SUPPLEMENTAL BRIDGE FOUNDATION REPORT

Retrofit of the Swinging Bridge over Arroyo Grande Creek
Arroyo Grande, California

Yeh Project No.: 219-099

August 23, 2019

Prepared for:

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Attn: Mr. Martin Pohll

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Attn: Mr. Martin Pohll

Subject: Supplemental Geotechnical Report, Retrofit of the Swinging Bridge over Arroyo Grande Creek, Arroyo Grande, California

Dear Mr. Pohll:

Yeh and Associates, Inc. is pleased to submit this supplemental geotechnical report for the design of the Retrofit of the Swinging Bridge over Arroyo Grande Creek in Arroyo Grande, California. This report was prepared in accordance with our agreement for professional services with Quincy executed on May 27, 2019. This report presents the results of our limited geotechnical evaluation. The purpose of our geotechnical evaluation was to use existing subsurface data to estimate the axial capacity of new cast-in-drilled-hole (CIDH) piles proposed to retrofit the existing bridge abutments, estimate the capacity of existing CIDH piles supporting the cable anchorages, provide LPILE soil parameters for Quincy's analysis of laterally loaded piles, and to evaluate liquefaction hazards at the site and the need to consider down drag, seismic settlement or lateral spreading in the foundation design. Our geotechnical evaluation consisted of analysis based on available subsurface data presented in Fugro Consultants' (2017) *Draft Geotechnical Engineering Report* and an Earth Systems Pacific (2014) report. A summary of geotechnical considerations for design of the bridge retrofit are as follows:

- The subsurface conditions encountered in the previous borings generally consisted of approximately 19 to 20 feet of alluvium overlying Paso Robles Formation. The alluvium consisted of soft to stiff lean clay with varying amounts of sand (CL) as well as medium dense to very dense sand with varying amounts of silt, clay, and gravel (SP-SC, SM, SC, SC-SM). The Paso Robles Formation was encountered below the alluvium to depths up to 41.5 feet below the bridge approach elevation. The Paso Robles Formation generally consisted of medium dense to very dense sand with varying amounts of silt, clay and gravel (SW, SP-SM, SC) as well as stiff sandy lean clay (CL).
- Arroyo Grande Creek flows southwest below the bridge. The 65-percent plans (Quincy n.d.) show the water surface approximately 27.5 feet below the bridge deck elevation. We understand scour is not a consideration for design. Groundwater was encountered in the

previous borings drilled in the abutment areas at depths ranging from 31 to 40 feet below the ground surface.

- This report provides foundation design recommendations for the retrofit of the abutments. The 65-percent design (Quincy n.d.) shows that the retrofit will consist of providing additional foundation support at the abutments using new 16-inch diameter cast-in-drilled-hole (CIDH) piles. The design will also consider the axial and lateral capacities of existing 24-inch diameter CIDH piles that support the existing buried cable anchorages behind the abutments. The design will only consider normal service loads and extreme event limit states (e.g. seismic loading) will not be considered for the design. Yeh was requested to evaluate the potential for liquefaction of the foundation soil encountered and the need to consider down drag, seismic settlement or lateral spreading in the foundation design.
- The design earthquake for Yeh's liquefaction analysis was estimated using the latest version of ARS Online, V2.3.09. The design earthquake for the liquefaction analysis is a M6.9 event occurring on the Los Osos 2011 Fault, mapped approximately 4.0 miles northeast from the project site, resulting in an estimated peak ground acceleration of approximately 0.66g. The Paso Robles Formation is not typically considered susceptible to liquefaction. The relative density of a unit encountered within the Paso Robles Formation in Fugro's (2017) boring DH-4 would be considered potentially liquefiable based on the NCEER Guidelines (Youd et al. 2001). However, our characterization of liquefaction potential is based on one blowcount from a boring drilled using methods that are typically not preferable for assessing liquefaction potential. The proposed retrofit is anticipated to result in a "no better, no worse" site condition, where the potential for liquefaction is not estimated to decrease or increase as a result of retrofitting the bridge.

