URS

FINAL GEOTECHNICAL ANALYSIS REPORT LAGUNITAS ROAD BRIDGE PROJECT ROSS, CALIFORNIA

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1.0 INTRODUCTION

The Lagunitas Road Bridge Project geotechnical analysis report was performed for the San Francisco District, US Army Corps of Engineers (COE) under Indefinite Delivery Contract (IDC) DACW07-00-D-0003. The notice-to-proceed with this phase of the project was received from Ben C. Gerwick, Inc. on March 5, 2002.

The project team for the geotechnical analysis consisted of Sverdrup/Gerwick JV (prime consultant) and URS Corporation, Inc. (geotechnical subconsultant). URS Corporation, Inc., will be referred to as "URS" in this report.

The purpose of the geotechnical analysis report is to achieve the following objectives:

- Provide assessment for (1) Seismic criteria, (2) Liquefaction potential, (3) Abutment stability, (4) Foundation stiffness, (5) Earth pressures, and (6) Sliding resistance.

- Evaluate three different scenarios: (1) Existing conditions, (2) Lowered streambed, and (3) Lowered streambed with box culvert in west abutment.

This report is organized into sections that describe the project, subsurface conditions, traffic loads and seismic criteria, liquefaction analysis, abutment stability, foundation stiffness, earth pressures, summary of conclusions, and presents our recommendations. Figures are used to illustrate the features of the existing bridge structure, show exploration locations, to present the results of our slope stability analyses, and to show earth pressure diagrams. Tables are used to summarize data from our liquefaction potential analyses, abutment stability analyses, and foundation stiffness analyses.

2.0 PROJECT DESCRIPTION

The Lagunitas Road Bridge is located about 200 feet west of Sir Francis Drake Boulevard in the Town of Ross, California as shown in Figure 1, Site Vicinity Map. The project site is in Marin County. Data research performed by Sverdrup/Gerwick JV has been unable to find any as-built documents/drawings for the Lagunitas Road Bridge. Information provided by the COE in their Request for Proposal (RFP) dated August 29, 2001 indicates that the bridge was constructed in 1908 by the Town of Ross and is identified as an architecturally significant structure.

The Lagunitas Road Bridge consists of concrete girder type construction that is supported on two piers. The bridge is skewed to the Corte Madera Creek channel as shown on Figure 2. The

bridge is about 26 feet wide and about 87 feet long. The existing streambed elevation at the time of the geotechnical investigation in October of 2001 (URS, 2002) was approximately +10.5 feet National Geodetic Vertical Datum (NGVD).

The project purpose for the COE is to provide flood protection for residential, commercial, and public property along Corte Madera Creek. The two alternatives considered in the vicinity of the Lagunitas Road Bridge are for the completion of the Corte Madera Creek Flood Control Project that started many years ago. The alternatives include (1) excavating the streambed down to elevation +4.0 feet (NGVD), or (2) using a combination of channel excavation, and adding a bypass box culvert located to the west of the bridge. It is our understanding that the box culvert will be approximately 19 feet wide, 10 feet high, with a wall thickness of 1.5 feet, and will be located a minimum of 10 feet to the west of the west bridge headwall.

3.0 SUBSURFACE CONDITIONS

A total of 25 auger sounding holes and 2 geotechnical borings were drilled during the geotechnical investigation that URS conducted in October 2001 at the Lagunitas Road Bridge site. The field exploration and subsurface conditions are discussed in detail in the URS report entitled "Geotechnical Data Report, Lagunitas Road Bridge Project, Ross, California", and dated January 30, 2002 (URS, 2002).

The geotechnical soil properties that were used in our analyses are outlined below.

3.1 WEST BRIDGE ABUTMENT

Below the upper 2.5 feet of asphalt and aggregate base material, the subsoils consist of about 32.5 feet of Sandy Lean Clay (CL) with a very stiff to medium stiff consistency, underlain by about 8 feet of Poorly-Graded Sandy Gravel with Silt (down to elevation –13.0 feet NGVD). During the course of the soils investigation, a 5-foot thick Sandy Clay (CL) strata was encountered in boring B-2 at a depth of approximately 30 feet (-5.0 feet NGVD). The clay strata at about 30 feet may be localized, therefore we did not consider its impact in our stability analyses. Below the poorly-graded sandy gravel, bedrock consisting of shale and sandstone (greywacke) is encountered. The clay layer was divided into 3 units due to its variability in shear strength. The groundwater table was assumed to be at elevation +12 feet (NGVD). The soil parameters used in our analyses are presented in Table 3.1.

TABLE 3-1 SOIL PROPERTIES USED IN ANALYSES FOR ABUTMENT AREAS			
Soil Type	Elevation (feet, NGVD)	Total Unit Weight (pcf)	Soil Strength Parameters
Pavement (AC/AB)	+25.0 to +22.5	140	-
Sandy Lean Clay (CL)	+22.5 to +12.0	125	Su = 2,000 psf $\phi = 0°$
	+12.0 to +7.5	125	Su = 1,500 psf $\phi = 0°$
	+7.5 to –5.0	125	Su = 700 psf $\phi = 0°$
Poorly-Graded Sandy Gravel with Silt (GP-GM)	-5.0 to –13.0	130	Su = 0 $\phi = 38°$
Bedrock (shale and greywacke)	Below –13.0	-	-

3.2 STREAMBED

Due to the heterogeneity of the soil profile, we considered in our analyses that the streambed subsoils consist predominantly of medium stiff Sandy Lean Clay (CL), and Well-Graded Sandy Gravel (GW) that extend to elevation of –5.0 feet (NGVD), underlain by Poorly-Graded Sandy Gravel with Silt (GP-GM) down to elevation –13.0 feet (NGVD). Table 3.2 summarizes the soil properties that were used in our analyses.

TABLE 3-2 SOIL PROPERTIES USED IN ANALYSES FOR STREAMBED AREA			
Soil Type	Elevation (NGVD)	Total Unit Weight (pcf)	Soil Strength Parameters
Sandy Lean Clay (CL)	+10.5 to –5.0	125	Su = 700 psf $\phi = 0°$
Well-Graded Sandy Gravel (GW)		140	Su = 0 $\phi = 32°$
Poorly-Graded Sandy Gravel with Silt (GP-GM)	-5.0 to –13.0	130	Su = 0 $\phi = 38°$
Bedrock (shale and greywacke)	Below –13.0	-	-

4.0 TRAFFIC LOAD AND SEISMIC CRITERIA

4.1 TRAFFIC

We understand that a vehicular surcharge load exists on the Lagunitas Road Bridge. The expected highway load for this type of area is classified as H-20 (AASHTO 2001), which corresponds to a surcharge of 480 psf.

4.2 SEISMIC

The bridge is located about 7.5 miles northeast of the active San Andreas Fault and 11 miles southwest of the active Hayward Fault (Walter et al., 1998). Therefore, the bridge will be exposed to strong shaking during future earthquakes. The maximum credible earthquakes (MCE) that can be produced by the Hayward Fault and San Andreas Fault are 7.5 and 8.0 in magnitude, respectively. According to a seismic hazard map published by Caltrans California (1996), the peak ground acceleration on rock based on MCE at the site location is 0.4g. Using correlations from Idriss (1991), estimated acceleration for this soft soil site should range from approximately 0.32 to 0.48g for a peak rock acceleration of 0.4g. For our analyses, we used values of 0.3 and 0.4g for the peak ground acceleration. SHAKE analyses were not performed to estimate the surface accelerations.

5.0 LIQUEFACTION ANALYSES

Liquefaction is the phenomenon in which earthquake ground vibrations induce a decrease in volume, a buildup of pore water pressure, and a resulting loss of strength in saturated, loose to medium dense, cohesionless soils. Liquefaction is manifested in the formation of sand boils and ground deformations, which include: (1) settlements due to volume change of the liquefied deposit; and (2) ground deformation or soil failure due to the greatly reduced strength of the liquefied layer. Ground deformations observed at liquefaction sites during previous earthquakes have included: (1) cracking due to dynamic displacements of the ground surface with respect to adjacent non-liquefied areas during ground shaking; (2) uplift of materials that are less dense than the adjacent ground due to loss of soil frictional support and increased buoyant forces; and (3) for sites on or adjacent to slopes, extensive lateral movement in response to gravitational forces, which is referred to as lateral spread.

Principal effects of liquefaction consist of (1) vertical strains due to settlement, or (2) transient or permanent lateral strains due to dynamic shaking. Because of the sandy gravel that exists in the streambed under the Lagunitas Road Bridge, evaluation of liquefaction resistance of the soils was performed.

5.1 ANALYSES METHODOLOGY

Liquefaction susceptibility using Standard Penetration Test data was analyzed using the method summarized by Youd and Idriss (1997). Liquefaction analyses performed for this investigation included: (1) evaluation of soil consistency influencing liquefaction; (2) correction of penetration resistance data; (3) evaluation of cyclic stress ratio for each penetration test depth and for representative ground shaking; and (4) evaluation of liquefaction potential based on the values computed above cyclic resistance ratio. Only penetration tests performed in the sandy gravel material at the project site were considered for the analyses for liquefaction potential.

5.1.1 Influence of Soil Consistency

Discussion of the soil consistency encountered in our exploration program is included in Section 3 of this report and in a previous URS report, “Geotechnical Data Report, Lagunitas Road Bridge Project, Ross, California,” Job. No. 43-00067008.00, dated January 30, 2002.

5.1.2 Correction of Penetration Resistance Data

Uncorrected SPT and U-sampler values for the sandy soils below the streambed (URS 2002) were modified and corrected to account for the following:

- Sampler type
- Overburden pressure
- Hammer energy
- Borehole diameter
- Rod length
- Sampling method
- Fines content and soil plasticity

These correction factors were implemented in accordance with the guidelines presented by Youd and Idriss (1997).

5.1.3 Evaluation of Cyclic Stress Ratio for Representative Ground Shaking

Liquefaction potential was analyzed for an earthquake scenario based on peak ground acceleration (PGA) levels presented in Section 4. These levels of ground shaking are assumed to be the result of Magnitude 7.5 and 8.0 earthquakes on the adjacent segment of the Hayward and San Andreas Faults, respectively.

Cyclic stress ratios (CSR) were computed for the earthquake corresponding to the in-situ stress conditions at the location and depth of each penetration tests. The CSR were computed for peak horizontal accelerations at the ground surface of 0.3g and 0.4g. The CSR refers to an estimate of the average drive horizontal stresses of the soils during an earthquake divided by the effective stress controlling soil strength at that depth.

5.1.4 Evaluation of Cyclic Resistance Ratio

The cyclic resistance ratio (CRR) refers to the capacity of soil to resist liquefaction. These ratios were computed by using a simplified base curve presented in Youd and Idriss (1997) that is dependent on the appropriate corrected SPT blow count for a magnitude 7.5 earthquake.

