FOR CONTRACT No.: 04-4S5904

INFORMATION HANDOUT

MATERIALS INFORMATION

Foundation Report for Distressed Slope Repair (Bridge No. 35E0035)
Dated March 15, 2011

Foundation Review
Dated July 11, 2011

Dist-Co-Rte-PM: 04-SM-84-22.1

Memorandum

Flex your power!
Be energy efficient!

To: MS. OFELIA ALCANTARA
Supervising Bridge Engineer
Office of Bridge Design - West

Attention: Gordon Danke
Phil Lutz

Date: March 15, 2011

File: 04-SM-84 PM 22.0/22.1
04-4S5901
Efis# 0400002051
Slope Failure Repair
Woodside Road

From: DAVID NESBITT
Transportation Engineer
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

M. Momenzadeh

MAHMOOD MOMENZADEH
Chief, Branch C
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

Subject: Foundation Report for Distressed Slope Repair

This memo supersedes our previous Foundation Report dated February 12, 2011 for the repair of the sliding slope located on the north side of westbound Route 84 (Woodside Road), near the intersection of Southgate Road in the City of Woodside, San Mateo County, California (Figure 1).

SCOPE OF WORK

We have performed a geotechnical investigation to determine the possible causes of the sliding slope, and developed a repair plan. The scope of work includes the following:

- Visual observations of the distressed roadway during our site visits on October 29, 2009 and January 14, 2010.
- Review of Route 84 As-built plans.
- Subsurface exploration consisting of two exploratory borings advanced to an approximate depth of 31.5 ft and 65.0 ft in March 2010.
- One slope inclinometer was installed to a depth of approximately 60 ft in one borehole, and one piezometer was installed to depth of approximately 30 ft in the other borehole. Periodic readings of the slope inclinometer and piezometer were recorded.
- Engineering analyses and preparation of the repair recommendation.

PROJECT SITE DESCRIPTION AND BACKGROUND

Site Description

The subject site is located along the north side of Route 84 (Woodside Road) near the intersection of Southgate Road (Figure 2). At this location, Route 84 is a four lane highway with a center turn lane and shoulders. The roadway is constructed of asphalt concrete (AC), which has been overlaid periodically within the slide area to compensate for the roadway settlement. The subject site is approximately located at the top of a 60 ft high slope. The slope is approximately 2 (H): 1 (V), and is vegetated with grass, shrubs, and small trees. There are no structures located on the slope at the subject site. A city street is located at the bottom of the slope.

Site History

The current configuration of Route 84 (Woodside Road), at the subject site, was completed in the early 1970s. This section of Woodside Road was originally Route 114 according to the as-built plans. According to the Caltrans records, there were three slope failures along westbound Route 84 between Route 280 and Southgate Road in 1983. The repairs were completed in 1984. The first two locations are west of the subject site, and required retaining structures to stabilize the slopes. The slope at the subject site was rebuilt. The repair included removal and re-compaction of the slide material, construction of a drainage blanket, and installation of horizontal drains. An earth buttress was constructed at the toe of the slide repair with material from the adjacent repair site. The horizontal drains continue to function, and periodic readings were recorded.

The Caltrans files indicate that cracking of the roadway appeared shortly after the repair was completed in 1984. Caltrans records indicate that a slope inclinometer and two piezometers were installed at the subject site in 1984. Caltrans files indicate the slope inclinometer was monitored through 1989. The slope inclinometer data indicate that there was movement at a depth approximately 30 ft below the ground surface. This slope inclinometer could not be located during the investigation due to the AC overlays of the subject site. The subject site has been monitored and repaired by Caltrans Maintenance over the years. The repairs consist of crack sealing and AC overlays. The Office of Geotechnical Design – West was requested to investigate the settlement, and pavement cracking in 2009.

Site Observations

Site observations indicated that the distressed roadway section has signs of settlement, and cracking of the pavement in a near circular pattern. The cracking of the pavement starts on the shoulder of westbound Route 84, crosses the right lane and extends half way into the left lane, and continues back to the westbound shoulder. The length of the distressed roadway section is approximately 160 ft.

SITE GEOLOGY AND SEISMICITY

Geology

The geology of the project site consists of marine sedimentary rocks of the middle to lower Eocene Whiskey Hill Formation (Figure 3). Brabb, *et al* (1998), describe the Whiskey Hill Formation as coarse-grained light-gray to buff sandstone, silty claystone, glauconitic sandstone, and tuffaceous siltstone with chaotic interbeds of claystone. Geologic mapping by Brabb *et al* (1998) show the bedrock to be gently folded at the project site with moderate dips (~45°) both northeast and southwest. Borings recovered from the project site indicate the bedrock consists primarily of a soft, moderately fractured grayish brown claystone.

Natural Slope Stability

Extensive construction of highways and roads has obscured natural slopes. Much of the topography seen at the present is man-made. There have been several slide repairs adjacent to the proposed project location. No other slides have been observed within the project limits.

Seismicity

The project lies in the seismically active San Francisco Bay area and is prone to strong ground shaking (Figure 4). Table No. 1 below lists the major faults in the region, their distance from the project site, maximum credible earthquake magnitudes, and peak bedrock accelerations anticipated at the site (Caltrans 2007 Fault Database).
(http://www.dot.ca.gov/hq/esc/earthquake_engineering/SDC_site/).

Table 1 Faults, Maximum Credible Earthquake Magnitudes, and Peak Bedrock Accelerations

FAULT	Distance (mi)*	Maximum Credible Earthquake Magnitude**	Maximum Peak Bedrock Acceleration
San Andreas	2	7.9	0.67 g
Hayward	16.5	7.3	0.23 g
Calaveras	22	7.4	0.17 g

**Closest portion of the fault, measured in miles.*

*** Moment Magnitude.*

FIELD INVESTIGATION AND FINDINGS

Field Exploration and In-Situ Testing

A subsurface investigation was conducted in March 2010, and consisted of two rotary wash borings. The borehole locations are shown on the Log-Of-Test Borings (LOTB). Borehole R-10-101 was drilled to a depth of 65 feet. Borehole R-10-102 was drilled to a depth of 31.5 feet. In-situ Standard Penetration Test (SPT) blow counts were recorded at every 5 ft interval to evaluate the consistency of the on-site soils and soil samples were obtained. Rock core samples were collected in borehole R-10-101. Selected soil and rock samples were then transported to geotechnical laboratory in Sacramento for testing.

A slope inclinometers (SI) was installed in borehole R-10-101, and a piezometer was installed in borehole R-10-102. Slope inclinometer results are located in Appendix A.

Laboratory Testing

Selected soil and rock samples retrieved from the borings were tested to evaluate the properties pertinent to our analysis. The types of laboratory tests performed include the following: Laboratory test results are located in Appendix A.

- Atterberg Limits (AASHTO T 89, AASHTO T 90).
- Moisture Content (AASHTO T 265, ASTM D 2216).
- Corrosion (Soil Resistivity and Soil pH); Content California Test Methods (CTM) 643.
- Unconfined Compression (ASTM D 2166, ASTM D 2938).

The corrosion tests results are summarized in Table No. 6.

Subsurface Soil Conditions

The soil and rock encountered during the subsurface investigation, as interpreted from borings R-10-101 and R-10-102 consists of a 20 to 25 feet thick layer of fill, a 20 to 25 feet layer of alluvium, overlaying sedimentary rock. The fill layer consists of a stiff to very stiff fat clay (CH) with gravel, and very stiff silt with sand (ML). The alluvium layer consists of a very stiff to hard fat clay (CH). The sedimentary rock is a soft, moderately fractured claystone.

Groundwater Conditions

Periodic readings of the groundwater levels, from the piezometer in boring R-10-102, are shown in Table No. 2.

Table 2- Periodic Groundwater Readings

Date	Depth to water level (ft)	Groundwater elevation (ft)
03/25/2010	12:30	172.7
04/13/2010	13.00	172.0
05/12/2010	12.80	172.2
06/09/2010	13.10	171.9
09/03/2010	13.90	171.1
10/01/2010	14.30	170.7
11/02/2010	14.55	170.4
11/23/2010	14.95	170.0
01/06/2011	12.60	172.4

Higher groundwater elevations can be anticipated depending on the amount of precipitation during the rainy season.

SLIDE MATERIAL ENGINEERING PROPERTIES

Soil/rock strength parameters of the sliding mass were determined using back-analysis of the landslide. The size of the sliding mass was estimated using slip-plane location (from SI data), head scarp location, and other geologic features. The soil parameters and pore water pressure parameters determined by back-analysis are summarized in Table No. 3.

Table 3. Back-calculated Soil/Rock Strength and Pore Water Pressure Parameters

Soil/Rock Type	Unit Weight (Pcf)	Internal Friction Angle (degrees)	Cohesion c, (Psf)	Pore water pressure Parameter, r_u	Safety Factor (SF)
Alluvium/ Claystone	130	20	0	0.41	~ 0.95

GEOTECHNICAL RECOMMENDATIONS

<u>Slope-Stability Analysis of the Proposed Repair</u>

<u>Wall Type</u>

The most viable repair strategy for this location is to construct a soldier pile tie-back wall. The retaining wall will be required from Station 101+42.31 to Station 103+32.31 for a length of 190 ft. The soldier beam tie-back wall shall be designed for a slide plane depth of 30 feet below ground surface (bgs), and a permanent retained height of 21 feet, and a temporary height of 23 feet above the finished grade. The lagging will need to be extended at least 2 feet below the finished grade.

Stability Analysis of the Proposed Repair

We performed a slope stability analysis of the proposed repair wall section using a specified slip-plane and the back-calculated soil/rock strength and water pressure parameters indicated in the above. The purpose of the analysis was to calculate the tie-back load required to increase the slide static factor of safety (FS) to 1.3 and seismic FS to 1.1. Our analysis shows that a retaining system that extends to the bottom of the slip-plane will require a two tie-backs rows anchor system. The results of the slope stability analysis are shown on Table No. 4. Horizontal drains will be installed near the base of the wall to reduce the pore water pressure behind the wall.

Table 4. Tie-back loads and safety factors

Soil/Rock Type	Unit Weight (Pcf)	Internal Friction Angle (degrees)	Cohesion c, (Psf)	Pore water pressure Parameter, r_u	Tie-back Load in each row (2 rows) (Kips/ft)	Safety Factor (SF)	Case
Alluvium/ Claystone	130	20	0	0.20	21	~1.3	Static
Alluvium/ Claystone	130	20	0	0.20	29	~1.1	Seismic

A horizontal seismic coefficient of 0.20 was used in the seismic stability analyses.

The piles should be spaced at a maximum spacing of 5 feet on center to create a more reasonable tie-back load for this site, and have full arching between the piles.

We recommend the following requirements/criteria for the proposed soldier beam pile tie-back retaining system design:

Tie-back Anchor

The minimum tie-back anchor loads per row (two rows) are given in Table No. 4, and based on the wall slope stability analysis. The minimum unbonded length is 50 feet for the upper tie-back anchor, and 45 ft for the lower tieback. We rely on passive pressure against the soldier piles as shown below.

