GEOTECHNICAL INVESTIGATION

STEELE CANYON ROAD AT P.M. 2.25
NAPA COUNTY, CALIFORNIA

Prepared for:

Napa County
Owner

and

Winzler & Kelly
Consulting Engineers
Design Engineer

Prepared by:
Blackburn Consulting

July 25, 2007

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Geotechnical • Construction Services • Forensics

BCI File No. 1223.1
July 25, 2007

Mr. Steve McHaney
Winzler & Kelly Consulting Engineers
633 Third Street
Eureka, California 95501

Subject: **Geotechnical Investigation**
Steele Canyon Road at P.M. 2.25
Napa County, California

Dear Mr. McHaney:

Blackburn Consulting (BCI) prepared this Geotechnical Investigation report for the 2006 storm damage on Steele Canyon Road located at P.M. 2.25, in accordance with our agreement dated April 25 2007.

Please call if you have questions on this report or require additional information. We appreciate this opportunity to be of service.

Sincerely,

BLACKBURN CONSULTING

W. Eric Nichols, C.E.G.
Project Manager

PROFESSIONAL GEOLOGIST
WILLIAM E. NICHOLS
NO. 2229
CERTIFIED ENGINEERING GEOLOGIST
EXP 1-31-08
STATE OF CALIFORNIA

Reviewed by:

Rick Sowers, P.E., C.E.G.
Senior Project Manager

Distribution: Client (4)

GEOTECHNICAL INVESTIGATION

STEELE CANYON ROAD SLIDE AT P.M. 2.25
Napa County, California

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1 PURPOSE

Blackburn Consulting (BCI) prepared this Geotechnical Report for the storm damage site at Steele Canyon Road, P.M. 2.25, in accordance with our April 25, 2007 agreement. This report includes the results of our subsurface investigation, opinion regarding the cause of road failure, and repair recommendations.

We prepared this report to provide geotechnical criteria for Winzler & Kelly Consulting Engineers to use during mitigation design. Do not use or rely upon this report for different locations or improvements without the written consent of BCI.

2 SCOPE OF SERVICES

To prepare this report, BCI:

- Reviewed published geologic mapping and topographic mapping.
- Performed geologic reconnaissance of the site.
- Drilled and sampled two exploratory test borings from road level.
- Completed a seismic refraction line along the outside roadway shoulder.
- Prepared an exploration summary memorandum dated June 13, 2007.
- Performed laboratory testing and geotechnical engineering evaluation in support of the recommendations contained herein.

3 PROJECT LOCATION AND SITE DESCRIPTION

3.1 Project Location

The project is located on Steele Canyon Road, approximately 2.25 miles north of its intersection with Highway 128 in Napa County, California. Site latitude is approximately N38.46772° and longitude is approximately W122.18174°. We show the project location on Figure 1.

3.2 Site Description

At P.M. 2.25, Steele Canyon Road traverses a west-facing slope and is a paved, two-lane section, about 24 to 28 feet wide. At this location, the roadway is established in a 25 foot high cut section on the inboard (east) side, with 10 to 15 feet of fill on the outboard (west) side. Profile grade descends northerly on a steep (10%) grade. Surface runoff flows north along an unpaved inner road ditch.

In the vicinity of the slipout, the slope below the road is at about 1.5H:1V (horizontal:vertical) gradient for a slope distance of about 50 to 55 feet, then transitions to about a 2H:1V slope below that level. The cut-slope above the road is at about 1:1 and exposes very intensely weathered and fractured mudstone/claystone.

The slipout occurred below the road within the steeper fill portion of the slope and involved a 70 to 80 foot wide section of embankment. The head scarp is near the outside road shoulder. The toe of the slip-out is located mid-slope (about 25 feet vertical distance) below the road. The

failure has not encroached into the travel-way, although we observed longitudinal pavement cracks in the outboard (southbound) lane that extend to near the centerline of the road. We did not observe slumping on the slope below the slipout and the creek does not appear to affect the slipout at this location.

4 GEOLOGIC SETTING

4.1 Regional Geology and Faulting

The project is located near the eastern margin of the Coast Ranges geomorphic province of California. This geomorphic province is characterized by a series of discontinuous northwest-trending mountain ranges extending from the Klamath Mountains on the north coast of California to the Transverse Ranges to the south. The Coast Ranges are composed of thick Mesozoic and Cenozoic sedimentary strata that have a complex structure due to intense folding and faulting.

No "active" faults (defined as showing evidence of surface displacement within the past 11,000 years) are shown in the immediate site vicinity, and the site is not within an Alquist-Priolo "Earthquake Fault Zone" for fault-rupture hazard. Published geologic mapping[1] shows several northwest-trending pre-Quaternary faults (>1.6 million years) located within about 100 to 3,000 feet west and southwest of the site. The nearest active fault is the Green Valley Fault, located about six miles south of the site.

4.2 Local Geology

The referenced geologic mapping shows the site to be underlain by rock identified as the Great Valley Sequence (Early Cretaceous and Late Jurassic) that is mainly composed of sandstone, shale and mudstone with lesser conglomerate. In the vicinity of the site, the mapping shows rock bedding striking northwest and dipping about 22° northeast. The trace of a northwest trending pre-Quaternary fault is shown about 100 feet west of the site, aligned roughly parallel to Steele Canyon Road. We show site geology on Figure 2.

We observed fractured claystone/mudstone, consistent with the published mapping, in the cut-slope above the road. The exposed rock is very intensely weathered/fractured, and is subject to sloughing along the face of the cut-slope. At this location, we measured rock bedding striking north and dipping about 60° east (into the slope). The cut-slope, not involved with the subject slip-out, appears to have performed well at this location.

The site is not mapped within a large-scale landslide feature. We did not observe evidence of regional landsliding, springs or surface seeps in the immediate vicinity of the site during our field investigation in May 2006.

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[1] Delattre, Marc P., et al, "Geologic Map of the Capell Valley 7.5' Quadrangle Napa County, California: A Digital Database" Version 1.0, 2006 (http://www.consrv.ca.gov/CGS/rghm/rgm/northern_region_quads.htm).

5 SUBSURFACE EXPLORATION

To characterize the subsurface conditions and obtain samples for laboratory testing, BCI retained Taber Consultants to drill two exploratory borings on May 28, 2007. The borings were advanced to a maximum depth of about 40 feet (lowest elev. 69) with solid-stem auger drilling methods using a truck-mounted CME 45 drill.

We supplemented the test borings with a seismic refraction line located along the outside shoulder of the roadway. We used a Seistronix RS-100 RadioSeis Wireless Seismic System with a 24 bit high resolution, single channel refraction seismograph. The energy source was by means of hand-actuated sledge hammer blows.

The drillers obtained soil and rock samples from the borings using 2.0-inch OD (1.4-inch ID) "Standard Penetration" and 3.0-inch OD (2.4-inch ID) "split-spoon" samplers equipped with brass liners. The samplers were driven into the ground with the force of a 140 pound hammer falling 30 inches using an automatic drop-hammer.

BCI's engineer, Kristy Chapman, logged soils and rock sampled from the borings consistent with the Unified Soil Classification System (USCS). BCI retained and sealed drive samples recovered from the samplers in moisture-proof containers for laboratory testing and reference. We also made groundwater observations in the borings during and after drilling operations. At the completion of field work, the borings were backfilled with cement grout.

BCI referenced the boring locations to existing site features. Subsequent to our field exploration, the locations and elevations of the test borings were determined by a survey made by Winzler & Kelly. We include the Log of Test Borings (LOTB) drawing showing boring locations, elevations, and laboratory test results in the Appendix.

6 SUBSURFACE CONDITIONS

6.1 Borings

We encountered two units in the test borings, as summarized below.

Unit 1: This unit, overlain by a pavement section comprised of 0.6 feet of asphalt concrete over 1.5 feet of gravelly silty sand, consists of firm and stiff-very stiff clay with sand and gravel to a depth of 8.5 feet below road grade. We interpret the majority of this unit to be roadway and embankment fill. The lower portions of this unit may represent colluvium or residual soil.