5.1.5 Evaluation of Liquefaction Potential

Determination of whether the soil is liquefiable or not is based on the computed factors of safety. Factors of safety are determined by dividing the CRR by the CSR and multiplying by a

magnitude scaling factor (Youd and Idriss, 1997). A factor of safety (FS) less than 1.0 indicates that the soil will liquefy under the given earthquake magnitude and ground acceleration. Soils with a FS between 1.0 and 1.1 are considered marginally safe. A FS greater than 1.1 indicates a soil that is not liquefiable.

5.2 RESULTS

Results of the analyses indicate in all scenarios, the upper 14 feet of soils in the streambed will liquefy. For an earthquake of magnitude 7.5 (Hayward fault), a strata of soil at depth 14 to 16 feet exists that was determined to be safe from liquefaction when subjected to a PGA of 0.3g. Soil located at 20 feet below the streambed surface was calculated to be liquefiable, and soil below 35 feet in the west abutment was determined to be marginally safe. For a PGA of 0.4g, there is a strata at depth 14 to 16 feet that was determined to be marginally safe from liquefaction. All other depths were determined to be liquefiable at this PGA.

For an earthquake of magnitude 8.0 (San Andreas fault), a strata of soil at depth 14 to 16 feet exists that was determined to be safe from liquefaction when subjected to a PGA of 0.3g. All other depths were calculated to be liquefiable. For a PGA of 0.4g, all soil layers analyzed were determined to be liquefiable.

Table 5.1 presents the results of our analyses. The calculation spreadsheets of our analyses are located in Appendix A.

TABLE 5-1

LIQUEFACTION POTENTIAL RESULTS

Depth below streambed (ft)	Maximum Credible Earthquake / Peak Ground Acceleration (g)			
	7.5	7.5	8.0	8.0
	0.3	0.4	0.3	0.4
4 – 14	Will Liquefy	Will Liquefy	Will Liquefy	Will Liquefy
14 – 16	Safe	Marginally Safe	Safe	Will Liquefy
20 – 22	Will Liquefy	Will Liquefy	Will Liquefy	Will Liquefy
35 – 36 (west abutment location)	Marginally Safe	Will Liquefy	Will Liquefy	Will Liquefy

6.0 ABUTMENT STABILITY ANALYSES

This section summarizes the results of static and pseudo-static stability analyses for the west abutment area of the Lagunitas Road Bridge. Static stability was evaluated to determine if a sufficient factor of safety (FS) against a stability failure exists for the option of excavating the streambed to a lower elevation. Evaluation of pseudo-static stability was performed to assess the effect of earthquake forces. The cross-section used for the analyses was based on a simplified version of the subsurface profile presented in URS (2002).

6.1 ANALYSES METHODOLOGY

The computer program SLOPE/W (GEO-SLOPE International Ltd.) was used to perform both static and pseudo-static stability analyses of the project area. SLOPE/W allows the consideration of multiple soil layers with varying properties. Spencer's method was used for the analyses, which enforces compatibility of both moment equilibrium and force equilibrium and provides a higher level of reliability than methods that compute only moment or force equilibrium. Critical failure surfaces, associated with minimum factors of safety (ratio of resisting forces to driving forces), were identified for both static and pseudo-static seismic stability analyses. Factors of safety less than 1.0 indicate potential instability while factors of safety in excess of 1.0 indicate stability. A static factor of safety of 1.5 or greater is desirable, while a pseudo-static factor greater than 1.0 is acceptable for short-term conditions (earthquake loads).

Soil strengths and unit weight parameters used for both static and pseudo-static evaluations are discussed in Section 3 of this report. No reduction in soil strength was included in the pseudo-static analyses as a result of earthquake shaking. Tension cracks were used in the upper clay layer due to its high cohesion. The depth of the tension cracks extended completely through the upper clay layer.

6.2 STATIC STABILITY ANALYSES RESULTS

Figures 3 and 4 present the critical stability surfaces for the analyzed cross-section. The figures show the location of the critical stability surface and the locus (point) of rotation for that circle. Once the failure surface reaches the upper clay layer, it extends vertically to the ground surface due to the presence of tension cracks (i.e. absence of soil shear strength). Contours of minimums factor of safety are shown for the loci of less critical stability circle evaluations. The computed static factors of safety for the cross-sections are presented in Table 6.1. The results show that excavations performed in the streambed will create an unstable condition.

TABLE 6-1 RESULTS OF STATIC STABILITY ANALYSES	
Elevation of Streambed (feet NGVD)	Minimum Factor of Safety
+11 (current condition)	1.5
+6.5	1.1

6.3 PSEUDO-STATIC STABILITY ANALYSES RESULTS

Seismic stability of the abutment was evaluated using a pseudo-static approach. In this method, earthquake forces are represented by an equivalent static horizontal force. This force is a product of a horizontal acceleration and the mass of the potential slide material. For the project site, the closest known fault is the San Andreas fault, which is capable of producing a maximum horizontal ground acceleration of 0.4g. Our pseudo-static analyses used a seismic coefficient (k_h) of 0.2 (i.e., one-half of the peak horizontal acceleration) to determine the resulting minimum factors.

Figures 5 and 6 present the critical stability surfaces for the analyzed cross-section. The same conditions as implemented in the static analyses were applied. The computed pseudo-static factors of safety for both conditions are presented in Table 6.2. The resulting factor of safety against failure is marginal (FS = 1.0) for the current streambed condition. The pseudo-static factor of safety is not sufficient (FS < 1.0) to ensure stability of the abutment if the streambed elevation is lowered.

TABLE 6-2 RESULTS OF PSEUDO-STATIC STABILITY ANALYSES (SEISMIC COEFFICIENT = 0.2g)	
Elevation of Streambed (feet NGVD)	Minimum Factor of Safety
+11 (current condition)	1.0
+6.5	0.9

7.0 FOUNDATION STIFFNESS

For the analysis of the existing bridge structure, Ben C. Gerwick, Inc. requested that URS develop the static and dynamic soil stiffnesses for the bridge piers and headwalls. The soil

spring constants were evaluated based on the subsurface information presented in URS (2002). Due to the variable shear strengths of the subsoils, our analyses focused on determining a range of appropriate soil parameters in determining the spring constants values.

Our analyses are based on the following assumptions:

- The exiting bridge headwalls are embedded 9 feet into the ground (i.e., base elevation of the headwalls is +1.5 ft NGVD).
- Soil conditions for both piers, and both headwalls, are similar to each other.
- Soil springs are compression-type elements that will not provide any resistance when subjected to tensile forces.
- Maximum bridge headwall lateral displacement under seismic loading will not exceed 1 inch.

7.1 VERTICAL STIFFNESS K_Z

To evaluate the soil-foundation interaction, we estimated the maximum low strain soil shear modulus, G_{max}, from (1) Standard Penetration Test (SPT) blowcounts from borings B-1 and B-2 for the bridge piers (URS, 2002), and (2) Consolidation Test results from boring B-2 for the bridge headwall.

As mentioned earlier, blowcounts obtained using the Dames & Moore U-Sampler and the Modified California Sampler were converted to equivalent SPT blowcounts.

For the bridge piers case, two approaches were adopted to evaluate G_{max}.

In the first approach, the shear wave velocity V_s can be evaluated from either laboratory tests, or in-situ tests such as the Standard Penetration Test. Various correlations between shear wave velocities and SPT blowcounts [Ohta and Goto (1978), Imai and Tonouchi (1982), Sykora and Stokoe (1983)] were used to evaluate V_s.

The low-strain shear modulus can be calculated from the shear wave velocity as follows:

$$Gmax = \rho \, . \, Vs^2$$

where ρ is the fill density, in units of (Mass / Length^3).

A second approach consisted of using Imai and Tonouchi (1982) to evaluate the low strain shear modulus G_{max} directly from SPT blowcounts.

Both approaches have given relatively comparable results. Due to the heterogeneous nature of the soil and its variable density, we concentrated in our analyses in determining a range of G_{max} values.

For the bridge headwalls case, the low strain shear modulus G_{max} was evaluated using the Hardin (1978) approach and the consolidation test results from boring B-2 (URS, 2002).

Based on the low strain shear modulii, empirical correlations from Fang (1991) were used to evaluate the vertical spring constants at the base of the bridge piers and headwalls.

Table 7-1 presents a summary of the stiffnesses at the base of the bridge piers and the headwalls.

TABLE 7-1

SUMMARY OF VERTICAL STIFFNESSES K_Z (lbs/inch)

	Bridge Pier	Bridge Headwall
Existing Streambed Condition (Elevation +10.5 feet NGVD)	1.9×10^7 to 2.9×10^7	1.8×10^7
Streambed Lowered by 4 feet (Elevation +6.5 feet NGVD)	1.7×10^7 to 2.7×10^7	1.5×10^7

7.2 LATERAL STIFFNESS K_H

The lateral load/deformation relationship of the bridge piers can be computed by use of the p-y data presented in Tables 7-2 and 7-3. The p-y values were developed along the height of the bridge piers. The computer program LPILE (Ensoft, 1997) was used to analyze the lateral load behavior of the bridge piers under static and dynamic conditions, and for the existing and dredged streambed elevations +10.5 feet and +6.5 feet NGVD, respectively. As illustrated in Figure 2, the longitudinal direction is perpendicular to the alignment of the bridge piers, whereas the transverse direction is aligned with the bridge piers.

