SOUTHERN CALIFORNIA SOIL & TESTING, INC.
A California Certified Small Business Enterprise (SBE)

San Diego
619.280.4321
6280 Riverdale Street
San Diego, CA 92120

Indio
760.775.5983
83-740 Citrus Avenue
Suite G
Indio, CA 92201-3438

Riverside
951.965.8711
1130 Palmyrita Avenue
Suite 330-A
Riverside, CA 92507

Toll Free
877.215.4321
www.scst.com

GEOTECHNICAL INVESTIGATION
PLANNED RETAINING WALL AND
QUARRY EXPANSION
ROSEMARY'S MOUNTAIN QUARRY
5600 HIGHWAY 76
SAN DIEGO COUNTY, CALIFORNIA

PDS2013-MUP-87-021W2, PDS2013-RC-87-001W2
Log No: 87-2-13

PREPARED FOR:

MR. GARY NOLAN
PLANT MANAGER
GRANITE CONSTRUCTION COMPANY
38000 MONROE STREET
INDIO, CALIFORNIA 92203

PREPARED BY:

SOUTHERN CALIFORNIA SOIL & TESTING, INC.
6280 RIVERDALE STREET
SAN DIEGO, CALIFORNIA 92120

Providing Professional Engineering Services Since 1959

SOUTHERN CALIFORNIA
SOIL & TESTING, INC.
A California Certified Small Business Enterprise (SBE)

San Diego
619.280.4321
6280 Riverdale Street
San Diego, CA 92120

Indio
760.775.5983
83-740 Citrus Avenue
Suite G
Indio, CA 92201-3438

Riverside
951.965.8711
1130 Palmyrita Avenue
Suite 330-A
Riverside, CA 92507

Toll Free
877.215.4321
www.scst.com

April 2, 2014

SCS&T No. 1311133
Report No. 1R3

Mr. Gary Nolan
Plant Manager
Granite Construction Company
38000 Monroe Street
Indio, California 92203

Subject: GEOTECHNICAL INVESTIGATION
PLANNED RETAINING WALL AND QUARRY EXPANSION
ROSEMARY'S MOUNTAIN QUARRY
5600 HIGHWAY 76
SAN DIEGO COUNTY, CALIFORNIA

Dear Mr. Nolan:

This letter transmits Southern California Soil & Testing Inc.'s (SCS&T) report describing the geotechnical investigation performed for the subject project. If you have any questions concerning this report, or need additional information, please call us at (619) 280-4321.

Respectfully Submitted,
SOUTHERN CALIFORNIA SOIL AND TESTING, INC.

REGISTERED PROFESSIONAL GEOLOGIST
DOUGLAS A. SKINNER
No. 2472
Exp. 8-31-14
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Douglas A. Skinner, CEG 2472
Senior Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER
GARRETT B. FOUNTAIN
NO. 2752
EXP. 09-30-14
GEOTECHNICAL
STATE OF CALIFORNIA

Garrett B. Fountain, GE 2752
Principal Geotechnical Engineer

GBF:DAS:aw

(2) Addressee
(1) Addressee via e-mail at Gary.Nolan@gcinc.com

TABLE OF CONTENTS

SECTION **PAGE**

ATTACHMENTS

FIGURES

APPENDICES

EXECUTIVE SUMMARY

This report presents the results of the geotechnical investigation Southern California Soil and Testing, Inc. (SCS&T) performed for the subject project at the Rosemary's Mountain Quarry. We understand the project consists of the expansion of the existing quarry to the north, the design and construction of mechanically stabilized earthen (MSE) retaining walls during mining phase 2, and the creation of a relatively large mining waste dump behind the planned cut slope during mining phase 3. The project is located at 5600 Highway 76 in the Fallbrook area of San Diego County, California. The purpose of our work is to provide conclusions and recommendations regarding the geotechnical aspects of the project.

Nineteen exploratory test borings were drilled using an air-rotary hammer drill rig. The borings extended to depths of between about 20 feet and 69 feet below the existing ground surface. SCS&T's geologist observed fill or residual soil underlain by granodiorite of Indian Mountain and gabbro rock in the borings. The fill is comprised of loose to medium dense silty sand with gravel. The residual soil consists of loose to medium dense clayey and silty sand. Groundwater was encountered at a depth of about 41 feet below the existing ground surface in boring B-13.

Temporary excavations are anticipated in the quarry expansion area. As part of the project's Reclamation Plan (Reclamation Plan No. RP87-001W2) filed with the County of San Diego, and as a condition of operation, mining shall be conducted in from a top down and from north to south or south to north along the face being worked. The working face shall be a maximum of approximately 33 feet in height with a flat working bench below it. The flat working bench shall be 66 feet wide whenever possible. This will result in an interim benched slope configuration as mining proceeds downward at 33-foot intervals. A Certified Engineering Geologist (CEG) or a Geotechnical Engineer (GE) shall map the exposed rock surface on a quarterly basis. Inspection and mapping of the mining face may be more frequent, as needed depending on field conditions. Each 33-foot vertical face may only be removed following on-site inspection and in accordance with the written recommendations of the CEG or GE.

During the Site Preparation Phase, temporary MSE walls will be constructed to create a pad at the 490-feet elevation in the central portion of the project. During mining phase 3, the north eastern portion of the project will be backfilled and a 2:1 (horizontal:vertical) fill slope comprised of overburden waste will be constructed. This fill slope will range in height from approximately 100 to 150 feet and will be on the 490-feet elevation pad. Our slope stability analyses indicate the 1:1 H:V excavations, temporary MSE walls, and proposed 2:1 fill slope have adequate factors of safety in respect to global stability provided the recommendations set forth in this report are

implemented. In our opinion, the proposed steeper slopes exposing rock will be stable and will not endanger any public or private property or result in the deposition of debris on any public way or interfere with any existing drainage course if the recommendations herein are implemented.

1. INTRODUCTION

1.1 GENERAL

This report presents the results of the geotechnical investigation Southern California Soil and Testing, Inc. (SCS&T) performed for the subject project at the Rosemary's Mountain Quarry. We understand the project consists of the expansion of the existing quarry to the north, the design and construction of mechanically stabilized earthen (MSE) retaining walls during mining phase 2, and the creation of a relatively large mining waste dump behind the planned cut slope during mining phase 3. The project is located at 5600 Highway 76 in the Fallbrook area of San Diego County, California. The purpose of our work is to provide conclusions and recommendations regarding the geotechnical aspects of the project. Figure 1 presents the site location.

1.2 SCOPE OF WORK

1.2.1 Field Exploration

Subsurface conditions were explored by drilling 19 exploratory test borings using an air-rotary hammer drill rig at the locations shown on Figure 2. The test borings extended to depths of between about 20 and 69 feet below the existing ground surface. An SCS&T geologist logged the test borings and obtained samples for examination and laboratory testing. The logs of the test borings are in Appendix I. The rate of time in seconds per foot of depth is presented on the boring logs. Soils are classified according to the Unified Soil Classification System illustrated on Figure I-1.

1.2.2 Laboratory Testing

The laboratory program consisted of tests for:

- Maximum Density and Optimum Moisture Content;
- Direct Shear.

The results of the laboratory tests, and brief explanations of test procedures, are in Appendix II.

1.2.3 Analysis and Report

The results of the field and laboratory tests were evaluated to develop conclusions and recommendations regarding:

1. Subsurface conditions
2. Criteria for seismic design
3. Alternatives for foundation support along with geotechnical engineering criteria for design of the foundations

4. Lateral loads on retaining walls, including earthquake forces
5. Resistance to lateral loads
6. Estimated foundation settlements
7. Slope stability

2. SITE AND SUBSURFACE CONDITIONS

2.1 SITE DESCRIPTION

The planned quarry expansion will be located along the northern portion of the site, north of the active quarry, within an existing citrus tree grove. Topographically, the area of the planned expansion slopes to the south with an elevation difference of about 25 feet over a span of about 100 feet. Vegetation consists of citrus trees, native shrubs and grasses.

The planned MSE wall locations are within the northern portion of the active quarry. This area consists of a level pad, at the 490-feet elevation, containing stockpiles of rock, and asphalt concrete material. Currently, a temporary fill slope about 30 feet high descends from the south side of the pad to the south.

2.2 SUBSURFACE CONDITIONS

2.2.1 MSE Wall Area

Fill underlain by granodiorite of Indian Mountain and gabbro rock was observed in the borings within the areas of the planned retaining walls in borings B-1 through B-5. The fill is comprised of loose to medium dense silty sand with gravel. The fill extends between about 5 feet and 19 feet below the ground surface. The grandiorite of Indian Mountain is moderately weathered to slightly weathered and hard. The gabbro is intensely weathered, moderately soft and can be broken down to silty sand under moderate hand pressure. The rock extends beyond the maximum depth explored of 50 feet.

2.2.2 Quarry Expansion Area

Fill and/or residual soil underlain by gabbro rock was observed in the borings drilled in the grove area. Fill was encountered in borings B-10, B-11 and B-18. The fill extends between about 5 feet and 13 feet below the ground surface and consists of loose to medium dense silty sand with gravel. Residual soil was encountered in borings B-6 through B-9, B-12 through B-17, and B-19. The residual soil extends between about 1 foot and 3 feet below the ground surface and consists of clayey and silty sand. The gabbro is intensely weathered, moderately soft and can be broken down to silty sand under moderate hand pressure. The rock extends beyond the maximum depth explored of 69 feet.