Unit 2: This unit underlies Unit 1 and is comprised of hard, moderately to well cemented clay. We encountered this unit to a depth of about 40 feet in each boring. We interpret these materials as intensely to moderately weathered, very intensely fractured and locally sheared claystone (shale) associated with the Great Valley Sequence.

Refer to the LOTB drawing in the Appendix for more specific soil/rock descriptions and sampling methods.

6.2 Seismic Refraction Line

The seismic refraction line shows primary wave (compression wave) velocities ranging from 1,100 feet per second (fps) to 7,000 fps. We interpret seismic velocities of 1,100 fps to represent in-place fill or residual soil, and the underlying velocity of 7,000 fps as weathered claystone. Based on the profile, we interpret the depth to rock to be about 11 feet below ground surface along the outside shoulder of the road.

We show the location and time-distance graph of the seismic refraction profile on the LOTB drawing.

6.3 Ground Water

We measured ground water at a depth of 27 feet in Boring 1. We did not observe free groundwater in Boring 2. We expect that groundwater is present during the wet season, likely "perched" over the underlying rock, and that Unit 1 fill soils are seasonally saturated. Ground water within the rock is likely discontinuous, controlled highly fractured zones and discontinuities. We consider the build-up of seepage pressures along the soil-rock interface as likely during the winter months or extended periods of rainfall.

7 LABORATORY TESTS

We performed the following laboratory tests on soil/rock samples obtained from the test borings:

- Moisture Content-Dry Density (ASTM D2937 & D2216)
- Unconfined Compressive Strength (ASTM 2166)
- Grain Size Analysis (ASTM D2487)
- Atterberg Limits (ASTM D4318)
- pH/Minimum Resistivity (CTM 643)
- Chloride (CTM 422)
- Sulfate (CTM 417)

Unit 1 embankment fill soils have dry densities ranging from 90 to 113 pcf, at moisture contents of 12 to 17%. Grain Size and Atterberg Limits results on a sample from Boring 2 shows 15% passing the No. 200 sieve, with a Liquid Limit of 35 and Plasticity Index of 16 (classed as SC per Unified Soil Classification). Results of an unconfined compressive strength test on one sample indicate 0.6 tsf. Unit 1 soils are at least moderately expansive based on Plasticity Index test results

Soil corrosion test results on a sample from Unit 1 indicate a soil pH of 6.5, minimum resistivity of 2,710 ohm-cm, chloride content of 9.9 parts per million (ppm), and a sulfate content of 0.9 ppm.

Unit 2 weathered and fractured claystone has dry densities ranging from 82 to 119 pcf, at moisture contents of 7 to 28%. Grain Size and Atterberg Limits results on a sample from Boring 2 show 29% passing the No. 200 sieve, with a Liquid Limit of 42 and a Plasticity Index of 21

(classed as SC per Unified Soil Classification). Results of an unconfined compressive strength test on one sample indicate 1.2 tsf. We interpret the relatively low unconfined compressive strength to reflect the fractured nature of the material and sample disturbance. Unit 2 claystone is at least moderately expansive based on Plasticity Index test results

Soil corrosion test results on a sample from Unit 2 indicate a soil pH of 5.8, minimum resistivity of 2,630 ohm-cm, chloride content of 13.6 parts per million (ppm), and a sulfate content of 7.8 ppm.

BCI performed laboratory tests in substantial conformance with the designated test procedures. The grain size analysis test results for claystone reflect the cemented nature of the materials tested (i.e., cemented fragments did not completely break-down). The LOTB drawing shows dry density, moisture content, and unconfined compressive strength test results. We attach other laboratory test results in the Appendix.

8 CONCLUSIONS

Based on the foregoing, we conclude that the cause of the slipout is likely due to a combination of seasonal storm water saturation of roadway fill overlying rock with associated build-up of seepage pressures along the soil-rock interface. Without remedial work, we expect that the slope will continue to move, extend headward, and eventually involve the roadway during future wet seasons.

The Unit 1 fill materials are directly involved with the slipout event. The Unit 2 rock materials appear "in-place" and not materially involved with the slope failure. Based on our boring data, the underlying rock is stable, but is intensely weathered, fractured and locally sheared (typical of rock weakened by fracturing, shearing and crushing along/near faults).

We consider the most straightforward approach for repair at this location to be a pile-bulkhead wall. This type of repair has the advantages of obtaining deep support within Unit 2 rock, provides a measure of internal structural flexibility with relative independence from subsequent down-slope or adjacent movement, and minimizes slope excavation and site/traffic disturbance during construction.

Below, we provide recommendations keyed to a 90-foot-long pile-bulkhead wall positioned approximately 5 feet from the existing outside edge of pavement. Figure 3 shows a plan view of the wall line and a schematic cross-section of the pile-bulkhead wall.

A drained, reconstructed embankment section could be considered at this location. However, site geometry suggests that the failed slope is marginal to "catch" a typical 2H:1V finished embankment slope. This approach would require excavation 35 to 40 feet below the roadway to engage "intact" Unit 2 claystone and involve much of the road prism to construct. The toe of the reconstructed embankment would "catch" mid-slope and be susceptible to damage should future movement occur either down-slope or adjacent to this site.

Road stabilization using a "flexible" gravity-wall system (e.g., "Hilfiker" or equivalent) could also be considered at this site, but would need to be at least 16 feet high to suitably engage

"intact" rock, involve significant excavation into the road prism and have limited flexibility under conditions of future downslope movement. Another disadvantage of this option is that on-site soils are moderately expansive and not suitable for placement between reinforcing elements, requiring materials to be replaced with a low-expansive, granular import.

Rigid wall systems (e.g., reinforced concrete cantilever or concrete crib walls) have limited tolerance to slope movement and/or adjacent events and we do not recommend their use at this site. We also do not recommend road retreat (into the cut-slope) at this site due to the steep slopes, weak rock materials and adverse effects on road alignment.

9 RECOMMENDATIONS

9.1 Pile-Bulkhead Wall

For vertical pile elements, we recommend:

- minimum 24-inch diameter cast-in-drilled-hole (CIDH) piles with H-pile cores for increased bending strength,
- piles spaced at 8 to 10 foot centers, and
- CIDH piles penetrate at least 20 feet into Unit 2 rock.

For design, pile "fixity" can be taken at 5 feet below the rock line. For a wall located 5 feet from the existing outside edge of pavement, we anticipate the rock line will vary along the wall from about elev. 105 on the south to elev. 97 on the north. To meet the above pile penetration criteria, specify CIDH pile tip elevations to at least elev. 85 at the south end, sloping to elev. 77 at the north end.

Actual pile tip elevations will depend on final wall location and realized loading per pile. Final pile penetration should be based on field-review by personnel from this office during construction.

Install timber or concrete lagging between H-pile flanges to retain backfill materials. To engage the rock, extend the lagging at least 13 feet below the top of wall. The lagging at each end of the wall can be adjusted upward to transition into the adjacent slope.

Unit 1 fill soils are suitable for re-use as wall backfill. Place and compact Unit 1 fill soils to 90% (per CTM 216) relative compaction (to 95% within 30-inches of finished road grade).

For Unit 1 soils re-used as backfill, use an active equivalent fluid pressure of 45 pcf to calculate lateral forces on the wall. For surcharge loads, apply an additional uniform lateral load behind the wall equivalent to 0.3-times the surcharge pressure.

Behind the wall, construct a 24-inch thick section of Permeable Material wrapped in filter fabric or install a pre-fabricated drain panel. Extend the drain element from the base of the lagging to within 18-inches of finished surface. Provide a perforated pipe and gravity outlet at the low point of the wall.

Alternatively, Permeable Materials or clean drain rock enclosed in permeable geotextile fabric can also be used for all backfill behind the wall. Under these conditions, calculate the lateral force on the wall based on an active equivalent fluid pressure of 36 pcf.

If needed for lateral restraint, use tie-backs connected to 24-inch (minimum) diameter CIDH pile-anchors installed along the inside of the roadway. We recommend that CIDH pile-anchors penetrate at least 15 feet into Unit 2 claystone. An allowable passive pressure of 350 pcf equivalent fluid pressure (based on $\phi = 38°$ and unit weight of 126 pcf) is available within Unit 2 rock at this level.

