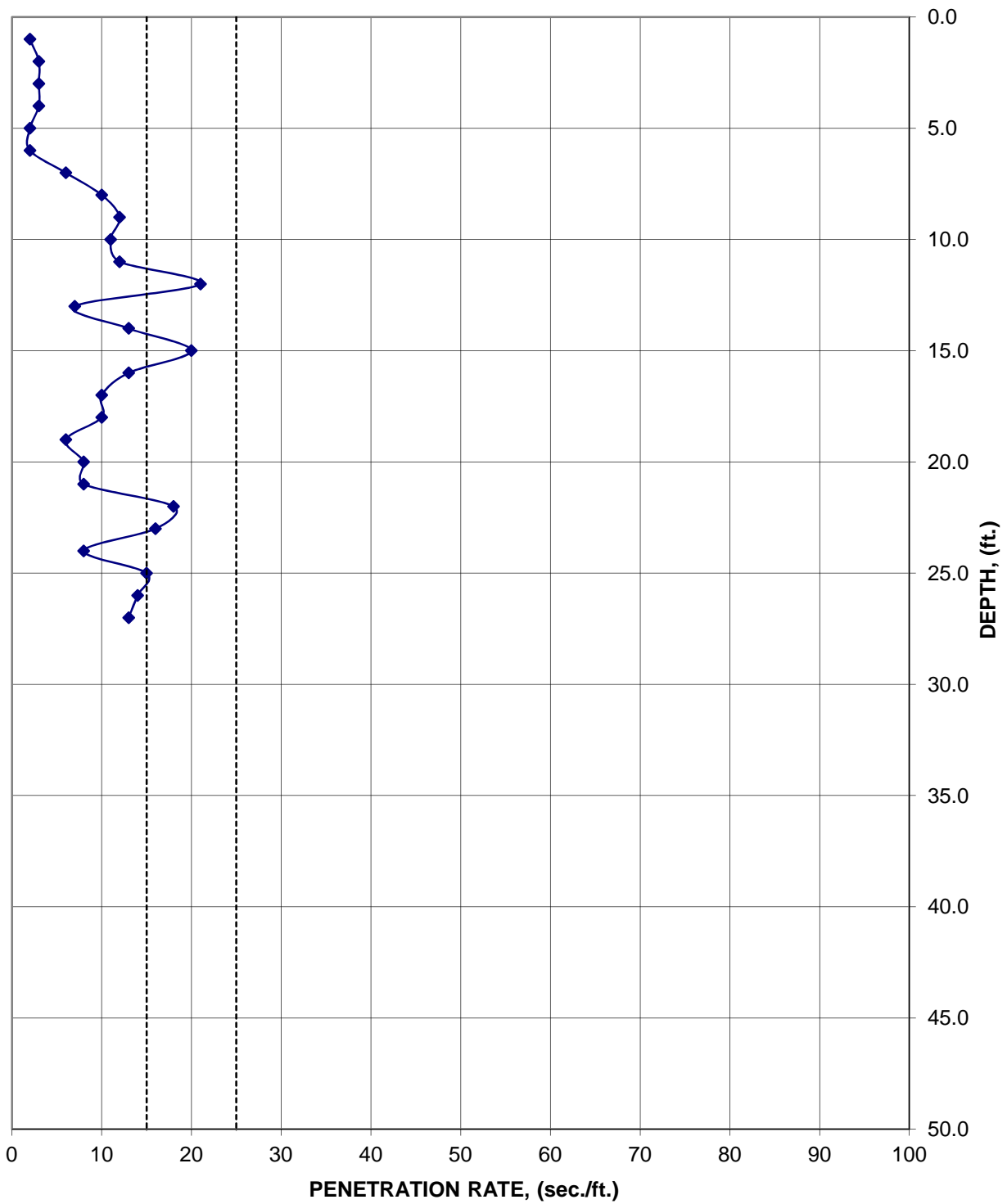


BORING AT-41



PERCUSSION DRILL PENETRATION RATE - Ingersol Rand ECM-370 w/4" Bit

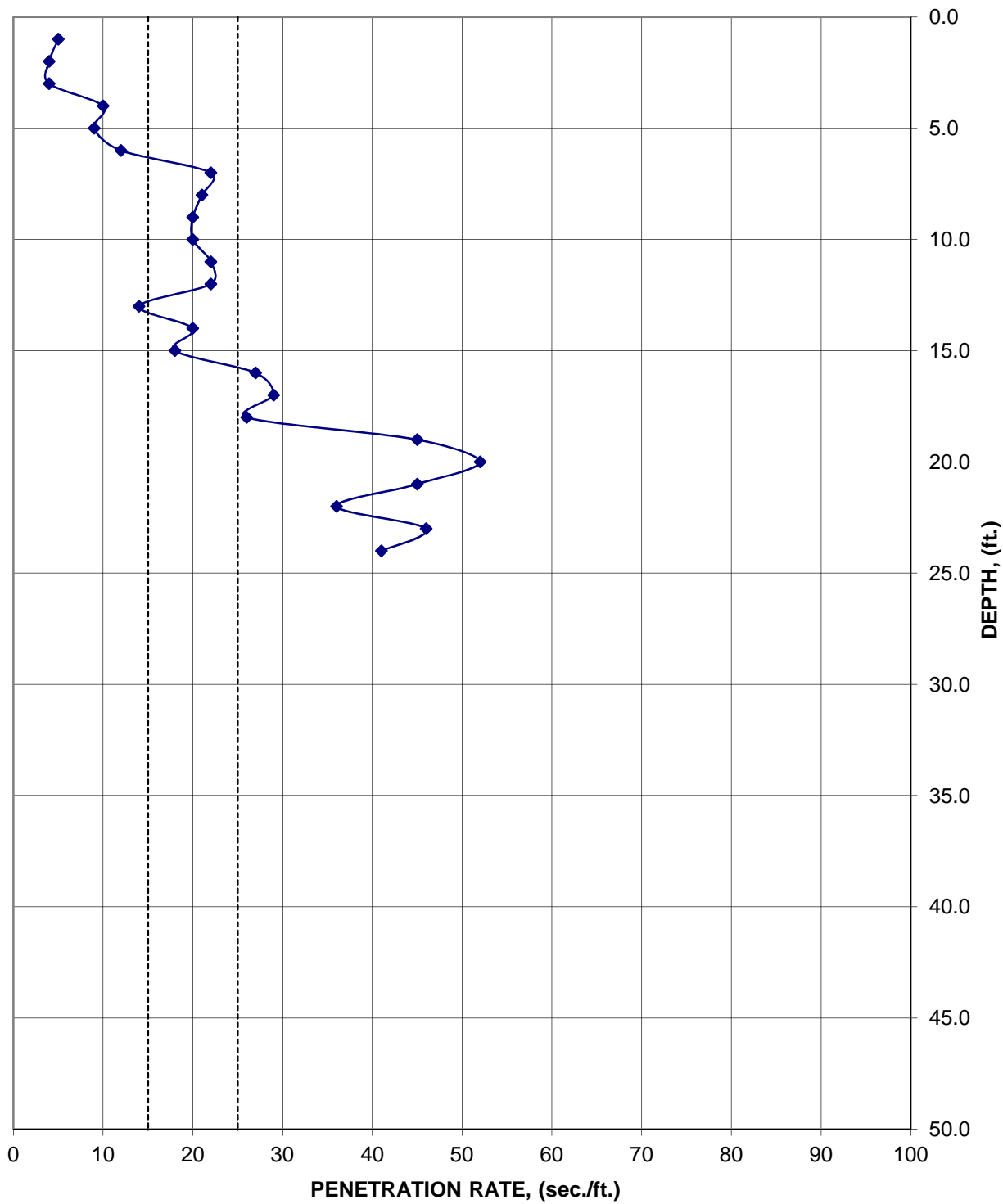
Project No.:
040084-013

Merriam Mountains

Figure No.:
BORING AT-41



BORING AT-42



PERCUSSION DRILL PENETRATION RATE - Ingersol Rand ECM-370 w/4" Bit

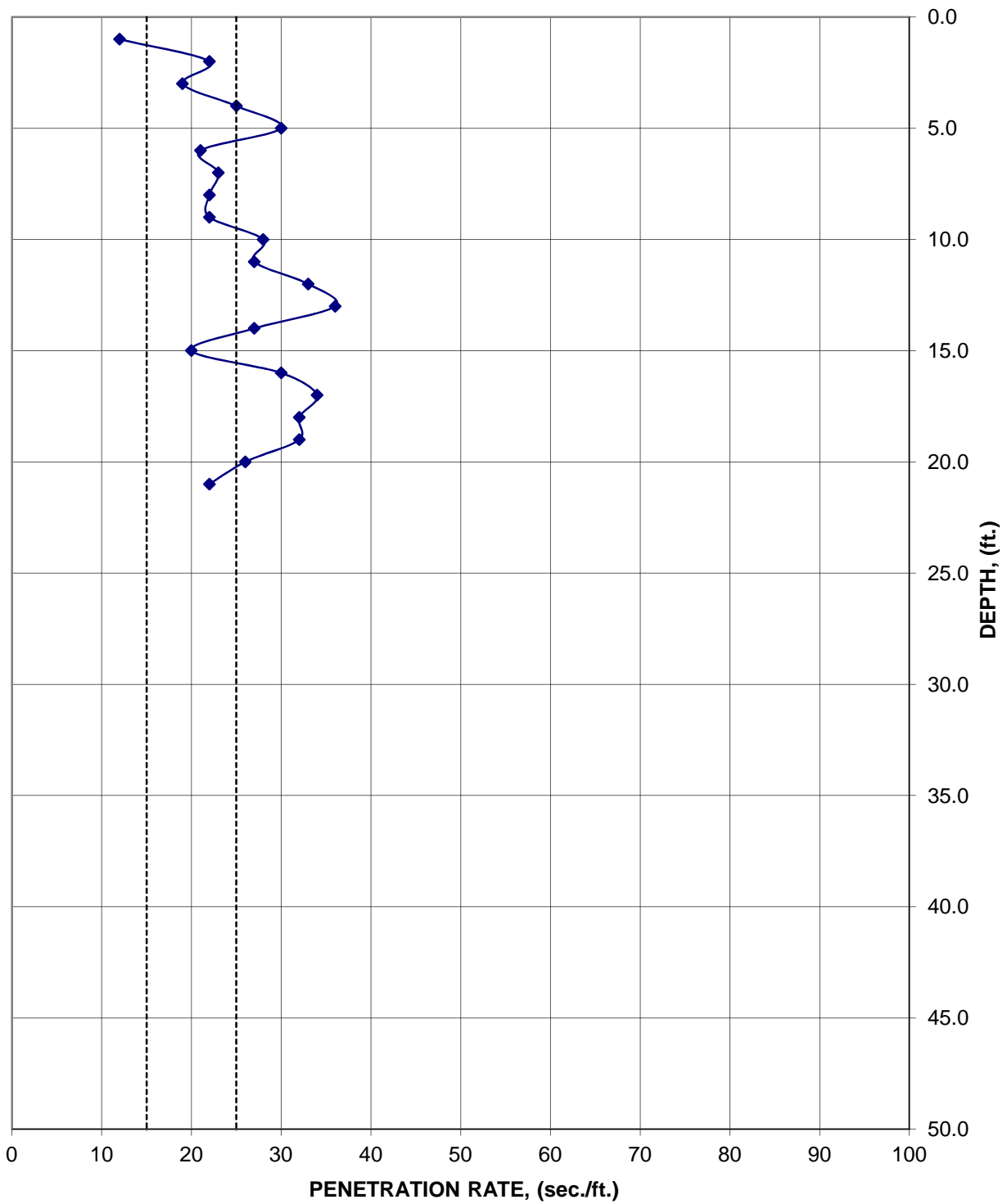
Project No.:
040084-013

Merriam Mountains

Figure No.:
BORING AT-42



BORING AT-43



PERCUSSION DRILL PENETRATION RATE - Ingersol Rand ECM-370 w/4" Bit

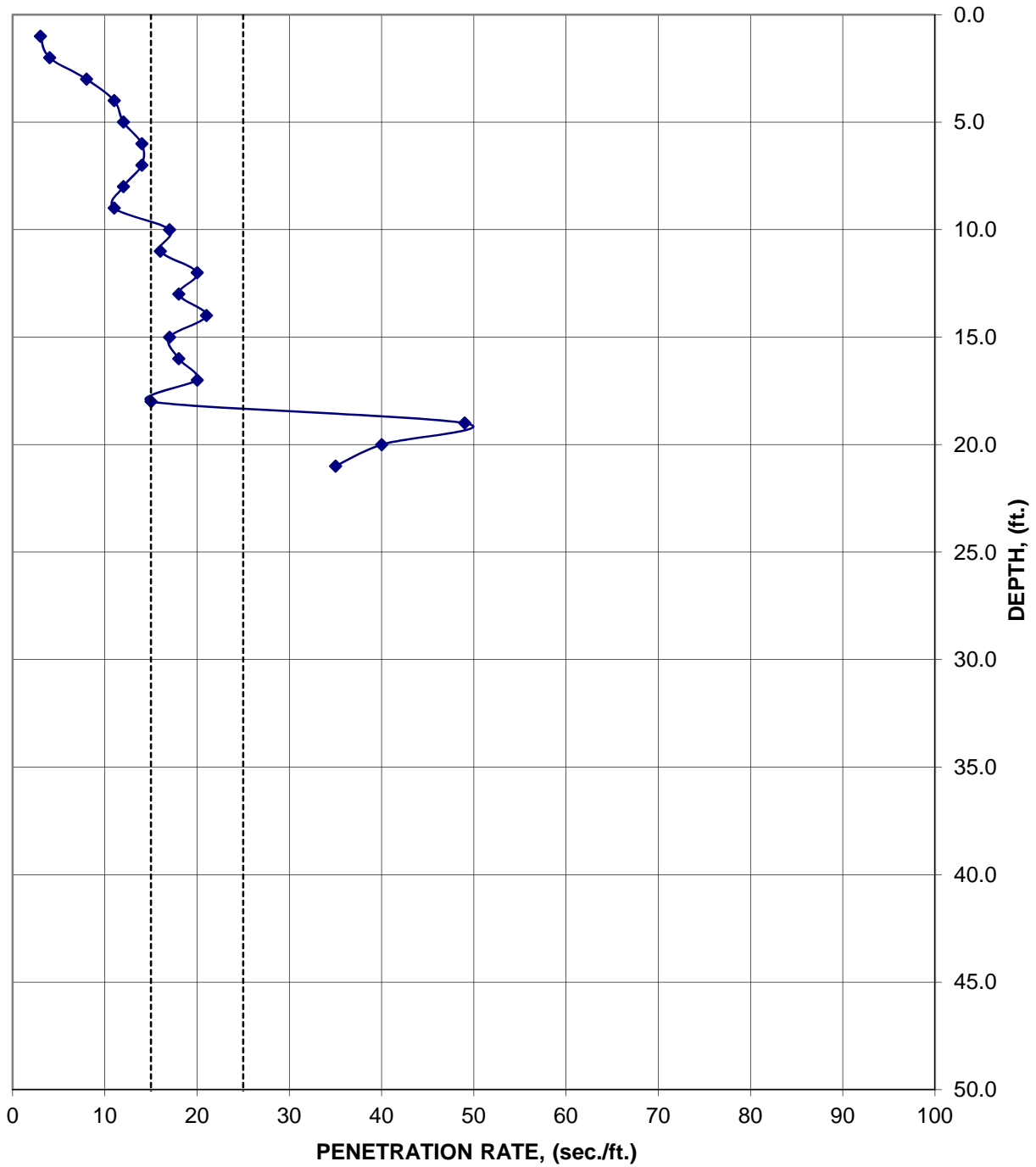
Project No.:
040084-013

Merriam Mountains

Figure No.:
BORING AT-43



BORING AT-44



PERCUSSION DRILL PENETRATION RATE - Ingersol Rand ECM-370 w/4" Bit

Project No.:
040084-013

Merriam Mountains

Figure No.:
BORING AT-44



APPENDIX B

FIELD PERCOLATION TEST DATA SHEET



Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	Newland Sierra	Project No.:	10618.002
Proj. Address:	Deer Springs Road , Ca		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type:	Brn silty sand		
Location:	P-1 (lower basin)		
Hole Dia:	4"		
Depth	6.0'		
Tested by:	SLR	Pre-Saturation Date:	6-11-15
		Test Date:	6-12-15
Notes: Measurements in 100ths of foot			

Time of Day	Interval / Notes	Water Level	Time of Day	Interval / Notes	Water Level
7:05		4.27	12:29	add water	4.4
8:05	60 min	5.18	12:59	30 min	4.68
8:06	add water	4.22			
8:36	30 min	4.68			
8:38	add water	4.4			
9:08	30 min	4.75			
9:09	add water	4.12			
9:42	33 min	4.6			
9:43	add water	4.05			
10:16	33 min	4.6			
10:17	add water	4.05			
10:52	35 min	4.52			
10:53	add water	4.1			
11:27	34 min	4.57			
11:28	add water	4.3			
12:28	60 min	4.85			

FOR OFFICE USE ONLY	DATE RECEIVED:	By:
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Notes: 8.9 min/inch or 6.7 inch/hour



Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	Newland Sierra	Project No.:	10618.002
Proj. Address:	Deer Springs Road , Ca		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type:	Brn silty sand		
Location:	P-2 (lower basin)		
Hole Dia:	4"		
Depth	6.9'		
Tested by:	SLR	Pre-Saturation Date:	6-11-15
		Test Date:	6-12-15
Notes: Measurements in 100ths of foot			

Time of Day	Interval / Notes	Water Level	Time of Day	Interval / Notes	Water Level
7:07		4.5			
8:07	60 min	5.01			
8:10	add water	4.46			
8:40	30 min	4.78			
8:43	add water	4.15			
9:13	30 min	4.5			
9:45	32 min	4.8			
9:46	add water	4.22			
10:19	33 min	4.6			
10:20	add water	4.04			
10:55	35 min	4.58			
11:33	38 min	4.95			
11:33	add water	4.61			
12:33	60 min	4.96			
12:34	add water	4.61			
1:04	30 min	4.78			

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Notes: 14.7 min/inch or 4.08 inch/hour



Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	Newland Sierra	Project No.:	10618.002
Proj. Address:	Deer Springs Road , Ca		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type:	Brn silty sand		
Location:	P-3 (upper basin)		
Hole Dia:	4"		
Depth	6.8'		
Tested by:	SLR	Pre-Saturation Date:	6-11-15
		Test Date:	6-12-15
Notes: Measurements in 100ths of foot			

Time of Day	Interval / Notes	Water Level	Time of Day	Interval / Notes	Water Level
7:35		4.65	12:42	37 min	4.55
8:19	44 min	5.3			
8:20	add water	4.22			
8:54	34 min	4.9			
8:55	add water	4.43			
9:29	34 min	4.92			
9:30	add water	4.45			
10:00	30 min	4.92			
10:02	add water	4.4			
10:32	30 min	4.9			
10:34	add water	3.95			
11:04	30 min	4.49			
11:34	30 min	5			
11:34	add water	4.5			
12:04	30 min	4.98			
12:05	add water	4.05			

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Notes: 6.2 min/inch or 9.7 inch/hour



Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	Newland Sierra	Project No.:	10618.002
Proj. Address:	Deer Springs Road , Ca		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type:	Brn silty sand		
Location:	P-4 (upper basin)		
Hole Dia:	4"		
Depth	6.7'		
Tested by:	SLR	Pre-Saturation Date:	6-11-15
		Test Date:	6-12-15
Notes: Measurements in 100ths of foot			

Time of Day	Interval / Notes	Water Level	Time of Day	Interval / Notes	Water Level
7:38		4.88	12:10	add water	4.7
8:23	45 min	5.41	12:43	33 min	5.3
8:24	add water	5			
8:58	34 min	5.4			
8:59	add water	5.05			
9:32	33 min	5.41			
9:33	add water	4.95			
10:03	30 min	5.31			
10:04	add water	4.9			
10:39	35 min	5.27			
10:40	add water	4.77			
11:10	30 min	5.15			
11:10	add water	4.81			
11:40	30 min	5.18			
11:40	add water	4.8			
12:10	30 min	5.11			

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Notes: 4.6 min/inch or 13 inch/hour

APPENDIX C
LABORATORY TEST RESULTS

APPENDIX C

Summary of Laboratory Test Results

Laboratory Maximum Dry Density and Optimum Moisture Content (ASTM D1557)

The maximum dry density and optimum moisture content of typical materials was determined in accordance with ASTM Test Method D1557. The results of these tests are presented in the table below.

Sample Location	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry weight)
T43, T44 @ 2-8 feet	Brown silty SAND with gravel	138.5	6.5
T46, T47, T48 @ 2-8 feet	Brown silty SAND with gravel	130.0	10.0

Moisture and Density Determination Tests: Moisture content and dry density determinations were performed on relatively undisturbed samples obtained from the test borings and/or trenches. The results of these tests are presented in the boring and/or trench logs. Where applicable, only moisture content was determined from "undisturbed" or disturbed samples.

Direct Shear Test Results (ASTM D3080)

Direct shear tests were performed on remolded samples that were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. The samples were tested under various normal loads, a motor-driven, strain-controlled, direct-shear testing apparatus. The rates of shearing used for the tests was 0.05 in/min. The test results are presented in the table below. The peak shear resistance was recorded to determine the peak strength parameters. The ultimate strength parameters were determined at the inflection point of the shear stress versus Horizontal Deformation plot. We have also reported the shear strength at 0.3 inches deformation on the accompany plot. Plots of the data presented in the table are also included in this appendix.

Location	Sample Depth (feet)	Peak Friction Angle (degrees)	Peak Apparent Cohesion (psf)	Ultimate Friction Angle (degrees)	Ultimate Apparent Cohesion (psf)
T43 & T44	2-8	47	700	46	400

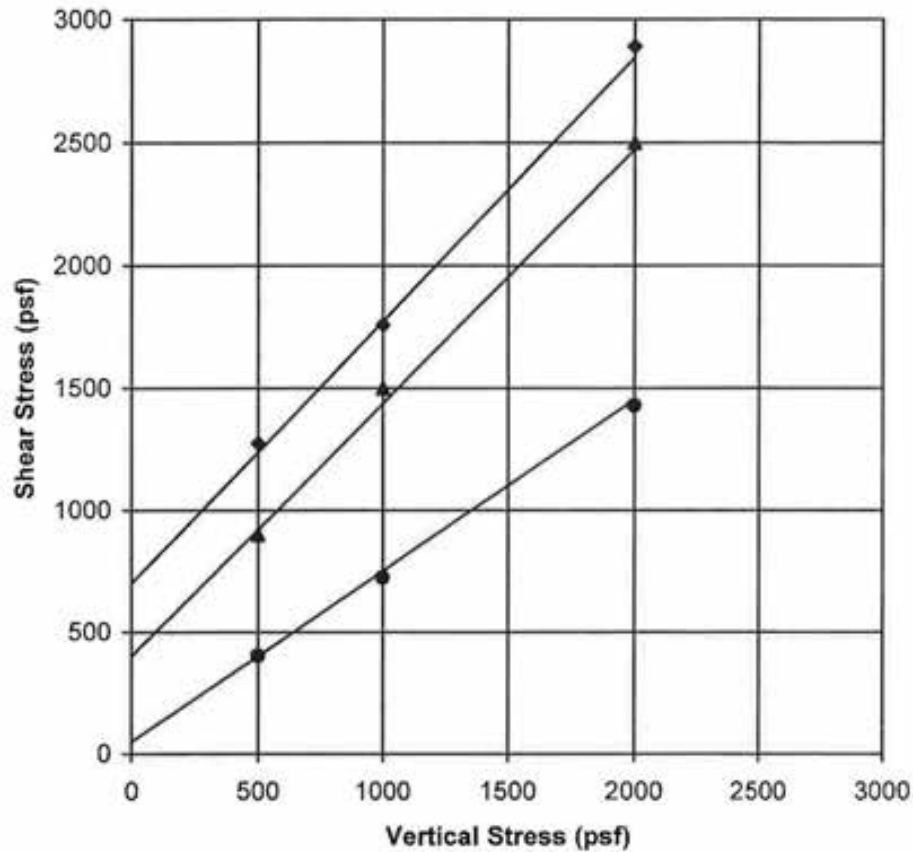
Consolidated Undrained Triaxial Test (ASTM D4767)

The consolidated undrained triaxial compression test for the remolded sample was performed in accordance with the ASTM D4767. The remolded sample was moisture conditioned and compacted in a split mold. The samples were then placed in the testing device and a small seating load was applied, to secure the sample in the testing device. The samples were saturated by the applying a back pressure. The axial load and chamber pressure were increased in small increments until the change in chamber pressure was within tolerance to the measured change in sample pore fluid pressure, indicating that the sample was fully saturated. Once the sample was fully saturated and had completed primary consolidation, the samples were loaded axially at a rate of 0.02 in/min for the remolded samples. The table below reports the peak total strength values for the samples obtained from the triaxial compression testing.

Boring Location	Sample Depth (feet)	Shear Type	Friction Angle (degrees)	Apparent Cohesion (psf)
T43 & T44	2-8	Peak*	28	2300
T46, T47, T48	2-8	Peak*	13	2450

*Remolded to approximately 90 percent of the maximum dry density at 2% above optimum moisture content.

Hydro-consolidation (Collapse-swell) Tests: Hydro-consolidation tests were performed on selected relatively undisturbed bulk samples and subsequently carved into ring samples. Samples were placed in a consolidometer and a load approximately equal to the in-situ overburden pressure was applied. Water was then added to the sample and the percent hydro-consolidation for the load cycle was recorded as the ratio of the amount of vertical compression to the original 1-inch height. The percent hydro-consolidation is presented on the attached figures.



Boring Location T43 & T44 Deformation Rate 0.05 in/min

Sample Depth (feet) 2-8

Sample Description Brown silty SAND with gravel @ 90%

Average Strength Parameters

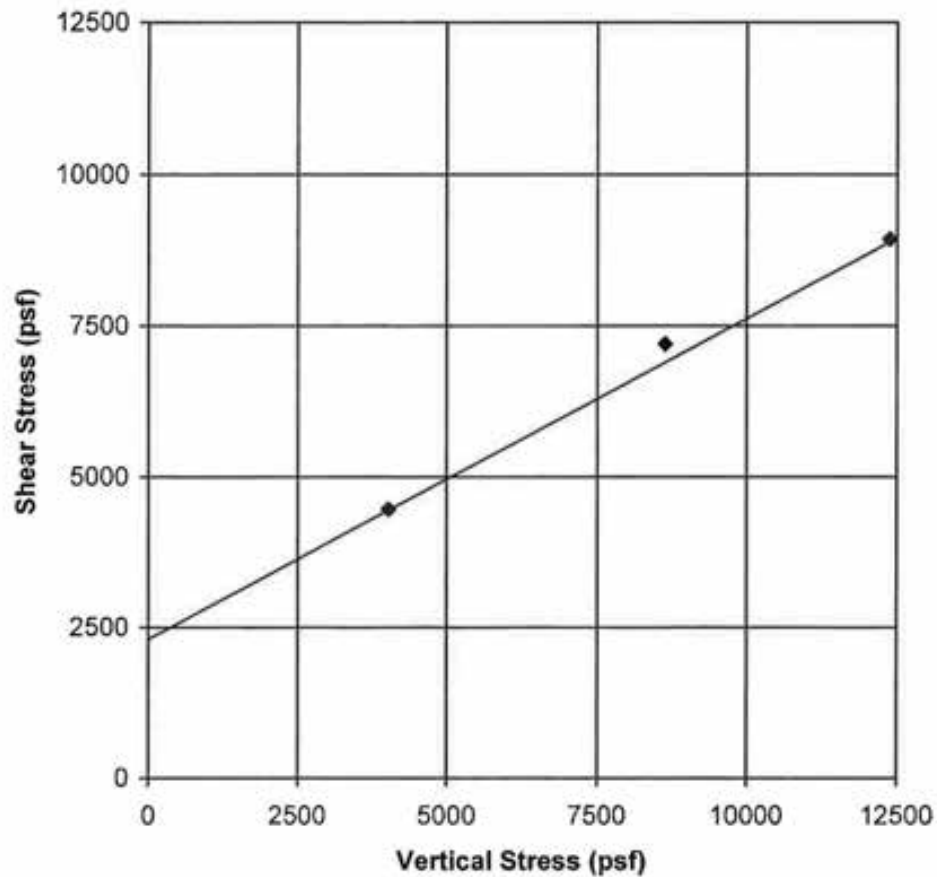
Peak	Friction Angle, ϕ'_{peak} (deg)	<u>47</u>	@ 0.3"	Friction Angle, $\phi'_{@ 0.3"}$ (deg)	<u>35</u>
	Cohesion, c'_{peak} (psf)	<u>700</u>		Cohesion, $c'_{@ 0.3"}$ (psf)	<u>50</u>
Ultimate	Friction Angle, ϕ'_{ult} (deg)	<u>46</u>			
	Cohesion, c'_{ult} (psf)	<u>400</u>			

DIRECT SHEAR SUMMARY

Project No.
Project Name

040084-004
Merriam Mountain





Boring Location T43 & T44

Sample Depth (feet) 2-8

Sample Description Brown silty SAND with gravel @ 90%

Total Strength Parameters

Friction Angle, ϕ (deg) 28

Cohesion, c (psf) 2300

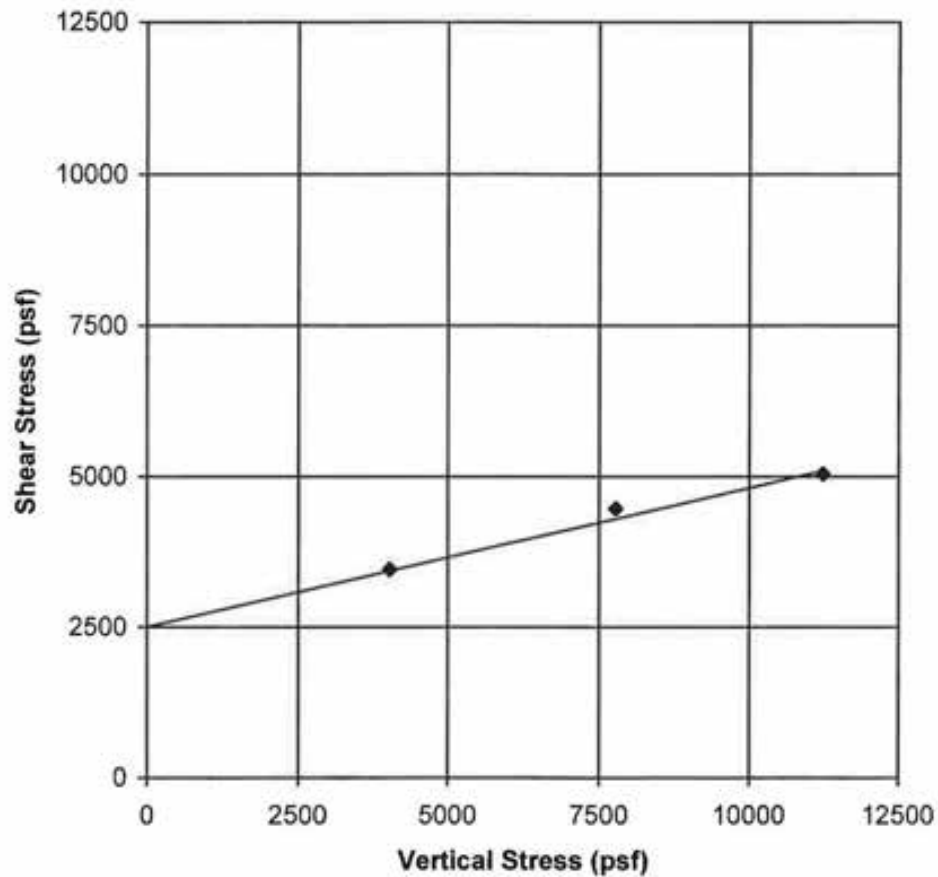
TRIAXIAL SHEAR

Project No. 040084-003

Project Name Merriam Mountain



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Boring Location T46, T47, & T48

Sample Depth (feet) 3-8

Sample Description Brown silty SAND with gravel @ 90%

Total Strength Parameters

Friction Angle, ϕ (deg) 13

Cohesion, c (psf) 2500

TRIAXIAL SHEAR

Project No. 040084-003
Project Name Merriam Mountain



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One-Dimensional Swell or Settlement Potential of Cohesive Soils (ASTM D 4546)

Project Name: MERRIAM MOUNTAIN

Project No.: 040084-013

Boring No.: BH-2

Sample No.: R-3

Sample Description: (SW-SM), BROWN WELL GRADED SAND WITH SILT.

