17-20	25

Joints	16	Irregular	0			21-25	2

JCR is the Joint Condition Rating, a classification system for discontinuities that ranges from 0 to 25 and is used with Bieniawski's 1976 Rock Mass Ratings system (Hoek, Kaiser, and Bawden, 2000)(see Section 3.4 below).

3.3.2 Analysis of Structural Data

The structural orientation data collected during mapping was plotted on stereonets using an equal-area, lower hemisphere projection, and contoured with the Schmidt method. Figure 4 is a compilation of all of the structural data, and shows that structures are generally moderately to steeply dipping. Faults, shears, and dikes (Figure 5) generally show the same structural orientations as joints (Figure 6); a concentration of poles striking east-northeast and dipping steeply, a smaller concentration of poles striking north-south and dipping steeply, and moderately to steeply southwest-dipping structures with variable strike. Plots of data collected from the upper benches (Figure 7), the pit bottom (Figure 8), and the exploration roadcut (Figure 9) show that the steep east-northeast-striking trend is present in all three areas. Steep north-south-striking structure is present in the upper benches and the roadcut, but is not well-developed in the pit bottom. Moderately to steeply southwest-dipping structures are present in all areas as well, but are more prominent in the pit bottom. Structural trends from the mapping data are summarized in Table 5 below.

Table 5 - Structural Sets from Mapping Data

Location	Set Number	Average Orientation		Concentration (%)	Comments
		Dip (°)	Direction (°)		
Upper Benches	1a	86	166	9	
	1b	88	352	7	Same Set as 1a
	2a	77	255	7	
	2b	85	80	4	Same Set as 2a
	3	57	194	3	
Pit Bottom	1a	89	157	9	
	1b	85	335	10.5	Same Set as 1a
	2a	60	265	7.5	
	3	50	227	13.5	
	4	54	313	7.5	
Exploration Roadcut	1a	86	138	10.5	
	1b	84	327	9	Same Set as 1a
	2a	73	264	7.5	

	4	68	301	7.5	

Most large faults (faults with six or more inches of infill; Figure 10) in the mapping data strike east-northeast and dip steeply or moderately southeast (Set 1), strike northeast and dip northwest (probably steep variations of Set 4) or strike more northerly and dip steeply east (Set 2). Dikes strike generally east-west and dip steeply (Set 1).

A planar failure was observed at the north end of the east wall and occurs along a fault and fault-parallel joint set oriented approximately 50°/250° (dip/dip direction). This fault and joint set are likely flatter-dipping variations of Set 2a. A toppling failure was observed in the north end of the west wall along a fracture in Set 1a oriented 90°/155°. Localized wedge failures are present in the west wall and occur along planes oriented approximately 60°/170° and 60°/110°, with intersections trending approximately 140°/55° (trend/plunge). The south-dipping structures are likely structures in Set 1a that dip flatter than average, but the southeast-dipping structures do not appear as a concentration of poles in the mapping data.

3.4 Rock Mass Shear Strengths

Golder characterized rock mass strengths using Hoek-Brown shear strength envelopes. The approach used by Golder requires computation of a Rock Mass Rating (RMR) value using Bieniawski's RMR System (1976) as discussed below, and then development of Hoek-Brown shear strength envelopes for the material. This approach is commonly used to evaluate global stability of rock slopes and has been in use for several decades.

3.4.1 Bieniawski Rock Mass Rating System

Bieniawski's RMR System[1] is a semi-quantitative assessment of rock quality based on the rocks intact strength as measured by unconfined compressive strengths (UCS), Rock Quality Designation (RQD)[2], joint spacing and condition, and groundwater conditions (see Table 6 next page).

[1] Bieniawski, Z.T., 1976. Rock Mass Classifications in Rock Engineering. Proceedings Symposium on Exploration for Rock Engineering, ed. Z.T. Bieniawski. A.A. Balkema Rotterdam, pp. 97-106

[2] RQD is the percentage of intact core lengths that are large than twice the core diameter

Table 6 - Rock Mass Rating System

Rock Mass Rating							
Parameter	Range of Values						
UCS Rating	>200 MPa (15)	100-200 MPa (12)	50-100 MPa (7)	25-50Mpa (4)	10-25 (2)	<3-10 (1)	1-3 (0)
RQD Rating	90-100% (20)	75-90% (17)	50-75% (13)	25-50% (8)	<25% (3)		
Joint Spacing Rating	>3m (30)	1-3m (25)	0.3-1m (20)	50-300mm (10)	<50mm (5)		
Joint Condition Rating	Very rough No separation Hard wall rock (25)	Slightly rough Separation<1mm Hard wall rock (20)	Slightly rough Separation<1mm Soft wall rock (12)	Slickensides, Separation or gouge <5mm (6)	Soft gouge or Separation >5mm (0)		
Groundwater Rating	Completely Dry (10)		Moist (7)	Mod. Pressure (4)	Severe (0)		

Total RMR Value (Sum of Ratings for 5 Items) =					
Rating	100 – 81	80 – 61	60 – 41	40 - 21	20 - 0
Description	I – Very Good	II – Good	III – Fair	IV - Poor	V - Very Poor

These parameters are summarized below for the Santa Margarita Quarry. Based on the above parameter characterization, Table 7 summarizes the computed RMR values for each of the three coreholes drilled at the site, and for weathered and fresh granite.

- Based on Golder's experience, a UCS value of 20,000 psi was assumed for the intact rock and 10,000 psi for the overlying weathered rock. The UCS for fresh granite typically ranges from 15,000 psi to 35,000 psi. Goodman(1980) notes example of granite UCS values that range from 20,500 psi to 32,800 psi. Based on our observation of the rock core and existing cut slopes in fresh and weathered granite at the quarry, the assumed UCS values are considered conservative.
- Based on the rock core from the site, Golder estimates that the average RQD for the weathered granite to be 40 percent and the intact granite ranging from 60 to 90 percent. Golder used an average 75 percent for the intact granite RQD.
- Based on the relatively high RQD values, and core photos, the joint spacing was assumed to range from 50 to 300 mm in the weathered granite and up to 0.3 to 1 meter for the intact granite.
- Based on observed drill core logs, the joints were assumed to have a separation of less than 5 mm for the weathered granite. The joints for the intact granite were assumed to be "slightly rough, have a separation of less than 1 mm with hard wall rock."
- Groundwater was assumed to be "moist" to "Semi-moist" for the weathered zone and "moderate" for the intact granite zone.

Based on the above parameter characterization, Table 7 below summarizes the computed RMR values for each geologic unit.

TABLE 7 - ESTIMATED RMR VALUES

COREHOLE	ROCK CHARACTER	ROCK MASS RATING VALUES					
		UCS	RQD	JOINT SPACING	JOINT CONDITION	GROUND-WATER	RMR TOTAL
SM06-1	Weathered	7	8	10	6	8	39
	Fresh	12	17	15	20	6	70
SMO6-2	Weathered	7	8	10	6	8	39
	Fresh	12	17	20	20	6	75
SMO6-3	Weathered	7	8	10	6	8	39
	Fresh	15	20	25	20	6	86

For Bieniawski's RMR System, an RMR value of 39 for the weathered rock represents the upper end of "Poor Quality" rock. RMR values of 70 to 75 for the fresh granite in borings 1 and 2 occur within the middle to upper range of "Good Quality" rock. An RMR value of 86 for the fresh granite in Boring 3 classifies as "Very Good Quality" rock.

3.4.2 Hoek-Brown Strength Criterion

The Hoek-Brown criterion is the most widely-accepted method of estimating rock mass shear strength in rock masses comprised of brittle, fractured rock. Non-linear, Hoek-Brown shear strength envelopes were developed for the weathered and fresh granite based on the assumed UCS values, the above RMR values, and rock types using the methodology developed by Hoek and Brown[3].

This criterion defines the relationship between major principal stress and minor principal stress at the time of failure based on the following equation:

$$\sigma_1 = \sigma_3 + \sqrt{\mathrm{m} \cdot \sigma_{\mathrm{ci}} \cdot \sigma_3 + \mathrm{s}\sigma_{\mathrm{ci}}{}^2}$$

where:

σ_1= the major principal stress at failure
σ_3 = the minor principal stress (confining stress)
m, s = Hoek-Brown material constants for rock mass
σ_{ci} = the uniaxial compressive strength of intact rock

[3] Hoek, E. and E.T. Brown, 1988, "The Hoek-Brown Failure Criterion – a 1988 Update." Proceedings of the 15th Canada Rock Mechanics Symposium, University of Toronto, pp. 31-38.

This is an empirical method, originally derived by fitting curves (i.e., shear strength envelopes) to the Mohr circle results from large-dimension triaxial compression tests on core samples of fractured rock. The curves represented shear strength envelopes for the fractured rock described in terms of “Hoek-Brown” parameters termed *m, s, and a,* which could be applied directly to field-scale rock masses. Based on Golder’s experience, the shear strength envelopes developed for the weathered and fresh granite appear appropriately conservative and applicable for global stability analyses for the Quarry.

4.0 SLOPE STABILITY EVALUATIONS

4.1 Regulatory Criteria

The Surface Mining and Reclamation Act (SMARA) does not specify a minimum factor of safety for slope stability for final reclaimed slopes. However, Section 3502(b)(3) indicates that final reclaimed slopes shall be flatter than the critical gradient, which implies that static factors of safety (FOS) should be greater than 1.0. This section further states "Wherever final slopes approach the critical gradient for the type of material involved, regulatory agencies shall require an engineering analysis of slope stability. Special emphasis on slope stability and design shall be taken when public safety or adjacent property are affected." Section 3704(f) states that "Cut slopes, including final highwalls and quarry faces, shall have a minimum slope stability factor of safety that is suitable for the proposed end use and conform with the surrounding topography and/or approved end use."

Given the significant height of the existing quarry slopes (approximately 400 feet), and potential proximity of the highwall to the ridgecrest (view shed), we consider a minimum factor of safety of 1.5 under static conditions for global stability to be appropriate and consistent with SMARA for the Quarry.

There are a number of methods that have been proposed in selecting a seismic coefficient for use in pseudo-static analyses. Pyke (1991) proposed a relationship between the PGA, seismic coefficient, and moment magnitude of an earthquake. For the Santa Margarita Quarry, this relationship implies a seismic coefficient of 0.10.

Golder used a more conservative approach based on the methodology proposed by Stewart et al. (2003). Considering a relatively small allowable permanent displacement of 2 inches or less, this approach yields a seismic coefficient to PGA ratio of approximately 0.74. Therefore, Golder used a seismic coefficient (k) of 0.20 in the pseudo-static analyses. This seismic coefficient was rounded up from the computed seismic coefficient of 0.18 (k= 0.24g x 0.74 = 0.18). A seismic coefficient of 0.2 is considered conservative for a design peak ground acceleration (PGA) of 0.24g. Using this approach, a pseudo-static factor of safety greater than 1.0 is considered acceptable.

As described below, the computed factors of safety for both static and pseudo-static slope stability exceed the minimum factors of safety deemed acceptable by Golder at this site by a significant margin.

4.2 Approach to Slope Stability Evaluations

Slope failures in relatively competent rock, such as the granite at the Santa Margarita quarry, are generally controlled by failure along one or more rock mass discontinuities, which is referred to as a kinematic failure mode. Rock mass discontinuities include joints, bedding planes, shear zones, faults, and foliation. Stability may be controlled by a single persistent structure, by several through-going joints defining blocks or wedges, or by numerous joints giving rise to a closely jointed, blocky rock mass.

Kinematic failure modes can include:

- Planar failures – sliding along a single discontinuity
- Wedge failure – sliding along two discontinuities
- Toppling failure – rotation out of the slope due to steeply dipping discontinuities

For all of these cases, stability is a function of the specific geometry and strength properties of the discontinuities present at a given location, groundwater and external loading conditions, and of the geometry of the slope under consideration. Viable kinematic failure modes were analyzed using industry standard methods and procedures (Section 4.3).

Rock slope stability can also be controlled by overall rock mass properties. In this case, slip surfaces are generally considered to be circular, and stability is in part a function of rock mass shear strength, rather than the orientations and properties of discrete structures. Golder evaluated global stability for representative highwall of the quarry, and also evaluated the potential for more limited failures within the weathered rock mass located within 50 feet of the original ground surface (Section 4.4).

4.3 Kinematic Slope Stability Analyses

The fresh granitic rock has a high strength relative to the proposed slope heights, which Golder believes will preclude the development of slope failures through intact fresh rock. Weathered granitic rocks that will be encountered in the upper portions of the slopes will also generally have adequate strength to preclude failure through intact rock. It is our opinion that any slope failures that develop within the fresh and weathered rock will be controlled by geological structures, or small-scale instability related to zones of local fracturing or blast damage. Therefore, the approach to evaluating stability of fresh rock is to evaluate the character and distribution of geological structures within the rock, and then to analyze the potential effect of these structures on bench-scale and overall slope stability. Where there are no structural controls of stability, then bench stability and overall slope designs will primarily be determined by operating practices, particularly blasting and excavation.

Kinematic analyses are used to identify <u>potentially</u> viable kinematic failure modes: The identification of potentially viable failure modes does not mean that a significant slope failure(s) will occur. They instead simply indicate that failure could occur if the discontinuities are 1) sufficiently continuous that they intersect the slope face and 2) also have low strength properties that will allow movement along the discontinuitie(s). Additional judgment regarding the continuity, frequency, and physical character of various discontinuities is applied to help assess the likelihood of kinematic failures.

4.3.1 Design Structural Sets

Design structural sets were developed from the structural mapping data (Section 3.3) and are tabulated below (Table 8). The design orientations for Sets 1a, 1b, 2a, 3, and 4 were obtained by averaging the values listed in the above Table 5 (Section 3.3). The southeast-dipping structures forming wedges with Set 1a were also included as Set 5.

Table 8 - Design Structural Sets

Set Number	Average Orientation		Comments
	Dip (°)	Direction (°)	
1a	87	154	Toppling failure at north end of current pit; moderately-dipping structures in this set form wedges with Set 5 in current pit
1b	86	338	Same Set as 1a
2a	70	261	Moderately-dipping variations form planar failures in northeast wall of current pit
2b	85	080	Same Set as 2a
3	54	211	Prominent at north end of pit, present in all areas mapped
4	61	307	Present in all areas mapped but not strongly developed
5	60	110	From wedges with Set 1a; not in mapping data

4.3.2 Kinematic Analysis of Slopes in Proposed Ultimate Pit

Because the stability of rock slopes controlled by geological structures is a function of the relative orientations of the slopes and the structures, the proposed pit has been divided into slope design sectors as shown in Figure 3. The pit extension area slopes are oriented mainly east-west (North sector), northwest-southeast (Northwest sector), and north-south (West sector), while the current pit slopes are oriented mainly northeast-southwest.

Where structurally-controlled failures are indicated to be kinematically possible, they have been summarized in Table 9 below. Where potential risks of failure are identified, appropriate mitigating design measures are recommended for the pitslope design (see Section 5.2.3).

The structural controls identified by the kinematic analysis are summarized in the following table and discussed in the following sections:

Table 9 - Summary of Structural Controls

<table>
<tr><th>Sector</th><th>Mode</th><th>Sets</th><th>Comments</th></tr>
<tr><td rowspan="4">Northwest</td><td>Planar</td><td>--</td><td>None indicated</td></tr>
<tr><td rowspan="2">Wedge</td><td>1 & 5</td><td>Intersection oriented 065°/50°; form wedges in current west wall</td></tr>
<tr><td>2b & 4</td><td>Intersection oriented 356°/50°</td></tr>
<tr><td>Toppling</td><td>--</td><td>May occur along structures parallel to Rinconada Splay</td></tr>
</table>

Sector	Mode	Sets	Comments
West	Planar	5	Strikes 30° from strike of slope
	Wedge	1 & 5	Intersection oriented 065°/50°; form wedges in current west wall
	Toppling	2a	Possible along structures steeper than average
North	Planar	3	Flatten bench face angles to reduce back-break
	Wedge	2a & 3	Intersection oriented 201°/54°
		2b & 3	Intersection oriented 165°/44°
		3 & 5	Intersection oriented 166°/44°
	Toppling	1b	Possible along structures dipping more to the north than average
Northeast	Planar	2a	Risk of local multi-bench planar failure; planar failures along flatter variations of this set in east wall of current pit
		4	Not strongly developed in mapping data; flatten bench face angles to reduce back-break
	Wedge	1a & 3	Intersection oriented 240°/50°
		1b & 3	Intersection oriented 252°/46°
		3 & 4	Intersection oriented 252°/46°
	Toppling	--	None indicated
East	Planar	4	Not strongly developed in mapping data; flatten bench face angles to reduce back-break
	Wedge	1b & 3	Intersection oriented 252°/46°
		2a & 4	Intersection oriented 310°/61°
		2b & 4	Intersection oriented 356°/50°.
	Toppling	--	None indicated

4.3.2.1 Northwest Sector

The general dip direction of the northwest wall is 032° (Figure 11). Toppling may occur in this slope if closely-spaced structures parallel to the Rinconada splay are present, although no systematic shallow structures to act as base planes are indicated by the mapping data. Wedges may form along the intersection of Sets 1 and 5, with wedge intersections oriented 065°/50° (trend/plunge). Sets 2b and 4 also form potential wedges, with intersections oriented 356°/50°.

4.3.2.2 West Sector

The dip direction of the walls in the West sector is 081° (Figure 11). Localized planar failures are possible along Set 5, which has an average dip direction about 30° from that of the slope. These are not expected to be extensive because the difference in dip direction between the slope and Set 5 is greater than 20°, and so the lateral extent of any such failures would be limited. Toppling may occur along steeper than average structures in Set 2a, a prominent set in the current pit, although these failures are not expected to be widespread due to the apparent lack of shallow structures to act as base planes. As in the northwest wall, the intersection of Sets 1 and 5 may form wedges with intersections oriented 065°/50° (trend/plunge).

4.3.2.3 North Sector

The general dip direction of the north wall is 187° (Figure 12). The dip direction of Set 3 is 24° from that of the north wall, and undercutting this set could cause planar failures if the set is well-developed in this area. Set 3 is present in all areas that were mapped, and is particularly strong in the north end of the current pit. This set presents a risk of bench-scale and multi-bench failures unless it is considered in the slope design for this sector. Recommended design slope angles are flattened to mitigate this potential (Section 5.2.3.3).

For this wall orientation, there is a potential for wedges to form along intersections of Set 2a and 2b with 3 (intersections 201°/54° and 165°/44°, respectively), and Set 3 with 5 (intersection 166°/44°). Wedges are also formed by Sets 1 and 4 and Sets 2b and 5, but with intersections too shallow to allow for failure. Additionally, isolated toppling failures are possible along structures in Set 1a where structures in that set dip more to the north than average.

4.3.2.4 Northeast Sector

In the Northeast sector, the average dip direction of the slopes is 282° (Figure 13). The average dip direction of Set 2a is 21° from the average dip direction of the wall in this sector. Set 2a is prominent in this area, as evidenced by bench-scale planar failures along it in the current east wall, and therefore presents a potential risk for planar failures in this sector. Recommended design slope angles are flattened to mitigate this potential (Section 5.2.3.3).

Set 4 is oriented nearly parallel to the pit wall, and planar failures are possible along this set if well developed, however, existing data indicates it is not strongly developed therefore the potential for failure is considered low. Wedges are formed by the intersection of Sets 1a and 1b with Set 3, and Set 3 with Set 4. The trend and plunge of the lines of intersection of the wedges formed by Sets 1b and 3 and Sets 3 and 4 are 252°/46°, and by Set 1a and 3 are 240°/50°. Recommended design slope angles are flattened to mitigate this potential (Section 5.2.3.3). Wedges are also formed by the intersection of Sets 1 and 4, however, intersections are too flat lying to allow for failure. No toppling risks in the east wall are indicated by the mapping data.

4.3.2.5 East Sector

The dip direction of the wall in the East sector is generally 313° (Figure 13). Set 4 is oriented within 6° of the pit wall. As in the Northeast Sector, planar failures are possible along this set if it is well developed in this area. Slope angles area flattened to mitigate this potential (Section 5.2.3.3). The intersection of Sets 2a and 2b with Set 4, and Set 1b with Set 3 form wedges. The lines of intersection are oriented 310/61° for Sets 2a and 4, 356/50° for Sets 2b and 4, and 252/46° for Sets 1b and 3. Recommended design slope angles are flattened to mitigate this potential (Section 5.2.3.3). No toppling risks in the east wall are indicated by the mapping data.