We recommend the following requirements/criteria for the anchored soldier pile retaining system design:

Lateral Earth Pressures for Anchor Wall

For active earth pressure against the wall/piles, use the following:

- For anchored wall constructed from the top down and restrained by ground anchors (tieback anchors) the lateral earth pressure acting on the wall height H, may be determined using the above calculated tieback load in accordance with Bridge Design Specifications dated August 2004 (BDS) Article 5.5.5.7.1, Figure 5.5.5.7.1-1.b (appendix B).

For passive earth pressure against the soldier piles, use the following:

- Use passive earth coefficient (Kp) of 3.0. This Kp was computed using engineering properties of Claystone (internal friction angle ϕ =30°, cohesion c = 0 psf, and total unit weight of soil/rock γ = 130 pcf).

For Friction Factor use $\delta = 17°$

For seismic earth pressure against the wall/piles, use a constant pressure of 19 H psf (rectangular pressure diagram), where H is the full wall design height.

For traffic loading use a constant pressure of 80 psf (rectangular pressure diagram) to a maximum depth of 10 ft below the top of the wall. This pressure is equivalent to a 2 ft of surcharge immediately behind the wall.

Vertical Capacity of Soldier Piles

Soldier piles should be embedded a minimum of 15 ft below slip-plane (Figure 5), and the pile spacing should not exceed 5 feet on center. We recommend a minimum total pile length of 50 feet.

The ultimate vertical compression and tension capacities of the piles are summarized in Table No. 5.

Table 5 Pile Friction and Tip Compression Capacities

	Ultimate ksf	Allowable ksf
Unit pile shaft friction per unit surface area of the pile length below the full design wall height.	2.0	1.0 (SF=2)
Pile tip compression bearing pressure per unit tip area of the soldier beam piles.	15	5.0 (SF=3)

Use 60% of the compression shaft resistance values to calculate the ultimate tension (uplift) resistance of the pile.

Drains

We recommend installation of horizontal drains 15 feet on center along the base of the soldier pile tie-back wall. The horizontal drains will be installed 5 degree angle upward for a length of 60 ft. The horizontal drain should consist of 1.5 inch diameter slotted PVC pipe (0.020" slots). The horizontal drains are needed to reduce pore water pressure for the tie-back anchors. If the finished grade in front of the wall is raised by the placement of fill, all horizontal drains outlets must be connected to an 8 inch PVC pipe anchored on wall face and extended outside of the fill with a minimum grade of 2.0%. The pipe outlet shall be discharge into a down drain pipe. Please consult with Central District Hydraulics Branch for details of the outlet requirements.

The as-built plans for the slide repair in 1983 indicate three horizontal drains were installed in 1984. Two of the three horizontal drains are located within the proposed retaining wall limits. There is one drain line from the drainage blanket of the previous slide repaired. A copy of the as-built plan for the horizontal drains and the drainage blanket are in Appendix A. The proposed soldier tie-back wall is located approximately between station 287+75 and station 289+75 on the A Line of the as-built plans for the horizontal drains. Caltrans has measured the outflow from these drains over the years, and copies of the outflows rates are located in Appendix A.

CORROSION

The pH and Resistivity test results indicate that the site is non-corrosive (per Caltrans Corrosion guide dated September 2003), and are shown in Table No. 6. Use Standard corrosion protection measures for this project. Corrosion test results are in Appendix A.

Table 6 Corrosion Test Results

Boring	Depth (ft)	pH	Resistivity ohm-cm
R-10-101	14-15	7.03	1081

CONSTRUCTION CONSIDERATIONS AND REQUIREMENTS

The following construction considerations and requirements should be included in the design and construction specifications for the proposed wall.

- The installation of the soldier beam piles must be completed prior to the placing of any temporary fill required for a construction bench in face of wall. No fill shall be placed on the slide area before all piles are in place.

- The contractor will encounter difficulties during the drilling of the soldier beam piles and tie-back anchor holes due to the high groundwater levels at the proposed site.

- During the drilling for the soldier pile beams, the contractor will encounter a drainage blanket, which consisting of filter fabric and gravel. The drainage blanket was installed as part of the previous slide repair. Two of the three horizontal drains installed from the previous slide repair maybe encountered during the drilling of the soldier beam piles (see as-built plans in Appendix A).

- During the drilling operation for the proposed soldier beam piles, we believe that some caving of the drilled holes will likely occur. Thus, use of temporary casing is required. Due to the groundwater elevations, the installation of soldier piles will likely require dewatering of the borehole before the concrete is placed. The current Caltrans practice for soldier beam pile construction does not allow the use of slurry. Therefore, the use of temporary casing and dewatering is required when groundwater is encountered during construction of the soldier beam pile.

- The maximum load for the testing of the tie-back anchors shall not exceed 1.3 of the design load.

- Installation of the soldier beam piles should be performed in accordance with Section 49-4 of the May 2006 Caltrans Standard Specifications.

- The drilling and concrete placement for soldier beam piles construction shall be staggered. No two adjacent holes can be open at the same time. The drilled hole for the soldier beam piles can't be left open overnight.

- Contact our office immediately, if any drilled hole encounters an existing horizontal drain, so we can evaluate the need for additional measures.

* * * * *

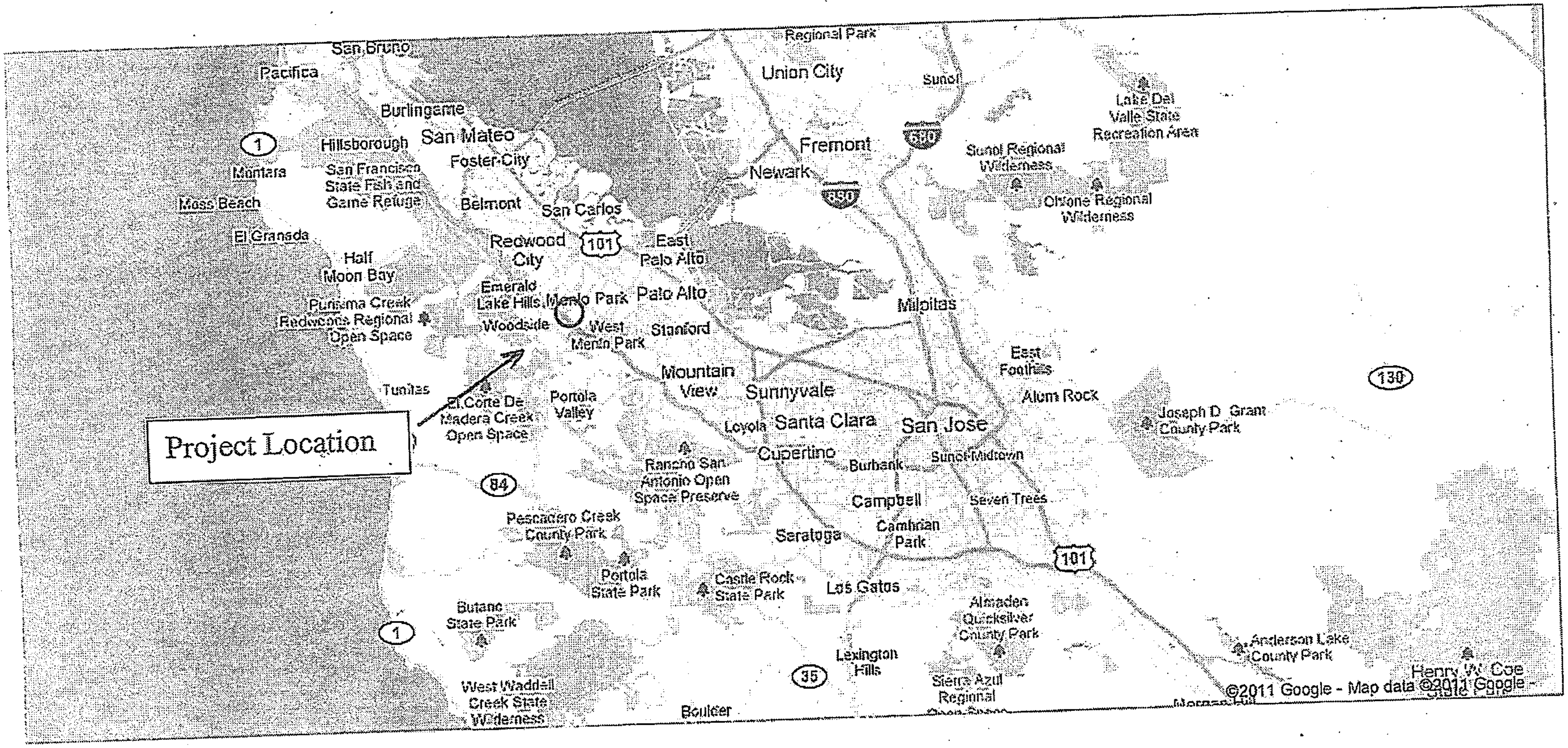
Should you have any questions, please call me at (510) 622-0104 or Mahmood Momenzadeh at (510) 286-5732.

Attachments:

c: TPokrywka, MMomenzadeh, NTachta, PLutz, GWilcox, JStayton, Structures RE, Pending File, MWillian, BKearney, Archive.