Tested By: JAP/VRO

Checked By: JMB

Sample Type: IN SITU

Depth (ft.) 20.0

Date: 11/1/07

Date: 11/2/07

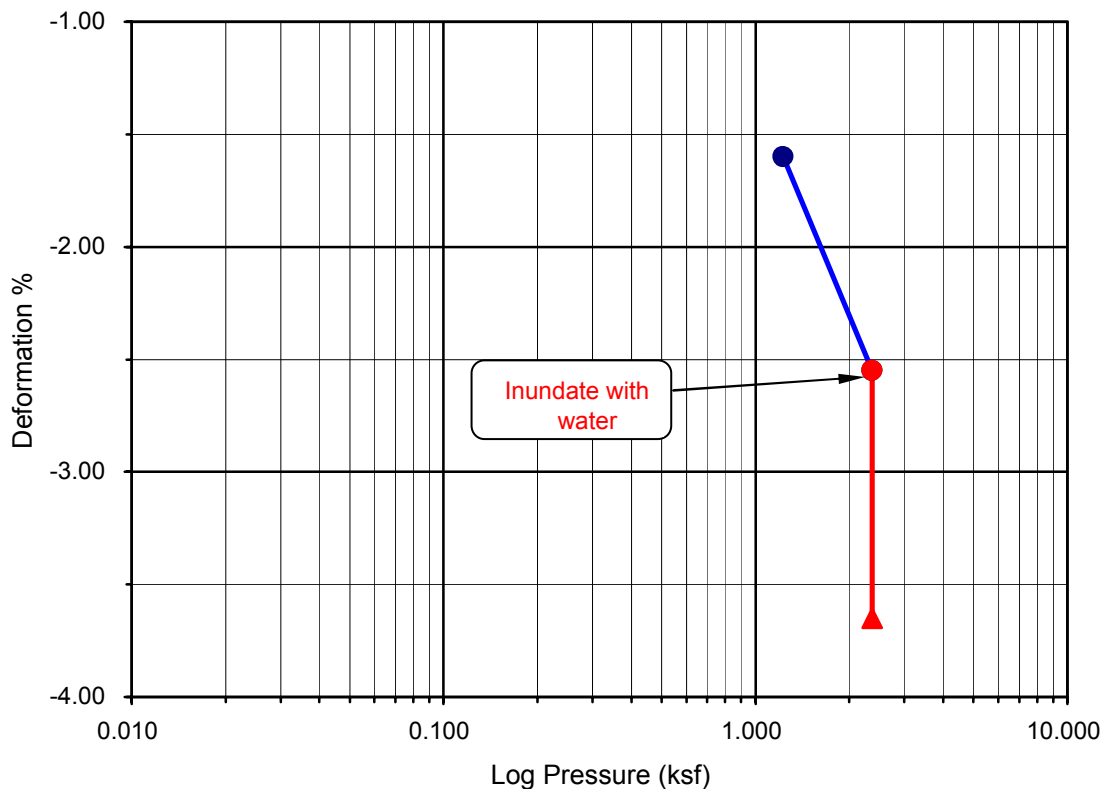
Initial Dry Density (pcf):	107.5
Initial Moisture (%):	5.4
Initial Length (in.):	1.0000
Initial Dial Reading:	0.0500
Diameter(in):	2.416

Final Dry Density (pcf):	111.6
Final Moisture (%) :	16.2
Initial Void ratio:	0.5676
Specific Gravity(assumed):	2.70
Initial Saturation (%)	25.5

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
1.225	0.0660	0.9840	0.00	-1.60	0.5425	-1.60
2.363	0.0755	0.9745	0.00	-2.55	0.5276	-2.55
H2O	0.0865	0.9635	0.00	-3.65	0.5103	-3.65

Percent Swell / Settlement After Inundation = **-1.13**

Deformation % - Log Pressure Curve



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One-Dimensional Swell or Settlement Potential of Cohesive Soils (ASTM D 4546)

Project Name: MERRIAM MOUNTAIN

Project No.: 040084-013

Boring No.: BH-2

Sample No.: R-2

Sample Description: (SW-SM), BROWN WELL GRADED SAND WITH SILT.

Tested By: JAP/VRO

Checked By: JMB

Sample Type: IN SITU

Depth (ft.) 10.0

Date: 11/1/07

Date: 11/2/07

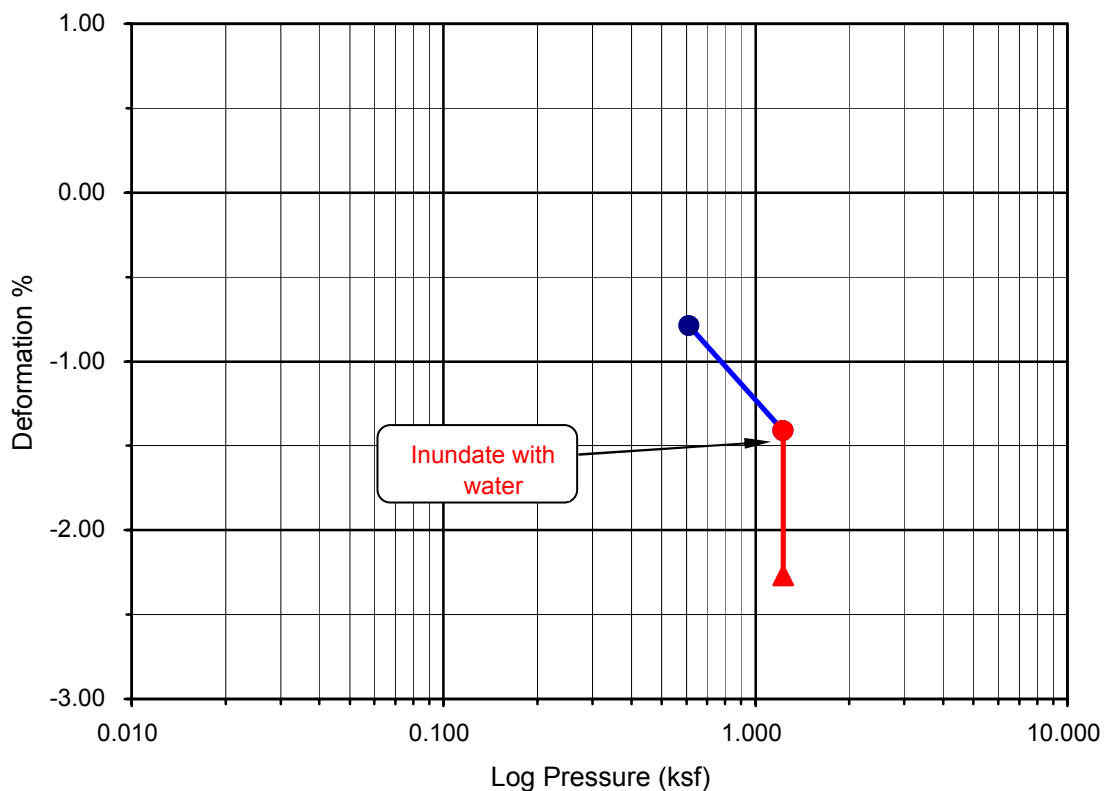
Initial Dry Density (pcf):	112.2
Initial Moisture (%):	5.1
Initial Length (in.):	1.0000
Initial Dial Reading:	0.0500
Diameter(in):	2.416

Final Dry Density (pcf):	114.8
Final Moisture (%) :	14.6
Initial Void ratio:	0.5025
Specific Gravity(assumed):	2.70
Initial Saturation (%)	27.5

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.613	0.0579	0.9921	0.00	-0.79	0.4907	-0.79
1.226	0.0641	0.9859	0.00	-1.41	0.4813	-1.41
H2O	0.0727	0.9773	0.00	-2.27	0.4684	-2.27

Percent Swell / Settlement After Inundation = **-0.87**

Deformation % - Log Pressure Curve



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One-Dimensional Swell or Settlement Potential of Cohesive Soils (ASTM D 4546)

Project Name: MERRIAM MOUNTAIN Tested By: JAP/VRO Date: 11/1/07
 Project No.: 040084-013 Checked By: JMB Date: 11/2/07
 Boring No.: BH-2 Sample Type: IN SITU
 Sample No.: R-1 Depth (ft.) 2.5
 Sample Description: (SC-SM), DARK BROWN SILTY, CLAYEY SAND. *** NOTE: 2 VOIDS PRESENT IN SAMPLE --- ~ 7% OF VOLUM

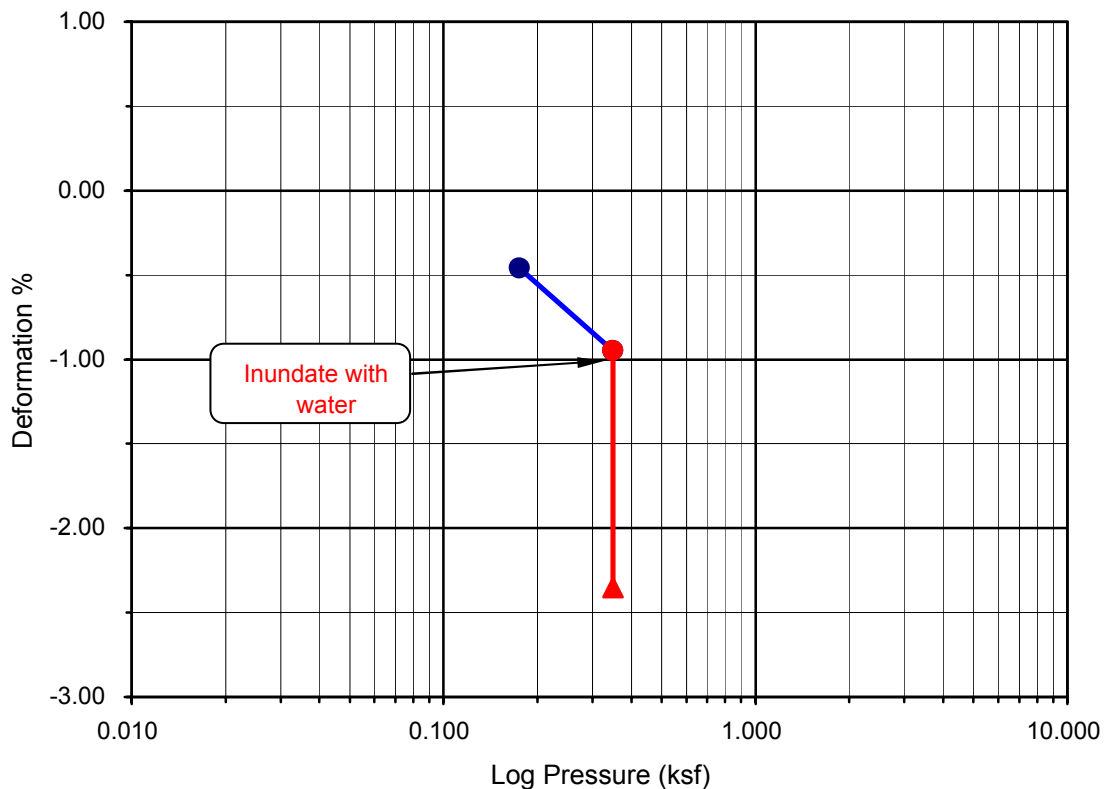
Initial Dry Density (pcf):	117.8
Initial Moisture (%):	6.4
Initial Length (in.):	1.0000
Initial Dial Reading:	0.0500
Diameter(in):	2.416

Final Dry Density (pcf):	120.6
Final Moisture (%) :	13.7
Initial Void ratio:	0.4315
Specific Gravity(assumed):	2.70
Initial Saturation (%)	39.7

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.175	0.0546	0.9954	0.00	-0.46	0.4249	-0.46
0.350	0.0595	0.9905	0.00	-0.95	0.4179	-0.95
H2O	0.0735	0.9765	0.00	-2.35	0.3978	-2.35

Percent Swell / Settlement After Inundation = **-1.41**

Deformation % - Log Pressure Curve



Rev. 08-04



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One-Dimensional Swell or Settlement Potential of Cohesive Soils (ASTM D 4546)

Project Name: MERRIAM MOUNTAIN
Project No.: 040084-013
Boring No.: BH-1
Sample No.: R-2
Sample Description: SM, LIGHT BROWN SILTY SAND.

Tested By: JAP/VRO Date: 11/1/07
Checked By: JMB Date: 11/2/07
Sample Type: IN SITU
Depth (ft.) 10.0

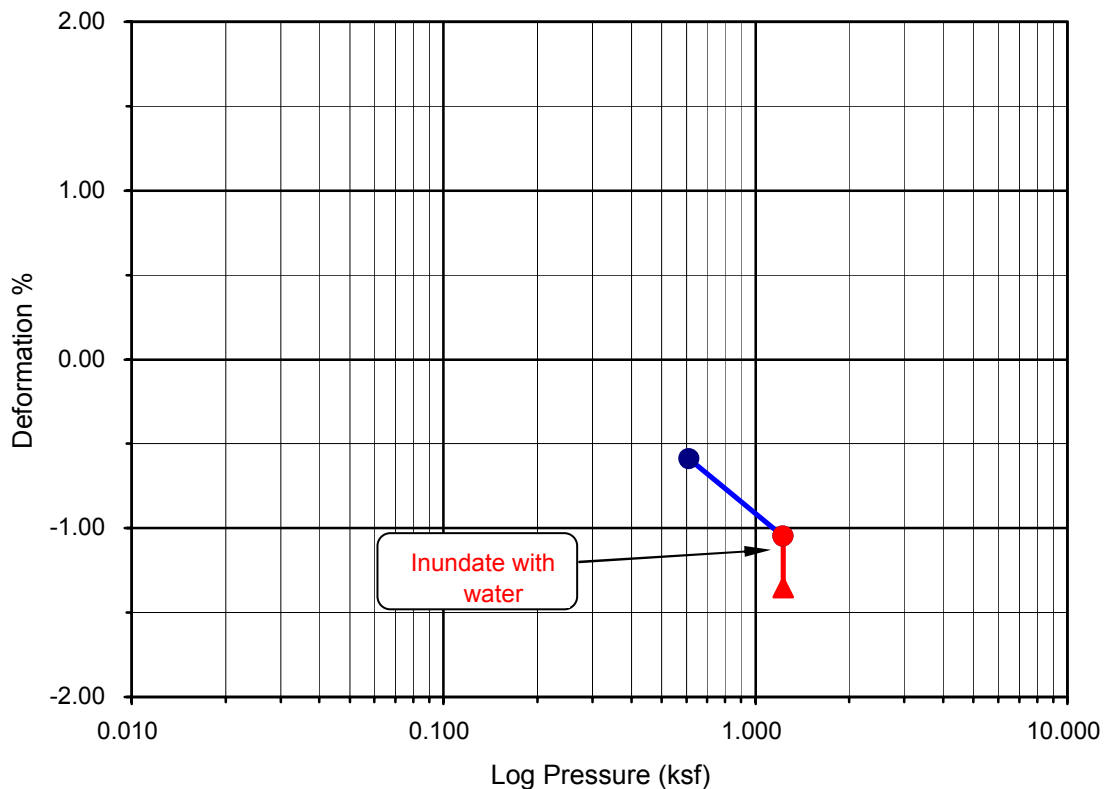
Initial Dry Density (pcf):	118.6
Initial Moisture (%):	8.5
Initial Length (in.):	1.0000
Initial Dial Reading:	0.0500
Diameter(in):	2.416

Final Dry Density (pcf):	120.2
Final Moisture (%) :	12.2
Initial Void ratio:	0.4215
Specific Gravity(assumed):	2.70
Initial Saturation (%)	54.8

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.613	0.0559	0.9941	0.00	-0.59	0.4131	-0.59
1.226	0.0605	0.9895	0.00	-1.05	0.4065	-1.05
H2O	0.0635	0.9865	0.00	-1.35	0.4023	-1.35

Percent Swell / Settlement After Inundation = **-0.30**

Deformation % - Log Pressure Curve



Rev. 08-04



Leighton

One-Dimensional Swell or Settlement Potential of Cohesive Soils (ASTM D 4546)

Project Name: MERRIAM MOUNTAIN
Project No.: 040084-013
Boring No.: BH-3
Sample No.: R-2
Sample Description: SM, REDDISH BROWN SILTY SAND.

Tested By: JAP/VRO Date: 11/1/07
Checked By: JMB Date: 11/2/07
Sample Type: IN SITU
Depth (ft.) 10.0

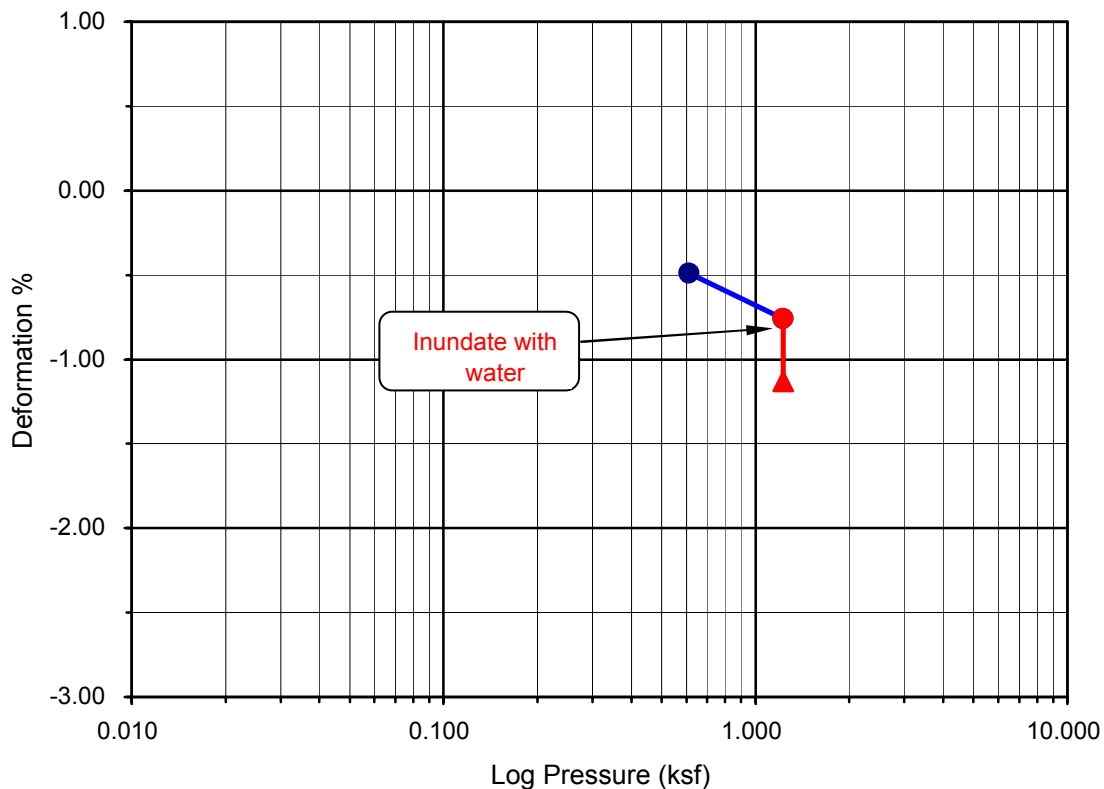
Initial Dry Density (pcf):	121.7
Initial Moisture (%):	7.1
Initial Length (in.):	1.0000
Initial Dial Reading:	0.0500
Diameter(in):	2.416

Final Dry Density (pcf):	123.1
Final Moisture (%) :	11.7
Initial Void ratio:	0.3846
Specific Gravity(assumed):	2.70
Initial Saturation (%)	50.1

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.613	0.0549	0.9951	0.00	-0.49	0.3778	-0.49
1.226	0.0576	0.9924	0.00	-0.76	0.3741	-0.76
H2O	0.0613	0.9887	0.00	-1.13	0.3689	-1.13

Percent Swell / Settlement After Inundation = **-0.37**

Deformation % - Log Pressure Curve



Rev. 08-04

APPENDIX D
SEISMIC REFRACTION STUDY



215 So. Highway 101, Suite 203 ♦ Solana Beach, CA 92075
Phone (619)481-8949 ♦ Fax (619)481-899
geop@subsurfacesurveys.com

September 20, 2002

Leighton & Associates, Inc.
3934 Murphy Canyon Rd, Suite B205
San Diego, CA 92123

Project Number: 02-387

Attn: **Brian Olson** re: Seismic refraction survey, Merriam Mtn/Stonegate, SD County

This brief letter report is to present the findings of a seismic refraction survey conducted in the hills across I-15 from Lawrence Welk Resort, north of Escondido, in San Diego County, California (Fig. 1) on September 9-10, 2002. The survey was carried out in the hills where residential structures are planned. Twelve lines were shot in places where bedrock is observed to be shallow or deep cut slopes are planned. Measured from off end shot to far offset geophone, all lines were 250 feet long. Purpose of the survey was to determine depth to bedrock, weathered and unweathered, and its rippability. These values were determined utilizing seismic refraction methodology.

A Bison 9024, 24 channel, seismograph system was applied to the task. This instrument has DIFP, digital instantaneous floating point. This translates into a computer-controlled seismograph that records incoming signals at all instrument settings, and these are analyzed in virtual real time by the computer, which then automatically selects the most appropriate settings and outputs optimum, balanced traces with maximum informational content.

Survey Design - Locations of the twelve single spread lines, numbered 1 through 12, are illustrated (Fig. 2). It is seen that the lines are variously laid out on ridges, noses, saddles and generally at locations where bedrock appears to be shallow or deep cut slopes are planned. Line lengths were 250 feet, and geophone interval was 10 feet. There was a 10 foot offset from off end shots to nearest geophone for the lines. There was also a 20 foot gap between geophones 12 and 13 where the mid split spread shot was fired. Asymmetrical split spread shots, between geophones 6 and 7, and 18 and 19, were also fired (hammer blows). This spread length allows for an investigation to depths of approximately 75 feet. The spreads were shot forward, split spread and reverse. This redundancy aids in determining dip and undulations in layer boundaries.

Source was a heavy-duty sledge hammer equipped with an inertial trigger. The accelerated weight drop source was available but was not needed. Vertical stacking was carried out as a noise abatement strategy, and to build energy. Elevations for all shot and geophone locations were surveyed in, as relative elevations, arbitrarily setting the forward end at 100 feet. These

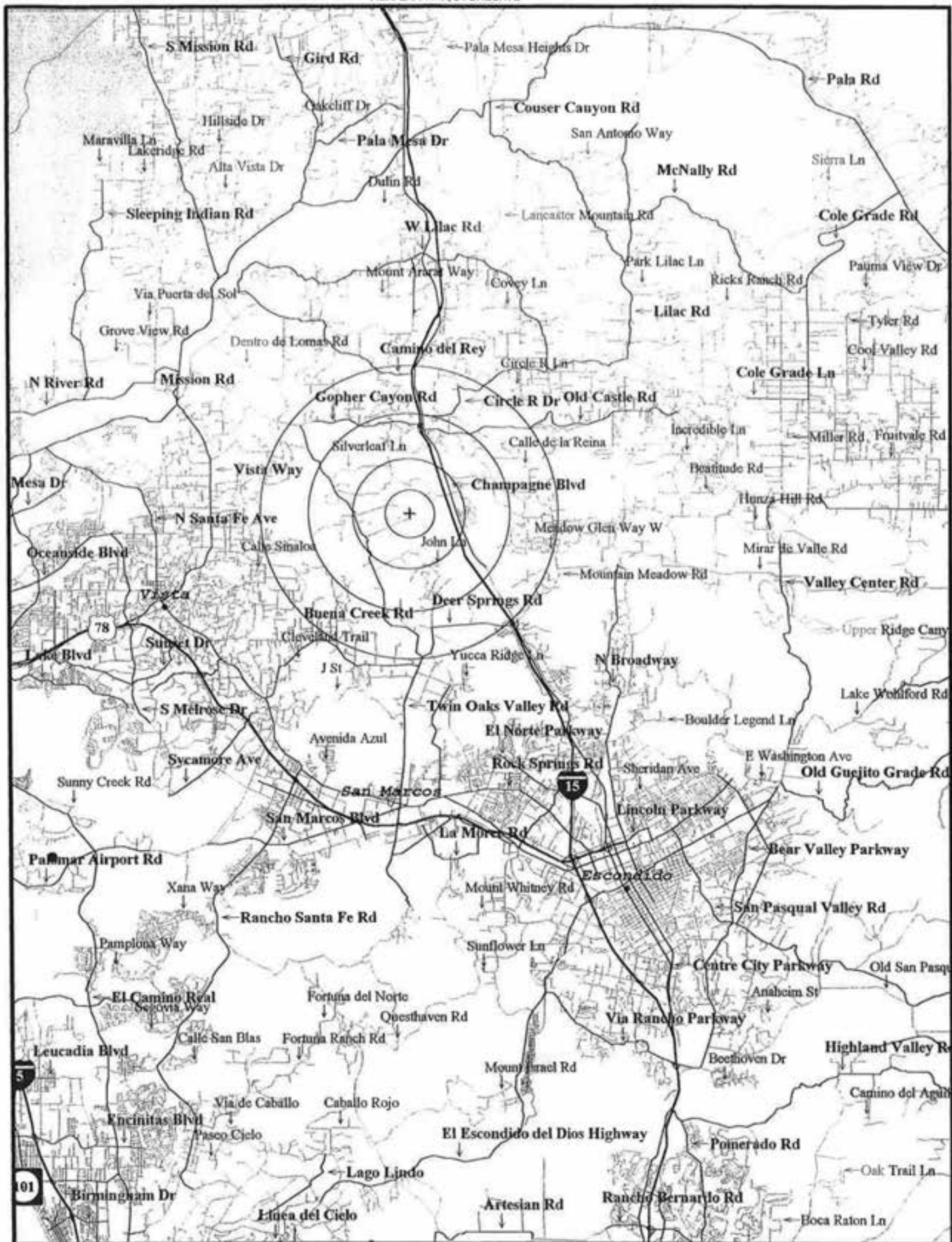


FIGURE 1



FIGURE 2

relative elevations were then converted to absolute by registering lines with a detailed topographic map furnished by the client. The site was somewhat away from freeways and busy streets; consequently, traffic noise was low. Wind noise was no more than a minor problem.

Geologic Setting - The site is on the Peninsular Ranges Batholith, consisting of a composite of individual Mesozoic intrusive bodies, mostly granitic clan rocks of Cretaceous age. Metamorphosed host rocks, locally found in roof pendants, are found here and there among the granities. None of the host rocks, however, appeared to be in the immediate area. The metasediments and metavolcanics are generally considered to be of Jurassic age. The intrusive rocks tend to be bi-modal. Small basic intrusive rock bodies are found locally within the granitic terrain, and outcrops of these basic rocks are found some distance to the north and east of the site, but apparently not in the immediate area. The local body, or bodies, of the composite batholith are very quartz rich. Free quartz is abundant enough in the matrix that it has a significant effect on the weathering process.

Seismic surveys in this crystalline rock setting have virtually always revealed a three layer case in the refraction data, when the investigation is deep enough. The topmost layer is commonly thin and is composed of soil and colluvium. The second layer is weathered crystallines and the deepest layer is unweathered crystalline rocks. Generally, this is the rule at this site.

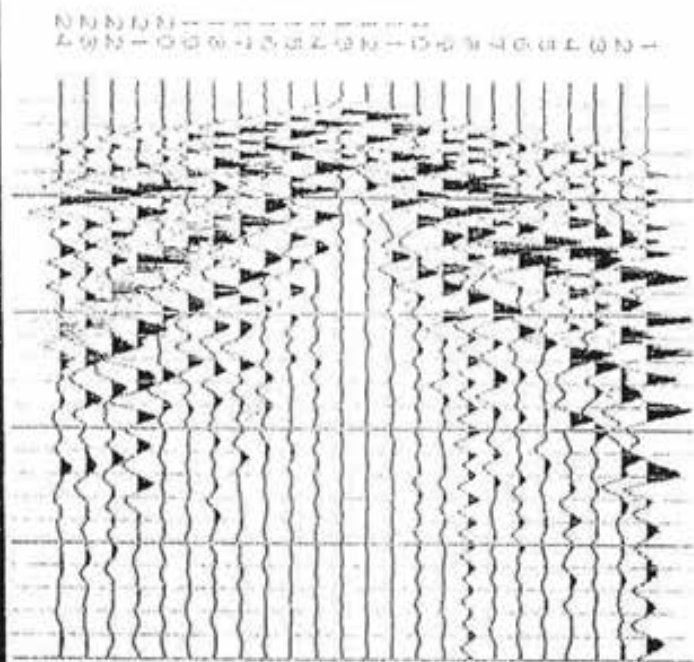
Brief Description of the Geophysical Method Applied - Seismic refraction investigates the subsurface by generating arrival time and offset distance information to determine the path and velocity of an elastic disturbance in the ground. The disturbance is created by shot, hammer, weight drop or some comparable method of putting impulsive energy into the ground. Detectors are laid out at regular intervals in a line to measure the first arrival energy and the time of its arrival. The data are plotted in time-distance graphs, from which velocity of, and depth to, layers can be calculated. This is possible because rays (a continuum point on an expanding wave front) of the disturbance wave follow a direct route, and are refracted across layer boundaries where there is a difference in elastic and density properties. The critically refracted ray travels along the layer interface, at the speed of the lower layer, and continuously "feeds" energy back to the surface, to be successively detected by the line of geophones.