4.3.3 Summary of Kinematic Analyses

The analyses indicate that wedge and planar failures are potentially viable kinematic failure modes, particularly for joints and shears in the North, Northeast, and East sectors. Where these discontinuities or intersections have been identified dipping more steeply than 30 degrees out of slope, slopes have been flattened to minimize the potential for these failure types (see Section 5.2.3) Although kinematically viable, the toppling failure mode in the granite is considered to be unlikely due to the lack of flat lying discontinuities to act as base planes.

In our opinion, the discontinuity data, in conjunction with the rock type, indicate that kinematic failures are likely to be limited to relatively localized bench scale failures that are common to most hard rock quarries. For the proposed end use of the quarry as open space, these types of failures do not result in significant impacts for the reclamation condition, and can be managed effectively during operations through sound blasting and excavation practices.

4.4 Global Stability – Limit Equilibrium Analyses

Limit equilibrium stability analysis was performed using the program SLIDE (Version 6.015). SLIDE uses a two-dimensional, limit equilibrium method of slices to compute factors of safety. Golder evaluated global stability for the following representative slope conditions:

- A unit weight of 165 pounds per cubic foot (PCF) was assumed for both types of granite material; weathered and fresh rock.
- Groundwater was assumed to occur about 100 feet below ground surface at the ridgecrest and daylights at the toe of the highwall.
- Section A – This section consists of a generalized 460-foot high slope with 1H:1V overall slopes in granite and 70 degree bench face angles. This section is considered a conservative representation of the majority of the quarry as it includes the tallest and steepest slopes proposed for the site. A tension crack was defined in the upper of the slope for Section A-A effectively reducing shear strength contribution of the slip surface segment from the weathered zone.
- Section D – This section consists of a 150-foot tall section of weathered granite exposed above the highwall. This condition is considered representative of the worst case condition in the southwest sector of the quarry where the slopes are tallest. The slopes were conservatively modeled at an inclination of 1.25H :1V, although most weathered rock slopes will be graded at 1.5H:1V. The weathered granite strength envelope was used for the modeling and the failure surface was constrained to the weathered profile.

The results of our stability analyses are summarized in Table 10 below. The output from the SLIDE program is included in Appendix C.

TABLE 10
COMPUTED FACTORS OF SAFETY

SECTION	COMPUTED FACTORS OF SAFETY	
	STATIC	PSEUDOSTATIC
Section A – Failure of the Entire Slope	5.7	4.4
Section D – Failure Only Within Weathered Rock	3.4	2.5

Golder’s conservative slope stability model results in computed factors of safety for global stability of the entire slope and the weathered rock profile exceed the minimum value of 1.5 for static conditions. Pseudostatic factors of safety for the above failure modes were computed to be 4.4 and 2.5, respectively, for a seismic coefficient of 0.2, which is considerably higher than the minimum of 1.0 that Golder considers appropriate for this evaluation.

5.0 SLOPE DESIGN

5.1 General Conditions

Rock quality and structural conditions generally appear favorable for the development of moderately steep to steep inter-ramp slopes throughout the pit unless extensive fault zones are encountered. Specific conclusions supported by the characterization and analyses described in the previous sections include:

- Rock mass strength is sufficiently high to preclude rock mass failure of overall slopes. The slope angles that can be achieved will depend on the stable bench configurations that can be developed safely; this will be a function of both structural conditions and operating practices.
- Groundwater is not expected to influence slope stability, and no slope dewatering for slope stability is currently anticipated.
- There is little clay alteration evident, and low shear strengths associated with clay alteration are not anticipated to be a control on large-scale or bench stability.
- Kinematic stability analyses indicate potential for structurally-controlled planar failures along joints and faults that will dip out of the North, Northeast, and East sector slopes of the pit expansion. The bench face angles are designed to account for these structures and mitigate the potential for this failure mode.
- In other pit sectors, where high risks of planar failures have not been identified, steep bench face angles should generally be achievable with careful blasting, excavation, and scaling. While wedge or toppling failures may occur locally, available structural data does not indicate these failure mechanisms to be a widespread control on bench design.
- Zones of intense fracturing that may be encountered in association with faulting or elsewhere, and that could result in a decrease in stable bench face angles, are currently understood to be relatively narrow, and will affect bench stability only over limited lengths and should not require major slope re-design.

5.2 Slope Design Recommendations

5.2.1 Slope Design Rationale

Using the average orientations of systematic discontinuity sets as a basis for evaluating structural control of bench stability is a reasonable practice given the currently available structure data. Kinematic analyses based on these average orientations indicated little potential for structural control of slope angles in the Northwest and West sectors, but a moderate to high potential for development of planar failures along joints and shears in the North, Northeast, and East sectors if design bench faces angles are too steep. Therefore recommendations are provided to mitigate these risks (Section 5.2.3).

Slopes could theoretically be designed to avoid all possible failures, but this would result in unreasonably flat slope angles and impacts on production that are unnecessary. Rather than designing to avoid all failures, it is generally acceptable from an operational perspective to assume that some minor bench-scale failures may occur, and that these failures can be minimized by good operational practices and effective catch benches.

5.2.2 General Design Parameters

Since there appears to be little potential for rock mass or structural control of overall or inter-ramp slope angles, achievable pit slope angles will be determined by the bench configurations that can be safely developed and maintained. Bench configurations are defined by production bench height, achievable bench face angle (BFA), and catch bench width, all of which combine to define the inter-ramp angle (IRA) as shown in Figure 14. Our recommended bench configurations are given in terms of these parameters, but include the following assumptions:

- Production bench height is 50 ft.
- All slopes will be developed using a single bench configuration.
- Catch bench widths should be sufficient to provide effective protection against rockfall. The following modified Ritchie criteria (Ryan and Pryor, 2000) is commonly used for initial estimates of design catch bench width:
  - Catch Bench Width (ft) = (0.2 x Bench Height) + 15 ft.
  - For 50 ft height between catch benches, this results in a design catch bench width of 25 ft.
- Minimum design catch bench widths in the mining industry are commonly taken as 20 ft to allow for back-break and hard toes due to imperfect blasting, and local bench crest failures due to structural conditions; narrower design catch benches are not generally assumed unless operating experience can demonstrate that effective catch benches can be constructed at narrower designs.
- BFA and IRA are limited by structural control for slopes oriented within +/-20° of the dip direction of structures susceptible to planar failure modes, and +/- 45° of the trend of potential wedge failure modes.
- If strong structural control at flatter angles is lacking, the following BFA ranges are typically achievable, depending on rock quality and blasting methods:
  - Standard production blasting - 55°-65°
  - Effective controlled blasting - 65°-70°
  - Best-case controlled blasting - 70°-75°
- Since catch bench width for a given bench height is constant according to the modified Ritchie criteria, maximizing the IRA will be contingent on excavating the BFA as steep as possible. We have assumed a BFA consistent with the implementation of effective controlled blasting at the pit limit.

Operational considerations to achieve maximum achievable and safe pit slope angles are provided in Appendix D.

5.2.3 Recommended Design Parameters by Sector

5.2.3.1 Weathered Granite, All Sectors

Weathered Granite is estimated to range from approximately 25 to 50 ft thick, should be excavated no steeper than 1.25(H):1(V) slope, and should be excavated back from the pit crest a minimum of 10 ft to form a catch bench to catch any raveling.

5.2.3.2 West and Northwest Sectors

Fresh Granite should be excavated in a single bench configuration with design 25-ft catch benches at 50-ft vertical intervals. Design bench face angles of 70° appear feasible provided an effective pre-split is implemented at the pit limit, and will result in an inter-ramp slope angle of 49° (Figure 15). Based on the bedrock design in the Northwest and West walls, an effective pre-split blast will be required to develop clean bench faces. Inter-ramp slope angles on the order of 5° lower can be expected without the use of an effective pre-split.

5.2.3.3 North, Northeast and East Sectors

In the North sector, structural mapping data indicates the presence of a set of faults and joints dipping 50° to 65° to the southwest in the pit bottom, upper benches, and exploration roadcut. Bench face angles in weathered and fresh granite in the North wall are designed at 60° to reduce the potential for planar failures and back-break along these structures so that effective catch benches can be developed per MSHA regulations.

Moderately to well-developed structures are also indicated to dip into the pit and approximately parallel the slopes in the Northeast and East sectors, and bench face angles in these sectors are also designed at 60° to account for the effects of these structures.

Fresh granite should be single-benched, with 25-ft catch benches at 50-ft vertical intervals, for an inter-ramp slope angle of 43°, as illustrated in Figure 15. It should be possible to develop these moderate bench face angles without pre-split blasting, but effective cushion blasting should be implemented to limit blast damage and back-break.

There is potential for increasing the bench face angle in these sectors if the in-dipping structures are not well-developed. If after the first few benches are developed in bedrock these structures are determined to be poorly developed or not present, the bench face angles could be increased to provide similar bench configurations and slope angles as in the Northwest and West walls. Alternatively, oriented coring could be used to determine the presence and prevalence of these structures prior to expansion of the pit.

5.2.4 Summary of Recommended Slope Designs

The recommended design slope configurations illustrated in Figure 15 and summarized below (Table 11) should be safely achievable if an effective pre-split is implemented in the fresh rock in the Northwest and West sectors. Effective cushion blasting should be adequate for the design bench configurations in weathered rock and for the flatter design bench face angles in fresh rock in the North, Northeast, and East sectors. Bench-scale failures may still occur where structural conditions are locally unfavorable.

Table 11 - Recommended Slope Designs

Sector	Granite Type	Bench Configuration	Bench Height (ft)	Catch Bench Width (ft)	Bench Face Angle (°)	Design Inter-Ramp Slope Angle (°)
All	Weathered	Single	Varies	10 min	1.25(H):1(V)	Varies with height
Northwest and West	Fresh	Single	50	25	70[1]	49
North, Northeast, and East	Fresh	Single	50	25	60	43

[1] Assumes effective inclined pre-split used to develop clean bench faces

6.0 SUMMARY AND CONCLUSIONS

1. There is no indication of large-scale, deep-seated slope failures involving the rock mass at the Santa Margarita quarry, which supports our expectation that the high strength of the fresh granitic rocks will preclude large-scale, global rock mass failure. We consider the overall stability of the slopes to be in compliance with Section 3704(f) of SMARA which states that "Cut slopes, including final highwalls and quarry faces, shall have a minimum slope stability factor of safety that is suitable for the proposed end use and conform with the surrounding topography and/or approved end use."

2. Slope failures controlled by continuous faults are limited in extent because of the steep dip of the faults and their orientation relative to the quarry slopes. Faults generally dip too steeply to control overall slope stability, and their influence on quarry slope stability in the pit expansion is expected to be limited to bench-scale instability over the exposed width of the fault zones. Unstable zones of limited width may extend over the full height of the slope, but will not result in large-scale instability or failure of the overall slope.

3. Appropriate design and operational recommendations will limit the potential for bench-scale failures controlled by slope-parallel structures provided that the benches are designed to account for this structure. Design bench face angles should correspond to the approximate dip of the structure where structures are well-developed, pervasive, dip into the pit, and strike within 20°-25° of the slope orientation. These conditions appear to apply at the North, Northeast, and East sectors.

4. Existing catch bench development varies from adequate in the upper benches of the west slope and the lowest bench of the same slope, to minimal in the middle and lower portion of the west slope. Operational considerations and practices directed toward overall slope performance, operational safety, and catch bench development are provided in Appendix D.

5. Our observations of the distribution of major structures and structural sets at the Santa Margarita quarry indicate that they are not ubiquitous, and so multiple structures and/or structural sets that could combine to form wedges occur only intermittently. This is supported by the local and intermittent occurrence of wedges in the west slope of the existing pit. For this reason, we do not consider that it is necessary to design slopes at flat angles to eliminate the risk of wedge failures, since the occurrence of occasional bench-scale wedge failures is generally acceptable to most mining operations.

6. Bench-scale toppling failures may occur locally, but are not expected to be a control on slope design. Good blasting practices will minimize the risk of toppling failures developing, and development of effective catch benches will mitigate the risks of rockfall hazards that can be associated with toppling failures.

7. Groundwater is not expected to be a significant factor for slope stability because the pit will not be deepened, and there is little indication of groundwater inflows except locally along structures. While increased inflows may occur as the west wall is pushed back, or as the north slope is developed closer to the Salinas River, it is unlikely that sufficient groundwater pressures would be encountered to affect stability in the slopes in competent granitic rocks. Increased pit inflows that could require additional dewatering for pit bottom operations may occur as the north slope is developed closer to the Salinas River. This should not affect stability in fresh granitic rocks.

6.1 Recommended Geotechnical Review

The slope design recommendations provided are based on a geotechnical model developed from available geological information that includes surface mapping and exploration core drilling. These recommendations are appropriately conservative because they reflect the assumption that actual conditions in the vicinity of the Ultimate pit slopes generally conform to the conditions observed and evaluated during Golder's investigation.

Golder recommends that a qualified engineering geologist or geotechnical engineer experienced in evaluating the stability of hard rock slopes inspect the quarry slopes annually. These inspections should summarize the rock types observed, provide detailed rock mass descriptions and measured discontinuity orientations, observed seepage conditions, and compare the observed conditions relative to that described in this report. If the conditions vary from Golder's characterization, the engineering geologist or geotechnical engineer should evaluate whether the changes have an adverse impact on slope stability, and if so, provide recommendations to mitigate the slope stability concerns.

7.0 REFERENCES

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8.0 CLOSING

It has been a pleasure to assist on this interesting project. Please call or e-mail if you have any questions or if we can be of further assistance.

GOLDER ASSOCIATES INC.

Graeme Major

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Principal Geotechnical Engineer

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William L. Fowler, P.G., C.E.G.
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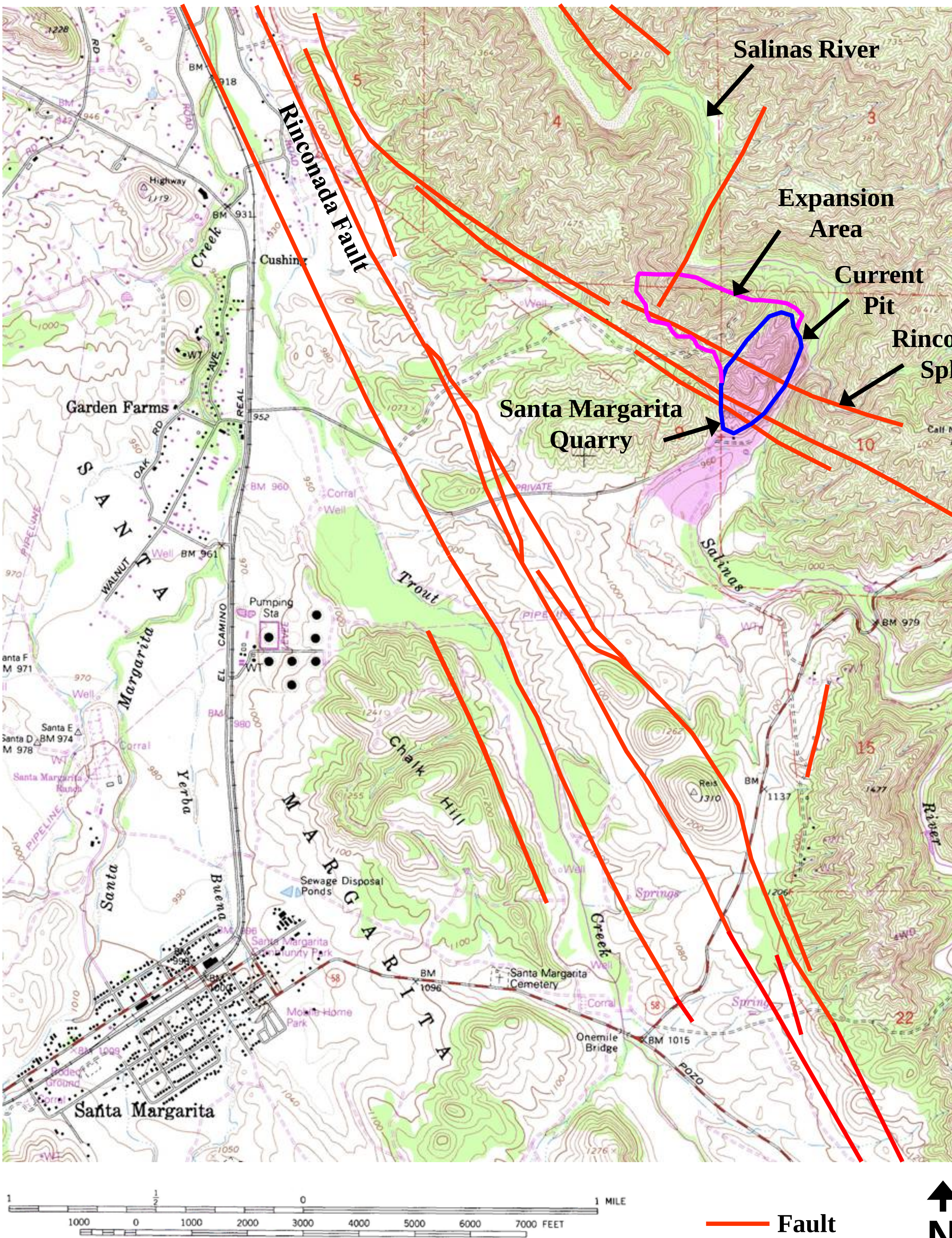
FIGURES