DNesbitt/mm

REGISTERED PROFESSIONAL ENGINEER
MAHMOOD MOMENZADEH
No. GE 2685
Exp. 12/31/2011
GEOTECHNICAL
STATE OF CALIFORNIA

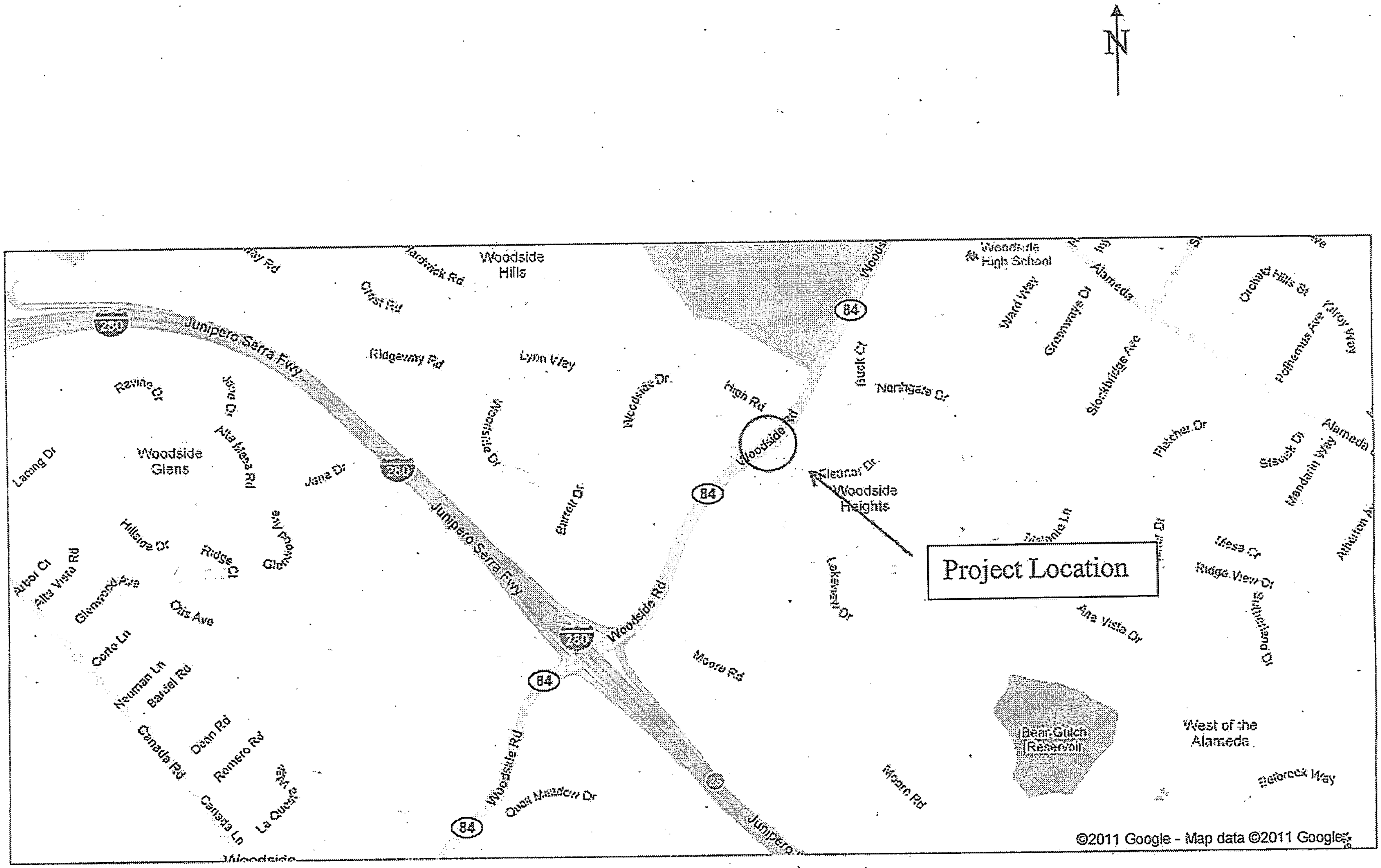


Scale 1" = 5 miles

Figure 1 - Location Map

04-SM 84
0400002051

PM 22.0-22.1
February 2011



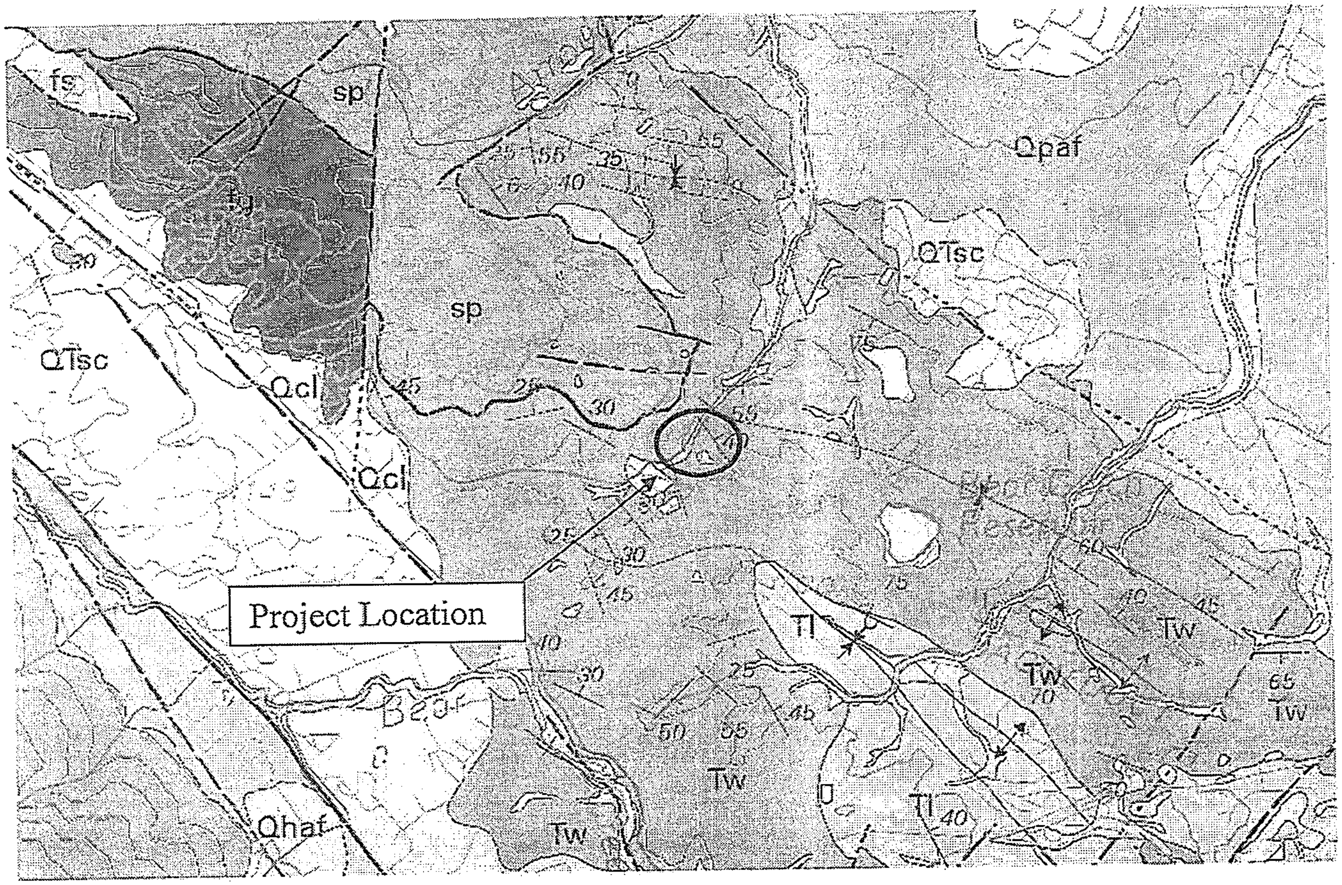
Scale 1" = 1000 feet

Figure 2 - Vicinity Map

04-SM 84
0400002051

PM 22.0-22.1
February 2011

Caltrans



Map Units

af	Artificial fill
alf	Artificial levee fill
Qhaf	Holocene alluvial fans
Qcl	Colluvium
QTsc	Santa Clara Formation
Qpaf	Older alluvial fans
Tl	Ladera Sandstone
Tw	Whiskey Hill Formation
KJf	Franciscan Formation undifferentiated
fg	Greenstone
fs	Sandstone
fc	Chert
	Limestone
sp	Serpentinite

Map Explanation

Contact—Depositional or intrusive contact, dashed where approximately located, dotted where concealed

Fault—Dashed where approximately located, small dashes where inferred, dotted where concealed, queried where location is uncertain.

Reverse or thrust fault—Dotted where concealed

Anticline—Shows fold axis, dotted where concealed

Syncline

Strike and dip of bedding

Overturned bedding

Flat bedding

Vertical bedding

Strike and dip of foliation

Vertical foliation

Strike and dip of joints in plutonic rocks

Vertical joint

Brabb, E.E, Graymer, R.W., and Jones, D.L., Geology of the Onshore Portion of San Mateo County, California, a Digital Database, USGS, 1998, OFR 98-137