2.2.3 Groundwater

SCS&T's geologist observed groundwater at a depth of about 41 feet below the ground surface in boring B-13. However, groundwater levels can fluctuate following periods of precipitation or irrigation. It is likely that water will become perched in the existing fill following heavy rains or irrigation. In addition, the presence or absence of groundwater encountered during mining operations is dependent on the size, spacing, and interconnectivity of fractures within the bedrock.

2.3 SEISMIC DESIGN PARAMETERS

A geologic hazard likely to affect the project is groundshaking as a result of movement along an active fault zone in the vicinity of the subject site. The site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters in accordance with the California Building Code are presented below:

Site Coordinates: Latitude 33.343°
Longitude -117.141°
Site Class: C
Site Coefficient F_a = 1.0
Site Coefficient F_v = 1.3
Spectral Response Acceleration at Short Periods S_s = 1.500
Spectral Response Acceleration at 1-Second Period S_1 = 0.599
$S_{MS} = F_a S_s$ = 1.500
$S_{M1} = F_v S_1$ = 0.778
S_{DS}=2/3* S_{MS} = 1.000
S_{D1}=2/3* S_{M1} = 0.519

2.4 SLOPE STABILITY ANALYSIS

Northern Expansion Temporary Cut Slopes

SCS&T performed a slope stability analysis on the temporary cut slope depicted in cross-section A-A' using the commercially available software GStabl7V.2. SCS&T used direct shear tests to assist in determining strength parameters. The static factor-of-safety determined using this method is 1.3 during construction. The pseudostatic factor-of-safety determined using earthquake loads is 1.1 during construction. Results of our analyses are presented in Appendix III.

Phase 3 Overburden Fill Slopes

As part of the project's Reclamation Plan (Reclamation Plan No. RP87-001W2) filed with the County of San Diego, the north-eastern portion of the project will be backfilled and a 2:1 fill slope will be constructed (fill area Phase 3). This slope will range in height between 60 feet and

80 feet in vertical height. We understand, the toe of this slope will be at least 200 feet behind the top of the planned 0.176:1 cut slope proposed for mining Phase 3.

SCS&T performed a slope stability analysis on the fill slope depicted in cross-section A-A' using the commercially available software GStabl7V.2. SCS&T used direct shear tests to assist in determining strength parameters. The static factor-of-safety determined using this method is 2.7. The pseudostatic factor-of-safety determined using earthquake loads is 1.7. Results of our analyses are presented in Appendix III. It is also our opinion that the fill slope will not generate surcharge loads on the proposed 0.176:1 cut slope.

2.5 SURFICIAL SLOPE STABILITY ANALYSIS

Northern Expansion Temporary Cut Slopes

Weathered gabbroic rock will be exposed along the surface of the planned cut slope. In our opinion, the most likely failure to occur at the site is a surficial slope failure. This type of failure is typical on steepened slopes with granular materials. SCS&T performed a surficial slope stability analysis for the planned 1:1 temporary cut slope. The factor-of-safety is 4.4 at a depth of 2 feet below the slopes surface. The result of our analysis is presented in Appendix III.

During construction, isolated areas of unstable rock may be exposed. SCS&T should perform periodic inspection of the slope to determine if rock bolts or other temporary rock restraining systems should be implemented.

Phase 3 Overburden Fill Slopes

In our opinion, the most likely failure to occur with respect to the fill slope is a surficial slope failure. SCS&T performed a surficial slope stability analysis for the planned 2:1 fill slope. The factor-of-safety is 1.6 at a depth of 2 feet below the slopes surface. The result of our analysis is presented in Appendix III.

To minimize the potential for surficial slope failure, the project's Civil Engineer should design the grades in the vicinity of the slope to direct water away from the tops of slopes and toward approved drainage areas. Drainage terraces should be constructed parallel to the fill slope face to intercept surface water. The vertical intervals of the drainage terraces should not exceed 30 feet. Paving of the drainage terraces and concrete brow ditches is not required, as they may hinder the revegetation efforts. Other alternatives that could be implemented consist of stabilizing the slope by planting and mechanical means with a cellular confinement system such as GeoWeb®.

3. CONCLUSIONS

Temporary excavations of up to about 105 feet in height at an inclination of about 1:1 (horizontal:vertical) are anticipated in the quarry expansion area. Our slope stability analyses indicate the 1:1 (H:V) excavations and proposed fill slopes have an adequate factor of safety in respect to global stability and surficial stability. In our opinion the proposed steeper slopes exposing competent rock will be stable and will not endanger any public or private property or result in the deposition of debris on any public way or interfere with any existing drainage course if the recommendations herein are implemented.

No specific wall foundation plans are available at this time. However, we expect the planned walls will extend about 40 feet in height with bottom levels on bedrock. A licensed geotechnical engineer should review the wall plans for global stability once they become available.

4. RECOMMENDATIONS

4.1 MSE WALL DESIGN PARAMETERS

The following soil parameters can be used for the design of Mechanically Stabilized Earth (MSE) walls.

TABLE 1

Mechanically Stabilized Earth Wall Design Parameters

	Reinforced Soil	Retained Soil	Foundation Soil
Internal Friction Angle (degrees)	30°	30°	30°
Cohesion (pounds per square foot)	0	0	0
Moist Unit Weight (pounds per cubic foot)	130	130	130

4.1.1 Passive Pressure

Passive pressure for the design of retaining walls can be taken as 300 psf per foot of depth. This pressure can be increased by ⅓ for seismic loading. The allowable friction coefficient of 0.3 can be used. The upper 12 inches of material in front of foundations should not be included in passive pressure calculations unless the surface is covered with pavement.

4.1.2 Active Pressure

The active soil pressure for the design of unrestrained earth retaining structures with level backfills can be taken as equivalent to the pressure of a fluid weighing 40 pounds per cubic foot (pcf). The active soil pressure for the design of restrained earth retaining structures with level backfills can be taken as equivalent to the pressure of a fluid weighing 60 pcf. An

additional 20 pcf should be added to the above values for walls with a 2:1 (horizontal: vertical) sloping backfill. An increase in soil pressure equivalent to an additional 2 feet of retained soil can be used to account for surcharge loads from light traffic. A granular and drained backfill condition has been assumed. If any other surcharge loads are anticipated, SCS&T should be contacted for the necessary increase in soil pressure.

4.1.3 Seismic Earth Pressure

The seismic earth pressures can be taken as an inverted triangular distribution with a maximum pressure at the top equal to 21H pounds per square foot (with H being the height of the wall in feet). This pressure is in addition to the static design wall load. The allowable passive pressure and bearing capacity can be increased by ⅓ in determining the stability of the wall. The seismic coefficient for design of MSE walls can be taken as 0.16.

4.1.4 Backfill

Backfill should consist of predominately granular soil free of organic material and rocks greater than 3 inches in maximum dimension. The backfill should be placed in lifts 6 inches or less in loose thickness, moisture conditioned to within 2 percentage points above optimum moisture content and compacted to at least 90% relative compaction determined in accordance with ASTM - D 1557. Expansive or clayey soils should not be used for backfill behind retaining walls.

4.1.5 Factor of Safety

The above values, with the exception of the allowable soil bearing pressure, do not include a factor of safety. Appropriate factors of safety should be incorporated into the design.