Source: USGS 7.5 Minute Topographic Map (Santa Margarita); Hart, 1976

Golder Associates

TITLE

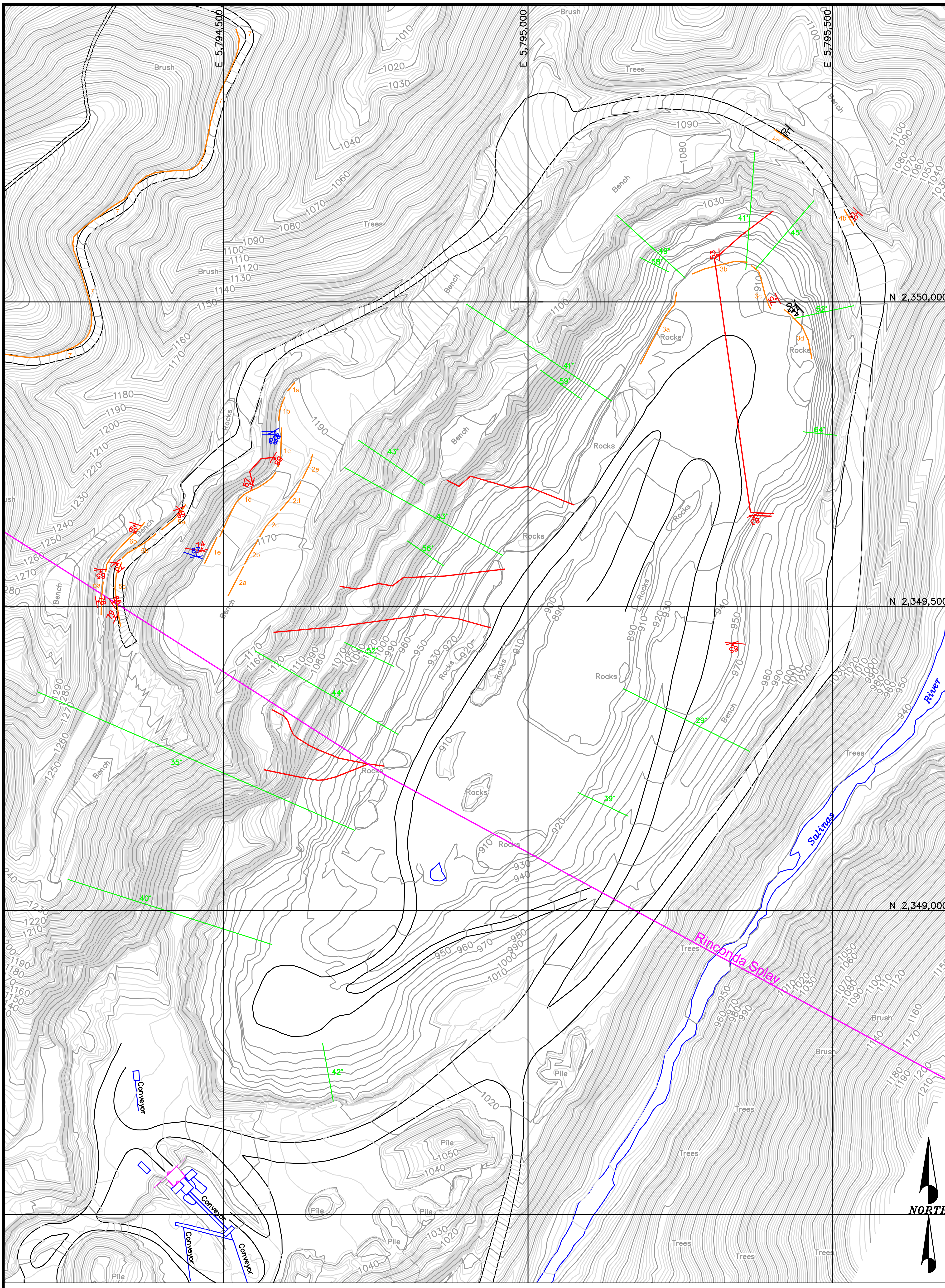
SITE LOCATION MAP

CLIENT/PROJECT

Hanson

SANTA MARGARITA QUARRY

DRAWN RK	DATE 3/28/08	JOB NO. 073-97199
CHECKED GM	SCALE NA	DWG. NO. / REV. NO.
REVIEWED GM	FILE NO.	FIGURE 1



Explanation

Strike and Dip of Fault

Strike and Dip of Dike

Strike and Dip of Joint

Structural Mapping Location

Fault

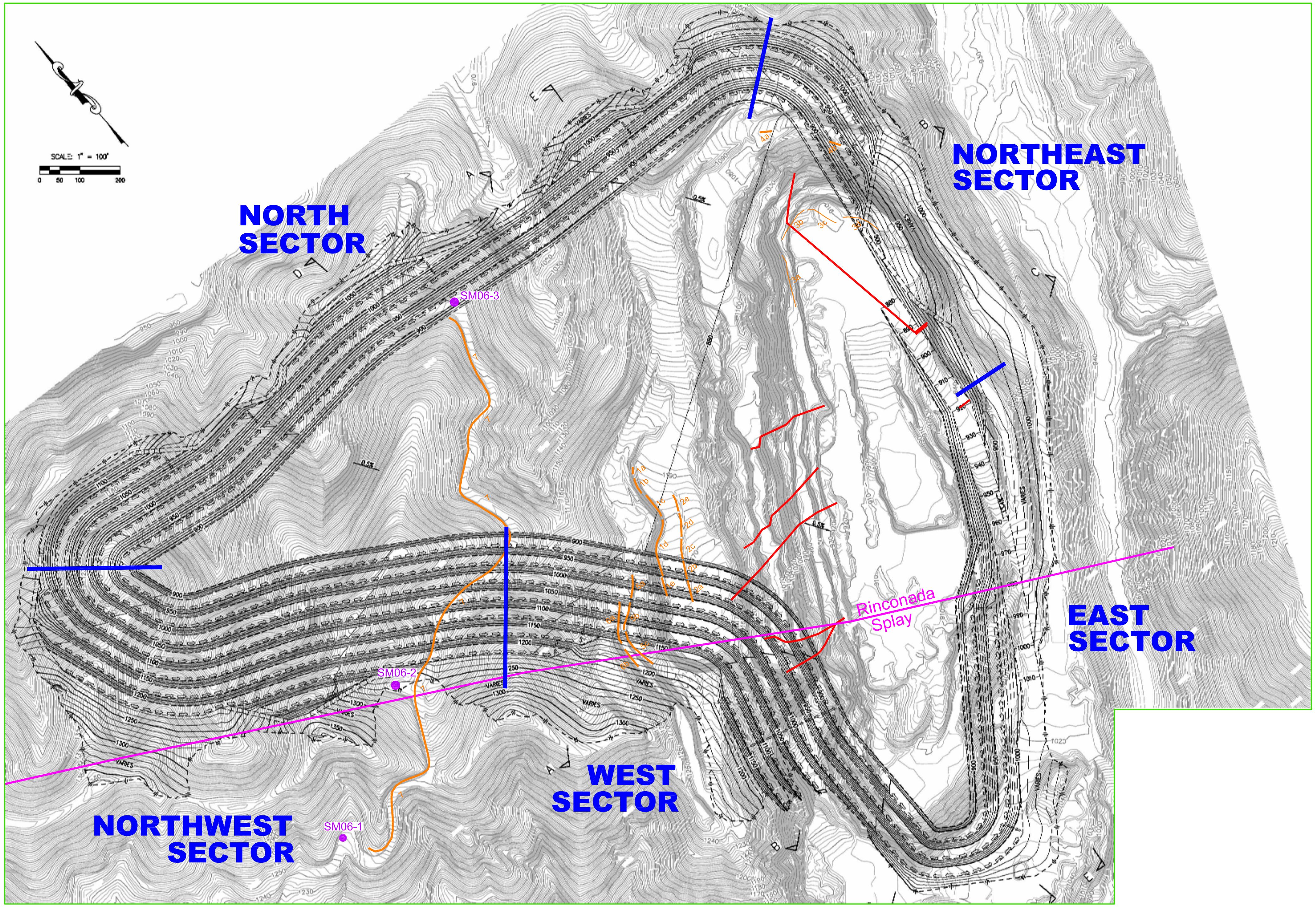
Splay of Rinconda Fault (Hart, 1976)

Slope Angle

Golder Associates
RENO, NEVADA

PROJECT No.073-97199 DATE 5/12/08

QUARRY GEOLOGY
FIGURE 2

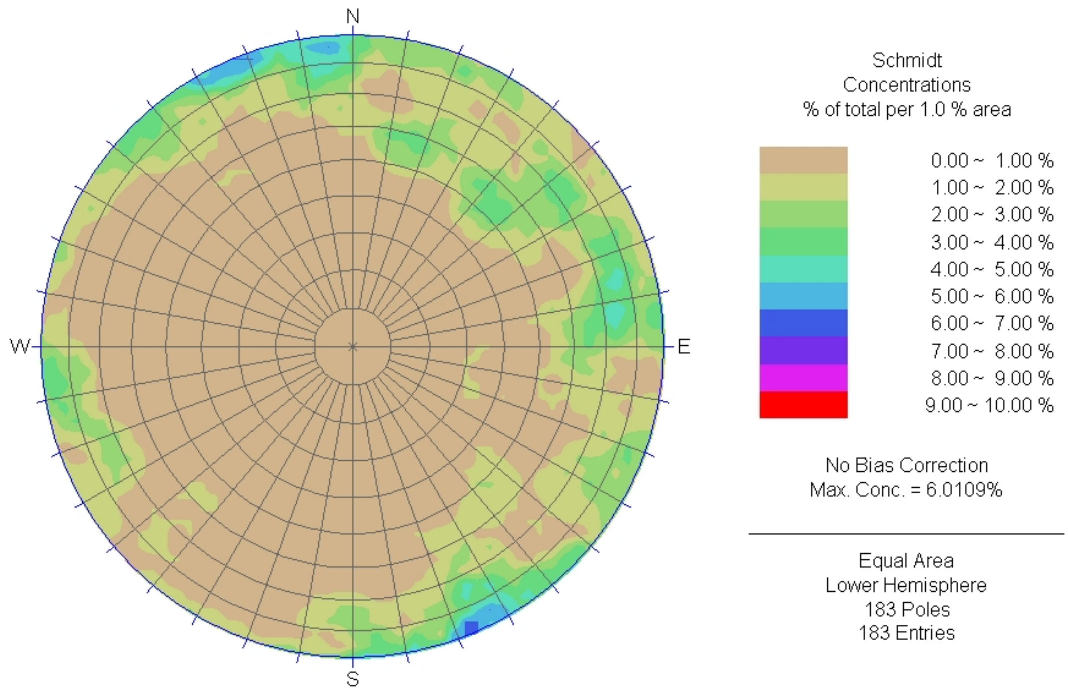


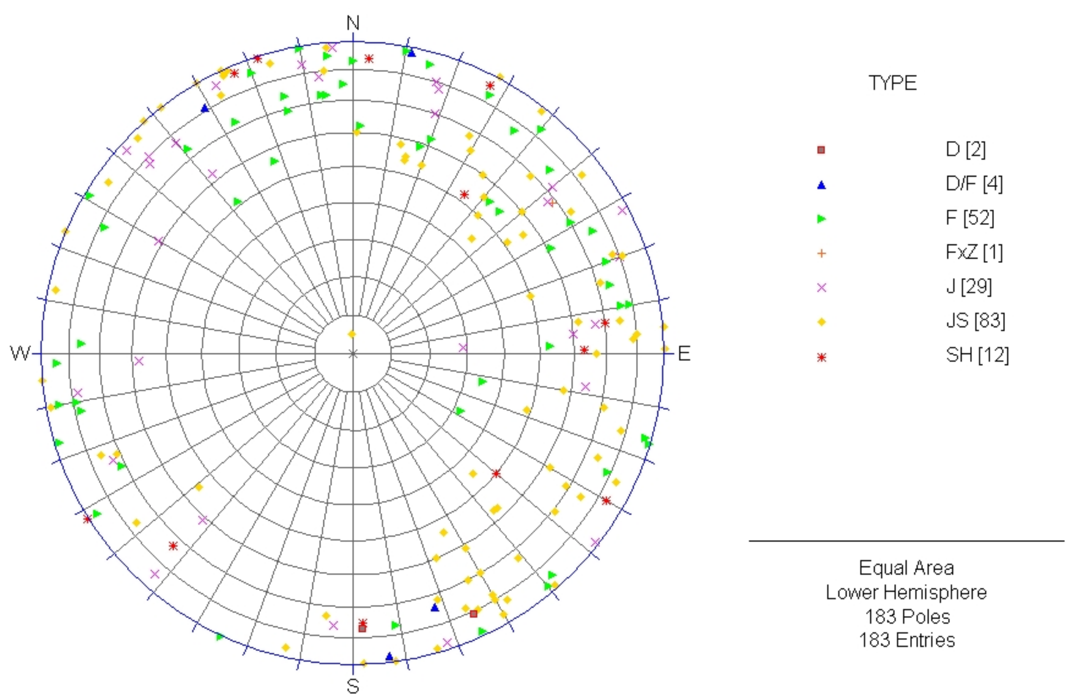
Explanation

Structural Mapping Location
SM06-2 Exploration Drillhole
Fault
Splay of Rinconada Fault (Hart, 1976)
Sector Boundary

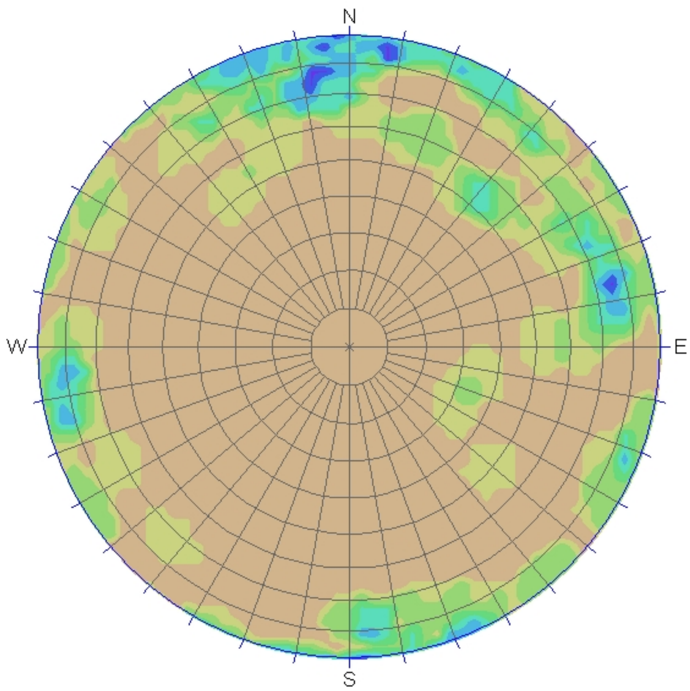
Golder Associates
RENO, NEVADA
PROJECT No. 073-97199 DATE 3/29/12_SUNNYVALE

PLANNED PIT EXTENSION
FIGURE 3





Golder Associates	TITLE **ALL STRUCTURES MAPPED**		
CLIENT/PROJECT Hanson **SANTA MARGARITA QUARRY**	DRAWN RK	DATE 3/28/08	JOB NO. 073-97199
	CHECKED GM	SCALE NA	DWG. NO. / REV. NO.
	REVIEWED GM	FILE NO.	**FIGURE 4**

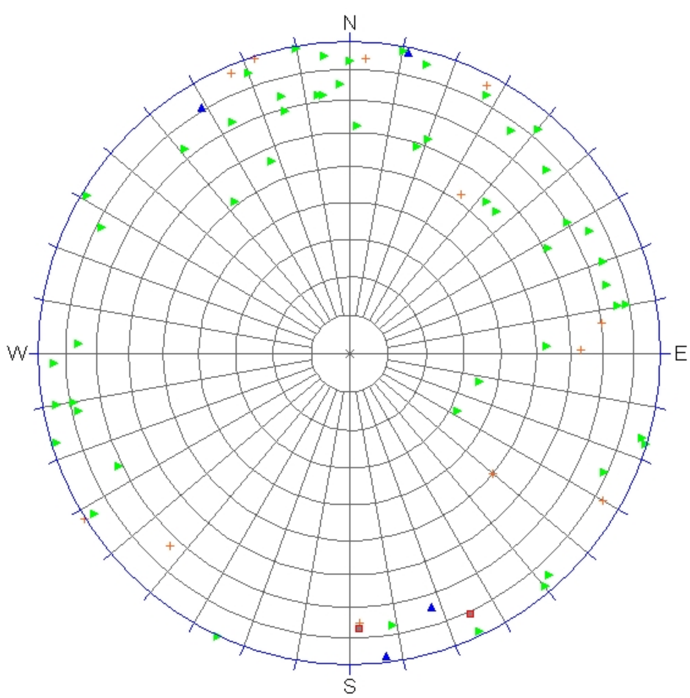


Schmidt
Concentrations
% of total per 1.0 % area

0.00 ~ 1.00 %
1.00 ~ 2.00 %
2.00 ~ 3.00 %
3.00 ~ 4.00 %
4.00 ~ 5.00 %
5.00 ~ 6.00 %
6.00 ~ 7.00 %
7.00 ~ 8.00 %
8.00 ~ 9.00 %
9.00 ~ 10.00 %

No Bias Correction
Max. Conc. = 7.1429%

Equal Area
Lower Hemisphere
70 Poles
70 Entries



TYPE

D [2]
D/F [4]
F [52]
SH [12]

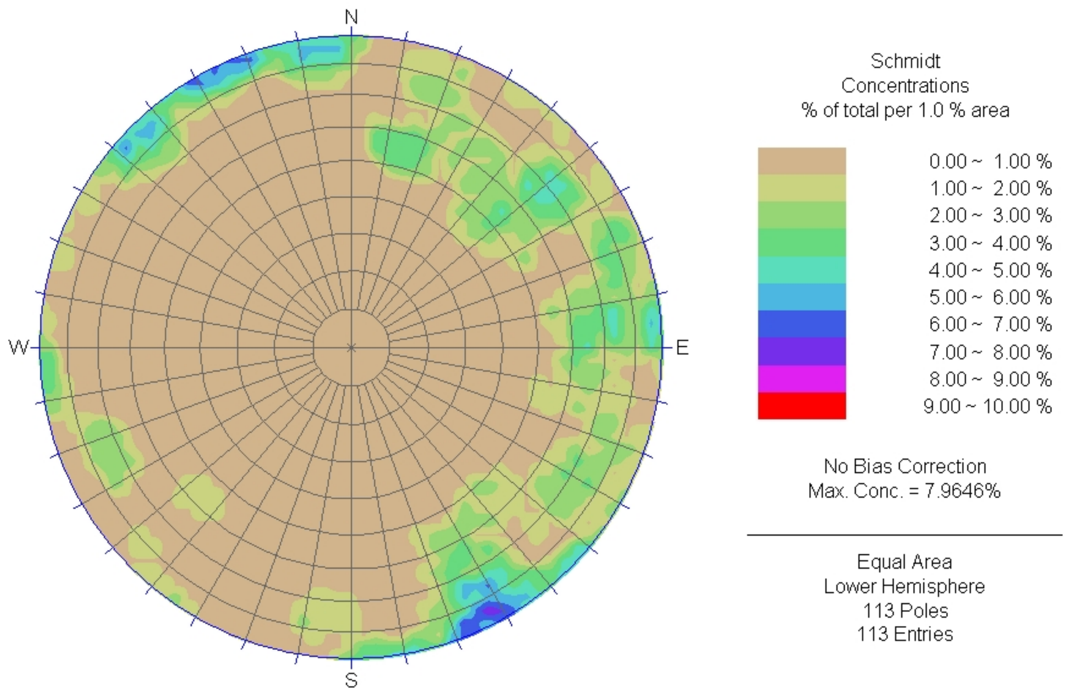
Equal Area
Lower Hemisphere
70 Poles
70 Entries