Shots are normally reversed from one end of the line to the other, to determine whether or not the layering is horizontal or dipping. And a split spread shot adds redundancy to improve the interpretation. The acquired data are computationally intense. A ray-tracing computer program, SIPT2 in this instance, is used to iteratively honor all refracting surfaces, velocities, and to be able to consider a large number of layers, where they are present. A first energy arrival picking program, with such features as zoom, filtering, time stretching, separation of traces, AGC and balancing of traces, is also applied.

Interpretation

Data Quality - Monitor records are produced in the field with each shot. These are prints of the raw data as it comes in to the recorder. They show the quality of the data, so that the operator can determine whether or not the data are pickable, or shots need to be repeated. Two representative monitor records, one a split spread shot from line 10 and a forward shot from line 11 (Fig. 3) are illustrated.

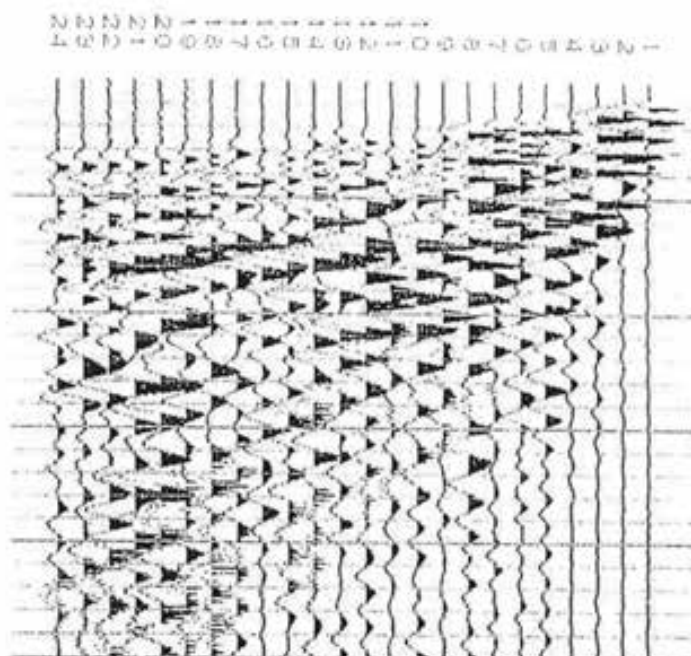
First energy arrivals are seen to be fairly sharp on the raw records, although some very minor



BISON 9000 SERIES

Record Name: LEIG0048
Date 09:10:02 Time 15:03
Hi-cut 2000 Lo-cut 16
Sample rt .500ms DFic Out
Delay(ms) DFhc Out
Channels 24 DFnt Out
Samples 500 DFbp Out
Rec len 250ms Agc Off
Time scale = 10 (ms)/division.

PCH	GN	STK	EX	PCH	GN	STK	EX
+ 01	M	0002	07	+ 13	M	0002	15
+ 02	M	0002	07	+ 14	M	0002	12
+ 03	M	0002	08	+ 15	M	0002	11
+ 04	M	0002	08	+ 16	M	0002	10
+ 05	M	0002	10	+ 17	M	0002	09
+ 06	M	0002	10	+ 18	M	0002	09
+ 07	M	0002	09	+ 19	M	0002	09
+ 08	M	0002	10	+ 20	M	0002	09
+ 09	M	0002	10	+ 21	M	0002	07
+ 10	M	0002	11	+ 22	M	0002	07
+ 11	M	0002	12	+ 23	M	0002	07
+ 12	M	0002	14	+ 24	M	0002	07



BISON 9000 SERIES

Record Name: LEIG0051
Date 09:10:02 Time 16:03
Hi-cut 2000 Lo-cut 16
Sample rt .500ms DFic Out
Delay(ms) DFhc Out
Channels 24 DFnt Out
Samples 500 DFbp Out
Rec len 250ms Agc Off
Time scale = 10 (ms)/division.

PCH	GN	STK	EX	PCH	GN	STK	EX
+ 01	M	0003	16	+ 13	M	0003	07
+ 02	M	0003	13	+ 14	M	0003	07
+ 03	M	0003	11	+ 15	M	0003	07
+ 04	M	0003	12	+ 16	M	0003	07
+ 05	M	0003	10	+ 17	M	0003	06
+ 06	M	0003	09	+ 18	M	0003	06
+ 07	M	0003	09	+ 19	M	0003	06
+ 08	M	0003	08	+ 20	M	0003	05
+ 09	M	0003	09	+ 21	M	0003	05
+ 10	M	0003	08	+ 22	M	0003	05
+ 11	M	0003	09	+ 23	M	0003	05
+ 12	M	0003	08	+ 24	M	0003	05

FIGURE 3

wind noise is coming in mostly on the far offset traces. This is not uncommon for off end shot records. Even so, with use of a computer picking program, with zoom, filtering, etc., there was no difficulty in picking the first energy arrivals. There should not be significant variation in picked arrival times should the first breaks be picked by several persons independently.

More of the shooting parameters are listed below the monitor records (Fig. 3).

The first pick information, geophone locations and geometry of the spreads, are input to a routine that produces a time-distance plot (e.g. Fig. 4, from line 6, all shots). The eight curves reflect the shots at the five positions along the line, as previously outlined. The split spread shots, however, produce two curves going in opposite directions. The data show the expected 3-layer case as is illustrated by generalized straight lines drawn through the reverse curve. Locally the curves appear "spikey"; this is thought to be due to the presence of core rocks.

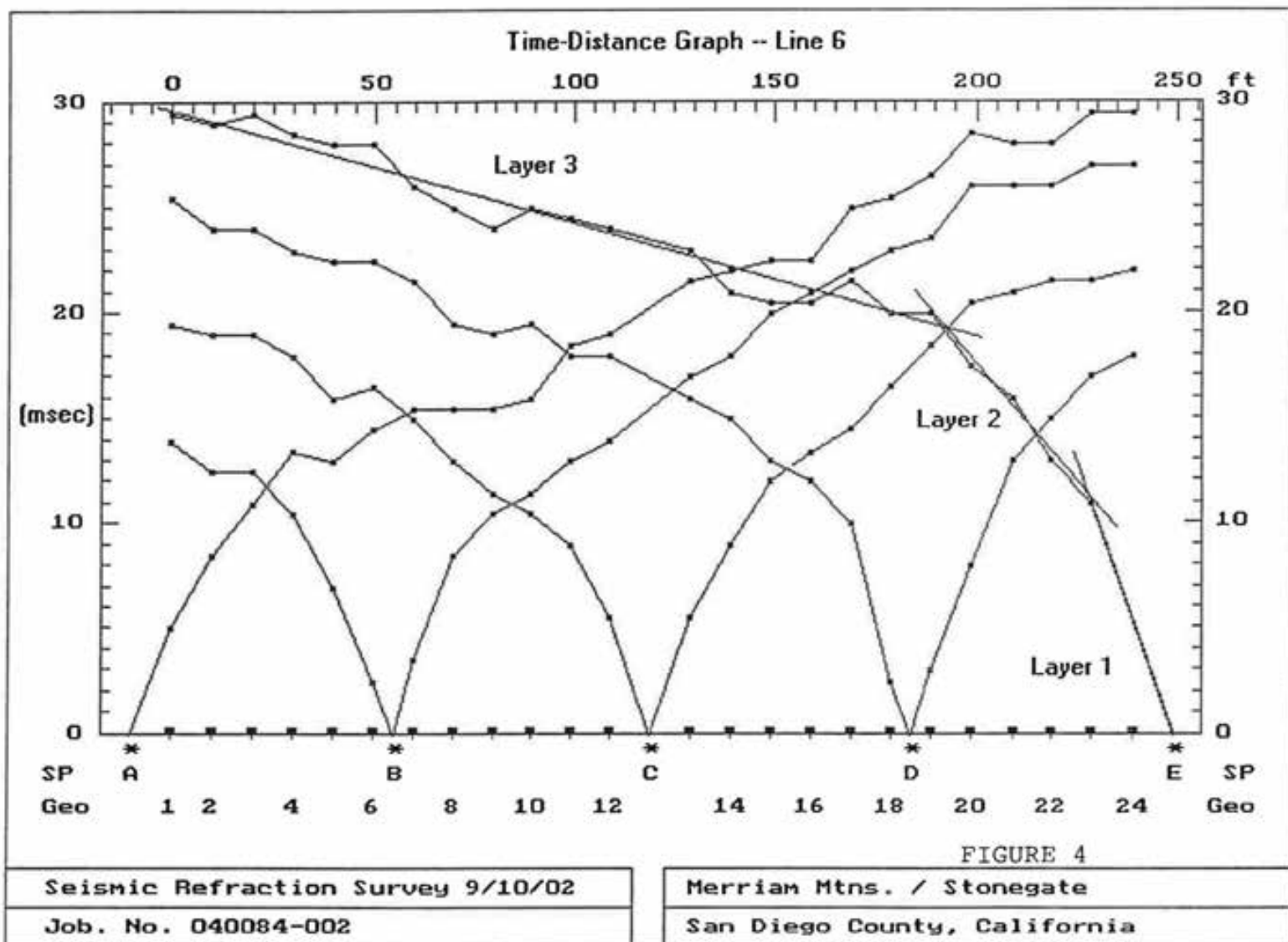
It is obvious that the topmost two layers are relatively thin in the T-D plot illustrated. The aforementioned undulations in the curves, due to the presence of core rocks, are, in part, explained by the fact that elevation corrections are not yet applied to the data in the time-distance plot, perhaps small disturbances from noise, and lower energy at far offset geophones. Minor variations in the positions of the "dog-legs" in the several curves are mostly an expression of the laterally changing thicknesses of the uppermost layers.

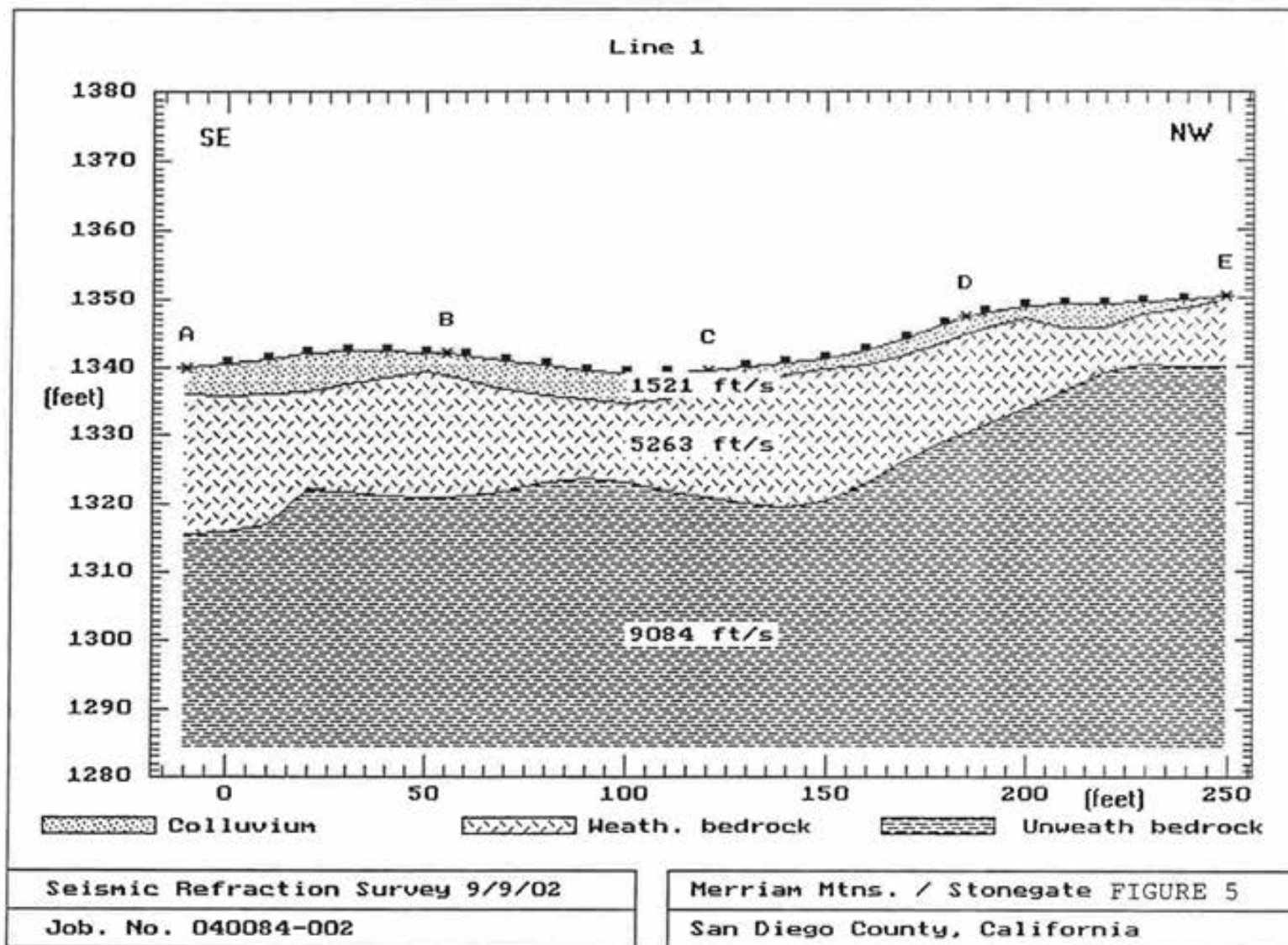
Findings – The geologic models were calculated utilizing the SIPT2 program, which includes an iterative ray tracing procedure. With the processing of the "corrected" data, including elevation corrections, geologic models were developed for the twelve lines (Figs. 5-16). Boundaries appear to be somewhat undulating; however, some vertical exaggeration in the structure sections makes this appear to be more than it is. In general the layer boundaries mimic the surface topography, which is typical of weathering-controlled layer boundary development. Departures, nevertheless, are partially explained by cultural disturbance of the surface in the preparation of pads and roads. There is considerable fill on line 4, for example.

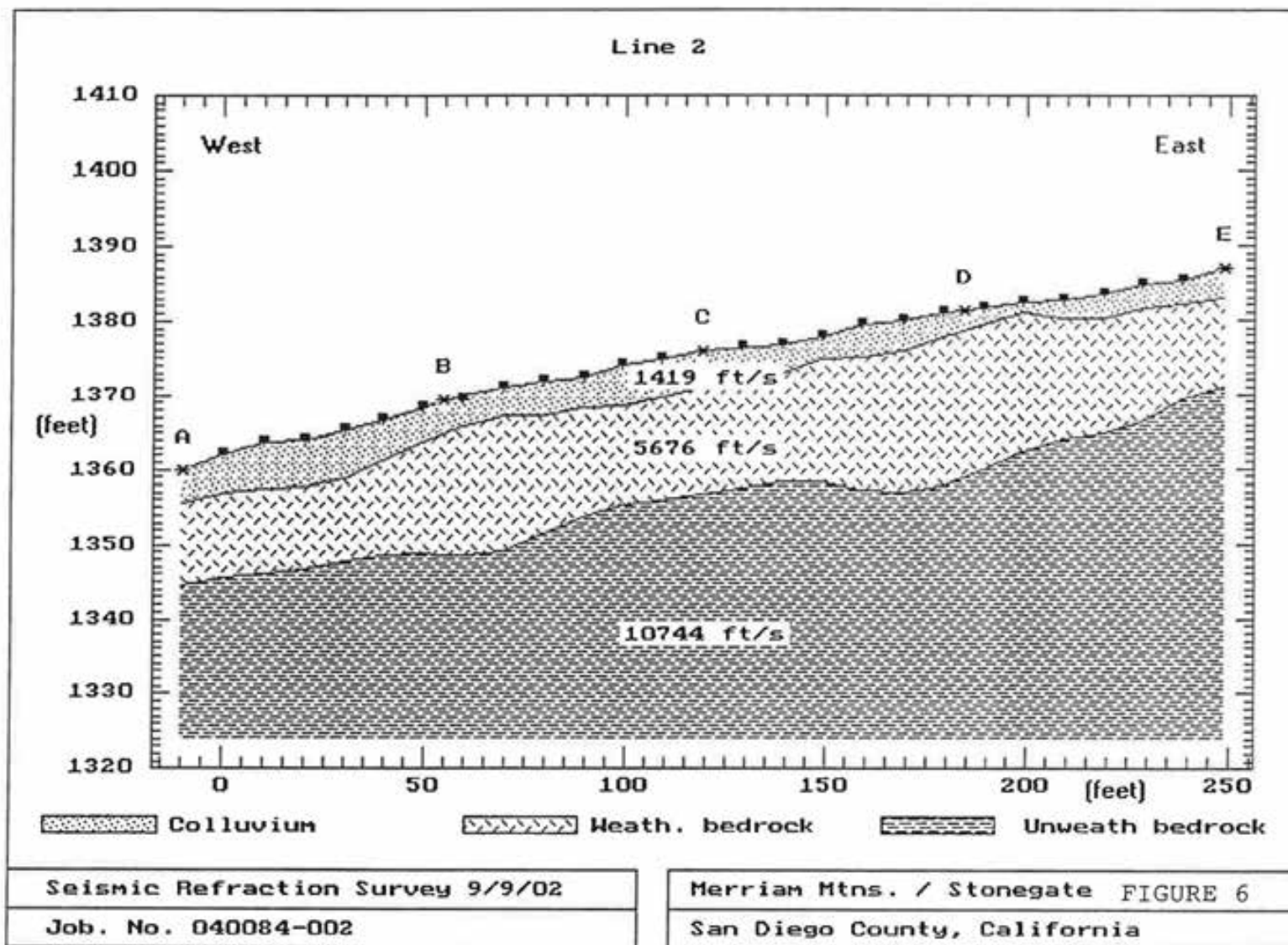
Thickness of the topmost soil/colluvium layer averages approximately 4.5 feet, but varies from feather edge to about 13 feet, where sampled. The variation is, in part, a function of the cultural reshaping of the near surface. Average velocity is about 1890 ft/sec. This is slightly above average for soil/colluvium in this geologic setting, but can still be characterized as being within normal range. Statistically, layer 1 velocity is higher in the north end of the area of investigation.

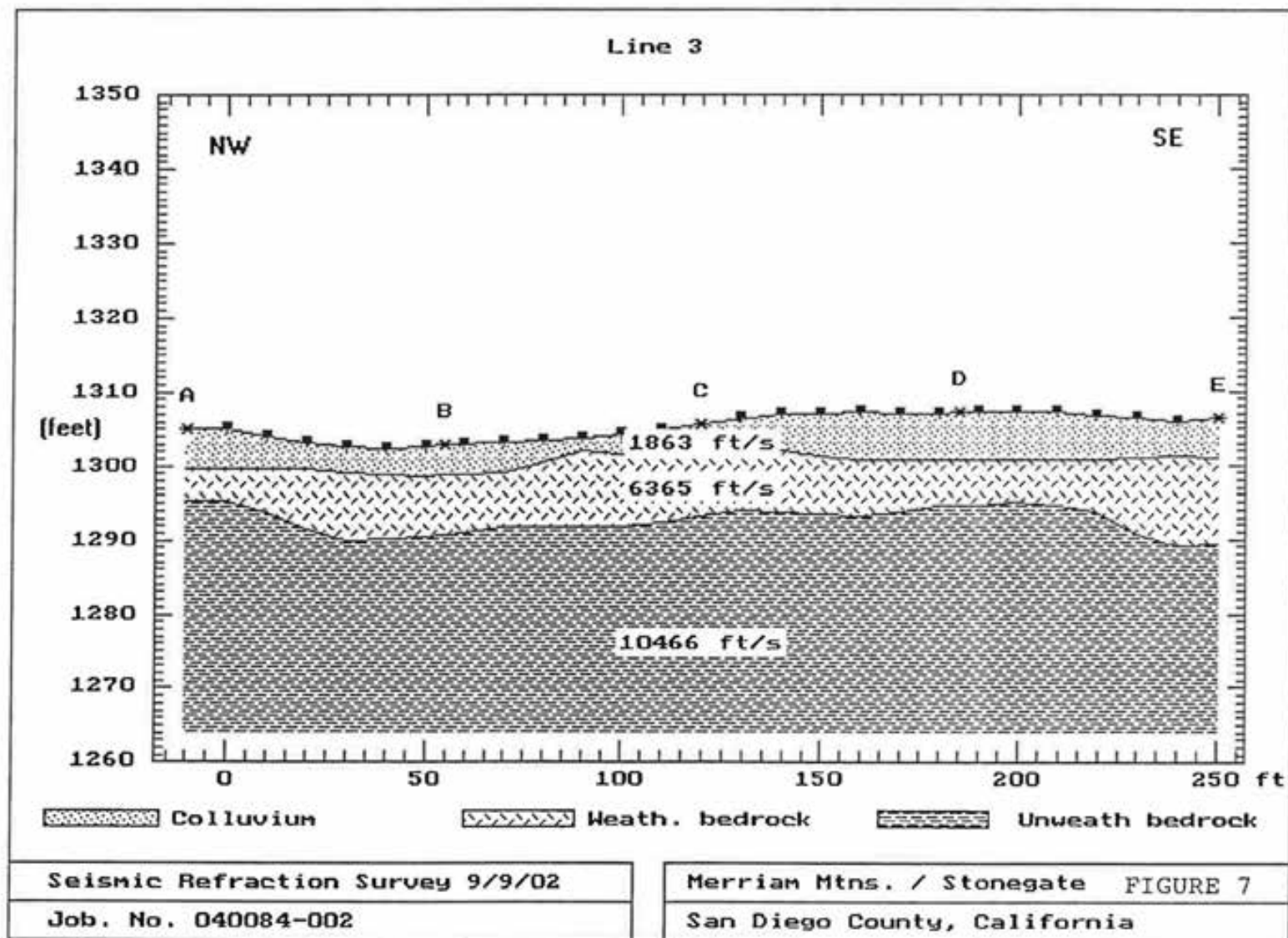
Layer 2 thickness is in the order of 20 feet. Average velocity is 5400 ft/sec. This statistical velocity is approximately twice the average found in other places (that we've worked) in the batholith, for weathered granite. This implies that weathering is less at this site than most other sites. It is our interpretation that the abundance of quartz in the matrix is the explanation of this aberration. It is the decay of feldspars to clay in the weathered zone that loosens the matrix and thereby lowers the seismic velocity. The presence of much quartz, which does not decay in the same manner as does feldspar minerals, that preserves more rigidity in the rock mass. The range of velocities in layer 2 is not excessive.

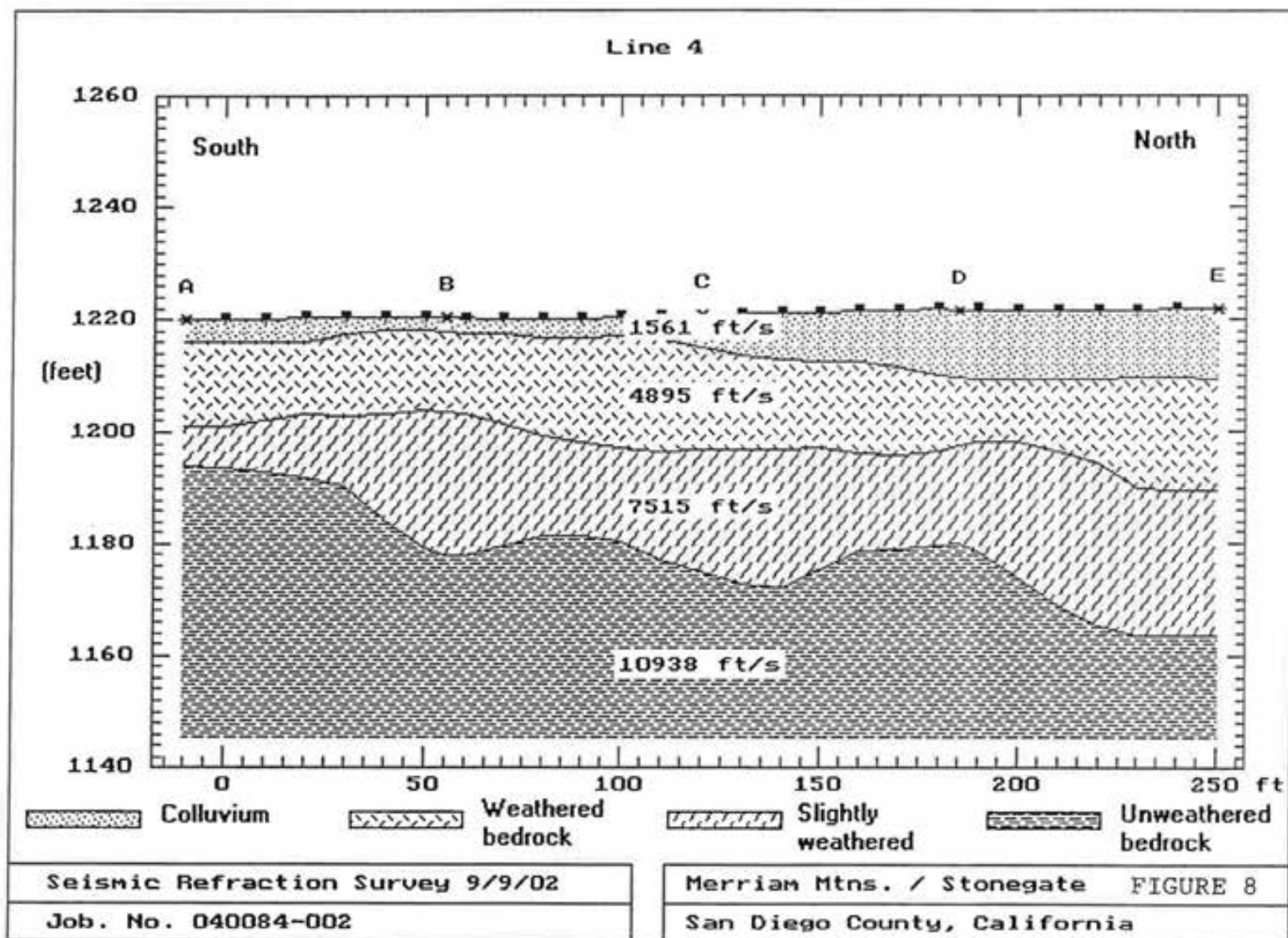
Finally, thickness of "unweathered" layer 3 (the deepest layer imaged) was not determined; presumably it extends to great depth. Its seismic velocity is in the order of 11,010 ft/sec, determined from samples on all twelve lines. This bulk velocity is higher than normal for unweathered granite. Abundance of quartz may be the explanation here also.

