State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. RAMIN RASHEDI
Division of Structure Design
Design Office 59-232

Attention Mr. Gary Blakesley

Date: May 5, 2000

File: 11-SD-5-KP 48.99
11-0301U1

Sorrento Viaduct (Widen)
Bridge No. 57-0513R/L

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Division of Structural Foundations - MS 5
Office of Structure Foundations

Subject: Revised Foundation Recommendations

Introduction

The proposed Sorrento Viaduct (Br. No. 57-0513R/L) Right and Left Widenings (both outside widenings and inside sliver widenings) are part of planned Rte. 5/805 Freeway improvements for the San Diego area. A request for Final Foundation Recommendations, dated October 22, 1998, for the subject bridge widenings was submitted to the Office of Structure Foundations (OSF) by Mr. Rashedi. Site specific ARS, liquefaction potential, and methods of liquefaction mitigation were requested in the above memorandum. A list of preliminary column/pile loads and shaft diameters were provided to OSF by Mr. Rashedi (dated December 18, 1998). As the 5/805 and 5/56 project has progressed, further revisions of the above pile load and shaft diameter list was sent to OSF including Revision 1 (dated February 24, 1999), Revision 2 (dated April 9, 1999), and Revision 3 (dated May 11 and 26, 1999). Final bent pile diameters were confirmed by Mr. Rashedi (personal communication, September 1999). Abutment pile diameters and axial service loads were provided by Mr. Rashedi (February 1, 2000). Mr. Gary Blakesley provided final bottom of footing/pile cutoff elevations for the proposed widenings (Caltrans facsimile copy, dated March 24, 2000) and requested p-y curves or COM624 soil profile information at the abutments. P-Y curves were also requested by Mr. Earl Seaberg on February 24, 1999. In the same memorandum it was mentioned that in order to mitigate the effects of potential liquefaction, large diameter cast-in-drilled-hole (CIDH) piles would be used for structures at the 5/805 Interchange (Seaberg, February 24, 1999). In preliminary evaluations of the As Built Log of Test Borings (LOTBs) performed by the Office of Geotechnical Earthquake Engineering (Perez-Cobo and Abghari, February 10 and April 7, 1999), potentially liquefiable soils are relatively thick at the site [estimated approximately as 12 m (40 ft) thick]. On February 3, 1995, Mr. Sharid Amiri also provided preliminary liquefaction analyses in this area based mostly on the As Built LOTBs. As Built pile capacities were also provided within the same Memorandums for each bridge (Left and Right).

Subsurface information was obtained by OSF drilling 22 - 94 mm diameter mud rotary borings which also involved extensive coring. Several 25 mm (1 in) diameter soil tube tests were attempted at the proposed Bent 2 Left and Right side widening for the Right Bridge, but could not penetrate the embankment fill with boulders/cobbles exposed at the surface and buried just beneath the surface. Results from the field studies will be shown on the LOTBs. In addition to the recent field work, the As Built LOTBs for the Sorrento Valley Overhead (Contract No. 11-022454, signed July 20, 1964), contained additional site and subsurface information and will be included within the new contract plans.