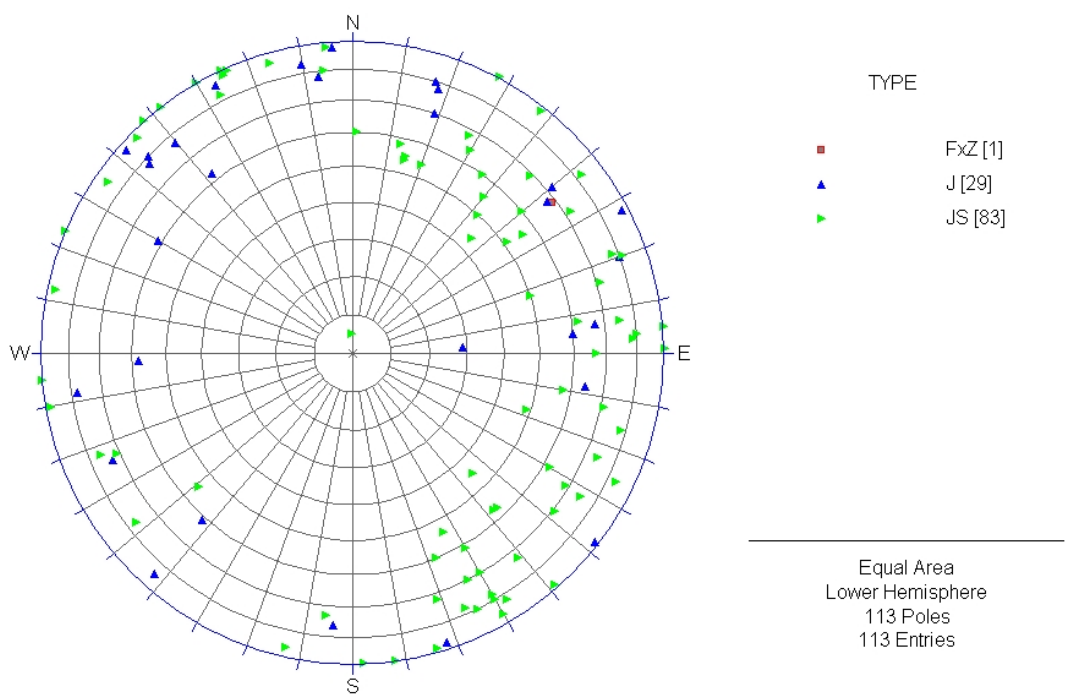
Golder Associates

TITLE
ALL FAULTS, SHEARS, AND DIKES MAPPED

CLIENT/PROJECT
Hanson
SANTA MARGARITA QUARRY

DRAWN	RK	DATE	3/28/08	JOB NO.	073-97199
CHECKED	GM	SCALE	NA	DWG. NO. / REV. NO.	
REVIEWED	GM	FILE NO.			**FIGURE 5**



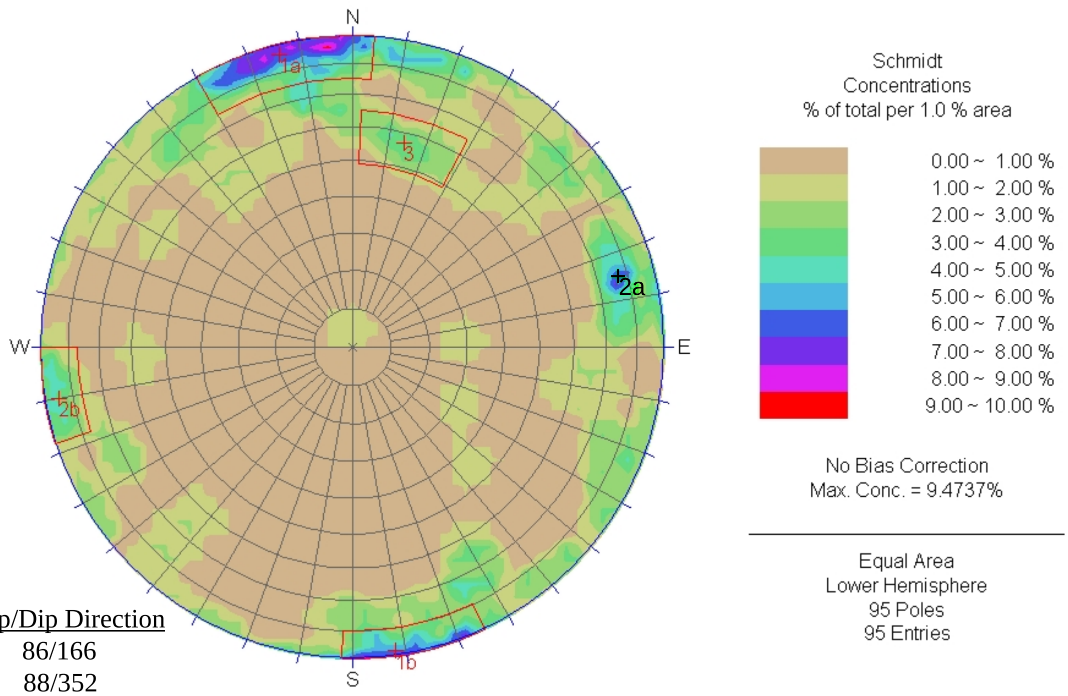


Golder Associates

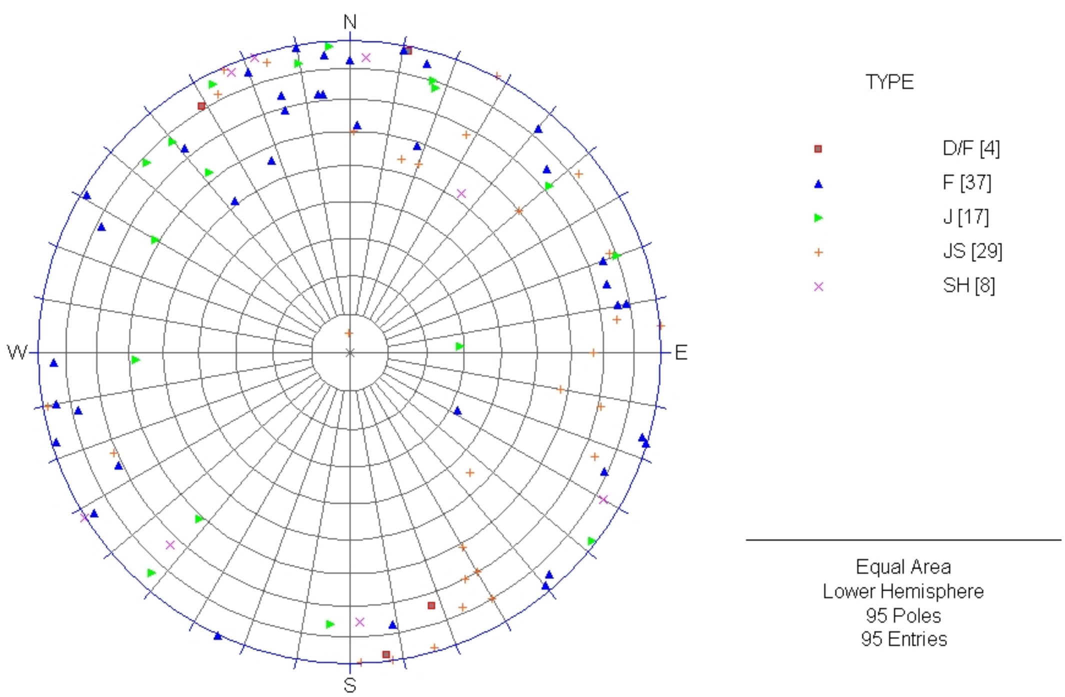
TITLE

ALL JOINTS MAPPED

CLIENT/PROJECT	DRAWN	DATE	JOB NO.
Hanson — SANTA MARGARITA QUARRY	RK	3/28/08	073-97199
	CHECKED GM	SCALE NA	DWG. NO. / REV. NO.
	REVIEWED GM	FILE NO.	**FIGURE 6**



Set	Dip/Dip Direction
1a	86/166
1b	88/352
2a	77/255
2b	85/080
3	57/194

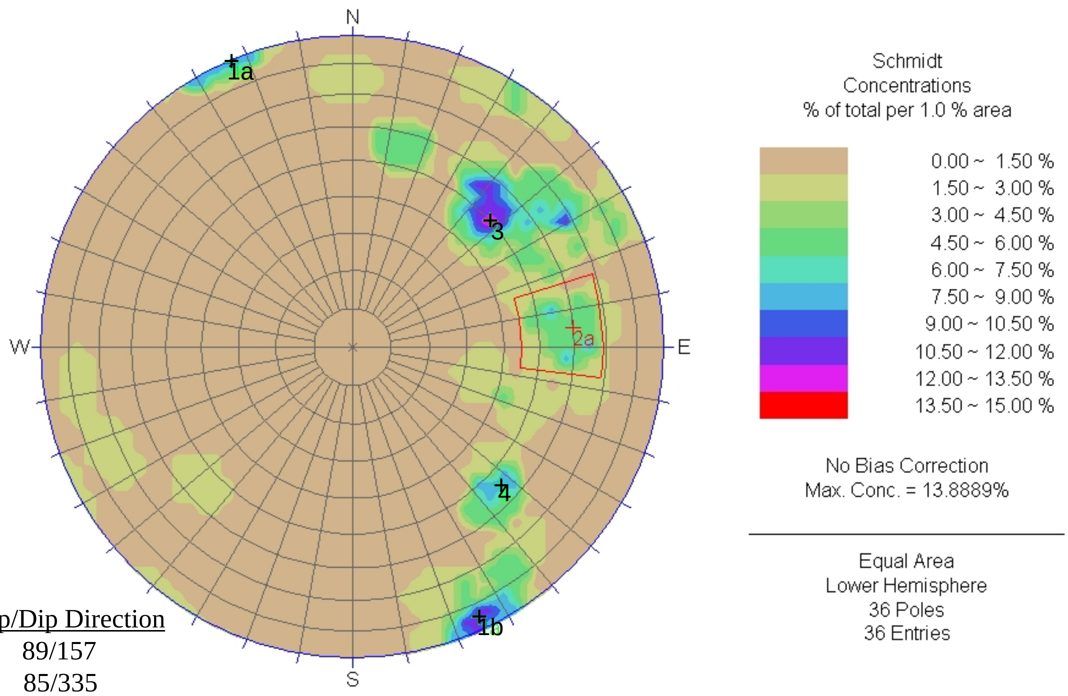


Golder Associates

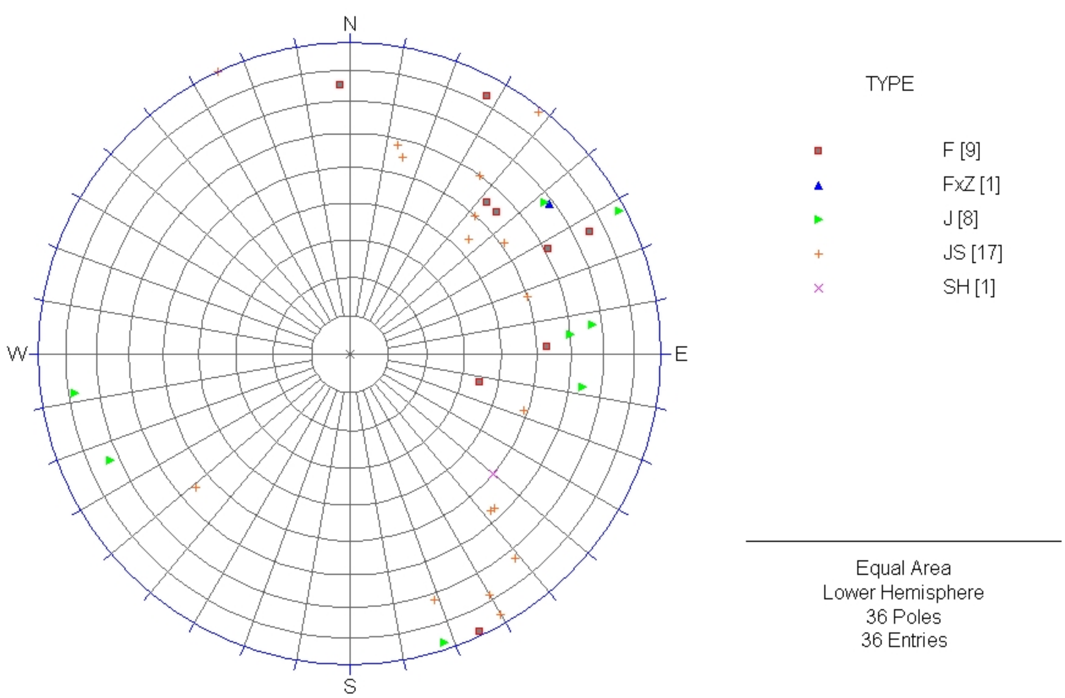
TITLE

ALL STRUCTURES MAPPED ON UPPER BENCHES

CLIENT/PROJECT	DRAWN	DATE	JOB NO.
Hanson — SANTA MARGARITA QUARRY	RK	3/28/08	073-97199
	CHECKED GM	SCALE NA	DWG. NO. / REV. NO.
	REVIEWED GM	FILE NO.	FIGURE 7



Set	Dip/Dip Direction
1a	89/157
1b	85/335
2a	60/265
3	50/227
4	54/313



Golder Associates

TITLE

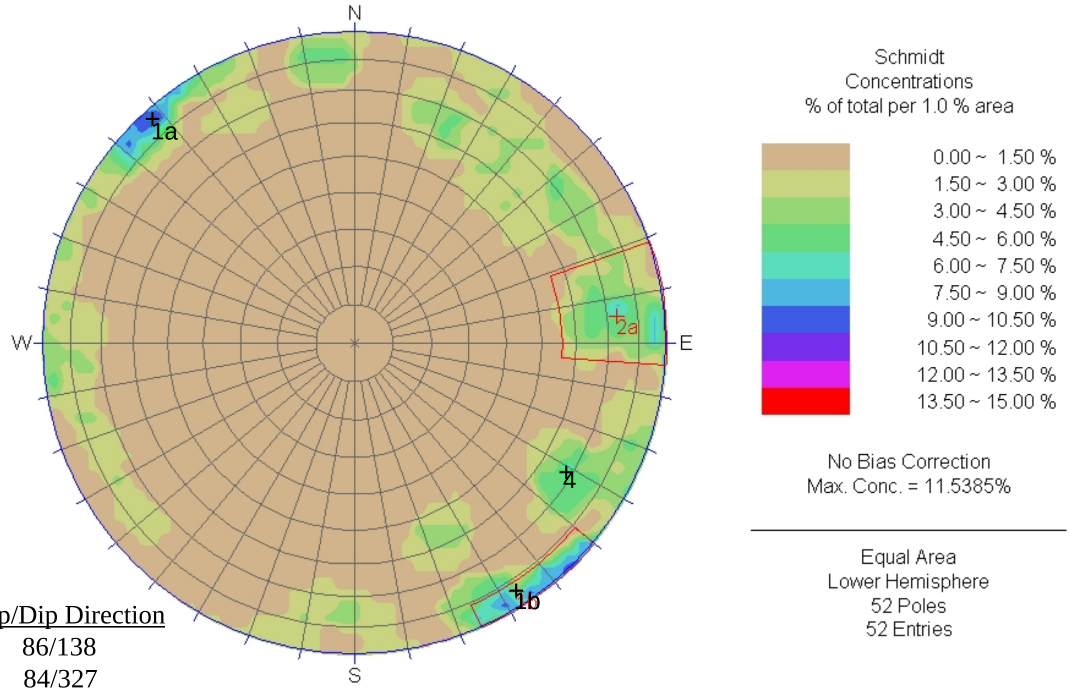
ALL STRUCTURES MAPPED IN PIT BOTTOM

CLIENT/PROJECT

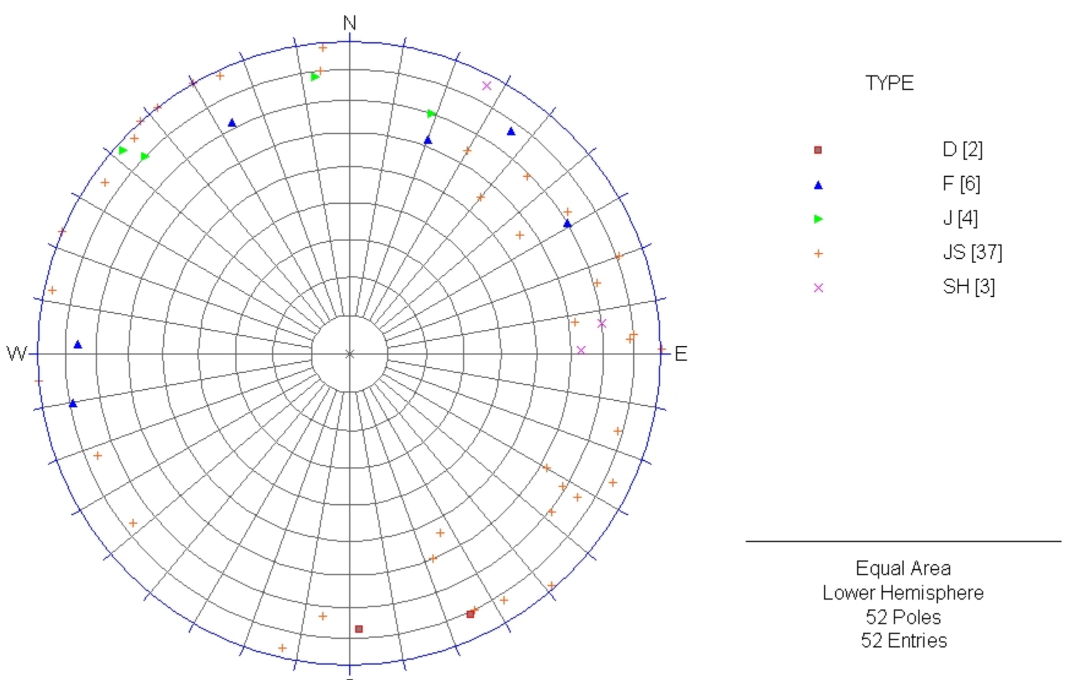
Hanson

SANTA MARGARITA QUARRY

DRAWN	RK	DATE	3/28/08	JOB NO.	073-97199
CHECKED	GM	SCALE	NA	DWG. NO. / REV. NO.	
REVIEWED	GM	FILE NO.			**FIGURE 8**



Set	Dip/Dip Direction
1a	86/138
1b	84/327
2a	73/264
4	68/301

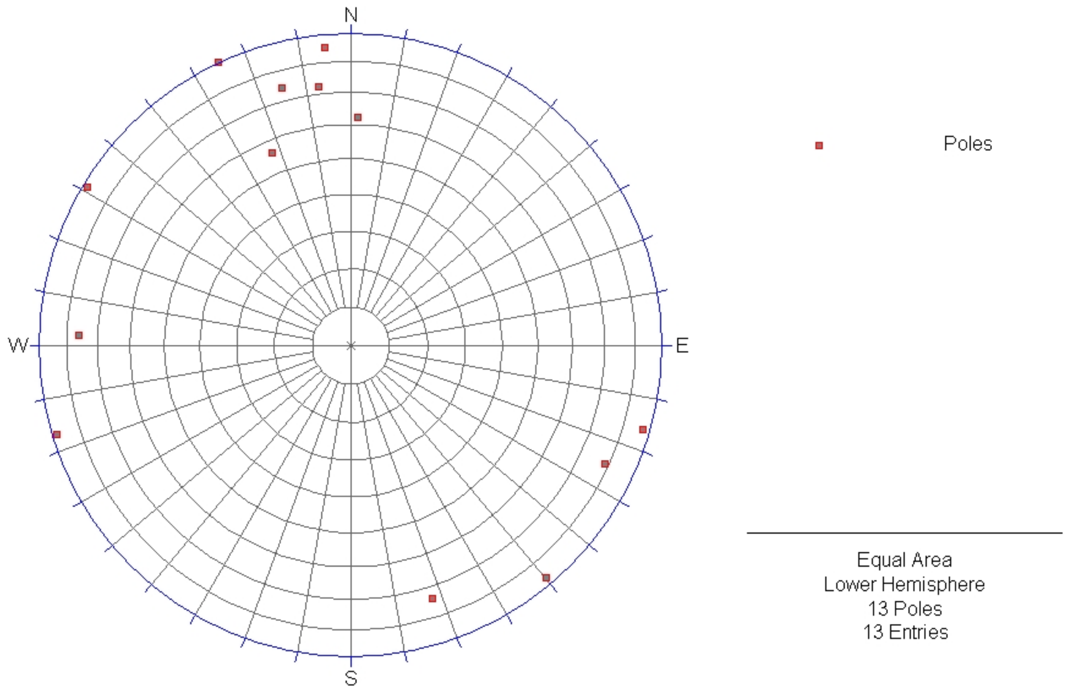


Golder Associates

TITLE

ALL STRUCTURES MAPPED IN EXPLORATION ROADCUT

CLIENT/PROJECT Hanson — SANTA MARGARITA QUARRY	DRAWN RK	DATE 3/28/08	JOB NO. 073-97199
	CHECKED GM	SCALE NA	DWG. NO. / REV. NO.
	REVIEWED GM	FILE NO.	**FIGURE 9**



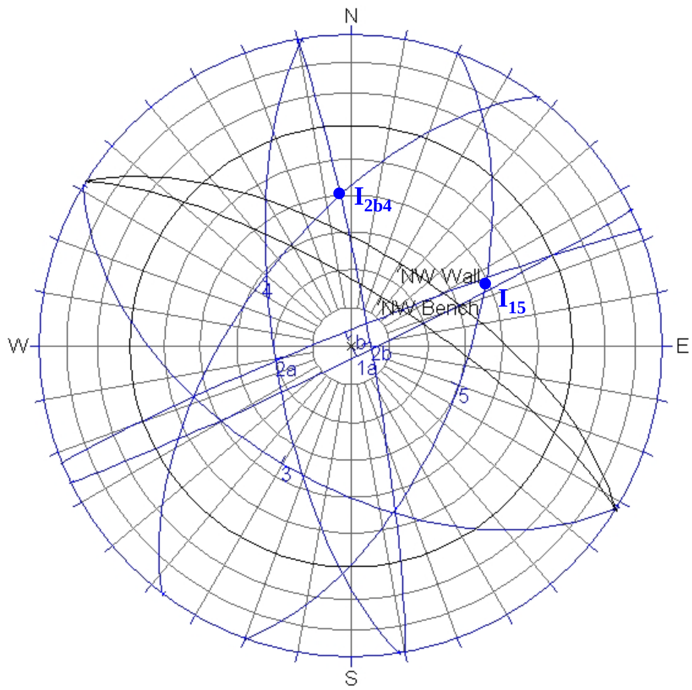
Golder Associates

TITLE

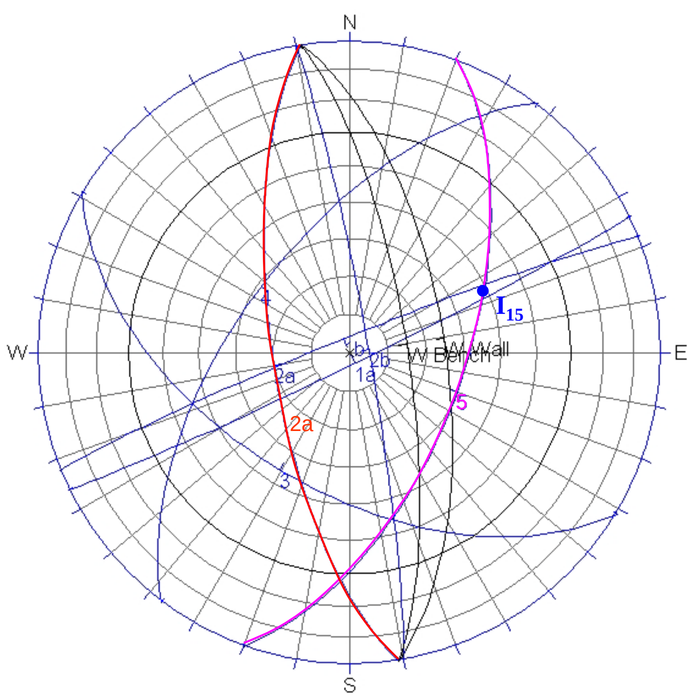
ALL FAULTS WITH
≥ 6 INCHES OF INFILL

CLIENT/PROJECT	DRAWN	RK	DATE	3/28/08	JOB NO.	073-97199
Hanson — SANTA MARGARITA QUARRY	CHECKED	GM	SCALE	NA	DWG. NO. / REV. NO.	
	REVIEWED	GM	FILE NO.			**FIGURE 10**