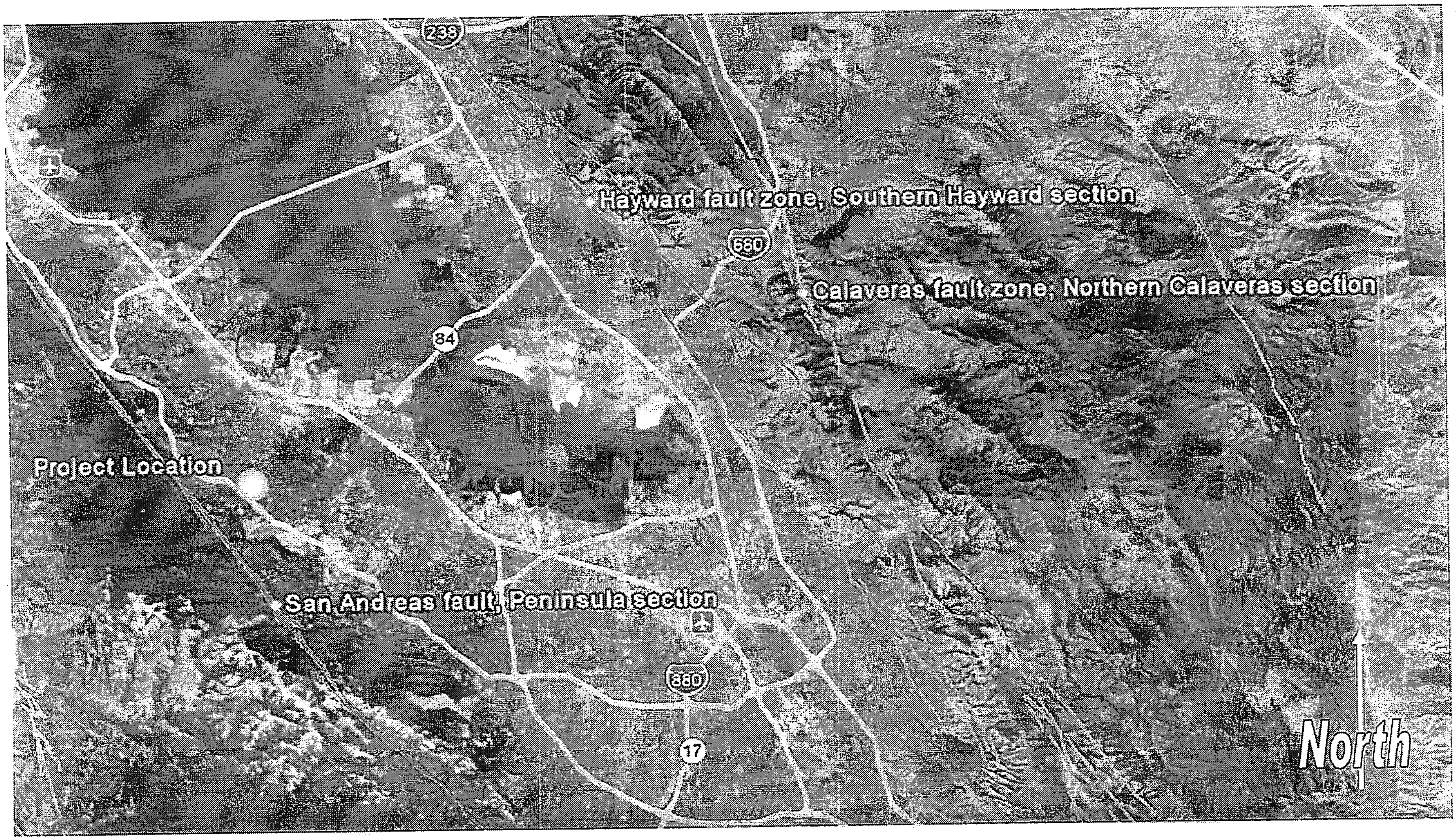
Scale 1:400000

Caltrans

Figure 3 - Geology Map

04-SM 84 **PM 22.0-22.1**

0400002051 **February 2011**



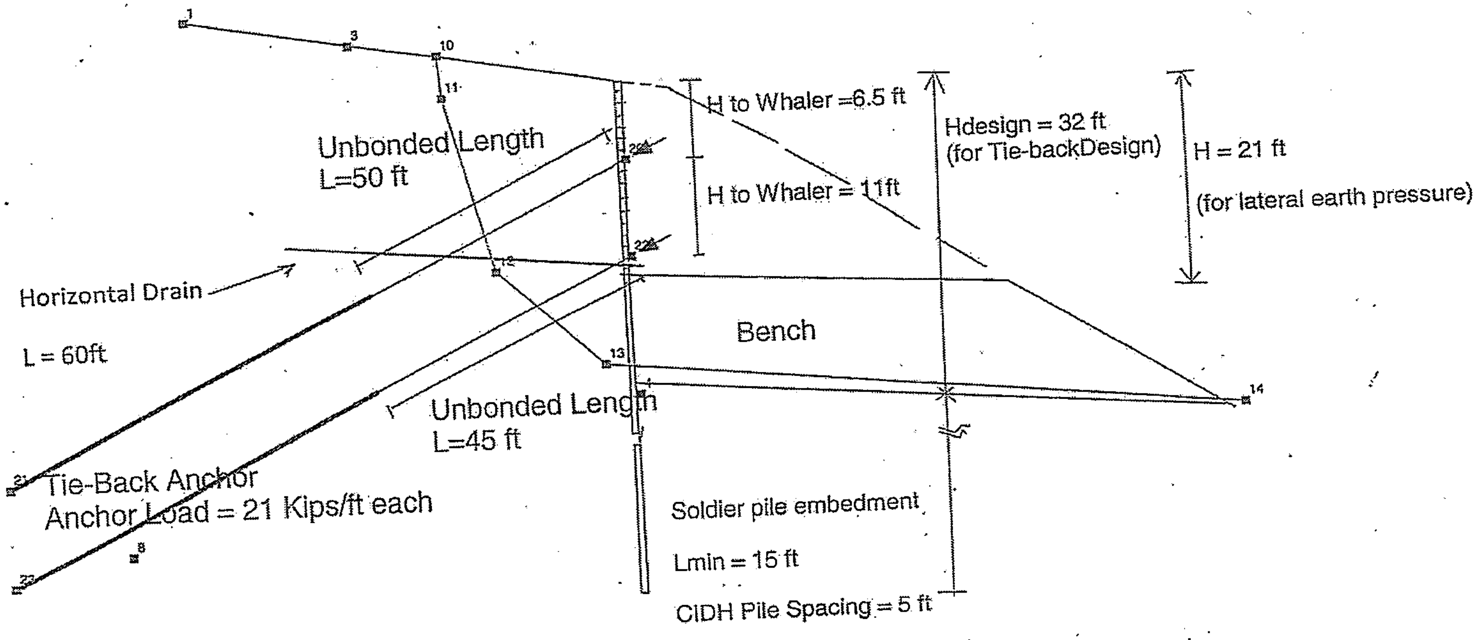
15 miles

U.S. Geological Survey and California Geological Survey, 2006, Quaternary fault and fold database for the United States, 12/01/2008, from USGS web site: http//earthquakes.usgs.gov/regional/qfaults/
Base map from Google Earth 2008

Caltrans

Figure 4 - Regional Fault Map

SM 84	PM 22.0-22.1
0400002051	February 2011



Not to scale

	Figure 5 - Typical Cross Section	
Caltrans	04-SM 84 0400002051	PM 22.0-22.1 February 2011

APPENDIX A

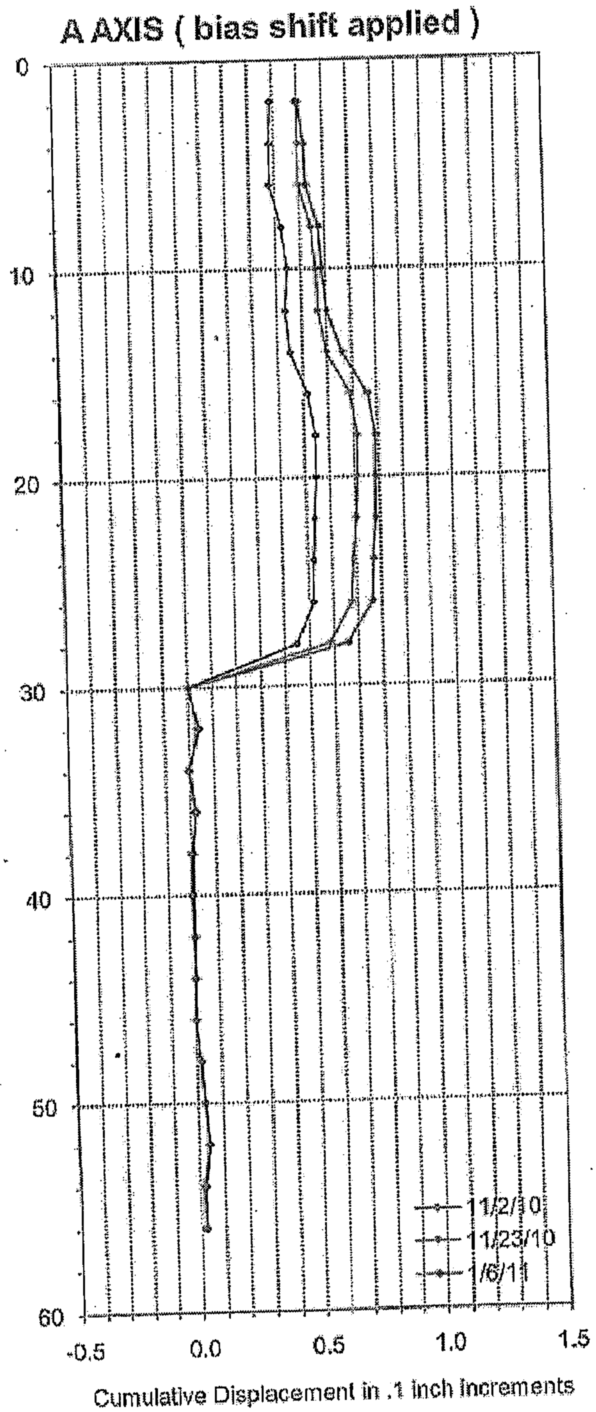
Slope Inclinometer Readings

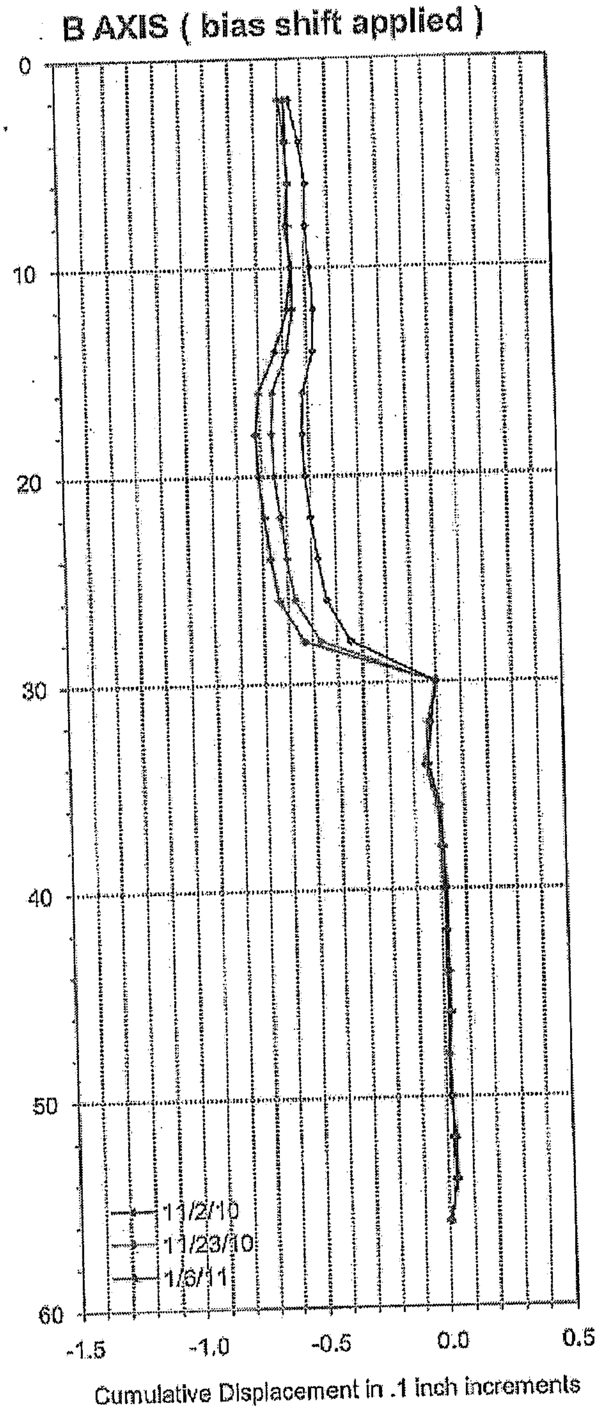
Laboratory Test Results

Out Flow Rates for Horizontal Drains

As-built for Horizontal Drains

Cumulative Displacement (inches) since 3/25/2010





SI-2 (R-10-101)
SOUTHGATE
04-SM-84-PM 22.0
January 7, 2011

CA-Department of Transportation
DES-OGDW Orinda Field Station
15 Camino Pablo (925) 254-6504

CALIFORNIA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL LABORATORY

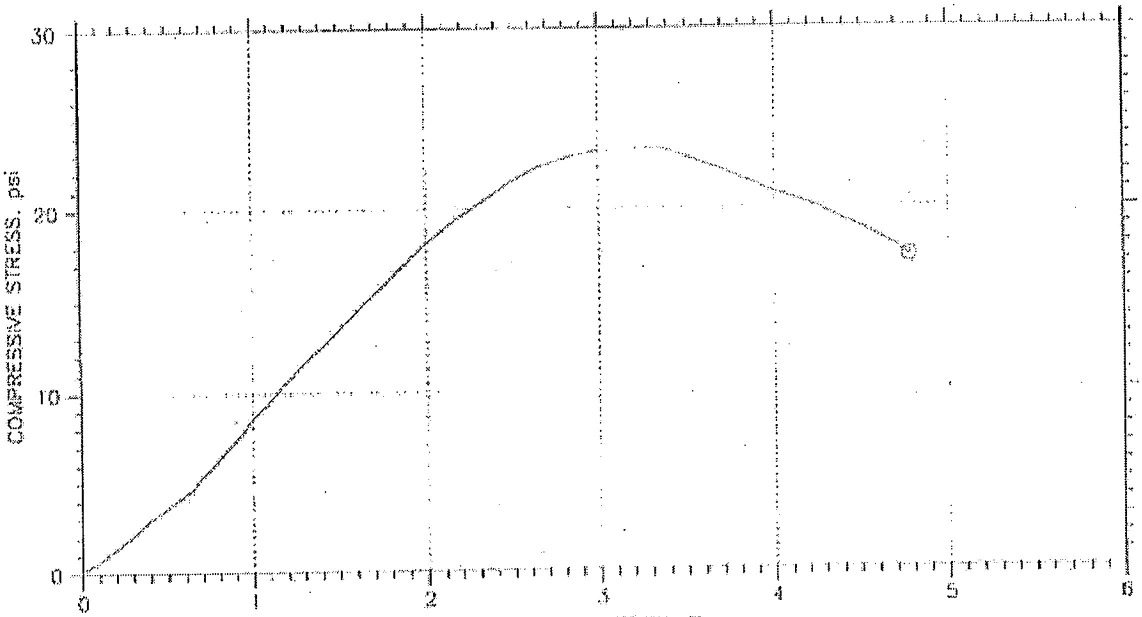
Caltrans

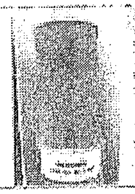
GL TRACKING NO : 10-028
Dist - EA: 04-4S5900
Report Date: July 6, 2010
Page: 1/1