Site Description

The existing abutments are dominantly founded in approach embankment fill material which ranges between approximately 12.80 and 14.94 m (42 to 49 ft) thick for the southern abutments (Abutment 1 for both the Right and Left Bridges) and 13.72 to 14.94 m (45 to 49 ft) thick for the northern abutments (Abutments 9 Left and 8 Right). Within the Sorrento Valley area artificial fill commonly ranges from 0 to 3.66 m (0 to 12 ft) thick. Underlying native material ranges from 1.52 to 16.46 m (5 to 54 ft) thick for the Left Bridge and from 4.27 to 18.29 m (14 to 60 ft) thick for the Right Bridge, and native material thins to the south towards the hillside. The top surface of the underlying Eocene Ardath Shale slopes to the north away from the Abutment 1 widen areas. At the Abutment 1 areas the top rock surface varies from approximately elevation +12.50 to +6.71 m (+41 to +22 ft) from west to east, respectively. The rock surface generally drops down to elevations ranging between -7.01 to -8.23 m (-23 to - 27 ft) near the Bent 5 and 6 widenings and also remains generally low to the north near the Abutment 8 and 9 areas -7.32 to -7.59 m (-24 to -24.9 ft). Locally, bedrock is encountered at shallow depth in the area of the Left Bridge - Left side widening near Bents 7 and 8 at elevations ranging from +0.94 to +0.09 m (+3.1 to +0.3 ft). Approach embankment fill material consists dominantly of stiff to hard, sandy lean clay with scattered gravel and cobbles (up to 150 mm diameter, composed mostly of soft mudstone and minor hard metavolcanic rock fragments) interlayered with medium dense to dense, silty sand with gravel and clayey gravel. Artificial fill within the valley generally consists of medium dense to loose/soft to very stiff, silty sand with scattered gravel and cobbles interlayered with clayey sand, sandy lean clay, and clayey gravel. Native material [mapped as Holocene alluvium and slope wash undifferentiated according to Kennedy (1975)] and probably including some older alluvium at depth, consists of dominantly very loose to minor dense/dominantly soft to firm and minor hard, sandy silt interbedded with clayey sand, silty sand, sandy lean clay with and without gravel and cobbles, elastic silt, and cobble/gravel (sporadic dense gravel/cobble unit generally directly overlies bedrock). Much of the loose and soft native material is considered potentially liquefiable and is being investigated by the Office of Geotechnical Earthquake Engineering (OGEE) for potential mitigation measures or adequacy of proposed mitigation measures. As mentioned earlier, final p-y (lateral resistance) curves are also being developed for use at proposed bridge support locations. The underlying Eocene Ardath Shale generally consists of interbedded very soft to moderately hard, mudstone, claystone, siltstone, and minor sandstone and rare conglomerate. The formation is often intensely weathered and very soft to soft in the upper 2.74 to 7.92 m (9 to 26 ft) with weak rock unconfined compressive strengths ranging from 120 to 180 psi. Below the upper weathered zone, generally soft to moderately hard formation (fairly strong rock) shows unconfined compressive strengths from at least 250 to 300 psi and higher. The two deepest borings for the bridge widenings, Boring 99-3 and 99-4 (drilled for the Left Bridge, Right sliver widen), were 47.24 to 48.25 m (155 to 158.3 ft) below the surface [elevations -37.92 to -37.06 m (-124.4 and -121.6 ft)], respectively. Downhole P-S logging (compression and shear wave) showed that the better quality formational mudstones had shear wave velocities greater than 457 meters per second (1500 fps) which appeared to correlate with unconfined compressive strengths of at least 250 psi and higher. Shear wave velocities ranging from 549 to 762 meters per second (1800 to 2500 fps) in pseudo-rock-like material correlated with unconfined compressive strengths of at least 300 psi and higher in the Ardath Shale below approximate elevation -21.95 m (-72 ft). The LOTBs should be reviewed for more specific details.

Ground Water

Static ground water was last measured on January 11, 2000, within Boring 99-1(near Bent 8, left side - left bridge widen) at elevation +7.77 m (+25.5 ft). The water level within Boring 99-1 did not vary more than 0.03 m (0.1 ft) repeatedly measured over 4 months time.

The As Built LOTB shows ground water was encountered from approximate elevations +6.10 to +4.11 m (+20 to +13.5 ft) based on the City of San Diego datum, which requires a +2.44 m (+8.00 ft) add (Schuh, Caltrans E-mail and Memorandum, February 14 and March 7, 2000) to adjust to the current metric datum (NAVD 88) upon which the recent plans and boring program are based. The adjusted to metric As Built elevations would then show ground water was encountered at elevations +8.53 to +6.55 m (+28.0 to +21.5 ft) for the earlier foundation investigation, with measurements taken in either April or August, 1962.

Seismicity

See the memorandum (dated February 10, 1999) concerning Preliminary Seismic Design Recommendations sent to Mr. Earl Seaberg from Mr. Angel Perez-Cobo and Dr. Abbas Abghari. Final Seismic Design Recommendations and Lateral Resistance, p-y Curves will be submitted by the OGEE.

As mentioned above (Perez-Cobo and Abghari, February 10, 1999) the proposed "structures are located approximately 5 km from the Newport-Inglewood-Rose Canyon fault which has a maximum credible earthquake moment magnitude of M=7.0 and based on the Caltrans California Seismic Hazard Map (Mualchin, 1995), these structures are within the peak horizontal bedrock acceleration zone of 0.5 g."