5. GEOTECHNICAL ENGINEERING DURING CONSTRUCTION

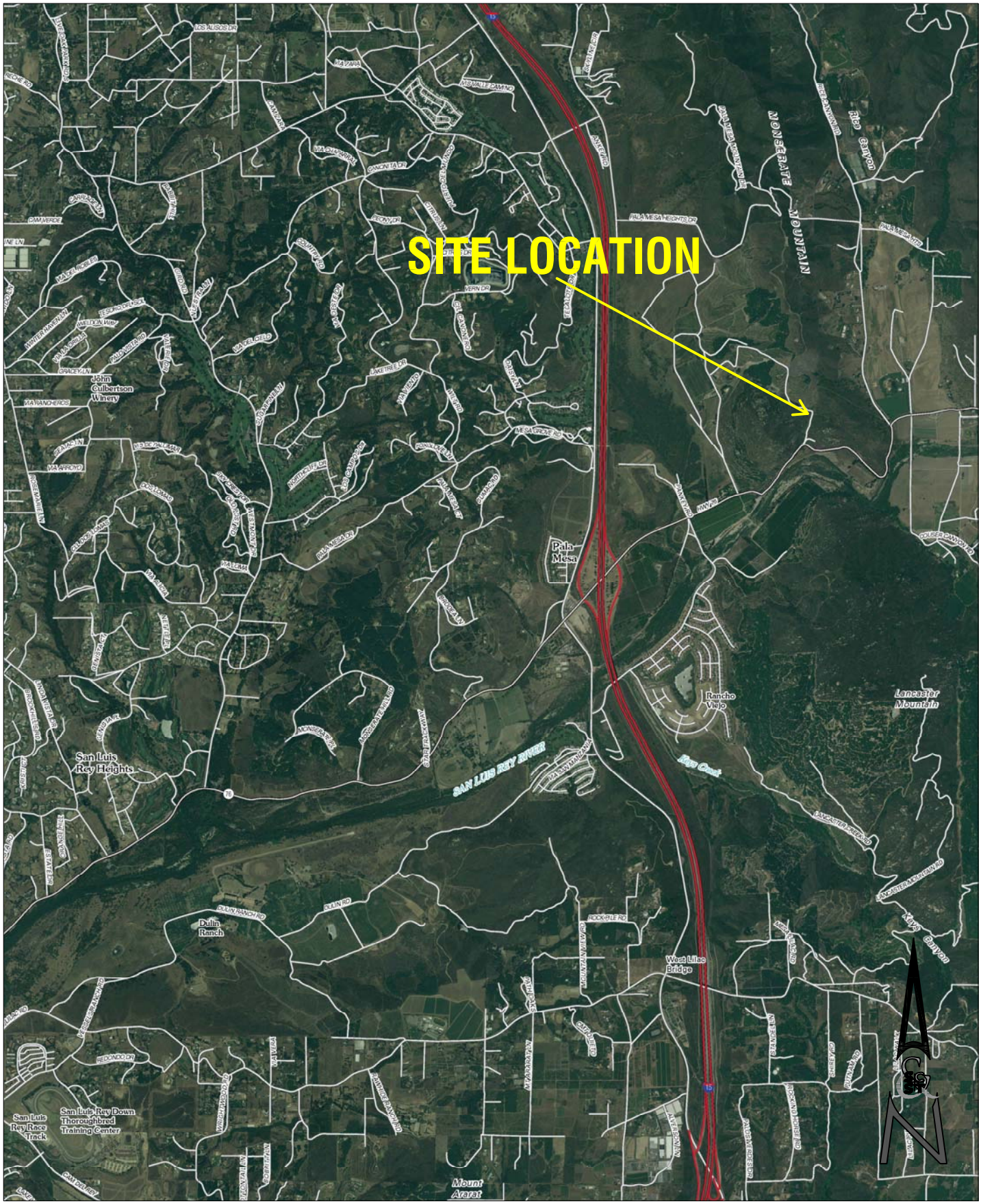
The geotechnical engineer should review project plans and specifications prior to bidding and construction to check that the intent of the recommendations in this report has been incorporated. Observations and tests should be performed during construction. If the conditions encountered during construction differ from those anticipated based on the subsurface exploration program, the presence of the geotechnical engineer during construction will enable an evaluation of the exposed conditions and modifications of the recommendations in this report or development of additional recommendations in a timely manner.

6. CLOSURE

SCS&T should be advised of any changes in the project scope so that the recommendations contained in this report can be evaluated with respect to the revised plans. Changes in

recommendations will be verified in writing. The findings in this report are valid as of the date of this report. Changes in the condition of the site can, however, occur with the passage of time, whether they are due to natural processes or work on this or adjacent areas. In addition, changes in the standards of practice and government regulations can occur. Thus, the findings in this report may be invalidated wholly or in part by changes beyond our control. This report should not be relied upon after a period of two years without a review by us verifying the suitability of the conclusions and recommendations to site conditions at that time.

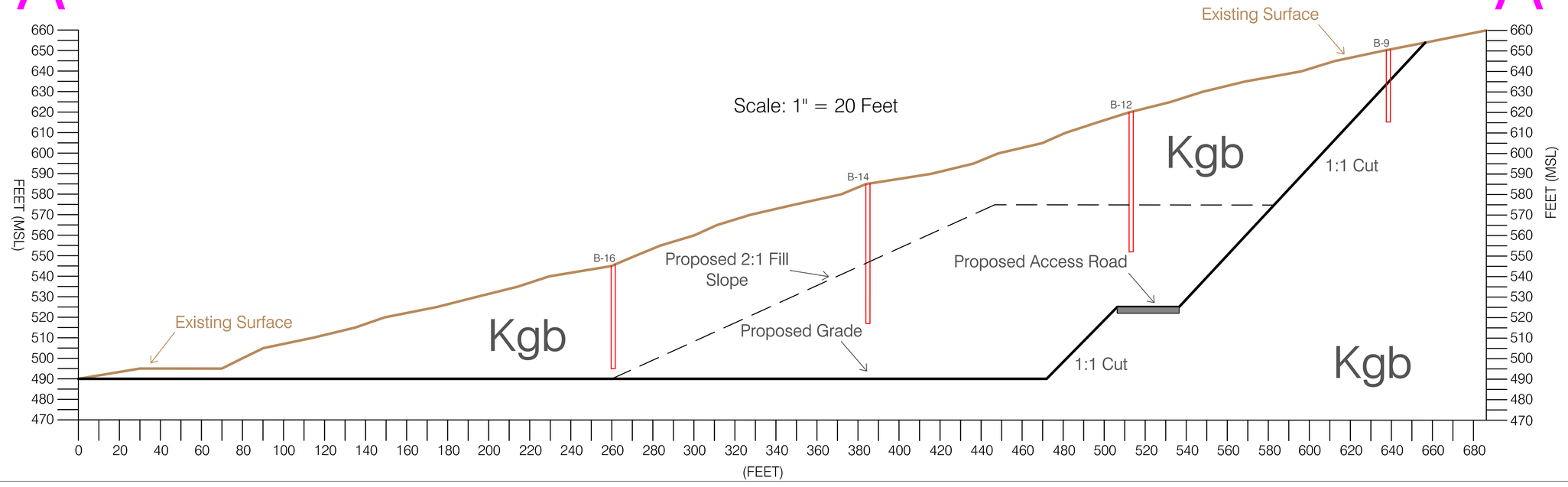
In the performance of our professional services, we comply with that level of care and skill ordinarily exercised by members of our profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the boring locations, and that our data, interpretations, and recommendations are based solely on the information obtained by us. We will be responsible for those data, interpretations, and recommendations, but shall not be responsible for interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty of any kind whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.



SOUTHERN CALIFORNIA SOIL & TESTING, INC.	SITE VICINITY MAP ROSEMARY'S MOUNTAIN RETAINING WALL AND QUARRY EXPANSION	Date: September 2013 By: AKN Job No.: 1311133-1 Scale: Not To Scale	Figure: 1

SCS&T LEGEND

- B-19 / 19 — Approximate Test Boring Location (Italicized number indicates depth of fill, if encountered)
- Kgb — Gabbro, Undivided
- Ki — Granodiorite of Indian Mountain
- Approximate Geologic Contact
- A — A' — Approximate Location of Cross Section A-A'



CROSS SECTION A-A'
ROSEMARY'S MOUNTAIN RETAINING WALL AND QUARRY EXPANSION
Pala, California

Date: April 2014
By: AKN/DAS/EAK
Job No.: 1311133-1R3
Scale: Various

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

Figure: 2

APPENDIX I
FIELD INVESTIGATION

Nineteen exploratory test borings were drilled using an air-rotary drill rig at the locations indicated on Figure 2 on August 12 and 13, 2013. The rate of drilling was recorded as time in seconds/foot of depth. The fieldwork was performed under the observation of a SCS&T geologist, who also logged the borings and obtained samples of the materials encountered.

Figures I-1 and I-2 present the results.

RATE (Average Seconds/3-foot Interval)

DEPTH BELOW SURFACE (ft)	AT-1	AT-2	AT-3	AT-4	AT-5	AT-6	AT-7	AT-8	AT-9	AT-10
0-3	5	9	10	7	4	5	5	2	9	4
3-6	21	7	5	7	6	8	3	7	9	2
6-9	20	6	5	3	12	8	9	11	8	2
9-12	15	10	10	9	9	7	10	11	10	2
12-15	12	10	13	12	7	8	10	11	10	10
15-18	14	11	13	14	7	9	10	9	11	10
18-21	15	16	15	12	8	11	11	10	11	10
21-24	13	13	17	14	14		12	9	10	11
24-27	18	14	16	15	12		14	11	13	11
27-30	20	12	17	13	12			12	11	
30-33	19	14	14	11	10				13	
33-36	17	17	16	16	13				13	
36-39	22	17	16	15	13					
39-42	21	16	14	21	12					
42-45	18	14	33	15	12					
45-48	23	18	16	15	16					
48-51	15	16	16	15	15					
51-54										
54-57										
57-60										
60-63										
63-66										
66-69										

Rate (sec./ft)	Hardness	Rippabililty
10-18	Soft-Medium	Rippable
18-25	Medium-Hard	Probable
>25	Hard	Heavy Ripping/ Blasting
>35	Very Hard	Blasting

Test Drilling data provided by M.J. Baxter Drilling Co. Based on ECM 590RC with a 4-inch bit.

ROSEMARY'S MOUNTAIN RETAINING WALL & QUARRY EXPANSION			
By:	AKN	Date:	4/2/2014
Job Number:	1311133-1R3	Figure:	I-1

RATE (Average Seconds/3-foot Interval)

DEPTH BELOW SURFACE (ft)	AT-11	AT-12	AT-13	AT-14	AT-15	AT-16	AT-17	AT-18	AT-19	
0-3	2	9	8	8	3	5	4	3	6	
3-6	4	22	8	7	4	9	3	8	9	
6-9	3	17	8	8	4	7	4	9	8	
9-12	2	18	9	9	4	7	4	11	9	
12-15	6	9	11	10	7	7	10	10	9	
15-18	10	22	11	11	6	11	32	11	13	
18-21	12	9	10	10	8	9	10	10	12	
21-24	17	9	10	10	9	11	7	10	10	
24-27	17	11	14	9	10	15	24	11	11	
27-30		19	18	12	10	11	10	10	12	
30-33		12	28	12	10	12	12	10		
33-36		11	28	10	8	14	16	10		
36-39		14	34	37	12	12	14	12		
39-42		15	39*	40	11	11	16	7		
42-45		13	35	38	9	10	17			
45-48		14	33	35	18	19	18			
48-51		7	42	41	22	15	24			
51-54		13	39	38	28		17			
54-57		16	45	34	26		9			
57-60		33	42	36	14					
60-63		27	46	39	22					
63-66		14	37	36	26					
66-69		13	29	29	18					

* Groundwater was encountered at about 41 feet below existing grade.

