State of California
DEPARTMENT OF TRANSPORTATION

California State Transportation Agency

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To: MR. DAVID SOON, CHIEF
Bridge Design Branch 10
Office of Bridge Design North

Date: February 14, 2019

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0512000134 (EA 05-1C360)

San Jose Creek Bridge (Replace)
Bridge No. 51-0217

Attn: Ms. Tracy Sanderson

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design South

Subject: **PRELIMINARY FOUNDATION REPORT (PFR) FOR SAN JOSE CREEK BRIDGE (REPLACE)**

Scope of Work

Per the request from Office of Bridge Design North (OBDN), Branch 10 dated October 4, 2017, this Preliminary Foundation Report (PFR) has been prepared for the proposed replacement of San Jose Creek Bridge. The purpose of this report is to summarize the investigations performed and to provide preliminary foundation recommendations for the San Jose Creek Bridge (Replace). The recommendations presented in this report are based on the location plan, APS, preliminary loads, foundation design data sheet provided by OBDN on October 7, 2017 and on January 17, 2019 (revised), preliminary hydraulic report dated October 16, 2018, a recent 2018 subsurface investigation consisting of three borings and two Cone Penetrometer Tests (CPTs) and review of the 1963 As-built Log of Test Borings (LOTB) for San Jose Creek Bridge (see As-built Data section of this report).

Project Description

The existing 4-lane bridge spans the San Jose Creek, carrying State Route 217 traffic to and from University of California Santa Barbara in the City of Goleta, Santa Barbara County. Reactive aggregates in all structure concrete elements has caused the deterioration of the bridge which resulted in peer review recommendation of bridge replacement. However, according to Structure Maintenance no sign of distress has been noted in any substructure elements. San Jose Creek coalesces with San Pedro Creek before draining into Goleta Slough and Pacific Ocean.

All elevations referenced within this report are based on the North American Vertical Datum of 1988 (NAVD 88), except 1963 As-built plans. To convert an elevation at this site from National Geodetic Vertical Datum of 1929 (NGVD 29) to NAVD 88, add 2.05 feet to the NGVD 29 elevations.

Field Investigation and Field Testing Program

The field investigation was begun on July 31, 2018 and completed on August 30, 2018. The investigation included drilling and sampling of three 4.3-inch diameter, mud rotary core (RC) borings, one at each proposed support locations, and two 2.0-inch diameter cone penetration test (CPT) soundings, one at each abutment. Drillings were performed by Caltrans Drilling Services and CPTs were performed by Gregg Drilling Services. Soils were logged and classified by Kleinfelder Consultants in accordance with the 2010 Caltrans Soil and Rock Logging, Classification and Presentation Manual.

Soil/formational samples were obtained within borings mostly using the Standard Penetration Test (SPT) Sampler and 2.5-inch inner diameter (I.D.) Punch Core and HQ Samplers, with minor use of 2.0-inch I.D. spilt-barrel modified California Sampler. Sampling was predominantly continuous with alternating SPT's and Punch Core Samplings. SPTs were at 5-foot depth intervals. SPTs were performed in accordance with ASTM D1586 using a 1.4-inch I.D. split spoon sampler with a 140 lbs. safety hammer dropped 30 inches, with a 78% hammer efficiency. SPT samples were collected in zip lock bags. All samples were collected in core boxes. Log of Test Borings (LOTB) will be provided to your office upon completion.

Laboratory Testing Program

Selected representative soil samples were tested in Caltrans laboratory to obtain or derive relevant physical and engineering soil properties. All laboratory tests were performed in general accordance with California Test Methods (CTM) or American Society for Testing and Materials (ASTM) Standards. Field and laboratory testing intervals are to be shown on the LOTB sheets.

Site Geology and Subsurface Conditions

The project site is located within the Western Transverse Ranges Geomorphic Province, along the coastal low lands of Santa Barbara Plain at southern foothills of Santa Ynez Mountains. A Geologic Map of the Goleta Quadrangle (Dibblee, 1987) shows that the site is underlain by Holocene surficial alluvium of unconsolidated flood plain deposits of silt, sand and gravel, and older dissected Pleistocene surficial alluvial deposits of weakly consolidated silt, sand and gravel. Outcrops of southerly dipping beds of Monterey Formation are mapped on the east and west of the site at 60° to 75°, and 40° to 45°, respectively. Depth to formational material is between 50 to 85 feet.

Based on information from recent site investigation, different soil units are encountered at the proposed bridge supports, as characterized below (given elevations are approximate).

Abutment 1 (Boring RC-18-003): Surface elevation of +16.2 to +6 ft. very stiff elastic silt (fill material); elevation +6 to -11 ft. medium dense to very dense silty sand with gravel and trace shell fragments at elevation -7 ft.; elevation -11 to -41 ft. very stiff to hard silt with little fine sand at elevation -19 ft. and little fine gravel at elevation -31 ft.; underlain by very soft to soft, shale unit of Monterey Formation.

Pier 2 (Boring RC-18-002): Bottom of creek elevation of +0.5 to -18 ft. dense fossiliferous poorly graded sand with silt; elevation -18 to -50 hard elastic silt; underlain by very soft to soft, shale unit of Monterey Formation.