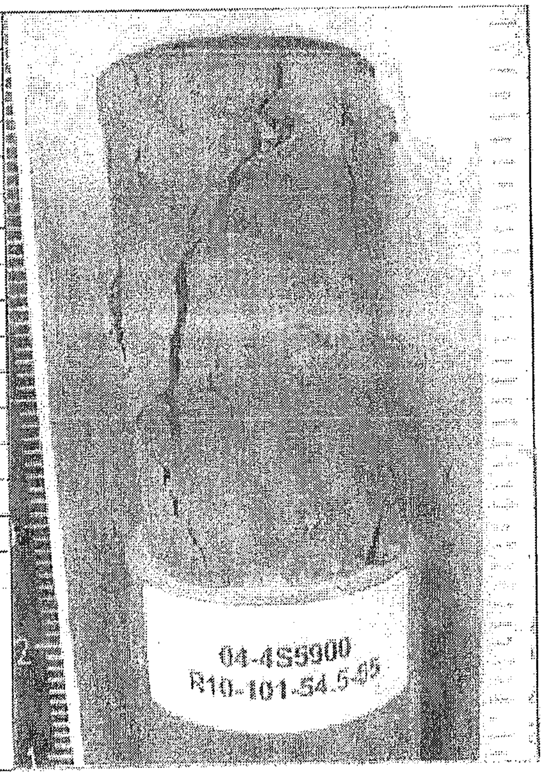
CLASSIFICATION TEST SUMMARY

SAMPLE ID	% FINER THAN																ATTERBERG LIMITS		AS RECEIVED		Gs
	3"	2 1/2"	2"	1 1/2"	1"	3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	5µ	1µ	LL	PI	γd (pcf)	%m	
R-10-101_14-15'																		47	19		24.1	
R-10-101_3																					19.4	
R-10-101_54.5-55'																					19.8	
R-10-101_56-57'																		60	34		27.4	
R-10-101_6																					18.0	
R-10-101_61-62'																		64	38		31.4	
R-10-102_5																						

UNCONFINED COMPRESSION TEST REPORT



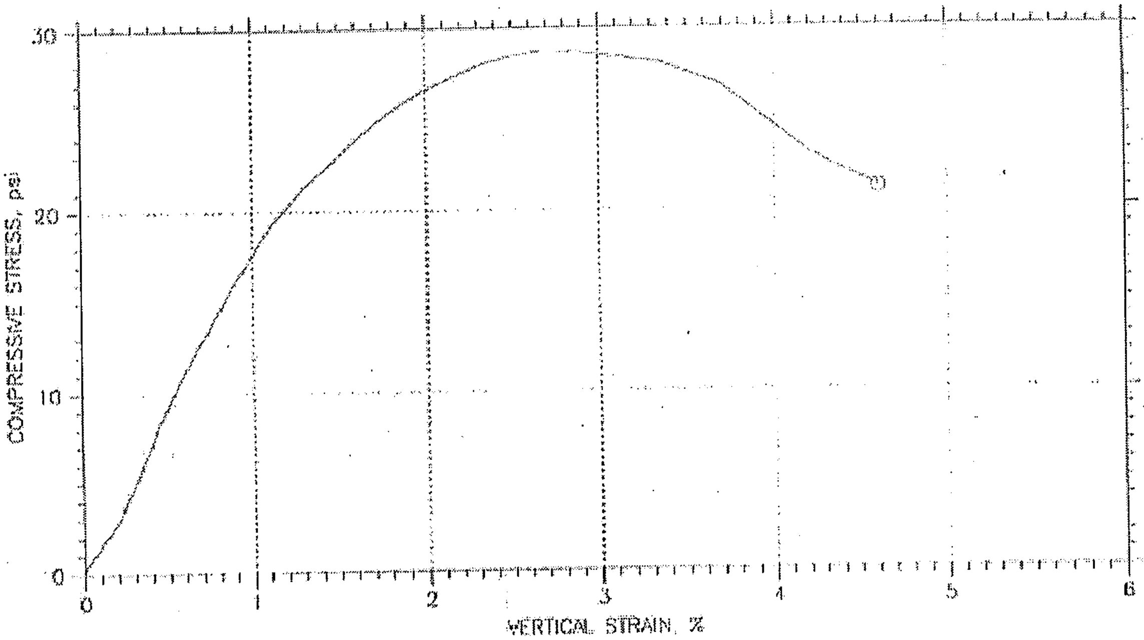
Symbol		⊙
Test No		010-071
Initial	Diameter, in	2.33
	Height, in	4.1
	Water Content, %	19.36
	Dry Density, pcf	106.8
	Saturation, %	
	Void Ratio	
Unconfined Compressive Strength, psi		23.29
Undrained Shear Strength, psi		
Time to Failure, min		
Strain Rate, %/min		1
Implied Specific Gravity		
Liquid Limit		---
Plastic Limit		---
Plasticity Index		---
Failure Sketch		

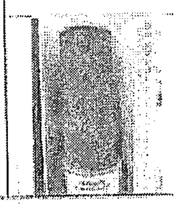


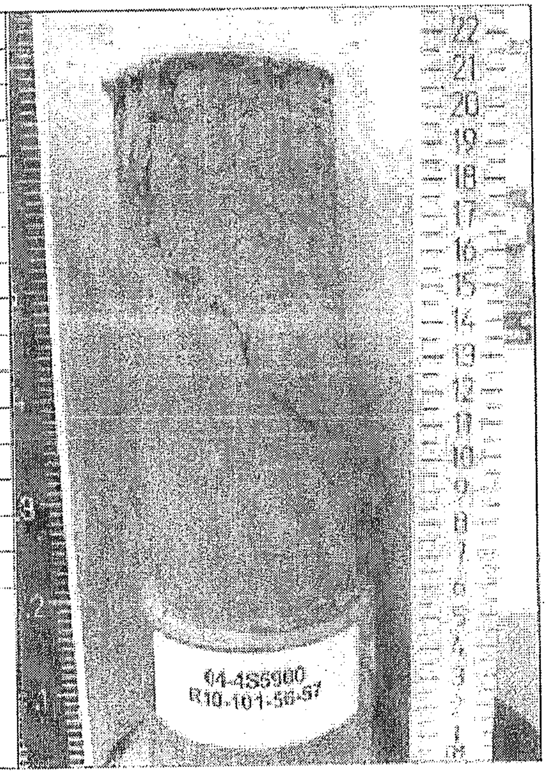
Caltrans	Project: Woodside Road
	Location: 04-SM-84-22-22.1
	Project No.: 04-4S5900
	Boring No.: R10-101
	Sample No. : 54.5-55
	Description: MOIST GRAY SILTY CLAY
	Remarks: ASTM D 2166

Thu, 13-MAY 2010 16:46:30

UNCONFINED COMPRESSION TEST REPORT



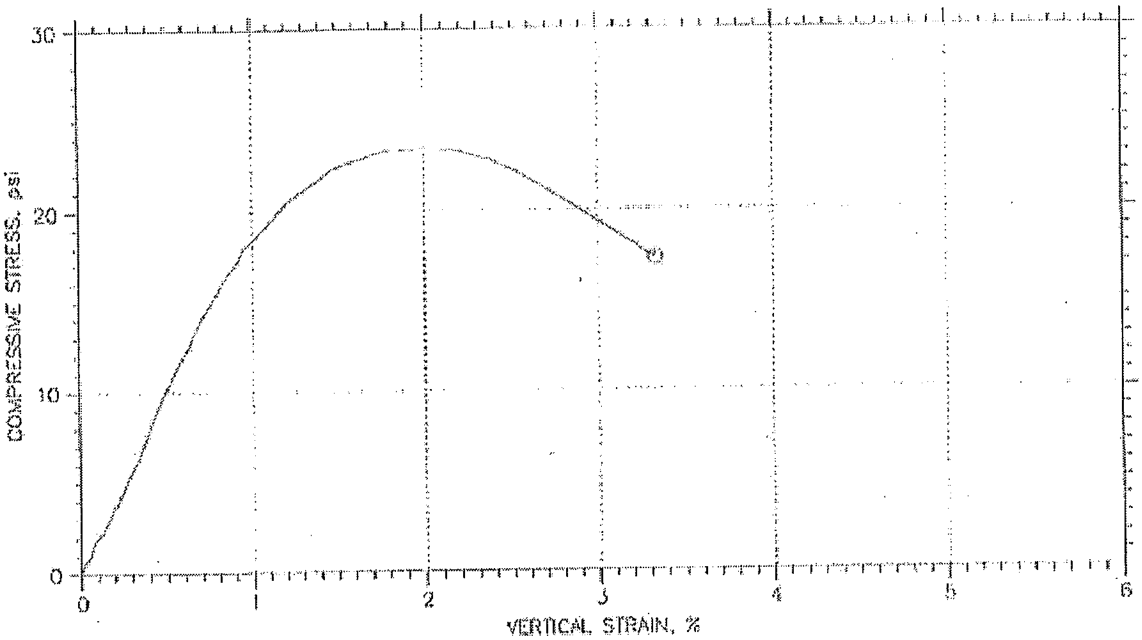
Symbol		⊙
Test No.		Q10-072
Initial	Diameter, in	2.4
	Height, in	5.58
	Water Content, %	19.8
	Dry Density, pcf	110.9
	Saturation, %	
	Void Ratio	
Unconfined Compressive Strength, psi		28.7
Undrained Shear Strength, psi		
Time to Failure, min		
Strain Rate, %/min		1
Implied Specific Gravity		
Liquid Limit		---
Plastic Limit		---
Plasticity Index		---
Failure Sketch		



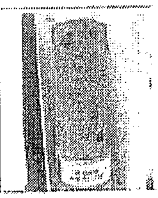
Caltrans	Project: Woodside Road
	Location: 04-SM-84-22-22.1
	Project No.: 04-4S5900
	Boring No.: R10-101
	Sample No. : 56-57
	Description: MOIST GRAY SILTY CLAY
	Remarks: ASTM D 2166