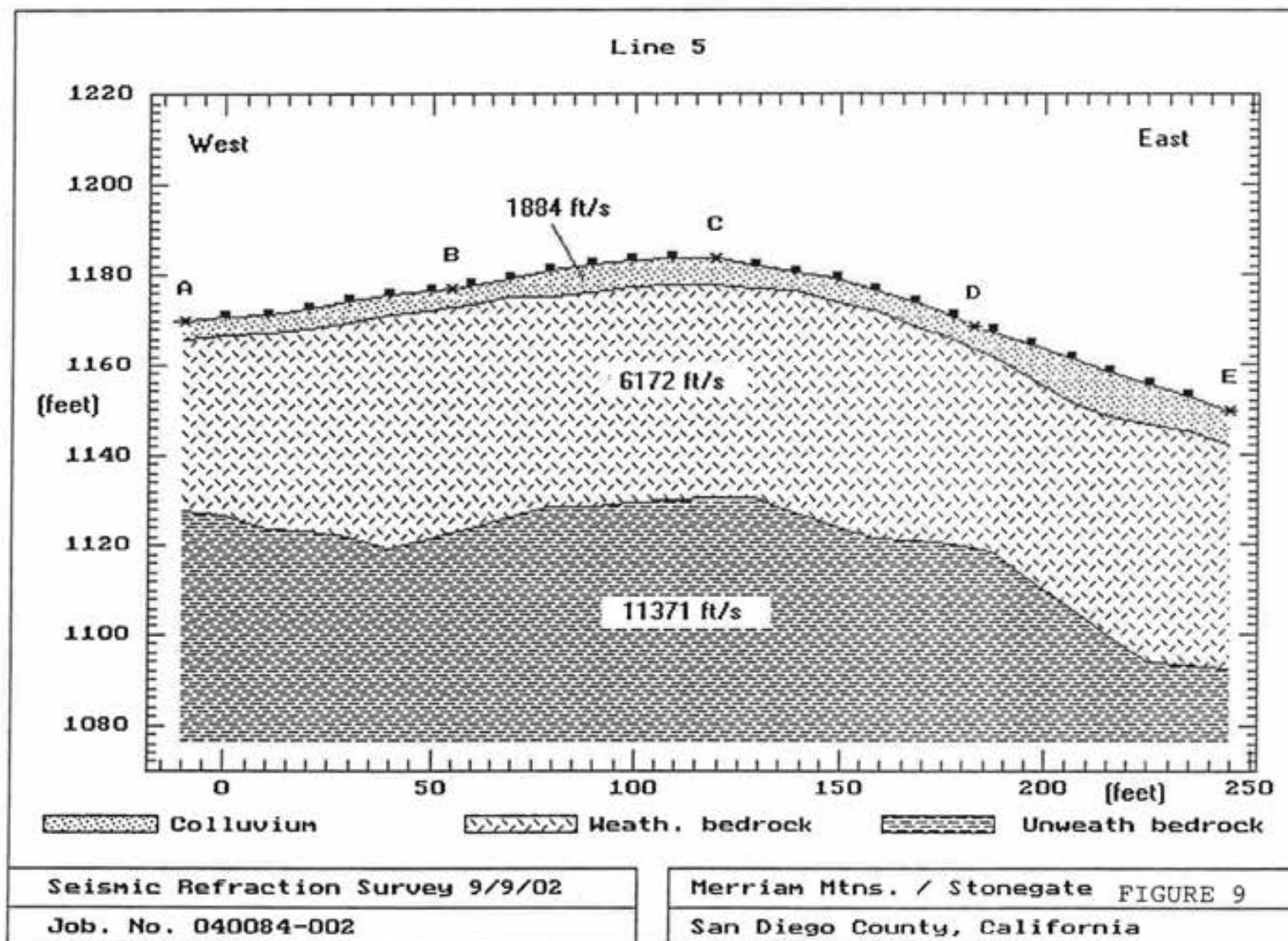


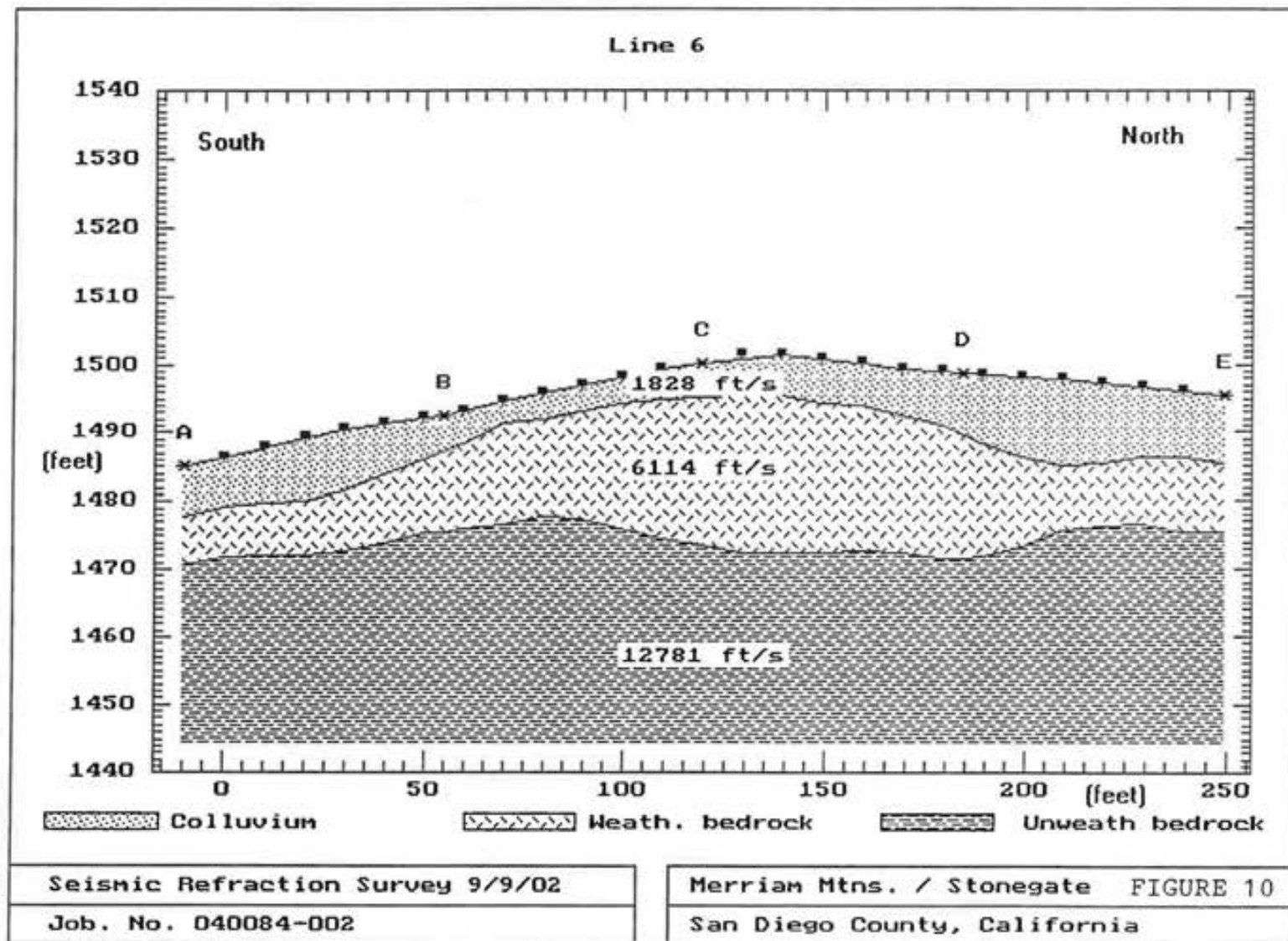


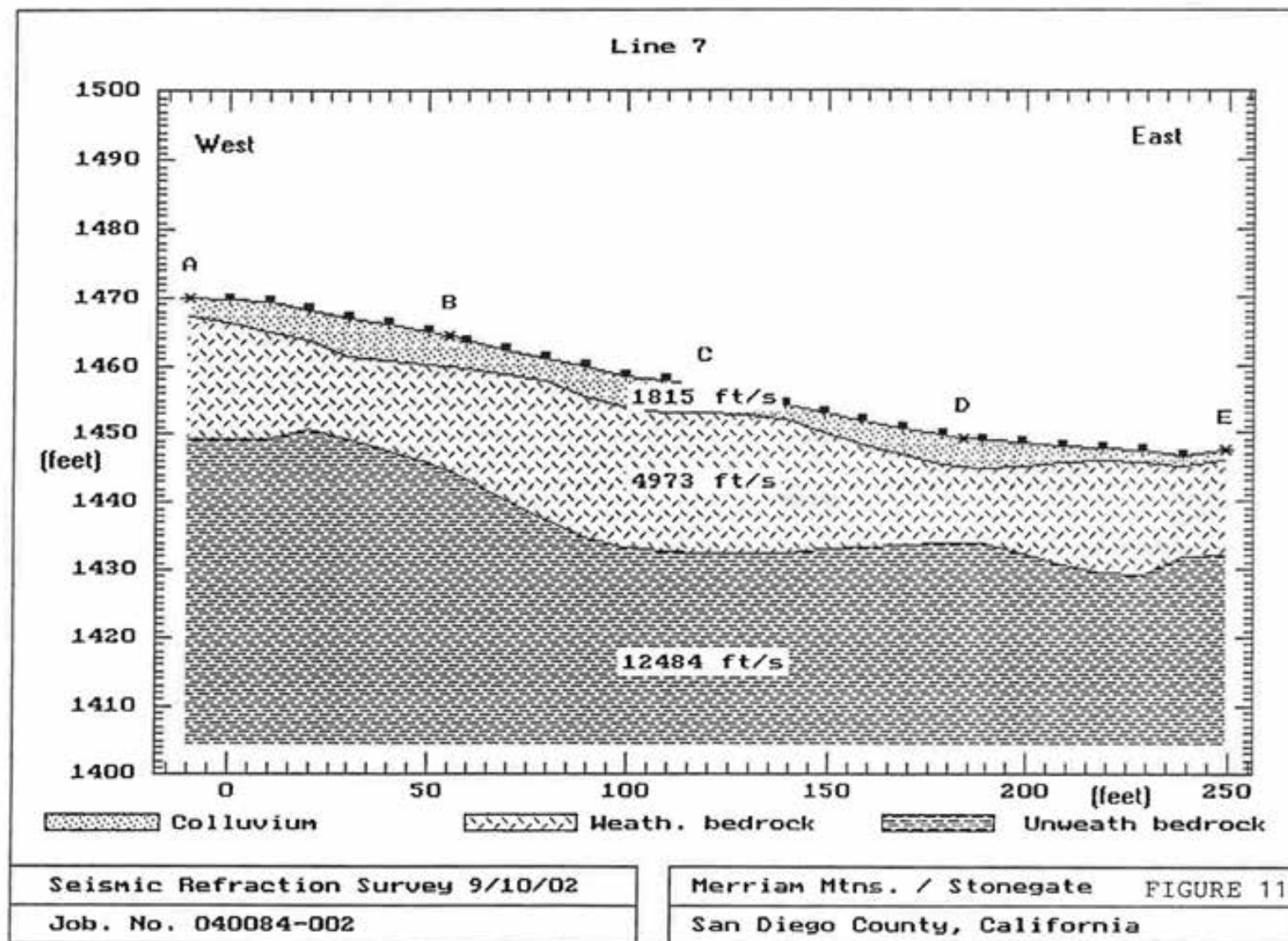


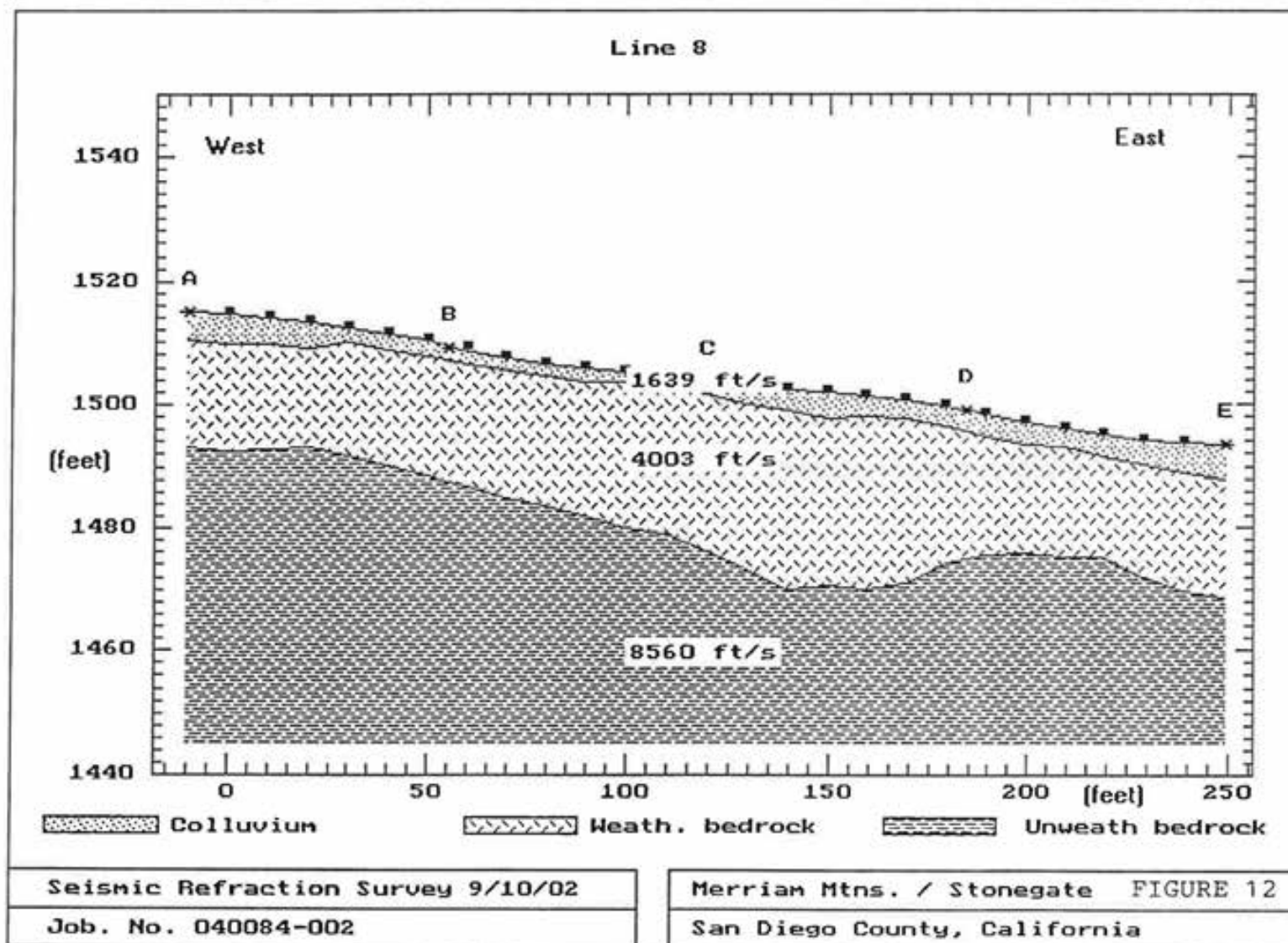


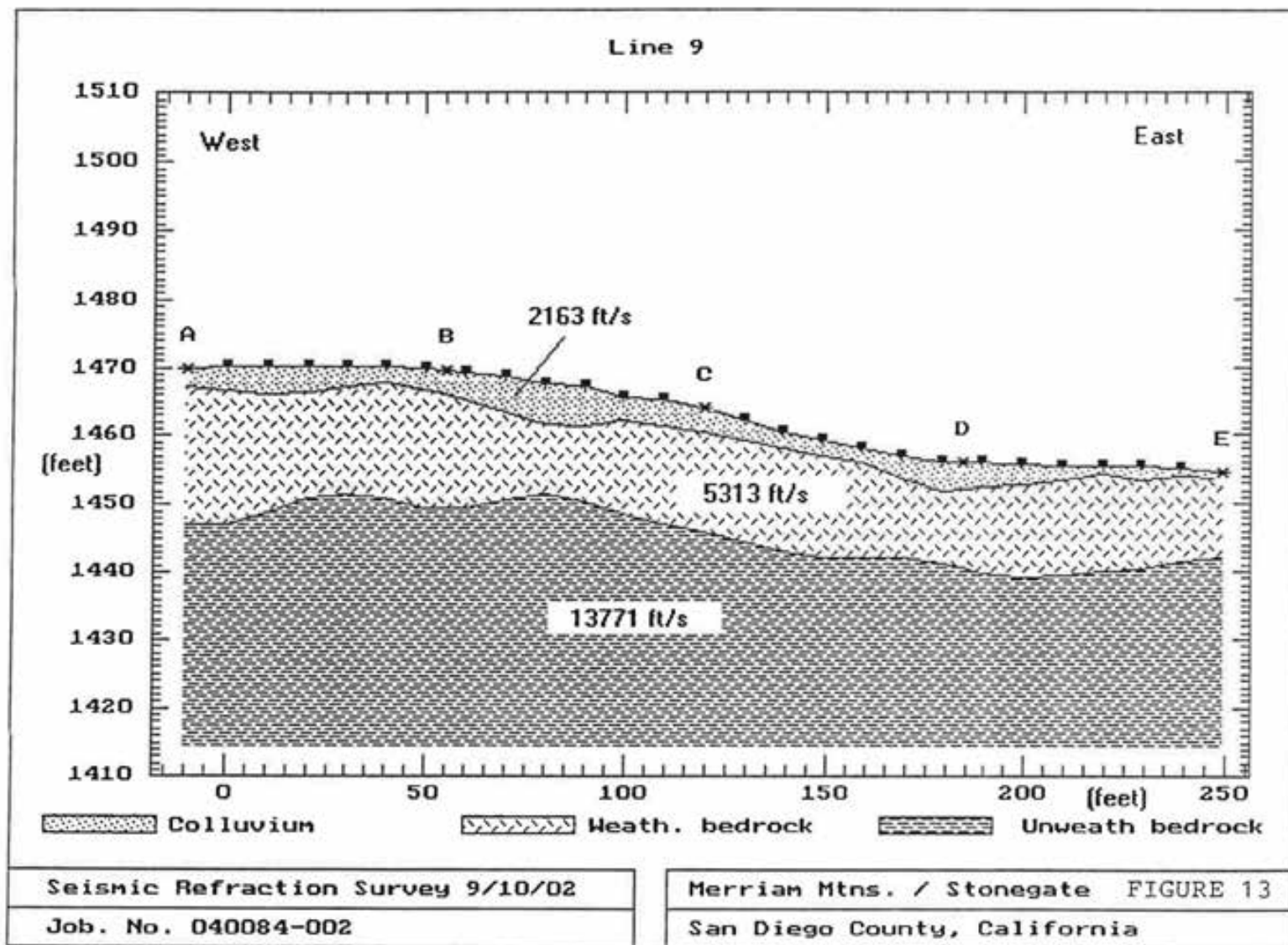


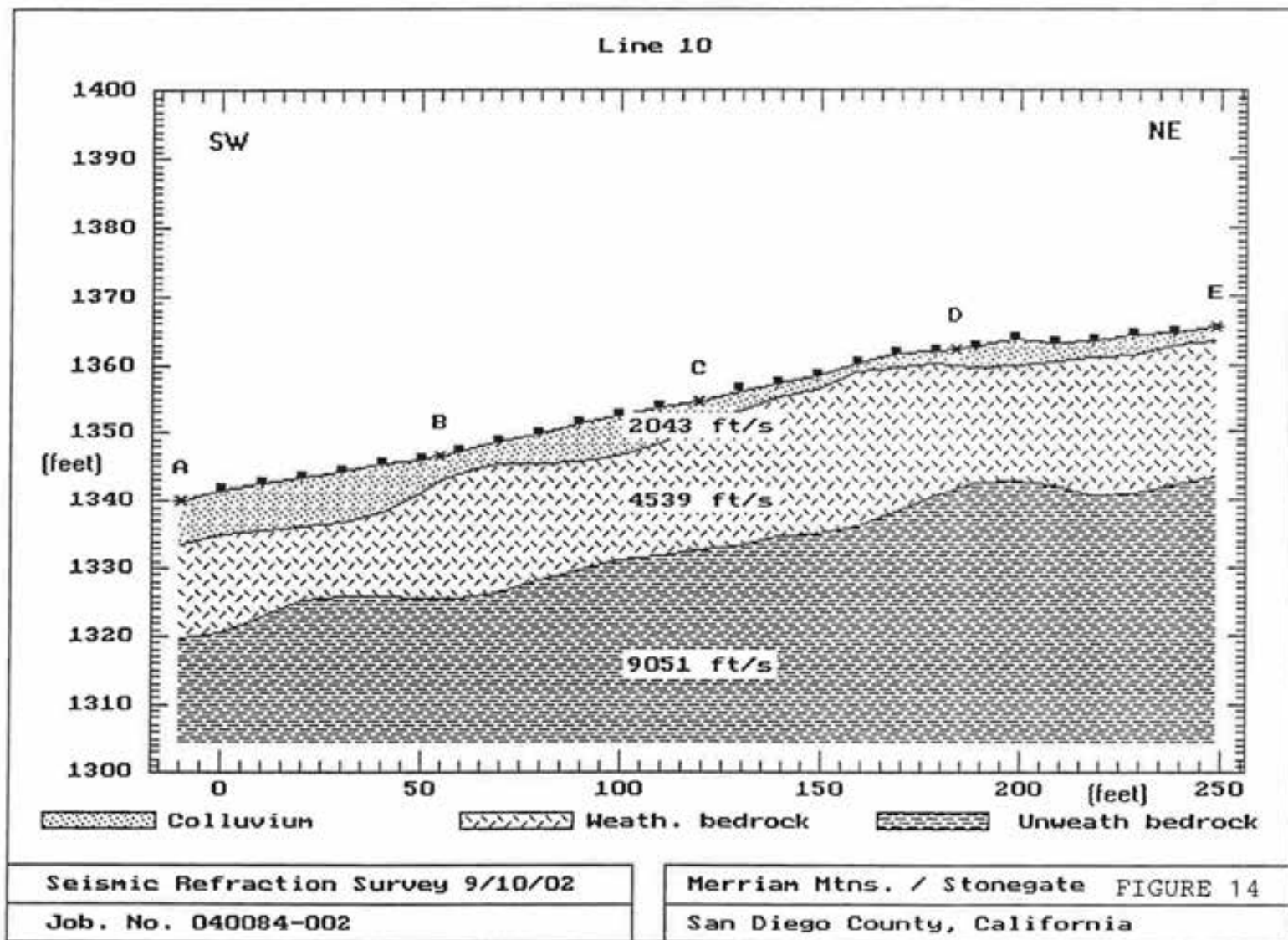


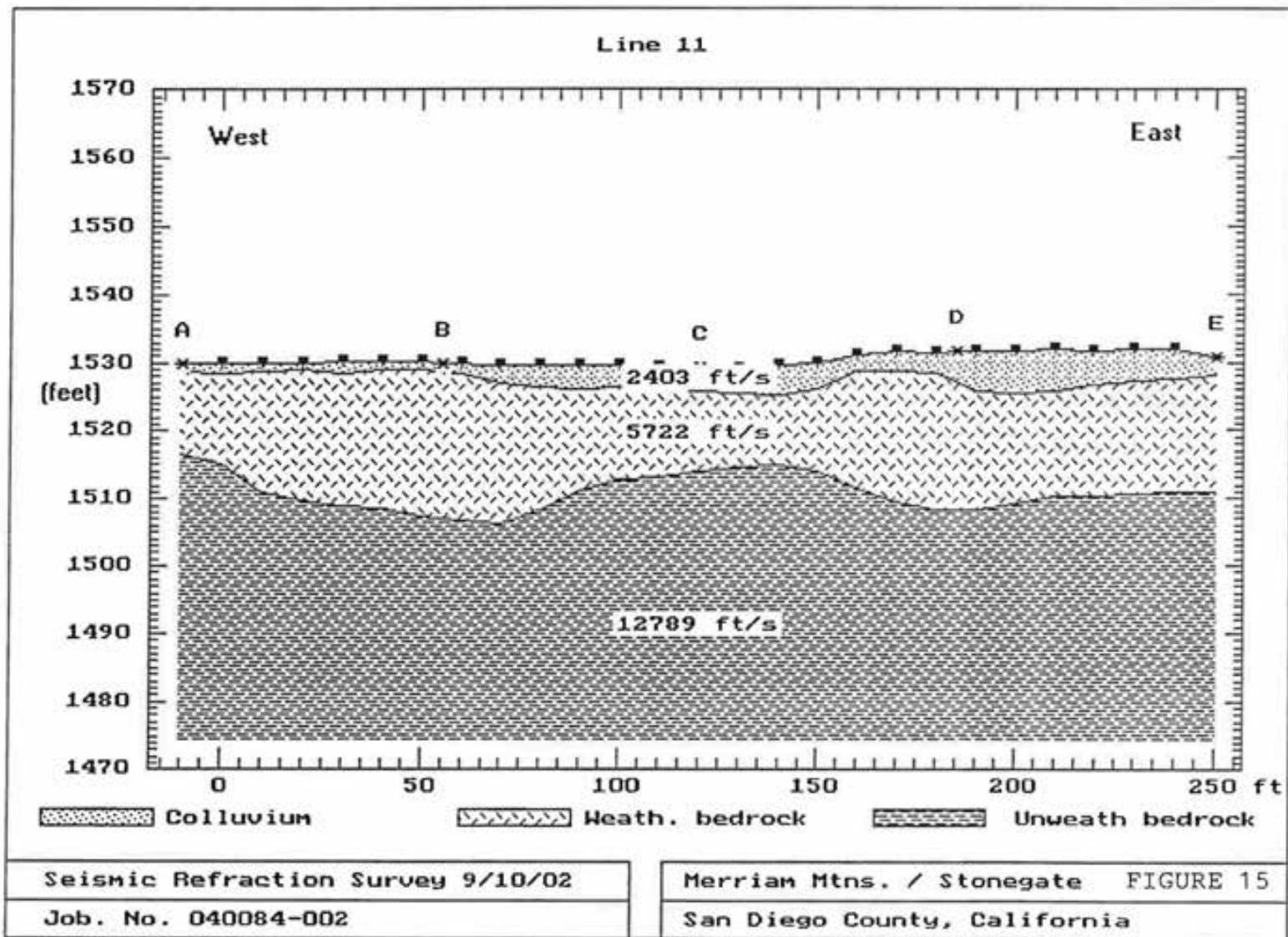


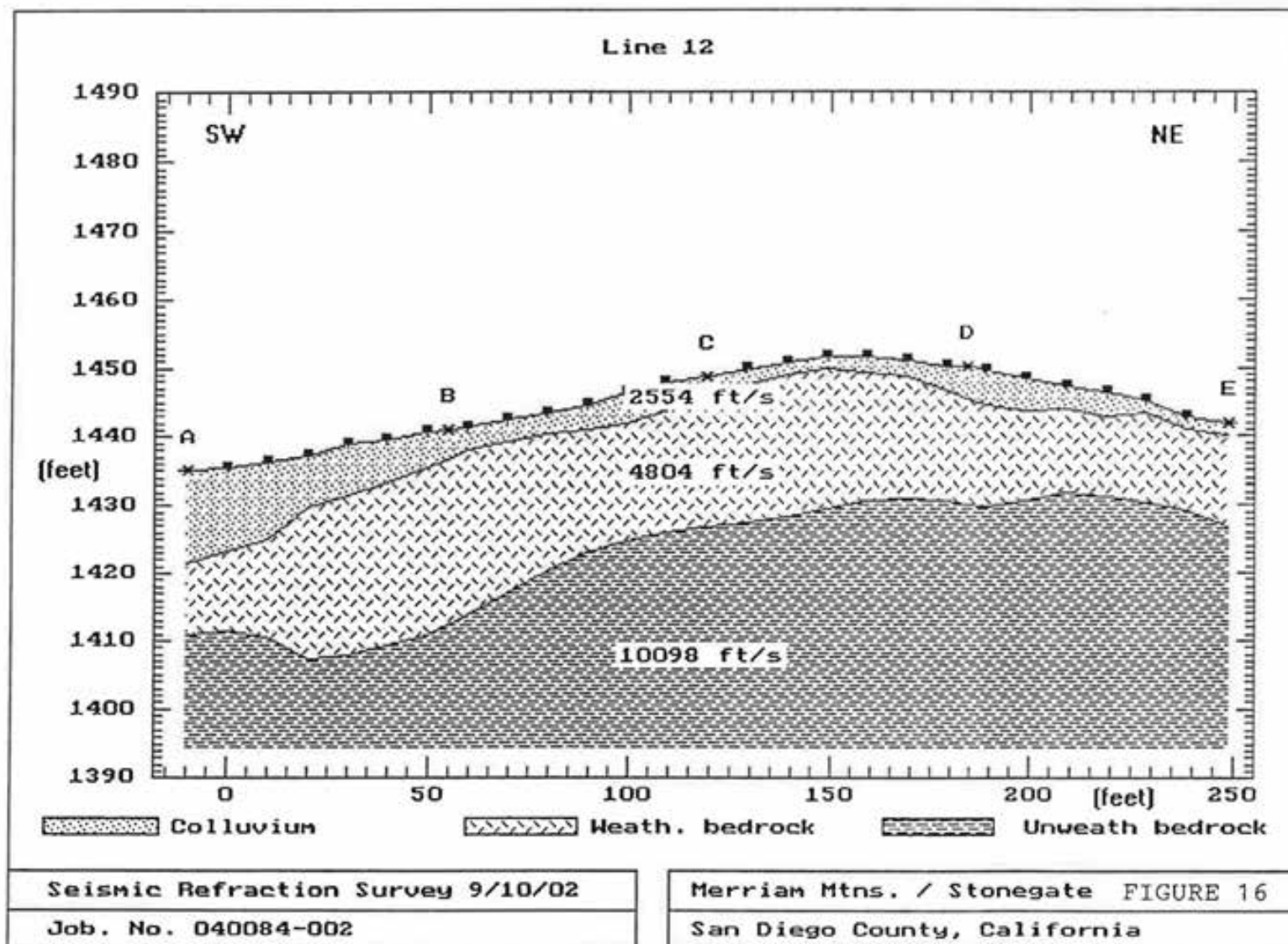












The geology under line 4 is an exception to the above-discussed factors; there are four layers revealed in the refraction data (Fig. 8). The average of layers 2 and 3 is virtually the same as the velocities of layer 2 in the other sections. This suggests that there is something unique in the weathering process at the position of line 4. Line 4 is in a major drainage, unlike the other lines. There are several geologic phenomena that can produce two weathering layers, for example, recent local uplift, recent lowering or raising of the water table, aggradation of sediments in a drainage, together with other possibilities.

The Caterpillar Rippability chart is illustrated and is the basis for determining rippability of the rocks encountered in the survey (Fig. 17). The chart is empirical, but is based on thousands of field and laboratory samples. It is seen that layer 1 is easily rippable everywhere sampled. Layer 2, except the layer 2 under line 4, is marginal, even with heaviest equipment. From the layer 3 velocities determined, it appears that for planning purposes the layer 3 rocks, unweathered granite, should be considered non-rippable everywhere.

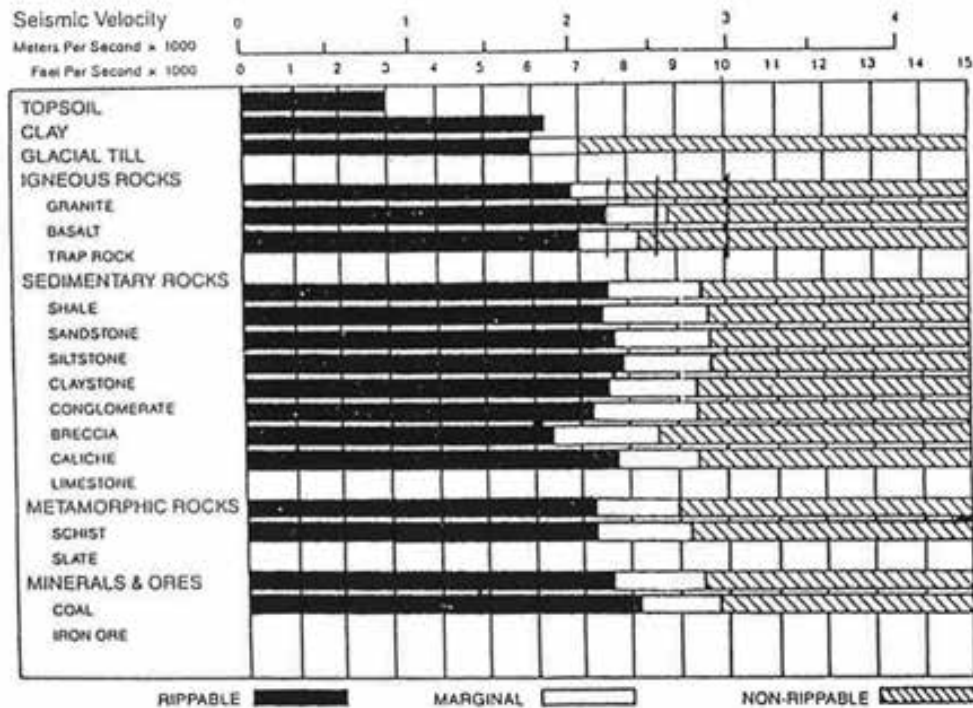


Figure 9. Caterpillar rippability chart

Two photographs (Figs. 18 & 19) are representative of the terrain and the flora at the site. The large granitic boulders seen in the slopes are the tops of core rocks. These bodies are non-rippable, and those that are too large to be moved by bulldozers will have to be broken up by use of explosives.

