# GEOTECHNICAL INVESTIGATION

STEELE CANYON ROAD AT P.M. 2.00
NAPA COUNTY, CALIFORNIA

Prepared for:

Napa County
Owner

and

Winzler & Kelly
Consulting Engineers
Design Engineer

Prepared by:
Blackburn Consulting

July 27, 2007

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Geotechnical • Construction Services • Forensics

BCI File No. 1216.1
July 27, 2007

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

Subject: **Geotechnical Investigation**
*Steele Canyon Road at P.M. 2.00*
*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. 2.00, in accordance with our agreement dated April 25 2007.

Please call if you have questions regarding 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 1-31-10 STATE OF CALIFORNIA

Reviewed by:

Bob Lokteff, C.E., G.E.
Principal

REGISTERED PROFESSIONAL ENGINEER ROBERT C. LOKTEFF No. 2589 Exp. 12/31/09 GEOTECHNICAL STATE OF CALIFORNIA

Distribution: Client (4)

**GEOTECHNICAL INVESTIGATION**

**STEELE CANYON ROAD SLIDE AT P.M. 2.00**
**Napa County, California**

# TABLE OF CONTENTS

# 1 PURPOSE

Blackburn Consulting (BCI) prepared this Geotechnical Investigation for the storm damage at Steele Canyon Road, P.M. 2.00, in accordance with our April 25, 2007 agreement. This report includes the results of our subsurface investigation, our geotechnical 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 of the area.
- Reviewed a "Landslide Site Assessment Report" dated March 9, 2006 and a "Project Worksheet" dated May 23, 2006, both prepared by the Federal Emergency Management Agency.
- Performed geologic reconnaissance of the site.
- Drilled, logged, and sampled three exploratory test borings from road level.
- Completed a double-ended seismic refraction line near the toe of the fill slope.
- 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 two miles north of its intersection with Highway 128 in Napa County, California. Site latitude is approximately N38.46532° and longitude is approximately W122.18097°. We show the project location on Figure 1.

## 3.2 Site Description

At P.M. 2.00, Steele Canyon Road traverses a west-facing slope on a slight concave-westerly curve. The road is a paved, two-lane section, about 24 to 28 feet wide. It is established in a 25 to 30 foot high cut section on the inboard (east) side, with 15 to 20 feet of fill on the outboard (west) side. Profile grade descends northerly on a steep (10%) grade.

The slipout occurred on the slope below the roadway and has affected about 140 feet of road length, undercutting portions of the paved shoulder and displacing a metal beam guardrail about 8 to 12 inches horizontally and vertically. The County has placed temporary fill onto the slipout area and placed a K-rail along the outside shoulder of the road. Incipient pavement cracks associated with the slipout event extend into the outboard (southbound) lane to near the centerline of the road.

About 80 feet south of the site, a 30-inch diameter pipe crosses under the road and carries surface water from a natural (ephemeral) drainage. The pipe connects to a concrete junction box located on-slope below the road, extends along the surface, and discharges near the toe of the slope just north of the slipout area. The existing slopes adjacent to the slipout are in-place at about 2:1 (horizontal:vertical).

The goal of repair at this location is to re-establish the integral ground and pre-slipout roadway width through the affected area.

# 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 200 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 and Faulting

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 22° northeast (into the slope at this location). The trace of a northwest trending pr-Quaternary fault is shown about 200 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, within inner road cuts in the immediate site vicinity. At this location, we did not observe any preferred fracture/bedding planes or rock structure. The exposed rock is decomposed and very intensely weathered/fractured (effectively soil-like), and is subject to sloughing along the face of the cut-slope. The cut-slope above the road is in-place at about 1:1 (or slightly steeper) and appears to have performed well.

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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).

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 during our field investigation in May 2006.

## 5 SUBSURFACE EXPLORATION

To characterize the subsurface conditions and obtain samples for laboratory testing, BCI retained Clear Heart Drilling, Inc. to drill three exploratory borings on May 22 and 23, 2007. The borings were advanced to a maximum depth of about 50 feet with hollow-stem auger drilling methods using a truck-mounted CME 750 drill.

We supplemented the test borings with a double-ended refraction seismic profile located on-slope below 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 soil samples recovered with the drive 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 and the seismic refraction line 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, laboratory test results, and the seismic refraction line in the Appendix.

## 6 SUBSURFACE CONDITIONS

### 6.1 Borings

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

Unit 1: This unit, underlain by about 0.15 to 1.0 feet of asphalt concrete, consists of roadway and embankment fill. It is comprised of loose clayey gravel and clayey sand in the upper 1 to 8.5 feet below road grade, then soft and firm (locally hard) silty clay and clay with gravel. We encountered this unit to a depth of about 14.5 feet in Boring 1, 17 feet in Boring 2, and 9 feet in Boring 3. Materials of this unit were directly involved with the slipout.

Unit 2: This unit underlies Unit 1 soils and is comprised of firm and stiff clay and gravelly clay, interpreted as slope colluvium and/or residual soil. We encountered this unit to a depth of about

19.5 feet in Boring 1 and 14.5 feet in Boring 3. In Boring 2, the lower portion described as Unit 1 may include colluvium and/or residual soil.

Unit 3: This unit underlies Unit 2 in Boring 1 and Boring 3, and Unit 1 in Boring 2. Materials of this unit are comprised of very stiff to hard, moderately to well cemented clay. We encountered this unit to a depth of about 50 feet in Boring 1 and 41 feet in Boring 2 and Boring 3. We interpret these materials as decomposed 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 descriptions and sampling methods.

### 6.2 Seismic Refraction Profile

The refraction seismic profile shows primary wave (compression wave) velocities ranging from 500 feet per second (fps) to as much as 7,500 fps. We interpret seismic velocities between 500 and 2,000 feet per second (fps) to represent unconsolidated slide debris/colluvium and residual soil, and the underlying velocities of 6,000 to 7,500 fps as weathered rock. Based on the profile, we interpret the depth to rock near the toe of the slipout to vary between 7 and 12 feet (measured normal to slope).

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

### 6.3 Ground Water

We measured ground water at a depth of 30 feet in Boring 1. We did not observe free groundwater in Boring 2 or Boring 3. 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 by 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 100 to 129 pcf, at moisture contents of 12 to 17%. Grain Size and Atterberg Limits results on samples from Boring 1 and Boring 2 show 14 to 49% passing the No. 200 sieve, with Liquid Limits of 36 and 37 and Plasticity Index of 16 and 18 (classed as SC and GC per Unified Soil Classification). Results of unconfined compressive strength tests on two samples show 0.12 tsf and 2.9 tsf. Unit 1 soils are at least moderately expansive based on Plasticity Index test results.

Soil corrosion test results on two samples from Unit 1 show a soil pH of 6.4 and 7.63, minimum resistivity of 1,260 and 1,530 ohm-cm, chloride content of 12.7 and 14.9 parts per million (ppm), and sulfate content of 21.6 and 25.9 ppm.

Unit 2 slope colluvium and residual soils have dry densities ranging from 69 to112 pcf, at moisture contents of 10 to 34%. An unconfined compressive strength test on one sample from Unit 2 shows 0.79 tsf.

Unit 3 weathered and fractured rock materials have 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 3 show 90% passing the No. 200 sieve, with a Liquid Limit of 50 and a Plasticity Index of 25 (classed as borderline CH per Unified Soil Classification). Results of unconfined compressive strength tests on two samples show 0.46 tsf and 0.71 tsf. We interpret the relatively low unconfined compressive strength to reflect the intensely fractured nature of the rock. Unit 3 rock materials are moderately to highly expansive based on Plasticity Index test results.

BCI performed laboratory tests in substantial conformance with the designated test procedures. 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 roadway distress and slipout is likely due to seasonal saturation of roadway fill overlying rock. Seasonal storm water infiltration and associated build-up of seepage pressures along the soil-rock interface may also have contributed to the slipout at this location.

The slip-plane extends to the rock line in the central portion of the slipout. The Unit 1 fill materials are directly involved with the slipout event and we identified a 5 to 6 foot thick zone of weak Unit 2 soil that remains in-place along the flanks of the slipout. The lowermost Unit 3 rock appears "in-place" and not materially involved with the slipout event. 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 show the estimated slip-plane on Figure 3.

The most straight-forward approach to re-establish the roadway at this location is a reconstructed, drained embankment section. The key elements of this work include toe support within Unit 3 rock; internal subdrainage with assured positive relief at the outlet; replacement and reconstruction of the embankment as compacted fill; and an interceptor underdrain along the inside road area to help control seasonally shallow groundwater infiltration/seepage. Below we provide recommendations for a reconstructed embankment section with a 2:1 (horizontal: vertical) finished slope established from the restored original hinge-line (see Figure 3).

A drained, reinforced embankment section with a steeper finished slope and toe support established at a higher level on the existing slope face could also be considered. However, due to their cohesive nature and moderate expansion potential, soils would need to be amended on-site or replaced with suitable import. This option would also be more susceptible to movement in the event of subsequent down-slope slide events depending on the geometry of the final finished slope configuration.

Structure support (i.e., crib wall, reinforced earth, cantilever wall, bulkhead wall, etc.) along the outer edge of original traveled-way could be considered, but is severely limited by the required height necessary to engage the "intact" rock materials. Wall options such as a gabion wall or reinforced embankment would also have limited flexibility under conditions of future movement (particularly down-slope), are sensitive to local defects in bearing, and involve relatively large space requirements to construct. Another disadvantage of a flexible wall system at this site is that the majority of 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.

We do not recommend road retreat (into the cut-slope) at this site due to the steep slopes, weak fill and rock materials and adverse effects on road alignment.

## 9 SLOPE STABILITY

We used the SLIDE program to evaluate slope stability of the reconstructed, drained embankment section recommended below. The results of our analysis indicate a Factor of Safety (FS) of about 1.4 for static conditions, and a FS of about 1.1 for pseudostatic (earthquake) conditions.

We used conservative soil parameters to model the backslope of the reconstructed embankment section within the zone between the drain "blanket" and rock surface (i.e., phi angle = 20° and cohesion = 50 psf). For pseudostatic conditions, we used a seismic coefficient of 0.125.

Based on our analysis we expect that the recommended reconstructed, drained embankment section will be appropriately stable. We show the graphical output from the slope stability analysis (including soil parameters) in the Appendix.

## 10 RECOMMENDATIONS

### 10.1 Reconstructed, Drained Embankment Section

We recommend the following for a reconstructed, drained embankment section:

- Reconstruct the embankment section the full length of the distressed area, approximately 140 feet along Steele Canyon Road.

- Construct a minimum 12 foot wide toe bench (keyway), established within "in-place" Unit 3 weathered rock as identified by BCI.

- Establish the toe bench level to slope from elev. 149 at the south end to elev. 143 at the north end, defined by a 1.5H:1V (horizontal;vertical) "control-line" from the re-established outer-roadway hinge. The actual toe bench elevation may require adjustment during construction to account for local variation of Unit 3 rock materials (particularly at the ends).
- We recommend that the excavation backslope extend at 1:1 from the heel of the toe to the top of Unit 3 rock materials, then follow the rock surface to where a 1:1 backslope can be constructed to maintain one-lane access through the site. Some trimming into the inboard slope may be necessary to maintain a single lane for traffic.