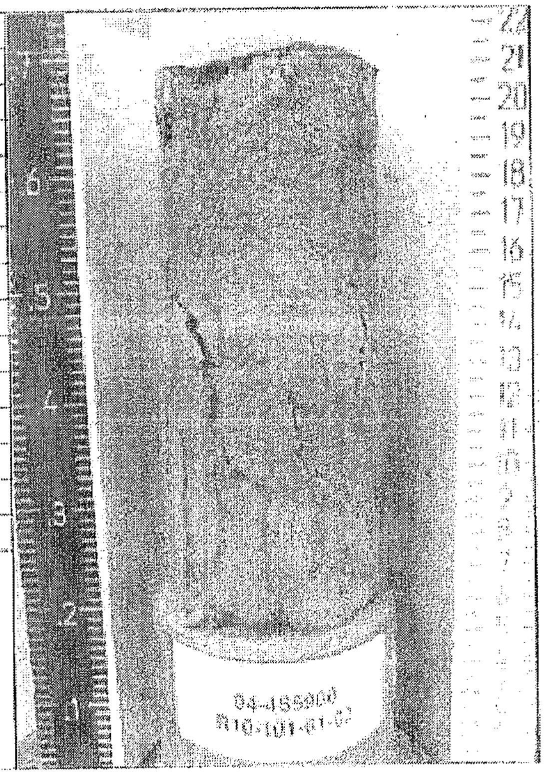
Thu, 13-MAY-2010 16:52:48

UNCONFINED COMPRESSION TEST REPORT



Symbol		⊙
Test No.		Q10-073
Initial	Diameter, in	2.46
	Height, in	5.6
	Water Content, %	18.03
	Dry Density, pcf	108.9
	Saturation, %	
	Void Ratio	
Unconfined Compressive Strength, psi		23.4
Undrained Shear Strength, psi		
Time to Failure, min		
Strain Rate, %/min		1
Implied Specific Gravity		
Liquid Limit		---
Plastic Limit		---
Plasticity Index		---
Failure Sketch		





Caltrans	Project: Woodside Road
	Location: 04-SM-84-22-22.1
	Project No.: 04-4S5900
	Boring No.: R10-101
	Sample No. : 61-62
	Description: MOIST GRAY SILTY CLAY
	Remarks: ASTM D 2166

Thu. 13-MAY-2010 16:59:06

<Rudy_C_Lopez@dot.ca.gov>
04/20/2010 12:37 PM

To <David_Nesbitt@dot.ca.gov>
cc
bcc
Subject Corrosion Test Summary Report - Soil, EA: 04-4S5900 (Corr. # CR100342)

Division of Engineering Services
Materials Engineering and Testing Services
Corrosion Technology Branch
Report Date: 4/20/2010
Reported By: Lopez, Rudy

CORROSION TEST SUMMARY REPORT - Soil/Water

Bridge Name:
Bridge Number:
EA No.: **04-4S5900**
Dist/Co/Rte/PM or KP: **04 / SM / 84 / 22.0/22.1**

SIC Number (TL101)	Sample Location	Sample Type	Sample Depth	Minimum Resistivity[1] (ohm-cm)	pH[2]	Chloride Content[3] (ppm)	Sulfate Content[4] (ppm)
C238661		SOIL	14-15 FT/R10-101	1081	7.03		

This site is not corrosive to foundation elements (see note below for MSE wall backfill).

Note: For MSE wall structure backfill material, minimum resisitivity must be 2000 ohm-cm or greater, pH must be between 5.5 and 10.0, chloride content must not be greater than 250 ppm, and sulfate content must not be greater than 500 ppm.

[1,2]CTM 643, [3]CTM 422, [4]CTM 417

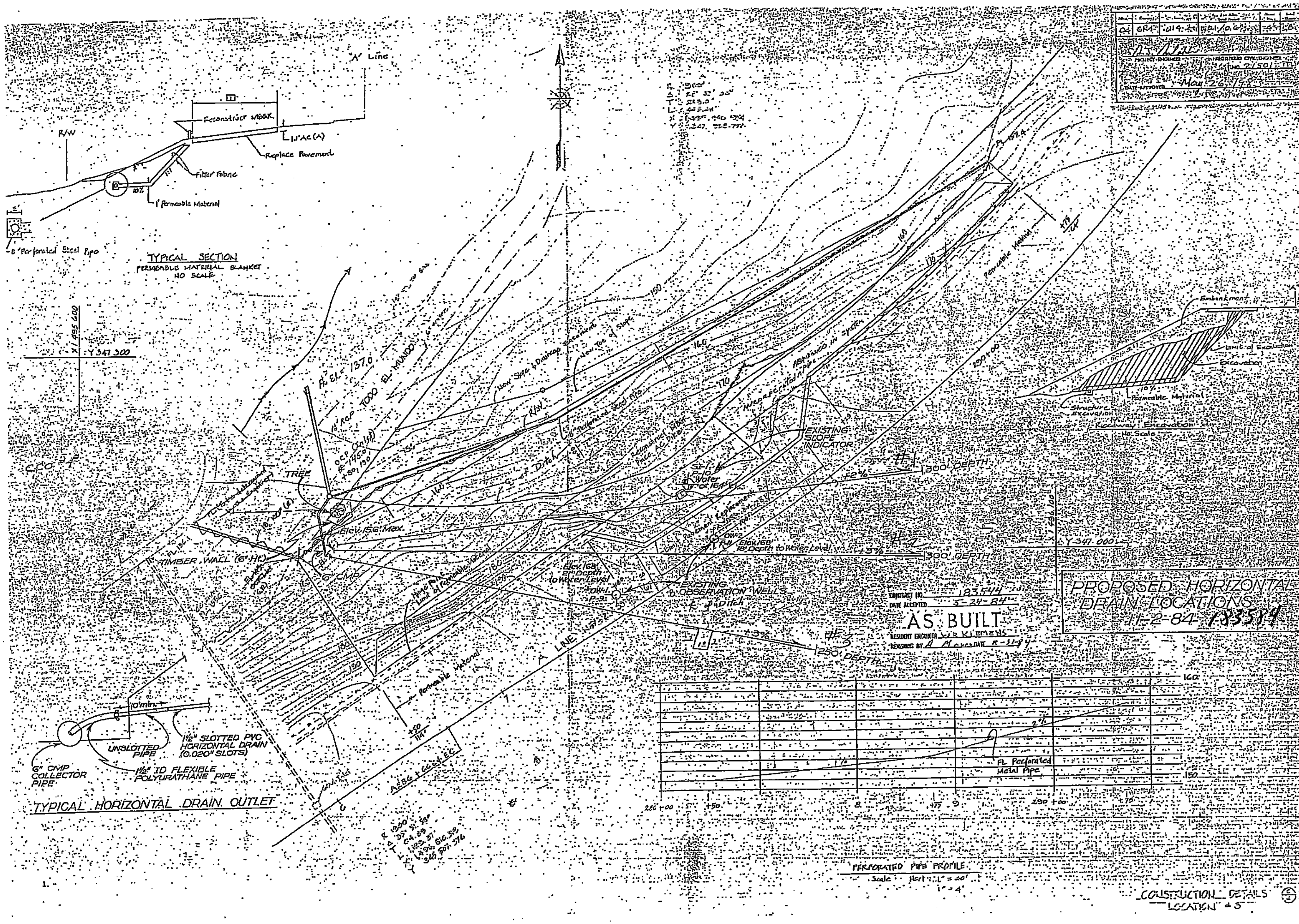
SOUTHGATE 04-SM-114-PM 0.5

State of California
Department of Transportation
DES-OGDW
Orinda Field Station

HORIZONTAL DRAIN DATA

LITERS PER DAY

YEAR	Date of Reading	HD-1	HD-2	HD-3	HD-4
1985	12-17-85	0	91	95	163
1986	1-10-86	0	79	102	185
	2-11-86	0	174	132	405
	4-4-86	0	462	405	2188
	5-22-86	0	167	246	693
	8-15-86	0	64	144	257
	9-10-86	0	144	159	174
	10-17-86	0	57	132	159
	12-5-86	0	76	114	121
1987	2-24-87	0	53	68	151
	6-11-87	0	26	30	76
	8-21-87	0	26	23	19
1989	5-1-89	0	26	19	61
1991	6-10-91	0	11	23	8
1992	4-29-92	68	38	19	0
1993	5-6-93	485	117	79	0
1994	5-4-94	49	26	0	0
1995	4-26-95	719	231	201	0
1997	3-20-97	201	257	102	0
2001	8-1-01	276	148	87	0
2008	5-12-08	0	230	219	763
	8-19-2008	281	81	132	0
	11-3-2008	138	276	68	0
	12-10-2008	271	158	104	0
2009	1-12-2009	415	121	156	0
	2-17-2009	749	104	265	0
	7-27-2009	179	190	181	0
	8-31-2009	151	170	160	0
	11-2-2009	194	219	202	0
2010	1-4-2010	259	324	278	0
	3-25-2010	986	792	645	0
	4-13-2010	1022	871	677	0
	5-12-2010	994	691	720	0
	6-9-2010	893	295	540	0
	9-3-2010	279	112	135	0
	10-1-2010	230	167	109	0
	11-23-2010	252	79	137	0
2011	1-6-211	756	472	432	0



'A' Line
Reconstruct MBGR
R/W
Replace Pavement
Filter Fabric
1' Permeable Material
10%
6" Perforated Steel Pipe
TYPICAL SECTION
PERMEABLE MATERIAL BLANKET
NO SCALE
X 1 495 600
Y 347 300
TREE
TIMBER WALL (6' Ht)
6" CMP
Elev. 156 Max.
EXISTING SLOPE INDICATOR
#1 300' DEPTH
#2 300' DEPTH
#3 250' DEPTH
EXISTING OBSERVATION WELLS
A LINE
Permeable Material
Limit of Excavation
Excavation
Roadway Excavation
No Scale
Y 347 000
PROPOSED HORIZONTAL DRAIN LOCATIONS
11-2-84 183544
CONTRACT NO. 183544
DATE ACCEPTED 5-24-84
AS BUILT
RESIDENT ENGINEER W.R. KLEMENS
1 1/2" SLOTTED PVC HORIZONTAL DRAIN (0.020" SLOTS)
UNSLOTTED PIPE
1 1/2" ID FLEXIBLE POLYURATHANE PIPE
6" CMP COLLECTOR PIPE
10' min.
TYPICAL HORIZONTAL DRAIN OUTLET
FL Perforated Metal Pipe
2%
1%
160
150
PERFORATED PIPE PROFILE
CONSTRUCTION DETAILS
LOCATION #5