Abutment 3 (Boring RC-18-001): Surface elevation of 13.6 to +8 ft. embankment fill; elevation +8 to +3 ft. very stiff sandy lean clay; elevation +3 to -23 ft. loose to very dense silty sand; elevation -23 to -35 ft. very stiff silt with sand; elevation -35 to -41 ft. very dense silty sand; elevation -41 to -61 ft. very stiff lean clay with thin interbeds of silt, sand and gravel; elevation -61 to -68 ft. hard and very stiff elastic silt; underlain by very soft to soft, shale unit of Monterey Formation.

Groundwater

During 2018 field investigation, groundwater levels were measured from elevations +2.2 ft. (at Abut. 1) to +0.65 ft. (at Abut. 3), and surface water was measured at +3.5 ft. (at Pier 2) in the San Jose creek beneath the bridge site. Groundwater was encountered in all borings during the 1959 subsurface investigation. The highest measured groundwater level is +3.2 feet (per NGVD 1929 Datum), or +5.2 (per NAVD 1988 Datum). It should be noted that groundwater levels can fluctuate with the change of season and other factors including sea level rise. The design ground water table was considered as +5.2 feet.

As-built Foundation Data

Construction of original San Jose Creek Bridge was completed in 1963 with all bents supported on driven concrete pile extensions and abutments and wing walls on driven concrete piles. The existing structure is a continuous 7-span RC slab bridge with 11-column bents and end diaphragm abutments. Table 1 presents a summary of the 1963 As-built Data.

Table 1: Summary of the 1963 As-built Data

Support Type	Foundation Type	Design Load	Estimated Tip Elevations (ft.)	Specified Pile Tip Elevation (ft.)
Open end Diaphragm Abutments	Class I driven Concrete Piles	45 ton	-25.0 (NGVD 29)	-20.0 (NGVD 29)
11-Column Bents	Class I driven Concrete Pile Extensions			

Scour Evaluation

Per Preliminary Hydraulic Report, dated October 16, 2018, prepared by Structures Hydraulics and Hydrology, there is no contraction scour and no degradation scour for this structure. The local (pier) scour depth is 7.9 ft. and the long-term scour elevation is approximately -3.0 ft.

Corrosion Evaluation

Nine soil samples taken from Boring No. RC-18-001, four soil samples taken from Boring No. RC-18-002, five soil samples taken from Boring No. RC-18-003, and a water sample from the San Jose creek were tested by Caltrans laboratory for corrosion testing. A summary of corrosion test results is presented in Table 2. Based on the results of corrosion tests, the site is considered corrosive to foundation elements.

Table 2. Corrosion Test Results

Boring No.	Sample Depth (ft)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
RC-18-001	6.5 - 10	8.14	989	445	82
	11.5 - 15	7.78	455	852	550
	36.5 - 40	7.63	130	1600	4100
	46.5 – 50	8.22	130	790	4600
	51.5 - 55	8.03	122	1000	3400
	61.5 - 65	7.31	154	780	4700
	71.5 - 75	7.16	195	900	3950
	91.5 - 95	5.22	629	3500	87
	116.5 - 120	6.78	465	4600	96
RC-18-002	31.5 - 35	8.01	100	1735	9813
	41.5 – 45	6.81	158	2829	5939
	76.5 - 80	7.44	795	1398	657
	105 - 110	6.94	623	2370	112
RC-18-003	0 - 5	7.75	1354	299	136
	31.5 - 35	8.06	1448	144	22
	41.5 – 45	7.75	95	2415	14499
	76.5 - 80	7.37	361	2867	981
	105 - 110	7.37	273	5069	1466
Surface water sample from the creek	N/A	7.72	29	3100	21000

Note: The Caltrans Corrosion Guidelines states that if the minimum resistivity is greater than 1100 Ohm-Cm the sample is considered to be non-corrosive and testing to determine sulfate and chloride is not performed. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 1500 ppm, or the pH is 5.5 or less.

Seismic Design Information and Recommendations

The project site may be subject to strong ground motions from nearby earthquake sources during the design life of the bridge. Based on the recent (2018) field investigation for subsurface information and the Standard Penetration Test correlations, the average shear wave velocity for the upper 100 feet (V_{S30}) of soil is estimated to be 755 ft/sec (230 m/sec).

The Design Spectrum was determined using the Caltrans ARS Online (v. 2.3.09) web tool. The Design Spectrum is the upper envelope of deterministic and probabilistic response spectrums. For this site, the Design Spectrum is controlled by the probabilistic approach. The probabilistic ARS curve corresponds to a ground motion return period of 975 year (i.e., 5% probability to be exceeded in 50 years).

Using the USGS Interactive Deaggregation Tool, the controlling probabilistic fault scenario for this site was determined to have a design magnitude of M = 7.3 and site-to-fault distance of approximately 5.67 km (3.52 miles).

The peak ground acceleration (PGA) is 0.7g. Seismic Design Data for San Jose Creek Bridge (Replace) are presented in Attachment 1.