Rate (sec./ft)	Hardness	Rippabililty
10-18	Soft-Medium	Rippable
18-25	Medium-Hard	Probable
>25	Hard	Heavy Ripping/ Blasting
>35	Very Hard	Blasting

Test Drilling data provided by M.J. Baxter Drilling Co. Based on ECM 590RC with a 4-inch bit.

ROSEMARY'S MOUNTAIN RETAINING WALL & QUARRY EXPANSION			
By:	AKN	Date:	4/2/2014
Job Number:	1311133-1R3	Figure:	I-2

APPENDIX II
LABORATORY TESTING

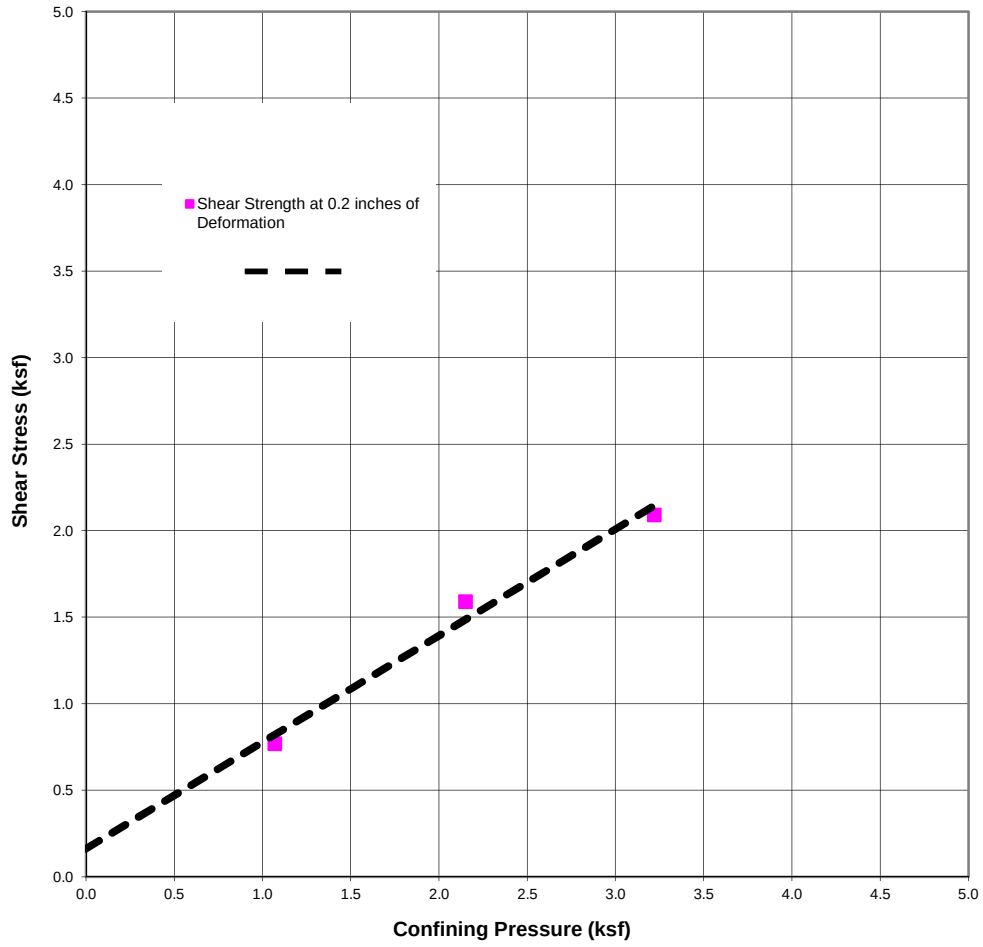
SUMMARY

Laboratory tests were performed to provide geotechnical parameters for engineering analyses. The following tests were conducted:

- **MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENT:** The maximum density and optimum moisture content of 5 samples were determined in accordance with ASTM D 1557. The results are presented on Figures II-1 through II-5.
- **DIRECT SHEAR:** Five direct shear tests were performed in accordance with ASTM D 3080. The shear stress was applied at a constant rate of strain of approximately 0.003 inch per minute. The results of these tests are presented on Figures II-1 through II-5.

Soil samples not tested are now stored in our laboratory for future reference and analysis, if needed. Unless notified to the contrary, all samples will be disposed of 30 days from the date of this report.

Direct Shear Test Results



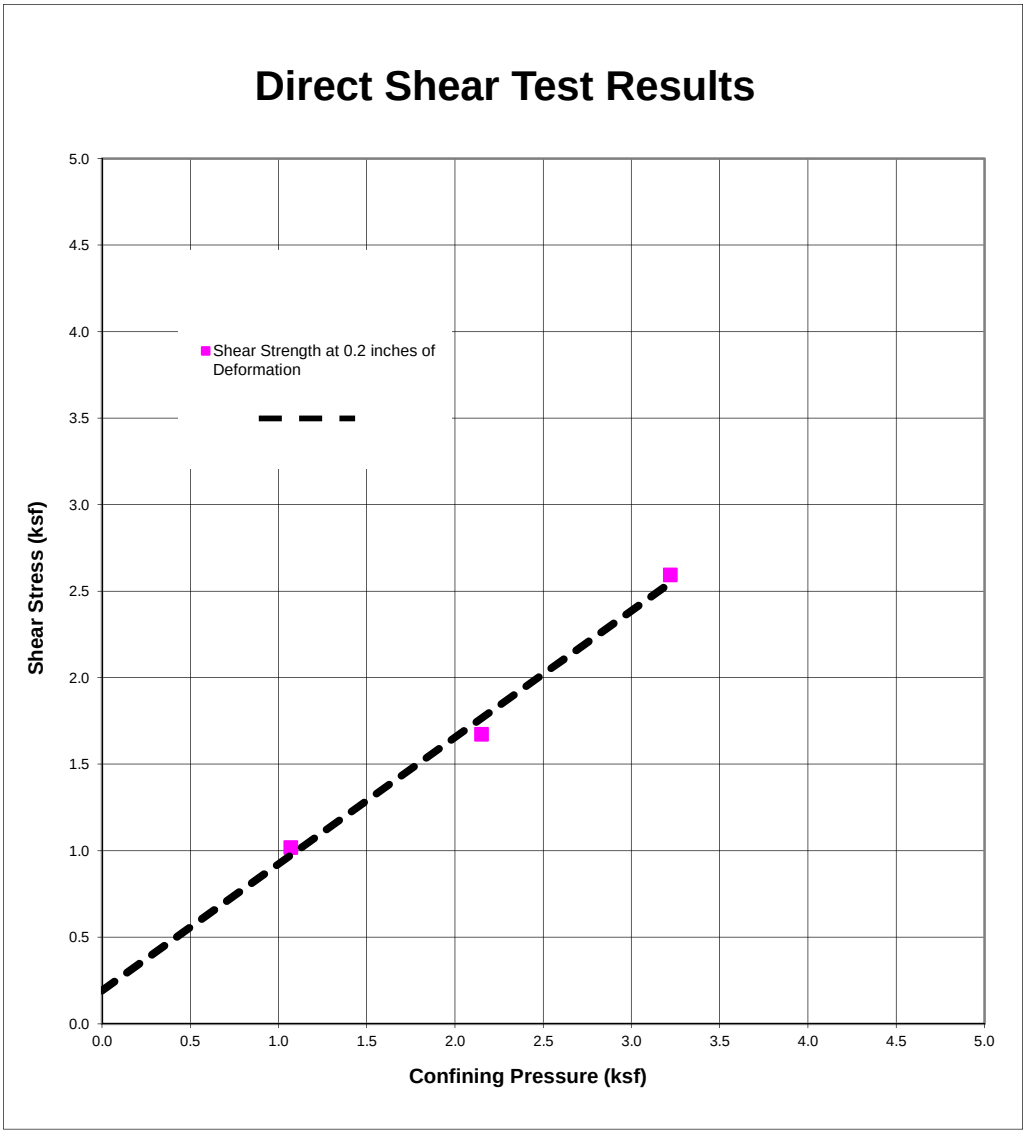
SAMPLE	DESCRIPTION	INTERNAL FRICTION ANGLE(DEG.)	COHESION INTERCEPT (PSF)
B-4 at 10 feet to 20 feet	SILTY SAND (Remolded to 90% relative compaction)		
Shear Strength at 0.2 inches of Deformation		32	163

MAXIMUM DENSITY & OPTIMUM MOISTURE CONTENT

123.7 pcf at 10.3% moisture content

SC & ST	**SOUTHERN CALIFORNIA SOIL & TESTING**	PLANNED RETAINING WALL AND QUARRY EXPANSION		
		By: AKN	Date:	4/2/2014
		Job No.: 1311133-1R3	Figure:	II-1