We appreciate the opportunity to be of service. Please contact Gresh Eckrich at 805-481-9590 or geckrich@yeh-eng.com if you have questions or require additional information.

Sincerely,
YEH AND ASSOCIATES, INC.

Jamie L. Cravens
Staff Engineer

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1. Existing Facility

The existing bridge is a suspension bridge consisting of a timber deck carried by timber beams suspended on ¾-inch steel cables. The pair of main support cables pass over the support towers located on either end of the bridge and attach to buried cable anchorages located approximately 50 feet behind both towers. The Schott (n.d.) plans show the buried cable anchorages consist of two piles spaced approximately 12 feet apart parallel to the bridge centerline. The piles for the anchorages are 2-foot diameter, 12-feet long, and connected by a 2-foot thick tie beam buried approximately 2 feet below the ground surface (Schott n.d.). Additional lateral support for the bridge is provided by lateral wind/sway cables anchored into the creek bank.

Based on field measurements by Fugro (2017), the tower at the south end of the bridge is supported by a concrete abutment wall approximately 12 feet long, varying between 18 inches to 3 feet high, and varying between 13 to 21 inches thick. The tower at the north end of the bridge is supported by a concrete abutment wall approximately 12 feet long, 7 feet high, and 9.5 inches thick (Fugro 2017). Arroyo Grande Creel flows westerly beneath the existing bridge within an incised channel. The creek thalweg is approximately 35 feet below the approaches to the bridge. The slope grades below the bridge vary from vertical to approximately 25 percent (Quincy n.d.).

2. Proposed Improvements

The retrofit of the bridge will provide additional foundation support for the abutments. The new foundations will consist of two 16-inch diameter cast-in-drilled-hole (CIDH) piles connected by a 3-foot deep pile cap across the abutment. The retrofit will be constructed on the creek side of the existing abutment wall and will be doweled into the existing abutments. The two retrofit piles will have a center-to-center spacing of approximately 8 feet and will be constructed on an approximately 1h:1v (horizontal:vertical) slope, as shown on the 65-percent plans (Quincy n.d.). The retrofit design will consider the estimated axial and lateral capacities of the existing buried cable anchorages.

The retrofit could also include replacing the existing lateral wind/sway cables to provide more horizontal-direction lateral support. The scope of this evaluation does not include design recommendations for the lateral cables. Fugro (2017) provided recommendations for the design of helical anchors for lateral support.

3. Subsurface Conditions

Fugro (2017) excavated two hand auger borings and drilled two hollow stem auger borings at the site to depths of approximately 17 to 41.5 feet below the ground surface. Additionally, Fugro (2017) presented data from previous explorations performed by Earth Systems Pacific (2014). Earth Systems drilled three hollow stem auger borings on Short Street, just south of the south abutment, to depths of approximately 26.5 to 36.5 feet below the road surface. The approaches to the bridge were underlain by approximately 19 to 20 feet of alluvium overlying Paso Robles Formation. The alluvium consisted of soft to stiff lean clay with varying amounts of sand (CL) as well as medium dense to very dense sand with varying amounts of silt, clay, and gravel (SP-SC, SM, SC, SC-SM). The Paso Robles Formation was encountered below the alluvium to depths up to 41.5 feet below the bridge approach elevation. The Paso Robles Formation generally consisted of medium dense to very dense sand with varying amounts of silt, clay and gravel (SW, SP-SM, SC) as well as stiff sandy lean clay (CL).

4. Geotechnical Recommendations

4.1 Geotechnical Properties

Table 1 presents a summary of the subsurface profile and predominant soil units interpreted from the previous design report (Fugro 2017) and used in Yeh's geotechnical analyses. Geotechnical engineering properties were assigned to various soil layers within the profile for use in evaluating the axial load capacity of CIDH pile foundations for the retrofitted bridge. The soil unit weights and shear strength parameters were estimated based on measured blow counts and the results of laboratory testing of selected samples presented in the Fugro (2017) report. A groundwater level of 27.5 feet below the ground surface was used for the analyses for the north and south abutments and cable anchorages. We understand Quincy will perform lateral analyses of the piles using the computer program LPILE. Recommended input parameters for LPILE are provided in Table 1.