Soil at Abutment 1 and Pier 2 is competent. Soil at Abutment 3 is poor. In the "Preliminary Foundation Recommendations" section of this report, 36-inch diameter CIDH piles have been suggested for Abutment 3, instead of 24-inch CDH piles proposed by OBDN. Once the pile type is selected by OBDN, soil modulus parameters for p-y curves for Abutment 3 will be provided.

The project is mapped within the tsunami inundation zone on the State of California Tsunami Inundation Map for Emergency Planning. Consideration of the effects of a tsunami on the proposed structure should be considered.

Surface Fault Rupture Hazard

The site is not located within any Alquist-Priolo Earthquake Fault Zone as established by the California Geological Survey and is not located within 1000 feet of a fault that is Holocene or younger in age. Therefore, potential for surface fault rupture does not exist.

Liquefaction Potential and Lateral Spreading Evaluation

Liquefaction potential exists at Abutment 3 and Liquefaction potential does not exist at Abutment 1 and Pier 2. There is a potential for lateral spreading potential at Abutment 3. Please refer to the Attachment 2 for Lateral Spreading Evaluation. OBDN and OGDS need to work together between the 0 phase and 1 phase to design piles to evaluate lateral capacity of pile for lateral spreading soil.

Preliminary Foundation Recommendations

These recommendations are intended to provide preliminary support type selection and feasibility, based on available data, constructability and suitability for this site. The actual foundation types and recommendations will depend on design requirements and environmental restrictions.

The existing structure is supported on piles at abutments and pile extensions at bents. The following recommendations are for the proposed replacement of San Jose Creek Bridge, as shown on the general layout plan provided by OBDN. CIDH pile foundation is presented in the general plan as the foundation support. Structure design has provided preliminary plan and design loading for this alternative. However, other foundation types such as driven concrete or H piles are also possible options for abutments, if environmentally acceptable. If steel piles are selected, they would need to be mitigated to account for the corrosive environment by adding a sacrificial steel thickness to the steel piles (please refer to the latest Caltrans Corrosion Guidelines and consult Caltrans Corrosion and Structural Concrete Field Investigation Branch (CSCFI) in Office of Structural Materials). Also, if steel piles are selected, they would require a closed ended tip to create a displacement pile. Driven piles for Pier 2 may not be suitable for the supports at this site due to environmental restrictions. If another foundation type is selected by Structure Design other than drilled shaft foundation, appropriate design loading needs to be provided by Structure Design.

The General Foundation Information and Preliminary Design Loads are provided by OBDN and presented in Tables 3 and 4.

Table 3. General Foundation Information Provided By OBDN

Support Location	Pile Type	Finished Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load*	Number of Piles per Support
				B	L		
Abut 1	24” dia. CIDH	11.0	5.0	9	131	1”	28
Pier 2	78” dia. CIDH pile with casing (Type II Shaft) **	0.0	-2.0	N/A	N/A	1”	8
Abut 3	24” dia. CIDH	11.7	5.5	9	130	1”	28

* Based on CALTRANS’ current practice, the total permissible settlement is one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.
** Below the casing tip elevation, the CIDH pile diameter is 66”.

Table 4. Preliminary Design Loads Provided By OBDN

Support Location	Service-1 Limit State (kips)		Strength Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load Per Support	Permanent Load Per Support	Compression		Tension		Compression		Tension	
			Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile
Abut 1	2436	1794	4913	271	0	0	1794	182	0	0
Pier 2	8916	7085	11500	1860	0	0	7085	1135	0	0
Abut 3	2436	1794	4913	271	0	0	1794	182	0	0

The following foundation recommendations were designed in accordance with the 2014 AASHTO LRFD Bridge Design Specification (6^{th} Edition) with CA Amendments.

Design tip elevations and controlling design tip elevation for abutments and the bent provided in Tables 5 and 6 are prepared by OGDS. At Abutment 3, liquefaction exists and there is a potential for lateral spreading. Structure Design (SD) needs to perform the lateral pile analysis and GS will provide the necessary soil parameters for this analysis in the design stage.

The calculated axial geotechnical capacities of the 24-inch diameter CIDH piles at Abutments 1 and 3 are based on the skin friction only. Based on the subsurface information gathered at the site, the pile types proposed with the preliminary design loads provided by OBDN, 24” diameter CIDH piles at the Abutment 1 is acceptable. However, at Abutment 3, the pile length exceeds 30 times diameter when the pile type proposed with the preliminary design loads provided by OBDN are considered (see Tables 5 and 6 below). Therefore, a 36-inch diameter CIDH is recommended at Abutment 3. Revised pile type and loading may be updated after the request for the Foundation Report.

At Pier 2, 78-inch diameter permanent casing will be utilized up to 20’ below the cutoff elevation per information received from OBDN to facilitate the construction of the Type II shaft. The axial geotechnical capacities for 66-inch CIDH piles below the casing are based on the skin friction only. If a shallow pile tip elevation is desired, the diameter of the CIDH pile shaft and casing should be increased.