Direct Shear Test Results



SAMPLE	DESCRIPTION	INTERNAL FRICTION ANGLE(DEG.)	COHESION INTERCEPT (PSF)
B-6 at 6 feet at 16 feet	SILTY SAND (Remolded to 90% relative compaction)		
Shear Strength at 0.2 inches of Deformation		36	192

MAXIMUM DENSITY & OPTIMUM MOISTURE CONTENT

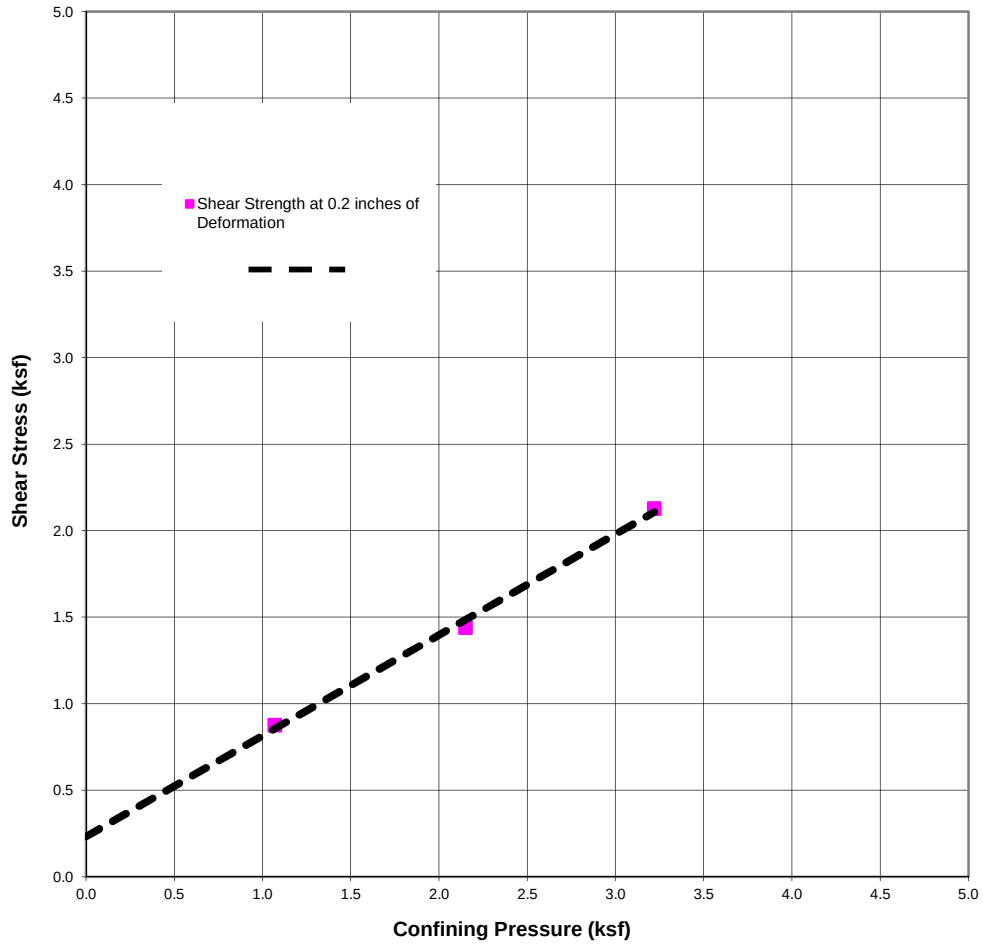
130.7 pcf at 8.5% moisture content

SOUTHERN CALIFORNIA SOIL & TESTING

PLANNED RETAINING WALL AND QUARRY EXPANSION

By:	AKN	Date:	4/2/2014
Job No.:	1311133-1R3	Figure:	II-2

Direct Shear Test Results



SAMPLE	DESCRIPTION	INTERNAL FRICTION ANGLE(DEG.)	COHESION INTERCEPT (PSF)
B-9 at 1 foot to 11 feet	SILTY SAND (Remolded to 90% relative compaction)		
Shear Strength at 0.2 inches of Deformation		30	232

MAXIMUM DENSITY & OPTIMUM MOISTURE CONTENT

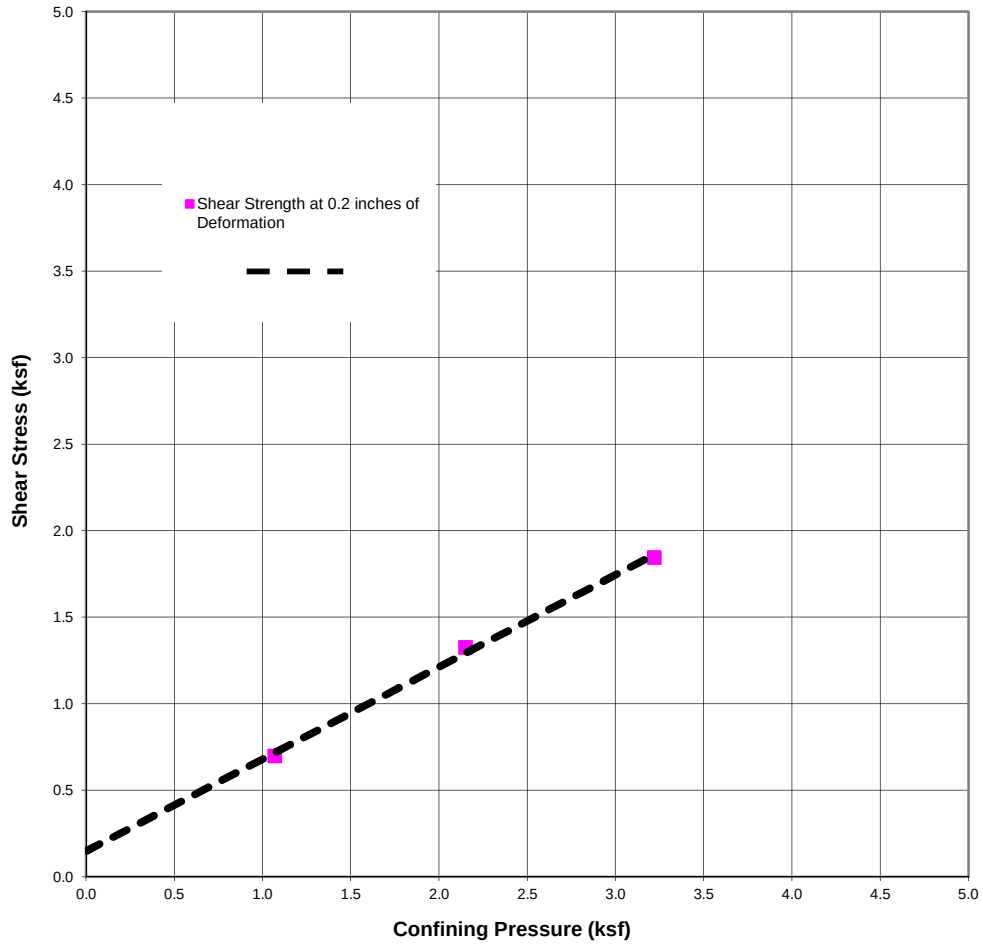
131.9 pcf at 8.3% moisture content

SOUTHERN CALIFORNIA SOIL & TESTING

PLANNED RETAINING WALL AND QUARRY EXPANSION

By:	AKN	Date:	4/2/2014
Job No.:	1311133-1R3	Figure:	II-3

Direct Shear Test Results



SAMPLE	DESCRIPTION	INTERNAL FRICTION ANGLE(DEG.)	COHESION INTERCEPT (PSF)
B-11 at 0 foot to 10 feet	SILTY SAND (Remolded to 90% relative compaction)		
Shear Strength at 0.2 inches of Deformation		28	147