Table 1: Geotechnical Properties for the North and South Abutments and Cable Anchorages

Geologic Unit	Depth Below Bridge Deck Elevation (ft)	General Material Description	SHAFT Model	LPile Model	Unit Weight (pcf)	Friction Angle (degrees)	ε_{50}[1] (in/in)	k (pci)
Qal_1[2]	0-4	Soft to stiff lean CLAY with varying amounts of SAND (CL)	--	--	--	--	--	--
Qal_2	4-20	Medium dense to very dense poorly graded SAND with varying amounts of SILT and CLAY (SP, SP-SC, SM, SC, SC-SM)	FHWA Sand	API Sand	119	36	--	150
QTp	>20	Medium dense to very dense, poorly to well-graded SAND with varying amounts of SILT, CLAY, AND GRAVEL (SP-SM, SP-SC, SW, SC, CL)	FHWA Sand	API Sand	120	38	--	200

4.2 Liquefaction Analysis

A screening level analysis was performed to evaluate the liquefaction potential of the soil encountered. Liquefaction is the loss of soil strength due to an increase in soil porewater pressure resulting from seismic ground shaking. Liquefaction typically occurs in loose to medium dense granular soil that is below the groundwater table. The extent and severity of liquefaction is dependent upon the intensity and duration of the strong ground motion. Additionally, seismic settlement can occur in loose to medium dense granular soil that is above the water table (and therefore not susceptible to liquefaction).

The design earthquake for Yeh's liquefaction analysis was estimated using the latest version of ARS Online, V2.3.09. Site coordinates were estimated as 35.1233° latitude and -120.5770° longitude. The shear wave velocity for the site was estimated to be approximately 265 meters per second, corresponding to site class D for stiff soil conditions. The shear wave velocity estimate is based on subsurface exploration field blow counts and classifications in conjunction with Caltrans' *Seismic Design Criteria* (Caltrans 2013) and Caltrans' *Methodology for Developing Design Response Spectrum for use in Seismic Design Recommendations* (Caltrans 2012b) for

[1] ε_{50} *is the strain corresponding to 50 percent of the failure stress.* k *is a constant relating to the stiffness of a soil versus displacement.*

[2] Qal_1 layer above cutoff elevation and not used in SHAFT analysis for axial capacity. Not recommended for consideration in lateral pile analysis.

estimation of shear wave velocity. The design earthquake estimated from ARS Online for the liquefaction analysis is a M6.9 earthquake occurring on the Los Osos 2011 Fault and resulting in an estimated peak ground acceleration of approximately 0.66g at the project site. The analysis assumed a groundwater depth of approximately 27.5 feet below the ground surface, corresponding to the highest groundwater elevation measured in previous borings drilled for the project (Fugro 2017).

The potential for liquefaction and seismic settlement to occur with the foundation support soil encountered was evaluated using NCEER Guidelines (Youd et al. 2001). The foundation support soil within the alluvium and Paso Robles Formation consisted of soft to stiff clay and medium dense to very dense sand. The Paso Robles Formation is generally composed of uncemented units of stiff to hard cohesive soil and medium dense to very dense granular soil that are not typically considered susceptible to liquefaction. Liquefaction resistance increases markedly with age (Youd et al. 2001), and sediments deposits within the past few thousand years are generally much more susceptible to liquefaction than sediments deposited during the Pleistocene age (10,000 to 2.6 million years ago) and Pliocene age (2.6 to 5.3 million years ago). Therefore, the susceptibility of the Pliocene- to Pleistocene-age Paso Robles Formation to liquefaction is generally considered low.