- Drain the buttress section with a 2-foot-thick "blanket" of ¾-inch crushed rock placed along the base of the toe bench and extended up the backslope to within 10 feet of road level to intercept/relieve subsurface water infiltration. Wrap the crushed rock with filter fabric (such as Mirafi 140N or equivalent) to prevent mobilization of fines into the drain.

- Collect water from the drain blanket with a 4-inch diameter perforated pipe installed along the heal of the bench and connect to a solid pipe outlet relieved by gravity. Daylight the pipe outlet at a location where it can be protected and maintained.

- Re-use Unit 1 fill materials as compacted embankment and place to 90% (per CTM 216) relative compaction (to 95% within 30-inches of finished road grade). Some materials may be wet upon excavation and require drying prior to placement.

- Construct the exterior finished slope to 2H:1V (or flatter) and trim to match the existing slope. Grade the finished slopes within the area of work to prevent surface water from ponding.

- Along the inner road area, construct a 24-inch wide underdrain to a minimum depth of 6 feet. Use ¾-inch crushed rock wrapped in filter fabric (such as Mirafi 140N or equivalent), with gravity relief downslope (north) of the slide area. Place 2 feet of native clayey soil at the surface (compacted to 95% per CTM 216) to inhibit surface infiltration into the drain. Discharge the underdrain downslope via cross culvert (within permeable backfill) at a suitable location.

- Reconstruct the pavement section.

- Re-establish, control and direct surface drainage from the road away from the slipout area (e.g., by means of an asphalt-concrete dike along the outer edge of road) and prevent surface water from ponding. Regular maintenance should be provided along the inner-roadway ditch to this area clear of debris from cut-slope sloughing.

# 11 CONSTRUCTION CONSIDERATIONS

## 11.1 Excavation and Shoring

We expect that excavation of soils and rock to the base of the toe bench can be achieved using typical heavy construction 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., shoring, 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.

## 11.2 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.

# 12 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, grading/foundation plans, and specifications prior to construction.
- Monitor construction to check and document our report assumptions. BCI should, at a minimum, review the toe bench excavation, construction backslope, subdrainage requirements, and drain outlet area.
- 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.

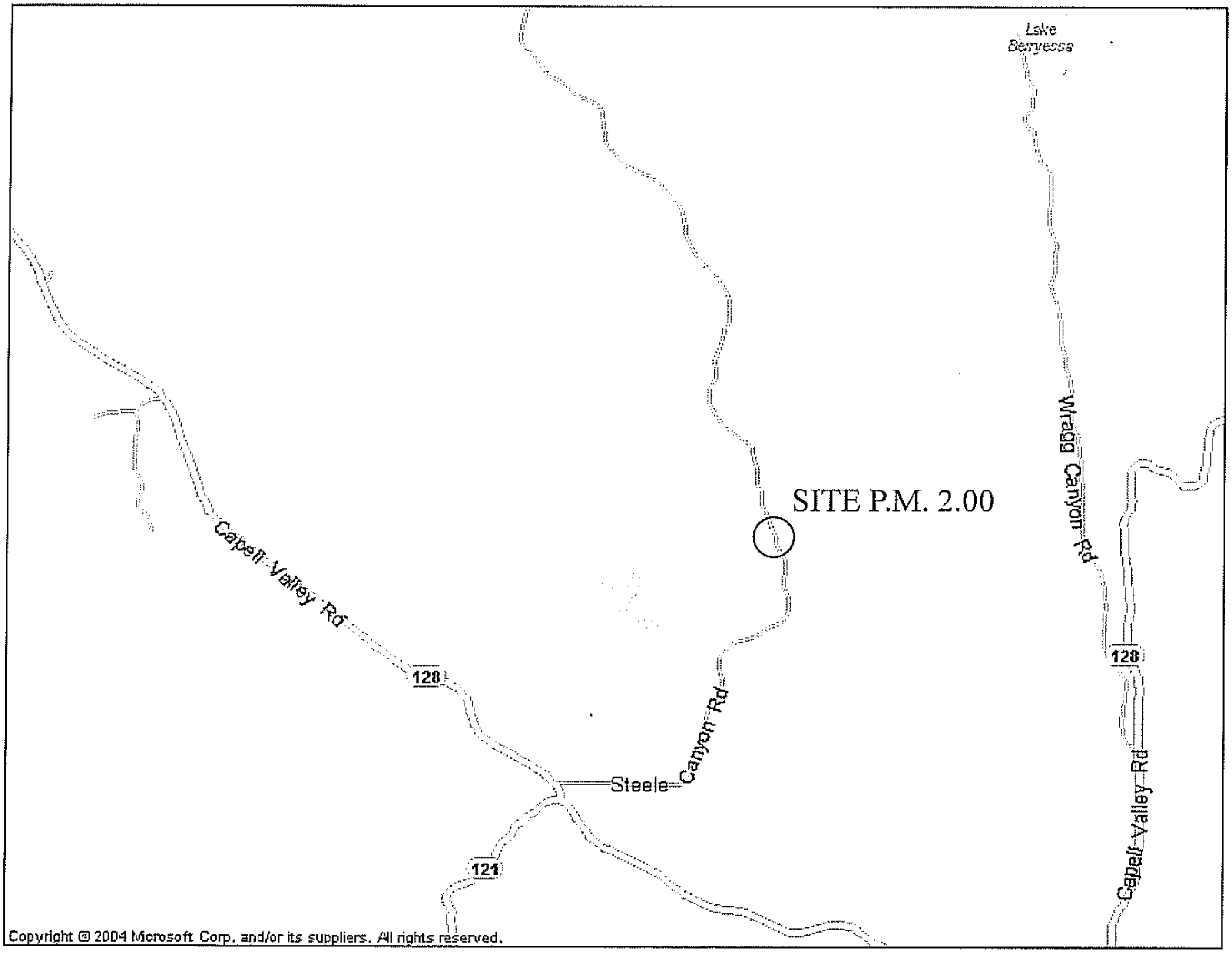
## 13 LIMITATIONS

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, rock, 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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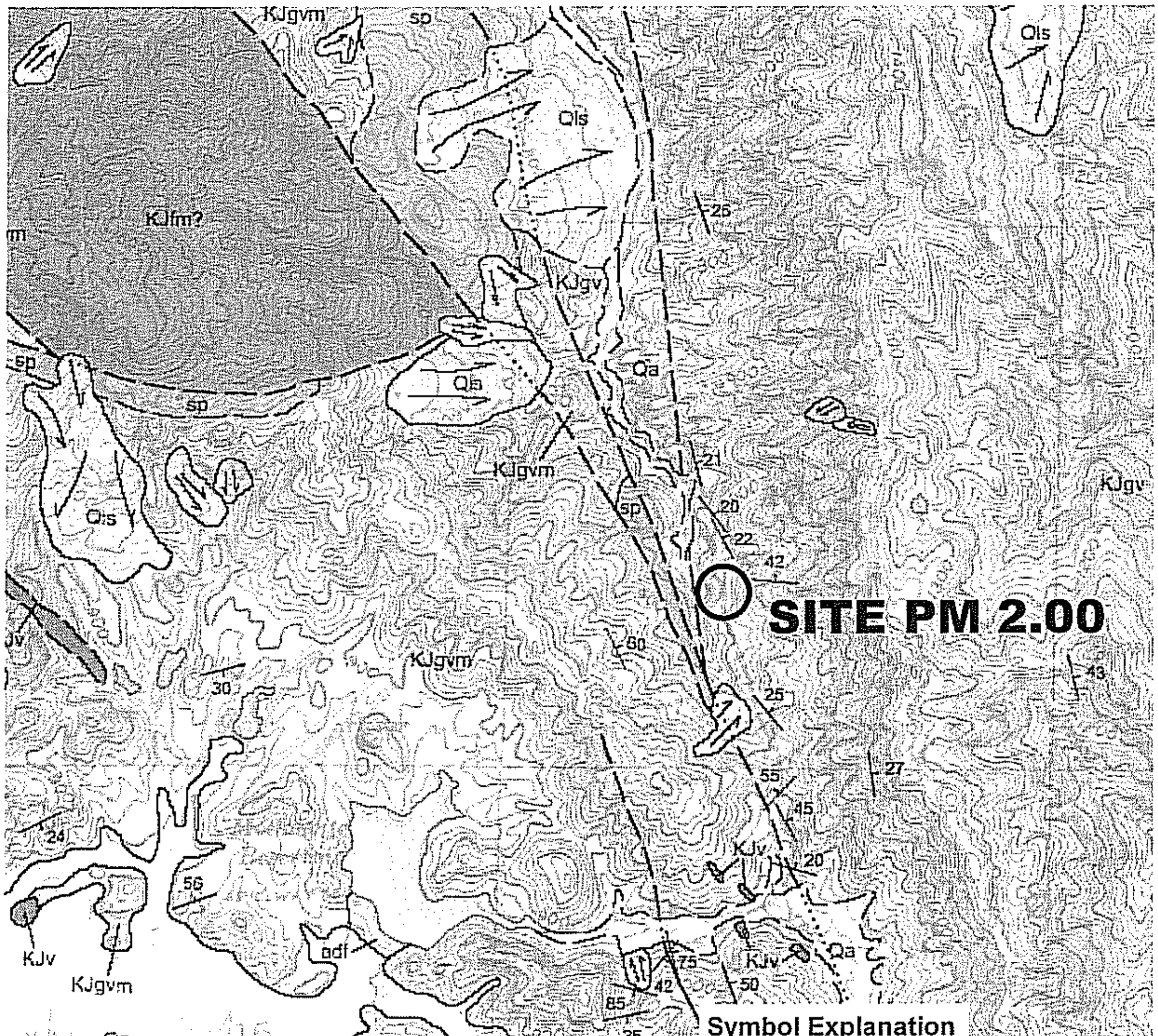
# VICINITY MAP

Steele Canyon Road at P.M. 2.00
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 2.00
Napa County, California

File No.1216.1

July 2007

Figure 2

## APPENDIX

- Log of Test Borings
- Laboratory Test Results
- Slope Stability Analysis

| DIST. | COUNTY | ROUTE | POST MILES TOTAL PROJECT | SHEET NO. | TOTAL SHEETS |
|---|---|---|---|---|---|
| 03 | NAP | CR | | | |