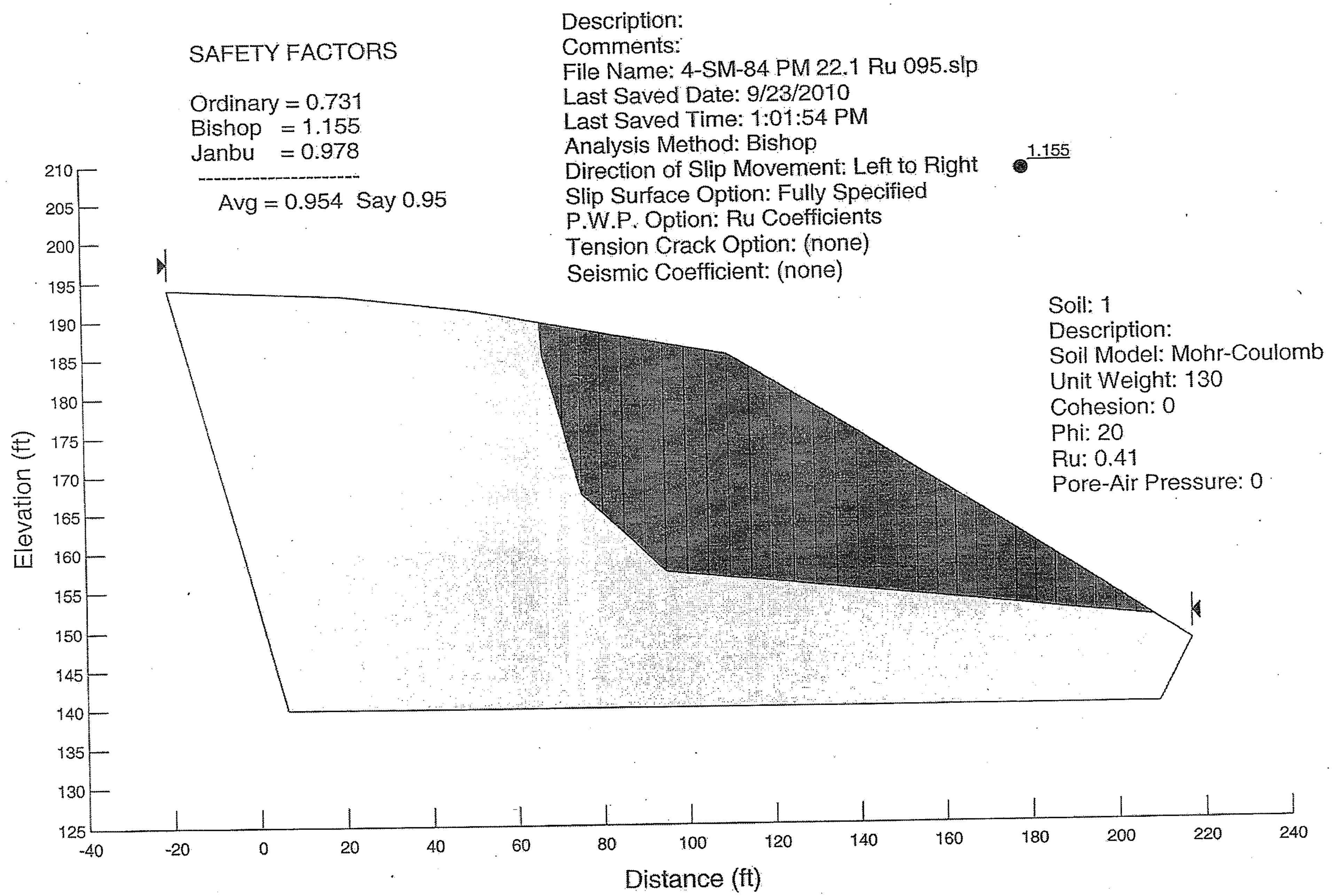
APPENDIX B

Stability Analysis:

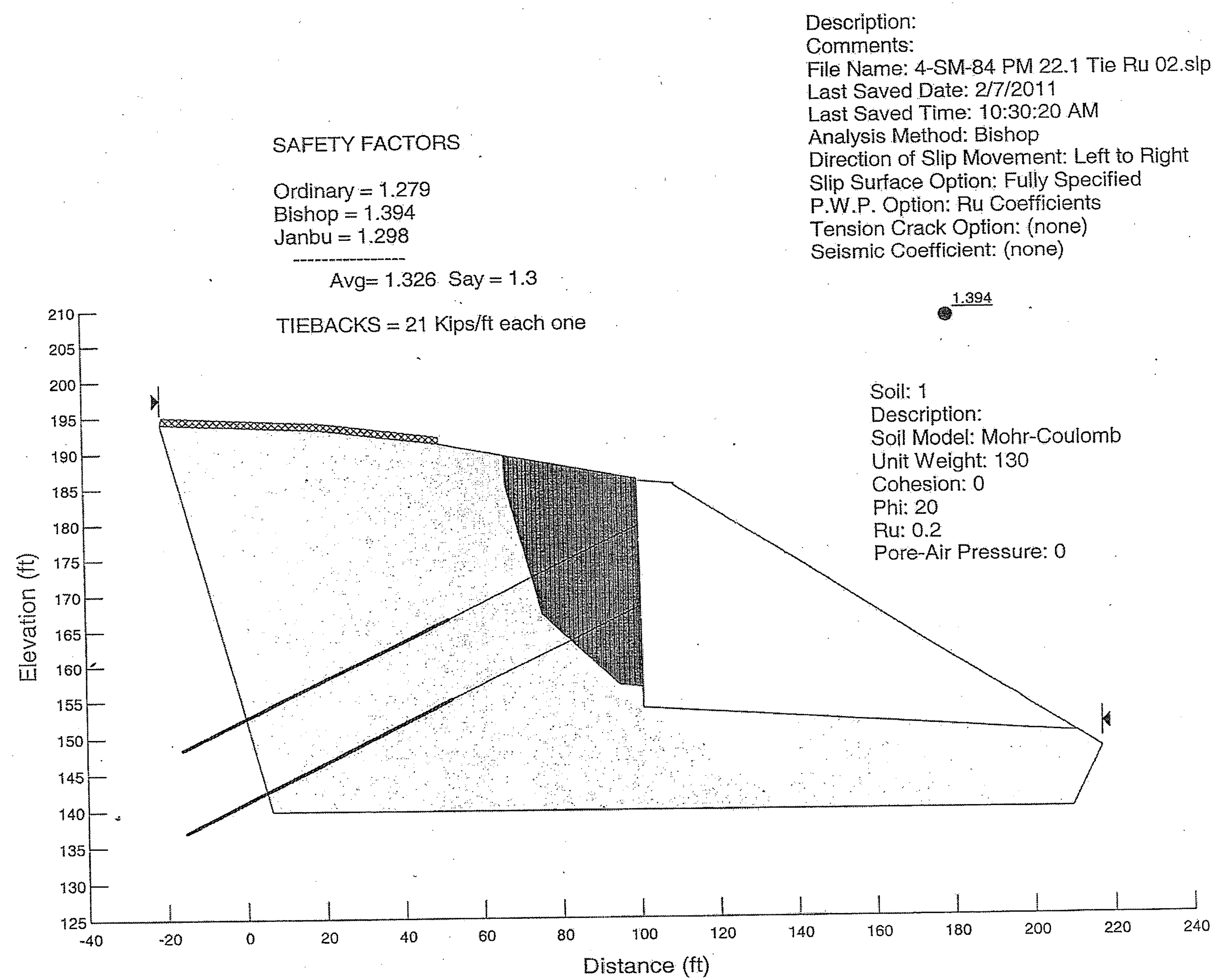
Slope – Back Analysis
Tie-back – Static, FS = 1.3
Tie-back – Seismic, FS = 1.1

Bridge Design Specifications

BDS Section 5, Page 5-33
BDS Section 5, Page 5-34



SAFETY FACTORS
Ordinary = 0.731
Bishop = 1.155
Janbu = 0.978
Avg = 0.954 Say 0.95
Description:
Comments:
File Name: 4-SM-84 PM 22.1 Ru 095.slp
Last Saved Date: 9/23/2010
Last Saved Time: 1:01:54 PM
Analysis Method: Bishop
Direction of Slip Movement: Left to Right
Slip Surface Option: Fully Specified
P.W.P. Option: Ru Coefficients
Tension Crack Option: (none)
Seismic Coefficient: (none)
1.155
Soil: 1
Description:
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 20
Ru: 0.41
Pore-Air Pressure: 0
Elevation (ft)
210
205
200
195
190
185
180
175
170
165
160
155
150
145
140
135
130
125
-40
-20
0
20
40
60
80
100
120
140
160
180
200
220
240
Distance (ft)


Description:
Comments:
File Name: 4-SM-84 PM 22.1 Tie Ru 02.slp
Last Saved Date: 2/7/2011
Last Saved Time: 10:30:20 AM
Analysis Method: Bishop
Direction of Slip Movement: Left to Right
Slip Surface Option: Fully Specified
P.W.P. Option: Ru Coefficients
Tension Crack Option: (none)
Seismic Coefficient: (none)
SAFETY FACTORS
Ordinary = 1.279
Bishop = 1.394
Janbu = 1.298
Avg= 1.326 Say = 1.3
TIEBACKS = 21 Kips/ft each one
1.394
Soil: 1
Description:
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 20
Ru: 0.2
Pore-Air Pressure: 0
Elevation (ft)
210
205
200
195
190
185
180
175
170
165
160
155
150
145
140
135
130
125
-40
-20
0
20
40
60
80
100
120
140
160
180
200
220
240
Distance (ft)