The proposed retrofit is anticipated to result in a “no better, no worse” site condition, where the potential for liquefaction is not estimated to decrease or increase as a result of retrofitting the bridge. The relative density of a layer of medium dense poorly graded sand with silt encountered within the Paso Robles Formation at a depth of approximately 40 feet in Fugro’s (2017) boring DH-4 would be considered potentially liquefiable based on the NCEER Guidelines (Youd et al. 2001). The potentially liquefiable soil unit was encountered near the maximum explored depth of the boring, and above the creek invert elevation. The thickness and extent of the layer is not known based on the one boring; however, there may be a potential for slope instability or lateral spreading of the creek banks associated with liquefaction. As noted above, it would be atypical for Paso Robles Formation to be considered vulnerable to liquefaction.

It should be noted the analysis was based on limited data that may not be representative of the Paso Robles Formation. This study’s characterization of liquefaction potential is based on one blowcount from a boring drilled using methods that are typically not preferable for assessing liquefaction potential. Additional analyses to better characterize the potential for liquefaction and associated impacts should be based on additional subsurface data collected using typical methods for assessing liquefaction (e.g., mud rotary drilling or cone penetrometer testing [CPT]).

If liquefaction were to occur, the estimated seismic settlement within the potentially liquefiable layer is approximately ¼ inch or less. The estimated settlement could occur below the north or south abutments and anchorages in association with liquefaction. There is a low potential for liquefaction to result in down-drag forces on the piles based on our screening level analysis of the limited data evaluated for this study. The potentially liquefiable units were encountered below the existing anchorage pile tip elevations and the anticipated tip elevations for the retrofit piles; therefore, there is a low potential for lateral spreading associated with liquefaction to result in lateral loading of the piles. No special geotechnical recommendations to mitigate liquefaction are recommended for the bridge design based on the analysis performed for this study.

4.3 Foundation Design

The proposed bridge abutment retrofit can be supported on CIDH pile foundations bearing in the alluvium and Paso Robles Formation. The recommendations for the design of deep foundations considered pile dimensions and cutoff elevations provided in Quincy's 65-percent plans (n.d.). Yeh estimated the axial resistances of the CIDH piles according to the latest Caltrans amendments to the AASHTO Bridge Design Specifications and considered the following conditions:

- A resistance factor of 0.7 was used in calculating the factored nominal axial compression and uplift resistance of the piles for skin friction for the Strength Limit State.
- Scour was not considered for design, and the factored nominal axial resistances for the Strength Limit State do not include the effects of scour.
- Cutoff elevations of approximately 4 feet below the ground surface elevation were assumed for the abutments and existing cable anchorages based on the Quincy (n.d.) 65-percent abutment plans and Schott (n.d.) cable anchorage as-built plans.

Pile Design. The additional foundation support for the abutments will be provided by new 16-inch diameter CIDH piles. The new piles can bear in either the alluvium or Paso Robles Formation. The existing cable anchorages are supported on CIDH piles bearing in the alluvium. The axial capacity of CIDH piles was estimated based on the frictional side resistance using the computer program SHAFT (Ensoft 2017). End bearing resistance as well as the skin friction resistance within the upper 5 feet of the pile was neglected. Pile group efficiency factors for axial resistance were not applied, assuming the center-to-center pile spacing is greater than two times the diameter, the pile cap is in intimate contact with the ground and that the soil at the pile cap elevation is medium dense or denser, and scour is not anticipated below the pile cap. The estimated factored nominal axial resistances for tension and compression of the abutment and cable anchorages for the Strength Limit State are provided in Figure 1. The depth shown in the figure represents the pile depth below the existing ground surface.