NORTHWEST SECTOR



WEST SECTOR



Golder Associates

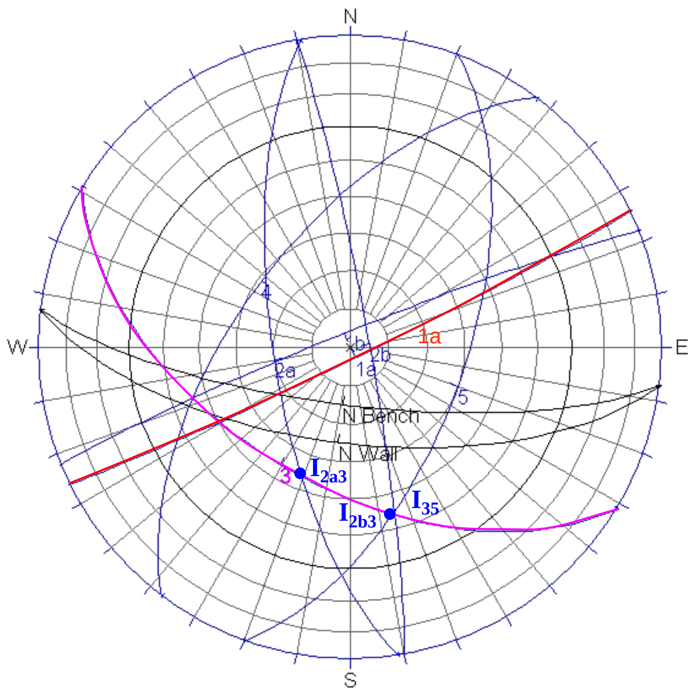
TITLE					
DESIGN STEREONETS, NORTHWEST AND WEST SECTORS					
DRAWN	RK	DATE	3/28/08	JOB NO.	073-97199
CHECKED	GM	SCALE	NA	DWG. NO. / REV. NO.	
REVIEWED	GM	FILE NO.			**FIGURE 11**

CLIENT/PROJECT

Hanson

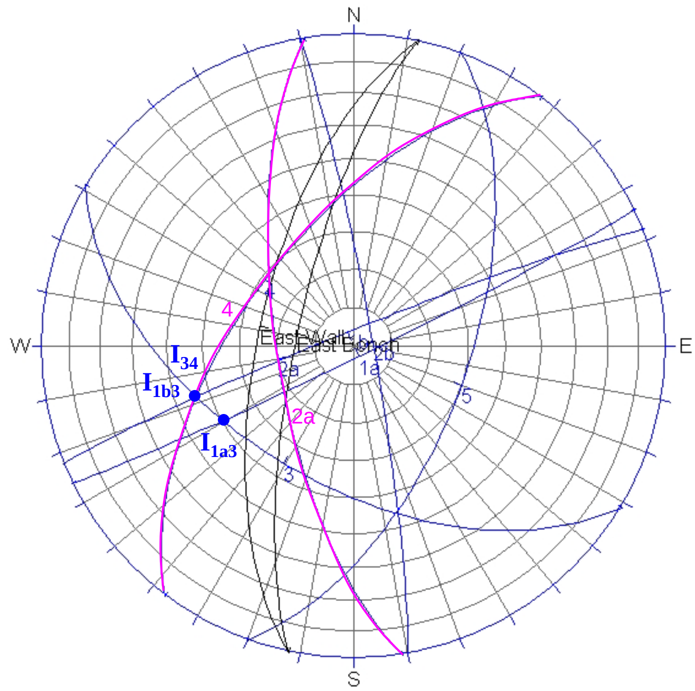
SANTA MARGARITA QUARRY

NORTH
SECTOR

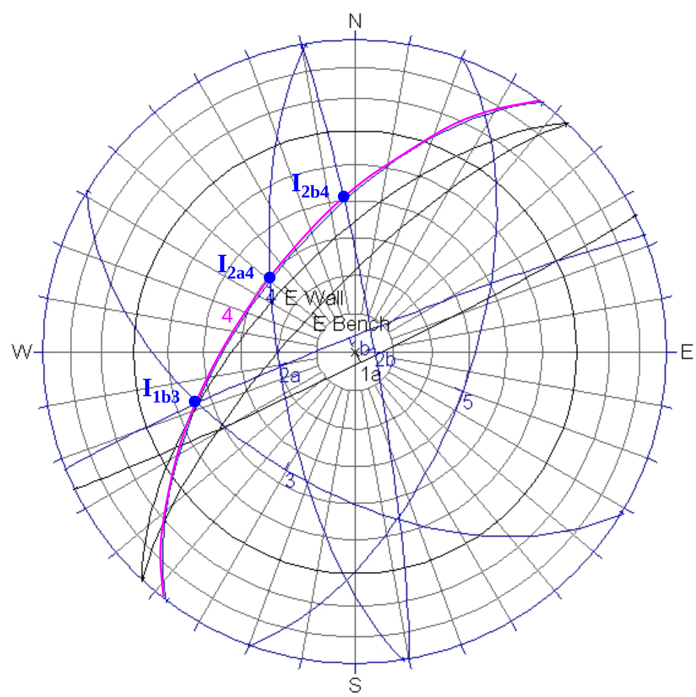


Golder Associates	TITLE DESIGN STEREONET, NORTH SECTOR				
CLIENT/PROJECT Hanson **SANTA MARGARITA QUARRY**	DRAWN RK	DATE 3/28/08	JOB NO. 073-97199		
	CHECKED GM	SCALE NA	DWG. NO. / REV. NO.		
	REVIEWED GM	FILE NO.	**FIGURE 12**		

NORTHEAST
SECTOR



EAST
SECTOR



Golder Associates

TITLE

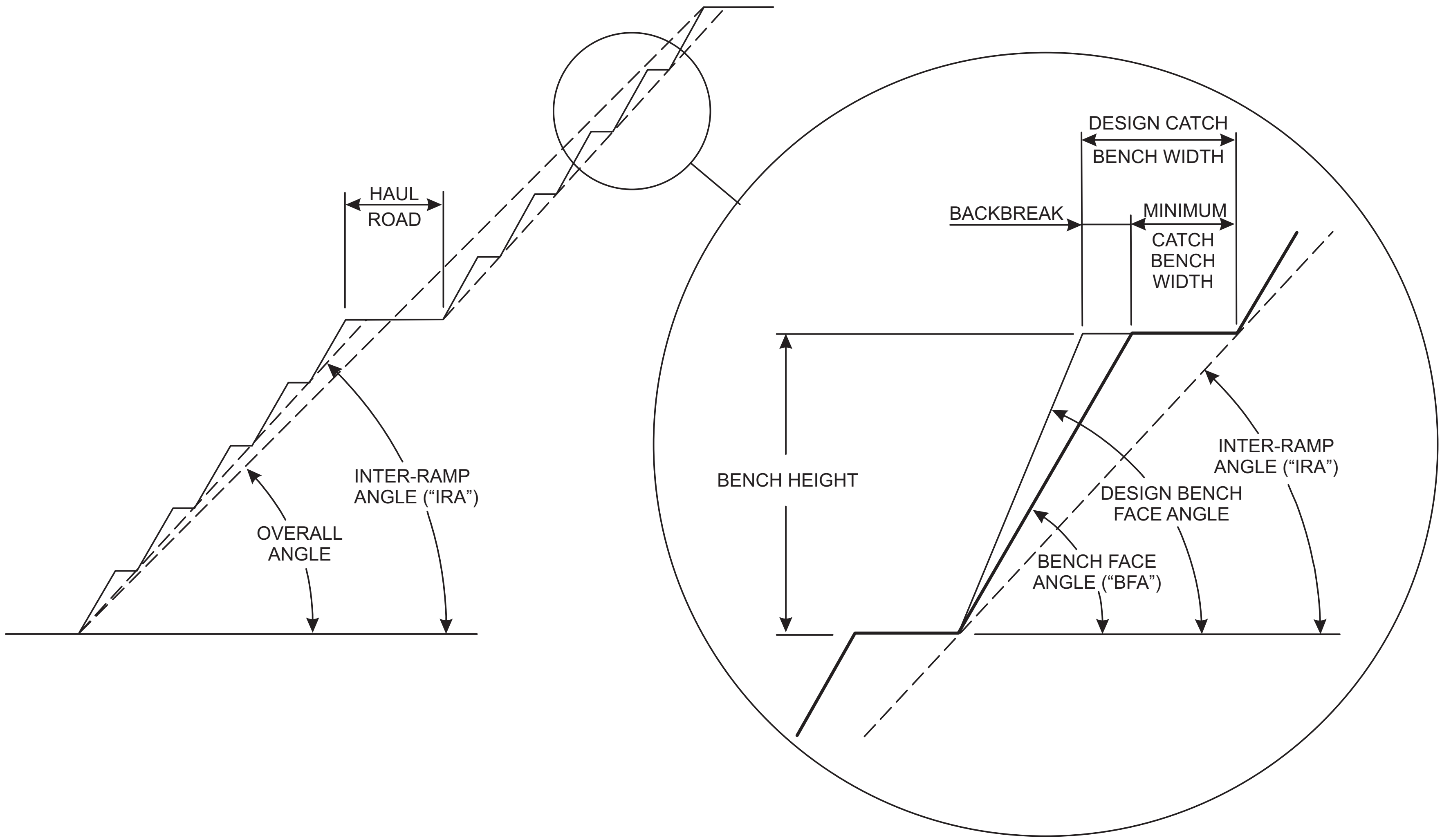
DESIGN STEREONETS, NORTHEAST AND EAST SECTORS

CLIENT/PROJECT

Hanson

SANTA MARGARITA QUARRY

DRAWN	RK	DATE	3/28/08	JOB NO.	073-97199
CHECKED	GM	SCALE	NA	DWG. NO. / REV. NO.	
REVIEWED	GM	FILE NO.			**FIGURE 13**



Golder Associates

595 DOUBLE EAGLE CT, SUITE 100
RENO, NEVADA 89521
PHONE: (775) 828-9604
FAX: (775) 828-9645

CLIENT/PROJECT

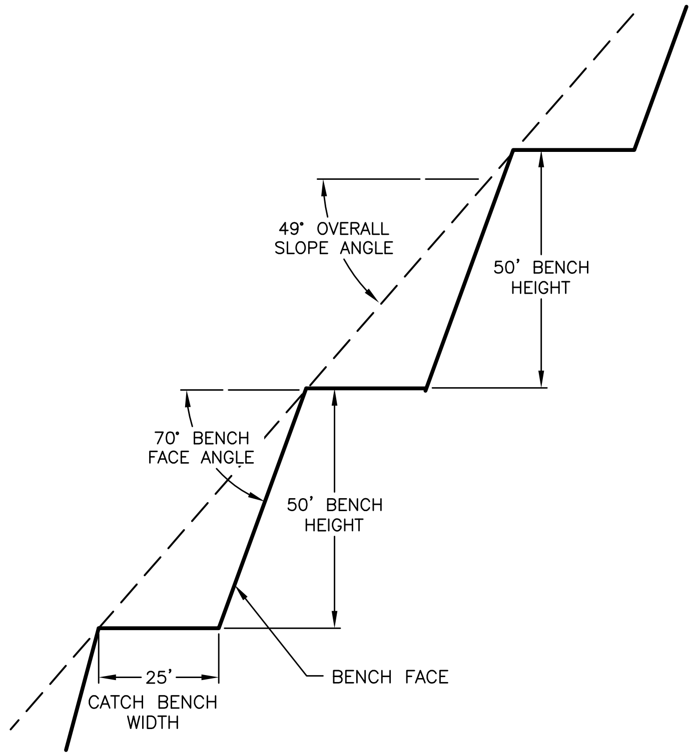
Hanson

SANTA MARGARITA QUARRY

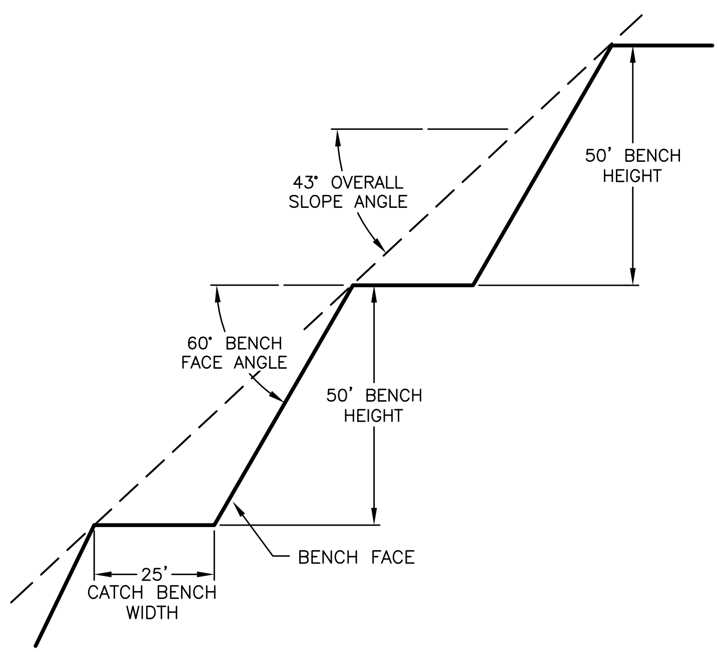
TITLE

BENCH AND INTER-RAMP SLOPE CONFIGURATION

DRAWN	RLK	DATE	5/12/08	JOB NO.	073-97199
CHECKED	GM	SCALE		DWG, NO./REV. NO.	
REVIEWED		FILE NO.			FIGURE 14



WEST AND NORTHWEST SECTORS, WEATHERED AND FRESH ROCK

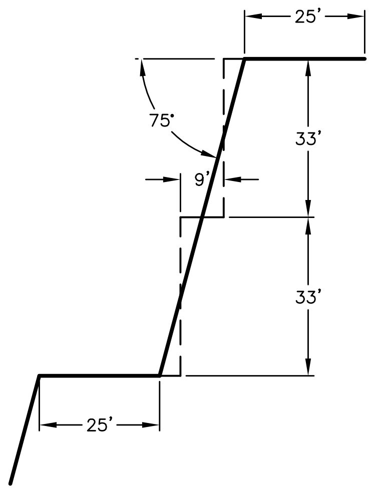


NORTH, NORTHEAST AND EAST SECTORS WEATHERED AND FRESH ROCK

Golder Associates
595 DOUBLE EAGLE CT SUITE 1000
RENO, NEVADA 89521
PHONE: (775) 828-9604
FAX: (775) 828-9645

TITLE	RECOMMENDED DESIGN SLOPE CONFIGURATIONS	
CLIENT/PROJECT: SANTA MARGARITA PROJECT	DRAWN: RJT	DATE: 02/24/12
S:\Hanson Santa Margarita\Hanson Logo.jpg	CHECKED: GM	SCALE: AS SHOWN
	REVIEWED:	FILE NO.
	JOB NO.: 073-97199	DWG NO./REV. NO.: S:\HANSON...FIGURES\FIGURE 15.DWG
	FIGURE 15	

VERTICAL PRE-SPLIT DRILLED SINGLE BENCH HEIGHT

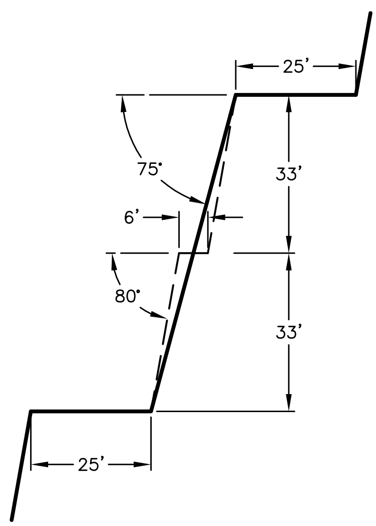


DRILL VERTICAL PRE-SPLIT ON APPROXIMATE CENTER OF DESIGN BENCH FACE WITH MINIMUM SAFE STANDOFF DISTANCE (NOMINAL 9 FT) BETWEEN LIFTS.

DESIGN BENCH

PRE-SPLIT DRILLHOLE

INCLINED PRESPLIT DRILLED SINGLE BENCH HEIGHT

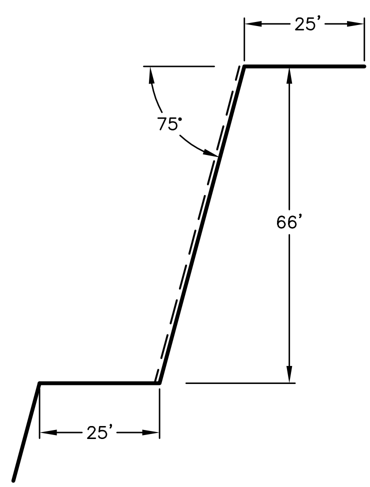


DRILL PRE-SPLIT AT 80° INCLINATION WITH MINIMUM SAFE STANDOFF DISTANCE (NOMINAL 6 FT) BETWEEN LIFTS

DESIGN BENCH

PRE-SPLIT DRILLHOLE

INCLINED PRE-SPLIT DRILLED DOUBLE BENCH HEIGHT



DRILL PRE-SPLIT AT 75° INCLINATION ALONG DESIGN BENCH FACE

DESIGN BENCH

PRE-SPLIT DRILLHOLE

Golder Associates
6165RIDGEVIEW CT., SUITE G
RENO, NEVADA 89519
PHONE: (775) 828-9604
FAX: (775) 828-9645

TITLE

TYPICAL DOUBLE BENCH PRE-SPLIT DESIGNS

CLIENT/PROJECT

S:\Hanson Santa Margarita\Hanson Logo.jpg

SANTA MARGARITA PROJECT

DRAWN RJT	DATE 05/12/08	JOB NO. 073-97199
CHECKED GM	SCALE AS SHOWN	DWG NO./REV. NO. S:\HANSON...FIGURES\FIGURE 16.DWG
REVIEWED	FILE NO.	FIGURE 16