Table 5. Preliminary Foundation Recommendations

Support Location	Pile Type	Cut-off Elevation (ft)	Service-1 Limit State Load per Support (kips)		Total Permissible Support Settlement	Nominal Resistance (kips)				Design Tip Elevations (ft)	Specified Design Tip Elevation (ft)
						Strength Limit		Extreme Event			
			Total	Perm.		Comp. (ϕ_{qs}=0.7)	Tension (ϕ_{qs}=0.7)	Comp. (ϕ_{qs}=1)	Tension (ϕ_{qs}=1)		
Abut 1	24” dia. CIDH	+5.0	2440	1800	1”	390	0	190	0	-50.0 (a-I) -32.0 (a-II)	-50.0
Pier 2	78” dia. CIDH pile with casing (Type II Shaft) **	-2.0	8920	7090	1”	2660	0	1140	0	-93.0 (a-I) -61.0 (a-II)	-93.0
Abut 3	24” dia. CIDH	+5.5	2440	1800	1”	390	0	190	0	-74.0 (a-I) -68.0 (a-II)	-74.0

Notes:
1. Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event).
2. A detailed assessment of settlement was waived as piles are embedded adequately into the formation material.
3. The CIDH specified tip elevation shall not be raised.
4. Design tip elevation for Lateral Load is typically provided by Structure Design (SD).
5. Below the casing tip elevation, the CIDH pile diameter is 66”.

Table 6. Preliminary Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Pile Tip Elevations (ft)	Specified Pile Tip Elevation (ft)
		Compression	Tension		
Abut 1	24” dia. CIDH	390	0	-50.0 (a)	-50.0
Pier 2	78” dia. CIDH pile with casing (Type II Shaft) **	2660	0	-93.0 (a)	-93.0
Abut 3	24” dia. CIDH	390	0	-74.0 (a)	-74.0

Notes: 1. Design tip elevations are controlled by: (a) Compression.
2. A detailed assessment of settlement was waived as piles are embedded adequately into the formation material.
3. The CIDH specified tip elevation shall not be raised.
4. Pile Data Table needs to be updated by SD since design tip elevation for Lateral Load is typically provided by SD.
5. Below the casing tip elevation, the CIDH pile diameter is 66”.

Notes to Structure Designer

The permanent casing may be smooth-wall casing or a CMP. Neither structural capacity nor geotechnical resistance of the permanent casing was used in the design of the pile. The permanent casing may be placed in a drilled hole, and the annular space backfilled with grout.

The creek water and soil samples have been determined to be corrosive for foundation material.

The Structure Designer must show on the plans, in the pile data table, the minimum pile design tip elevation required to meet the lateral load demands. If the specified pile tip elevation required to meet lateral load demands is lower than the specified pile tip elevation given within this report, the Office of Geotechnical Design South, Branch A should be contacted for further evaluation.

Construction Considerations

- Due to surface water and shallow ground water, wet method is recommended for CIDH pile construction.
- Maintaining a positive head in temporary casing is necessary to prevent caving and water inflow.
- For work in the creek, construction of a cofferdam and /or seal course and diversion of the creek may be necessary.
- Due to the environmental restrictions, impact driving should not be used during construction.

- It is anticipated concrete placement for the CIDH piles will require slurry displacement method. The zones used to calculate the skin friction of the CIDH concrete piles are shown in Table 7.

Table 7. CIDH Concrete Pile Skin Friction Zone Elevations

Support Location	Skin Friction Zone Top Elevation (ft)	Skin Friction Zone Bottom Elevation (ft)
Abut 1	+0.0	-45.0
Pier 2	-22.0	-87.0
Abut 3	+0.5	-69.0

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, Faramarz Gerami at (213) 620-2149, or Chris Harris at (213) 620-2147.

Prepared by: Date: 2/15/19

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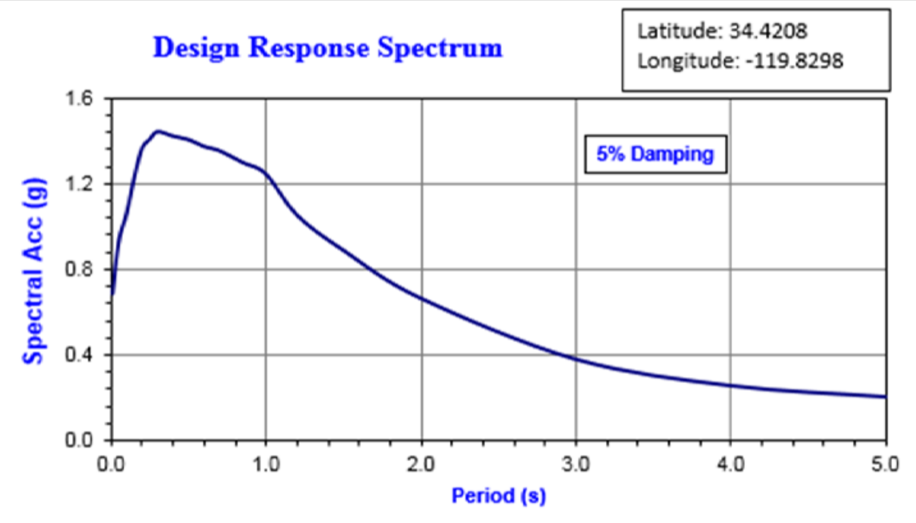
Appendices:
Appendix 1: Seismic Design Data
Appendix 2: Seismic Design Recommendations Report