As mentioned above, approximate depth to pseudo-rock-like material [Vs greater than 549 to 762 meters per second (1800 to 2500 fps)] occurs below approximate elevation -21.95 m (-72 ft).

Liquefaction

Liquefaction potential is considered moderate to high. Quaternary alluvium at the site is dominantly composed of loose to medium dense/soft to firm, sandy silt interbedded with clayey sand, silty sand, and sandy lean clay. Ground water is also rather shallow [measured within the recent investigation at 3.44 m (11.3 ft) below the surface]. As mentioned above, liquefaction potential is being determined by the OGEE.

Foundation Recommendations

The following recommendations are based on the Sorrento Valley Viaduct (Right and Left Widen) General Plan (revised March 26, 1999), Structure Plan No. 1 and 2 (revised October 1 and March 26, 1999), and Foundation Plans (revised July 1998), the above mentioned memorandums and personal communications from Mr. Rashedi (Caltrans facsimile copy dated May 26, 1999, personal communications regarding pile loads and pile diameter, September 1999, and a memorandum supplying abutment pile diameters and service loads, February 1, 2000), Mr. Blakesley (Caltrans facsimile copy with final bottom of footing elevations, dated March 24, 2000), and discussions with Mr. Ron Jones.

For the inside sliver widenings, sliver fills can be placed in accordance with Section 19-6 of the Standard Specifications. End dumping is not permitted. In the Abutment 1 area, any settlement due to the addition of the sliver fill [2.6 m (8.5 ft) widenings should be negligible in the foundation soils as existing embankment has been in place since 1965 and added load to the existing embankment will be minor. At the more significant Left Bridge - Left Widen, approximately 8.53 m (28 ft) of fill will be added with a calculated maximum settlement (Hough's Method) of 97 mm (3.8 in). The settlement period is estimated at approximately 60 days, however the actual settlement period will be determined by the project engineer on the basis of settlement data in the field. For the Right Bridge - Right Widen, approximately 2.4 m (8 ft) of fill will be added with a calculated maximum settlement (Hough's Method) of 56 mm (2.2 in).

For the inside sliver widenings in the area of Abutment 8 and Abutment 9, settlement should also be negligible with an estimated 1.2 to 1.5 m (4 to 5 ft) height of fill added. For the proposed Abutment 8 – Right Widen, additional fill is estimated at 8.53 m (28 ft) maximum height. Estimated settlement was calculated at approximately 305 mm (12 in). For the Abutment 9 – Left Widen, additional fill is estimated at 4.88 m (16 ft) maximum height. Settlement was calculated at 226 mm (8.9 in). Again, all fills can be placed in accordance with the Section 19-6 of the Standard Specifications. OSF recommends a fill settlement of up to 180 days for the outside widening in this area; however, the actual settlement period will be determined by the project engineer on the basis of settlement data in the field. Both outside widenings will have Retaining Walls extending from the abutments to the north to prevent encroachment of embankment upon local businesses.

Due to high fills OSF assumes that structure approach slabs will be incorporated within all the widenings but approach slabs were not denoted on available plans.

Plumb, 1.2m (4 ft) diameter, Cast-in-Drilled-Hole (CIDH) Piles can be used to support the bridge abutment widenings. Plumb, 2.4 m (8 ft) diameter drilled shafts will be used at the bent supports as shown below. CIDH pile capacities were calculated using the Federal Highway Administration's Drilled Shaft Manual (Pub. No. FHWA-HI-88-042) published July 1988. Permanent casing is recommended to be placed into bedrock to facilitate construction of the drilled shafts, prevent caving of loose soils and gravel/cobble lenses into the pile borings, and seal off ground water from entering the pile borings. OSF feels that permanent steel casing can be emplaced at least near specified tip elevation using a vibratory hammer. In discussions between Mr. Ron Jones and the author (March and April, 2000) the practice of drilling ahead of the casing before dropping the casing into place is considered undesireable as caving of loose soils and gravel/cobble lenses would create voids between the casing and surrounding soil, thus compromising the lateral capacity of the pile. However, drilling slightly ahead of casing in the basal gravel/cobble lenses and within bedrock will probably be necessary. OSF assumes no additional axial geotechnical capacity for permanent steel casing that will be installed to aid in construction of CIDH piles shown below.