TABLE 7-2-A

SUMMARY OF BRIDGE PIER P-Y DATA

EXISTING CONDITIONS (STREAMBED ELEVATION = +10.5 Feet NGVD)

LONGITUDINAL DIRECTION

STATIC CASE

2.5 ft depth		5.0 ft depth		7.5 ft depth		10.0 ft depth	
P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)
0	0	0	0	0	0	0	0
285	0.475	570	0.475	855	0.475	1140	0.475
570	0.950	1140	0.950	1710	0.950	2280	0.950
855	1.42	1710	1.42	2560	1.42	3420	1.42
1140	1.90	2280	1.90	3420	1.90	4560	1.90
1430	2.38	2850	2.38	4280	2.38	5700	2.38
1710	2.85	3420	2.85	5130	2.85	6840	2.85
2000	3.33	3990	3.33	5990	3.33	7980	3.33
2280	3.80	4560	3.80	6840	3.80	9120	3.80
2440	4.28	5060	4.28	7700	4.28	10300	4.28
2510	4.75	5200	4.75	8040	4.75	11000	4.75
2580	5.22	5330	5.22	8250	5.22	11300	5.22
2630	5.70	5460	5.70	8440	5.70	11600	5.70
3500	12.8	7270	12.8	11300	12.8	15600	12.8

TABLE 7-2-B

SUMMARY OF BRIDGE PIER P-Y DATA

EXISTING CONDITIONS (STREAMBED ELEVATION = +10.5 Feet NGVD)

LONGITUDINAL DIRECTION

DYNAMIC CASE (M_W = 8.0)

2.5 ft depth		5.0 ft depth		7.5 ft depth		10.0 ft depth	
P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)
0	0	0	0	0	0	0	0
285	0.475	570	0.475	855	0.475	1140	0.475
408	0.950	953	0.950	1650	0.950	2280	0.950
463	1.42	1070	1.42	1840	1.42	2800	1.42
506	1.90	1170	1.90	1990	1.90	3010	1.90
543	2.38	1240	2.38	2120	2.38	3190	2.38
574	2.85	1310	2.85	2230	2.85	3340	2.85
602	3.33	1370	3.33	2320	3.33	3480	3.33
628	3.80	1430	3.80	2410	3.80	3600	3.80
651	4.28	1470	4.28	2490	4.28	3710	4.28
673	4.75	1520	4.75	2560	4.75	3810	4.75
693	5.22	1560	5.22	2630	5.22	3900	5.22
712	5.70	1600	5.70	2690	5.70	3990	5.70
987	12.8	2190	12.8	3610	12.8	5270	12.8

TABLE 7-2-C

SUMMARY OF BRIDGE PIER P-Y DATA

EXISTING CONDITIONS (STREAMBED ELEVATION = +10.5 Feet NGVD)

TRANSVERSE DIRECTION

STATIC CASE

2.5 ft depth		5.0 ft depth		7.5 ft depth		10.0 ft depth	
P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)
0	0	0	0	0	0	0	0
20	0.033	40	0.033	60	0.033	80	0.033
40	0.067	80	0.067	120	0.067	160	0.067
60	0.100	120	0.100	180	0.100	240	0.100
80	0.133	160	0.133	240	0.133	320	0.133
100	0.167	200	0.167	300	0.167	400	0.167
120	0.200	240	0.200	360	0.200	480	0.200
140	0.233	280	0.233	420	0.233	560	0.233
160	0.267	320	0.267	455	0.267	632	0.267
180	0.300	360	0.300	483	0.300	679	0.300
200	0.333	394	0.333	509	0.333	724	0.333
220	0.367	407	0.367	535	0.367	767	0.367
229	0.400	419	0.400	559	0.400	809	0.400
316	0.900	599	0.900	914	0.900	1420	0.900

TABLE 7-2-D

SUMMARY OF BRIDGE PIER P-Y DATA

EXISTING CONDITIONS (STREAMBED ELEVATION = +10.5 Feet NGVD)

TRANSVERSE DIRECTION

DYNAMIC CASE (M_W = 8.0)

2.5 ft depth		5.0 ft depth		7.5 ft depth		10.0 ft depth	
P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)
0	0	0	0	0	0	0	0
20	0.033	40	0.033	60	0.033	80	0.033
40	0.067	80	0.067	120	0.067	160	0.067
60	0.100	120	0.100	180	0.100	240	0.100
80	0.133	160	0.133	240	0.133	320	0.133
100	0.167	200	0.167	300	0.167	400	0.167
118	0.200	240	0.200	360	0.200	480	0.200
122	0.233	280	0.233	420	0.233	560	0.233
126	0.267	320	0.267	478	0.267	640	0.267
129	0.300	352	0.300	503	0.300	720	0.300
132	0.333	361	0.333	527	0.333	800	0.333
135	0.367	368	0.367	550	0.367	853	0.367
138	0.400	375	0.400	571	0.400	889	0.400
178	0.900	478	0.900	884	0.900	1420	0.900

TABLE 7-3-A

SUMMARY OF BRIDGE PIER P-Y DATA

DREDGED CONDITIONS (STREAMBED ELEVATION = +6.5 Feet NGVD)

LONGITUDINAL DIRECTION

STATIC CASE

2.0 ft depth		4.0 ft depth		6.0 ft depth	
P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)
0	0	0	0	0	0
228	0.475	456	0.475	684	0.475
456	0.950	912	0.950	1370	0.950
684	1.42	1370	1.42	2050	1.42
912	1.90	1820	1.90	2740	1.90
1140	2.38	2280	2.38	3420	2.38
1370	2.85	2740	2.85	4100	2.85
1600	3.33	3190	3.33	4790	3.33
1820	3.80	3650	3.80	5470	3.80
1940	4.28	3990	4.28	6140	4.28
1990	4.75	4110	4.75	6320	4.75
2040	5.22	4210	5.22	6480	5.22
2090	5.70	4310	5.70	6630	5.70
2770	12.8	5730	12.8	8860	12.8

TABLE 7-3-B SUMMARY OF BRIDGE PIER P-Y DATA DREDGED CONDITIONS (STREAMBED ELEVATION = +6.5 Feet NGVD) LONGITUDINAL DIRECTION DYNAMIC CASE (M_W = 8.0)					
2.0 ft depth		4.0 ft depth		6.0 ft depth	
P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)
0	0	0	0	0	0
228	0.475	456	0.475	684	0.475
316	0.950	718	0.950	1210	0.950
359	1.42	810	1.42	1360	1.42
393	1.90	882	1.90	1480	1.90
422	2.38	943	2.38	1570	2.38
447	2.85	996	2.85	1660	2.85
469	3.33	1040	3.33	1730	3.33
489	3.80	1080	3.80	1800	3.80
507	4.28	1120	4.28	1860	4.28
525	4.75	1160	4.75	1910	4.75
540	5.22	1190	5.22	1970	5.22
555	5.70	1220	5.70	2010	5.70
774	12.8	1680	12.8	2730	12.8

TABLE 7-3-C

SUMMARY OF BRIDGE PIER P-Y DATA

DREDGED CONDITIONS (STREAMBED ELEVATION = +6.5 Feet NGVD)

TRANSVERSE DIRECTION

STATIC CASE

2.0 ft depth		4.0 ft depth		6.0 ft depth	
P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)
0	0	0	0	0	0
16	0.033	32	0.033	48	0.033
32	0.067	64	0.067	96	0.067
48	0.100	96	0.100	144	0.100
64	0.133	128	0.133	192	0.133
80	0.167	160	0.167	240	0.167
96	0.200	192	0.200	288	0.200
112	0.233	224	0.233	336	0.233
128	0.267	256	0.267	384	0.267
144	0.300	288	0.300	420	0.300
160	0.333	320	0.333	437	0.333
176	0.367	345	0.367	454	0.367
181	0.400	355	0.400	470	0.400
249	0.900	501	0.900	702	0.900

TABLE 7-3-D SUMMARY OF BRIDGE PIER P-Y DATA DREDGED CONDITIONS (STREAMBED ELEVATION = +6.5 Feet NGVD) TRANSVERSE DIRECTION DYNAMIC CASE (M_W = 8.0)					
2.0 ft depth		4.0 ft depth		6.0 ft depth	
P (lbs/in)	Y (in)	P (lbs/in)	Y (in)	P (lbs/in)	Y (in)
0	0	0	0	0	0
16	0.033	32	0.033	48	0.033
32	0.067	64	0.067	96	0.067
48	0.100	96	0.100	144	0.100
64	0.133	128	0.133	192	0.133
79.9	0.167	160	0.167	240	0.167
83.3	0.200	192	0.200	288	0.200
86.3	0.233	224	0.233	336	0.233
89	0.267	255	0.267	384	0.267
91.5	0.300	262	0.300	428	0.300
93.8	0.333	267	0.333	441	0.333
95.8	0.367	273	0.367	452	0.367
97.8	0.400	278	0.400	463	0.400
126	0.900	349	0.900	622	0.900

Constant horizontal spring constants K_H along the headwall height were determined based on Skempton (1951) and Davisson (1970). Table 7-4 presents a summary of these stiffnesses.

TABLE 7-4 SUMMARY OF HORIZONTAL STIFFNESSES K_H (lbs/in^2.in)	
Behind the Headwall (Road Side)	1.7
Facing the Headwall (Streambed Side)	1.1

7.3 SHEAR STIFFNESS K_S

The shear or sliding stiffnesses at the base of the bridge piers and headwalls were calculated based on the bearing pressures provided in the "Ben C. Gerwick, Structural Evaluation Report, March 2002". Table 7-5-A presents a summary of the shear stiffnesses.

TABLE 7-5-A SUMMARY OF SHEAR STIFFNESSES K_S (lbs/in^2.in)			
Base of the West Bridge Headwall	Base of Bridge Pier #1 (West Side)	Base of Bridge Pier #2 (East Side)	Base of the East Bridge Headwall
11.39/δ	15.42/δ	15.83/δ	11.11/δ

Delta (δ) is the lateral deflection in inches at the base of the bridge pier or bridge headwall. For example, the shear stiffnesses for a displacement of ¼ inch are presented in Table 7-5-B.

TABLE 7-5-B SHEAR STIFFNESSES K_S for ¼ inch of displacement (lbs/in^2.in)			
Base of the West Bridge Headwall	Base of Bridge Pier #1 (West Side)	Base of Bridge Bier #2 (East Side)	Base of the East Bridge Headwall
45.6	61.7	63.3	44.4

8.0 LATERAL EARTH PRESSURES

Lateral earth pressures (i.e., at rest, active, and passive) were determined to estimate the amount of force acting along the headwalls. The analyses considered two scenarios: (1) the streambed at its current elevation of 10.5 feet NGVD, and (2) the streambed excavated to an elevation of 6.5 feet NGVD. Both static and seismic conditions were considered in the analyses.

8.1 ANALYSES METHODOLOGY

Soil properties used in the analyses for static and seismic conditions are discussed in Section 4.0. The water level behind the headwall was assumed to be located at an elevation of 12 feet NGVD. Within the streambed, the water level was assumed to be at the ground surface for both scenarios. Hydrostatic pressures will be acting on the headwall below these water levels. Surcharge loading from the traffic (see Section 4) was also taken into account in the analyses for

both static and seismic conditions. The Rankine theory for determining lateral earth pressures was implemented in determining active and passive pressures acting on the headwall.

8.1.1 Static Conditions

The soil behind the west headwall was assumed to be under at-rest conditions, due to the restriction of lateral movement of the wall provided by the bridge at the top and the soil embedment at the bottom. Rankine theory was used to determine the amount of passive earth pressure acting on the outboard of the headwall.

8.1.2 Seismic Conditions

Under seismic conditions, the soil behind the west headwall was assumed to be under active conditions from the movement induced by an earthquake event. Rankine theory was used to determine both the amounts of active and passive earth pressures acting along the headwall. Using simplified procedures adopted from Seed and Whitman (1970) and Towhata and Islam (1987), and assuming a failure wedge both behind and in the front of the headwall, an additional surcharge from an earthquake event producing a ground acceleration of 0.4g was estimated.

8.2 RESULTS

Figures 7 and 8 show a pressure diagrams of both scenarios for static and seismic conditions, respectively. The results indicate that in both conditions, excavation of the streambed results in lower passive resistance acting on the headwall. Due to the stiff cohesive soils behind the headwall, a tension crack filled with water was assumed to be present and was determined to extend past the headwall depth.