Control and direct surface runoff away from the slipout area (e.g., by means of an asphalt-concrete dike along the outer edge of road). Initial wall yield may open surface cracks in back of the wall; these should be carefully sealed. Regular maintenance should be provided along the inner-roadway ditch to keep this area clear of debris from cut-slope sloughing.

Re-grade the slope in front of the wall to drain and prevent surface water from ponding. The finished slope should be vegetated to minimize erosion potential.

10 CONSTRUCTION CONSIDERATIONS

10.1 Excavation and Shoring

We expect that excavation of soils and rock can be achieved using heavy-duty foundation drilling equipment. We also expect a 1:1 (or flatter) construction backslope will be appropriately stable during dry-season construction (June through October), although may require supplemental treatment (e.g., benching or subdrainage) to control local sloughing and seepage.

The contractor is responsible for design and construction of all excavations and shoring in accordance with CalOSHA requirements.

10.2 De-watering

We expect ground water within the rock unit to be discontinuous and of limited quantity, likely transmitted as seepage through rock discontinuities (e.g., fractures, bedding planes, etc.). In general, we do not expect ground water inflow to be substantial during dry-season construction (June through October) and typically controllable by pumping.

10.3 Over-optimum Soil Moisture

Excessively over-optimum (wet) soil conditions can make proper compaction difficult or impossible. Wet soil is commonly encountered during the winter and spring months, or in excavations where ground water is encountered.

In general, wet soil can be mitigated by:

- Discing to aerate the soil during prolonged periods of dry weather
- Overexcavation and replacement with drier material
- Stabilization using aggregate and stabilization fabric or grid

If wet, unstable soil is encountered, BCI can observe the conditions and provide more specific mitigation recommendations.

11 RISK MANAGEMENT

Our experience and that of our profession clearly indicates that the risks of costly design, construction, and maintenance problems can be significantly lowered by retaining the geotechnical engineer of record to provide additional services during design and construction.

For this project, BCI should be retained to:

- Review and provide comments on the civil plans, foundation plans, and specifications prior to construction.
- Monitor construction to check and document our report assumptions. BCI should, at a minimum, review CIDH pile excavations, construction backslope, and drainage requirements.
- Update this report if design changes occur, one year or more lapses between this report and construction, and/or site conditions have changed.

If we are not retained to perform the above applicable services, we are not responsible for any other party's interpretation of our report, and subsequent addendums, letters, and discussions.

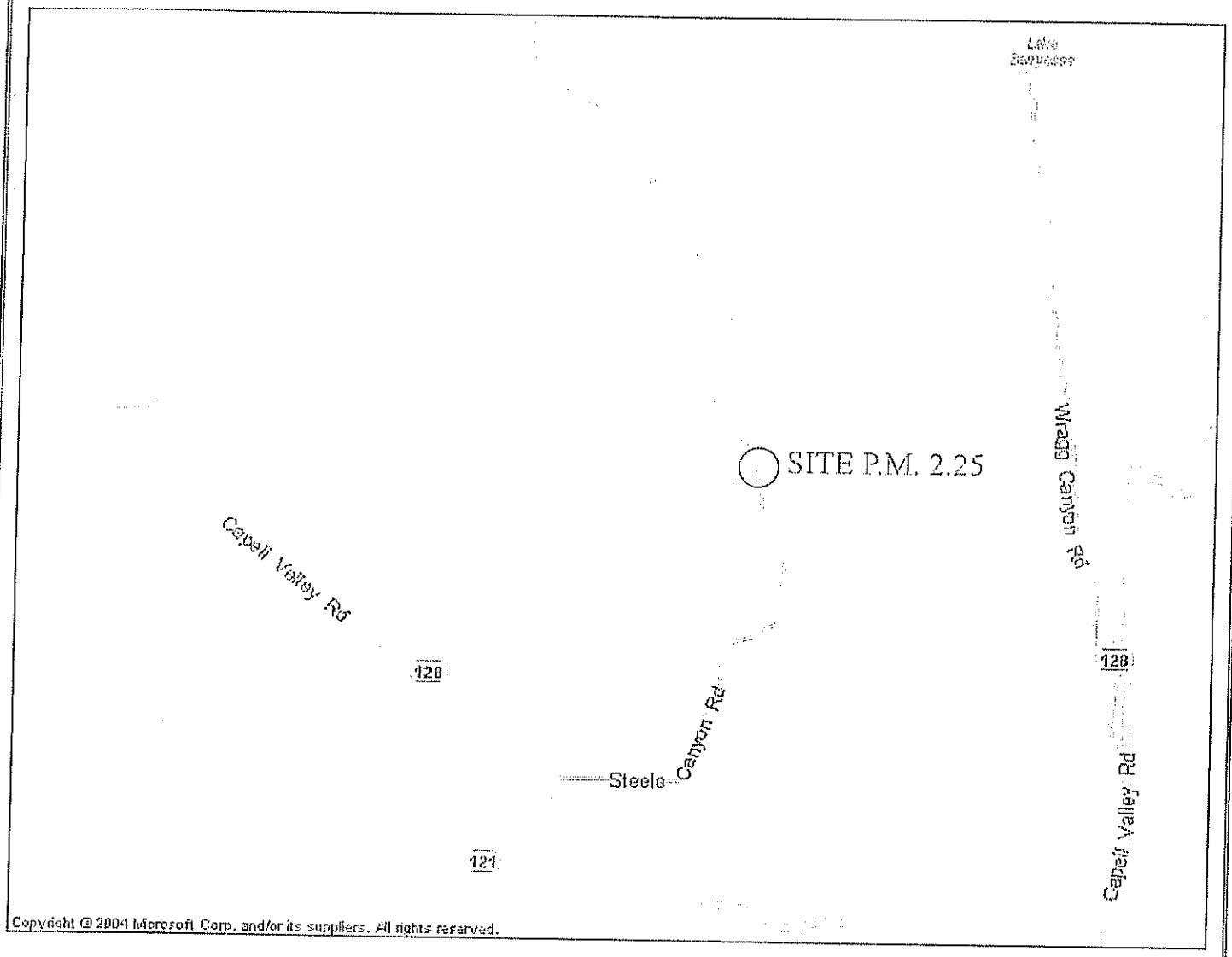
12 LIMITATIONS

BCI performed services in accordance with generally accepted geotechnical engineering principles and practices currently used in this area. We do not warranty our services.

BCI based this report on the current site conditions. We assume the soil and groundwater conditions observed in our borings are representative of the subsurface conditions on the site. Actual conditions between borings could be different.

Modern design and construction are complex, with many regulatory sources/restrictions, involved parties, construction alternatives, etc. It is common to experience changes and delays. The owner should set aside a reasonable contingency fund based on complexities and cost estimates to cover changes and delays.

The Log of Test Borings is presented in the Appendix. The lines designating the interface between soil/rock types are approximate. The transition between soil types may be abrupt or gradual. Our recommendations are based on the final logs, which represent our interpretation of the field logs and general knowledge of the site and geologic conditions.