CERTIFIED ENGINEERING GEOLOGIST

PLANS APPROVAL DATE


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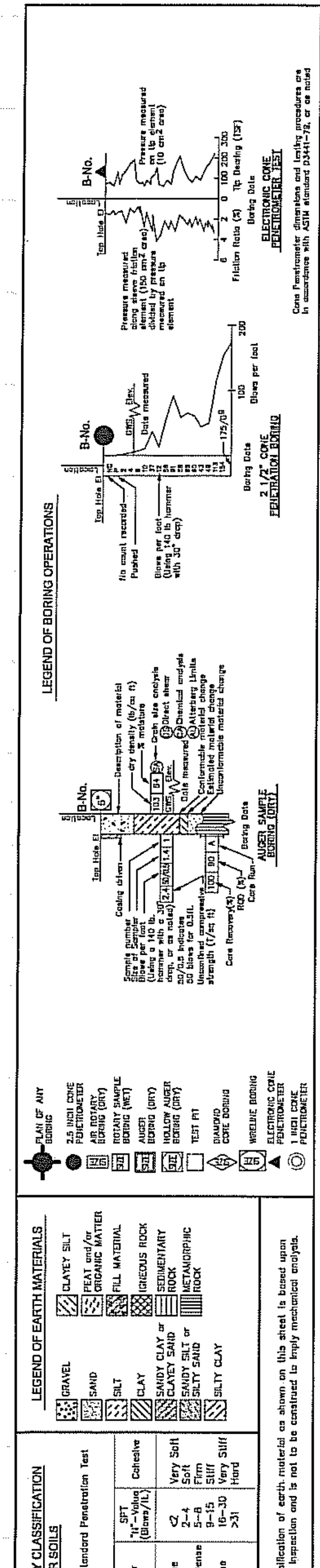
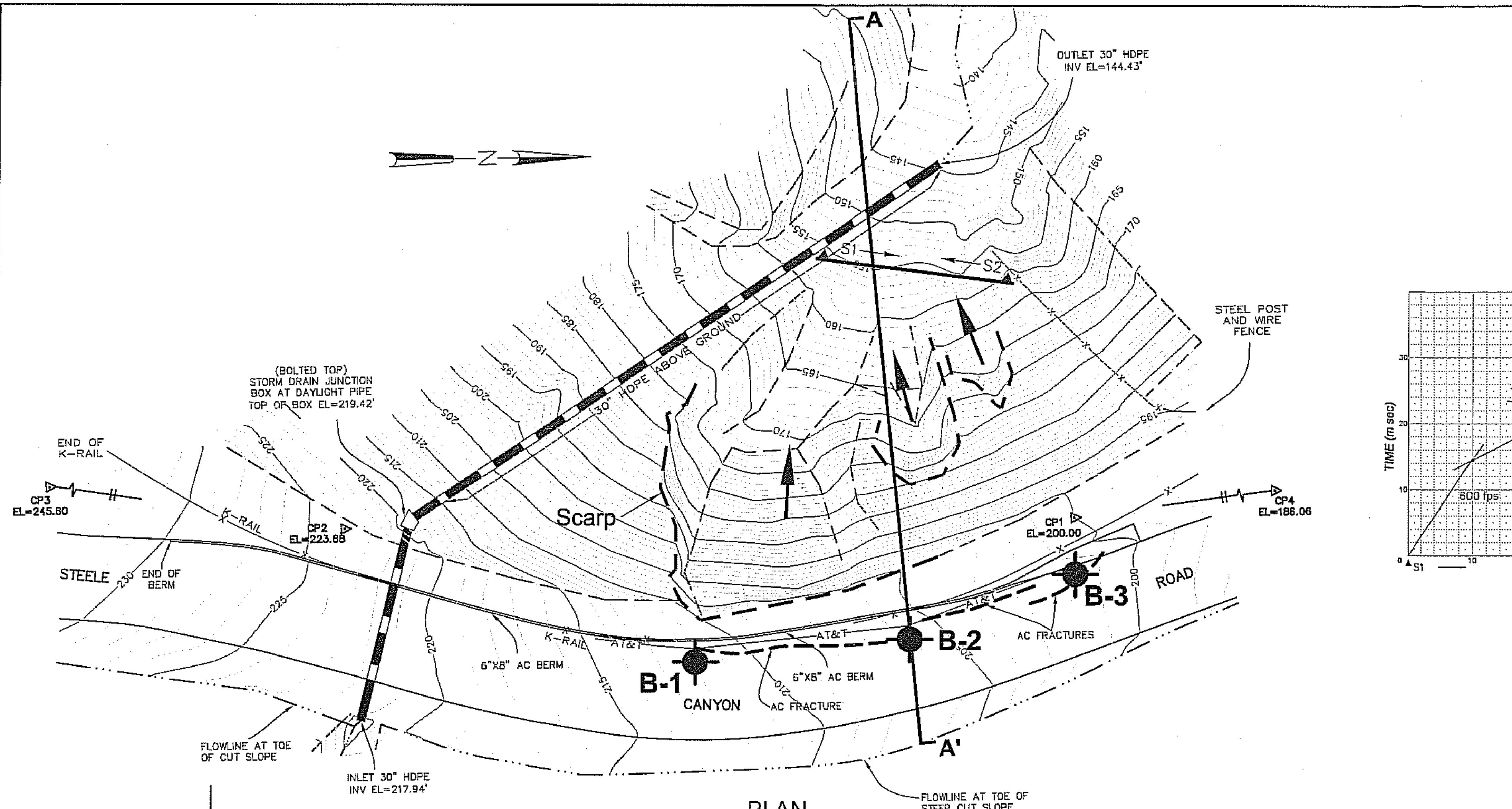
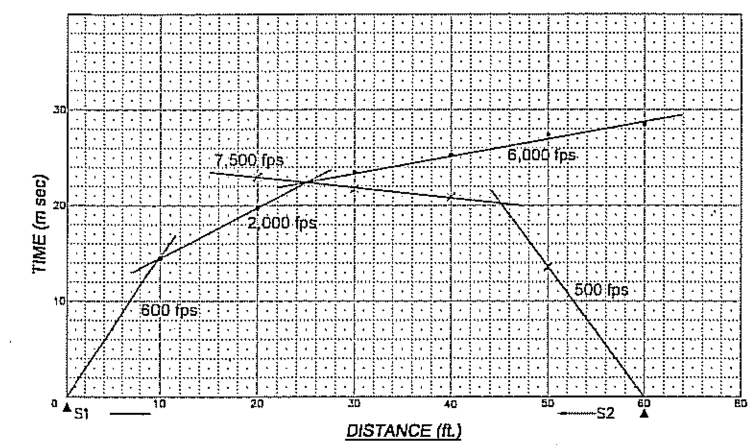
File No. 1216.1

REGISTERED GEOLOGIST WILLIAM E. NICHOLS No. 2229 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

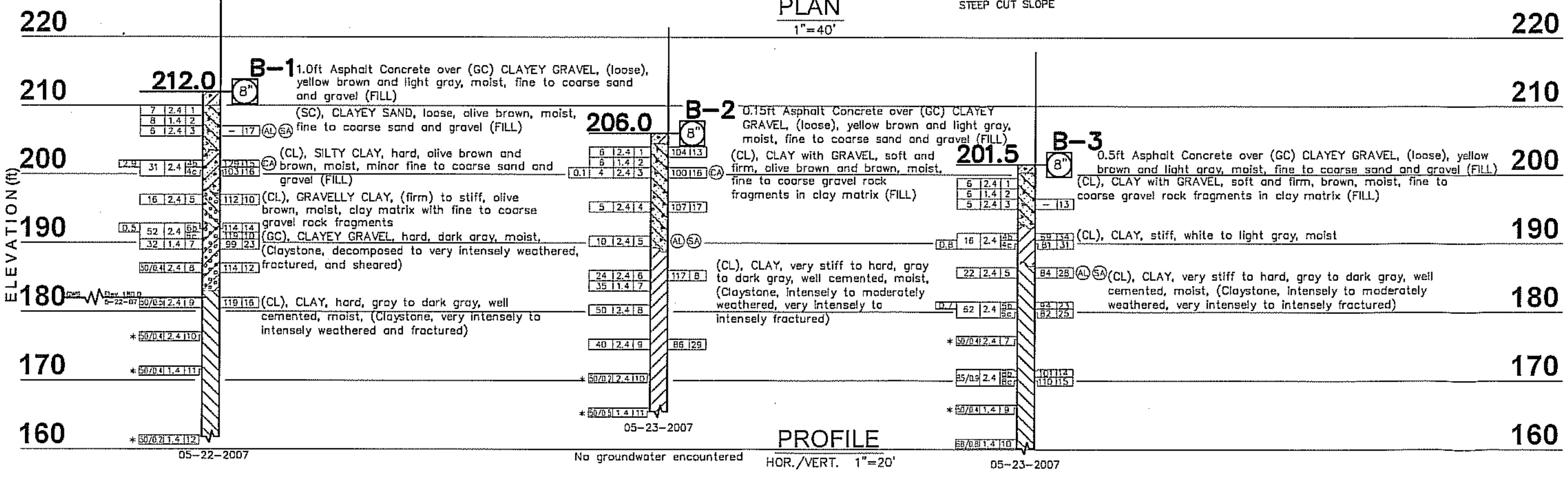



***REFRACTION SEISMIC RECORD***

NOTES:
1. Field classification of soils was in accordance with ASTM D 2488-00 "Description and Identification of Soils (Visual-Manual Procedure)".
2. Standard Penetration tests were performed in accordance with ASTM D 1586-99 using a hammer operated with an automated drop system. Drill rods were 1 5/8-inch diameter "A"-rods; sampler was driven with brass liners.
3. "2.4 inch sampler": ID=2.4 inch, OD=3.0 inch. Driven in same manner as SPT ("1.4 inch") sampler.
4. The length of each sampled interval is shown graphically on the boring log. Whole number blow counts ("N") represent the "standard penetration resistance" interval in accordance with ASTM D1586-99. Where less than 1 foot of penetration is achieved, the blow count shown is for that fraction of the "standard penetration resistance" interval actually penetrated.
5. Where indicated by an asterisk (*) the number of blows shown is for only that fraction of the initial 0.5 ft. "seating drive" interval penetrated.
6. Consistency of soils shown in ( ) where estimated.
7. Groundwater surface (GWS) elevations in the borings indicated on the Log of Test Boring Sheets reflect the fluid level in the borings on the specified date.
8. Groundwater surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
9. Refraction seismic profiles were made using a sledge-hammer actuated, single channel, signal enhancement timer.
10. Boring elevations were provided by Winzler & Kelly.
12. Electronic media for plan view provided by Winzler & Kelly, 2007.




| | | | |
|---|---|---|---|
| DESIGN OVERSIGHT | DRAWN BY | M. D. Robertson | K. A. Chapman — FIELD INVESTIGATOR |
| SIGN OFF DATE | CHECKED BY | W. E. Nichols | DATE May 2007 |

PREPARED FOR
COUNTY OF NAPA
DEPARTMENT OF TRANSPORTATION

PROJECT ENGINEER

BRIDGE NO.