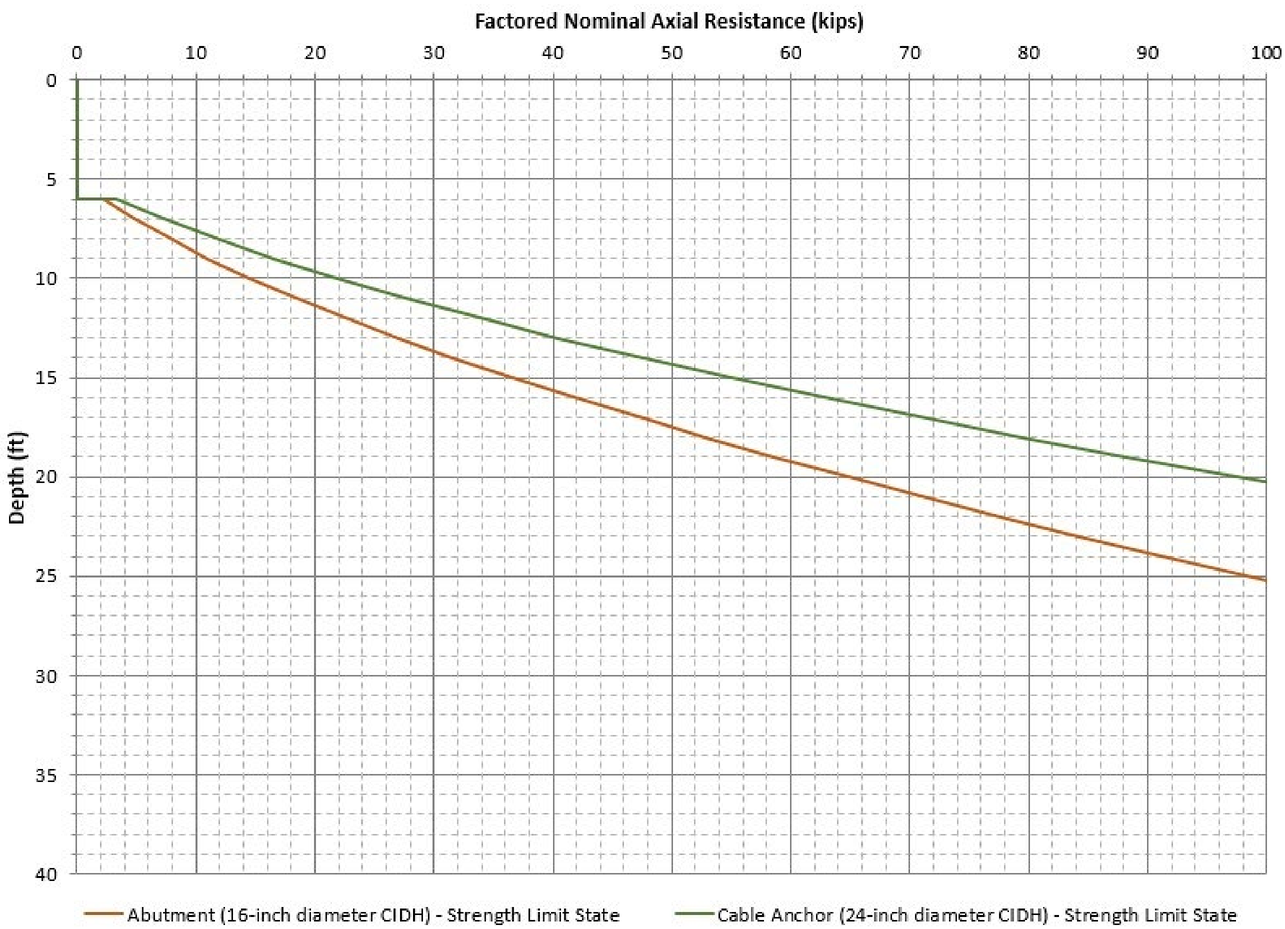

Figure 1: Factored Nominal Axial Resistance Versus Depth for the Abutment and Cable Anchorage Piles

P-Y Analyses. We understand that the lateral load resistance of the piles will be estimated by Quincy, and Quincy will use a soil resistance-pile deflection model (p-y analysis) and the computer program LPile by Ensoft. The depth of the retrofit CIDH piles into the alluvium and Paso Robles Formation for lateral loads resulting from the Service I Limit states should be estimated using the nominal soil resistance (or the factored nominal resistance with a resistance factor of 1.0), and by multiplying the estimated critical pile embedment by 1.3 to essentially fix the pile within the alluvium and Paso Robles Formation. Piles should be designed to tolerate the estimated bending moments and shear forces for the range of deflections being considered, and should consider lateral group effects (if needed) and sloping ground conditions. The recommended geotechnical input parameters and material models for use with LPile are presented in Section 4.1.