9.0 IMPACT OF WEST ABUTMENT BOX CULVERT

The proposed box culvert (10 feet high and 19 feet wide) will not impact the long-term performance of the west headwall. This is based on the conditions that the box culvert location will be a minimum of 10 feet to the west of the west bridge headwall, the base elevation is +12.5 feet NGVD, and the weight of the box culvert filled with water is less than or equal to the weight of the excavated soil.

There may be impact on the west headwall and bridge depending on how the excavation for the box culvert is braced and shored. This impact could soften the abutment soils if significant lateral movement occurs during construction.

10.0 SUMMARY OF CONCLUSIONS

The following conclusions are made with respect to the analyses performed for the Lagunitas Road Bridge site:

- Under the Caltrans seismic criteria, our results indicate that a significant portion of the subsoils at the pier locations will liquefy.

- Static factors of safety for the west abutment area are acceptable (FS > 1.5) under the existing conditions (streambed elevation at 10.5 feet NGVD). Assuming that the subsoil conditions are similar in the east abutment area, the static factors of safety would be acceptable for the east abutment. However, excavation of the streambed to an elevation of 6.5 feet NGVD will result in an unstable static condition in the abutments.

- Under current conditions, a horizontal acceleration of 0.4g produces marginal seismic factors of safety (FS = 1.0) in the west abutment area, and also likely in the east abutment.

- Long-term performance of the west headwall will not be affected by the proposed box culvert provided the structure is located a minimum 10 feet away and is lighter than the excavated soil.

11.0 RECOMMENDATIONS

Our analyses are based on surface soil data from the west abutment area of the Lagunitas Road Bridge and streambed area between the piers. In order to gain a complete understanding of the soils present at the site, it is advisable to verify the conditions in the east abutment area. We recommend that a limited investigation be performed to assess the soil conditions present in this location.

Currently, the base elevations of the west and east headwalls are unknown. It is recommended that the base elevations of the headwalls be determined to verify that the embedment depth of the headwalls is 9 feet or greater.

Based on available data and assumptions made of the site, our analyses indicate that it is not recommended to perform any excavation operations within the streambed under current soil conditions. Doing so will reduce the global stability of the west abutment area. Our analyses also indicate that the soils in the streambed area are susceptible to liquefaction and must be

mitigated. It is recommended that ground improvements such as grouting, jet grouting, or soil mixing be performed to strengthen the streambed soils. This will stiffen the soils, mitigate the liquefaction potential, and increase the minimum factors of safety to ensure adequate global stability of the abutments.

To fill tension cracks and gaps between the abutment soils and headwalls, it is recommended that a lean grouting program be undertaken in the abutment areas. A lean-mix cement grout would be used for the grouting program.

12.0 REFERENCES

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April 30, 2002
Job No. 43-00067008.01

Ben C. Gerwick, Inc.
1300 Clay Street, Suite 450
Oakland, CA 94612

Attention: Mr. Robert J. Filgas, PE
Project Manager

Dear Mr. Filgas:

Final Geotechnical Analysis Report
San Francisco District COE
IDC DACW07-00-D0003
Lagunitas Road Bridge Project
Ross, California

We are pleased to submit seven (7) copies of our final geotechnical analysis report for the Phase II portion of the Lagunitas Road Bridge Project. Our geotechnical analyses and preparation of this report completes the scope of services proposed on January 16, 2002 and authorized by the San Francisco District COE on February 21, 2002. Notice-to-proceed was received from Ben C. Gerwick, Inc. on March 5, 2002.

Our geotechnical analysis report provides assessments for (1) Seismic criteria, (2) Liquefaction potential, (3) Abutment stability, (4) Foundation stiffness, (5) Earth pressures, and (6) Sliding resistance. Three different scenarios were evaluated, consisting of the existing conditions, a lowered streambed, and a lowered streambed with a box culvert in the west abutment.

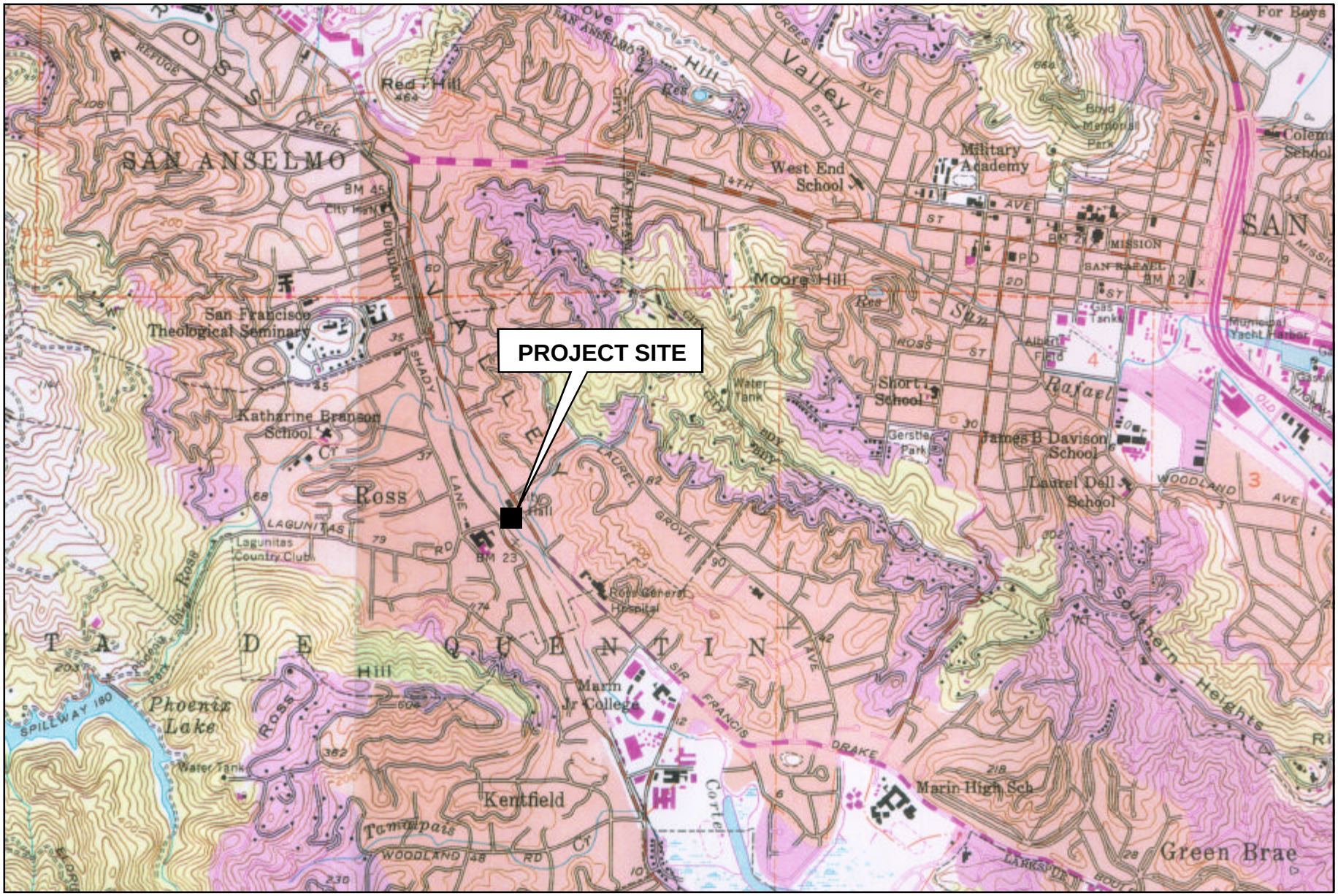
The foundation stiffness and sliding resistance analyses were performed by Makram El Sabbagh. The liquefaction and abutment stability analyses and earth pressure computations were performed by Peter Chiu.

We appreciated the opportunity to provide the geotechnical services to assess the subsurface conditions and to develop foundation design criteria. If you have any questions regarding this analysis report, please contact Francis R. Greguras at (415) 243-3808.

Very truly yours,

URS CORPORATION

Francis R. Greguras, PE, GE
Project Manager



Source:
USGS 7.5 Min. Series, San Rafael, California
Quadrangle, photorevised 1980.

N

0 0.5 1

Scale in Miles

SITE VICINITY MAP

Lagunitas Road Bridge Project Phase 2
Geotechnical Analysis Report
Ross, California

April 2002
43-00067008.01

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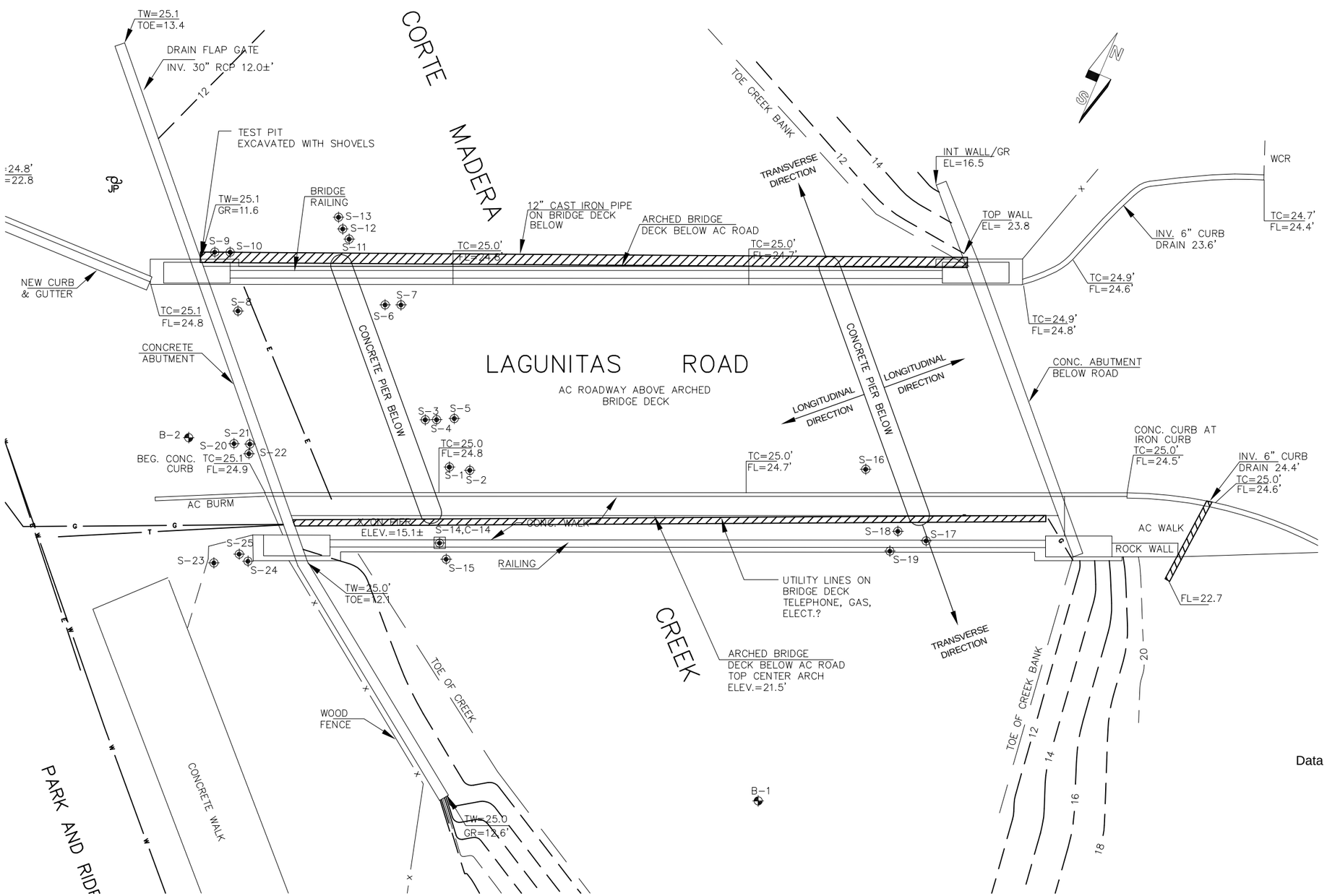
FIGURE 1