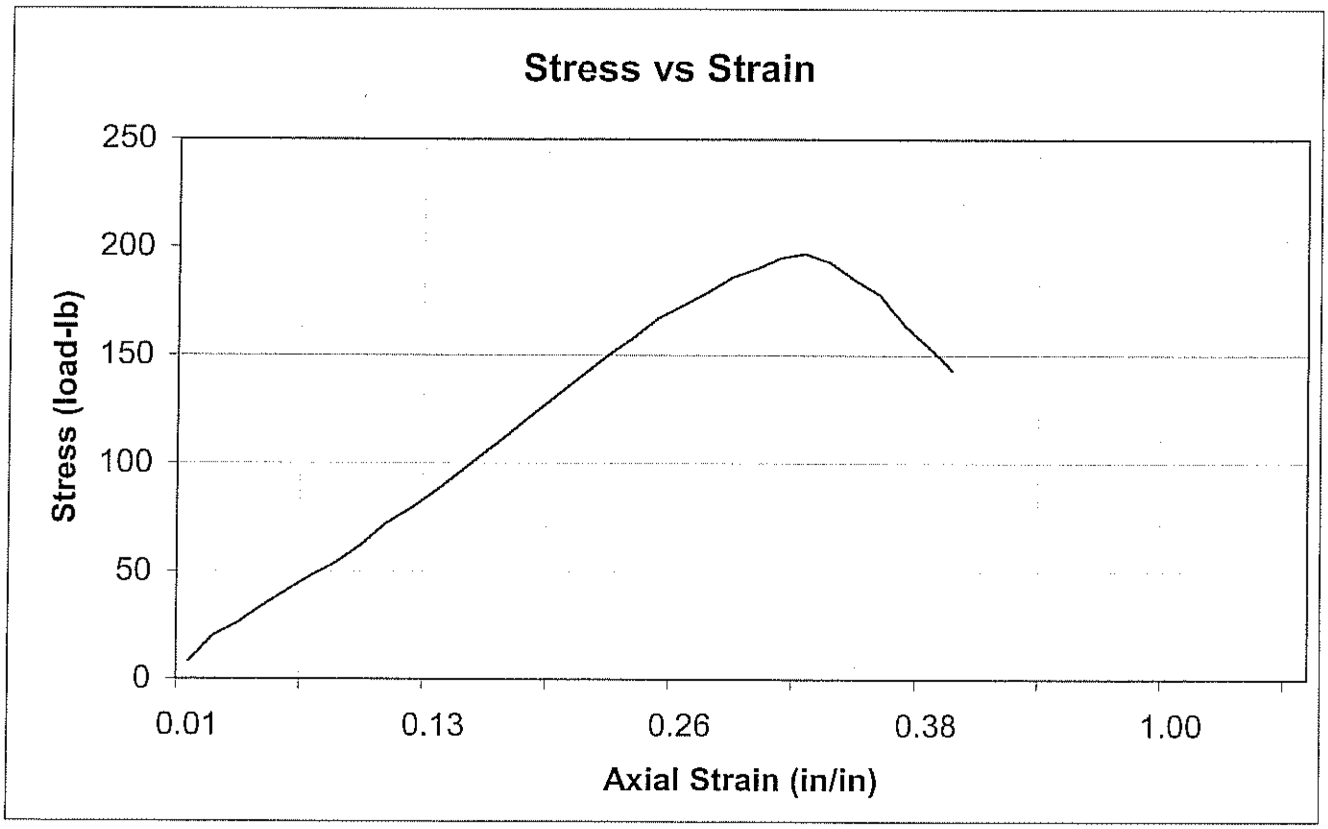
POST MILE

# STEELE CANYON ROAD AT PM 2.00

# LOG OF TEST BORINGS

**Project**
Steele Canyon Rd. P.M. 2.00
**Project Number**
1216.1
**Sample Number**
B1-4b
**Material Description**
Olive silty clay/clayey silt
**Tested By**
JBR

**ASTM D 2166-00**

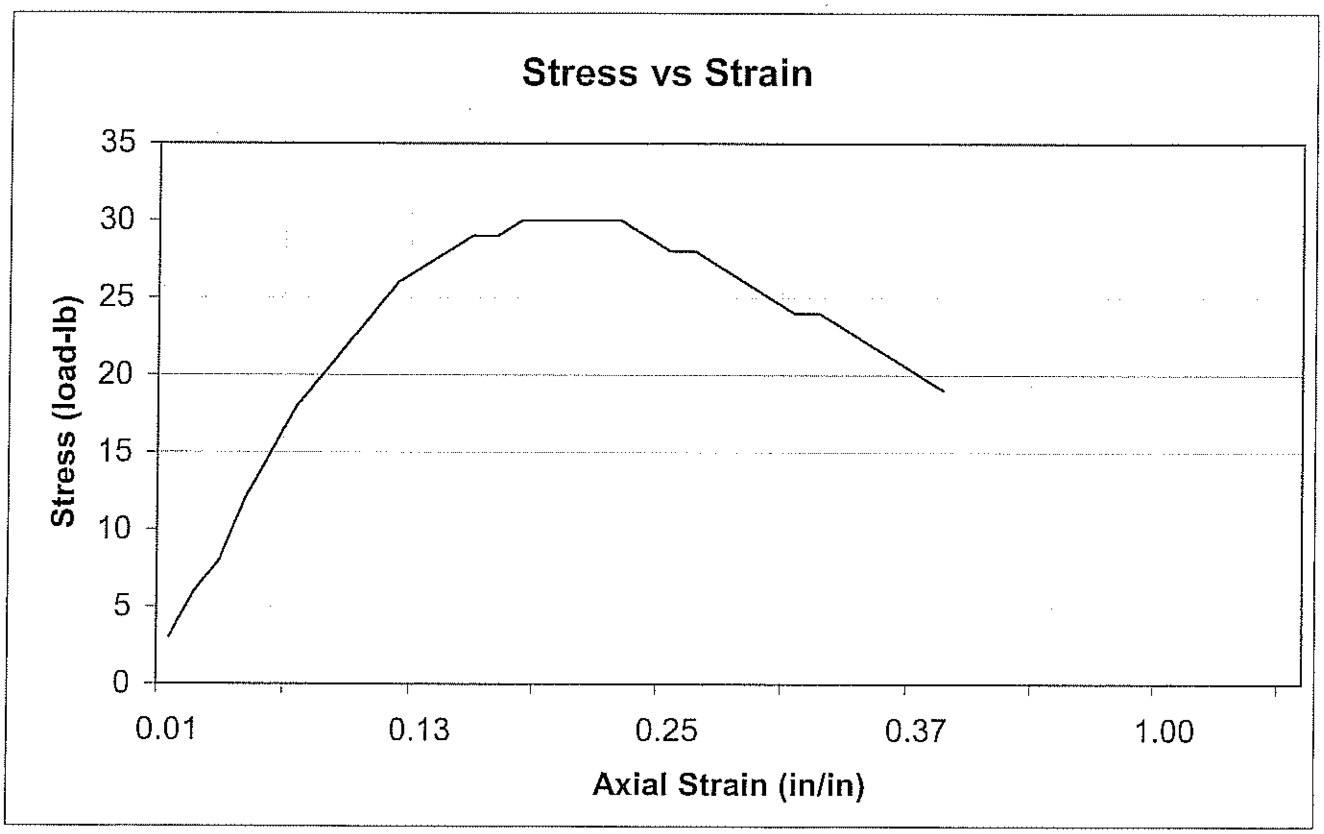



| | |
|---|---|
| Total Density (pcf) | 148.8 |
| Dry Density (pcf) | 129.3 |
| % Moisture | 15.1 |

Unconfined Compressive Strength (tsf) 2.94

**Project**
Steele Canyon Rd. P.M. 2.00
**Project Number**
1216.1
**Sample Number**
B1-6b
**Material Description**
Dark greenish gray clay with claystone
**Tested By**
JBR

**ASTM D 2166-00**

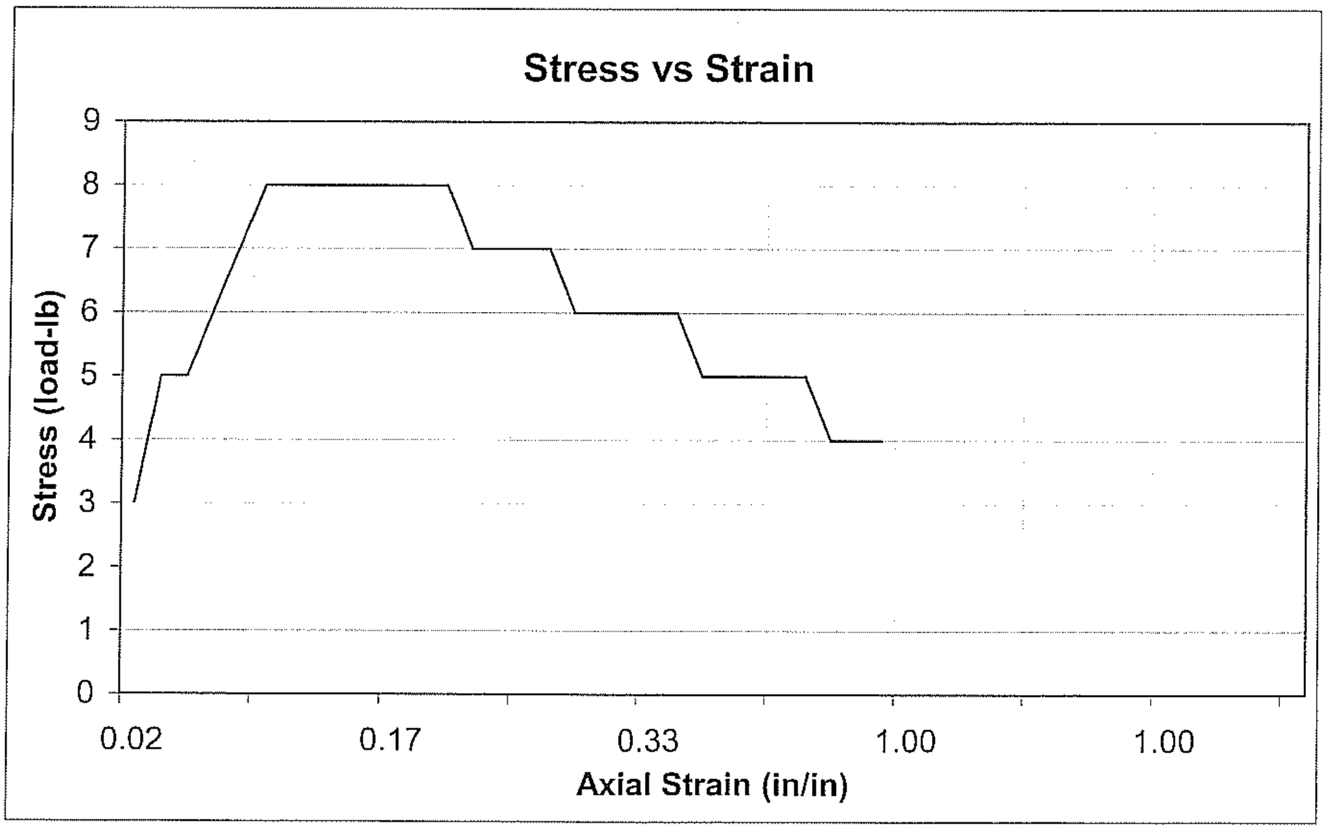



| Total Density (pcf) | 129.8 |
|---|---|
| Dry Density (pcf) | 114.4 |
| % Moisture | 13.5 |

Unconfined Compressive Strength (tsf) 0.46

**Project**
Steele Canyon Rd. P.M. 2.00
**Project Number**
1216.1
**Sample Number**
B2-3b
**Material Description**
Olive clay with claystone
**Tested By**
JBR