MAXIMUM DENSITY & OPTIMUM MOISTURE CONTENT

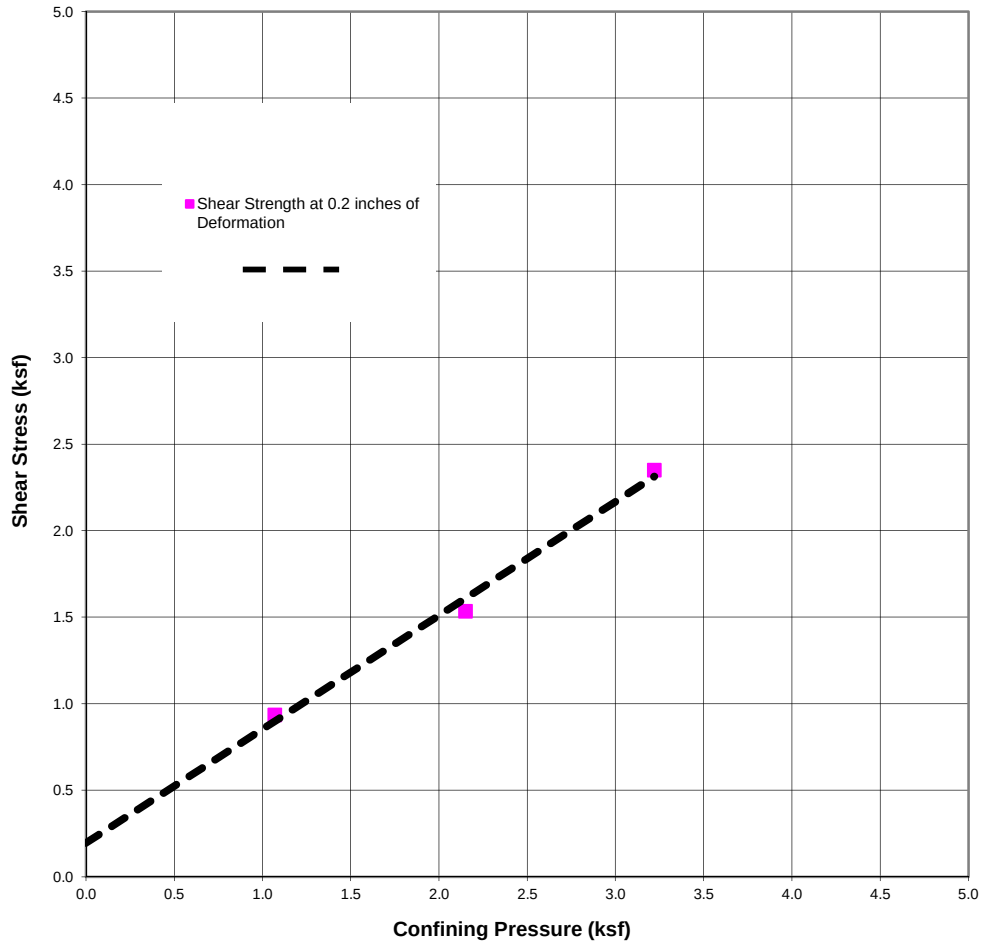
128.8 pcf at 7.5% moisture content

SOUTHERN CALIFORNIA SOIL & TESTING

PLANNED RETAINING WALL AND QUARRY EXPANSION

By:	AKN	Date:	4/2/2014
Job No.:	1311133-1R3	Figure:	II-4

Direct Shear Test Results



SAMPLE	DESCRIPTION	INTERNAL FRICTION ANGLE(DEG.)	COHESION INTERCEPT (PSF)
B-15 at 5 feet to 15 feet	SILTY SAND (Remolded to 90% relative compaction)		
Shear Strength at 0.2 inches of Deformation		33	196

MAXIMUM DENSITY & OPTIMUM MOISTURE CONTENT

127.0 pcf at 8.7% moisture content

SOUTHERN CALIFORNIA SOIL & TESTING

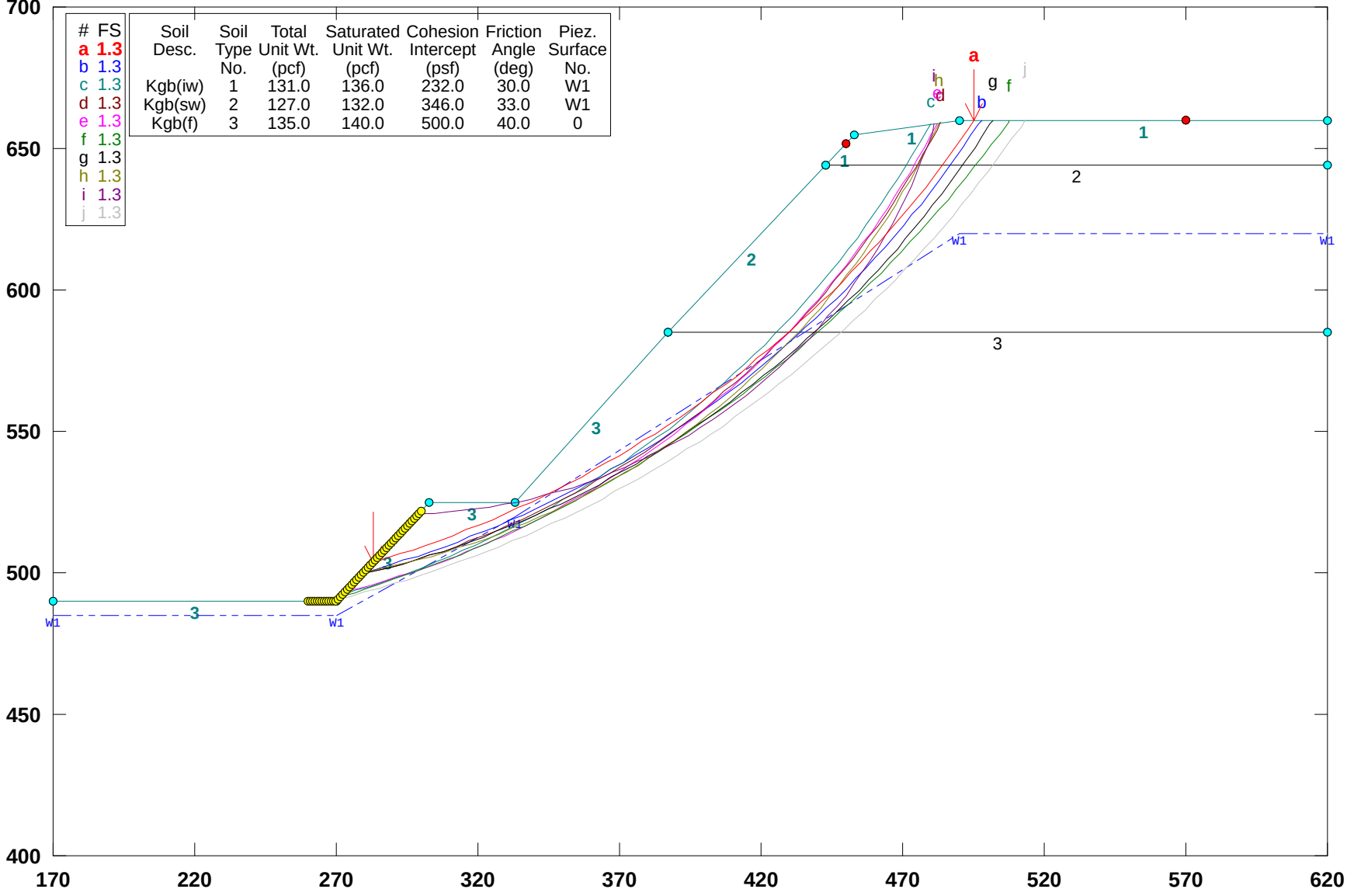
PLANNED RETAINING WALL AND QUARRY EXPANSION

By:	AKN	Date:	4/2/2014
Job No.:	1311133-1R3	Figure:	II-5

APPENDIX III
SLOPE STABILITY ANALYSIS

Rosemary's Mountain Quarry Expansion Cross Section A-A'

j:\geotechnical (11)\2013 job info\1311133 - rosemary's mt retaining walls & quarry\reports\1311133-1 geo inv\slope stability\-newfile.pl2 Run By: AKN 9/16/2013 07:37AM