Appendix 1: Seismic Design Data

Seismic Design Data for San Jose Creek Bridge *(Bridge No. 51-0217)*

Period (s)	Spectral Acceleration, Sa(g)
0.01	0.686
0.05	0.929
0.1	1.058
0.15	1.227
0.2	1.363
0.25	1.405
0.3	1.441
0.4	1.42
0.5	1.404
0.6	1.372
0.7	1.352
0.85	1.297
1	1.243
1.2	1.052
1.5	0.888
2	0.664
3	0.38
4	0.257
5	0.205



The Design Response Spectrum is the upper envelope of the deterministic and probabilistic response spectrum, but not less than the Minimum Deterministic Spectrum for California. The deterministic spectrum is obtained by using the average using the 2008 Campbell-Bozorgnia and the 2008 Chiou-Youngs ground motion prediction equations. Probabilistic response spectrum is obtained for 5 percent probability of exceedance in 50 years from the 2008 USGS Interactive

Seismic Loading Table (per MTD 1-47)

Soil Profile (VS30): 754 ft/s

Magnitude: **M** = 7.3

PGA: 0.7g

Design Response Spectrum

Appendix 2:

Lateral Spreading Analysis by Jeon Jongkoo (OGDPP, Geotechnical Services)

per Request by OGDS – Branch A

LATERAL SPREADING ANALYSIS

This section presents the results of a lateral spreading analysis performed for the San Jose Creek Bridge (Replace) projecct , in accordance with the procedures included in the following documents:

- MTD 20-15 (2017): Lateral Spreading Analysis for New and Existing Bridges
- Geotechnical Manual (2019): Example of A Geotechnical Lateral Spreading Analysis as per MTD 20-15 (2017), Under Preparation.

The evaluated design ground motion parameters corresponding to a 975-years return period, assuming $(V_s)_{30}$ = 230 m/sec, are:

- Design Horizontal Peak Ground Acceleration (Design HPGA) =0.73 g
- Moment Magnitude of the Casuative Earthquake, M_w =7.3

Based on the results of the liquefaction hazard evaluation, the upper most layer of subsurface soils predicted to liquefy due to the design ground motion at this bridge site occurs at Abutment 3. The depths from the top of the approach embankment at Abutment 3 to the top and bottom of this liquefied soil layers are approximately 10 and 20 feet, respectively. Liquefaction of this relatively shallow layer of soils is considered conductive to lateral spreading of the overlying abutment-soil-foundation system, which is also located on the bank of a creek. Subsurface soils at Abutment 1 and Pier 2 are not considered liquefiable due to existing dense to very dense materials. Therefore, grounds at Abutment 1 and Bent 2 are not considered susceptible to liquefaction-induced lateral spreading hazards. Based on this information, further analysis is necessary to evaluate liquefaction-induced lateral spreading hazards at Abutment 3.

Information presented in Table 1 on an idealized subsurface profile and recommended soil parameters was developed to perform a lateral spreading analysis at Abutment 3 as per MTD 20-15 (2017). This information was used to develop a representative geometric/cross section model of the Abutment 3, which is necessary to perform this lateral spreading analysis. As recommended in the MTD20-15, the SPT blow count based empirical correlation developed by Kramer and Wang (2015) was used to evaluate the residual undrained shear strength (S_r) of the liquefied soil.

Figure 1 shows the basic digital model of the Abutment 3 developed in the computer software SLOPE/W (Geo-Slope, 2014) based on the available topographic/geometric information, and the idealized soil profile and the soil parameters discussed above.

Table 1. Idealized Soil Profile with Assigned Soil Parameters for Lateral Spreading Analysis

Depth (ft)	Soil Description	Average Corrected SPT Blow Counts, $(N_1)_{60}$	Total Unit Weight, γ_t (pcf)	Shear Strength Parameters	
				Cohesion or Undrained Shear Strength, C_u or S_r (psf)	Friction Angle, ϕ (degrees)
0-10	Sandy Lean Clay (CL)	19	110	1100	0
10-20	Silty Sand (SM) (Liquefiable Soil Layer)	13	115	330	0
20-37	Silty Sand (SM)	55	120	0	36
37-70	Elastic Silt (MH) or Silt with Sand (ML)	50 or 14	115	3500 or 1000	0

Note: Groundwater level was assigned to a depth of 9 feet below existing ground surface.

The computer software SLOPE/W was used to performed Generalized Limit Equilibrium (GLE) based pseudo-static slope stability analyses as part of the lateral spreading analysis at Abutment 3.