**ASTM D 2166-00**

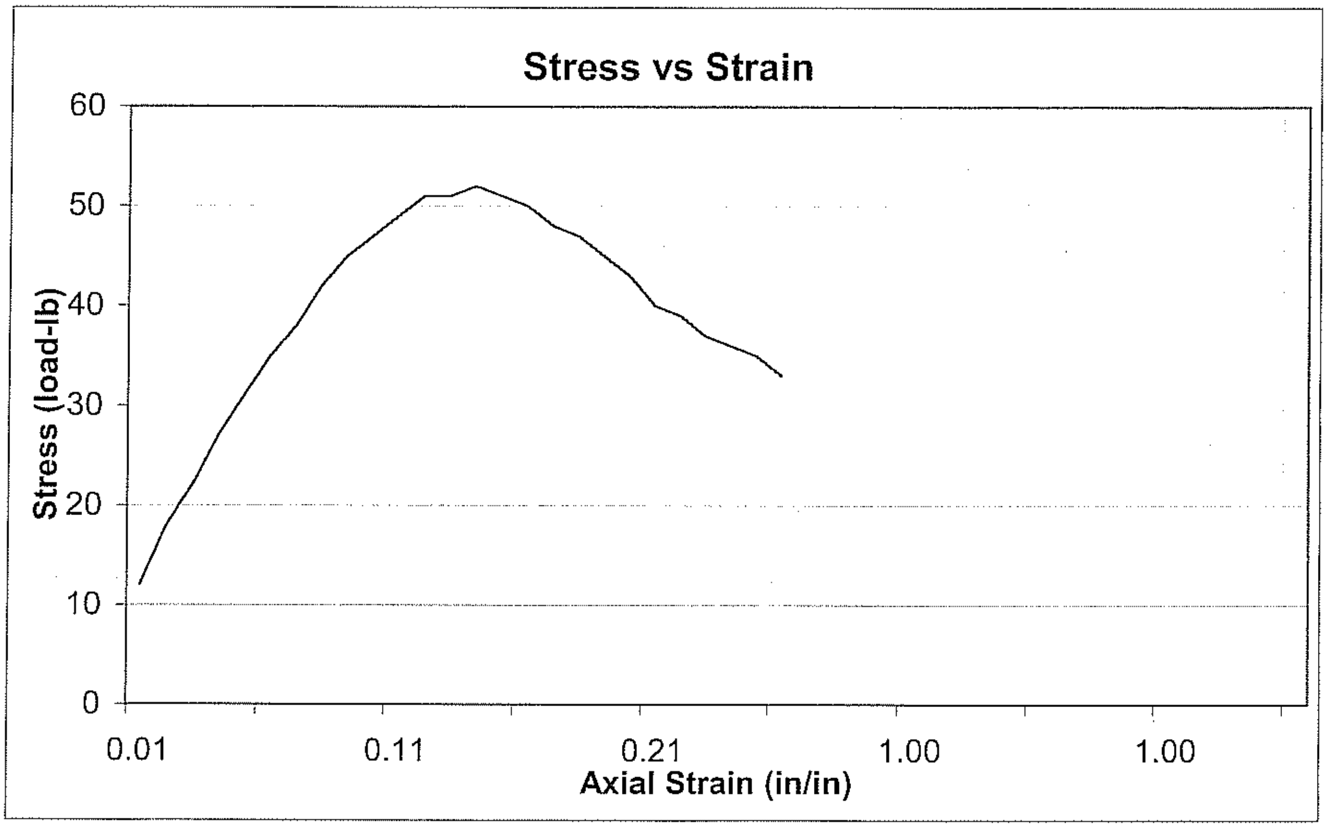



| | |
|---|---|
| Total Density (pcf) | 115.5 |
| Dry Density (pcf) | 99.5 |
| % Moisture | 16.1 |

Unconfined Compressive Strength (tsf) 0.12

**Project**
Steele Canyon Rd. P.M. 2.00
**Project Number**
1216.1
**Sample Number**
B3-4c
**Material Description**
Light greenish gray silt
**Tested By**
JBR

**ASTM D 2166-00**

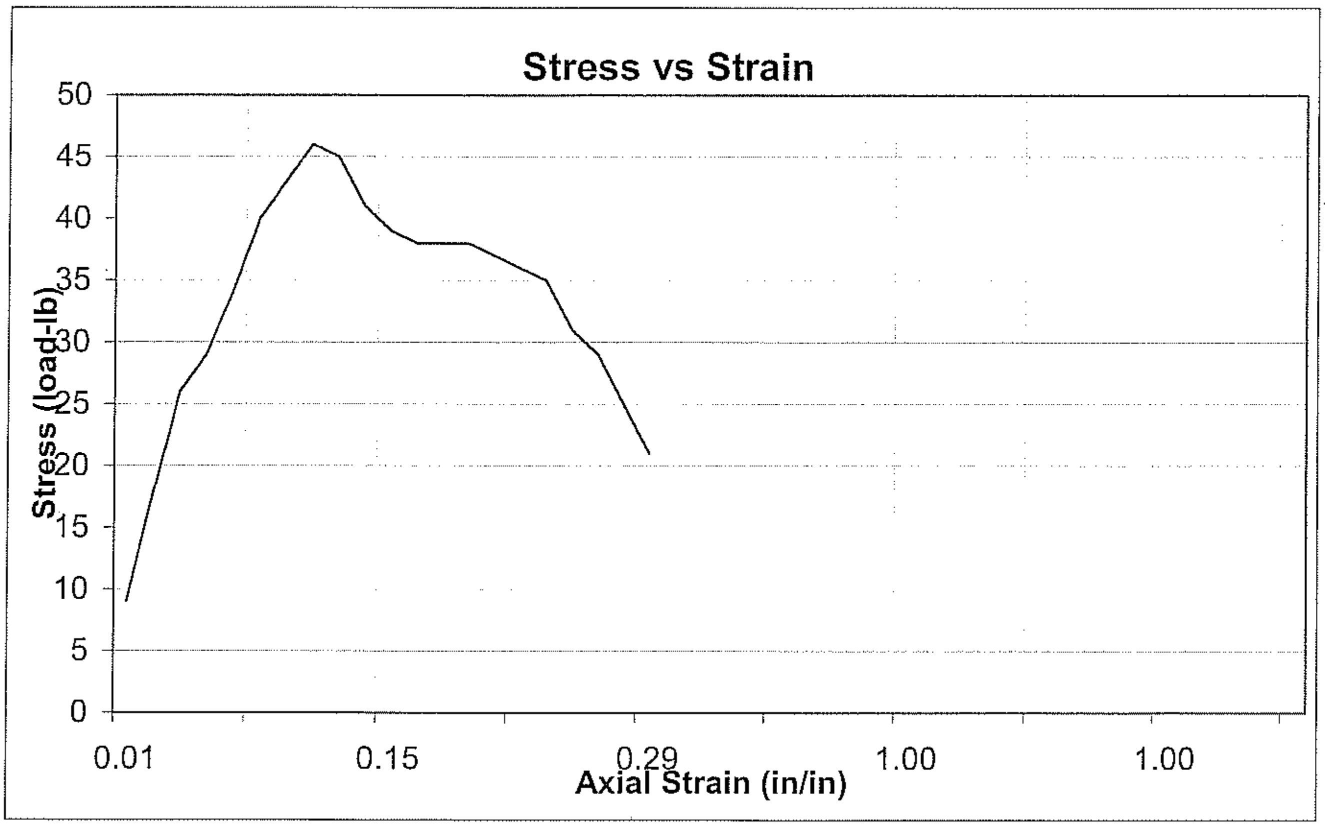



| | |
|---|---|
| Total Density (pcf) | 105.2 |
| Dry Density (pcf) | 80.5 |
| % Moisture | 30.7 |

Unconfined Compressive Strength (tsf) 0.79

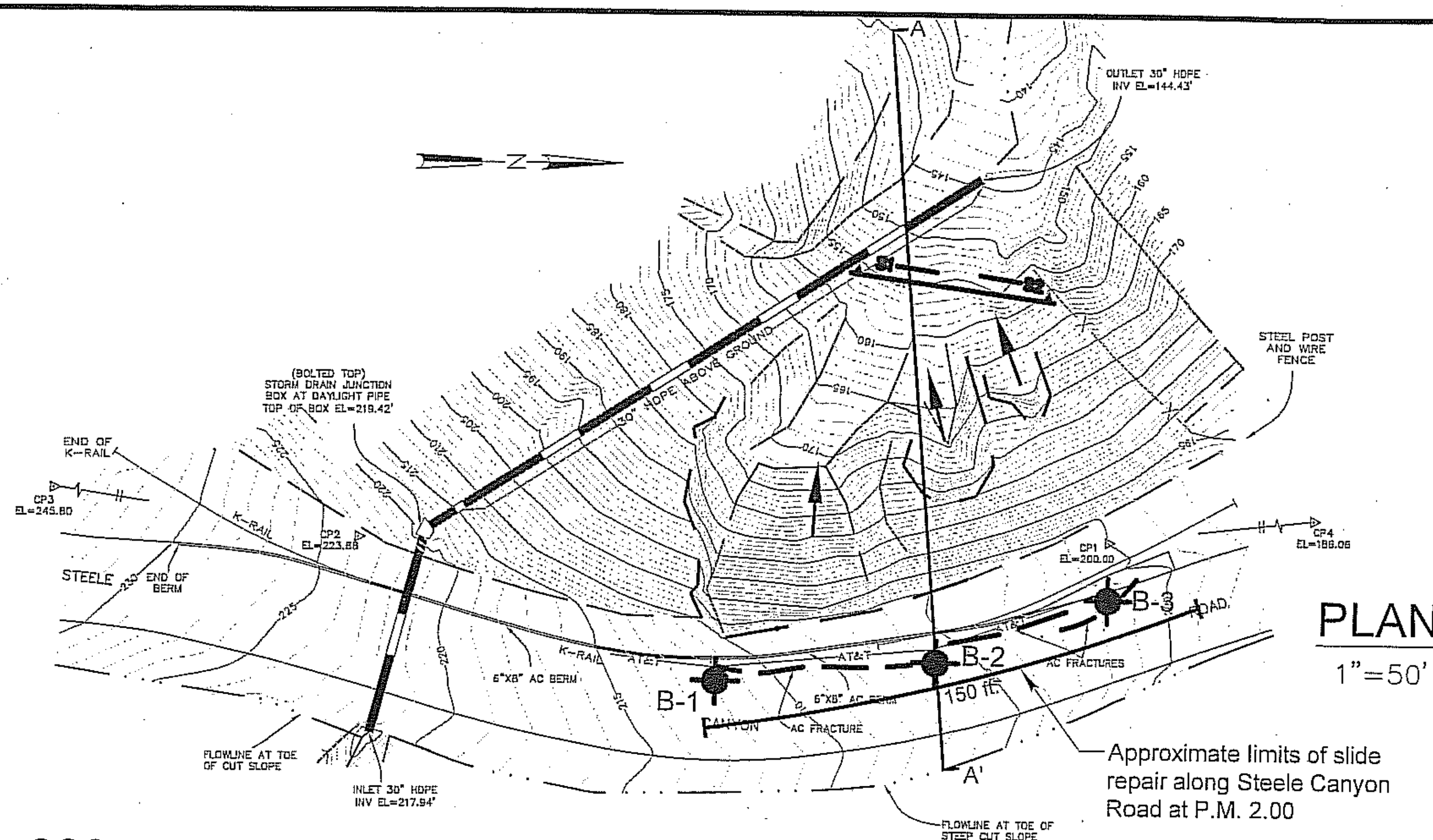
**Project**
Steele Canyon Rd. P.M. 2.00
**Project Number**
1216.1
**Sample Number**
B3-6b
**Material Description**
Olive silt
**Tested By**
JBR