N

Not to Scale

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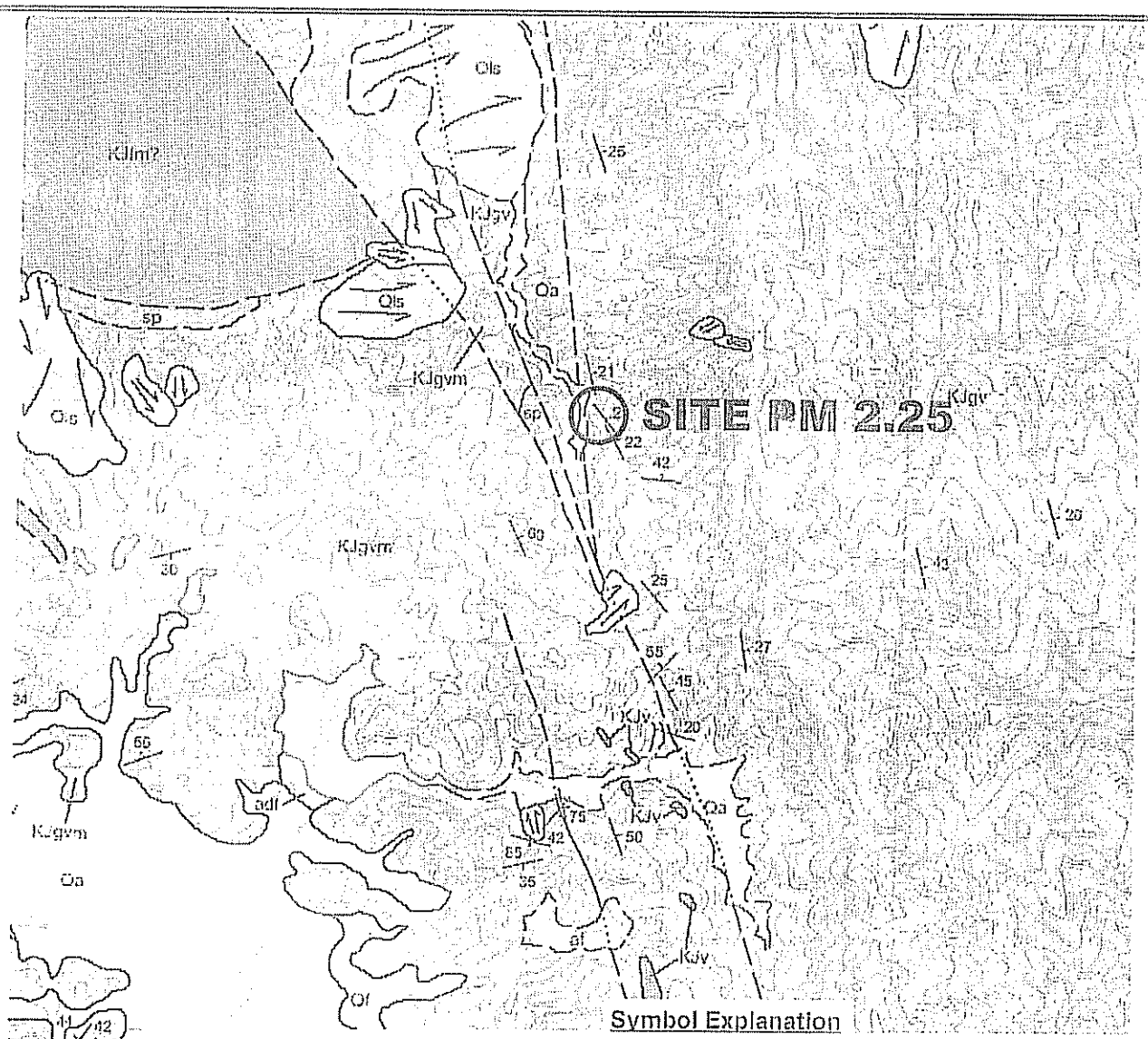
VICINITY MAP

Steele Canyon Road at P.M. 2.25
Napa County, California

File No. 1216.1

July 2007

Figure 1



Symbol Explanation

Contact between map units—Solid where accurately located, dashed where approximately located, dotted where concealed.

Fault—Solid where accurately located, dashed where approximately located, short dash where inferred, dotted where concealed, queried where uncertain.

Strike and dip of inclined bedding.

Landslide deposits-Arrows indicate general direction of movement.

N Not to Scale

Qa	Alluvium, undivided (latest Pleistocene to Holocene)-Flat, relatively undissected fan, terrace, and basin deposits.
Qls	Landslide deposits (Holocene and Pleistocene)-Includes debris flows and block slides.
KJgv / KJss	Great Valley Sequence (Early Cretaceous and Late Jurassic)-Interbedded sandstone, shale, mudstone, and occasional conglomerate. KJss-Resistant sandstone members that form prominent strike ridges.

Reference: Geologic Map of the Capell Valley 7.5' Quadrangle, Napa County, California, 1:24,000: A Digital Database, Version 1.0 by Marc P. Delattre and Janet M. Sowers, 2006.

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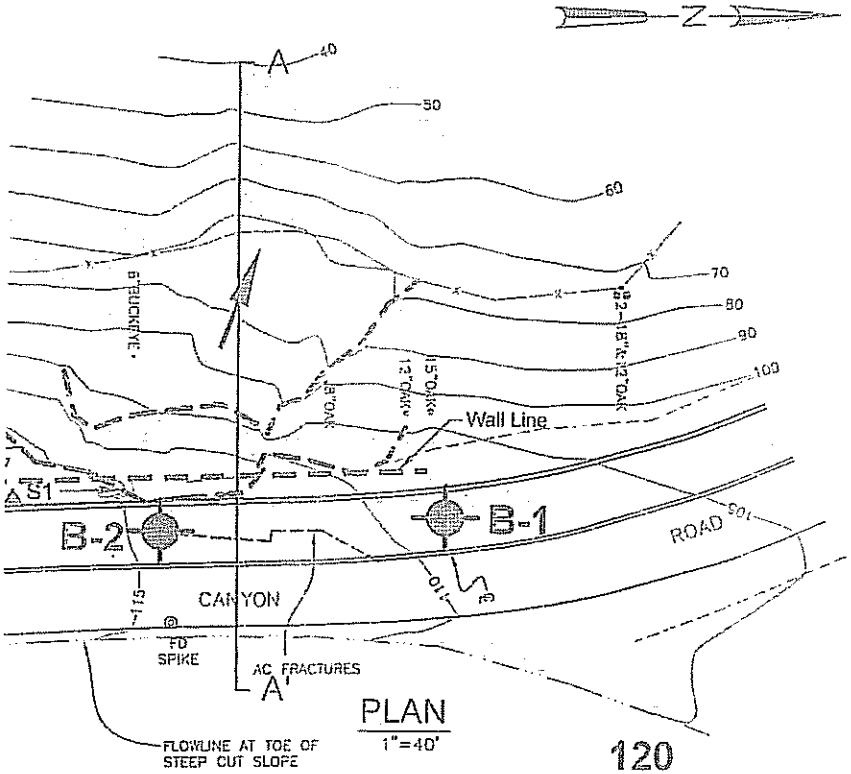
GEOLOGIC MAP