APPENDIX A

GEOLOGIC MAPPING DATA

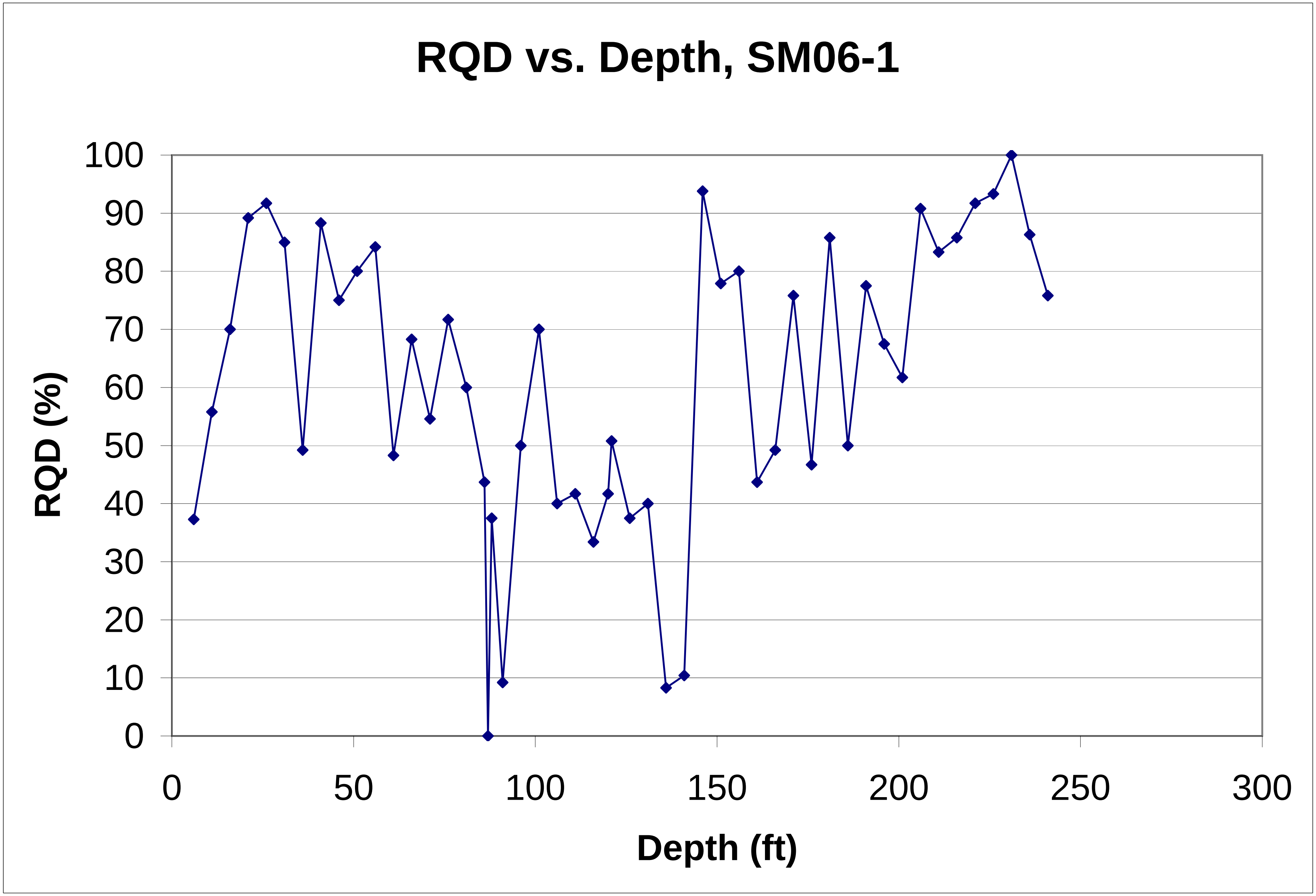
Station	Location	Type	Dip	Dip Direction	Persistence (ft)	Termination	Width (mm)	Infill	Infill Strength	Roughness	Shape	Spacing (ft)	JCR	Quantity	Comment
1a	1	J	58	88	8	1	NA	Patchy lim		R	P	0.3-2	18	3	
1a	2	J	77	140	10	0	NA	Patchy lim		R	P	1-2	18	1	
1a	2	J	78	133	8	1	NA	Patchy lim		R	P	1-2	18	1	
1a	3	J	61	42	12	0	NA	Patchy lim		R	P	na	18	1	Well-developed but apparently just 1
1a	4	J	29	267	12	0	4	Patchy lim		R	P	0.3 OR 15	16	2	Only 1 "pair" visible
1a	5	J	78	198	8	1	1	Patchy lim		R	U	1.5	16	3	
1a	5	J	80	197	6	1	1	Patchy lim		R	U	1.5	16	2	
1a	6	J	61	120	5	1	NA	Str lim		VR	P	0.5	16	1	Slope-parallel
1a	6	J	62	142	7	1	NA	Patchy lim		R	P	0.5	16	1	
1b	1	F	51	143	25	0	0.5	Patchy lim		VR	U		18	1	Small flt
1b	2	F	78	117		0	10	Bx & Go	R0	P/K	C		2	1	Shattered rock 2 inches away from flt, small flt
1b	3	F	77	351	10	0	50.8	Bx & Go	R0	P	C		0	1	Small flt merges with next
1b	4	F	87	320	25	0	152.4	Bx & Go	R0	P	C		0	1	Small flt merges with prevous
1c	1	D/F	81	149	35	0	101.6	Bx & Go	R0	P/R	P		0	1	Silt & clay with rock frags, strong lim stain; dike w/faulted contacts
1c	2	D/F	88	191	35	0	20	Cl, Silt, Go	R0/S2	P	U		0	1	Clay & silt filled fault, granite to south and leuco to north; dike w/faulted contacts
1c	3	F	88	121	35	0	609.6	Bx, Go, Cl	R0	P/R	P		0	1	Rel large flt, infill mostly fault bx - rounded rocks, but plastic clay near fault surface
1c	4	F	89	287	35	0	0.5	Patchy lim		K	U		3	1	
1c	5	F	90	25	35	0	0.5-75	Bx, Go, Cl	R0	P/K	P		2	1	Opposite direction of others
1d	1	F	83	220	35	0	0.5	Patchy lim		K	U	0.1-0.2	4	6	Set of 6 subparallel shears in 2 ft wide zone
1d	2	FZ	75	227	40	0	101.6	Bx, Go, Cl	R0/S2	K/P	U		0	1	~12 ft wide flt zone; series of faults between this and next
1d	3	F	88	73	40	0	304.8	Bx, Go, Cl	R0/S2	K/P	C		0	1	From afar; series of faults between this and previous
1d	4	F	87	286	40	0	610-915	Bx, Go, Cl,	R0/S2	K/P	P		0	1	
1d	5	F	79	260	10	0	50-200	Bx, Go, Cl	R0/S2	K/P	P		0	2	Splays of one fault up higher
1d	6	F	76	260	15	0	50-150	Bx, Go, Cl	R0/S2	K/P	C		0	1	Splays of one fault up higher
1d	7	J	88	176	4	0	NA	Lim		R	P	0.2-1	18	6	Slope-face-parallel set of joints, fairly wide
1d	8	F	85	318	8	1	0.5	Patchy lim		K	U	1	4	2	Estimated, face-parallel fault?
1d	9	F	85	80	15	0	5	Bx, Go, Cl	R0/S2	RP	U		2	1	Estimated
1d	10	F	85	195	40	0	10	Bx, Go, Cl	R0/S2	VR/K	P		0	1	Estimated
1d	11	F	32	298	80	0	5-50	Bx, Go, Cl	R0/S2	VR/K	C		0	1	Merges with flt parallel to 5 and 6
1e	12	F	74	255	35	0	100	Bx, Go, Cl	R0/S2	R	P		0	1	
1e	13	F	83	180	30	0	10-30	Sandy clay	R0/S2	R	C		0	1	
1e	14	D/F	74	342	35	0	203.2	Bx, Go, Cl	R0/S1	K	P		0	1	North margin of dike
1e	15	D/F	87	353	35	0	10	Bx, Go, Cl	R0/S1	K	C		0	1	
2a		JS	54	195	10	2	8	Patchy lim		R	P	2	14	3	
2b	2	JS?	88	344	25	0	2	Patchy lim		R	U	1-6	14	6	Possibly a fault
2c	3	JS?	82	153	25	0	10	Str lim		R	P	1-8	14	7	
2d	4	JS	5	178	25	0	NA	Patchy lim		R	P	6	18	2	Fairly regular spacing. Estimated orientation.
2d	4	JS	60	181	20	1	0.5	Patchy lim		R	P	6	18	1	
2d	5	JS	89	358	25	0	2-3	Patchy lim		R	P		14	1	Blast-loosened
2d	4	JS	54	200	10	0	10	Patchy lim		R	P	6	14	1	
2e	1	JS	45	315	20	1	3	Str. lim	S2	R	P	0.5	12	10	
2e	2	J	84	42	20	0	1	Str. lim	S2	R	P	1-5	14	5	
2e	3	J	88	308	0.8	1	NA	Str. lim	S2	VR	P	0.1-0.3	16	2	Face-parallel, crude
2e	1	JS	61	330	5	1	0.5	Str. lim	S2	R	P	0.5	14	2	
2e	2	F	71	64	20	0	10	Alt Bx & Go	R0/S2	R	P	1-5	0	1	
2e	4	F/J	75	250	20	0	10	Cl Go	S2	R	P	0.1-2	0/12	15	1 fault, rest are associated joints
2e	5	JS	80	330	5	0	NA	Str. lim	S2	R	P	6	12	1	
2e	5	JS	70	330	6	1	NA	Lim		R	P	6	14	1	
3		JS	90	155	15	0	152.4				P	1-2		6	Joint toppling in pit
3a	1	F	55	226	25	0	10	Cl Go	S3	R	U		0	1	
3a	2	F	83	208	25	0	4	Patchy lim		K	P	0.8-6	4	3	V. small flt, parallel to persistent joint set
3a	2	F	75	178	25	0	3	Patchy lim		K	U	0.8-6	2	2	
3a	3	J	73	66	25	0	1	Patchy lim		VR	U		16	1	
3a	4	J	78	82	15	0	NA	Lim		VR	U	0.2-0.9	18	4	Crudely developed slope-parallel set
3a	5	JS	88	218	25	0	1	Patchy lim		R	U	0.2-0.9	16	7	
3a	6	F	88	335	25	0	20	Cl Go	S2	R	U		0	1	At north end of wet spot
3b	1	JS	58	193	20	0		Patchy lim		R?	P	6	14	3	
3b	2	FxZ	69	233	25	0	0.5	Lim	S3	R	U	0.1-0.3	16	20	
3b	3	F	35	282	20	0	1-100	Bx, Go, Cl	R0/S2	P	U		0	1	
3b	4	Fx	64	278	20	0	5	Cl?		R	U	0.1-0.5	8	100	
3b	5	F	53	268	50	0	1-2	Bx	R0/S2	R	C		0	1	Bx = rocks in clay matrix
3c	1	JS	51	234	20	0		NA		R	U	0.5-3	18	8	
3c	2	JS	78	330	20	0	3	Cl?	NA	R	U	1-4	18	4	
3c	1	JS	60	216	20	0		Patchy lim	S2	R	U	0.5-3	18	3	Coated w/white material from surface water (gypsum?)
3c	2	JS	86	330	20	0		Patchy lim		R	U	1-4	18	3	Coated w/white material from surface water (gypsum?)
3c	3	J	87	242	20	0	NA	NA		VR	U	2	22	1	
3c	3	F	75	243	20	0	NA	NA		VR/K	U	2	6	1	
3d	1	JS	50	252	40	1		Patchy lim		R	U	0.5-8	18	0.5	2 spots on same fracture
3d	1	JS	44	226				Patchy lim					18	0.5	2 spots on same fracture

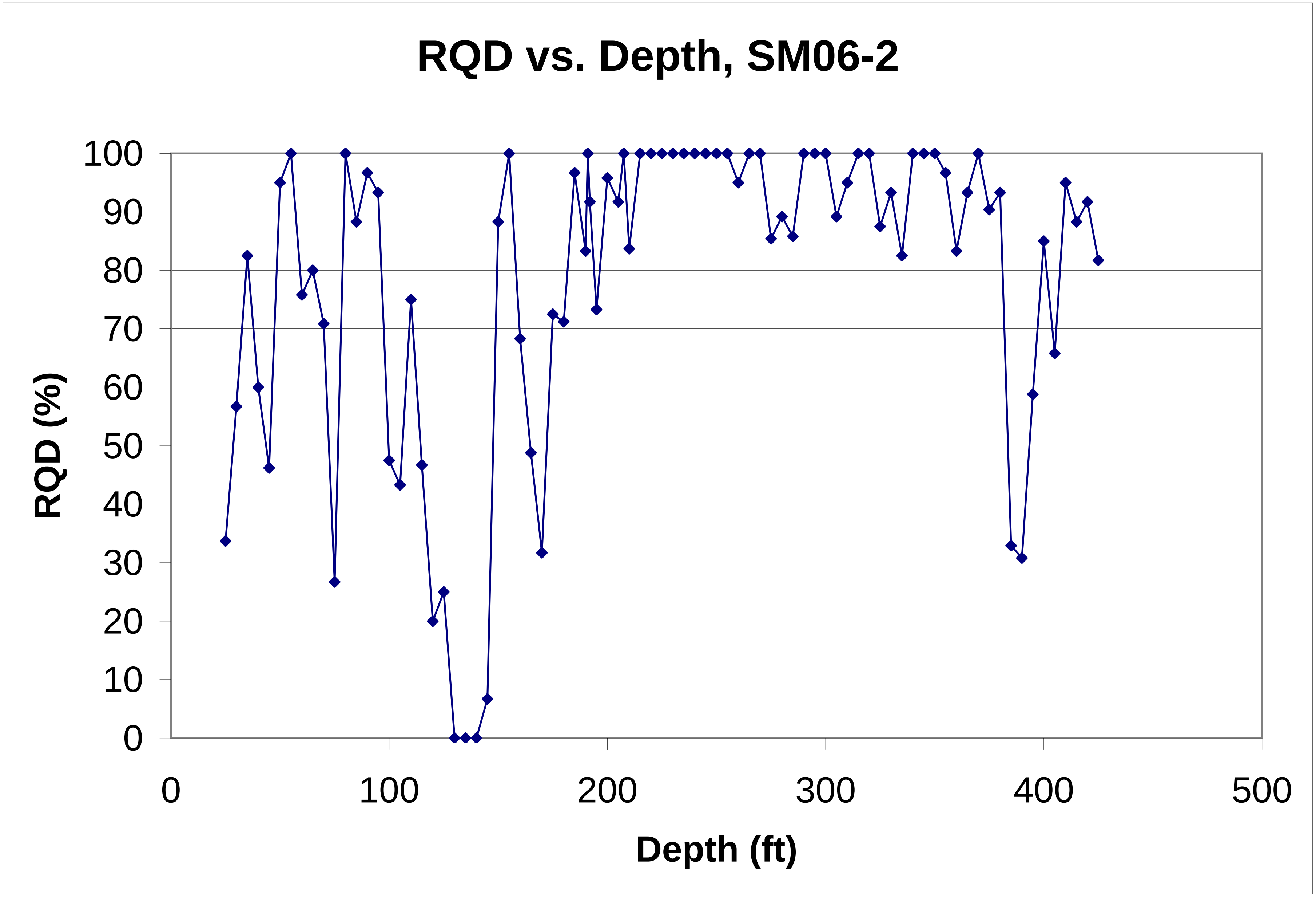
Station	Location	Type	Dip	Dip Direction	Persistence (ft)	Termination	Width (mm)	Infill	Infill Strength	Roughness	Shape	Spacing (ft)	JCR	Quantity	Comment
3d	2	JS	49	288	6	0	2	Lim		VR	U	1-10	20	1	
3d	3	J	68	232	30	0	NA	NA	NA	K/VR	C	2	8	1	
3d	4	SH	50	310			0.5	Patchy lim		R/K	C	3-5	8	1	
3d	5	JS	57	318	6	1	NA	NA	NA	R	P		20	1	
3d	6	J	67	263	5	0	NA	NA	NA	VR	C		20	1	
3d	7	JS	57	317	25	0	NA	Lim	NA	R	U		18	1	
3d	8	F	61	242	25	0	4	Lim	NA	VR/K	U		8	1	
3d	9	J	60	265	50	0	NA	Lim	NA	R	U	8-20	18	1	
3d	10	J	87	342	50	0	3	Lim		R	P	3-6	14	1	
5a	1	SH	90	58	8	0	0.5	Patchy lim		K/R	U	0.2-2	6	6	
5a	1	J	80	250	8	0	0.5	Patchy lim		R	ST	0.2-2	14	6	
5a	1	J	72	230	12	0	2	Patchy lim		R	U	0.2-2	16	8	
5a	2	JS	70	333	5	1	0.5	Patchy lim		R	U	0.5-1.5	18	4	Few but well-developed
5a	3	F	85	160	12	0	2	Lim	S4	VR	P		4	1	Maybe a fault? Not sure.
5a	4	F	56	158	12	0	457.2	Bx, Go, Cl	R0/S3	R/K	U		0	0.5	Major fault. Infill is altered rocks in bx and very soft
5a	4	F	69	165										0.5	Same fault different spot
5a	5	F	89	170	12	0	10	Bx, Go, Cl	R0/S2	VR/K	U		2	1	
5a	6	J	76	4	4	1	NA	Lim		R	U	0.2-1	18	8	Variable orientation
5a	7	JS	60	230	10	0	1	Lim		R	P	0.2-2	18	10	
5a	7	JS	68	208	5	0	5	Lim		VR	P	0.2-2	16	10	
5a	8	JS	78	336	3	0	NA	Lim		VR	P	0.8	16	5	Face-parallel set
5a	9	SH	52	215	12	0	0.5	Patchy lim		R/K	U		4	1	Parallel joints for 2 ft in footwall
5a	10	F	84	88	12	0	50-150	Bx, Go, Cl	R0/S2	K	U		0	1	
5a	11	SH	84	183	12	0	0.5	Patchy lim		K	P		4	1	
5b	1	F	72	174	20	0	125	Clay + rock	R0/S3	R/K	U	3	0	1	Infill mostly clay w/occasional rounded rock frags
5b	1	F	72	173	20	0	200	Bx, Go, Cl	R0/S2	R/K	U	3	0	1	
5b	2	F	88	190	20	0	50	Clay + rock	R0/S3	R/K	U	1	0	1	
5b	3	F	62	182	20	0	75-450	Clay + rock	R0/S2	K	U		0	1	Clay infill somewhat plastic
5b	4	JS	90	208	20	0	0.5	Patchy lim		R	U	0.1-0.2	18	4	
5b	4	JS	90	265	20	0	0.5	Patchy lim		R	C	0.1-0.2	18	4	
5b	5	SH	89	162	20	0	0.5	Patchy lim		K/R	P		4	1	
5b	6	JS	67	270	20	0	0.5	Patchy lim		S	C		16	10	Strongly developed
5b	6	JS	78	249	20	0	0.5	Str lim		S	C	0.1-0.8	14	10	Set south of 5
5b	7	F	74	165	20	0	600	Bx, Go, Cl	R0/S1	K	U		0	1	Large fault, infill ~2 inches of soft pure plastic clay, then usual clay gouge and bx
5c	1	F	73	141	20	0	2-50	Rk, Bx, Go	R0/S2	K/R	U		0	1	
5c	2	JS	82	232	15	0	0.5	Patchy lim		R	U	0.5-2	16	3	
5c	2	JS	75	263	15	0	0.5	Patchy lim		R	U	0.5-2	18	3	
5c	3	SH	83	300	7	1	5-50	Rk, Bx, Go	R0/S2	K	U		2	1	
5c	4	F	86	58	20	0	10-50	Rk, Bx, Go	R0/S2	K	U		0	1	Half planar, half wedge failure along this big fault; orientation variable
5c	5	J	86	153	6	0	NA	NA	NA	R	P		18	1	North plane of wedge
5c	6	SH	75	358	30	0	10-20	Bx, Cl, Go	R0/S2	K	U		2	1	
5c	7	F	79	295	30	0	10-240	Alt Bx, Cl, (	R0/S1	VR/K	U		0	1	
5c	8	SH	73	43	10	1	0.5	Cl	S1	K	U		3	1	Related to #4
6a	1	JS	71	67	12	0	NA	Patchy lim		R	U	1-2	22	8	
6a	2	SH	87	157	12	0	10	Patchy lim		R/K	U	1-3	18	2	
6a	1	JS	88	80	6	1	0.5	Patchy lim		R	U	1-2	22	2	
6a	2	JS	89	156	5	0	NA	Patchy lim		R	U	1-3	18	2	
6a	3	F	78	78	12	0	3	Bx & Go	R0 local R1	R/P	U		0	1	Same fault as big plane below
6a	4	F	85	175	4	0	250	Clay alt Bx	R0/S0	S	P		0	1	Cataclasic, stronger than usual, strong lim stain, near surface
6b	1	JS	71	282	15	0	0.5	Patchy lim		S	U	0.2-600	16	20	
6b	2	JS	86	164	15	0	0.5	Patchy lim		S	ST	0.2-0.3	16	5	
6b	3	JS	74	293	3	1	0.5	Lim		R	U	0.1-0.4	18	25	Anastamozing fx set cut off by #4
6b	4	J	83	170	8	0	NA	Patchy lim		S	P		18	1	
6b	5	JS	58	280	10	0	0.5	Str. Lim		S	C	0.1-0.4	14	30	
6b	6	JS	89	352	6	0	NA	Patchy lim		S	P	0.1-0.8	16	15	Prominent at north end
6b	7	F	59	198	5	0	100	Bx + Cl	R0/S2	R/K	U		0	1	
4a		JS	50	222	12	0	NA	Patchy lim		R	ST	0.2-0.5	20	4	~Slope-parallel
4a		JS	72	341	12	0	0.5	Patchy lim		S	P	0.3	18	3	~Slope-parallel
4b		F	55	222	15	0	100	Bx, Go, Cl	R0/S2	K	U		0	1	
4b		JS	73	321	10	0	0.5	Str. Lim		R	P	0.3-2	16	7	
7	1	SH	70	263	3	0	NA	Patchy lim		K	P		2	1	R2 20 ft away (west) or gate
7	2	F	72	153	15	0	3	Lim/cl	S2	S	P	1-3	2	6	Rock strength R0
7	3	JS	76	68	15	0	1	Patchy lim		S	P	2-4	14	1	Rock strength R0
7	2	JS	88	155	15	0	0.5	Str. Lim		S	P	1-3	14	1	Rock strength R0
7	2	JS	90	142	12	0	0.5	Str. Lim		R	P	1-3	12	1	Rock strength R0
7	2	JS	85	125	3	1	0.5	Str. Lim		R	C	1-3	14	1	
7	2	JS	80	334	2	2	0.5	Str. Lim		R	P	1-3	12	1	
7	2	JS	90	138	0.5	0	NA	Lim		R	C	0.3-2	18	1	
7	1	F/JS	70	239	2	0	50	Go, bx?, cl	R0/S1	S	P	0.3-1	0	5	Rest are joints, weathered - hard to tell infill