**ASTM D 2166-00**

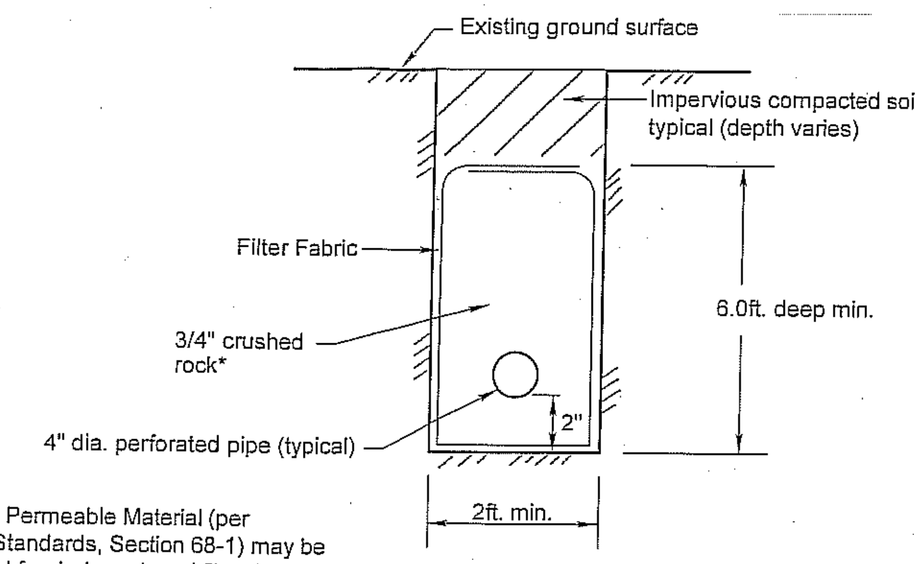



| Total Density (pcf) | 115.6 |
|---|---|
| Dry Density (pcf) | 94.1 |
| % Moisture | 22.9 |

Unconfined Compressive Strength (tsf) 0.71

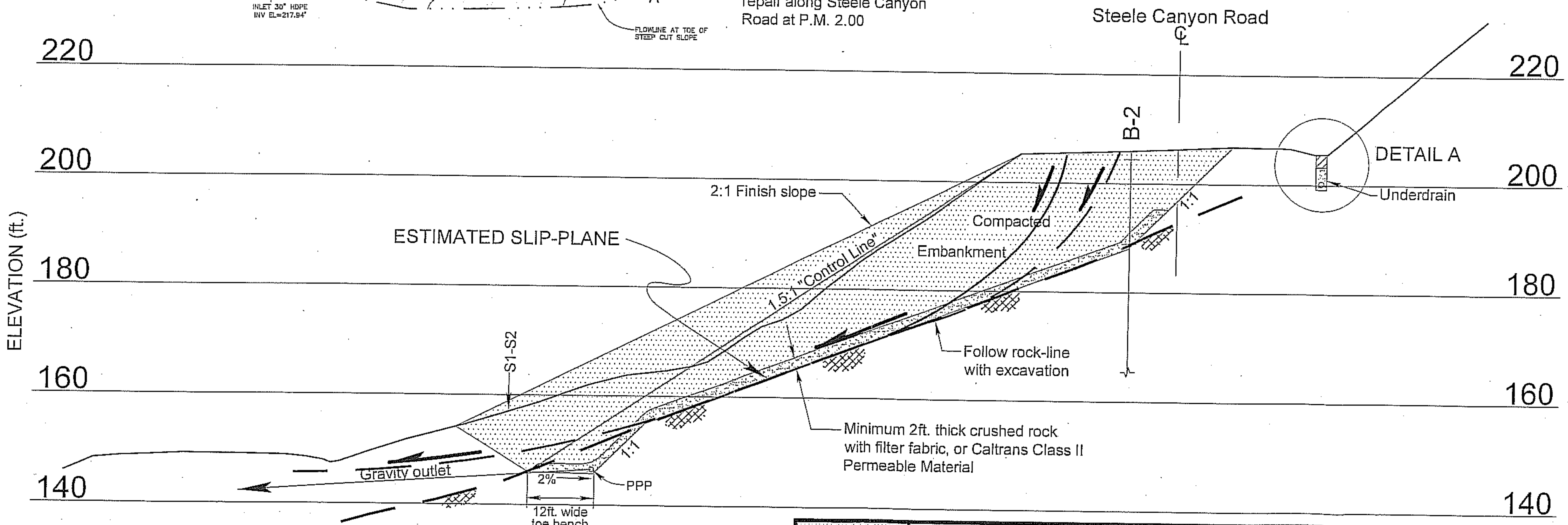



PLAN
1"=50'




* Class II Permeable Material (per Caltrans Standards, Section 68-1) may be substituted for drain rock and filter fabric.

UNDERDRAIN DETAIL A
Not to Scale




CROSS SECTION A-A'
1"=20'

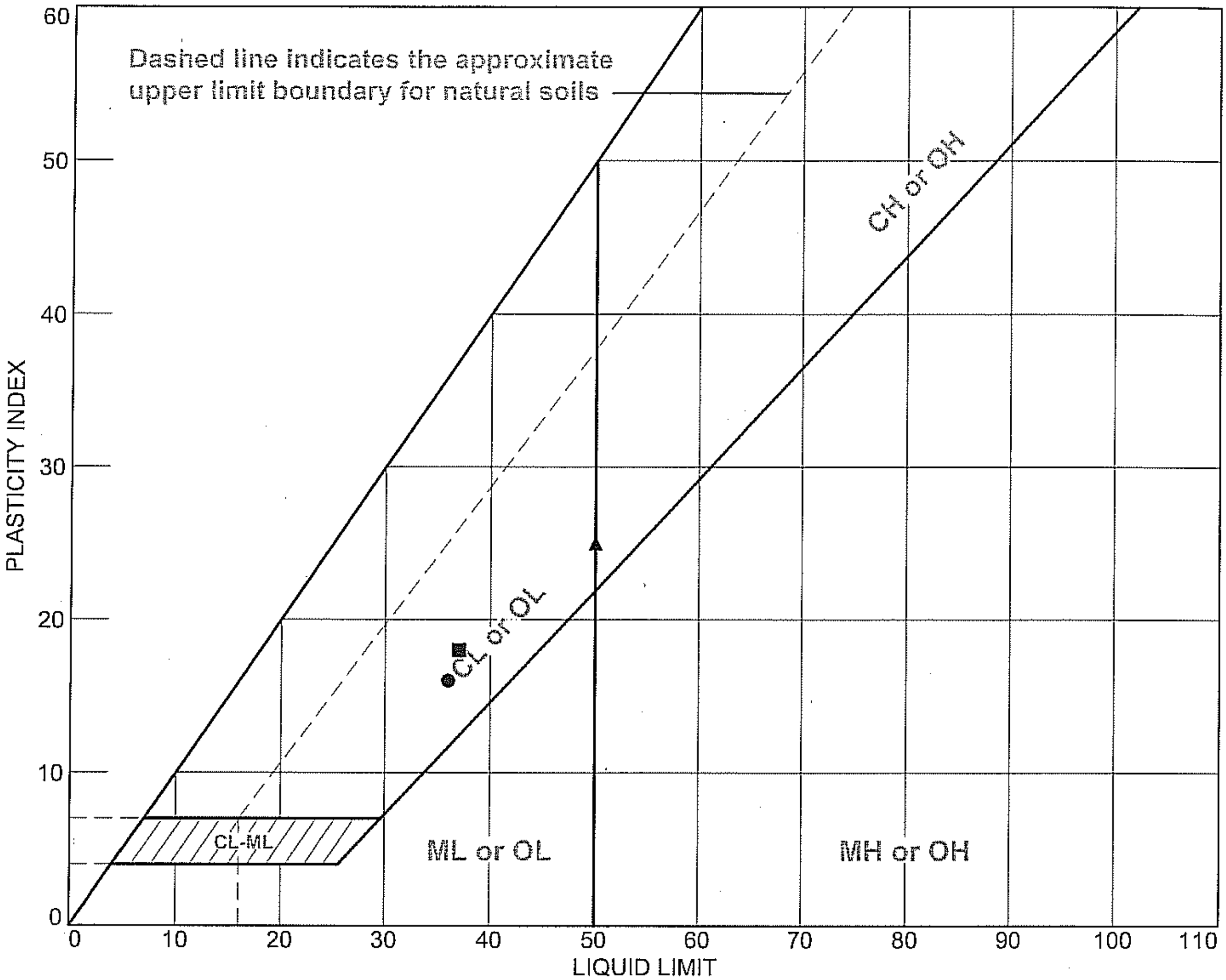
blackburn consulting
2437 Front Street
West Sacramento,CA 95691
Phone: (916) 375-8706
Fax: (916) 375-8709
www.blackburnconsulting.com