4/30/02 sv ..\43-00067008.01\svm.cdr



LEGEND:

- Auger Soundings
- Auger Sounding and Foundation Core Hole
- Geotechnical Boreholes

0 5 10 20
SCALE IN FEET

Data Reference: Topographic Survey Prepared By Carruthers Land Surveying for Sverdrup/Gerwick JV, November 2001

BORING AND SOUNDING LOCATION PLAN

Lagunitas Road Bridge Project, Phase 2
Geotechnical Analysis Report
Ross, California

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43-00067008.01

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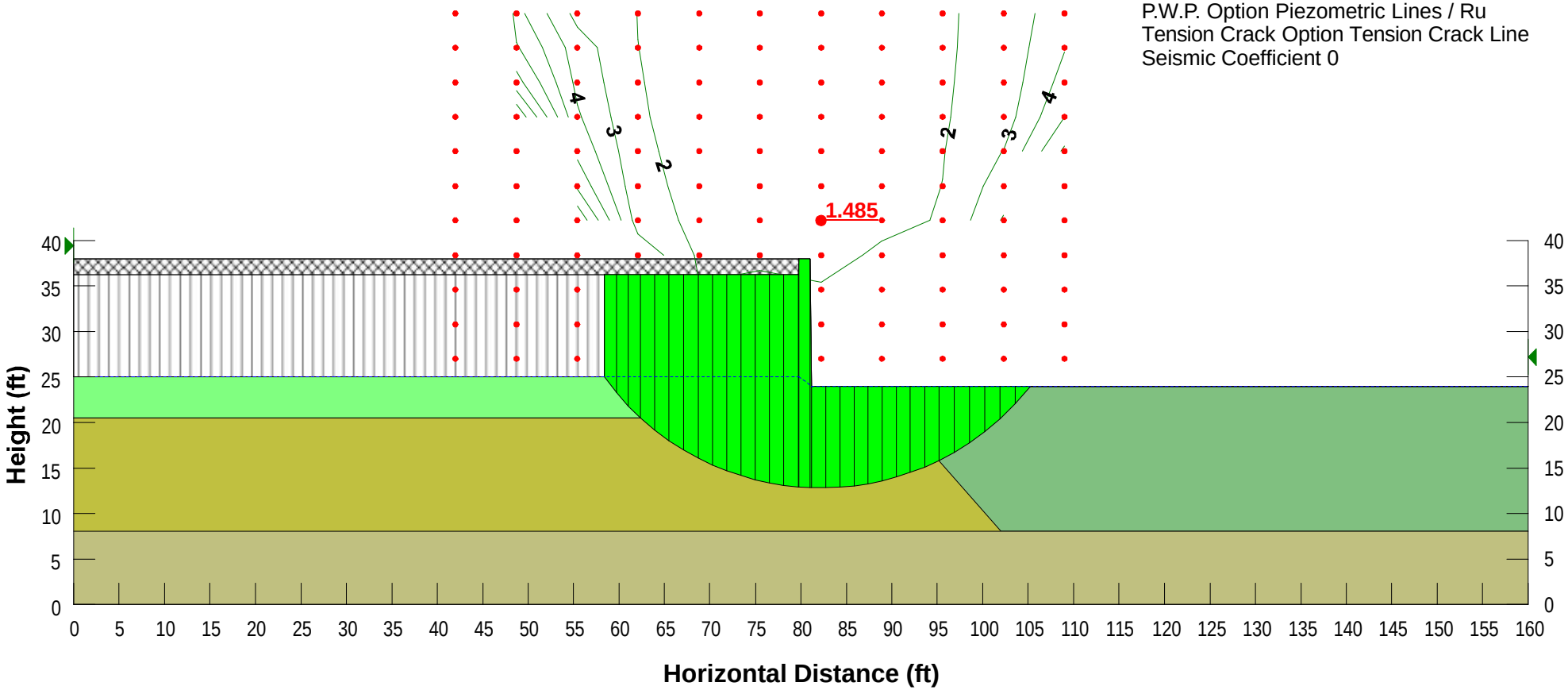
FIGURE 2

T:\43-00067008.01 Lagunitas Road Bridge\FIG_2.dwg, Layout1, 10/15/02 04:16:12 PM, Acrobat PDFWriter, Tabloid [11 x 17 in], 1:1.05399, Plotted By: DAF

Lagunitas Road Bridge
Streambed elevation as of October, 2001
02/04/04 17:11:00
File Name 10'S.SLP
Analysis Method Spencer
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option Tension Crack Line
Seismic Coefficient 0



WEST ABUTMENT STABILITY ANALYSIS

Lagunitas Road Bridge Project Phase 2
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Ross, California

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43-00067008.01

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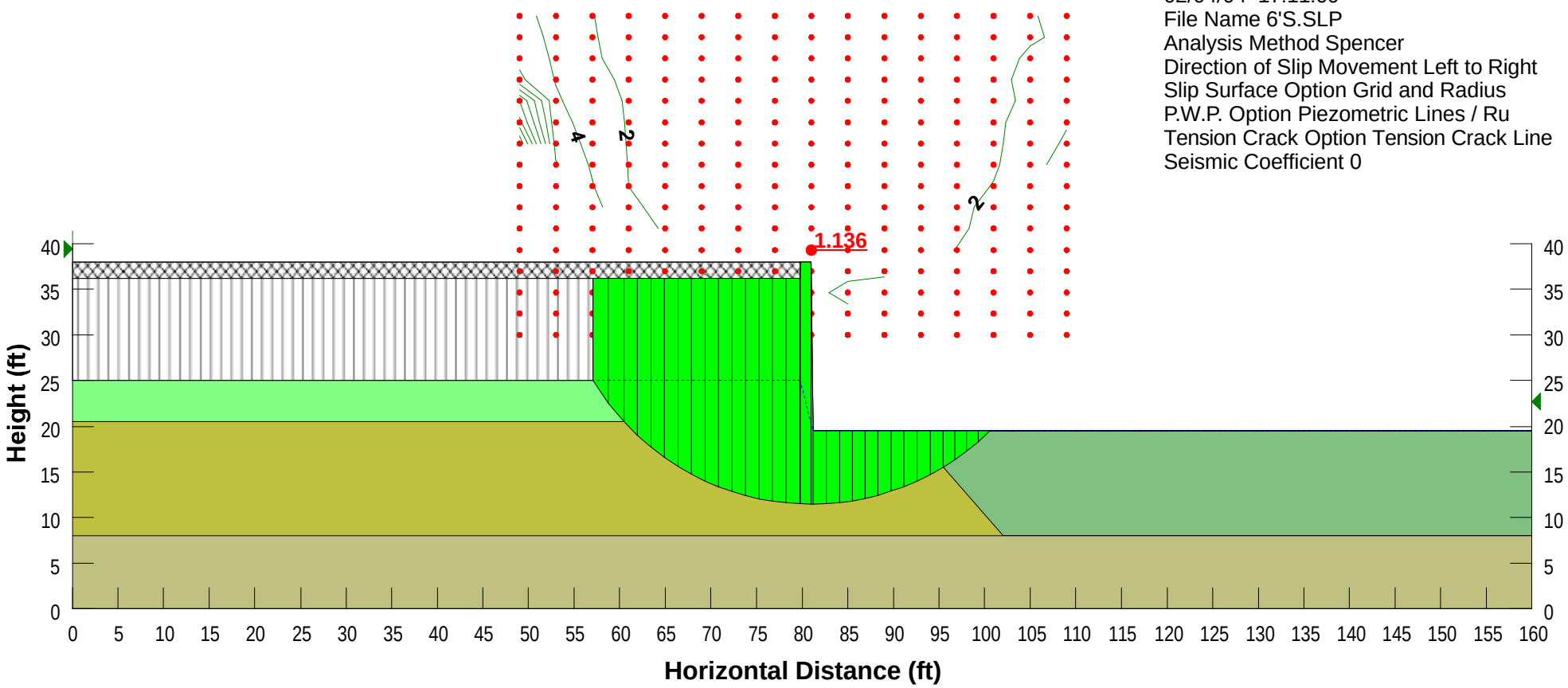
FIGURE 3

4/25/02 sv ..\43-00067008.01\fig3.cdr

Lagunitas Road Bridge
Streambed elevation excavated to 6.5'
02/04/04 17:11:00
File Name 6'S.SLP
Analysis Method Spencer
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option Tension Crack Line
Seismic Coefficient 0



WEST ABUTMENT STABILITY ANALYSIS

Lagunitas Road Bridge Project Phase 2
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FIGURE 4