This analysis includes several simplifying assumptions, including a wedge-shape potentially unstable soils soil mass with the bottom sliding failure surface located at the mid elevation of causative liquefiedsoil layer. It also specifies constraints on the horizontal limits of the potential sliding wedge or the failure surface as per MTD 20-15 when searching for the slip surface with the minimum pseudo-static factor of safety (FS) for a given seismic loading condition as represented by the parameter $k_{h,}$. The parameter k_h, termed as the coefficient of equivalent horizontal ground acceleration, is defined as, k_h= (Equivalent Horizontal Ground Acceleration)/g, where is the acceleration due to gravity.

Table 2 presents the pseudo-static factors of safety for different values of k_h without considering any pile lateral resistances. Results presented in Table 2 sand Figure 2 show that minimum FS = 3.6 for the case of k_h=0.0 and no pile lateral resistance (i.e,, R_{Tot} = 0.0 kips/ft). This indicates that the Abutment 3 is not susceptible to liquefaction-induced flow failure even if there is no addition lateral resistance offered by the pile foundations.

Table 2. Minimum FS for Different k_h values with R_{Tot} = 0.0 kips/ft

Coefficient of Horizontal Acceleration (k_h)	Minimum Factor of Safety
0.000	3.62
0.243	1.32
0.365	1.00
0.730	0.58

Results of the analysis performed for the case of k_h= k_{max} =(Design HPGA)/g and no pile lateral resistance (i.e,, R_{Tot} = 0.0 kips/ft) analysis are presented in Figure 3. A minimum FS of about 0.58 was obtained for this case. A FS<1.0 for this case indicates that, in the absence of any pile lateral resistance, Abutment 3 is susceptible to liquefaction-induced lateral spreading when subjected to the design ground motion.Based on the results presnented in Table 2 the estimated value of the coefficient of the equivalent yield horizontal ground acceleration, $(k_h)_y$, with no pile lateral resistancecontribution is about 0.365. This indicates that, in the absense of any pile lateral resistance, Abutment 3 is susceptible to lateral spreading for any value of the Design HPGA>0.365g. Since the Design HPGA is 0.73g for the subject bridge site, Abutment 3 is predicted to experience liquefaction-induced lateral spreading displacements.

Based on the above $(k_h)_y$ value of 0.365 and the correlation developed by Bray and Travasarou (2007) and included in MTD 20-15, in the absence of any pile lateral resistance, the estimated median lateral spreading displacement at Abutment 3 due the design seismic shaking (i.e., Design HGPA=0.73g) is about two (2) inches.

The above results also indicate that, in the presence of any pile lateral resistance, the liquefaction-induced lateral spreading displacement, if any, should be less than two (2) inches.

Based on additional analysis, it is estimated that lateral spreading is not likely to occur at Abutment 3 provided the proposed pile foundations at this support contribute to the lateral resistance of the sliding soil mass by an amount equal to, or greater than 33.3 kips/ft of abutment (i.e.., $R_{Tot} \geq 33.3$ kips/ft of abutment width).

REFERENCES

Caltrans (2017), Memos to Designer 20-15, *Lateral Spreading Analysis For New and Existing Bridges*

Kramer, S. and Wang, C.H., (2015). *Empirical Model for Estimation of the Residual Strength of Liquefied Soil*, Journal of Geotechnical and Geo-environmental Engineering.