**RECONSTRUCTED EMBANKMENT**
**CROSS SECTION A-A'**
STEELE CANYON ROAD SLIDE PM 2.00

| | |
|---|---|
| File No. 1216.1 | |
| July 2007 | |
| Figure 3 | |

07-13-07 1216.1 Steele Canyon Road PM 2.00 Figure 3.dwg

# LIQUID AND PLASTIC LIMITS TEST REPORT



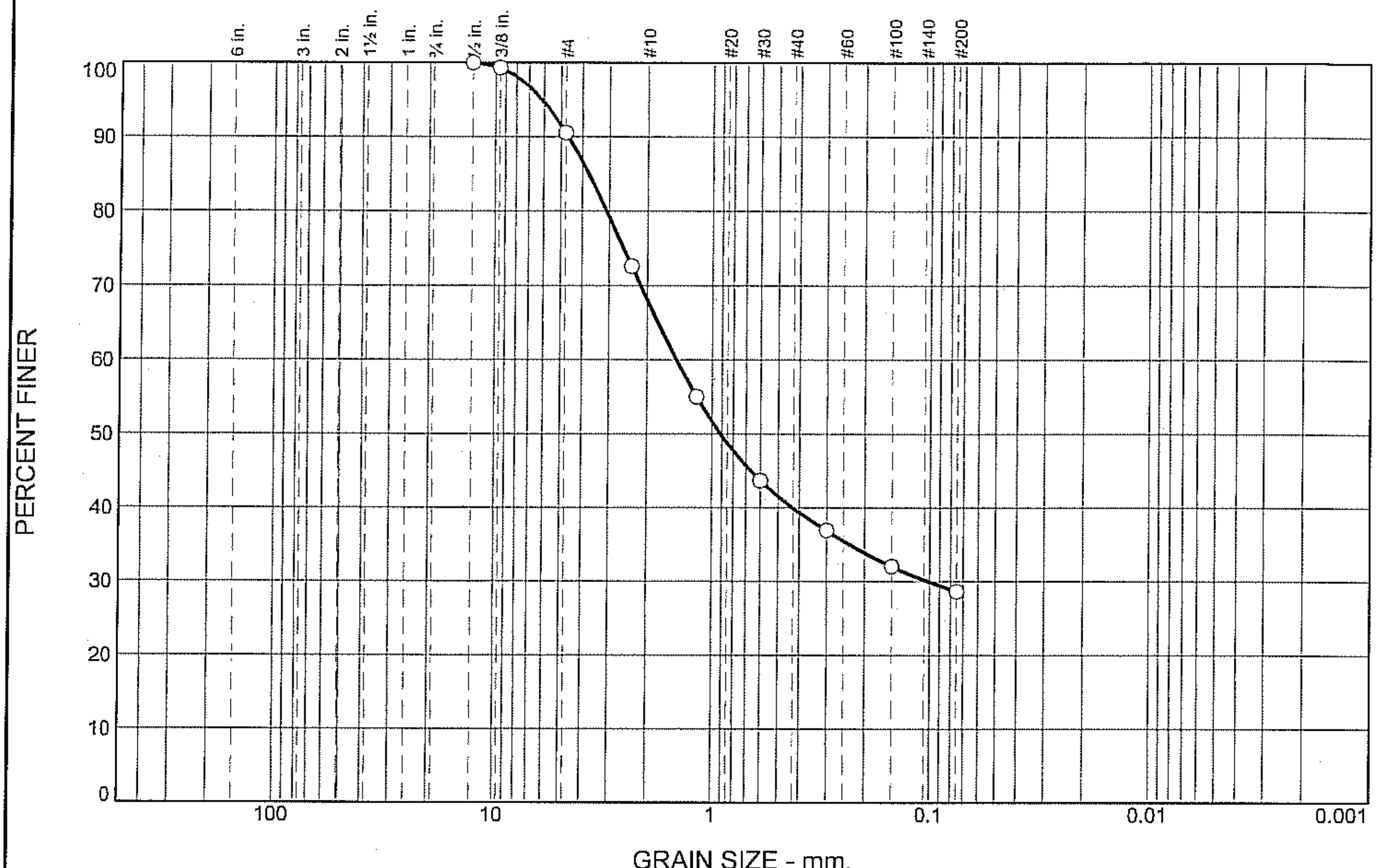

| SOIL DATA | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| SYMBOL | SOURCE | SAMPLE NO. | DEPTH | NATURAL WATER CONTENT (%) | PLASTIC LIMIT (%) | LIQUID LIMIT (%) | PLASTICITY INDEX (%) | USCS |
| ● | | B1-3c | 6.0-6.5 ft | | 20 | 36 | 16 | CL |
| ■ | | B2-5b | 15.5-16.0 ft | | 19 | 37 | 18 | CL |
| ▲ | | B3-5b | 15.5-16.0 ft | | 25 | 50 | 25 | CH |

**Blackburn Consulting**

**W. Sacramento, CA**

**Client:**
**Project:** Steele Canyon Rd. P.M. 2.00
**Project No.:** 1216.1 **Figure**

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 9.4 | 22.5 | 28.2 | 11.3 | 28.6 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 1/2" | 100.0 | | |
| 3/8" | 99.3 | | |
| #4 | 90.6 | | |
| #8 | 72.6 | | |
| #16 | 55.0 | | |
| #30 | 43.7 | | |
| #50 | 36.9 | | |
| #100 | 32.0 | | |
| #200 | 28.6 | | |

* (no specification provided)

**Material Description**

Olive brown clayey sand

**Atterberg Limits**

PL= 20 LL= 36 PI= 16

**Coefficients**

$D_{85}$= 3.7310 $D_{60}$= 1.4640 $D_{50}$= 0.9129
$D_{30}$= 0.1014 $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= SC AASHTO= A-2-6(1)

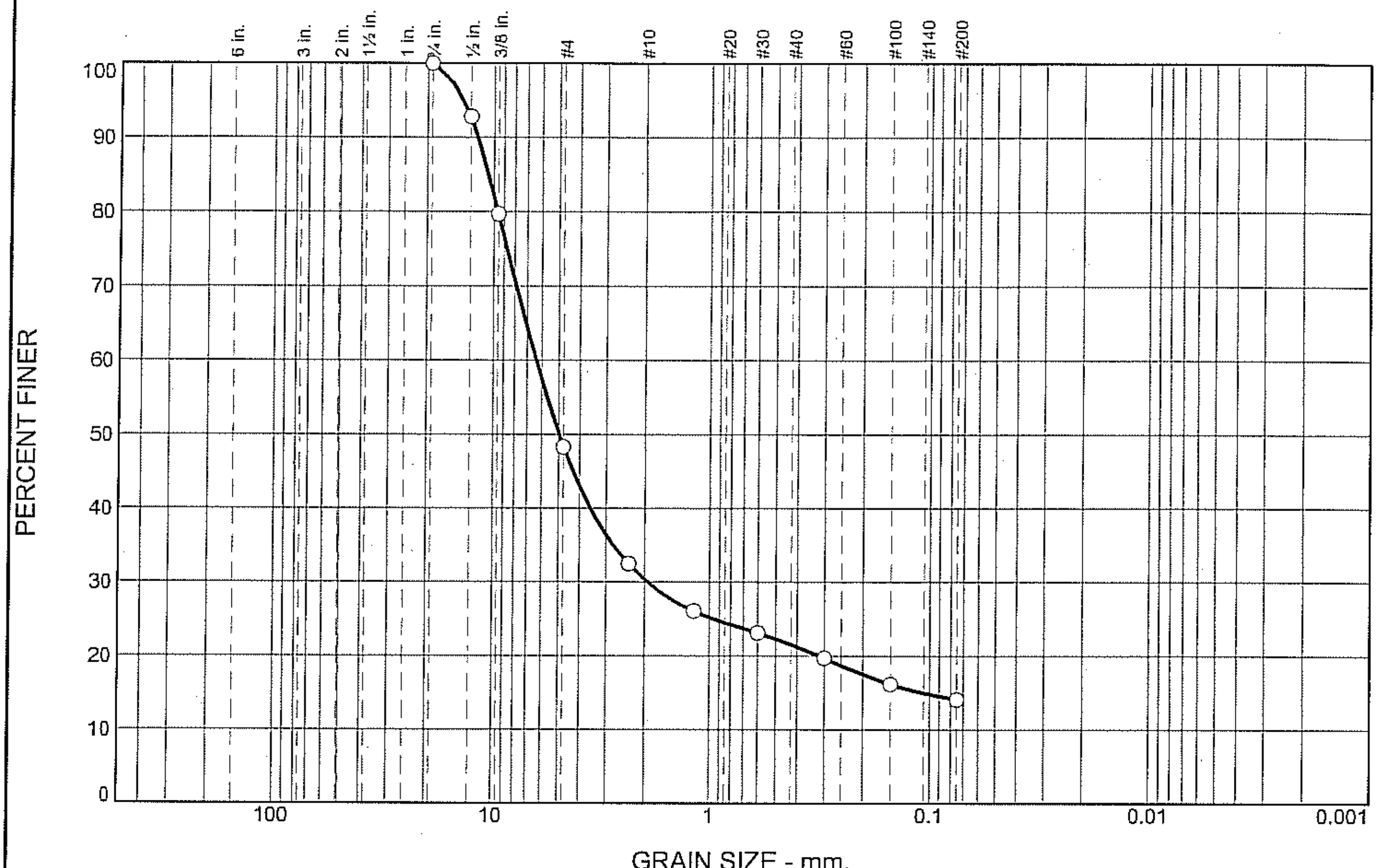
**Remarks**

Sample included cemented fragments

**Sample Number:** B1-3c **Depth:** 6.0-6.5 ft **Date:** 6-15-07

| **Blackburn Consulting**<br>**W. Sacramento, CA** | **Client:**<br>**Project:** Steele Canyon Rd, P.M. 2.00<br>**Project No:** 1216.1 **Figure** |
|---|---|

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 51.7 | 17.9 | 8.9 | 7.4 | 14.1 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/4" | 100.0 | | |
| 1/2" | 92.8 | | |
| 3/8" | 79.7 | | |
| #4 | 48.3 | | |
| #8 | 32.5 | | |
| #16 | 26.0 | | |
| #30 | 23.1 | | |
| #50 | 19.7 | | |
| #100 | 16.2 | | |
| #200 | 14.1 | | |

* (no specification provided)

**Material Description**

Olive brown clayey gravel

**Atterberg Limits**

PL= 19 LL= 37 PI= 18

**Coefficients**

$D_{85}$= 10.5828 $D_{60}$= 6.3596 $D_{50}$= 4.9866
$D_{30}$= 1.9301 $D_{15}$= 0.1069 $D_{10}$=
$C_u$= $C_c$=

**Classification**

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

**Remarks**

Sample included cemented fragments

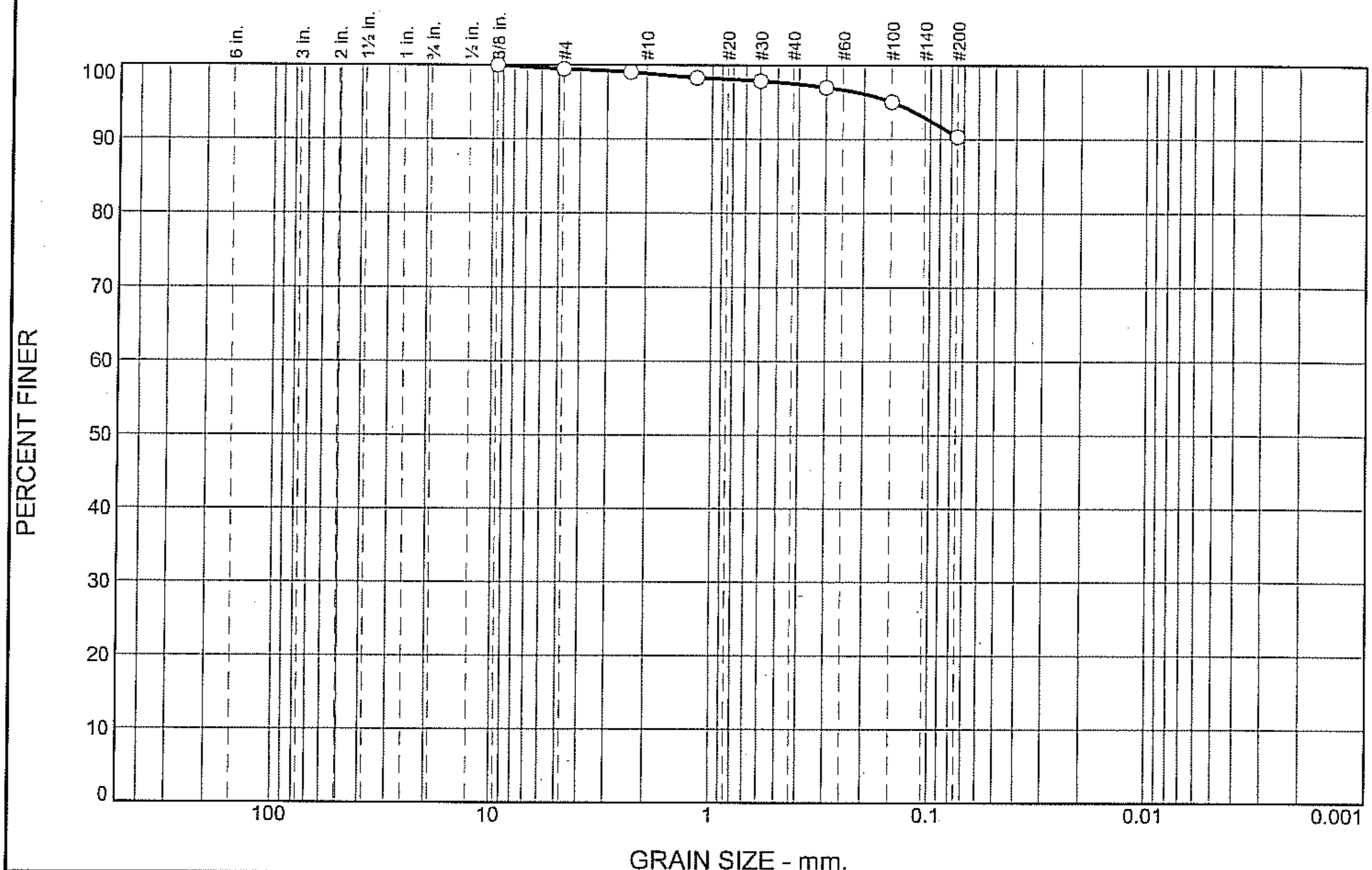
**Sample Number:** B2-5b **Depth:** 15.5-16.0 ft **Date:** 6-15-07

**Blackburn Consulting**

**W. Sacramento, CA**

**Client:**
**Project:** Steele Canyon Rd. P.M. 2.00
**Project No:** 1216.1 **Figure**

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.6 | 0.6 | 1.3 | 7.1 | 90.4 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/8" | 100.0 | | |
| #4 | 99.4 | | |
| #8 | 99.0 | | |
| #16 | 98.3 | | |
| #30 | 97.9 | | |
| #50 | 97.0 | | |
| #100 | 95.1 | | |
| #200 | 90.4 | | |

* (no specification provided)

**Material Description**

Light olive brown fat clay

**Atterberg Limits**

PL= 25 LL= 50 PI= 25

**Coefficients**

$D_{85}$= $D_{60}$= $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= CH AASHTO= A-7-6(25)

**Remarks**

**Sample Number:** B3-5b **Depth:** 15.5-16.0 ft **Date:** 6-15-07

**Blackburn Consulting**

**W. Sacramento, CA**

**Client:**
**Project:** Steele Canyon Rd. P.M. 2.00
**Project No:** 1216.1 **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.PM2.00 Site ID : B1-4B.
Thank you for your business.