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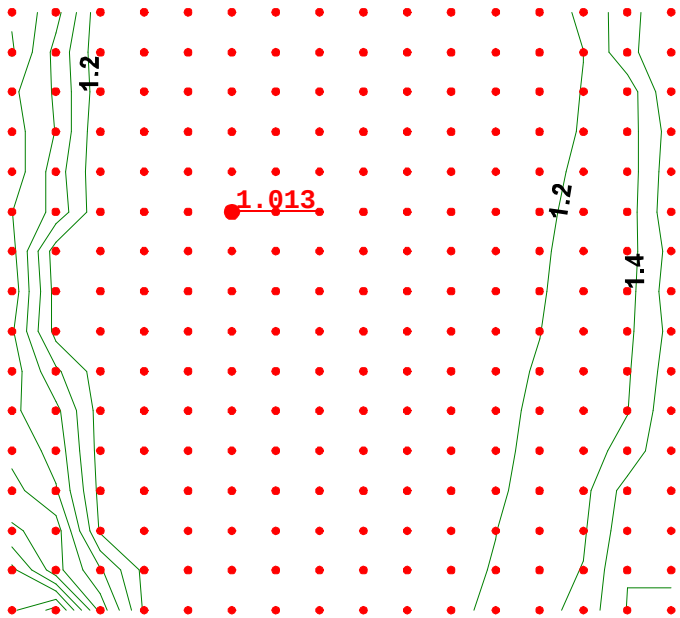
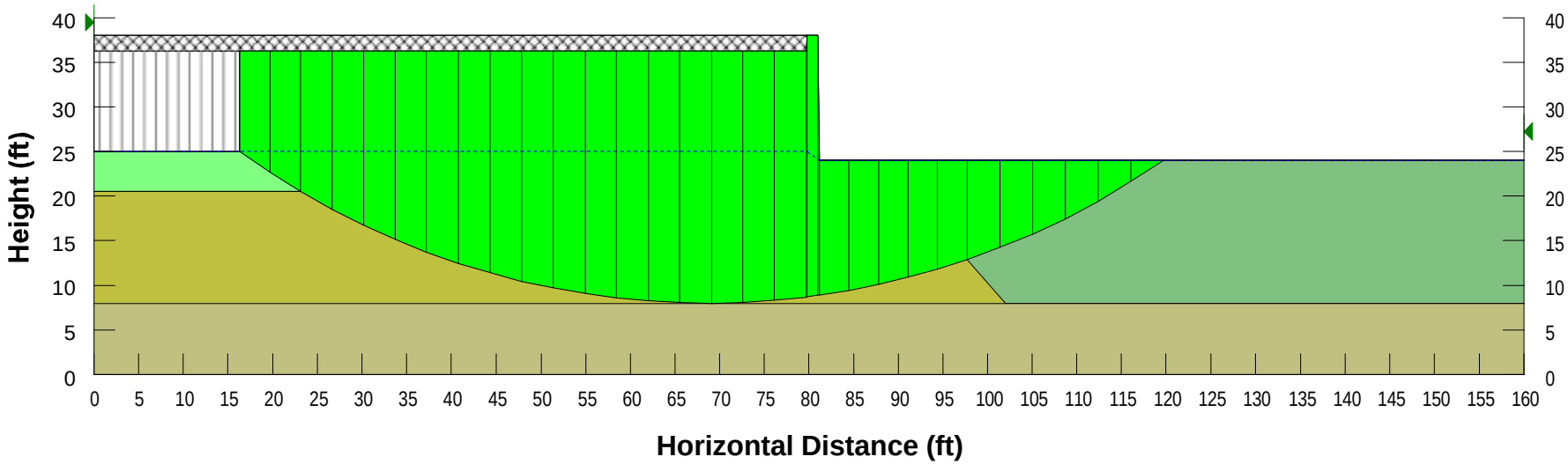


Lagunitas Road Bridge
Streambed elevation as of October, 2001
02/04/04 17:11:00
File Name 10'P.SLP
Analysis Method Spencer
Direction of Slip Movement Left to Right
Slip Surface Option Grid and Radius
P.W.P. Option Piezometric Lines / Ru
Tension Crack Option Tension Crack Line
Seismic Coefficient 0.2



WEST ABUTMENT STABILITY ANALYSIS

Lagunitas Road Bridge Project Phase 2
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Ross, California

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43-00067008.01

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FIGURE 5

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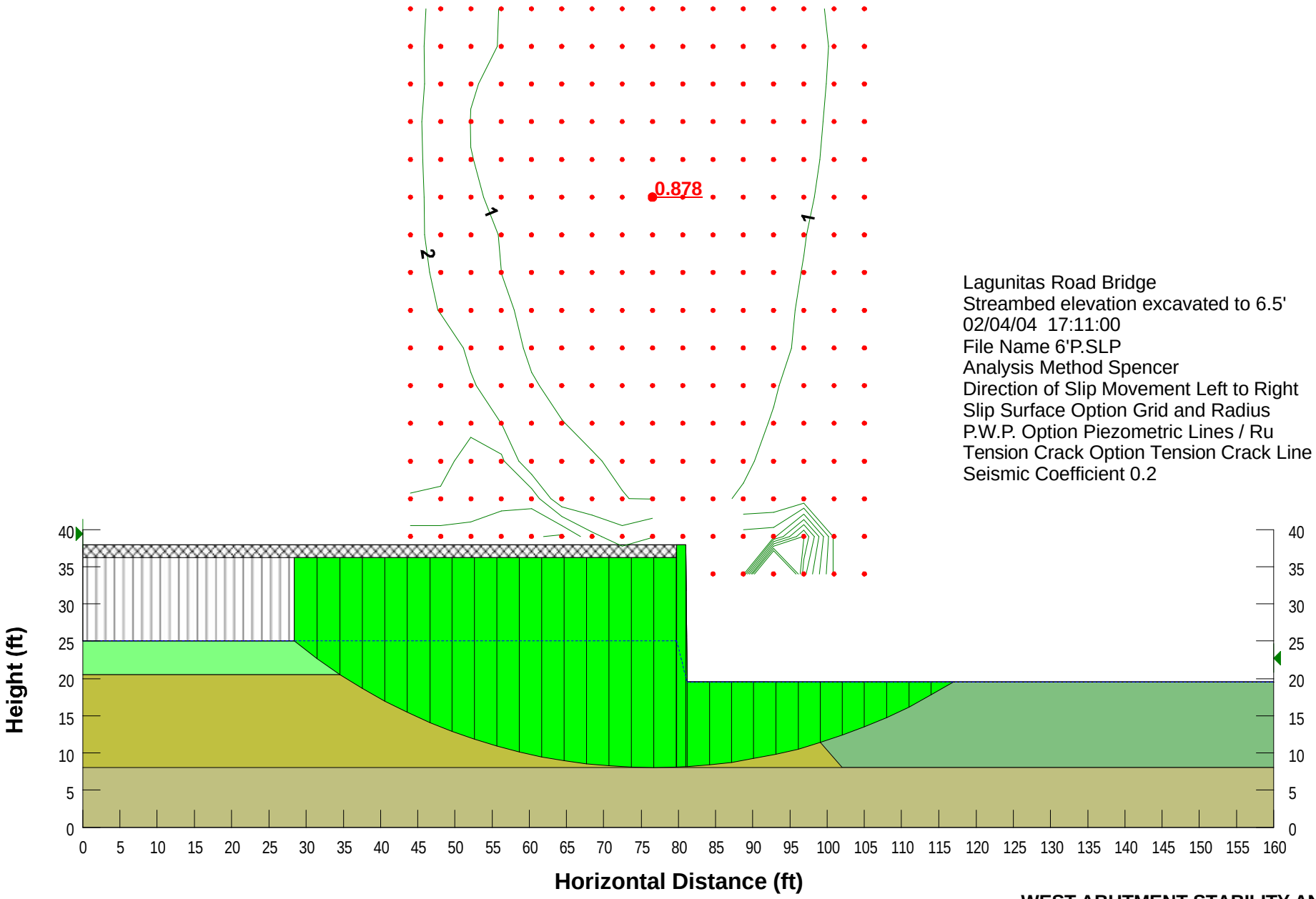

WEST ABUTMENT STABILITY ANALYSIS

Lagunitas Road Bridge Project Phase 2
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FIGURE 6

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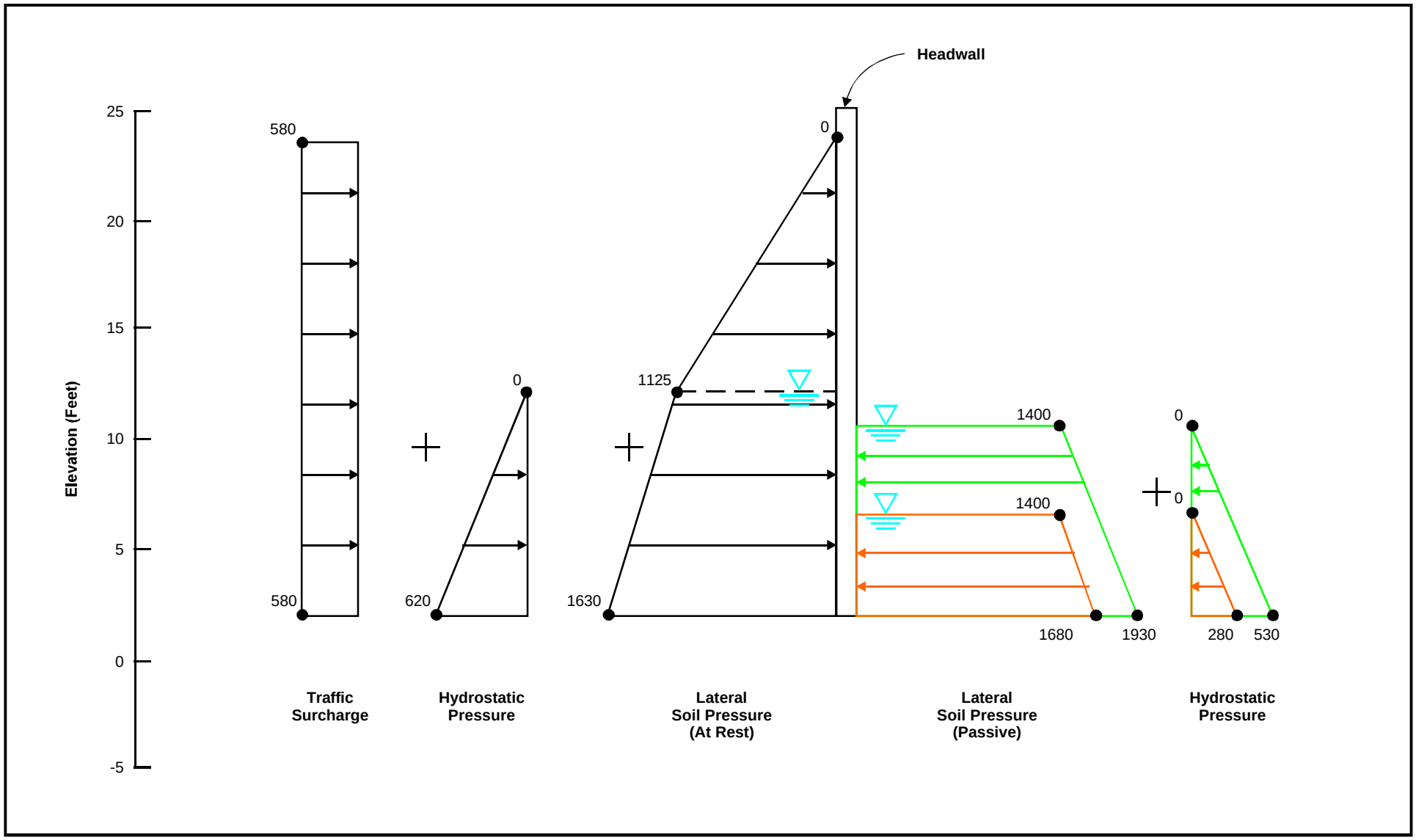