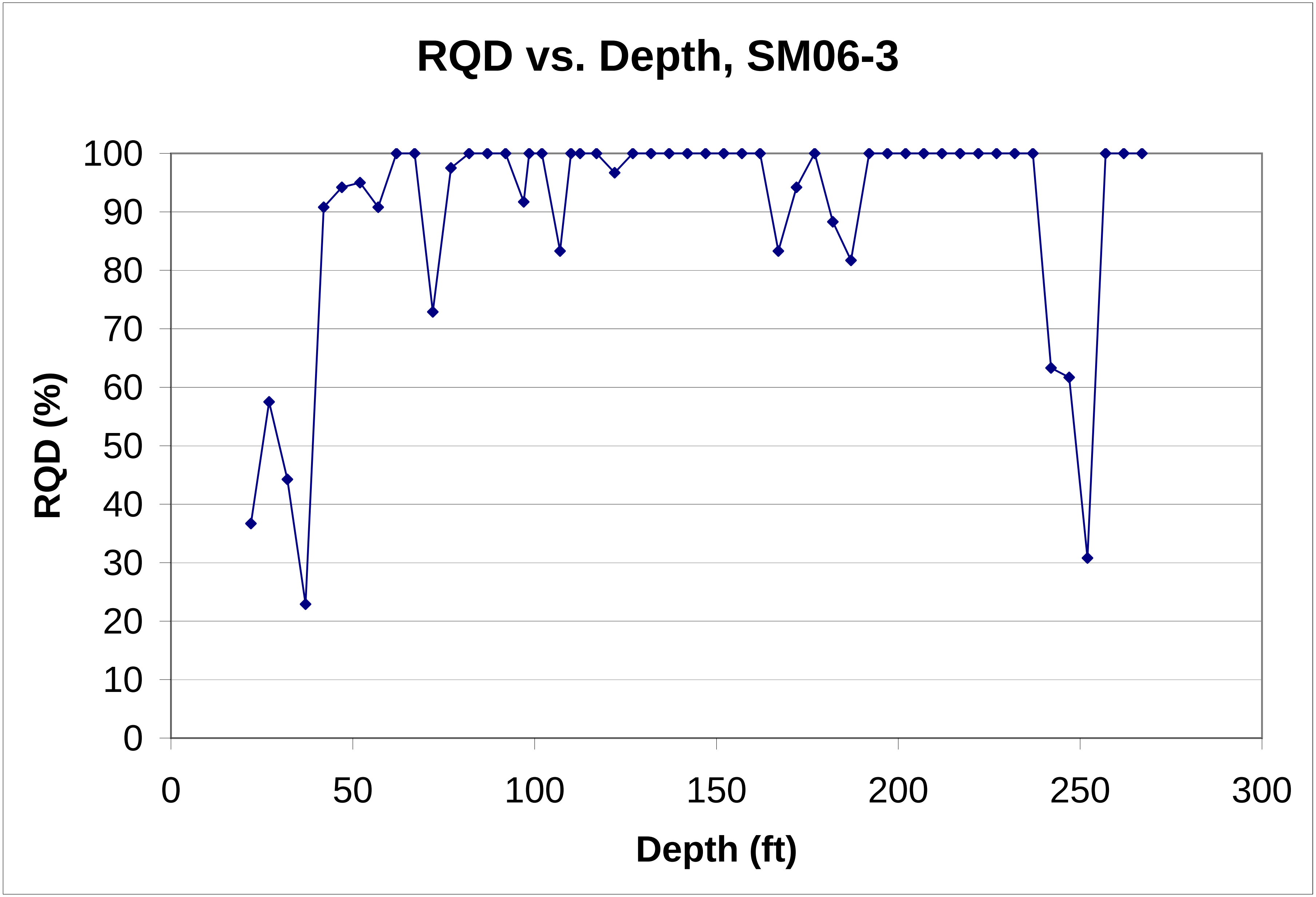
Station	Location	Type	Dip	Dip Direction	Persistence (ft)	Termination	Width (mm)	Infill	Infill Strength	Roughness	Shape	Spacing (ft)	JCR	Quantity	Comment
7	4	JS	87	102	3	0		Patchy lim		R	ST	0.2-1	14	7	
7	5	JS	56	235	2	1	0.5	Patchy lim		R	ST	0.8-1.5	14	8	
7	2	JS	90	150	10	0		Patchy lim		R	P	0.3-1.6	14	12	
7	2	JS	88	319	3	0	NA	Str. Lim		R	P	0.3	14	3	Crude, slope-parallel
7	6	JS	69	302	8	0	1	Str. Lim		R	U	0.1-0.3	14	13	Very dominant set here
7	7	J	70	199	8	0	1	Str. Lim		VR	P		12	1	
7	6	JS	62	300	8	0	0.5	Str. Lim		R	U	0.1-0.3	12	1	
7	7	SH/J	86	207	8	0	2	Patchy lim		VR/K	U	0.1-1	5	8	Rest are joints
7	8	D	77	358	10	0	NA	Patchy lim		VR	C	0.2-3	18	1	Pegmatite dike
7	8	J	78	173	10	0	NA	Patchy lim		R	C	0.2-3	16	7	
7	2	J	87	132	3	1		NA		VR	U	0.2-2	18	4	
7	2	J	80	134	8	0	2	NA		VR	C	2	18	2	
7	9	JS	80	174	7	0		NA		R	P	0.2-3	12	5	Strong set locally
7	9	JS	88	175	7	0	NA	NA		S	P	0.2-3	18	5	
7	10	JS	78	286	4	1		NA		VR	U	0.9	18	5	Poorly developed but definitely present
7	2	JS	87	135	6	0	0.5	NA		S	C	0.2-2	16	3	
7	11	JS	90	113	5	0	0.5	NA		VR	U	0.2-1.5	12	9	
7	10	JS	83	296	7	0	0.5	Str. Lim		VR	U	0.1-1.5	14	7	
7	10	JS	90	85	7	0	0.5	Str. Lim		R	P	0.1-1.5	14	6	
7	12	FZ	77	216	6	0	1-2	NA		S	C	0.1-2	12	20	Fracture zone related to fault ??
7	1	JS	69	225	3	0	0.5	Clean		S	P	0.1-1	18	3	No stain
7	10	JS	90	269	6	0	0.5	NA		S	P	0.1-2	18	5	Local set only
7	2	JS	82	328	2	0	0.5	NA		R	P	0.1-0.5	18	8	
7	13	JS	60	338	3	1	0.5	Str. Lim		R	U	0.2-0.3	16	15	
7	10	JS	79	267	7	0	1	NA		R	U	0.1-0.9	16	8	
7	3	JS	77	52	10	0	1	NA		R	C	0.5	16	6	
7	1	JS	72	237	8	0	1	NA		S	U	0.1-0.8	14	4	
7	1	JS	62	262	8	0	1	NA		S	U	0.2	16	50	Very dominant set!
7	1	JS	71	254	10	0	0.5	Patchy lim		S	P	0.1-8	16	50	
7	1	JS	80	266	10	0	0.5	Patchy lim		S	P	0.1-8	16	50	
7	2	D	81	335	12	0	0.5	Lim		S	ST	0.1-1	18	9	Some parallel joints also
7	14	F	76	92	10	0	600	Go, Bx, Cl	R0/S0	K	U		0	1	
7	1	SH	63	269	10	0	1	Patchy lim		K	U	0.2	4	50	Stopped by #14
7	14	F	79	80	12	0	5-50	Cl, Go, Bx	R0/S0	K	P	0.2-0.3	0	8	Many parallel joints
7	15	JS	75	302	6	1	0.5	Patchy lim		VR	U	0.1	16	50	
7	1	JS	81	250	10	0	1	Patchy lim		R	U	0.1-0.8	14	50	
7	16	JS	73	6	6	1	0.5	Patchy lim		VR	U	0.2-0.3	18	25	Some parallel pegmatite veins as well
7	17	F	62	200	8	0	100	Cl	S4	R	P		0	1	
7	15	JS	71	308	30	0	1	Patchy lim		S	U	0.3-1	14	10	
7	18	JS	55	220	12	0	2	Patchy lim		S	P	0.3-2	18	9	
7	18	JS	64	210	8	0	0.5	Patchy lim		S	U	0.2-0.3	16	20	
7	16	JS	86	13	6	0	NA	Patchy lim		S	ST/U	0.2-1	16	6	at gate
7	19	JS	54	333	12	0	1	Patchy lim		S	U	0.2-0.3	14	15	Strong set
3		JS	55	49										1	Wedge in west wall?
3		JS	55	195										1	Wedge in west wall?

APPENDIX B

REPRESENTATIVE ROCK CORE DATA


RQD vs. Depth, SM06-1
RQD (%)
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
0
50
100
150
200
250
300

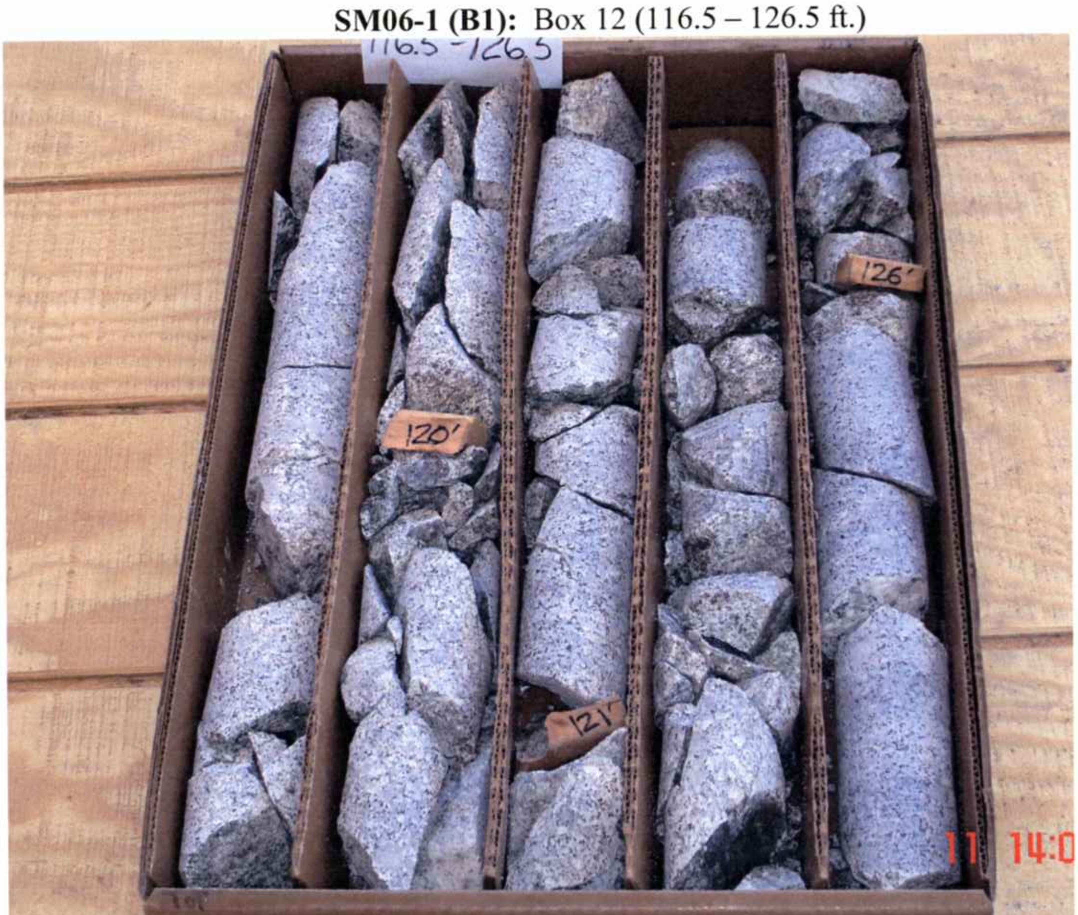

RQD vs. Depth, SM06-2
RQD (%)
100
90
80
70
60
50
40
30
20
10
0
0
100
200
300
400
500
Depth (ft)



RQD vs. Depth, SM06-3
RQD (%)
100
90
80
70
60
50
40
30
20
10
0
0
50
100
150
200
250
300
Depth (ft)

SM06-1 (B1): Box 11 (106.5 – 116.5 ft.)

SM06-1 (B1): Box 12 (116.5 – 126.5 ft.)



SM06-2 (B2): Box 10 (108.7 – 117.5 ft.)

SM06-2 (B2): Box 11 (117.5 – 127.5 ft.)

SM06-2 (B2): Box 22 (226.0 – 235.0 ft.)

SM06-2 (B2): Box 23 (235.0 – 244.2 ft.)

SM06-3 (B3): Box 9 (96.5 – 105.4 ft.)

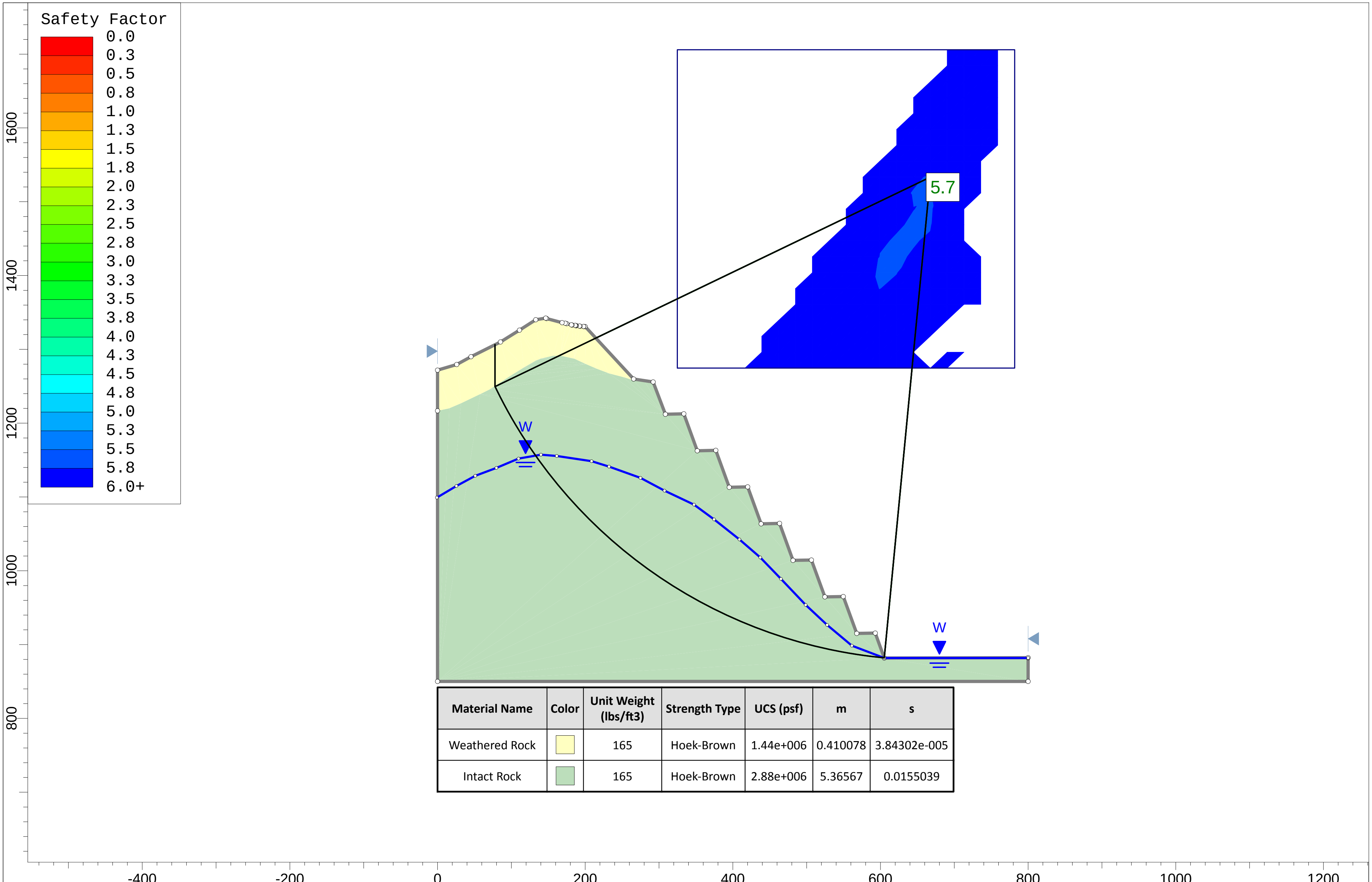
SM06-3 (B3): Box 10 (105.4 – 114.4 ft.)

APPENDIX C
SLIDE Stability Output

N:\Projects_2007\073-97199 (SM Quarry)\07397199-WORKSHEET-EXC VOL.dwg, 03/22/12 02:47:49pm, JRaub