Conclusions – For planning purposes, layer 3 rocks, unweathered quartz-rich granites, are well within the non-rippable range, and if cuts into this unit are planned, drilling and blasting will be required. Unweathered granites come to within about 10 feet of the surface locally, but generally they are deeper than 20 feet. It also appears that layer 2 rocks will be marginal to unrippable; consequently, some blasting will be necessary at shallow depths. The presence of much quartz

Site Photographs
Merriam Mountains / Stonegate Project



FIGURE 18

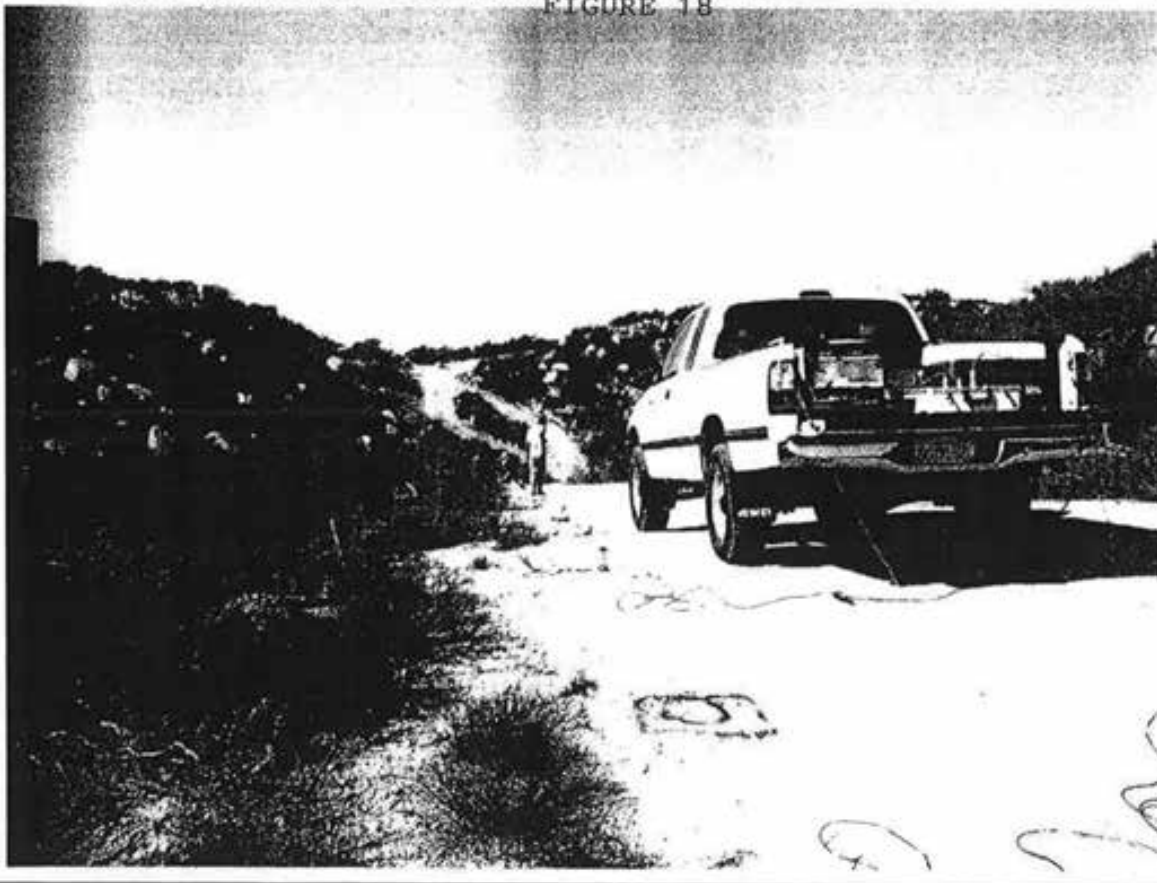
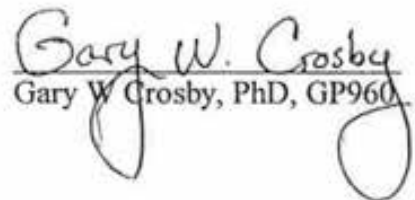


FIGURE 19

in the matrix is the probable cause of this unusual mechanical strength of the unweathered and weathered rocks.

SubSurface Survey's professional personnel are trained and experienced and have completed thousands of projects since the company's inception in 1988. It is our policy to work diligently to bring this training and experience to bear to acquire quality data sets, which in turn, can provide clues useful in formulating our interpretations. Still, non-uniqueness of interpretations, methodological limitations, and non-target interferences are prevailing problems. SubSurface Surveys makes no guarantee either expressed or implied regarding the accuracy of the interpretations presented. And, in no event will SubSurface Surveys be liable for any direct, indirect, special, incidental, or consequential damage resulting from interpretations presented herewith.

All data generated on this project are in confidential file in this office, and are available for review by authorized persons at any time. The opportunity to participate in this investigation is very much appreciated. Please call, if there are questions.


Gary W Crosby, PhD, GP960

GWC:arr



Leighton Group
3934 Murphy Canyon Road
Suite B-205
San Diego, CA 92123

October 26, 2007

Attn: Brian Olson

Re: Merriam Mtns. Seismic Survey

This report covers the results of a seismic refraction survey on the Merriam Mountains property located northwest of Deer Springs Road and Interstate 15 in San Diego County, California. The purpose of the survey was to measure the compressional wave velocity of bedrock for rippability assessment and to provide cross sections showing depth and configuration of bedrock. This should be useful to geologists and engineers planning cuts and other earth work for future development of the site.

The field work was performed on October 11, 2007 across moderate to steep slopes of undeveloped land. Six seismic refraction lines were recorded at locations selected by Leighton, most of which were laid out adjacent to dirt access roads. A survey location map is provided on Figure 1. Twelve seismic lines were previously recorded on this property during 2002 and are discussed in Subsurface Surveys report dated September 20. To avoid confusion with the previous work, this group of lines were numbered 13-18.

GEOLOGIC SETTING

A review of the "Geologic Map of California, Santa Ana Sheet", (California Division of Mines and Geology, 1966) indicates the local area is underlain Mesozoic granitic rocks that are primarily granodiorite. Large massive boulders were observed along steep slopes and along ridge tops, particularly in the area of Lines 17 and 18. Surface deposits were colluvium and decomposed bedrock.

DATA ACQUISITION AND FIELD METHODS

Seismic refraction data were recorded with a Bison 9024 signal enhancement seismograph and 30 Hz geophones. The standard spread layout used 24 geophones with a 10 foot spacing. Each spread used five shotpoints, one off each end (10 foot offset) and

three within the interior of the spread.

Compressional wave energy was created by sledge hammer impacts on a steel plate. The signal enhancement feature of the 9024 allowed returns from repeated hits to be stacked, thus improving the signal over the ambient noise. Vehicle traffic noise was not a factor during this survey. Each record was stored digitally on an internal hard disk and printed copies of each seismogram were made in the field on thermal paper. Example seismic records from this survey are shown of Figure 2.

Relative elevations of all shotpoints and geophones were determined by differential leveling with a hand level. Geophone 1 (distance = 0 ft.) at the beginning of each line was assigned a elevation value of 0.0 feet. All other elevation measurements along the line are relative to this point. A labeled wooden stake marks the beginning and end of each traverse.

SEISMIC REFRACTION METHOD

The refraction method involves measuring the total time for compressional waves to travel from a shotpoint through the subsurface to a set of geophones placed along the ground. Based on Snell's Law, when two or more layers are present with increasingly higher acoustic velocity, waves become critically refracted across the layer boundaries and begin traveling at the speed of the underlying layer. The advancing waves then generate new wavefronts back to the ground surface. The first surge of energy hitting the geophone is termed the "first arrival" and is depicted on the seismogram as a high angle deflection along each trace. Example field records from this survey that show the first arriving energy are provided on Figure 2.

Recognition of direct wave arrivals (non-refracted) verses refracted waves is a key element of refraction interpretation. To assist this process, the first arrival times measured from the seismic records are plotted on graphs of time verses distance called Time-Distance graphs. An example T-D graph from Line 16 is shown on Figure 3. Based on changes in slope on the graphs, a preliminary layer number is assigned to each segment of the T-D graph. The layer assignments together with time, distance and elevation data are input to a computer for additional processing.

DATA REDUCTION AND VELOCITY DETERMINATION

Processing and interpretation of this data set was accomplished with SIPT2, an interactive inversion modeling program developed by James Scott for the U.S. Bureau of Mines. The inversion algorithm uses the delay time method to construct a first pass depth

model. The model is then adjusted by an iterative ray tracing process that attempts to minimize the discrepancies between the total travel times calculated along ray paths and the observed travel times measured in the field.

This program calculates refractor velocity in two ways. First, apparent velocities from each shot are determined by the inverse slope of a best fit (least squares) line through datum-corrected travel times. True velocity is estimated from the apparent velocities by using the following equation:

$$V_t = 2(V_u \times V_d)/(V_u + V_d)$$

where V_t = true velocity
 V_u = apparent up dip velocity
 V_d = apparent down dip velocity

The second method uses a more sophisticated set of equations (the Hobson-Overton formula) developed by the Canadian Geological Survey. The final velocity assigned to the refractor is a weighted average of the results of the two methods. The weighting is based on the number of arrival times used in the computations.

SUMMARY OF RESULTS

Results from refraction analysis show a three layer solution beneath all survey lines. Velocities posted on the cross sections represent averages as described in the previous section. Therefore, minor localized changes in velocity may occur along any profile. A description of the layers is provided below and a velocity summary is shown in Table 1.

Layer 1 - is colluvium and decomposed bedrock. Thickness is generally less than 10 feet. The velocity range is 1537-2291 feet/sec.

Layer 2 - is interpreted to be weathered bedrock that varies in thickness from 4-43 feet, with an average of about 20 feet. The velocity of this layer is quite variable and reflects various degrees of weathering and the presence of hard core rock. For the most part, this layer is rippable. However there are a few localized hard (high velocity) zones that may be marginal. Most notable is the west end of Line 17 (6667 ft/sec) and the east end of Line 18 (6600 ft/sec).

Layer 3 - represents hard unweathered bedrock and is considered non-rippable. The depth to this interface is generally greater than 20 feet for Lines 13-16. For Lines 17 and 18, which were recorded along the ridge top, the depth to unweathered rock is mostly less

than 20 feet.

Table 1. Cross Section Summary Velocity in (ft/sec) Depth in (feet)

<u>Line</u>	<u>Layer 1</u>	<u>Layer 2</u>	<u>Layer 3</u>	<u>Depth Range</u>
13	2054	5036	10763	27-40
14	2291	4863	9353	20-27
15	1624	2855-5330	9361	20-46
16	1678	3650	10086	40-47
17	1537-2008	4818-6667	11122	15-20
18	1775	4996-6600	12188	7-22

Figure 4 presents a rippability chart (courtesy of Caterpillar Tractor Co.) for a D9N Ripper. Bar graphs show the relationship between seismic compressional wave velocity and ripper performance for various rock types in three categories: rippable, marginal, and non-rippable. Granitic rocks are listed as marginally rippable at approximately 6700 ft/sec and are considered non-rippable above 8000 ft/sec. This chart is provided only as a guide and should not be considered absolute. Other geologic factors that may influence bedrock rippability at this site include fracturing, jointing and the presence of unweathered core rock.

All data acquired during this survey is considered confidential and is available for review by your staff at any time. We appreciate the opportunity to participate in this project.

Please call if there are any questions.



Phillip A. Walen
Senior Geophysicist
CA Registration No. GP917

Seismic Survey Location Map

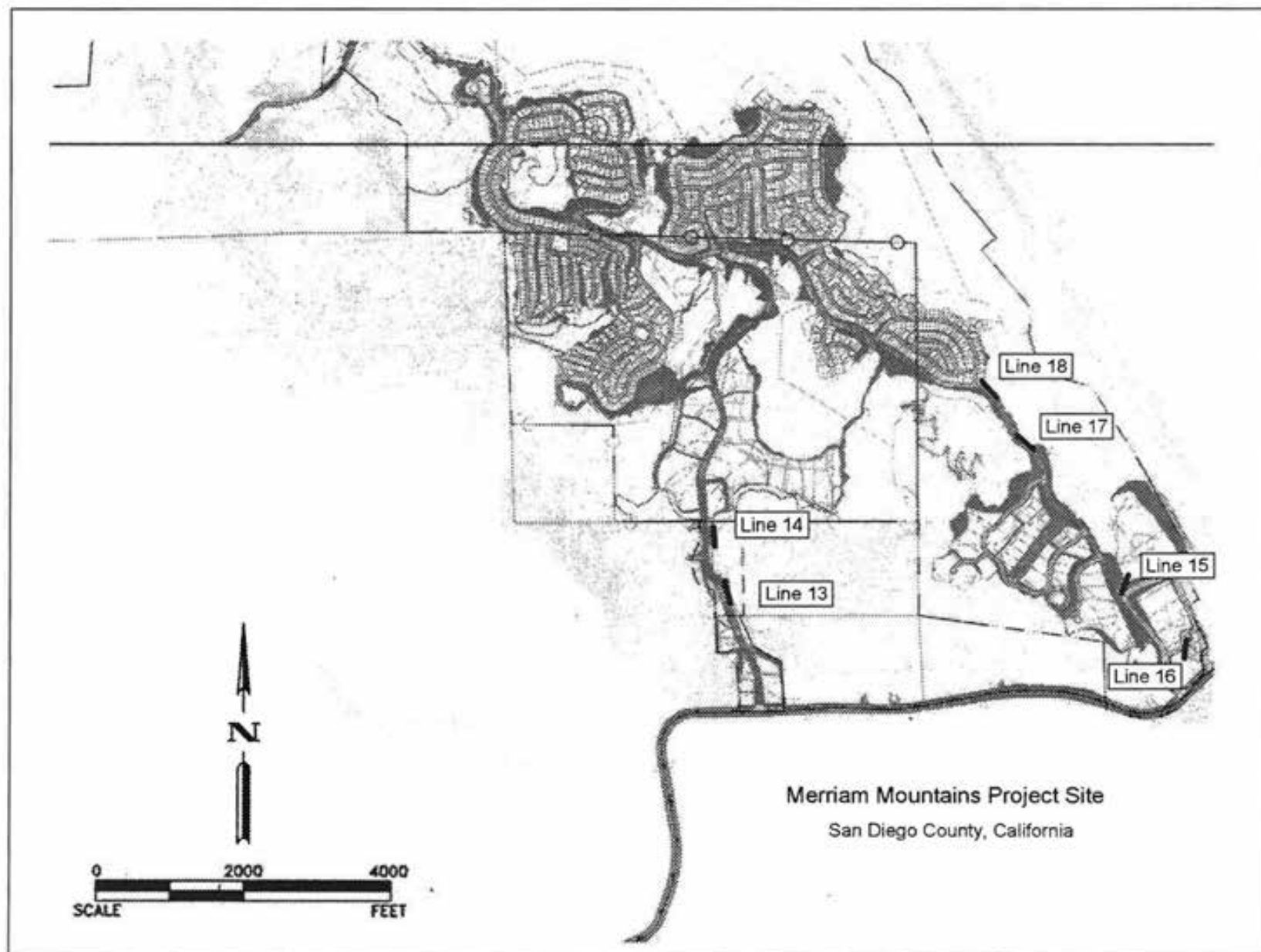


Figure 1

Example Seismic Field Records

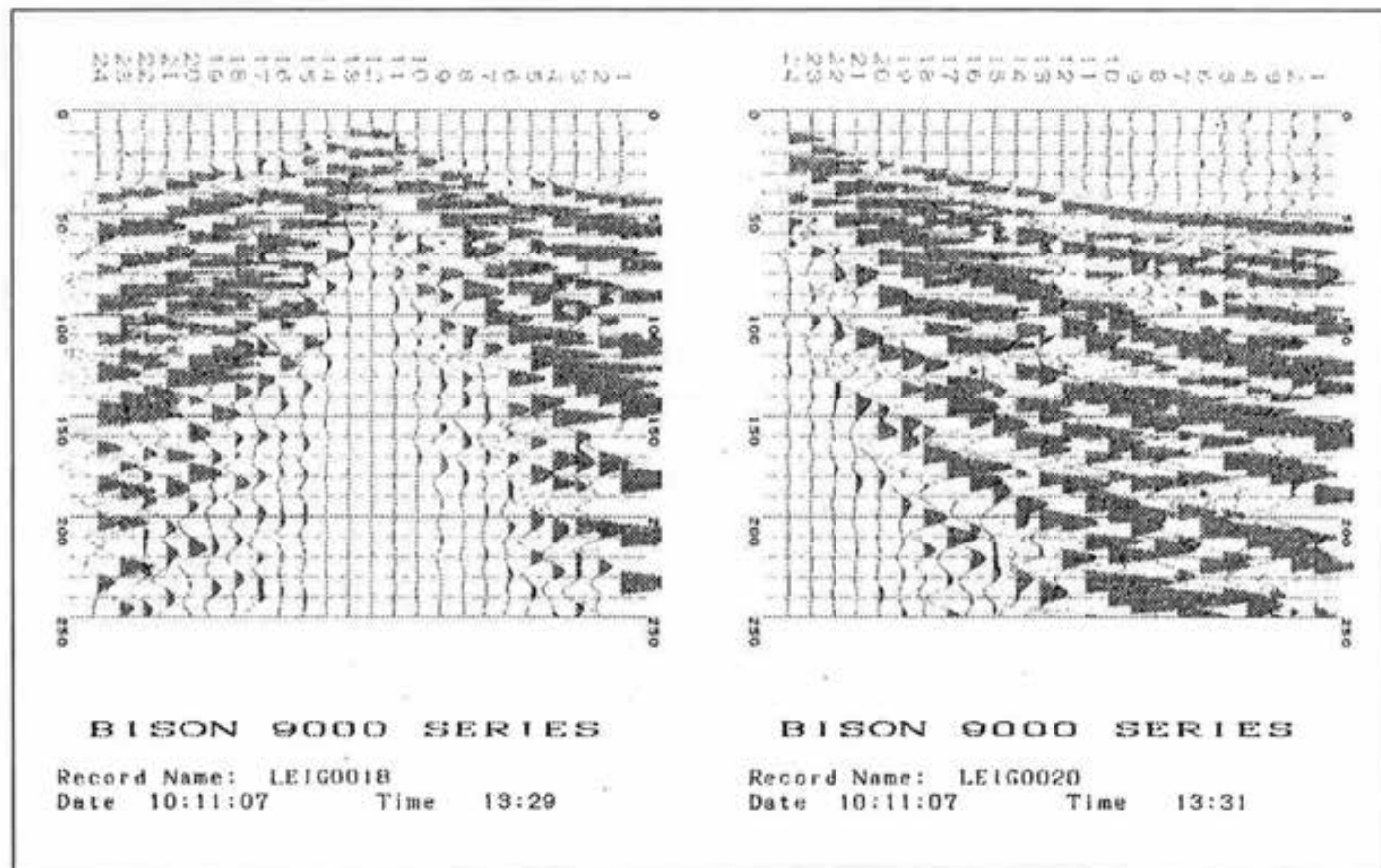
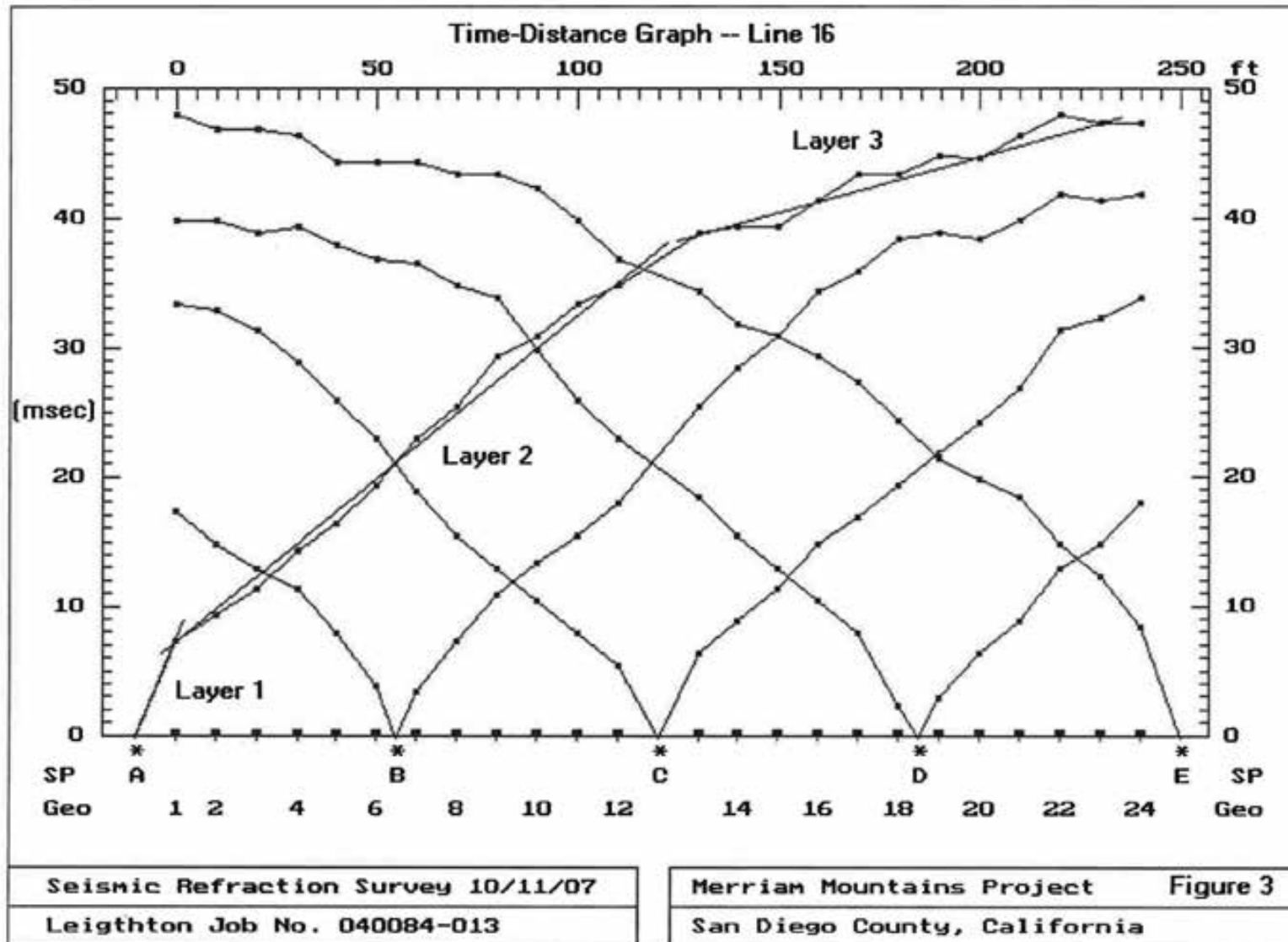


Figure 2



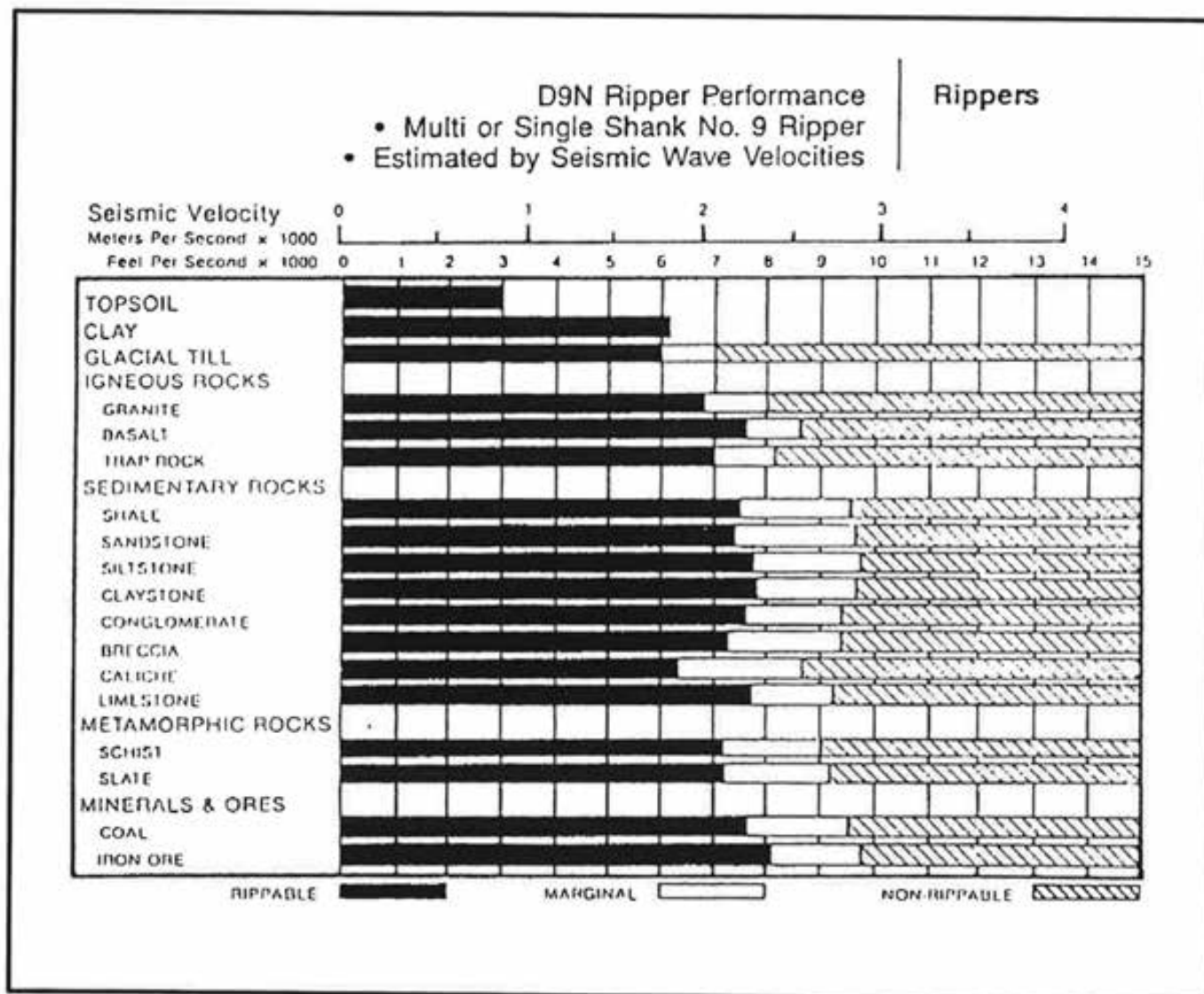
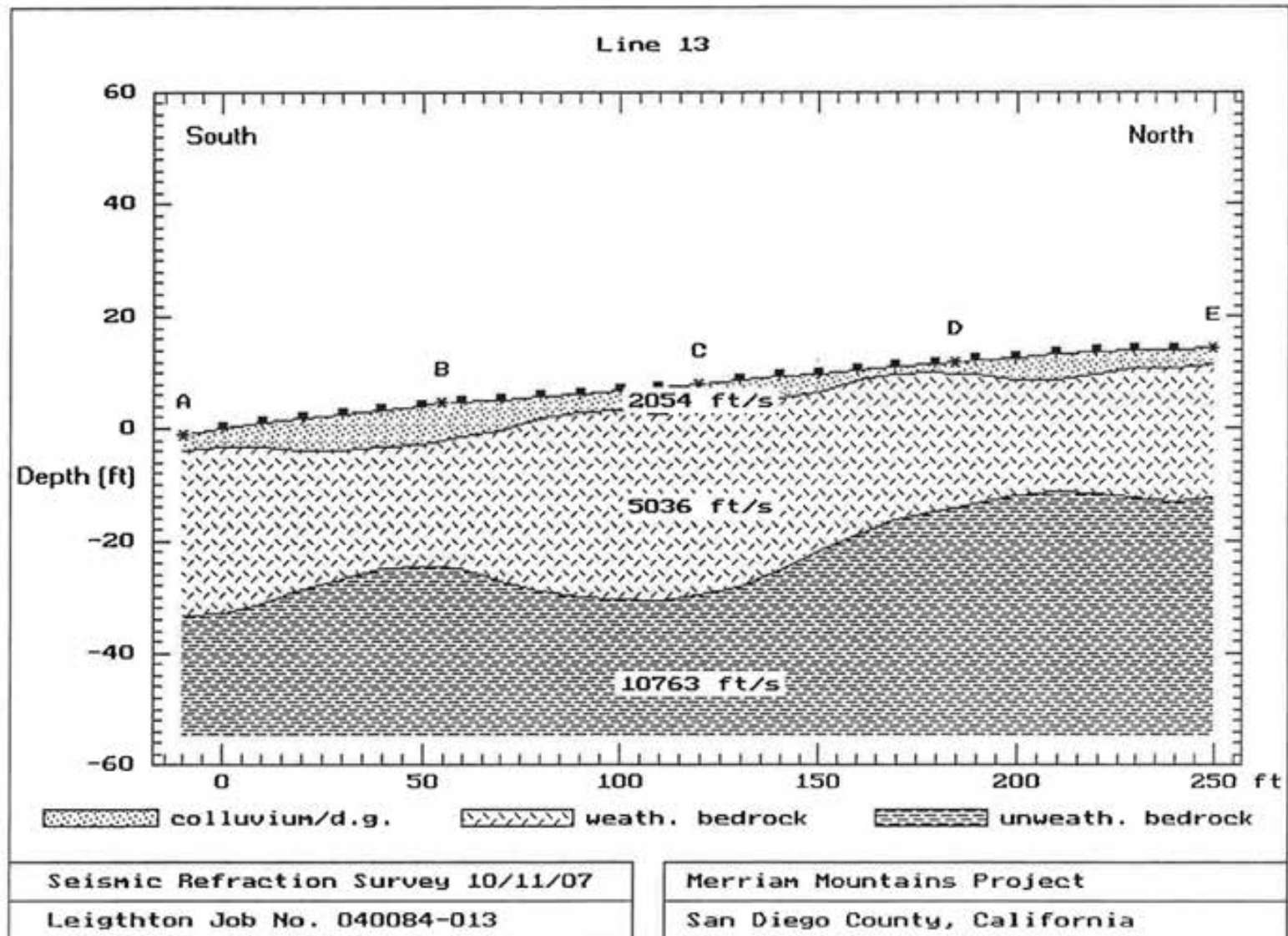
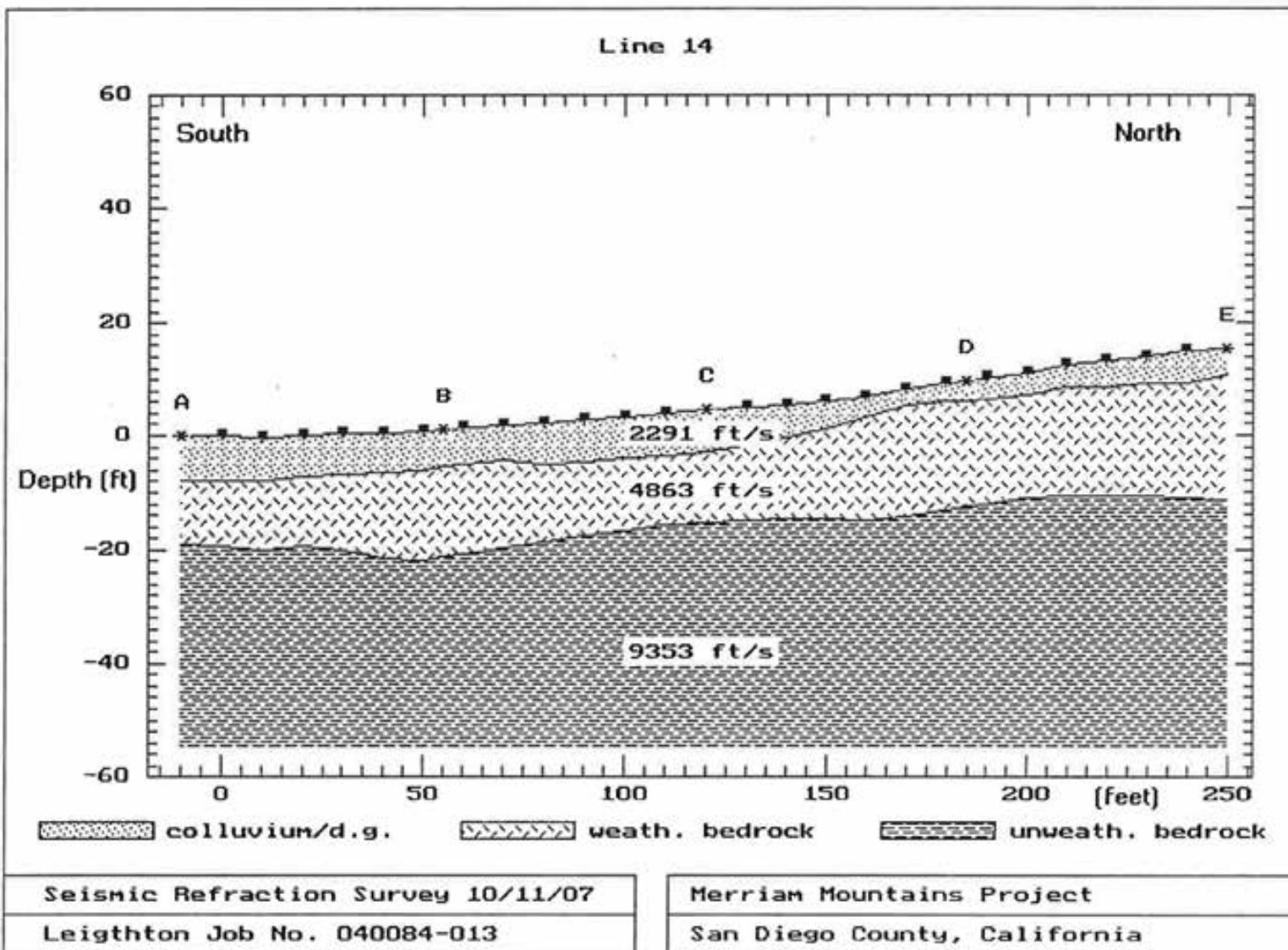
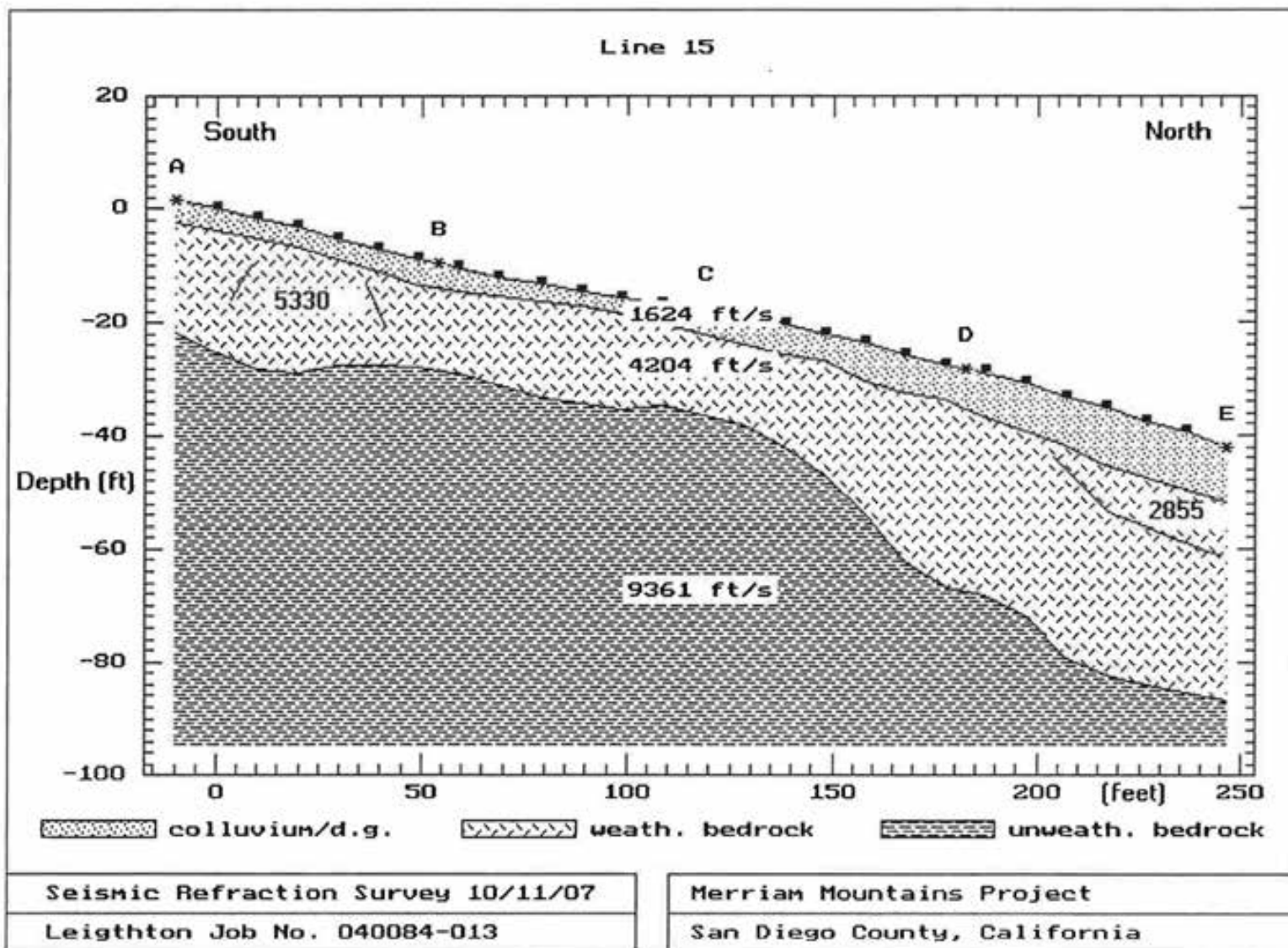