LEGEND

Scenario 1	Existing conditions as of October, 2001
Scenario 2	Streambed excavated to 6.5' elevation

PRESSURE DIAGRAM (STATIC CONDITIONS)

Lagunitas Road Bridge Project Phase 2
Geotechnical Analysis Report
Ross, California

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FIGURE 7

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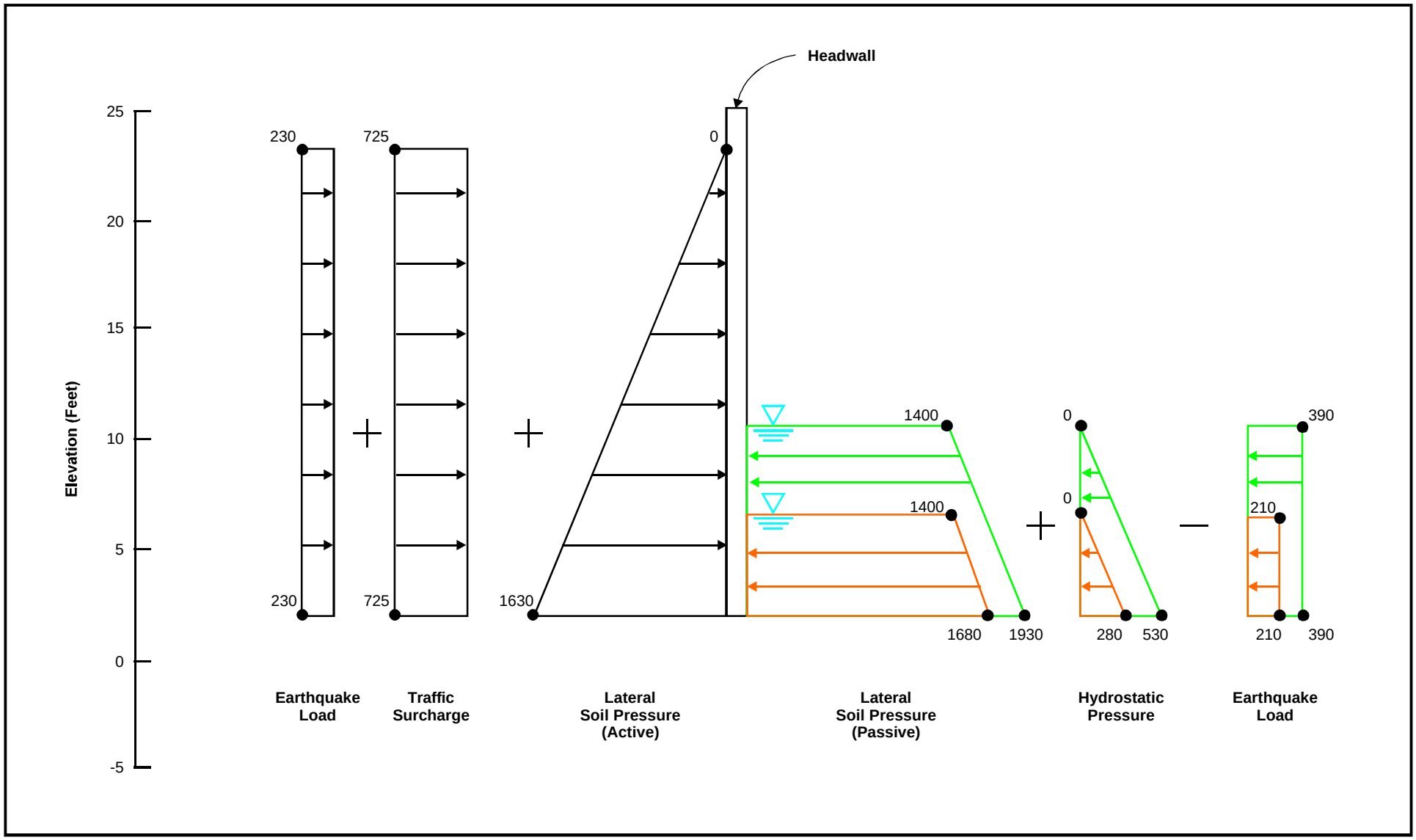

LEGEND

Scenario 1	Existing conditions as of October, 2001
Scenario 2	Streambed excavated to 6.5' elevation

PRESSURE DIAGRAM (SEISMIC CONDITIONS)

Lagunitas Road Bridge Project Phase 2
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Ross, California

April 2002
43-00067008.01

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FIGURE 8

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LAGUNITAS ROAD BRIDGE PROJECT -ROSS, CALIFORNIA
JOB NO. 43-00067008.01
LIQUEFACTION ANALYSIS - ADOPTED FROM YOUD & IDRISS 1997 PUBLICATION

Maximum Credible Event =	**7.5**	
PGA =	**0.3**	**g**

Assume the following Fill in-situ density for B-1:
γ_{sat} = 140 pcf
γ_{water} = 62.4 pcf
Water table at ground surface

Assume the following Fill in-situ density for B-2 and S-20:
γ_{sat} = 125 pcf (for 0-25' depth)
γ_{sat} = 140 pcf (for >25' depth)
γ_{water} = 62.4 pcf
Water table at 14' depth

CRR Curve for Clean Sand, M=7.5

a	4.80E-02
b	-1.25E-01
c	-4.72E-03
d	9.58E-03
e	6.14E-04
f	-3.29E-04
g	-1.67E-05
h	3.71E-06

Boring	Sample Depth (ft)	Mid-layer depth (ft)	N-value	Sampler Type	σ_{vo} (psf)	σ'_{vo} (psf)	σ'_{vo} (tsf)	Soil Type	Fines Contents (%)	Drill Notes	[1] $C_{H/S}$	[2] C_N	[3] C_E	[4] C_B	[5] C_R	[6] C_S
B-1	4.5 - 5.5	5	13	M&D	700	388.0	0.19	GP-GM	***2.2***		0.42	2.00	1	1.05	0.75	1
	9.5 - 10.5	10	32	M&D	1400	776.0	0.39	GW	***9.4***		0.42	1.61	1	1.05	0.75	1
	12.5 - 13.5	13	46	M&D	1820	1008.8	0.50	SW-SM	***5.9***		0.42	1.41	1	1.05	0.75	1
	14.5 - 15.5	15	27	SPT	2100	1164.0	0.58	GW-GM	***6.0***		1	1.31	1	1.05	0.85	1.1
	20.5 - 21.5	21	49	M&D	2940	1629.6	0.81	GP-GM	***11.6***		0.42	1.11	1	1.05	0.95	1
B-2	35 - 36	35.5	41	M&D	4595	3253.4	1.63	GP-GM	***11.6***		0.42	0.78	1.25	1.15	1	1.1
S-20	35 - 36	35.5	19	SPT	4595	3253.4	1.63	GP-GM	***11.6***		1	0.78	1.25	1.15	1	1

Boring	Sample Depth (ft)	Mid-layer depth (ft)	[7] α	[8] β	[9] $(N_1)_{60}$	[10] $(N_1)_{60\text{-}CS}$ (Idriss)	[11] Max M160CS, 30	[12] $CRR_{M=7.5}$	Depth (m)	[13] r_d	[14] $CSR_{M=7.5}$	[15] MSF	[16] CSR / MSF	[17] F.S	F.S	[18] Remarks
B-1	4.5 - 5.5	5	0.000	1.000	9	9	**9**	0.09	1.52	0.99	0.35	1.00	0.35	0.27	0.27	Will Liquefy
	9.5 - 10.5	10	0.677	1.019	17	18	**18**	0.19	3.05	0.98	0.34	1.00	0.34	0.56	0.56	Will Liquefy
	12.5 - 13.5	13	0.025	1.004	21	22	**22**	0.23	3.96	0.97	0.34	1.00	0.34	0.69	0.69	Will Liquefy
	14.5 - 15.5	15	0.030	1.005	35	35	**30**	0.46	4.57	0.97	0.34	1.00	0.34	Dense	1.35	Safe
	20.5 - 21.5	21	1.416	1.030	23	25	**25**	0.28	6.40	0.95	0.33	1.00	0.33	0.84	0.84	Will Liquefy
B-2	35 - 36	35.5	1.416	1.030	21	23	**23**	0.26	10.82	0.92	0.25	1.00	0.25	1.02	1.02	Marginally Safe
S-20	35 - 36	35.5	1.416	1.030	21	23	**23**	0.26	10.82	0.92	0.25	1.00	0.25	1.03	1.03	Marginally Safe

LAGUNITAS ROAD BRIDGE PROJECT -ROSS, CALIFORNIA
JOB NO. 43-00067008.01
LIQUEFACTION ANALYSIS - ADOPTED FROM YOUD & IDRISS 1997 PUBLICATION

Maximum Credible Event =	**7.5**	
PGA =	**0.4**	**g**

Assume the following Fill in-situ density:
γ_t = 140 pcf
γ_{water} = 62.4 pcf
Water table at ground surface for B-1
Water table at 14' depth for B-2 and S-20

CRR Curve for Clean Sand, M=7.5

a	4.80E-02
b	-1.25E-01
c	-4.72E-03
d	9.58E-03
e	6.14E-04
f	-3.29E-04
g	-1.67E-05
h	3.71E-06

Boring	Sample Depth (ft)	Mid-layer depth (ft)	N-value	Sampler Type	σ_{vo} (psf)	$\sigma_v'_o$ (psf)	$\sigma_v'_o$ (tsf)	Soil Type	Fines Contents (%)	Drill Notes	[1] $C_{H/S}$	[2] C_N	[3] C_E	[4] C_B	[5] C_R	[6] C_S
B-1	4.5 - 5.5	5	13	M&D	700	388.0	0.19	GP-GM	***2.2***		0.42	2.00	1	1.05	0.75	1
	9.5 - 10.5	10	32	M&D	1400	776.0	0.39	GW	***9.4***		0.42	1.61	1	1.05	0.75	1
	12.5 - 13.5	13	46	M&D	1820	1008.8	0.50	SW-SM	***5.9***		0.42	1.41	1	1.05	0.75	1
	14.5 - 15.5	15	27	SPT	2100	1164.0	0.58	GW-GM	***6.0***		1	1.31	1	1.05	0.85	1.1
	20.5 - 21.5	21	49	M&D	2940	1629.6	0.81	GP-GM	***11.6***		0.42	1.11	1	1.05	0.95	1
B-2	35 - 36	35.5	41	M&D	4595	3253.4	1.63	GP-GM	***11.6***		0.42	0.78	1.25	1.15	1	1.1
S-20	35 - 36	35.5	19	SPT	4595	3253.4	1.63	GP-GM	***11.6***		1	0.78	1.25	1.15	1	1