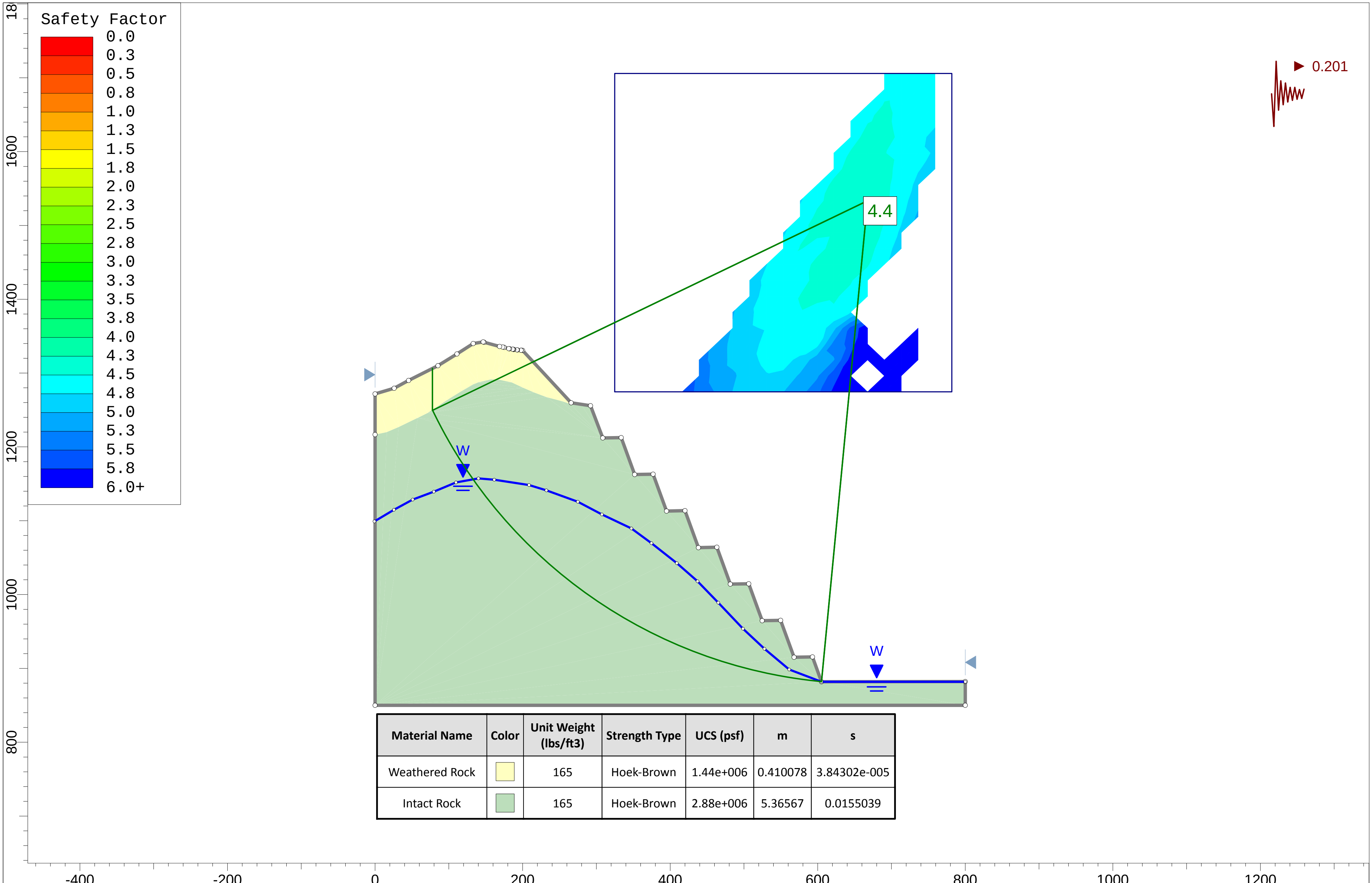
Golder Associates
Roseville, California

FIGURE 1
LIMIT EQUILIBRIUM STABILITY ANALYSIS
SANTA MARGARITA QUARRY EXPANSION
LEHIGH HANSON



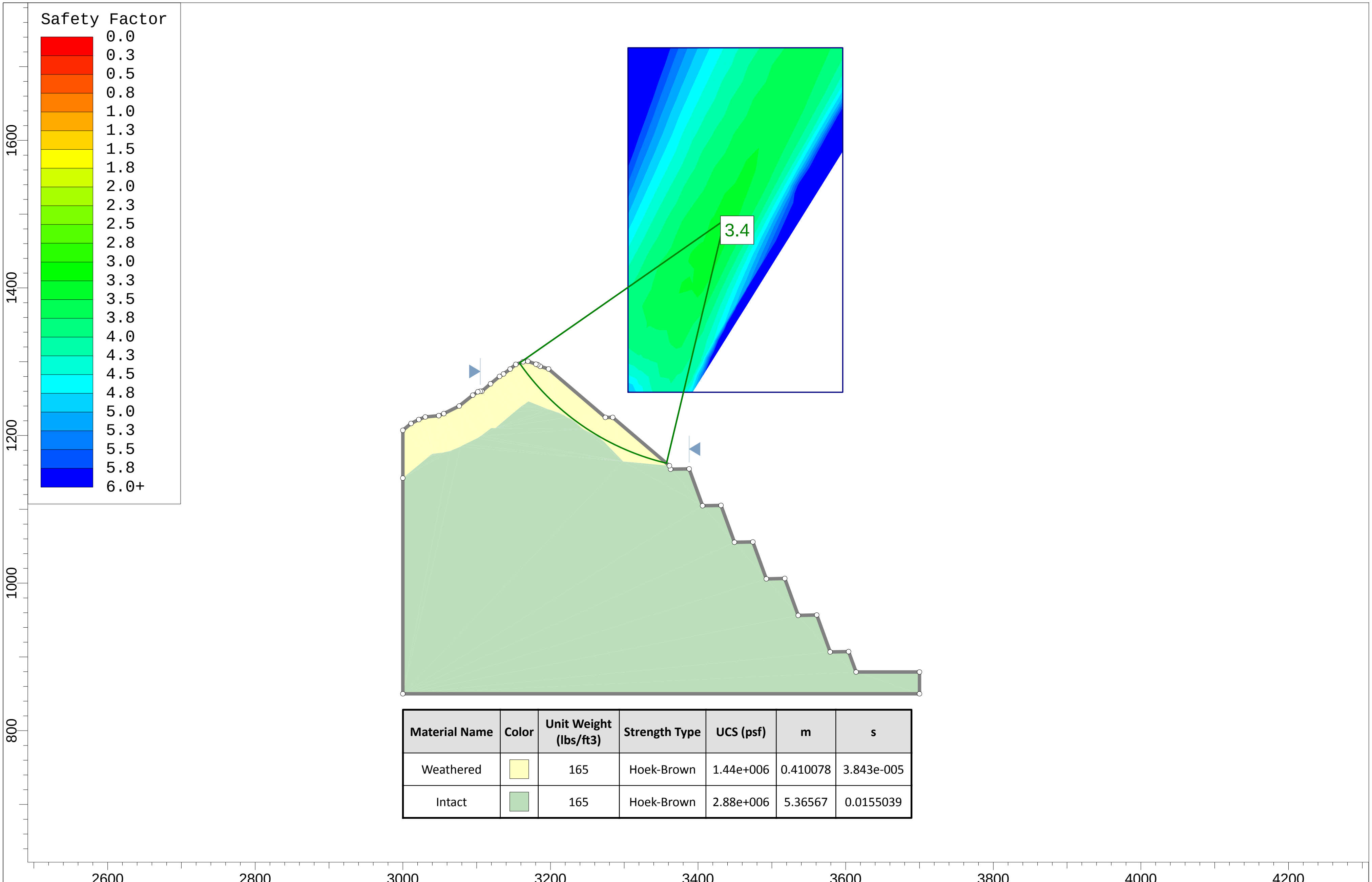
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (psf)	m	s
Weathered Rock		165	Hoek-Brown	1.44e+006	0.410078	3.84302e-005
Intact Rock		165	Hoek-Brown	2.88e+006	5.36567	0.0155039

Section A-A
Static Analysis



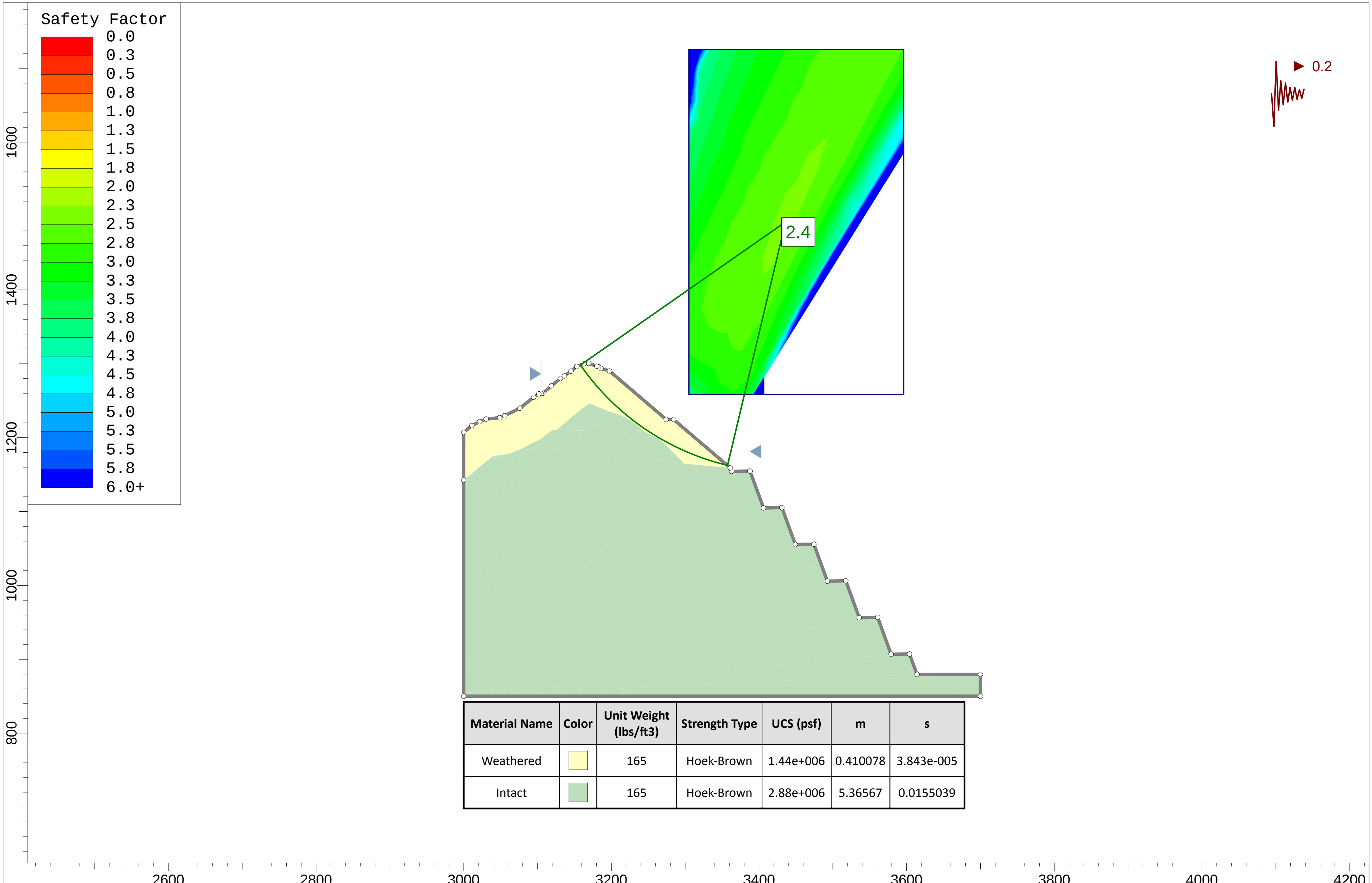
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (psf)	m	s
Weathered Rock		165	Hoek-Brown	1.44e+006	0.410078	3.84302e-005
Intact Rock		165	Hoek-Brown	2.88e+006	5.36567	0.0155039

Section A-A
Pseudo-Static Analysis



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (psf)	m	s
Weathered		165	Hoek-Brown	1.44e+006	0.410078	3.843e-005
Intact		165	Hoek-Brown	2.88e+006	5.36567	0.0155039

Section D-D, Upper Slope
Static Analysis



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (psf)	m	s
Weathered		165	Hoek-Brown	1.44e+006	0.410078	3.843e-005
Intact		165	Hoek-Brown	2.88e+006	5.36567	0.0155039

Section D-D, Upper Slope
Pseudo-Static Analysis

APPENDIX D
Operational Considerations

OPERATING CONSIDERATIONS FOR SANTA MARGARITA QUARRY

1.0 STANDARD OF CARE IN QUARRY SLOPE DEVELOPMENT

Rock slope design for open pit mines and quarries includes consideration of both mining economics (the steepness and overall stability of the slopes) and operating safety (particularly mitigation of rockfall hazards). Design factors related to safety must be of paramount importance, whether for permanent or temporary slopes, and slope designs must be implemented to meet the current standard of care in the mining industry for operating safely below rock slopes. This standard includes incorporating effective catch benches into pit slopes.

The minimum standard of care for safety in development of mine slopes is defined by Federal regulations that are enforced by Mine Safety and Health Administration (MSHA), or by equivalent State agencies using State regulations that can be no less stringent than Federal regulations. In addition, operating practices and slope designs to enhance operator safety are often developed at the corporate level, and these may be supplemented at the Operating level based on site conditions at individual pits.

Mine slope stability requirements are regulated by Title 30 of the Code of Federal Regulations, Section 56.3130. This Section requires that mining methods shall maintain slope stability in places where persons work or travel in performing their assigned tasks, and that bench configurations be based on the type of equipment used for scaling.

MSHA provides interpretation guidelines for ground control. These indicate that MSHA requires that a bench adequate to retain rockfall must be maintained above work or travel areas. Where there is not an effective catch bench above a work or travel area, other measures must be taken to protect the miners, such as berming off or ceasing mining in the affected area.

Mine slopes in rock are generally designed with catch benches for the sole purpose of providing safe working conditions; catch benches do not generally enhance the overall stability of the slope. Ineffective catch benches may arise due to geotechnical conditions that are different from those assumed for the design, or due to operating practices that do not achieve the design bench configurations. In either case, it is important to recognize the failure to meet design criteria early so that suitable modifications to the slope design and/or to the operating procedures can be implemented to maintain an adequate level of safety and protection against rockfall during continued development of the slope.

When operating procedures do not result in adequate catch benches, then design modifications to the slope may be required. Most commonly, this will incorporate a step-out to provide suitable catchment for

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any rockfall hazard above that level, and a flatter slope design with wider catch benches below the level at which the design changes are implemented. An alternative where there are no property or geological constraints at the slope crest is to lay back the slope to a flatter angle with wider catch benches from the top of the slope, which may result in increased stripping. Mechanical stabilization of benches or restraint of rockfall by fences or berms can be warranted locally in some circumstances, but are rarely feasible alternatives on a large scale.

Because catch benches are designed to provide safe working conditions below the slope, it is equally important to develop effective catch benches for both Ultimate and interim or Phase slopes, since the safety risks are identical for both cases. Where the Ultimate slope is designed assuming expensive perimeter blasting techniques, it may be warranted to avoid perimeter blasting costs for Phase slopes by accepting flatter interim slopes with alternative design bench configurations.

2.0 BENCHING PRACTICES

All slopes will be single benched, which reduces the operating risks associated with double benching operations. However, 50 ft benches are high, and warrant an appropriate level of care in drilling, blasting, and scaling operations to control the potential risks of rockfall hazards from such high benches. Operating safety is generally enhanced by implementing the following practices:

- Controlled blasting plan to reduce damage due to blasting, particularly in the upper bench;
- Blast optimization program to determine the optimum blasting procedures for site conditions;
- Thorough bench face scaling to reduce risks of rockfall using equipment that can safely reach the top of the bench to scale loose rock;
- Inspection and monitoring program to ensure that conditions are safe for initiating drilling and loading of blast holes below existing slopes;
- Geological documentation and geotechnical evaluation program to ensure that the conditions assumed for the slope and bench design are met in the field;
- Operator awareness training to train operators in safe practices, and to educate operators regarding potential rockfall hazards.

Mining a single bench configuration provides flexibility in enabling operations to be restricted in the area of bench toes, but it does not eliminate all need for operations, access, and mapping in areas that can be subject to significant rockfall hazards. Developing stable bench faces and controlling rockfall hazards with effective catch benches is therefore important even for single bench operations.

3.0 CONTROLLED BLASTING

3.1 Blasting

Where geological structure does not control slope and bench stability, the achievable inter-ramp slope angle depends on the stable bench face angles that can be developed. In the absence of structural control, achievable bench face angles are largely a function of drilling, blasting, and operating practices. Poor blasting that damages and disturbs the rock behind the design bench face, and poor scaling or excavation of the bench faces, will result in flatter bench face angles and flatter inter-ramp slopes. Blasting and operating practices that develop steep, stable bench faces enable the safe development of steeper inter-ramp slopes. The costs of the improved drilling and blasting practices result in benefits of improved operator safety and reduced stripping from more stable benches and steeper slope angles.

A careful wall control blasting program will be essential to develop steep, stable bench faces and to maximize inter-ramp slope angles at Santa Margarita. In massive, unfractured to lightly fractured rock, pre-split blasting using standard size production drillholes or slightly reduced drillhole diameters on reduced spacing is generally effective. However, as the intensity of fracturing increases, it is generally necessary to reduce the diameter and spacing of pre-split holes. In more fractured rock, cushion blasting is generally more cost effective, but cushion blasting rarely achieves the steep bench face angles assumed for the steeper slope designs in fresh rock at Santa Margarita.

The fresh granite at the Santa Margarita quarry is generally characterized by a fair to good quality rock mass, with average RQDs in the exploration drillholes ranging from 63% to 90%. Pre-split blasting is generally effective in good quality rock. Buffer blasting is generally more effective in poor quality rock, such as where the RQD is less than about 50%, and also generally produces satisfactory results where flatter bench face angles are controlled by geological structure.

3.2 Controlled Blasting Recommendations

3.2.1 Pre-Split Blast Design

Pre-split blasting should be used to maximize stable bench face angles and overall slope angles in the West and Northwest slopes. Pre-split blasting consists of drilling a row of closely-spaced holes along the design excavation limit, charging them lightly, and then detonating them simultaneously or in groups separated by short delays. Firing the pre-split row creates a crack that forms the excavation limit and helps to prevent wall rock damage by venting explosive gases and reflecting shock waves. The pre-split row is fired in advance of the adjacent trim blast, which must be designed to limit damage beyond the pre-split line. The trim blast includes a “Buffer” row adjacent to the pre-split row to break back to, but not beyond, the pre-split, by ensuring that the Buffer row is fired with good horizontal relief.

Optimizing the Buffer row location and charge is often the most challenging item when implementing a pre-split, since it must be close enough to the pre-split to break the toe, but should not create excessive crest back-break. The Buffer row must have a reduced spacing to enable clean breaking along the pre-split line, and a reduced burden so that it can easily push its burden away from the pre-split. Also, the Buffer row must be fired with effective horizontal relief; otherwise, instead of moving the broken ground forward to the free face, if it is fired in a choked condition it will break equally in both directions and will cause excessive back-break across the pre-split line due to flexural rupture and block heave.

While every pre-split design should be optimized based on site conditions, the following general guidelines provide a starting point for initial design of a pre-split in competent rock using 5-inch diameter blastholes:

- Spacing between blastholes in the pre-split line is typically about 12 blasthole diameters in hard rock, or about 5 ft with 5-inch pre-split holes
- Charge weight per pre-split hole = 1 kg/m^2 of wall area created, or 1 lb/ft for pre-split holes on 5 ft spacings, or 50 lbs for 50 ft holes on 5 ft spacings
- Distributed charges will give better results than toe charges, and are necessary for longer blastholes; continuous decoupled charges should be used for initial blasting trials
- If de-coupled distributed charges are used, load to about 8 blasthole diameters (3.5 ft) from the drillhole collar
- Inclining the pre-split line 10° to 20° off vertical will reduce back-break and improve toe breakage, but vertical pre-split holes can be used if the drill rig is not capable of drilling inclined holes
- Pre-split holes should be detonated simultaneously, or nearly so, and at least 50 ms (milliseconds) before the first holes of the adjacent trim rows
- The Buffer row (adjacent to the pre-split row) should be located ⅓ to ½ the normal burden distance in front of the pre-split line – for a 12 ft pattern this indicates a standoff distance from the toe of the pre-split row of 4 ft to 6 ft
- The burden and spacing of the Buffer row is typically about ½ to ⅔ of normal production holes – 50% is often used to facilitate pattern tie-in; this suggests about 6 ft spacing and a 6 ft to 8 ft burden for a typical 12 ft production pattern
- Powder factor for the Buffer row should be the same as for the production holes; charge weights of the Buffer holes are reduced to account for their reduced burden and spacing –the charge weight in Buffer row blastholes with 6 ft spacing and 8 ft burden should be reduced to about 33% of the charge of a 12 ft x 12 ft production pattern
- No sub-grade drilling above final benches or in the vicinity of bench crests
- No stemming in pre-split holes; if stemming is necessary for noise control, stem only at collar using minimum stemming
- Air deck Buffer row to reduce confinement and limit crest damage, or alternatively reduce stemming if oversize rock from the upper portion of the bench is a concern
- The trim shot in front of pre-split line, and particularly the Buffer row, must not be confined but must fire to a free face to prevent damage behind the pre-split line

The design of the pre-split should be optimized in the field based on performance. A successful pre-split will include:

- Stable bench faces with the pre-split barrels visible over most of the bench height
- Clean toes
- Well defined and linear bench crests at close to the design crest line
- Effective catch benches at close to the design width.

3.2.2 Cushion Blast Designs

Pre-split blasting is not necessary where flatter bench face angles and flatter overall slopes are acceptable. Flatter design bench face angles are recommended in the North, Northeast, and East sector walls because of the indicated structural control, and for weathered rock that may not be able to hold a pre-split. Cushion blasting could also be used for fresh rock in the Northwest and West walls for phase slopes, or for final slopes, if it is determined that steeper slopes do not warrant the cost of a pre-split. In this case, design slope angles should be reduced by 5° from design slope angles that are used assuming a good presplit.

Cushion blasting consists of a trim blast that incorporates a Toe row of reduced burden and spacing to reduce damage to the bench face and to help define the toe of the bench. The design of the Toe row is similar to that of the Buffer row in the pre-split, and air decking or reduced stemming is generally required to limit crest damage. The spacing is less than the burden to promote breakage between the toe holes. The purpose of the Toe row is to define the toe of the bench, and it is generally drilled within 3 feet of the design toe of the bench. The row adjacent to the Toe row is designed to define the crest of the bench, and reduced charges and air decking are commonly necessary to limit crest damage.

4.0 SCALING

Effective scaling must remove potential rockfall from bench faces and crests before drilling activities are initiated on the underlying bench. The bench crest should be inspected from the crest level bench to identify potential rockfall that should be removed. Design bench heights of 50 ft exceed the reach of a standard 988 loader, and so the upper portion of the bench faces could only be reached by the loader if it is worked off a muck pile or berm. If loaders cannot effectively scale upper bench faces and bench crests, then alternative methods such as chaining the upper slope and crest using a dozer on the crest level bench, or trimming bench crests with a dozer in advance of excavating the shot rock, should be implemented.

5.0 RISKS ASSOCIATED WITH STEEP SLOPE DESIGNS

There appears to be little risk of large-scale slope failures developing based on our current understanding of engineering geologic conditions at the Quarry. Risks of failure to achieve design slope angles appear to be predominantly related to the risk of encountering locally unfavorable structural conditions, or from damage to the benches due to poor operating practices. In either case, the result is likely to be a safety hazard resulting from the increased risk of rockfall to benches below, particularly during or immediately following precipitation events or earthquakes. With conservative slope designs, the risk of encountering conditions that will require a modification to the slope design is always less than with more aggressive slope designs. If mining has proceeded below a hazardous area, there is often little alternative for re-establishing safe working conditions other than incorporating a step-out into the slope design to provide additional catchment for rockfall. With an aggressive slope design, the effects of a step-out on inter-ramp slope angles is greater, and the potential for recovering to the original slope design by locally over-steepening the slope is less. This is particularly the case for high slopes that do not incorporate haul ramps to flatten the overall slope angle to less than the inter-ramp slope angle.