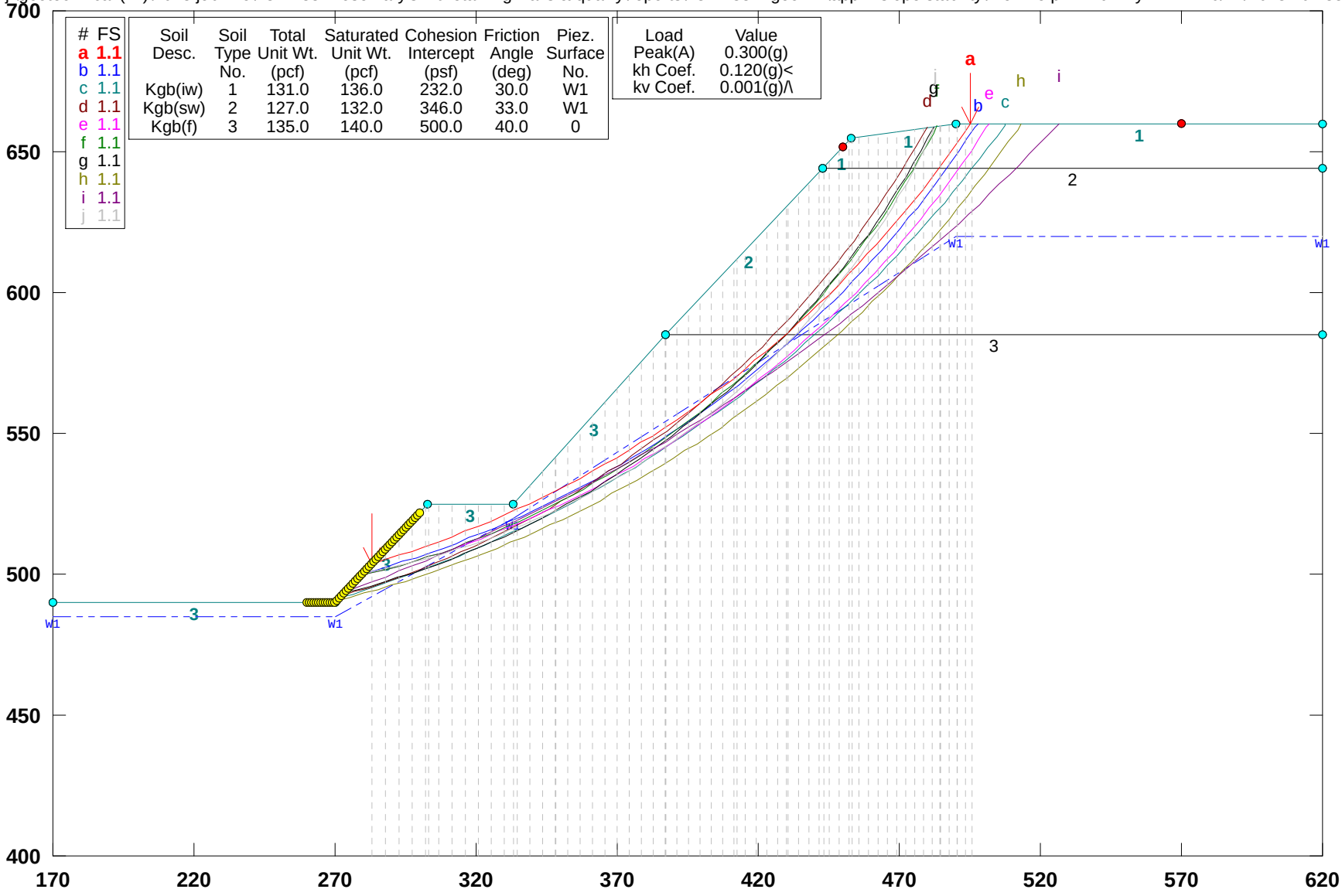


GSTABL7 v.2 FSmin=1.3

Safety Factors Are Calculated By The Modified Bishop Method

Rosemary's Mountain Quarry Expansion Cross Section A-A'

j:\geotechnical (11)\2013 job info\1311133 - rosemary's mt retaining walls & quarry\reports\1311133-1 geo inv\app iii slope stability\-newfile.pl2 Run By: AKN 10/22/2013 01:35PM

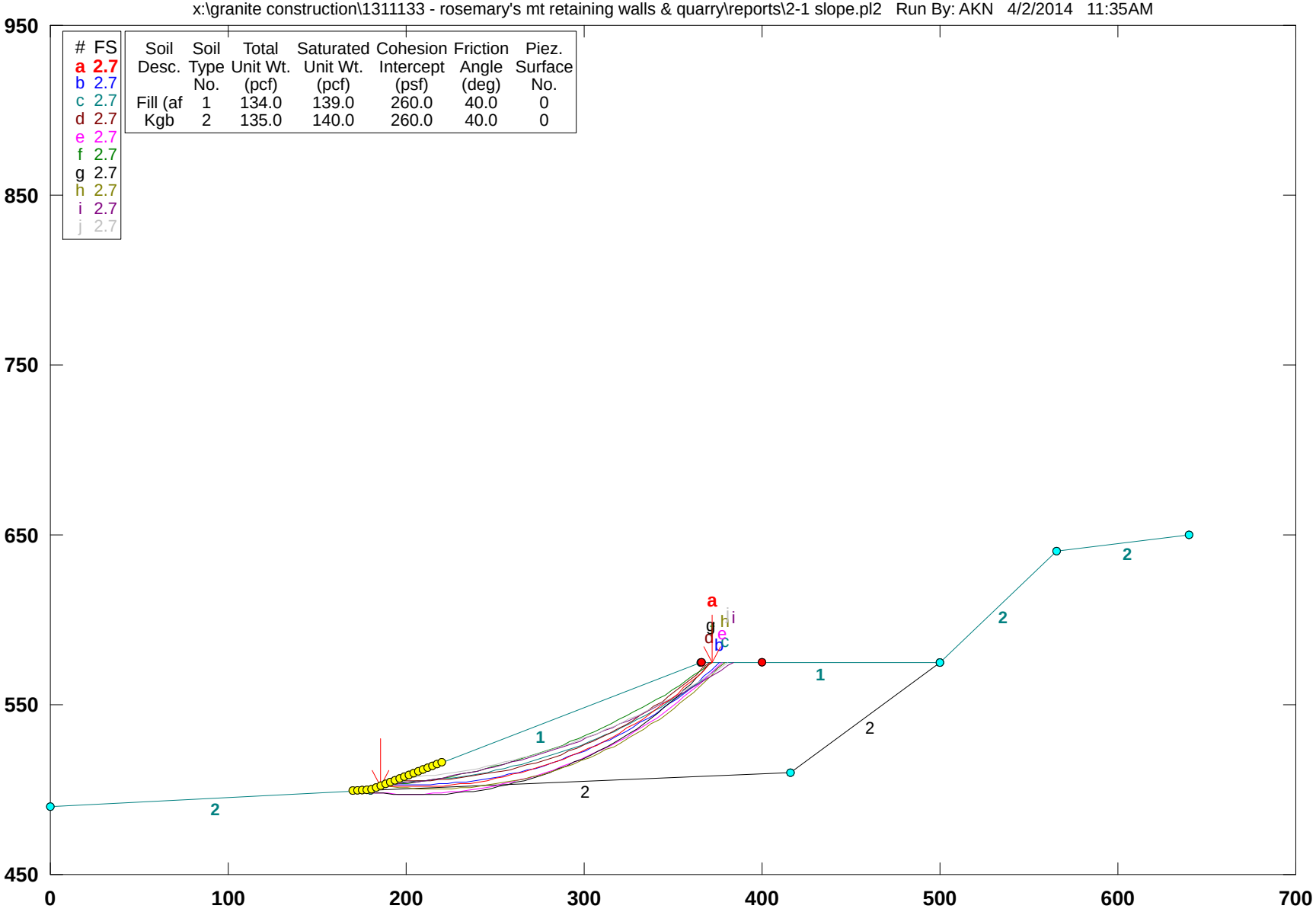


GSTABL7 v.2 FSmin=1.1

Safety Factors Are Calculated By The Modified Bishop Method

Rosemary's Mtn 2:1 Fill Slope 1311133-1R3

x:\granite construction\1311133 - rosemary's mt retaining walls & quarry\reports\2-1 slope.pl2 Run By: AKN 4/2/2014 11:35AM