Boring	Sample Depth (ft)	Mid-layer depth (ft)	[7] α	[8] β	[9] $(N_1)_{60}$	[10] $(N_1)_{60\text{-}CS}$ (Idriss)	[11] Max M160CS, 30	[12] $CRR_{M=7.5}$	Depth (m)	[13] r_d	[14] $CSR_{M=7.5}$	[15] MSF	[16] CSR / MSF	[17] F.S	F.S	[18] Remarks
B-1	4.5 - 5.5	5	0.000	1.000	9	9	**9**	0.09	1.52	0.99	0.46	1.00	0.46	0.20	0.20	Will Liquefy
	9.5 - 10.5	10	0.677	1.019	17	18	**18**	0.19	3.05	0.98	0.46	1.00	0.46	0.42	0.42	Will Liquefy
	12.5 - 13.5	13	0.025	1.004	21	22	**22**	0.23	3.96	0.97	0.45	1.00	0.46	0.52	0.52	Will Liquefy
	14.5 - 15.5	15	0.030	1.005	35	35	**30**	0.46	4.57	0.97	0.45	1.00	0.45	Dense	1.01	Marginally Safe
	20.5 - 21.5	21	1.416	1.030	23	25	**25**	0.28	6.40	0.95	0.45	1.00	0.45	0.63	0.63	Will Liquefy
B-2	35 - 36	35.5	1.416	1.030	21	23	**23**	0.26	10.82	0.92	0.34	1.00	0.34	0.77	0.77	Will Liquefy
S-20	35 - 36	35.5	1.416	1.030	21	23	**23**	0.26	10.82	0.92	0.34	1.00	0.34	0.77	0.77	Will Liquefy

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Maximum Credible Event =	**8**	
PGA =	**0.3**	**g**

Assume the following Fill in-situ density:
γ_t = 140 pcf
γ_{water} = 62.4 pcf
Water table at ground surface for B-1
Water table at 14' depth for B-2 and S-20

CRR Curve for Clean Sand, M=7.5

a	4.80E-02
b	-1.25E-01
c	-4.72E-03
d	9.58E-03
e	6.14E-04
f	-3.29E-04
g	-1.67E-05
h	3.71E-06

Boring	Sample Depth (ft)	Mid-layer depth (ft)	N-value	Sampler Type	σ_{vo} (psf)	$\sigma_v'_o$ (psf)	$\sigma_v'_o$ (tsf)	Soil Type	Fines Contents (%)	Drill Notes	[1] $C_{H/S}$	[2] C_N	[3] C_E	[4] C_B	[5] C_R	[6] C_S
B-1	4.5 - 5.5	5	13	M&D	700	388.0	0.19	GP-GM	***2.2***		0.42	2.00	1	1.05	0.75	1
	9.5 - 10.5	10	32	M&D	1400	776.0	0.39	GW	***9.4***		0.42	1.61	1	1.05	0.75	1
	12.5 - 13.5	13	46	M&D	1820	1008.8	0.50	SW-SM	***5.9***		0.42	1.41	1	1.05	0.75	1
	14.5 - 15.5	15	27	SPT	2100	1164.0	0.58	GW-GM	***6.0***		1	1.31	1	1.05	0.85	1.1
	20.5 - 21.5	21	49	M&D	2940	1629.6	0.81	GP-GM	***11.6***		0.42	1.11	1	1.05	0.95	1
B-2	35 - 36	35.5	41	M&D	4595	3253.4	1.63	GP-GM	***11.6***		0.42	0.78	1.25	1.15	1	1.1
S-20	35 - 36	35.5	19	SPT	4595	3253.4	1.63	GP-GM	***11.6***		1	0.78	1.25	1.15	1	1

Boring	Sample Depth (ft)	Mid-layer depth (ft)	[7] α	[8] β	[9] $(N_1)_{60}$	[10] $(N_1)_{60\text{-}CS}$ (Idriss)	[11] Max M160CS, 30	[12] $CRR_{M=7.5}$	Depth (m)	[13] r_d	[14] $CSR_{M=7.5}$	[15] MSF	[16] CSR / MSF	[17] F.S	F.S	[18] Remarks
B-1	4.5 - 5.5	5	0.000	1.000	9	9	**9**	0.09	1.52	0.99	0.35	0.85	0.41	0.23	0.23	Will Liquefy
	9.5 - 10.5	10	0.677	1.019	17	18	**18**	0.19	3.05	0.98	0.34	0.85	0.41	0.48	0.48	Will Liquefy
	12.5 - 13.5	13	0.025	1.004	21	22	**22**	0.23	3.96	0.97	0.34	0.85	0.40	0.58	0.58	Will Liquefy
	14.5 - 15.5	15	0.030	1.005	35	35	**30**	0.46	4.57	0.97	0.34	0.85	0.40	Dense	1.15	Safe
	20.5 - 21.5	21	1.416	1.030	23	25	**25**	0.28	6.40	0.95	0.33	0.85	0.39	0.71	0.71	Will Liquefy
B-2	35 - 36	35.5	1.416	1.030	21	23	**23**	0.26	10.82	0.92	0.25	0.85	0.30	0.87	0.87	Will Liquefy
S-20	35 - 36	35.5	1.416	1.030	21	23	**23**	0.26	10.82	0.92	0.25	0.85	0.30	0.87	0.87	Will Liquefy

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Maximum Credible Event =	**8**	
PGA =	**0.4**	**g**

Assume the following Fill in-situ density:
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γ_{water} = 62.4 pcf
Water table at ground surface for B-1
Water table at 14' depth for B-2 and S-20

CRR Curve for Clean Sand, M=7.5

a	4.80E-02
b	-1.25E-01
c	-4.72E-03
d	9.58E-03
e	6.14E-04
f	-3.29E-04
g	-1.67E-05
h	3.71E-06

Boring	Sample Depth (ft)	Mid-layer depth (ft)	N-value	Sampler Type	σ_{vo} (psf)	$\sigma_v'_o$ (psf)	$\sigma_v'_o$ (tsf)	Soil Type	Fines Contents (%)	Drill Notes	[1] $C_{H/S}$	[2] C_N	[3] C_E	[4] C_B	[5] C_R	[6] C_S
B-1	4.5 - 5.5	5	13	M&D	700	388.0	0.19	GP-GM	***2.2***		0.42	2.00	1	1.05	0.75	1
	9.5 - 10.5	10	32	M&D	1400	776.0	0.39	GW	***9.4***		0.42	1.61	1	1.05	0.75	1
	12.5 - 13.5	13	46	M&D	1820	1008.8	0.50	SW-SM	***5.9***		0.42	1.41	1	1.05	0.75	1
	14.5 - 15.5	15	27	SPT	2100	1164.0	0.58	GW-GM	***6.0***		1	1.31	1	1.05	0.85	1.1
	20.5 - 21.5	21	49	M&D	2940	1629.6	0.81	GP-GM	***11.6***		0.42	1.11	1	1.05	0.95	1
B-2	35 - 36	35.5	41	M&D	4595	3253.4	1.63	GP-GM	***11.6***		0.42	0.78	1.25	1.15	1	1.1
S-20	35 - 36	35.5	19	SPT	4595	3253.4	1.63	GP-GM	***11.6***		1	0.78	1.25	1.15	1	1

Boring	Sample Depth (ft)	Mid-layer depth (ft)	[7] α	[8] β	[9] $(N_1)_{60}$	[10] $(N_1)_{60\text{-}CS}$ (Idriss)	[11] Max M160CS, 30	[12] $CRR_{M=7.5}$	Depth (m)	[13] r_d	[14] $CSR_{M=7.5}$	[15] MSF	[16] CSR / MSF	[17] F.S	F.S	[18] Remarks
B-1	4.5 - 5.5	5	0.000	1.000	9	9	**9**	0.09	1.52	0.99	0.46	0.85	0.55	0.17	0.17	Will Liquefy
	9.5 - 10.5	10	0.677	1.019	17	18	**18**	0.19	3.05	0.98	0.46	0.85	0.54	0.36	0.36	Will Liquefy
	12.5 - 13.5	13	0.025	1.004	21	22	**22**	0.23	3.96	0.97	0.45	0.85	0.54	0.44	0.44	Will Liquefy
	14.5 - 15.5	15	0.030	1.005	35	35	**30**	0.46	4.57	0.97	0.45	0.85	0.53	Dense	0.86	Will Liquefy
	20.5 - 21.5	21	1.416	1.030	23	25	**25**	0.28	6.40	0.95	0.45	0.85	0.53	0.53	0.53	Will Liquefy
B-2	35 - 36	35.5	1.416	1.030	21	23	**23**	0.26	10.82	0.92	0.34	0.85	0.40	0.65	0.65	Will Liquefy
S-20	35 - 36	35.5	1.416	1.030	21	23	**23**	0.26	10.82	0.92	0.34	0.85	0.40	0.65	0.65	Will Liquefy

NOTES - page numbers refer to Youd & Idriss 1997 publication

[1] Sampler correction: 0.42 for D & M; 1.0 for SPT

[2] Overburden correction: Square root ($1/\sigma'_{vo}$) in tsf

[3] Hammer energy correction: Used 60% Hammer Efficiency for Rope and Cathead (C_E = 1) and 75% for CME Auto Hammer (C_E = 1.25)

[4] Borehole diameter correction: 1.15 for 8", 1.05 for 6", 1.0 for 2.5-4.5"

[5] Rod correction: 0.75 for 0-13', 0.85 for 13-20', 0.95 for 20-32', 1.00 for over 32'

[6] Sampling method correction: 1.1 to 1.3 for sampler w/o liners, 1.0 for standard sampler

[7] Fines content and soil plasticity correction (pg. 7)

[8] Fines content and soil plasticity correction (pg. 7)

[9] Corrected Blow Count: $(N_1)_{60} = N \times C_{H/S} * C_N * C_E * C_B * C_R * C_S$

[10] Blow count with Fines content and soil plasticity correction (pg. 7)

[11] 30 is the maximum blow count that can be used in this procedure

[12] From simplified base curve (Seed et al. 1985) using Blake equation (pgs. 6-7)

[13] Stress reduction coefficient (pg 3, eq. 2a)

[14] Cyclic stress ratio (pg. 3, eq. 1)

[15] Magnitude scaling factor (pg. 27, eq. 25)

[16] CSR x MSF

[17] Factor of safety: If corrected blow count > 30, soil is dense
$(CRR_{7.5}/CSR)MSF$ (pg. 24)

[18]

FS < 1	Will liquefy
FS 1 - 1.1	Marginally safe
FS > 1.1	Safe