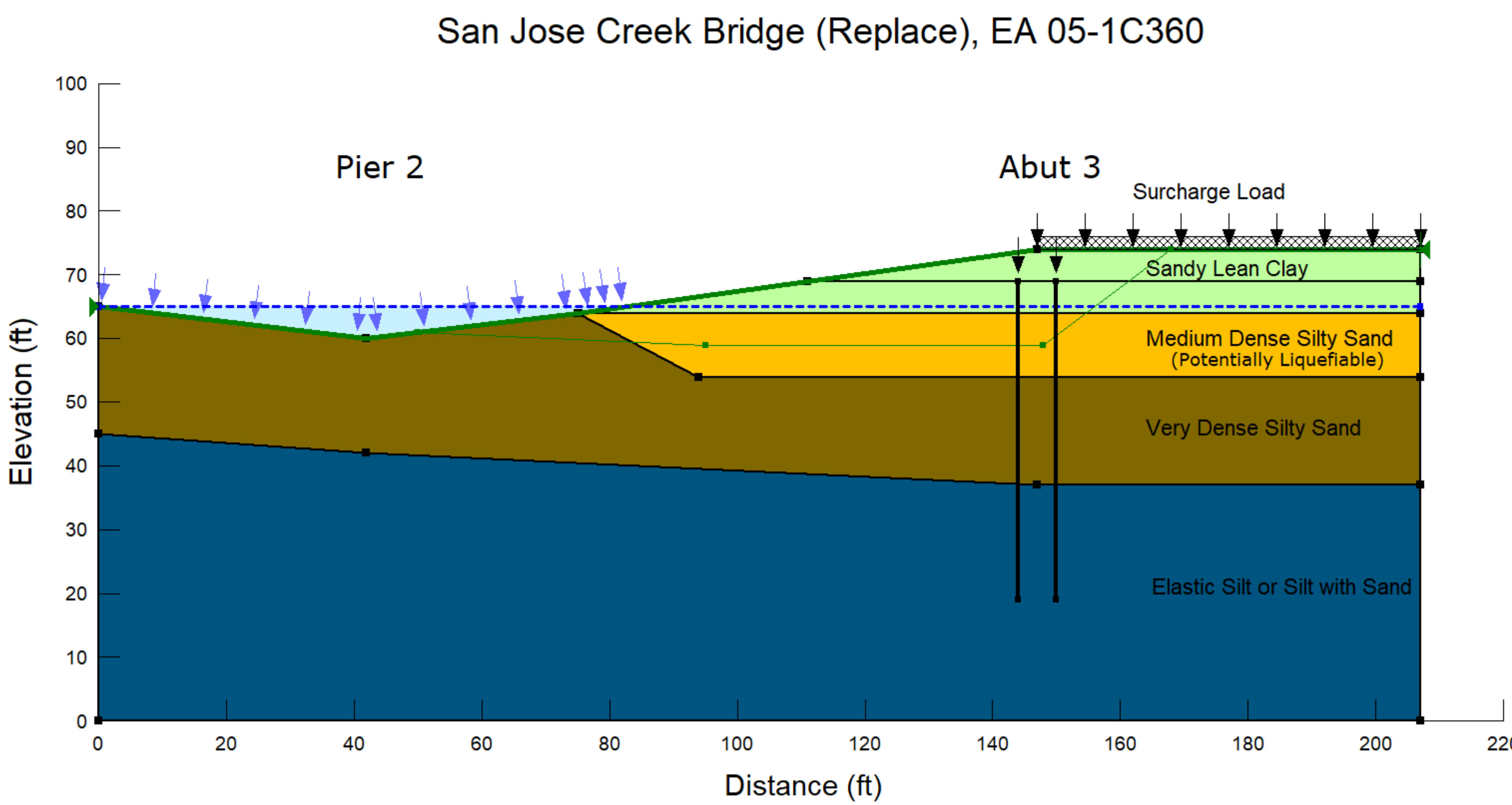


Figure 1. A Digital SLOPE/W Model for Pseudo-Static Slope Stability Analysis

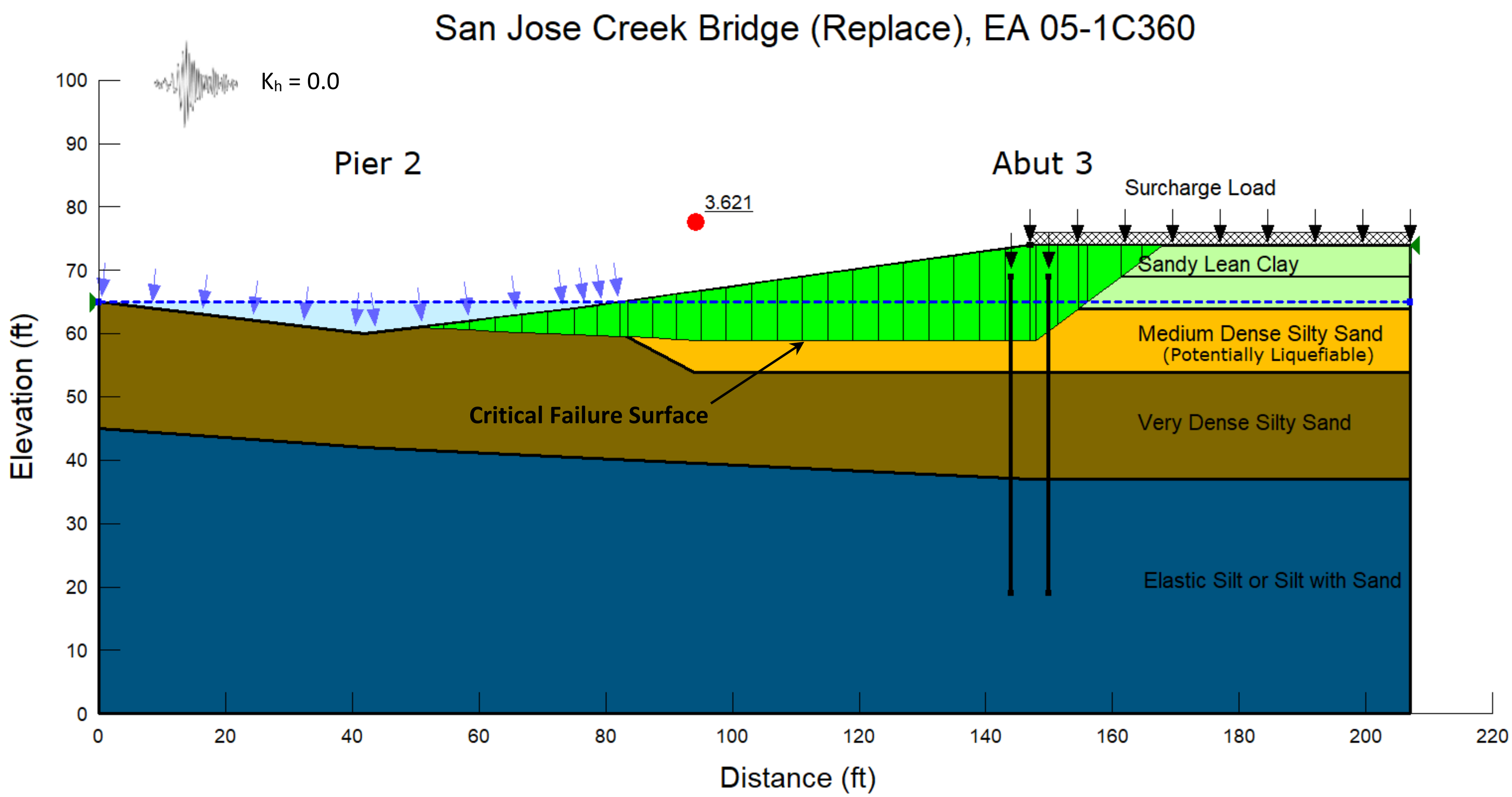


Figure 2. Results of the Pseudo-Static