SAFETY FACTORS

Ordinary = 1.148
Bishop = 1.249
Janbu = 1.048

Avg = 1.148 Say = 1.1

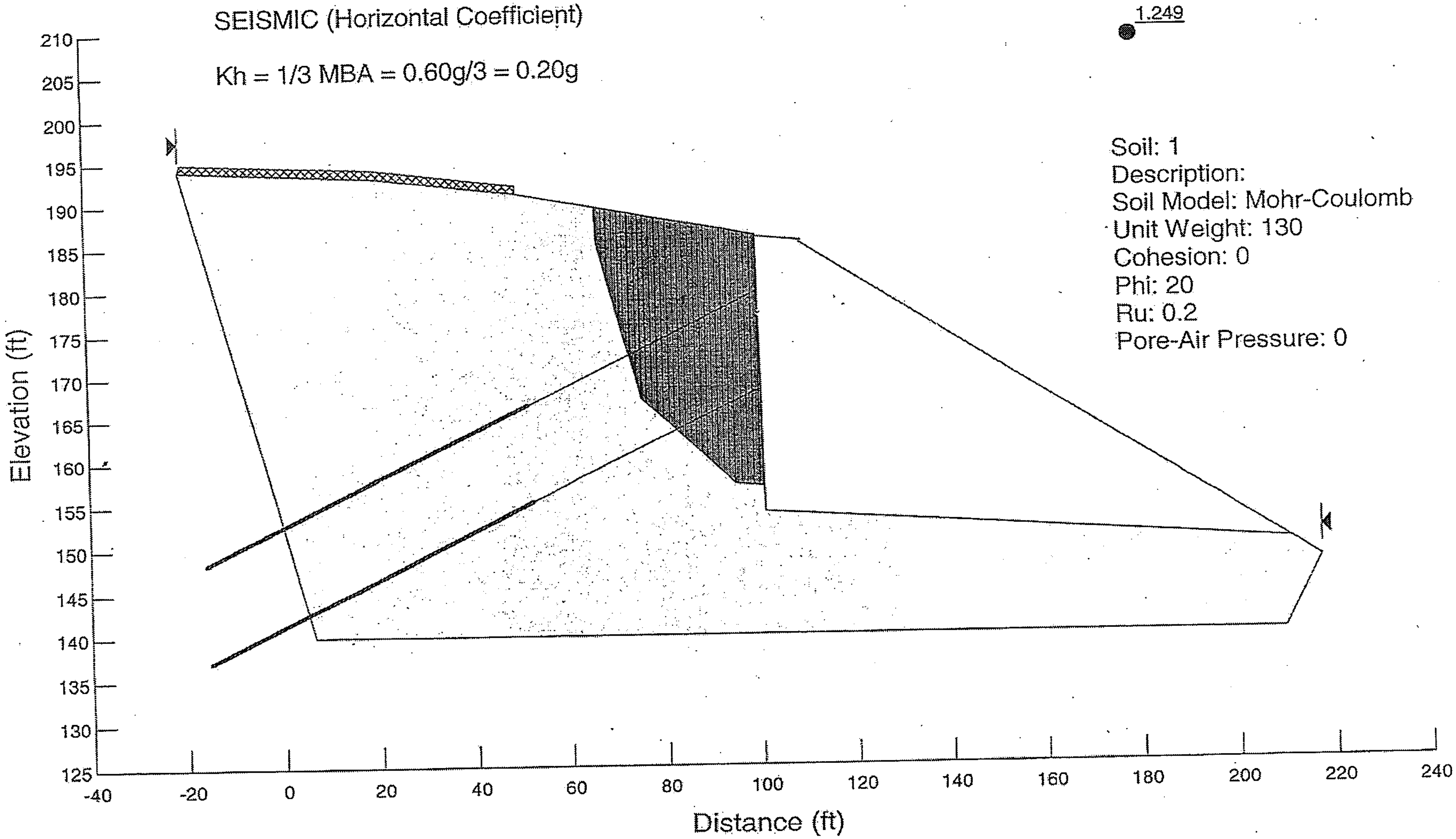
TIEBACKS (Seismic)
Two tiebacks of 29 Kips/ft each

SEISMIC (Horizontal Coefficient)

Kh = 1/3 MBA = 0.60g/3 = 0.20g

Description:
Comments:
File Name: 4-SM-84 PM 22.1 Seis Tie Ru 02.slp
Last Saved Date: 2/7/2011
Last Saved Time: 11:02:18 AM
Analysis Method: Bishop
Direction of Slip Movement: Left to Right
Slip Surface Option: Fully Specified
P.W.P. Option: Ru Coefficients
Tension Crack Option: (none)
Seismic Coefficient: Horizontal

Soil: 1
Description:
Soil Model: Mohr-Coulomb
Unit Weight: 130
Cohesion: 0
Phi: 20
Ru: 0.2
Pore-Air Pressure: 0



5.5.5.7 Lateral Earth Pressures for Anchored Walls

For anchored walls restrained by tie rods and structural anchors, the lateral earth pressure acting on the wall may be determined in accordance with Article 5.5.5.6.

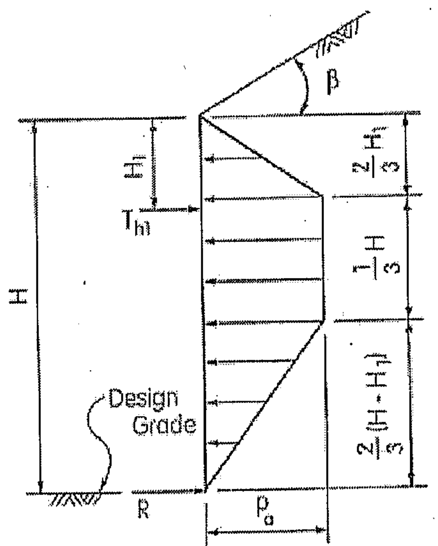
For anchored walls constructed from the top down and restrained by ground anchors (tieback anchors), the lateral earth pressure acting on the wall height, H, may be determined in accordance with Articles 5.5.5.7.1 and 5.5.5.7.2.

For anchored walls constructed from the bottom up and restrained by a single level of ground anchors located not more than one third of the wall height, H, above the bottom of the wall, the total lateral earth pressure, P_{Total}, acting on the wall height, H, may be determined in accordance with Article 5.5.5.7.1 with distribution assumed to be linearly proportional to depth and a maximum pressure equal to, $\frac{2P_{Total}}{H}$. For anchored walls constructed from the bottom up and restrained by multiple levels of ground anchors, the lateral earth pressure acting on the wall height, H, may be determined in accordance with Article 5.5.5.7.1.

In developing the lateral earth pressure for design of an anchored wall, consideration shall be given to wall displacements that may affect adjacent structures and/or underground utilities.

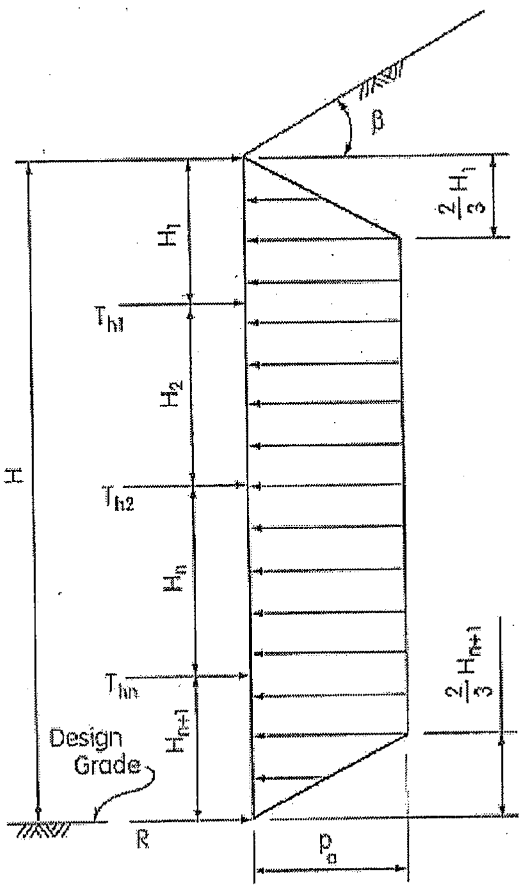
C5.5.5.7

In the development of lateral earth pressures, the method and sequence of wall construction, the rigidity of the wall/anchor system, the physical characteristics and stability of the ground mass to be supported/retained, allowable wall deflections, anchor spacing and prestress and the potential for anchor yield should be considered.



Note: $H_1 \leq \frac{2}{3}H$

a) Wall with a single level of anchors



b) Wall with multiple levels of anchors

Figure 5.5.5.7.1-1 Lateral Earth Pressure Distributions for Anchored Walls Constructed from the Top Down in Cohesionless Soils

5.5.5.7.1 Cohesionless Soils

The lateral earth pressure distribution for the design of temporary or permanent anchored walls constructed in cohesionless soils may be determined using Figure 5.5.5.7.1-1, for which the maximum ordinate, p_a, of the pressure diagram is determined as follows:

For walls with a single level of anchors :

$$p_a = \frac{P_{Total}}{\frac{2}{3}H} \qquad (5.5.5.7.1\text{-}1)$$

For walls with multiple levels of anchors:

$$p_a = \frac{P_{Total}}{\left(H - \frac{1}{3}H_1 - \frac{1}{3}H_{n+1}\right)} \qquad (5.5.5.7.1\text{-}2)$$

where:

p_a = maximum ordinate of pressure diagram (KSF)

P_{Total} = total lateral load required to be applied to the wall face to provide a factor of safety equal to 1.3 for the retained soil mass when stability is analyzed using an appropriate limiting equilibrium method of analysis. Except that P_{Total} shall not be less than $1.44\,P_a$. (KIP)

P_a = active lateral earth pressure resultant acting on the wall height, H, and determined using Coulomb's theory with a wall friction angle, δ, equal to zero. (KIP)

H = wall design height (FT)

H_1 = distance from ground surface at top of wall to uppermost level of anchors. (FT)

H_{n+1} = distance from design grade at bottom of a wall to lowermost level of anchors (FT)

T_{hi} = horizontal component of design force in anchor at level i (KIP/FT)

R = design reaction force at design grade at bottom of wall to be resisted by embedded portion of wall (KIP/FT)

5.5.5.7.2 Cohesive Soils

The lateral earth pressure distribution for cohesive soils is related to the stability number, N_S, which is defined as:

$$N_s = \frac{\gamma_s H}{S_u}$$

where:

γ_s = total unit weight of soil (KCF)

H = wall design height (FT)

S_u = average undrained shear strength of soil (KSF)

5.5.5.7.2a Stiff to Hard

For temporary anchored walls in stiff to hard cohesive soils ($N_s \leq 4$), and β = zero, the lateral earth pressure may be determined using Figure 5.5.5.7.1-1, with the maximum ordinate, p_a, of the pressure diagram determined as:

$$p_a = 0.2\gamma_s H \text{ to } 0.4\gamma_s H \qquad (5.5.5.7.2a-1)$$

where:

p_a = maximum ordinate of pressure diagram (KSF)

γ_s = total unit weight of soil (KCF)

H = wall design height (FT)

For permanent anchored walls in stiff to hard cohesive soils, the lateral earth pressure distributions described in Article 5.5.5.7.1 may be used with, P_{Total}, based on the drained friction angle of the cohesive soil. For permanent walls, the distribution (permanent or temporary) resulting in the maximum total force shall be used for design.

FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: **Structure Design**

1. Design
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services

1. GD - North ; South ; West
2. GS File Room

Date: 7/11/11

Woodside Tieback Wall
Structure Name

04 - SM - 84 - 22.1
District County Route ~~km~~ Post mi.

District Project Development
District Project Engineer

0400002051
04-455901
E.A. Number

35E0035
Structure Number

Foundation Report By: D. Nesbitt Dated: 3/15/11

Reviewed By: P. Lutz (SD) (GS)

General Plan Dated: 7/5/11 Foundation Plan Dated: 7/7/11

☑ ~~No~~ changes. ☐ The following changes are necessary.

FOUNDATION CHECKLIST

✓ Pile Types and Design Loads	___ Footing Elevations, Design Loads, and Locations	✓ Effect of Fills on Abutments and Bents
✓ Pile Lengths	✓ Seismic Data	✓ Fill Surcharge
✓ Predrilling	✓ Location of Adjacent Structures and Utilities	✓ Approach Paving Slabs
✓ Pile Load Test	✓ Stability of Cuts or Fills	✓ Scour
✓ Substitution of H Piles For Concrete Piles ☐ Yes ☑ No	✓ Fill Time Delay	✓ Ground Water
		✓ Tremie Seals/Type D Excavation

Philip L. Lutz 9
Structure Design Bridge Design Branch No.

Geotechnical Services

Rev. 06/02