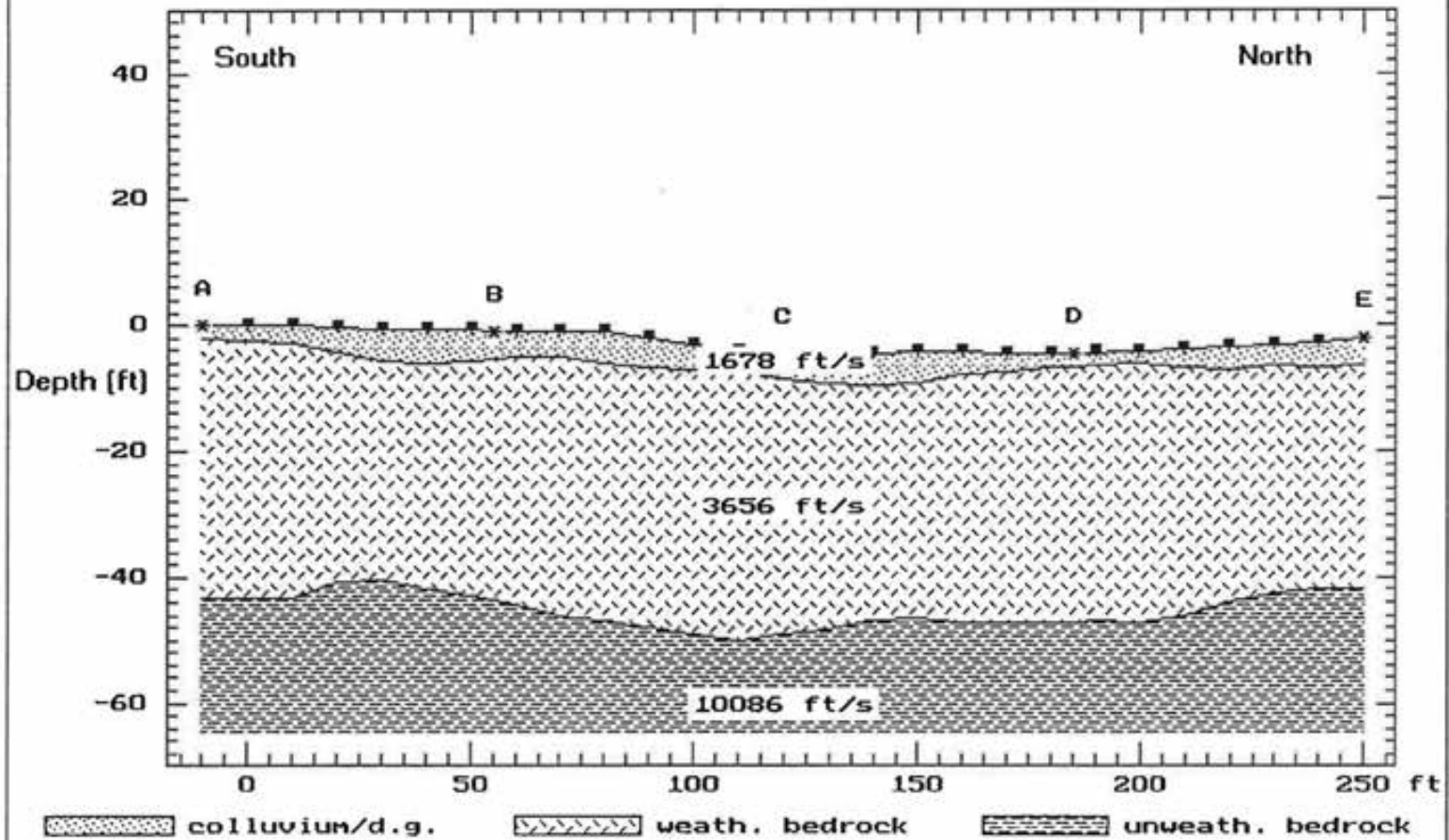
Figure 4. Rippability Chart







Line 16



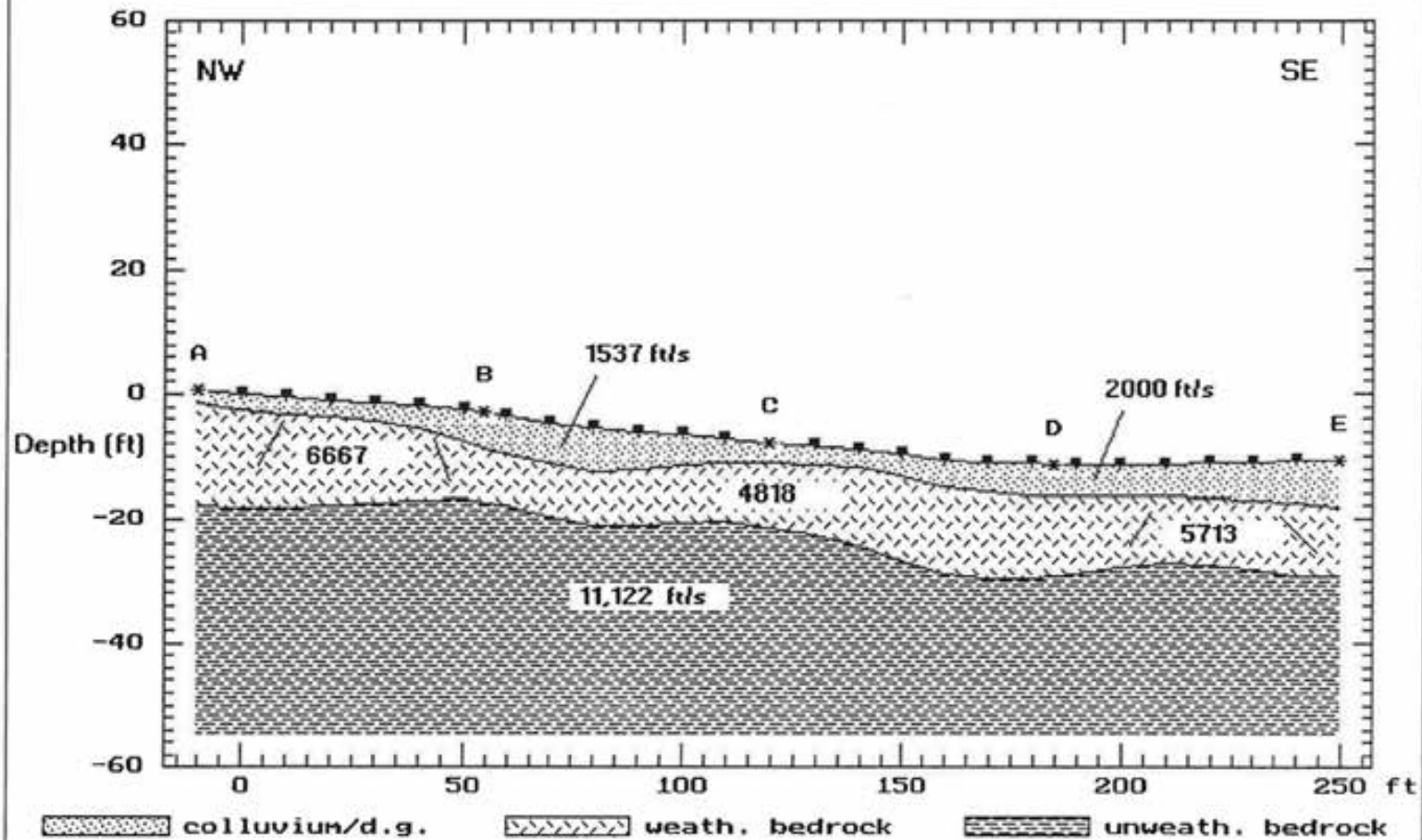
Seismic Refraction Survey 10/11/07

Leighton Job No. 040084-013

Merriam Mountains Project

San Diego County, California

Line 17

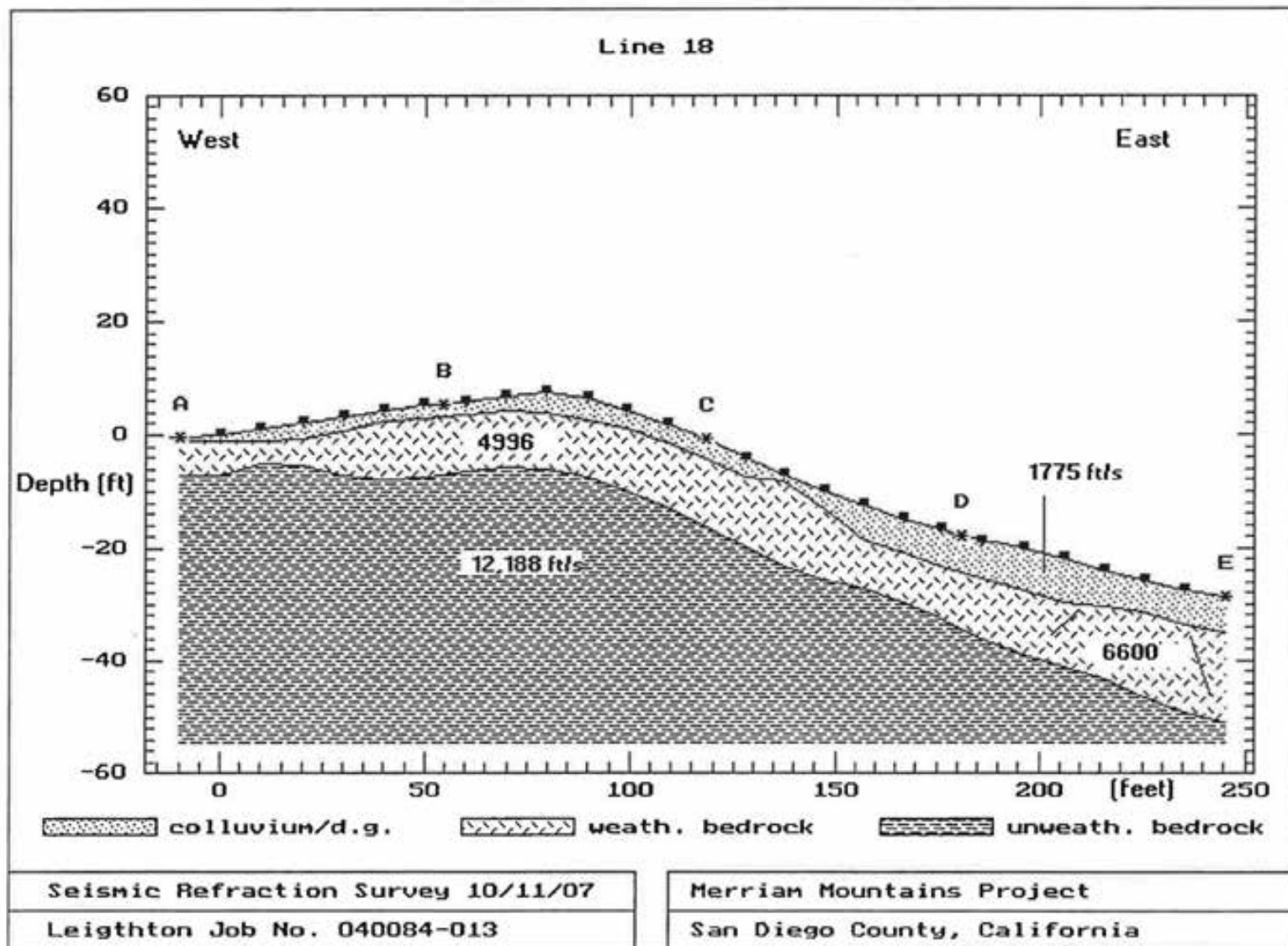


Seismic Refraction Survey 10/11/07

Leighton Job No. 040084-013

Merrian Mountains Project

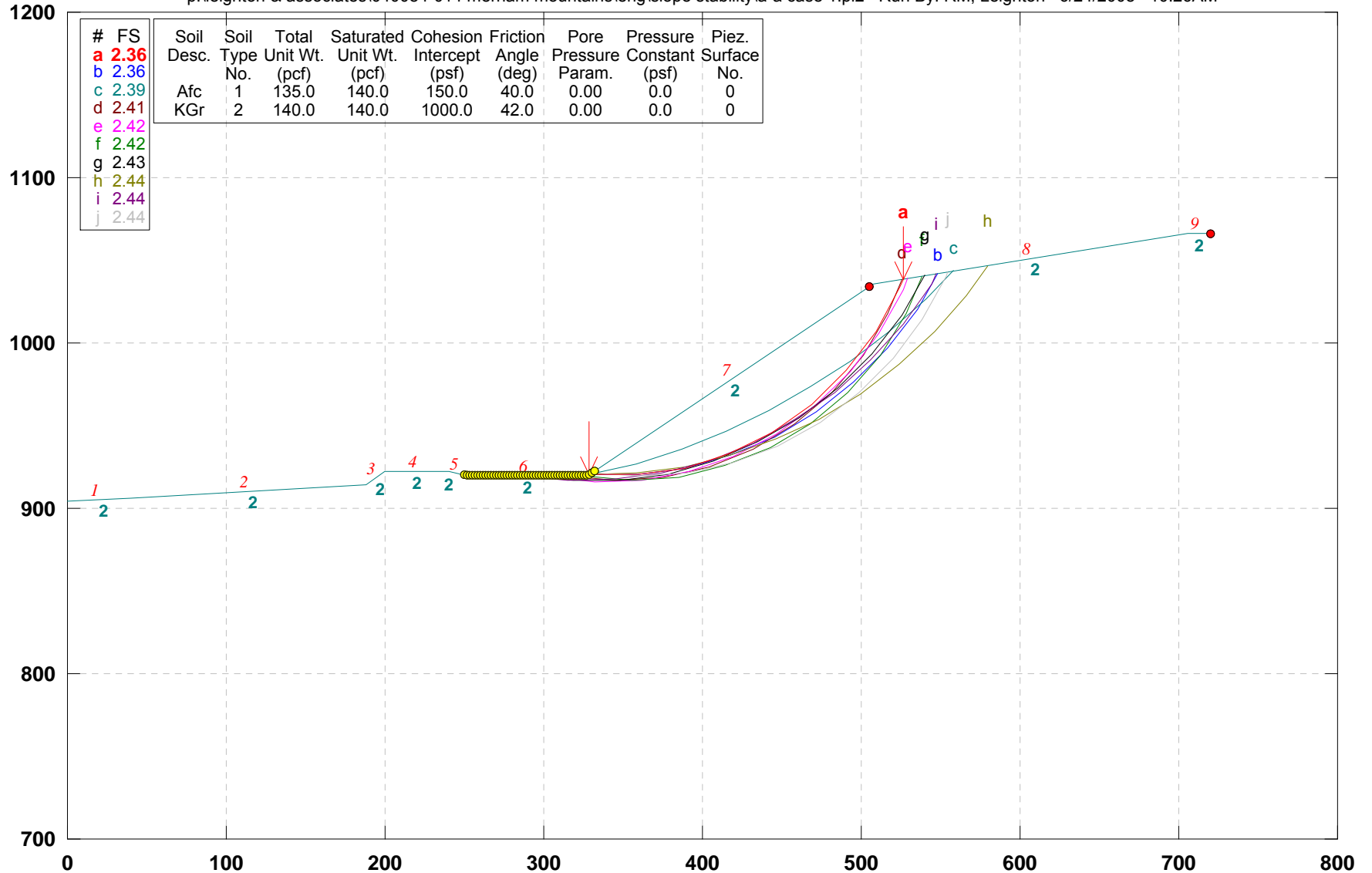
San Diego County, California



APPENDIX E
SLOPE STABILITY

Merriam Mountains - Cross Section A-A Proposed 1:1 Cut Slope Case 1

p:\Veighton & associates\040084-014 merriam mountains\eng\slope stability\A-A case 1.pl2 Run By: RM, Leighton 6/24/2008 10:26AM



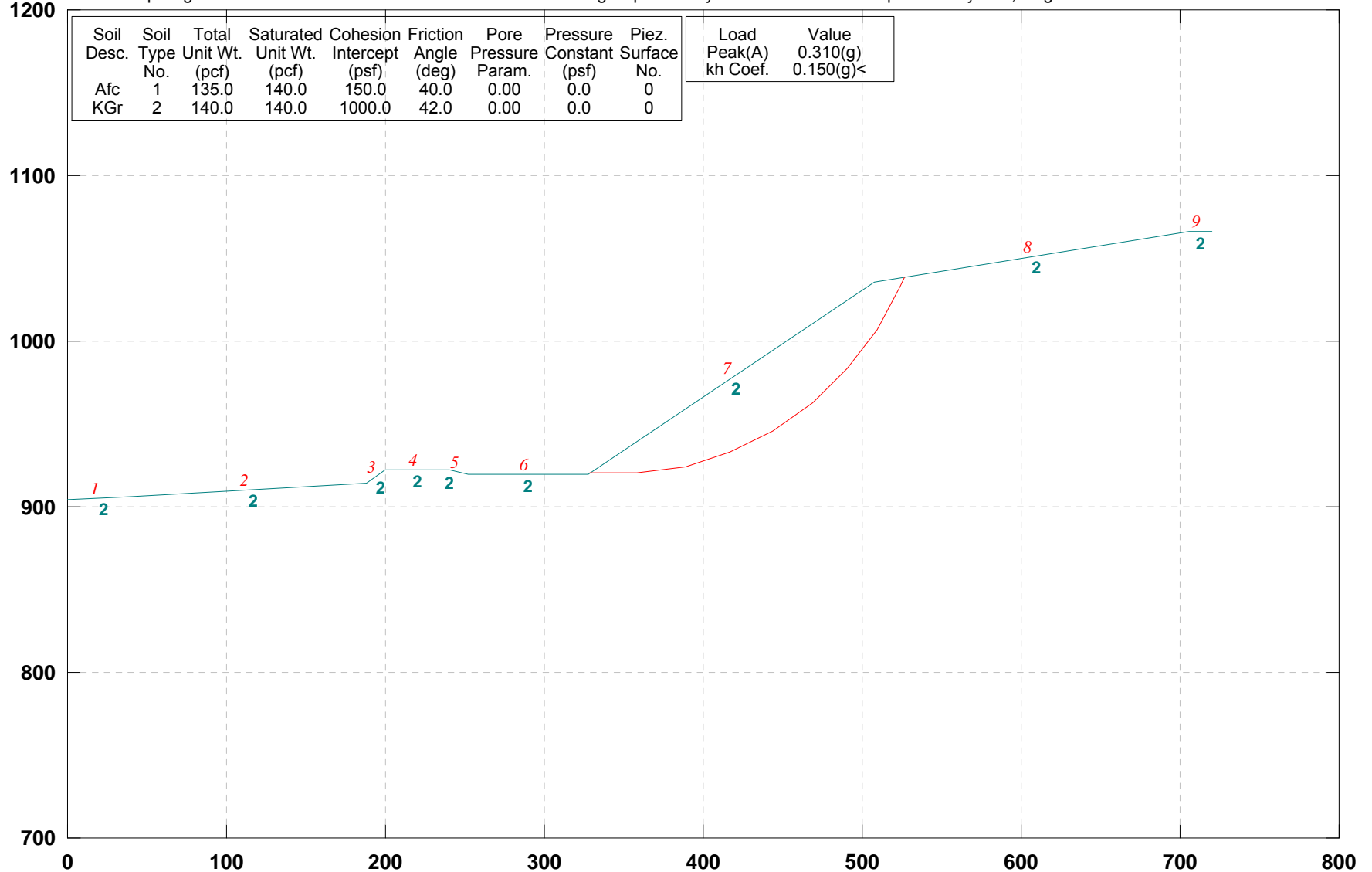
GSTABL7 v.2 FSmin=2.36

Safety Factors Are Calculated By The Modified Bishop Method



Merriam Mountains - Cross Section A-A Proposed 1:1 Cut Slope Case 1

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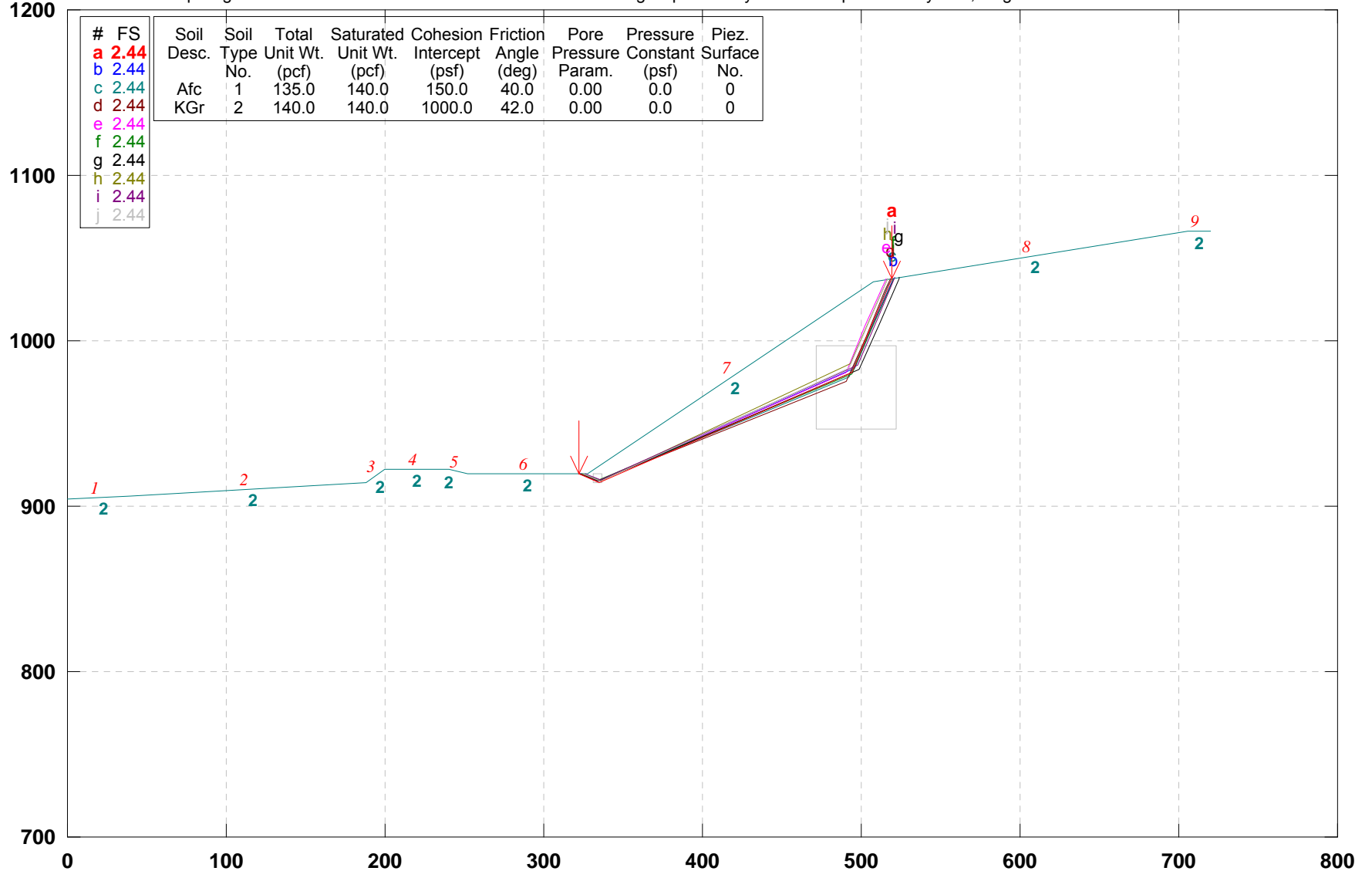
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Factor Of Safety Is Calculated By The Modified Bishop Method