Slope Stability Analysis for R_{Tot} = 0 and K_h =0.0 g

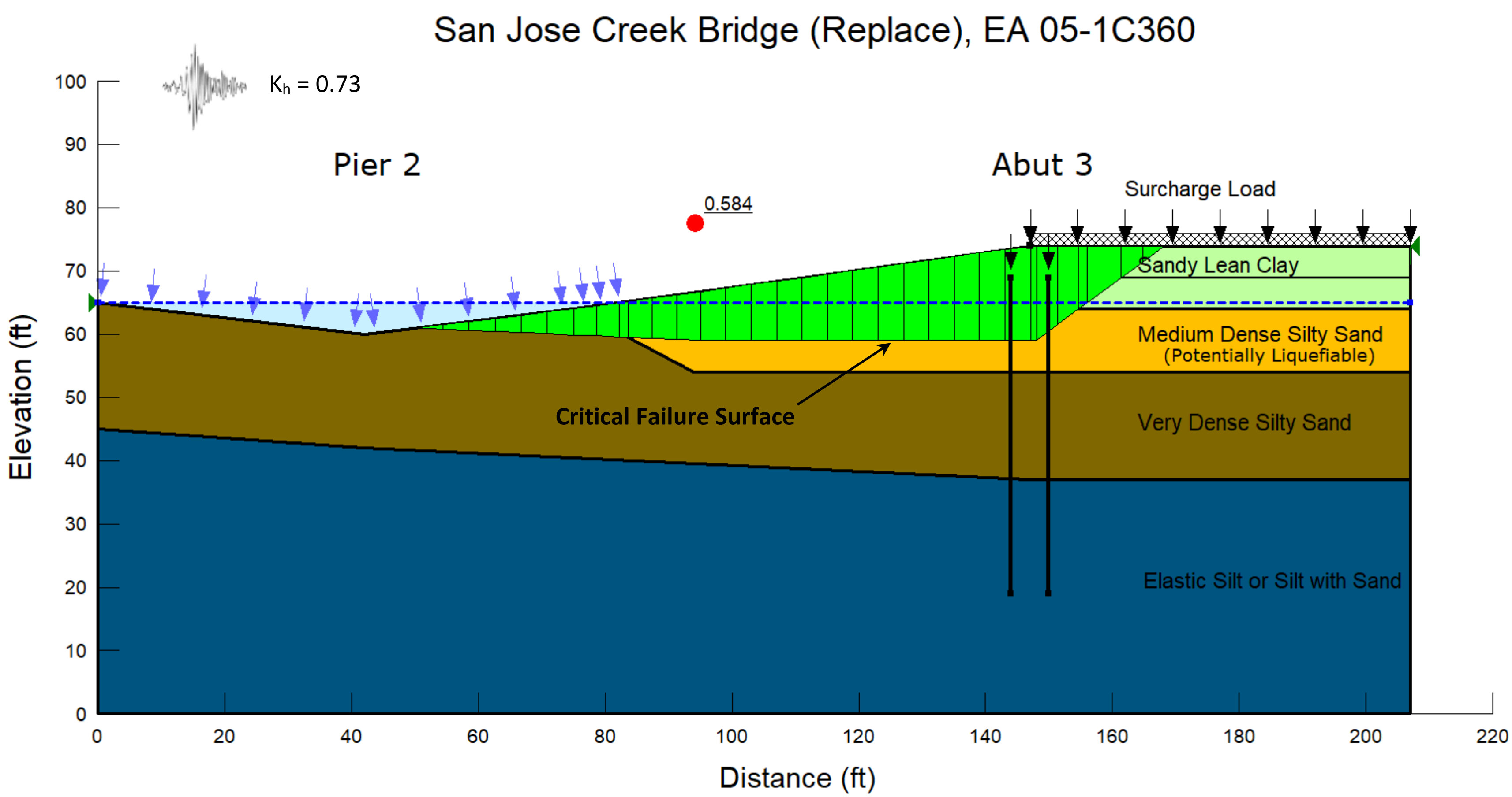


Figure 3. Results of the Pseudo-Static Slope Stability Analysis for R_{Tot} = 0 and K_h = (HPGA/g) =0.73