* For future reference to this analysis please use SUN # 50743-101310.

---

EVALUATION FOR SOIL CORROSION

| | | |
|---|---|---|
| Soil pH | 6.40 | |
| Minimum Resistivity | 1.53 ohm-cm (x1000) | |
| Chloride | 12.7 ppm | 00.00127 % |
| Sulfate | 21.6 ppm | 00.00216 % |

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.PM2.00 Site ID : B2-3C.
Thank you for your business.

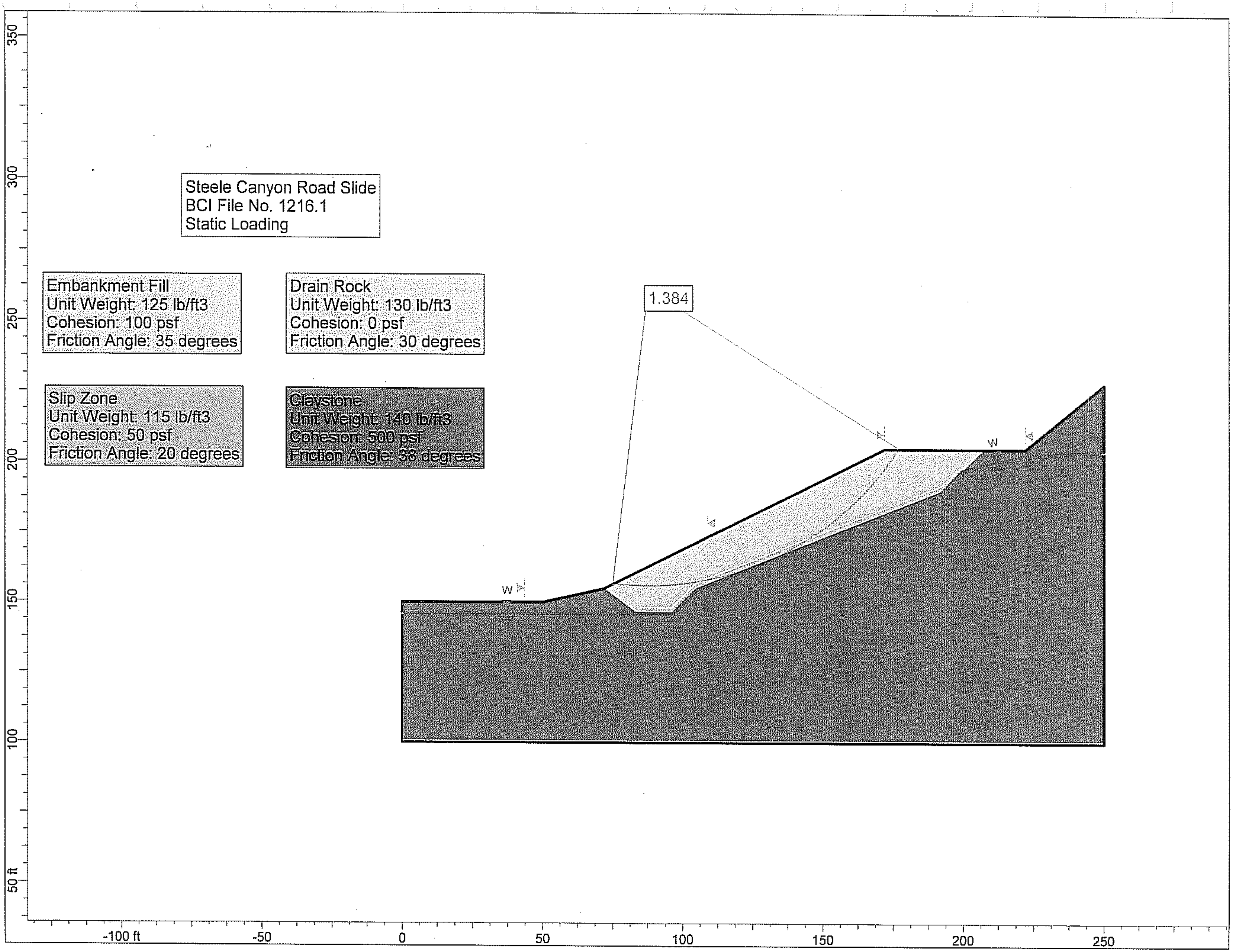
* For future reference to this analysis please use SUN # 50743-101311.

---

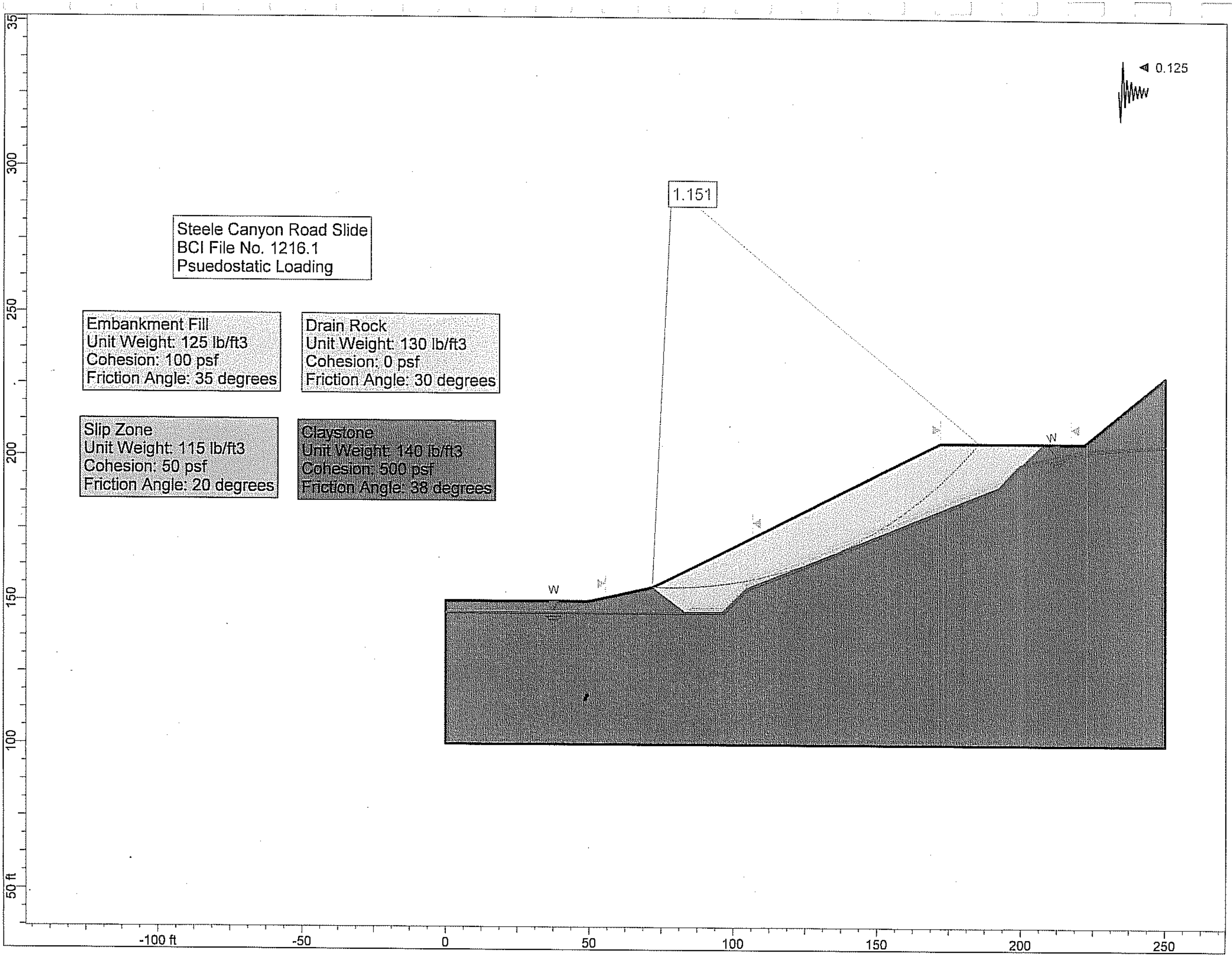
EVALUATION FOR SOIL CORROSION

| | | | |
|---|---|---|---|
| Soil pH | 7.63 | | |
| Minimum Resistivity | 1.26 ohm-cm (x1000) | | |
| Chloride | 14.9 ppm | 00.00149 | % |
| Sulfate | 25.9 ppm | 00.00259 | % |

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



Steele Canyon Road Slide
BCI File No. 1216.1
Static Loading
Embankment Fill
Unit Weight: 125 lb/ft3
Cohesion: 100 psf
Friction Angle: 35 degrees
Drain Rock
Unit Weight: 130 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees
Slip Zone
Unit Weight: 115 lb/ft3
Cohesion: 50 psf
Friction Angle: 20 degrees
Claystone
Unit Weight: 140 lb/ft3
Cohesion: 500 psf
Friction Angle: 38 degrees
1.384
W
W
350
300
250
200
150
100
50 ft
-100 ft
-50
0
50
100
150
200
250


0.125
1.151
Steele Canyon Road Slide
BCI File No. 1216.1
Psuedostatic Loading
Embankment Fill
Unit Weight: 125 lb/ft3
Cohesion: 100 psf
Friction Angle: 35 degrees
Drain Rock
Unit Weight: 130 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees
Slip Zone
Unit Weight: 115 lb/ft3
Cohesion: 50 psf
Friction Angle: 20 degrees
Claystone
Unit Weight: 140 lb/ft3
Cohesion: 500 psf
Friction Angle: 38 degrees
W
W
350
300
250
200
150
100
50 ft
-100 ft
-50
0
50
100
150
200
250