Merriam Mountains - Cross Section A-A Proposed 1:1 Cut Slope Case 2

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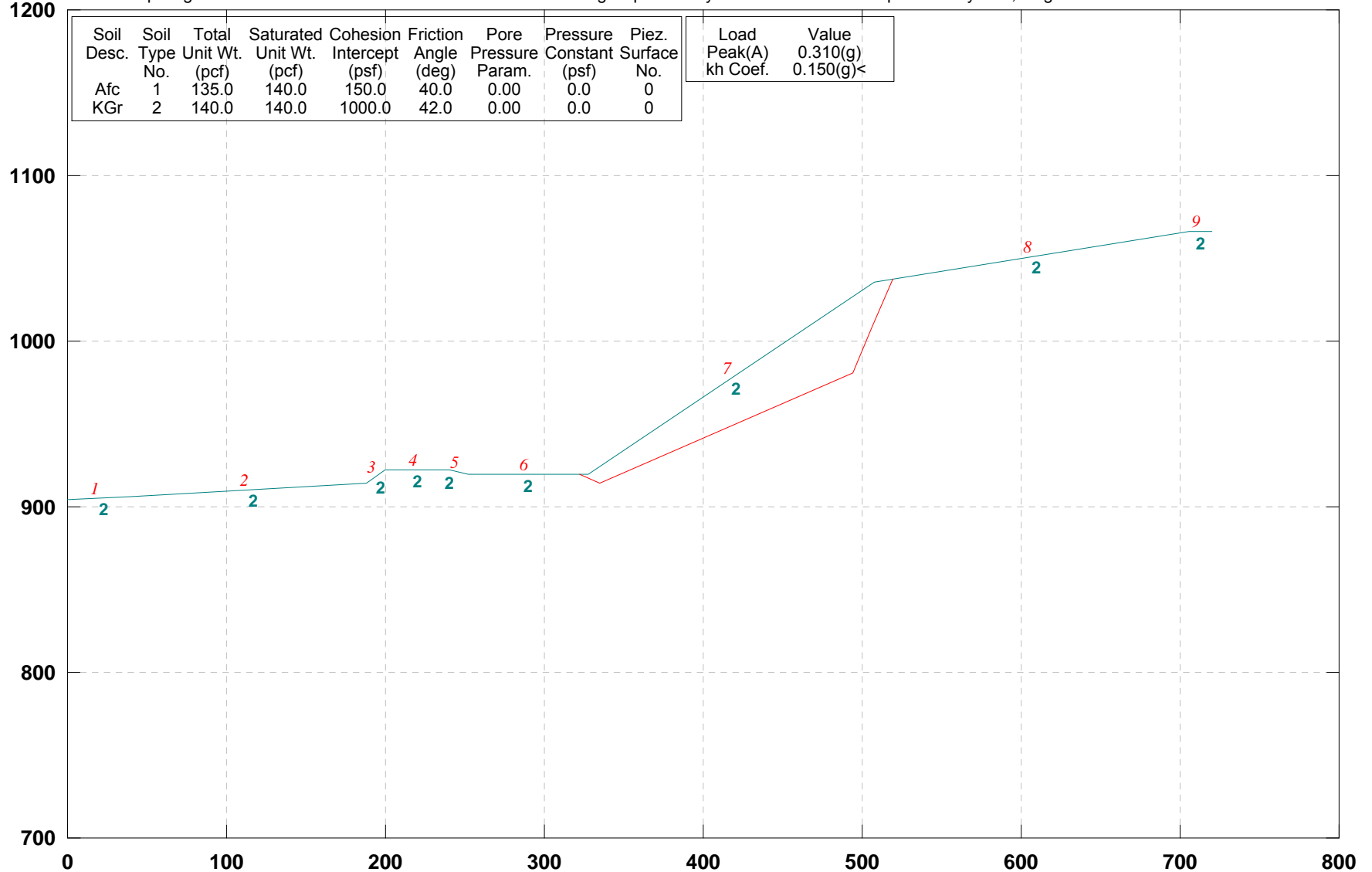
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Safety Factors Are Calculated By The Simplified Janbu Method



Merriam Mountains - Cross Section A-A Proposed 1:1 Cut Slope Case 2

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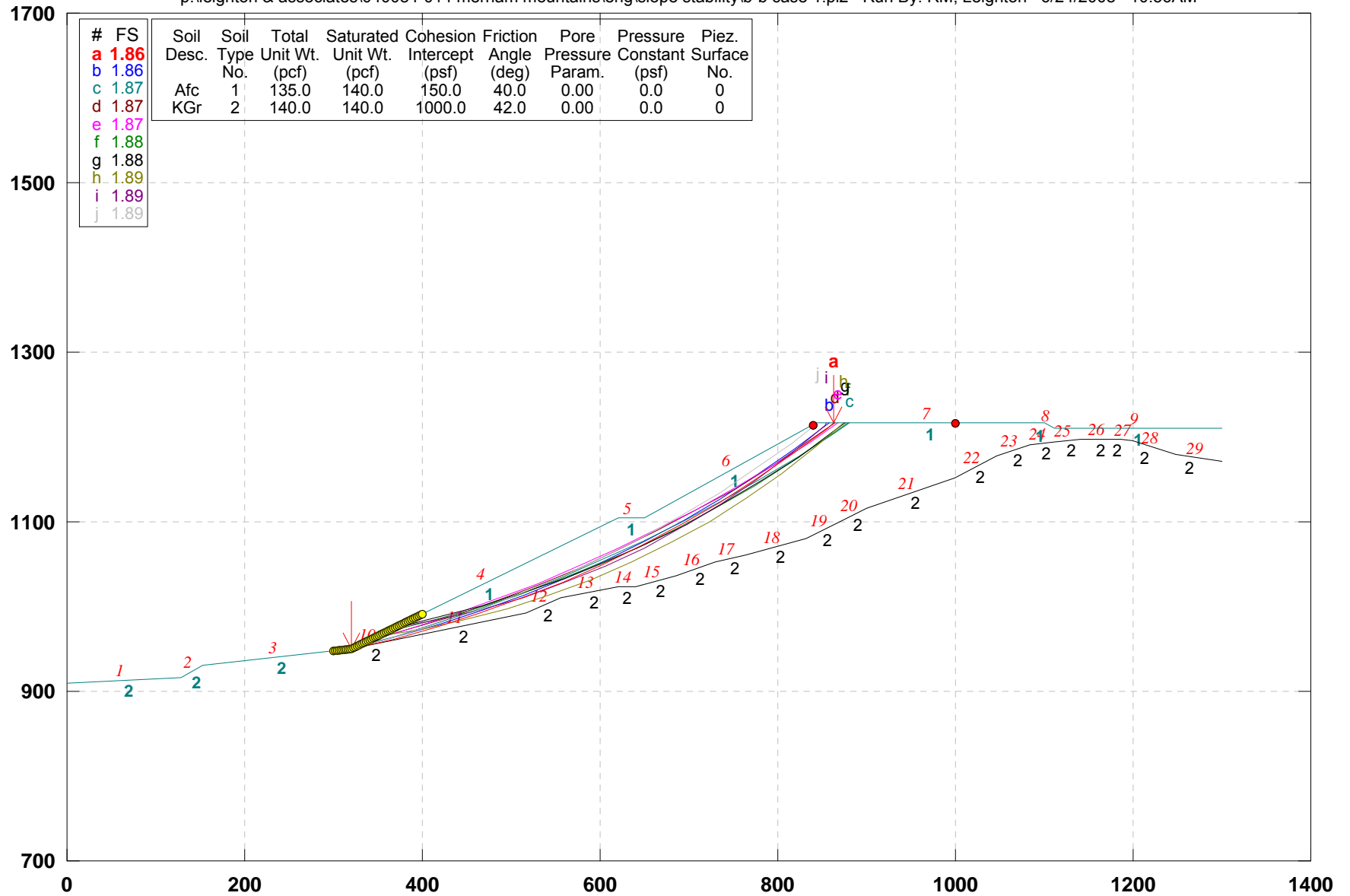
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Factor Of Safety Is Calculated By The Simplified Janbu Method



Merriam Mountains - Cross Section B-B Proposed 1.5:1 Fill Slope

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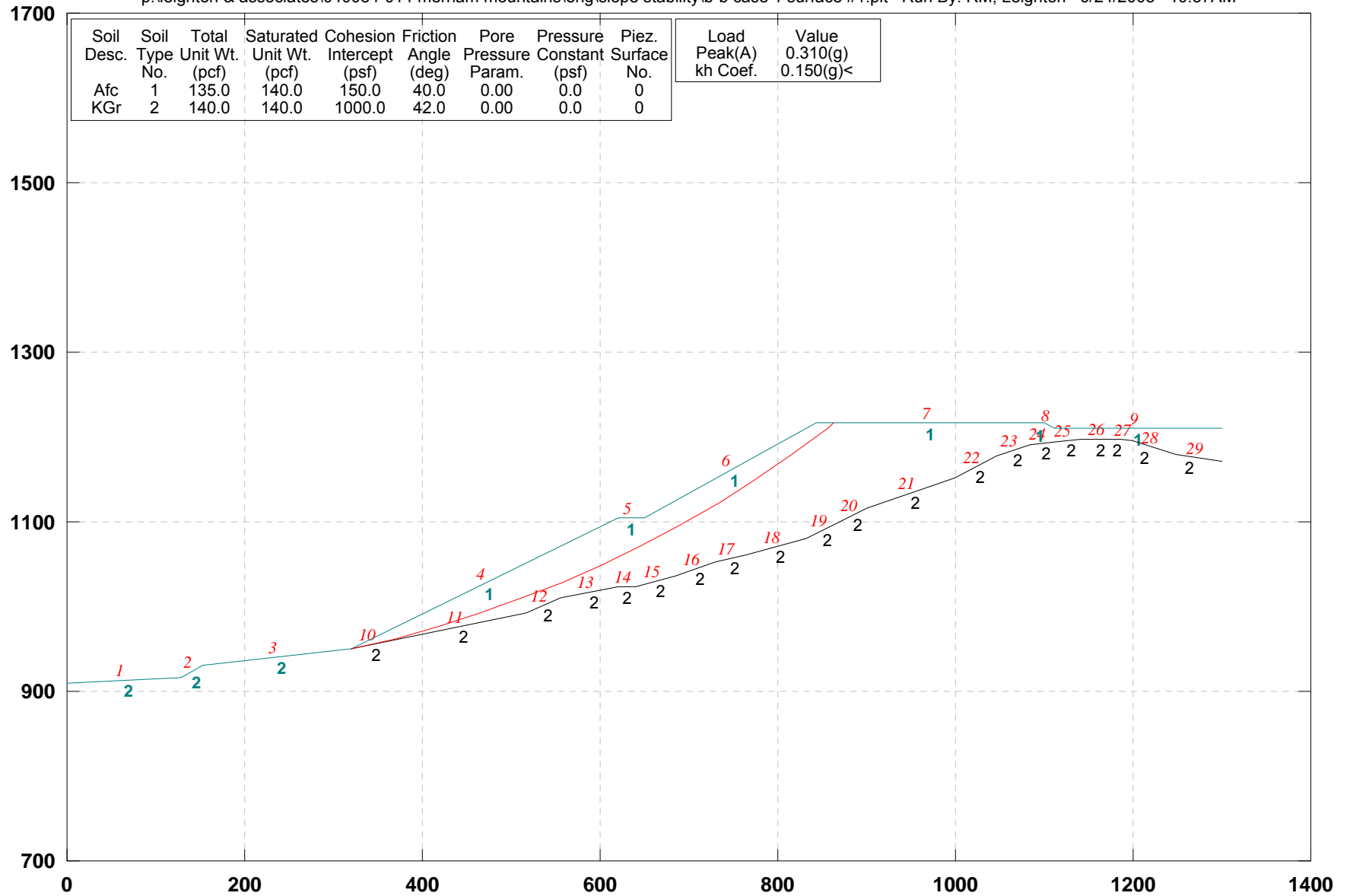
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Safety Factors Are Calculated By The Modified Bishop Method



Merriam Mountains - Cross Section B-B Proposed 1.5:1 Fill Slope

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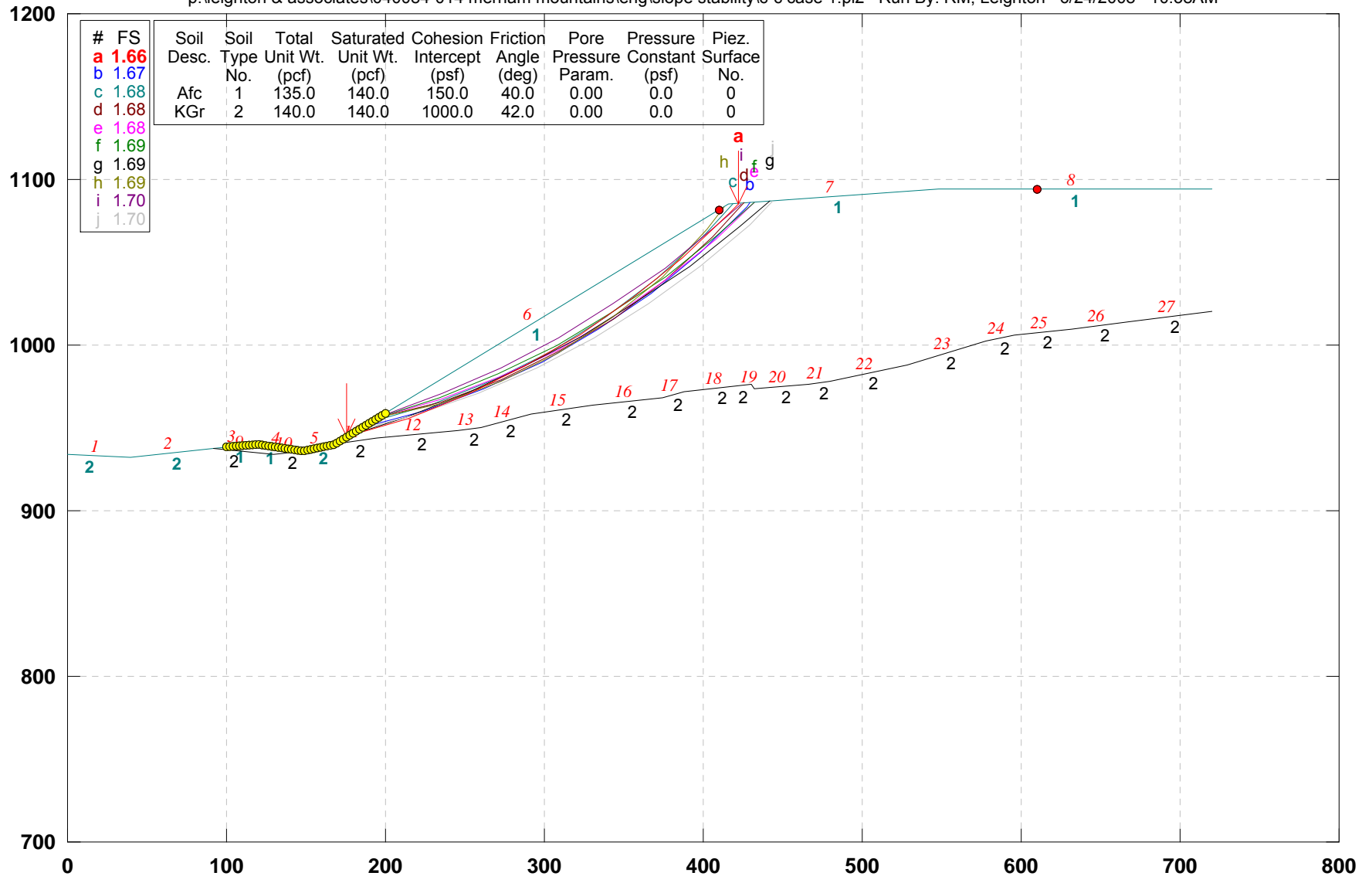
GSTABL7 v.2 FSmin=1.33

Factor Of Safety Is Calculated By The Modified Bishop Method



Merriam Mountains - Cross Section C-C Proposed 1.5:1 Fill Slope

p:\leighton & associates\040084-014 merriam mountains\eng\slope stability\c-c case 1.pl2 Run By: RM, Leighton 6/24/2008 10:38AM



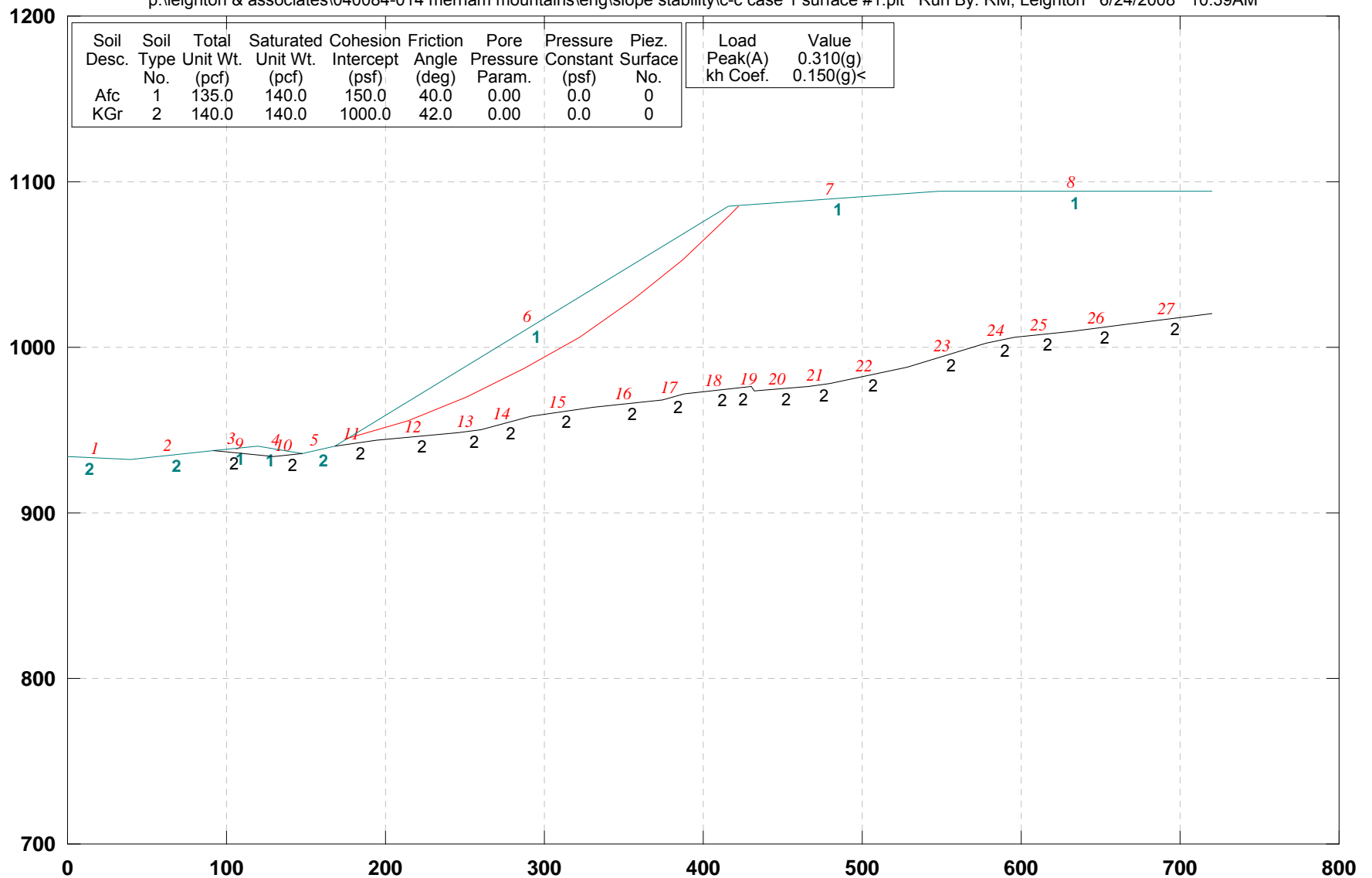
GSTABL7 v.2 FSmin=1.66

Safety Factors Are Calculated By The Modified Bishop Method



Merriam Mountains - Cross Section C-C Proposed 1.5:1 Fill Slope

p:\leighton & associates\040084-014 merriam mountains\eng\slope stability\c-c case 1 surface #1.plt Run By: RM, Leighton 6/24/2008 10:39AM



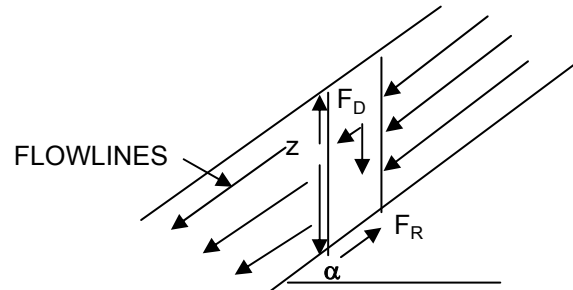
GSTABL7 v.2 FSmin=1.23

Factor Of Safety Is Calculated By The Modified Bishop Method



Proposed cut slope: assuming 3 feet deep saturation

SURFICIAL SLOPE STABILITY ANALYSIS



Project No.: 040084-014

Case: **1:1 Cut Slopes**

Depth of Saturation (ft), Z	=	3
Buoyant Unit Weight of Soil (pcf), γ_b	=	77.6
Total Unit Weight of Soil (pcf), γ_t	=	140
Slope Angle, α	=	45
Angle of Internal Friction, ϕ	=	42
Cohesion (psf), c	=	1000

Force Tending To Cause Movement:

$$F_D = Z\gamma_t \sin 2\alpha / 2 = \mathbf{210.00 \text{ lb/ft}}$$

Force Tending To Resist Movement:

$$F_R = Z\gamma_b \cos^2 \alpha \tan \phi + (c) = \mathbf{1104.81 \text{ lb/ft}}$$

$$F.S. = \frac{2Z\gamma_b \cos^2 \alpha \tan \phi + 2c}{Z\gamma_t \sin 2\alpha}$$

$$F.S. = \mathbf{5.3}$$

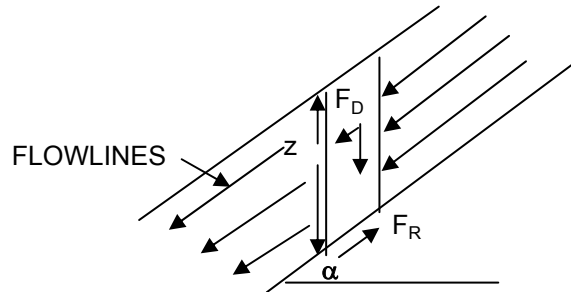
SURFICIAL STABILITY

Project Name : Merriam Mountains
Project Number : 040084-014



Proposed fill slope: assuming 3 feet deep saturation

SURFICIAL SLOPE STABILITY ANALYSIS



Project No.: 040084-014

Case: **1.5:1 Fill Slopes**

Depth of Saturation (ft), Z	=	3
Buoyant Unit Weight of Soil (pcf), γ_b	=	72.6
Total Unit Weight of Soil (pcf), γ_t	=	135
Slope Angle, α	=	33.7
Angle of Internal Friction, ϕ	=	40
Cohesion (psf), c	=	150

Force Tending To Cause Movement:

$$F_D = Z\gamma_t \sin 2\alpha / 2 = 186.95 \text{ lb/ft}$$

Force Tending To Resist Movement:

$$F_R = Z\gamma_b \cos^2 \alpha \tan \phi + (c) = 276.49 \text{ lb/ft}$$

$$F.S. = \frac{2Z\gamma_b \cos^2 \alpha \tan \phi + 2c}{Z\gamma_t \sin 2\alpha}$$

$$F.S. = 1.5$$

SURFICIAL STABILITY

Project Name : Merriam Mountains
Project Number : 040084-014



APPENDIX F

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

FOR ROUGH GRADING

1.0 General

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

LEIGHTON AND ASSOCIATES, INC.
General Earthwork and Grading Specifications

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical

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Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 **Evaluation/Acceptance of Fill Areas**

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 **Fill Material**

3.1 **General**

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 **Oversize**

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 **Import**

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to

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General Earthwork and Grading Specifications

inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 ($SE > 30$). The bedding shall be placed to 1 foot over the top of the conduit and densified. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

7.4 Observation and Testing

The densification of the bedding around the conduits shall be observed by the Geotechnical Consultant.

SLOPE

PROJECTED PLANE 1:1
(HORIZONTAL: VERTICAL)
MAXIMUM FROM TOE
OF SLOPE TO
APPROVED GROUND

EXISTING
GROUND SURFACE

2% MIN.

2 FEET MIN.
KEY DEPTH

15 FEET MIN.
LOWEST
BENCH (KEY)

BENCH

BENCH HEIGHT
(4 FEET TYPICAL)

COMPACTED
FILL

REMOVE
UNSUITABLE
MATERIAL

Diagram illustrating a road cut and fill construction detail. The diagram shows a cross-section of a road cut and fill area. Key features and labels include:

- EXISTING GROUND SURFACE:** Indicated by a dashed line.
- 2% MIN.:** Minimum slope requirement for the cut face.
- 15 FEET MIN. LOWEST BENCH (KEY):** Minimum width of the lowest bench.
- 2 FEET MIN. KEY DEPTH:** Minimum depth of the key.
- CUT FACE SHALL BE CONSTRUCTED PRIOR TO FILL PLACEMENT TO ALLOW VIEWING OF GEOLOGIC CONDITIONS:** Instruction for construction sequence.
- COMPACTED FILL:** Material placed above the cut face.
- BENCH:** A horizontal step in the fill.
- BENCH HEIGHT (4 FEET TYPICAL):** Typical height of a bench.
- REMOVE UNSUITABLE MATERIAL:** Instruction for material removal.

CUT-OVER-FILL SLOPE

EXISTING GROUND SURFACE

CUT FACE SHALL BE CONSTRUCTED PRIOR TO FILL PLACEMENT

OVERBUILD AND TRIM BACK

DESIGN SLOPE

COMPACTED FILL

REMOVE UNSUITABLE MATERIAL

PROJECTED PLANE 1 TO 1 MAXIMUM FROM TOE OF SLOPE TO APPROVED GROUND

2% MIN.

2 FEET MIN. KEY DEPTH

15 FEET MIN. LOWEST BENCH (KEY)

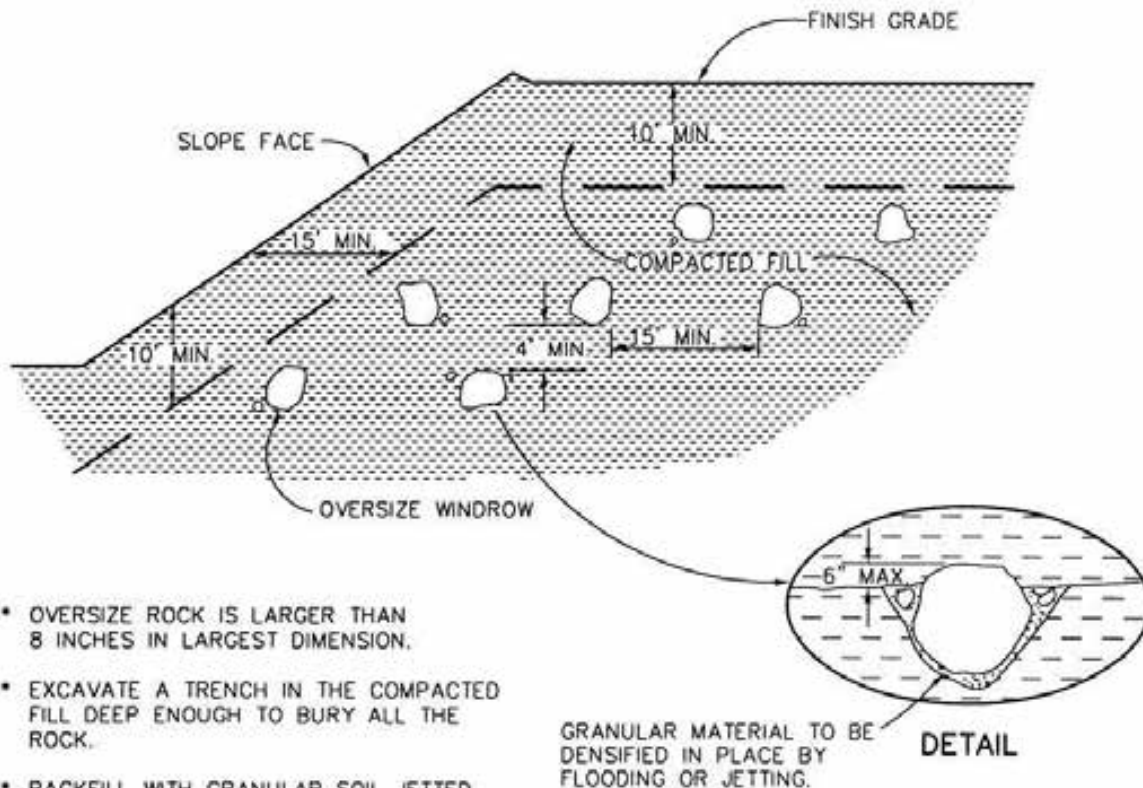
BENCH

BENCH HEIGHT (4 FEET TYPICAL)

BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

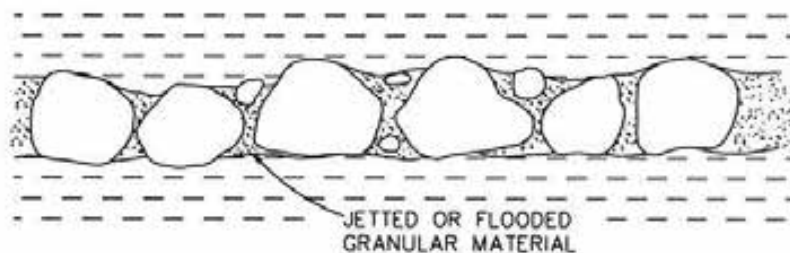
GENERAL EARTH WORK AND GRADING SPECIFICATIONS STANDARD DETAIL A





- OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
- DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- WINDROW OF BURIED ROCK SHALL BE PARALLEL TO THE FINISHED SLOPE.

GRANULAR MATERIAL TO BE DENSIFIED IN PLACE BY FLOODING OR JETTING.