Steele Canyon Road Slide PM 2.25
Napa County, California

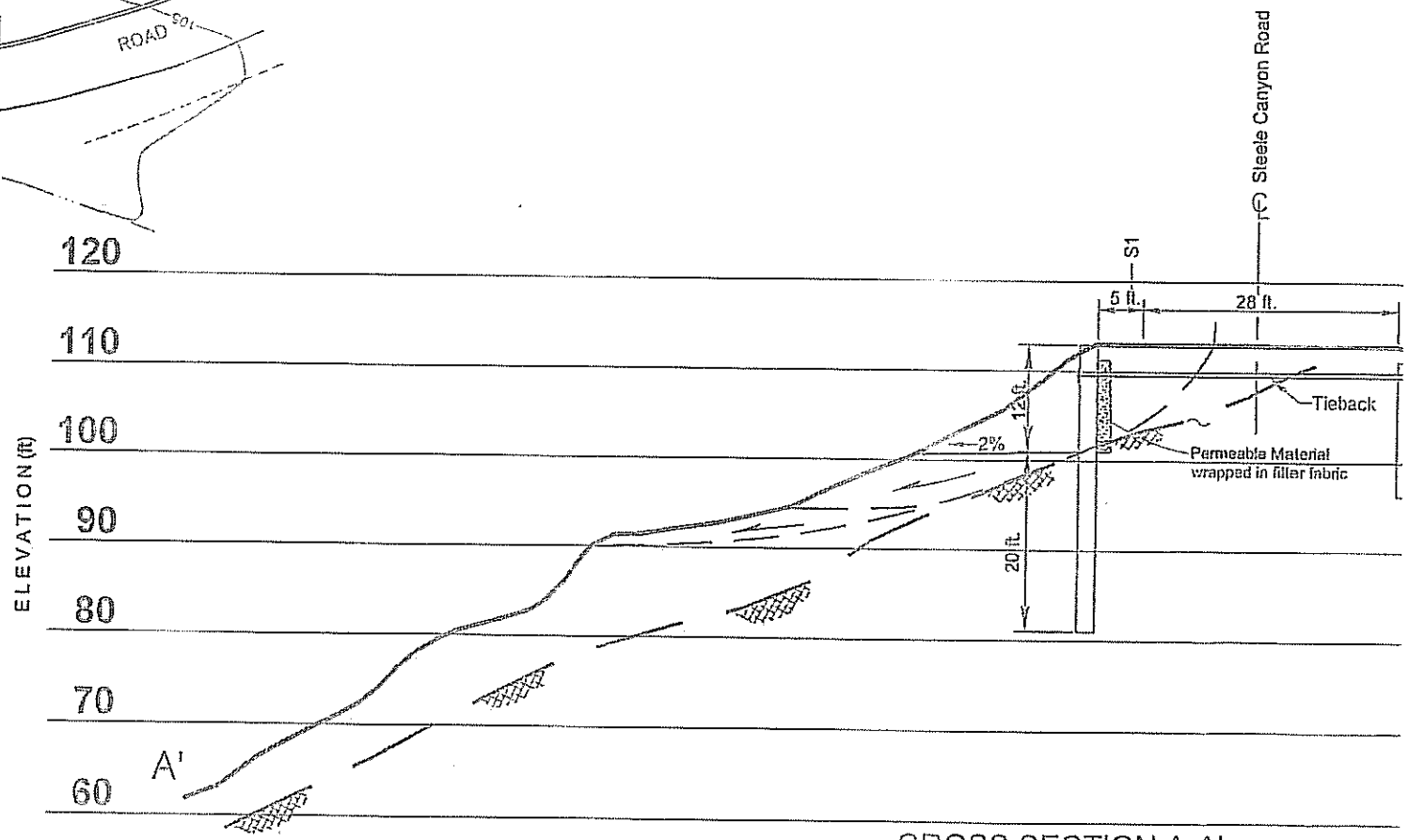
File No.1223.1

July 2007

Figure 2



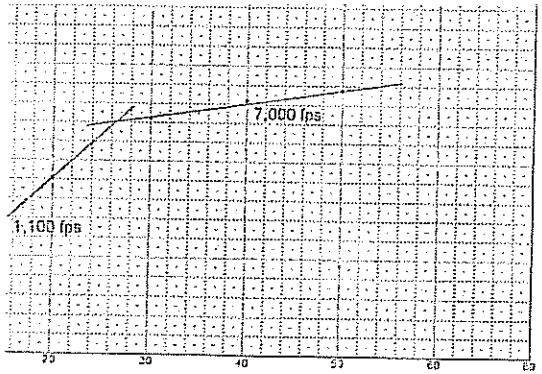
PLAN
1"=40'



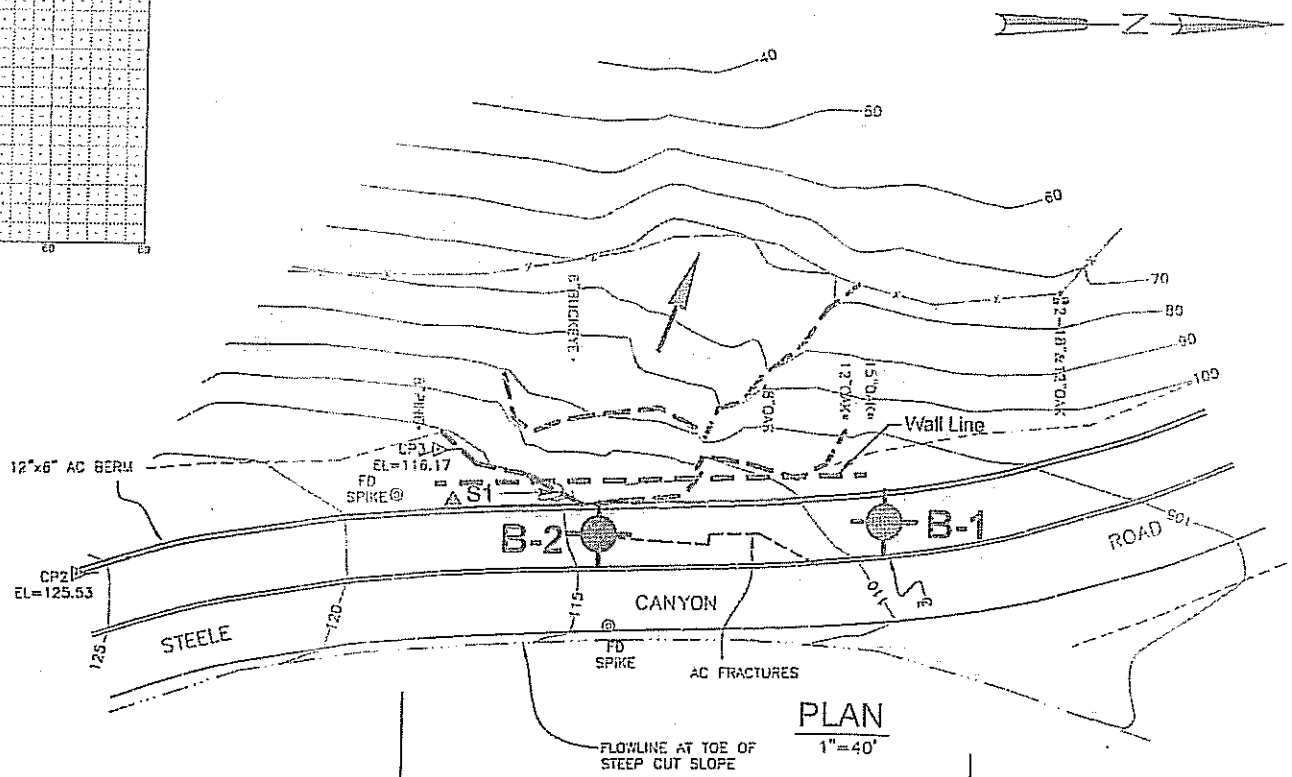
CROSS SECTION A-A'
HOR./VERT. 1"=20'

APPENDIX

- Log of Test Borings
- Laboratory Test Results

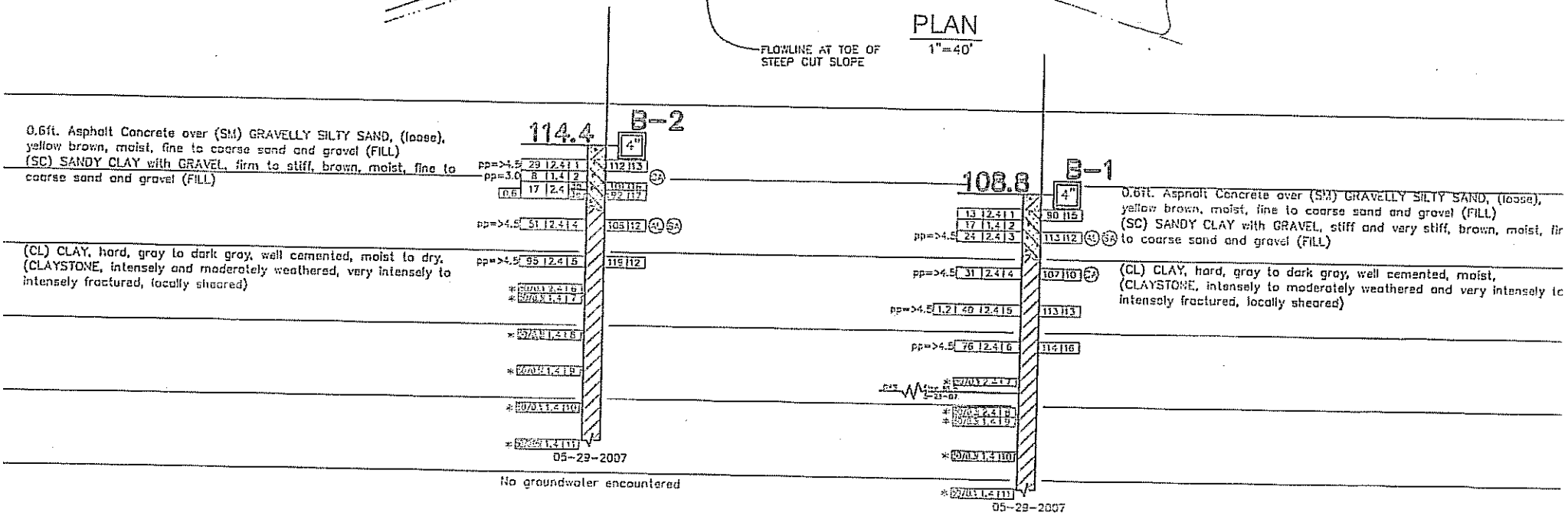


REFRACTION SEISMIC RECORD



PLAN
1"=40'