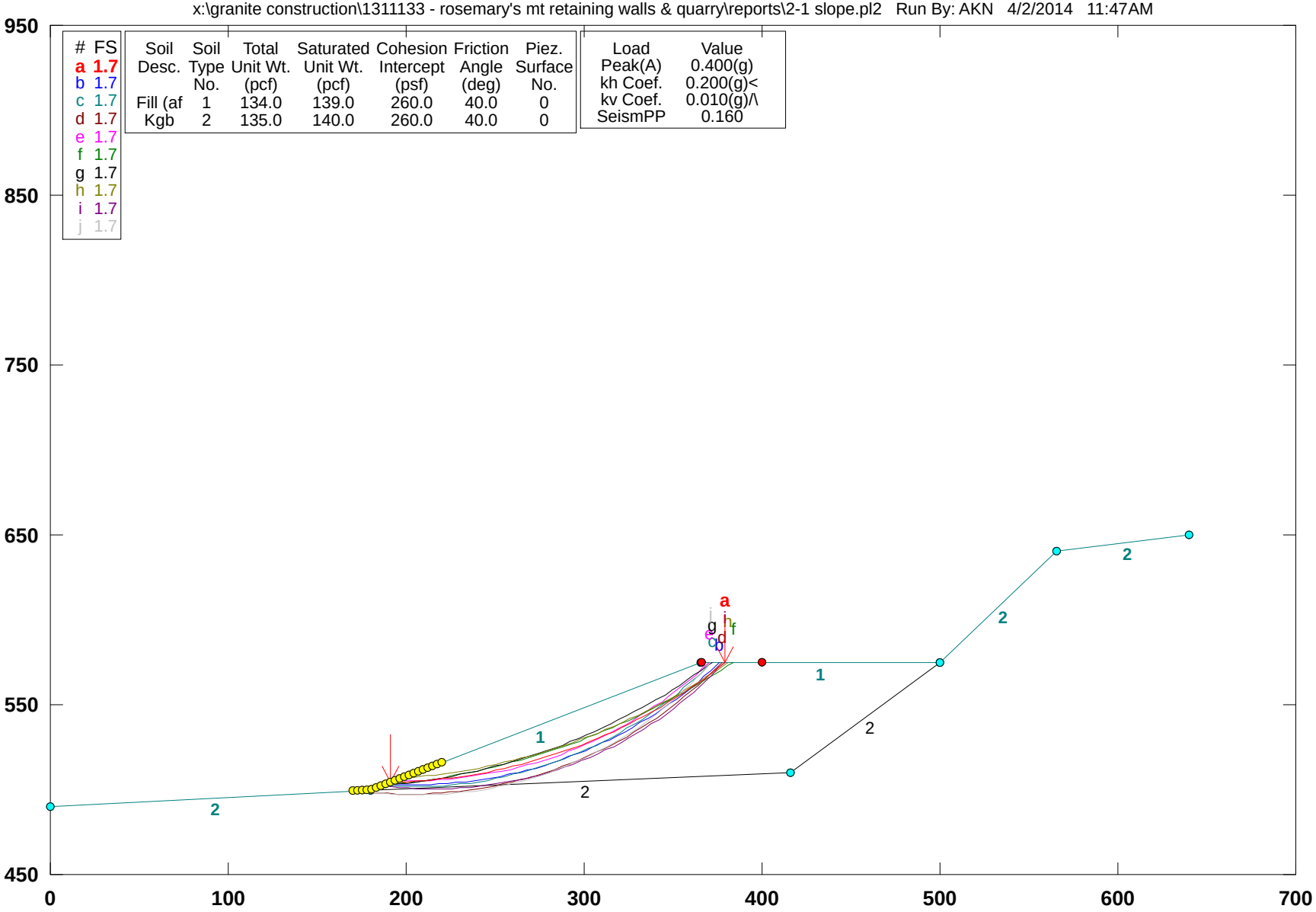


GSTABL7 v.2 FSmin=2.7

Safety Factors Are Calculated By The Modified Bishop Method

Rosemary's Mtn 2:1 Fill Slope 1311133-1R3

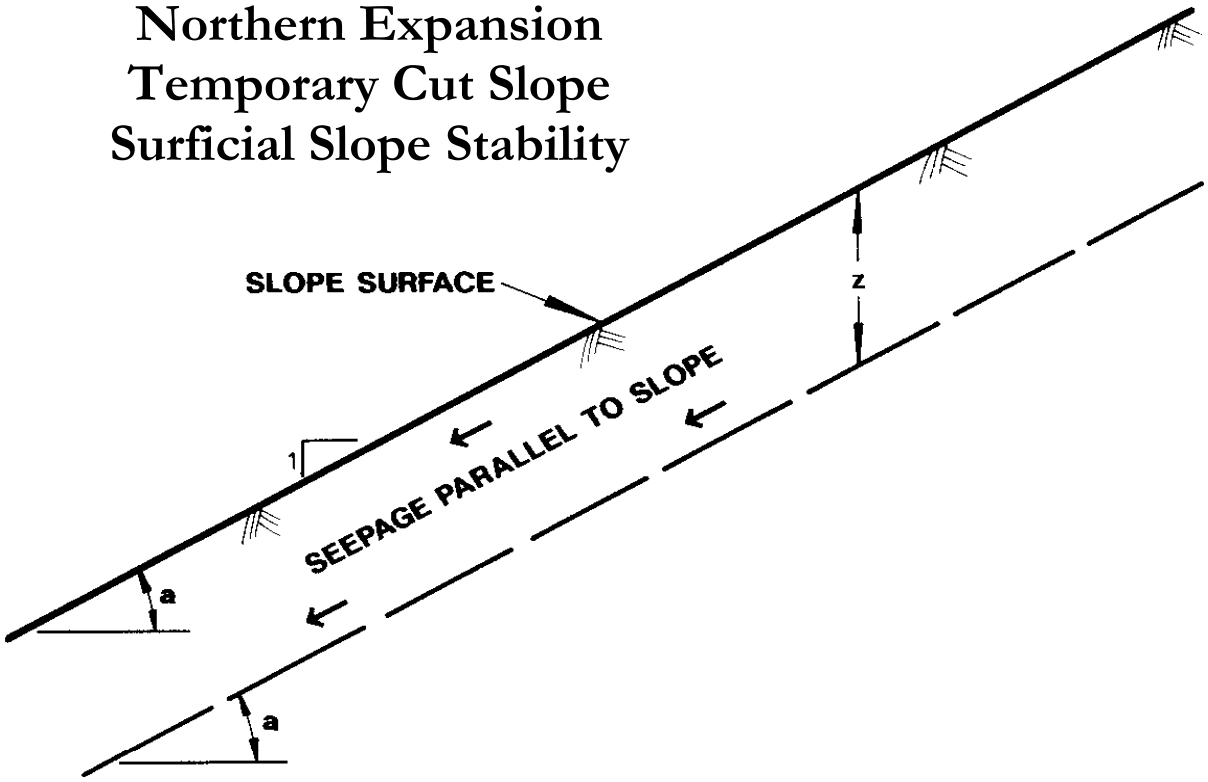
x:\granite construction\1311133 - rosemary's mt retaining walls & quarry\reports\2-1 slope.pl2 Run By: AKN 4/2/2014 11:47AM



GSTABL7 v.2 FSmin=1.7

Safety Factors Are Calculated By The Modified Bishop Method

Northern Expansion
Temporary Cut Slope
Surficial Slope Stability



ASSUMED PARAMETERS

z = DEPTH OF SATURATION = 2 Feet

a = SLOPE ANGLE = 45°

γ_w = UNIT WEIGHT OF WATER = 62.4 pounds per cubic foot

γ_T = SATURATED UNIT WEIGHT OF SOIL = 136 pounds per cubic foot

ϕ = APPARENT ANGLE OF INTERNAL FRICTION ALONG PLANE OF FAILURE = 45°

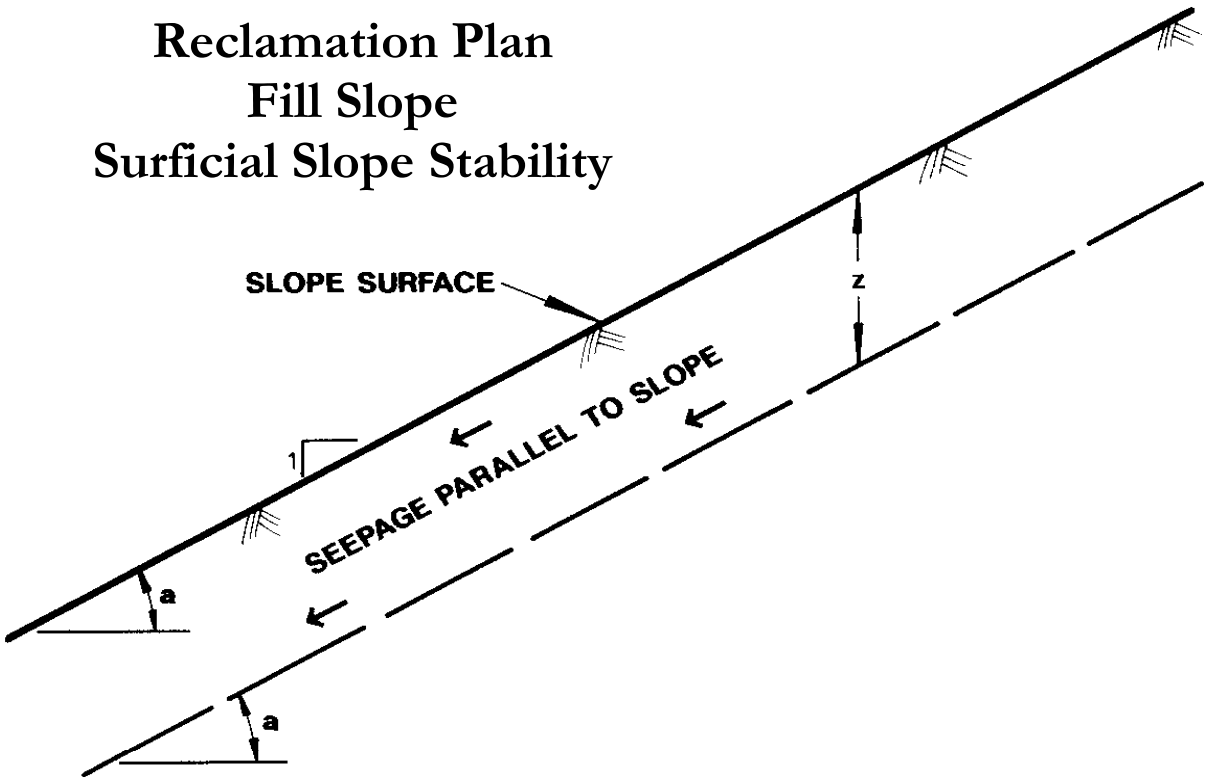
c = APPARENT COHESION ALONG PLANE OF FAILURE = 500 pounds per square foot

$$FS = \frac{c + T \tan \phi}{T} = \frac{c + (\gamma_T - \gamma_w)\, z \cos a \tan \phi}{\gamma_T\, z \sin a \cos a}$$

FS = 4.4

SC&ST	SOUTHERN CALIFORNIA SOIL & TESTING, INC.	ROSEMARY'S MOUNTAIN RETAINING WALL AND QUARRY EXPANSION		
		By:	DAS	Date: 4/2/2014
		Job Number:	1311133-3R	Figure: III-5

Reclamation Plan
Fill Slope
Surficial Slope Stability



ASSUMED PARAMETERS

z = DEPTH OF SATURATION = 2 Feet

a = SLOPE ANGLE = 26.5°

γ_w = UNIT WEIGHT OF WATER = 62.4 pounds per cubic foot

γ_T = SATURATED UNIT WEIGHT OF SOIL = 139 pounds per cubic foot

ϕ = APPARENT ANGLE OF INTERNAL FRICTION ALONG PLANE OF FAILURE = 30°

c = APPARENT COHESION ALONG PLANE OF FAILURE = 100 pounds per square foot

$$FS = \frac{c + T \tan \phi}{T} = \frac{c + (\gamma_T - \gamma_w)\, z \cos a \tan \phi}{\gamma_T\, z \sin a \cos a}$$

FS = 1.6

SC&ST	SOUTHERN CALIFORNIA SOIL & TESTING, INC.	ROSEMARY'S MOUNTAIN RETAINING WALL AND QUARRY EXPANSION		
		By:	DAS	Date: 4/2/2014
		Job Number:	1311133-3R	Figure: III-6