Group Effects and Sloping Ground. Group effects result from shadowing of piles when the direction of the lateral load is coincident with the alignment of piles within a pile group. Without group effects, the total lateral capacity of the pile group would be estimated as sum of the individual capacities of the piles for a given lateral deflection. The group efficiency is the percentage of the sum of individual pile capacities that can be used for design considering the shadowing effects of all the piles within the group. P-reduction factors should be applied in the analysis to estimate the group efficiency of the pile group considering lateral loading. P-reduction factors were selected from the Caltrans amendments to the AASHTO LRFD *Bridge Design Specifications* for the appropriate pile spacings and direction of loading. P-reduction factors for pile spacings at the abutments and cable anchorage piles are presented in Table 2.

Table 2: P-reduction Factors for Lateral Loading due to Pile Spacing

	Loading Direction Perpendicular to Bridge Centerline		Loading Direction Parallel to Bridge Centerline	
Support Location	Pile Spacing[3]	P-Reduction Factors, P_m	Pile Spacing	P-Reduction Factors, P_m
Abutments	6.0B	Row 1: $P_m = 1.0$ Row 2: $P_m = 0.93$	6.0B (or >4B)	$P_m = 1.0$[4]
Cable Anchorages	(only 1 row)	$P_m = 1.0$	6.0B	Row 1: $P_m = 1.0$ Row 2: $P_m = 0.93$

[3] Pile spacing based on Quincy (n.d.) 65% plans for abutments and Schott (n.d.) As-Built plans for cable anchorages. P-reduction factors will vary with pile spacing and should be adjusted if pile spacing is increased or decreased.
[4] See Table 3 for additional p-reduction factors for sloping ground conditions.

In addition, p-reduction factors should be applied in the analysis of the abutment retrofit piles to account for strength reductions due to adjacent sloping ground, where loading is parallel to the bridge centerline. The p-reduction factors for pile spacing (see Table 2) should be multiplied by the p-reduction factors for sloping ground conditions at the abutments (see Table 3).

Table 3: P-reduction Factors for Lateral Loading due to Sloping Ground at Abutments

Depth Along Pile (feet)[5]	P-Reduction Factors, P_m
0 – 5	$P_m = 0.3$
5 – 13	$P_m = 0.4$
13 - ?	$P_m = 1.0$

5. LIMITATIONS

This study has been conducted in general accordance with currently accepted geotechnical practices in this area for use by the client for design purposes. The conclusions and recommendations submitted in this report using geotechnical data presented in the Fugro (2017) report. If there are any changes in the project or site conditions, Yeh should review those changes and provide additional recommendations, if needed. Any modifications to the recommendations of this report or approval of changes made to the project should not be considered valid unless they are made in writing. The report and drawings contained in this report are intended for design-input; and are not intended to act as construction drawings or specifications.

Site conditions will vary between points of observation or sampling, seasonally, and with time. The nature and extent of subsurface variations across the site may not become evident until excavation is performed. If during construction, fill, soil, or water conditions appear to be

[5] Assumes 16-inch pile diameters at abutments. P-reduction factor depths will vary with pile diameter and should be adjusted if pile diameter is increased or decreased.

different from those described herein, Yeh should be advised and provided the opportunity to evaluate those conditions and provide additional recommendations, if necessary. The geotechnical professional should observe portions of the construction and site conditions, such as excavations, exposed subgrades and earthwork, to evaluate whether or not the conditions encountered are consistent with those assumed for design, and to provide additional recommendations during construction, if needed.

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