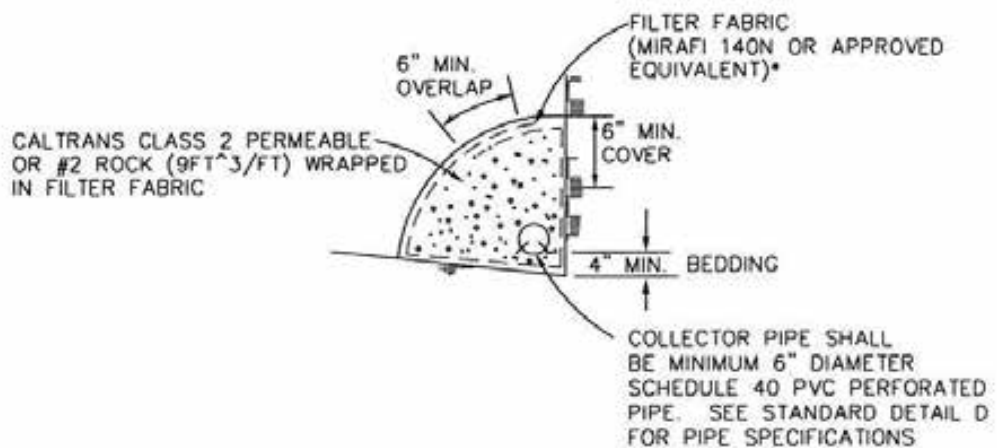
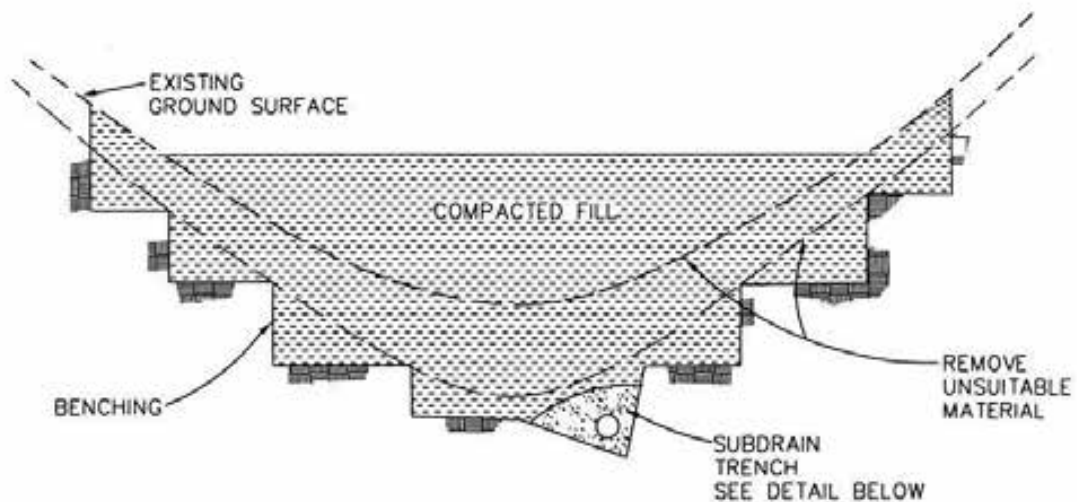


TYPICAL PROFILE ALONG WINDROW

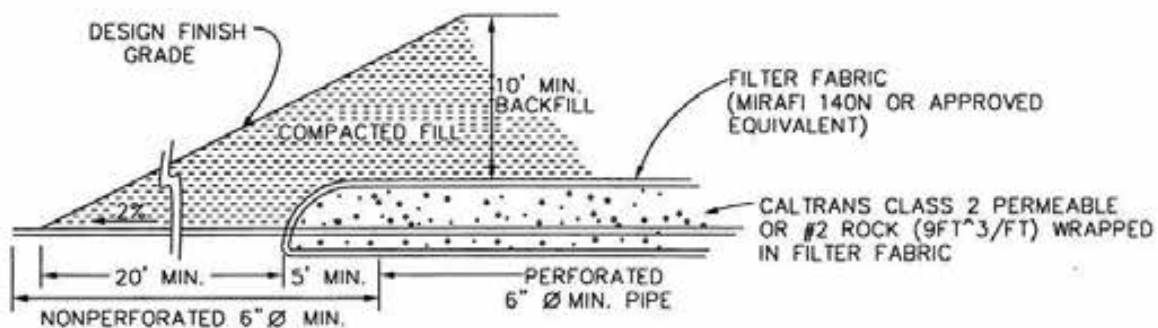
OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL B





SUBDRAIN DETAIL

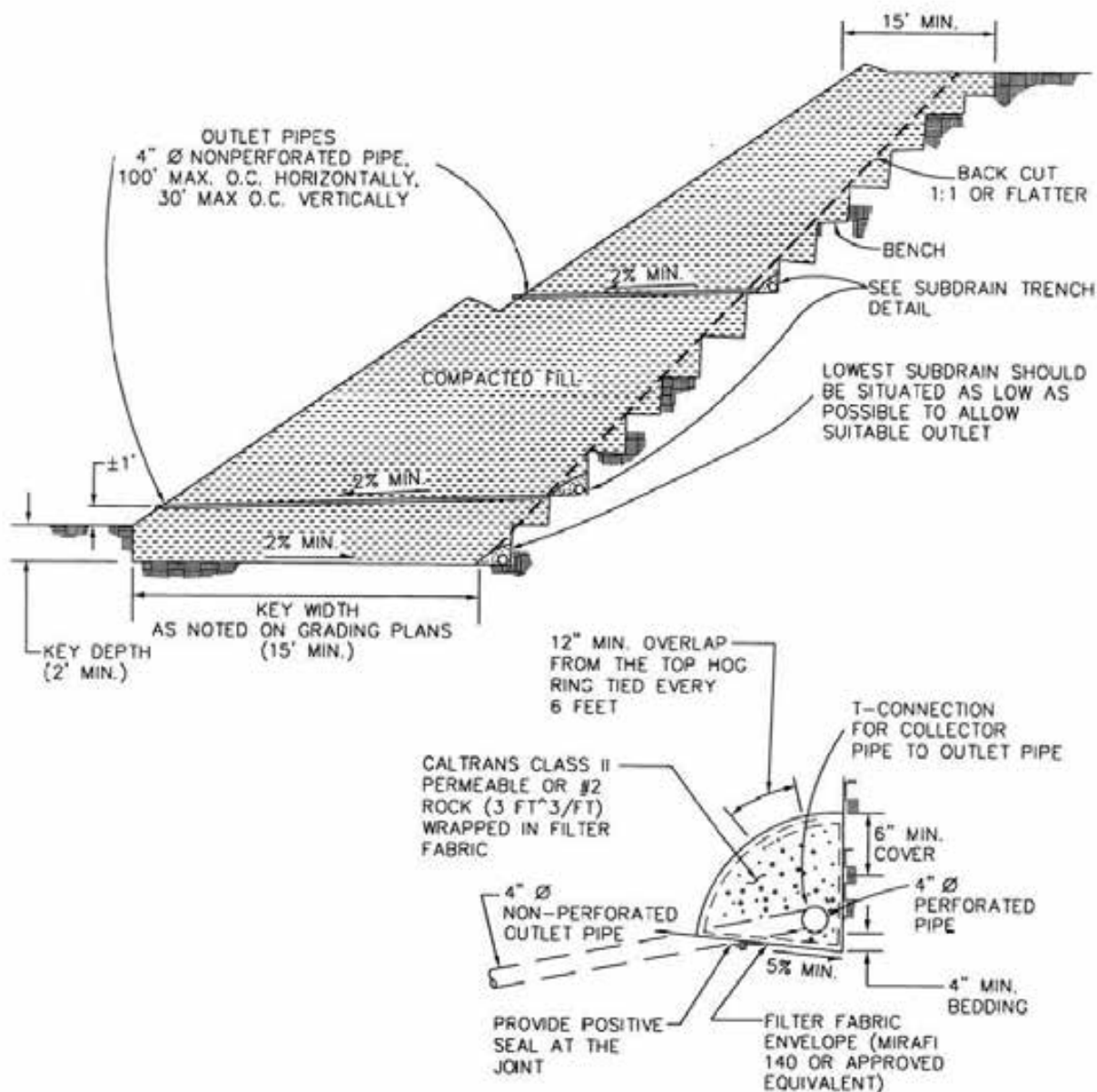


DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL C





SUBDRAIN TRENCH DETAIL

SUBDRAIN INSTALLATION – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drill holes are used. All subdrain pipes shall have a gradient of at least 2% towards the outlet.

SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

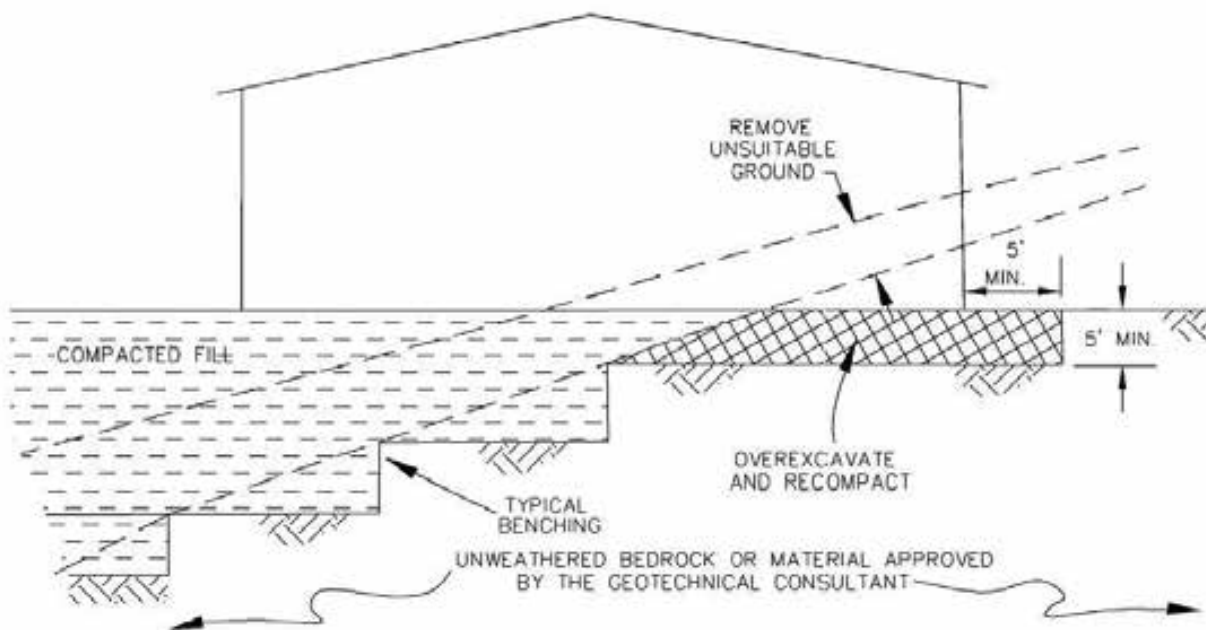
All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

BUTTRESS OR
REPLACEMENT
FILL SUBDRAINS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL D



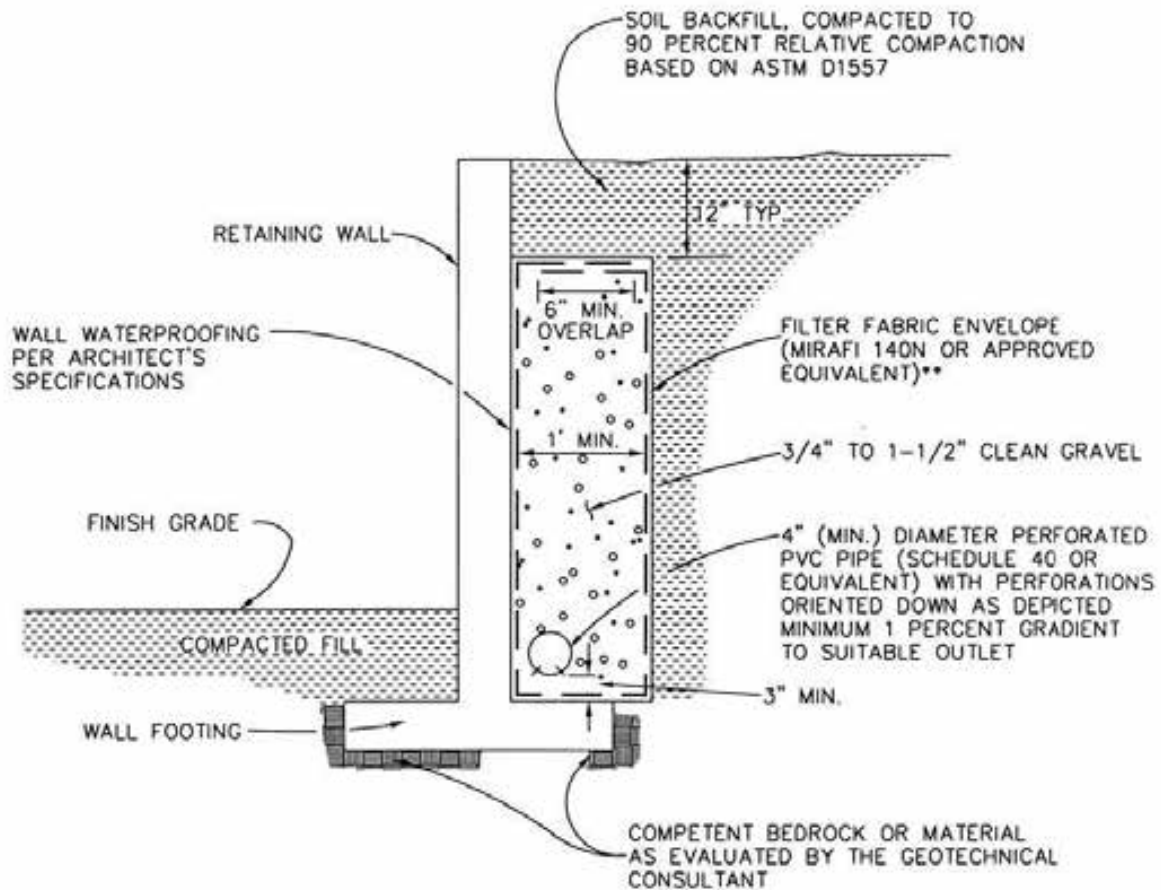
CUT-FILL TRANSITION LOT OVEREXCAVATION



TRANSITION LOT FILLS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E



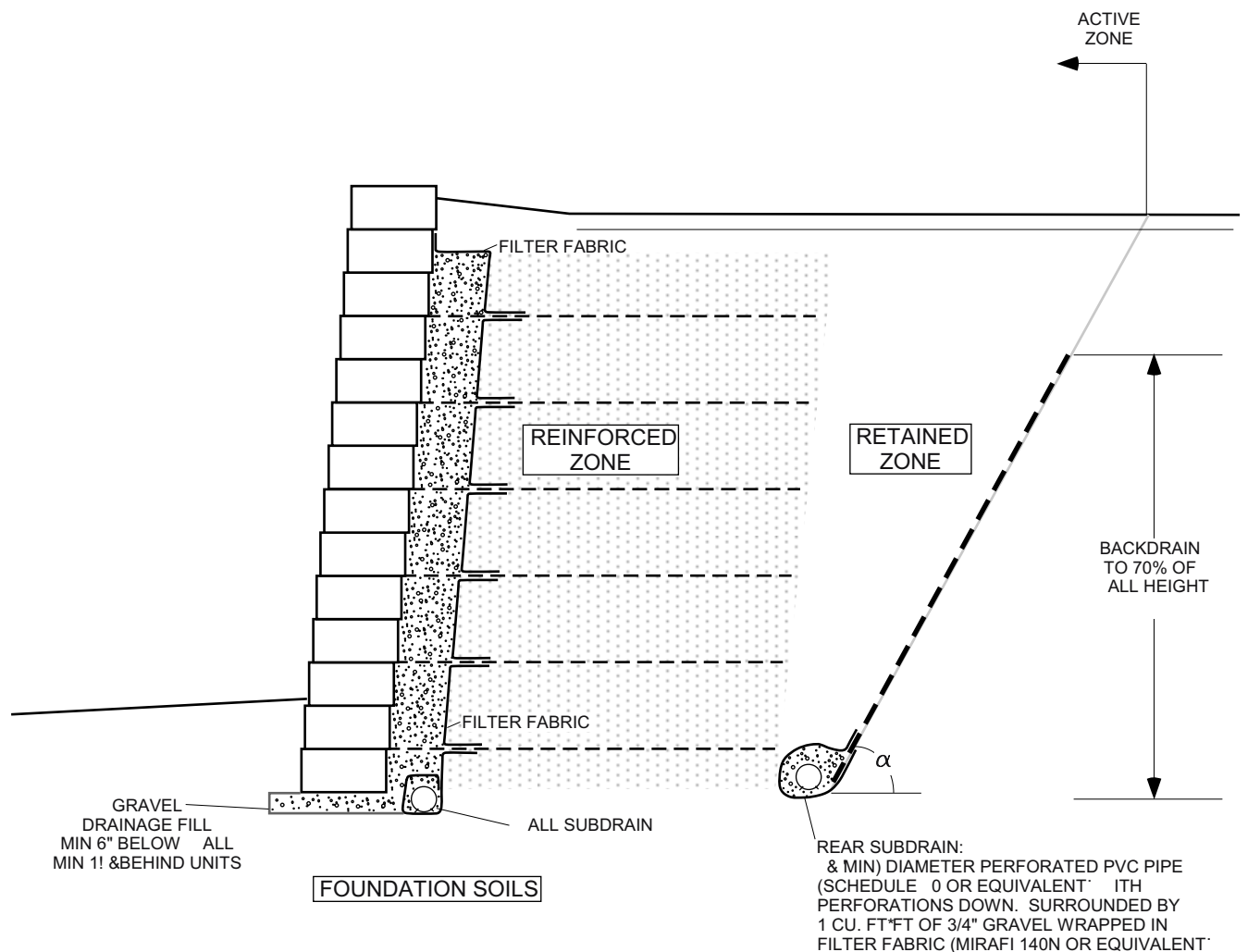


NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.

RETAINING WALL
DRAINAGE

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL F





NOTES

MATERIAL GRADATION AND PLASTICITY INDEX

REINFORCED ZONE:

SIEVE SIZE	% PASSING
1/2" INCH	100
NO. 4	20-100
NO. 40	0-60
NO. 200	0-5%

FOR WALL HEIGHT / 10 FEET, PLASTICITY INDEX / 20

FOR WALL HEIGHT 10 TO 20 FEET, PLASTICITY INDEX / 10

FOR TIERED WALLS, USE COMBINED WALL HEIGHTS

ALL DESIGNER TO REQUEST SITE-SPECIFIC CRITERIA FOR WALL HEIGHT 0 20 FEET

GRAVEL DRAINAGE FILL:

SIEVE SIZE	% PASSING
1/2" INCH	100
3/4" INCH	75-100
NO. 4	0-60
NO. 40	0-50
NO. 200	0-5%

1. CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH REQUIREMENTS OF WALL DESIGN.

2. GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL+EXTERNAL+AND COMPOUND STABILITY

3. GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

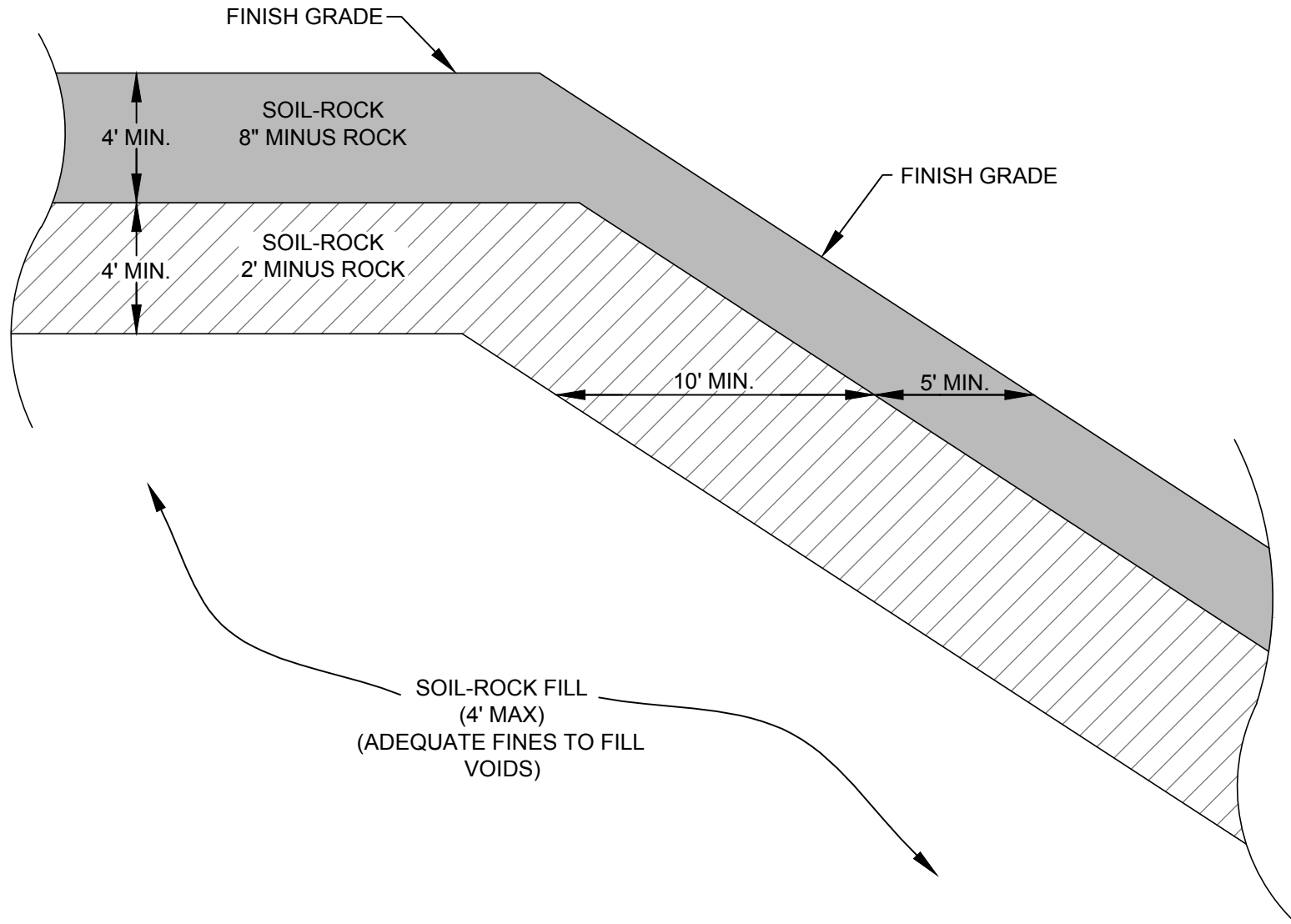
4. IMPROVEMENTS WITHIN THE ACTIVE ZONE ARE SUSCEPTIBLE TO POST-CONSTRUCTION SETTLEMENT. ANGLE $\alpha = \phi - \phi'$ HERE ϕ IS THE FRICTION ANGLE OF THE MATERIAL IN THE RETAINED ZONE.

5. BACKDRAIN SHOULD CONSIST OF J-DRAIN 302 (OR EQUIVALENT) OR 6-INCH THICK DRAINAGE FILL WRAPPED IN FILTER FABRIC. PERCENT COVERAGE OF BACKDRAIN TO BE PER GEOTECHNICAL REVIEW

SEGMENTAL RETAINING WALLS

GENERAL EARTH WORK AND GRADING SPECIFICATIONS STANDARD DETAIL G





NOT TO SCALE

Project: 10618.002	Eng/Geol: WDO/RCS
Scale: NTS	Date: November 2015
Reference:	
Author: MAM	

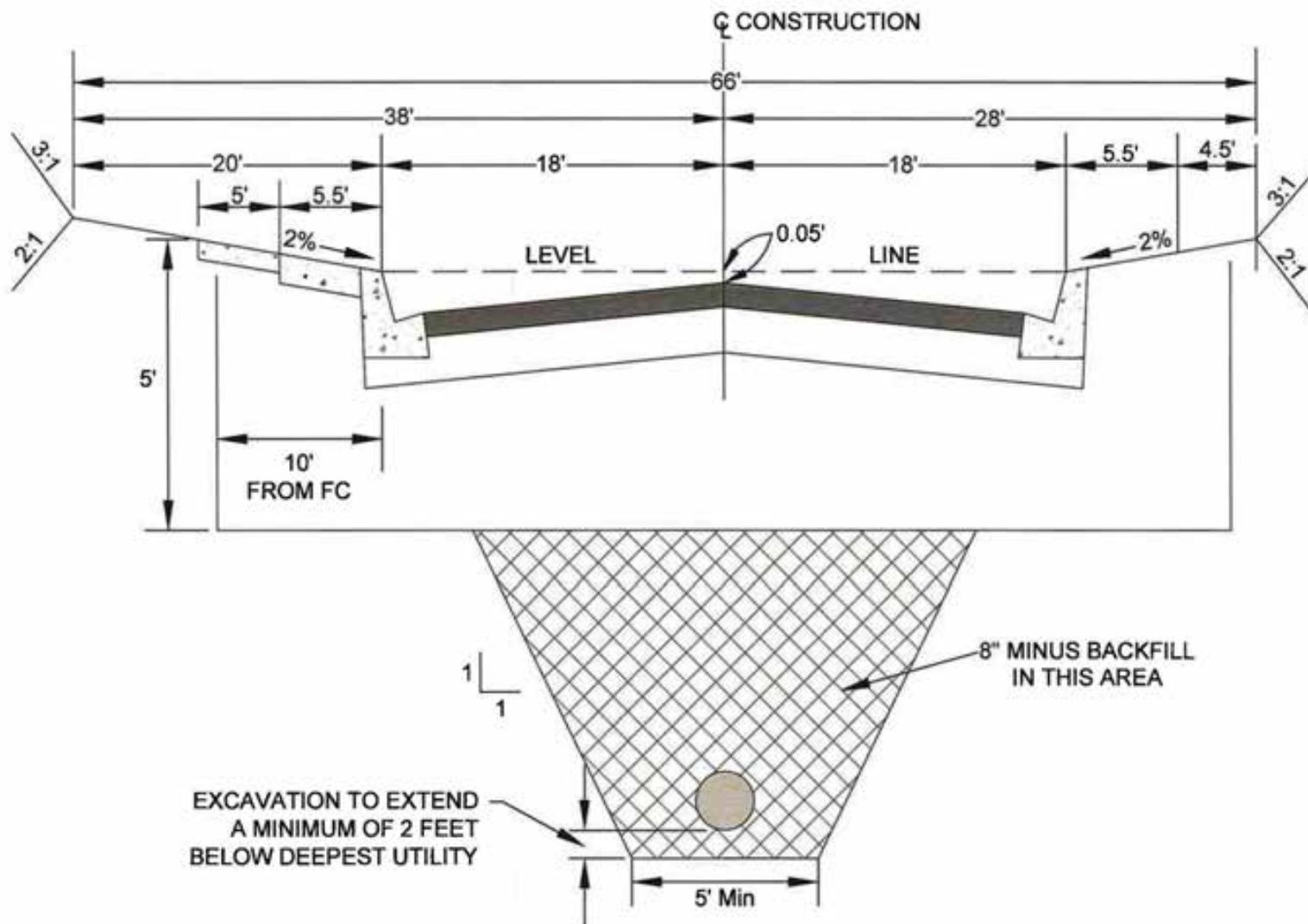
TYPICAL SOIL ROCK FILL DETAIL

Newland Sierra
San Diego, California

Figure A



Leighton



TAKE CARE TO AVOID
OVER-EXCAVATION OF
ADJACENT CUT
SLOPES IF PRESENT

NOT TO SCALE

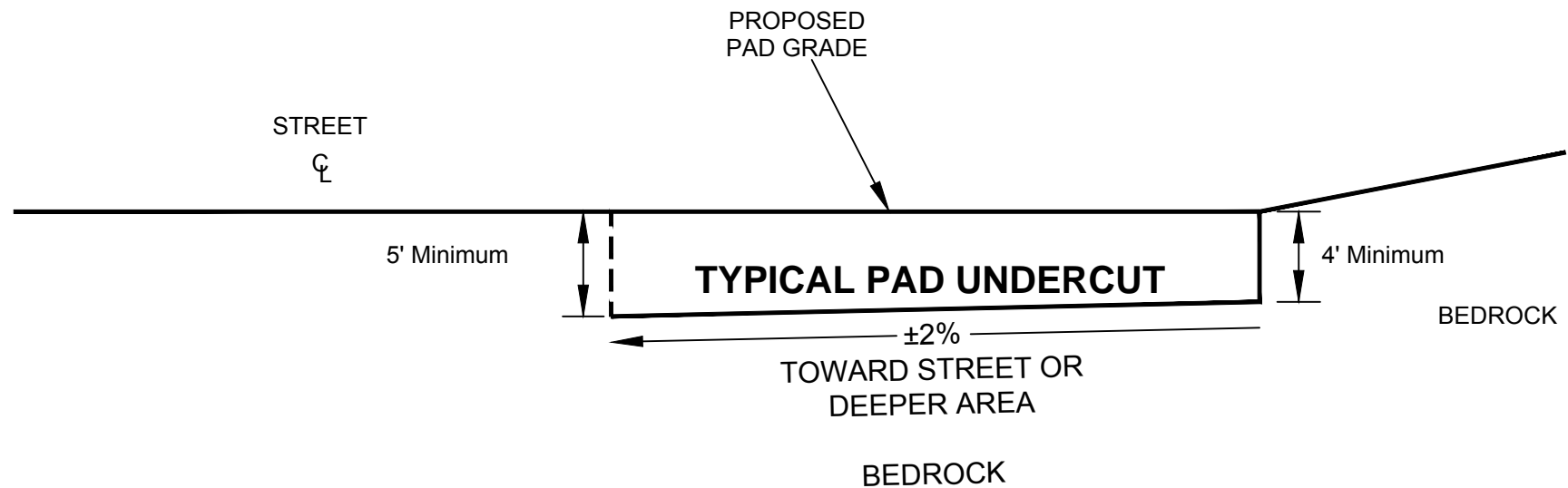
Project: 10618.002	Eng/Geol: WDO/RCS
Scale: NTS	Date: November 2015
Reference:	
Author: MAM	

TYPICAL STREET UTILITY UNDERCUT DETAIL

Newland Sierra
San Diego, California

Figure B





NOT TO SCALE

Project: 10618.002 Eng/Geol: WDO/RCS

Scale: NTS Date: November 2015

Reference:

Author: MAM

TYPICAL DETAIL FOR PAD UNDERCUT IN BEDROCK AREAS

Newland Sierra
San Diego, California

Figure C



Leighton