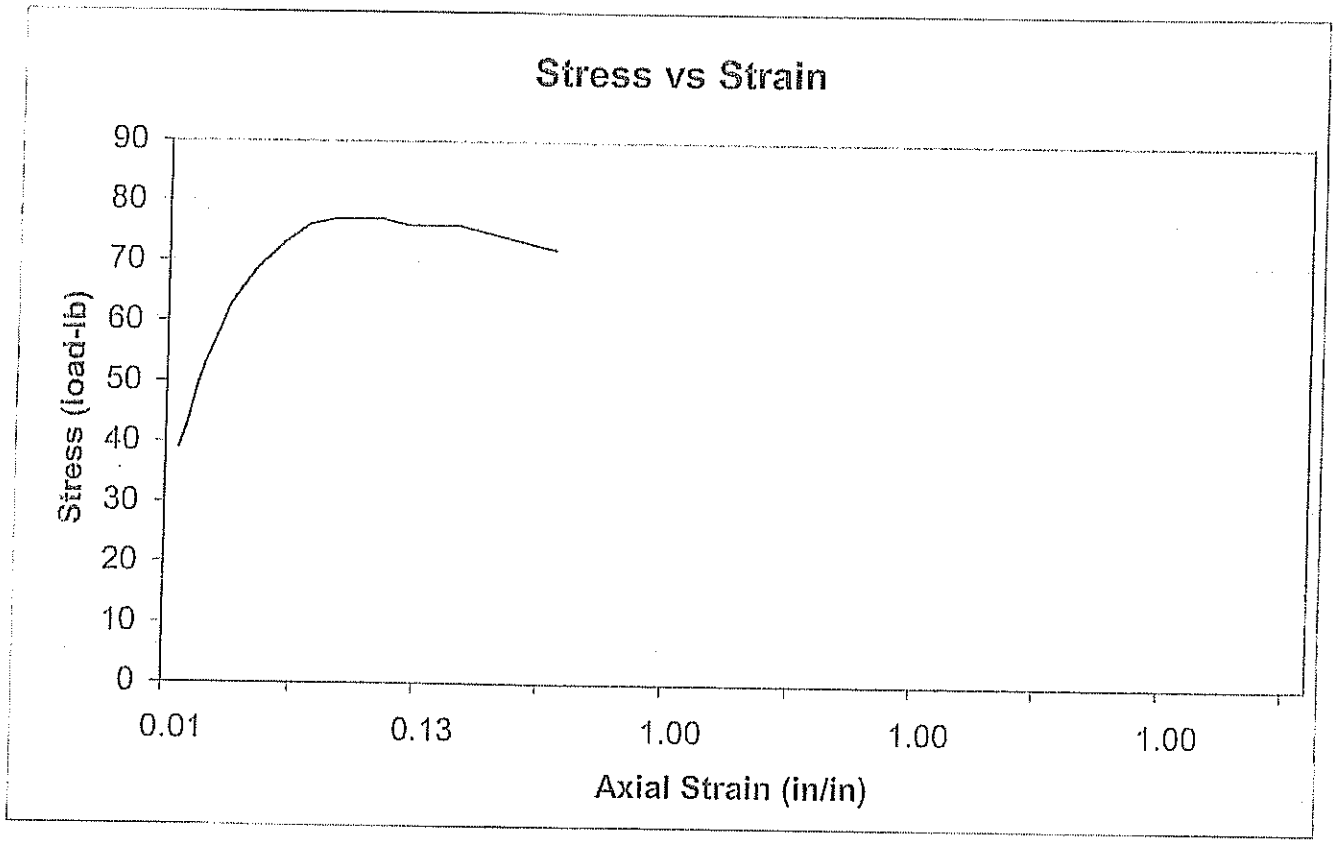
NOTES:
1. Field clo
"Descriptio
2. Standar
D 1586-99
Drill rods v
brass liner
3. 2.4 inc
manner as
4. The leng
boring log.
penetration
Where less
shown is f
interval act
5. Where in
only that f
penetrated.
6. Consiste
7. Groundw
Log of Test
specified d
8. Groundw
and may o
conditions
9. Refractio
actuated, s
10. Boring
12. Electron



PROFILE

Project
Steele Canyon Rd. P.M. 2.25
Project Number
1223.1
Sample Number
B1-5c
Material Description
Dark olive gray weathered claystone with clay
Tested By
KAC

ASTM D 2166-00

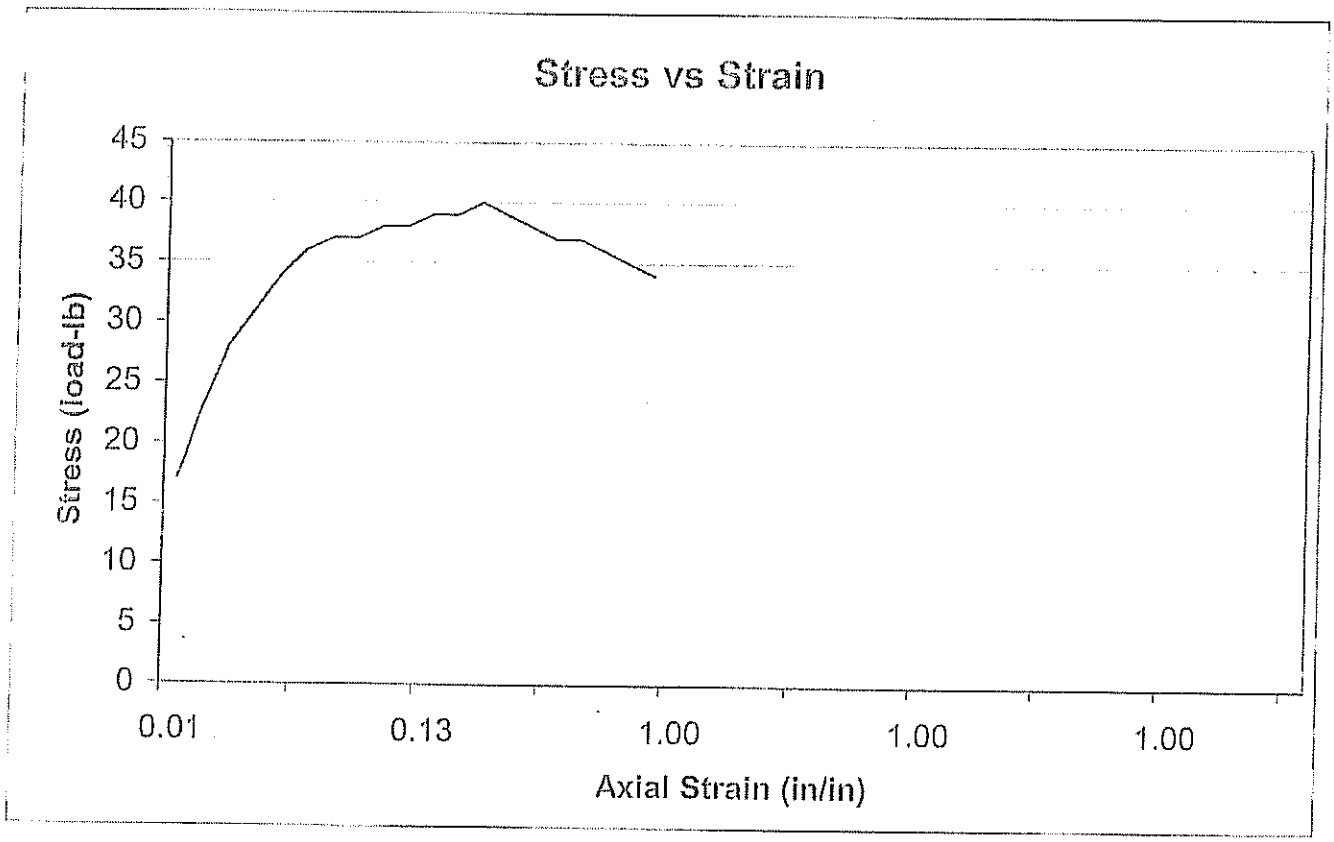


Total Density (pcf)	127.8
Dry Density (pcf)	113.2
% Moisture	12.9

Unconfined Compressive Strength (tsf) 1.18

Project
Steele Canyon Rd. P.M. 2.25
Project Number
1223.1
Sample Number
B2-3c
Material Description
Dark olive gray weathered clayey silt
Tested By
KAC

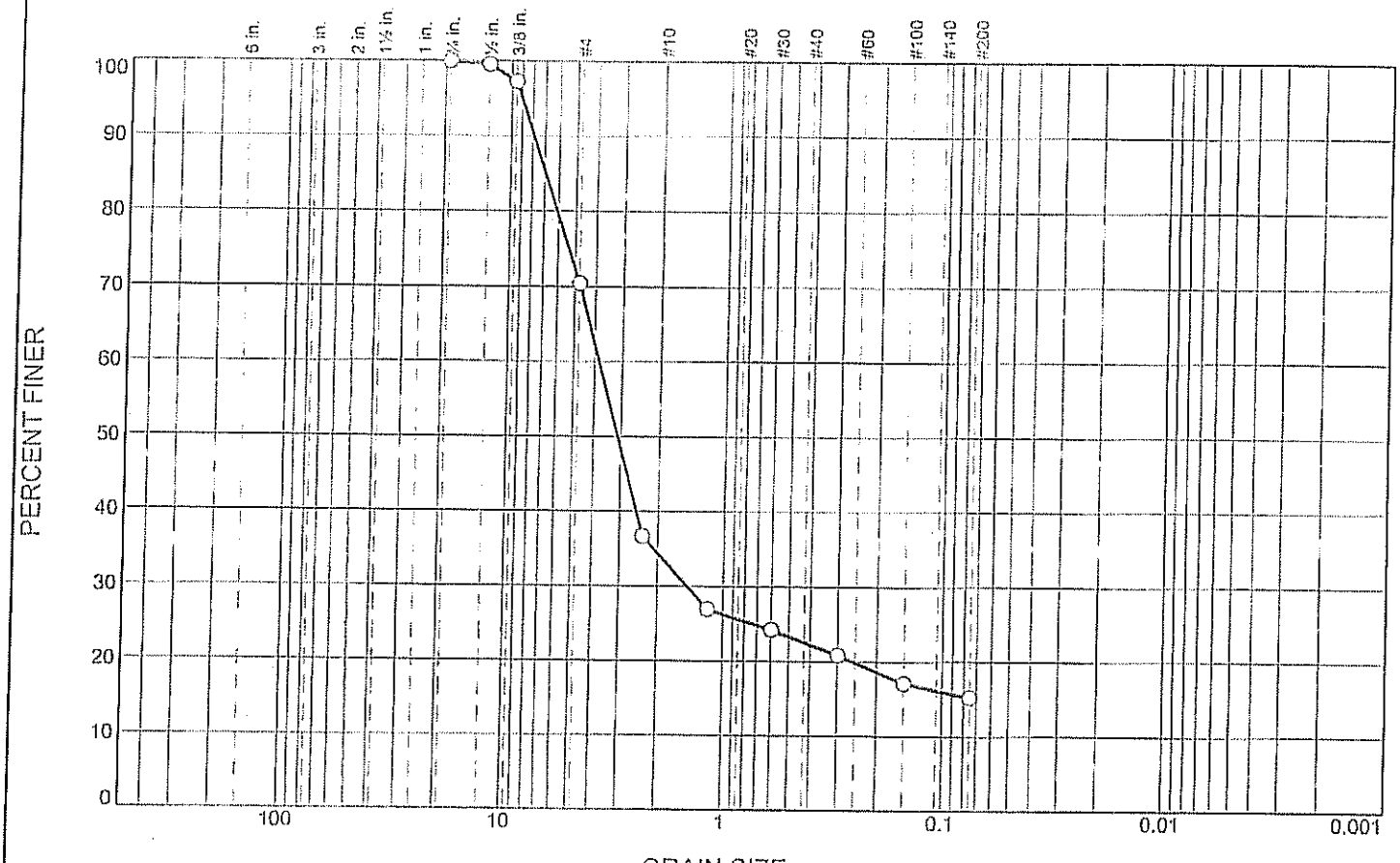
ASTM D 2166-00



Total Density (pcf)	107.0
Dry Density (pcf)	91.7
% Moisture	16.7

Unconfined Compressive Strength (tsf) 0.61

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	29.6	36.1	11.8	7.3	15.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	99.5		
3/8"	97.2		
#4	70.4		
#8	36.7		
#16	26.9		
#30	24.2		
#50	20.9		
#100	17.1		
#200	15.2		

* (no specification provided)

Material Description

Olive brown clayey sand with gravel (weathered/fractured claystone)

Atterberg Limits

PL= 19 LL= 35 PI= 16

Coefficients

D_{85}= 6.9312 D_{60}= 3.8263 D_{50}= 3.1104
D_{30}= 1.4669 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO= A-2-6(0)

Remarks

Sample Number: B1-3c Depth: 6.0-6.5 ft Date: 6-18-07

Blackburn Consulting

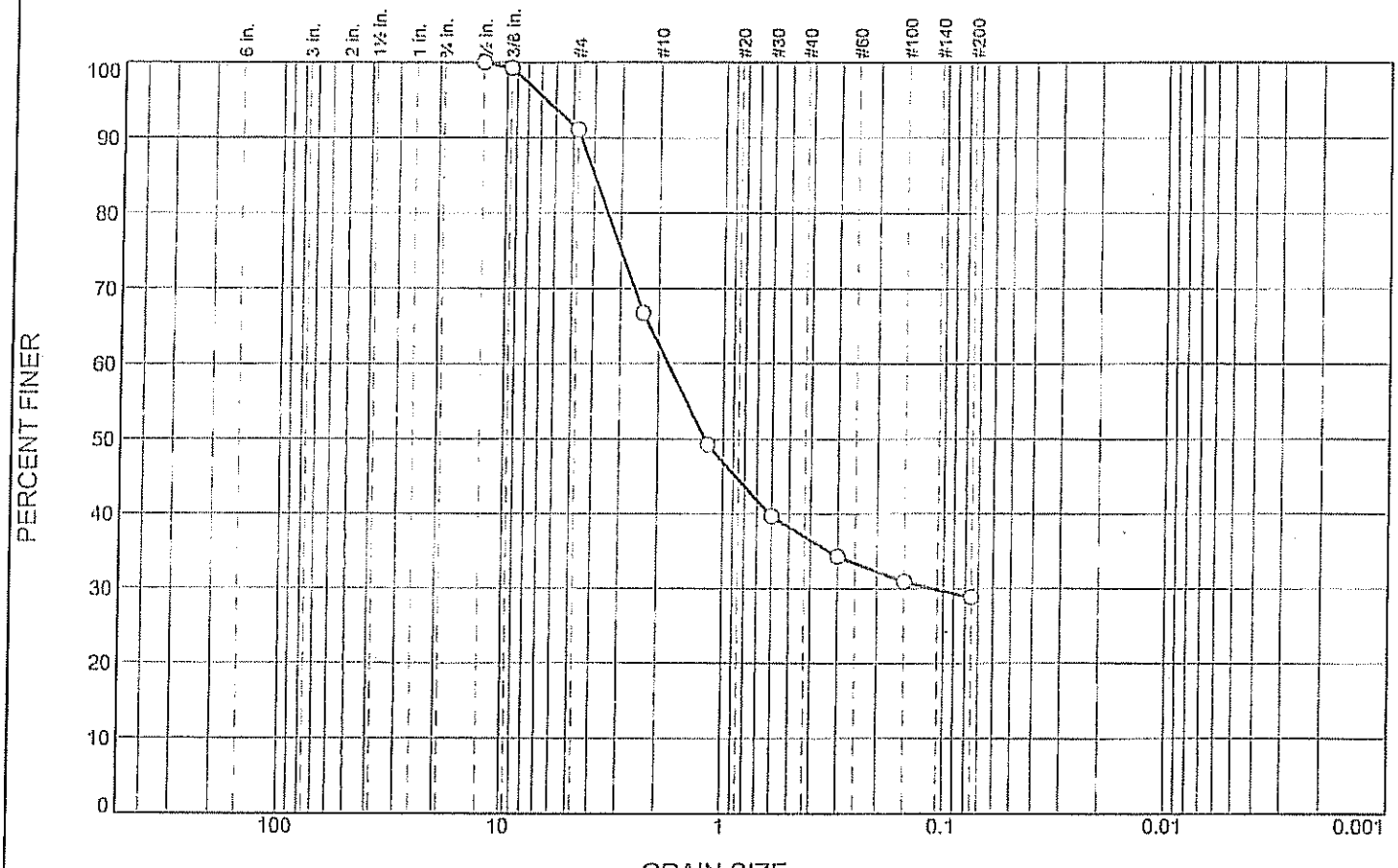
W. Sacramento, CA

Client:

Project: Steele Canyon Road at PM 2.25

Project No: 1223.1 Figure

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	8.9	28.5	25.6	8.1	28.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100.0		
3/8"	99.3		
#4	91.1		
#8	66.8		
#16	49.2		
#30	39.7		
#50	34.3		
#100	30.9		
#200	28.9		

* (no specification provided)

Material Description

Olive brown clayey sand (weathered/fractured claystone)

Atterberg Limits

PL= 21 LL= 42 PI= 21

Coefficients

D_{85}= 3.9875 D_{60}= 1.8043 D_{50}= 1.2159
D_{30}= 0.1100 D_{15}= D_{10}=
C_u= C_c=

Classification

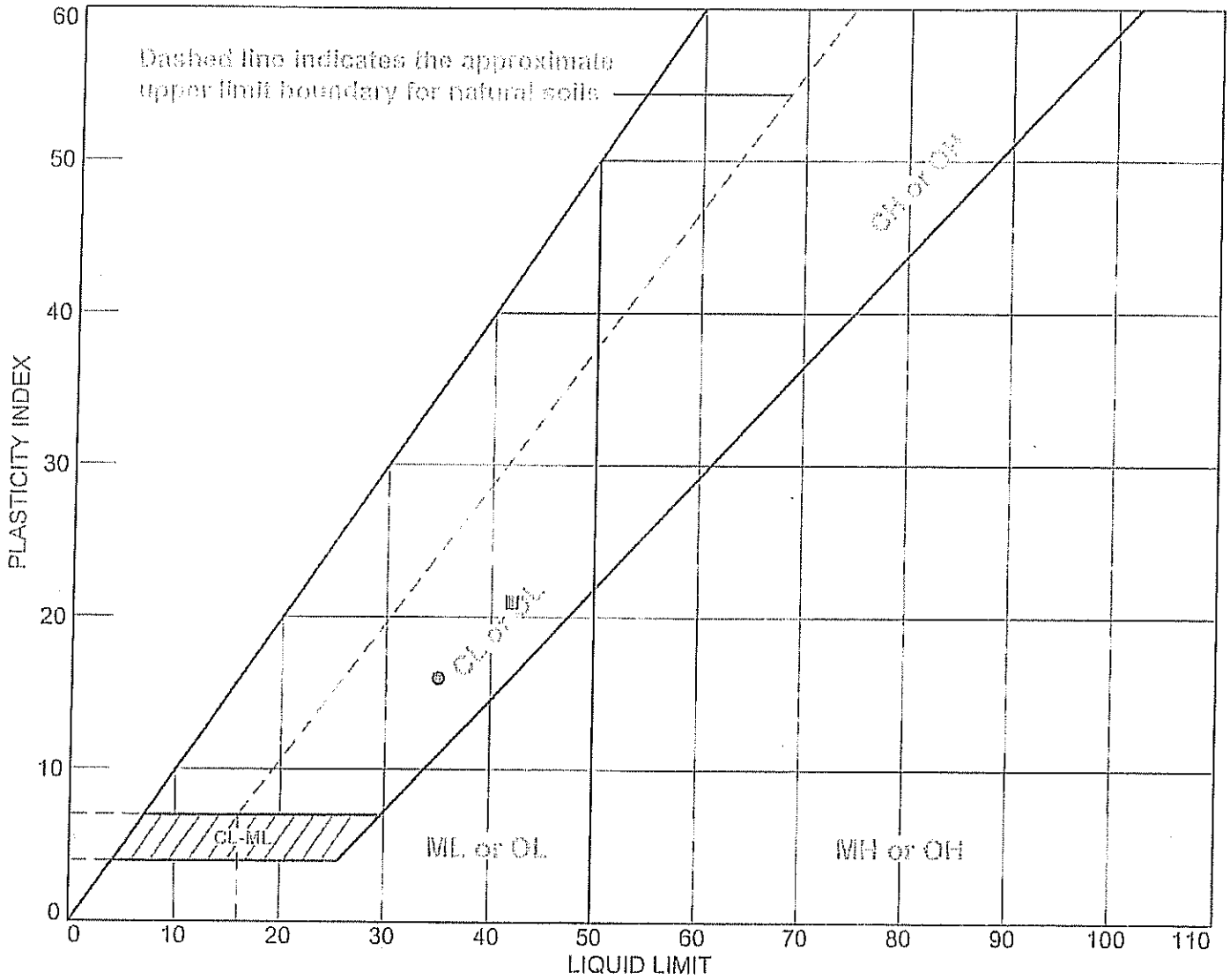
USCS= SC AASHTO= A-2-7(2)

Remarks

Sample Number: B2-4b Depth: 10.5-11.0 ft Date: 6-18-07

Blackburn Consulting W. Sacramento, CA	Client: Project: Steele Canyon Road at PM 2.25 Project No: 1223.1 Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		B1-3c	6.0-6.5 ft		19	35	16	SC
■		B2-4b	10.5-11.0 ft		21	42	21	

Blackburn Consulting W. Sacramento, CA	Client: Project: Steele Canyon Road at PM 2.25 Project No.: 1223.1 Figure

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 06/13/2007
Date Submitted 06/08/2007

To: Eric Nichols
Blackburn Consulting
2437 Front Street
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : STEELE CYN.RD.PM2.25 Site ID : B1-4B.
Thank you for your business.

* For future reference to this analysis please use SUN # 50744-101312.

EVALUATION FOR SOIL CORROSION

Soil pH	6.54	
Minimum Resistivity	2.71 ohm-cm (x1000)	
Chloride	9.9 ppm	00.00099 %
Sulfate	0.9 ppm	00.00009 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 06/13/2007
Date Submitted 06/08/2007

To: Eric Nichols
Blackburn Consulting
2437 Front Street
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : STEELE CYN.RD.PM2.25 Site ID : B2-2.
Thank you for your business.

* For future reference to this analysis please use SUN # 50744-101313.

EVALUATION FOR SOIL CORROSION

Soil pH	5.81	
Minimum Resistivity	2.63 ohm-cm (x1000)	
Chloride	13.6 ppm	00.00136 %
Sulfate	7.8 ppm	00.00078 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422