Sorrento Viaduct (Right Bridge), Br. No. 57-0513R - right side widen

Support Location/ Type & Diameter	Design Loading			Nominal Resistance		Intended Length of Rock Socket m (ft)	Bottom of Pile Footing/Cutoff Elevation m (ft)	Permanent Casing Specified Tip Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
	Compression kN (tons)	Tension kN (tons)	Lateral kN (tons)	Compression kN (tons)	Tension kN (tons)					
Abut 1/CIDH 1.2 m (4 ft)	N/A			**3050 (340)**	**0**	6.10 (20.0)	+27.00 (+88.6)	+5.49 (+18.0)	-0.61(1) (-2.0)(1)	-0.61 (-2.0)
Bent 2/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.97 (36.0)	+14.00 (+45.9)	+4.57 (+15.0)	-6.40(1) (-21.0)(1)	-6.40 (-21.0)
Bent 3/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.82 (35.5)	+8.00 (+26.2)	+4.11 (+13.5)	-6.71(1) (-22.0)(1)	-6.71 (-22.0)
Bent 4/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.97 (36.0)	+8.00 (+26.2)	-7.01 (-23.0)	-17.98(1) (-59.0)(1)	-17.98 (-59.0)
Bent 5/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.97 (36.0)	+8.00 (+26.2)	-7.01 (-23.0)	-17.98(1) (-59.0)(1)	-17.98 (-59.0)
Bent 6/CIDH 2.4 m (8 ft)				**16,050 (1800)**		9.88 (32.4)	+8.00 (+26.2)	-6.89 (-22.6)	-16.76(1) (-55.0)(1)	-16.76 (-55.0)
Bent 7/CIDH 2.4 m (8 ft)				**16,050 (1800)**		9.75 (32.0)	+10.00 (+32.8)	-7.13 (-23.4)	-16.76(1) (-55.0)(1)	-16.76 (-55.0)
Abut 8/CIDH 1.2 m (4 ft)	N/A			**2900 (325)**	**0**	4.57 (15.0)	+22.00 (+72.2)	-7.62 (-25.0)	-12.19(1) (-40.0)(1)	-12.19 (-40.0)

Notes: Design tip elevation is controlled by the following demands: (1) Compression; (2) Tension; (3) Lateral Loads

Sorrento Viaduct (Right Bridge), Br. No. 57-0513R - left side widen

Support Location/ Type & Diameter	Design Loading Compression kN (tons)	Design Loading Tension kN (tons)	Design Loading Lateral kN (tons)	Nominal Resistance Compression kN (tons)	Nominal Resistance Tension kN (tons)	Intended Length of Rock Socket m (ft)	Bottom of Pile Footing/Cutoff Elevation m (ft)	Permanent Casing Specified Tip Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
Abut 1/CIDH 1.2 m (4 ft)	N/A			**3050 (340)**	**0**	6.10 (20.0)	+27.00 (+88.6)	+4.57 (+15.0)	-1.52(1) (-5.0)(1)	-1.52 (-5.0)
Bent 2/CIDH 2.4 m (8 ft)				**16,050 (1800)**		11.28 (37.0)	+14.00 (+45.9)	+4.27 (+14.0)	-7.01(1) (-23.0)(1)	-7.01 (-23.0)
Bent 3/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.67 (35.0)	+8.00 (+26.2)	+1.52 (+5.0)	-9.14(1) (-30.0)(1)	-9.14 (-30.0)
Bent 4/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.67 (35.0)	+8.00 (+26.2)	-6.10 (-20.0)	-16.76(1) (-55.0)(1)	-16.76 (-55.0)
Bent 5/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.67 (35.0)	+8.00 (+26.2)	-6.10 (-20.0)	-17.37(1) (-57.0)(1)	-17.37 (-57.0)
Bent 6/CIDH 2.4 m (8 ft)				**16,050 (1800)**		9.60 (31.5)	+8.00 (+26.2)	-7.77 (-25.5)	-17.37(1) (-57.0)(1)	-17.37 (-57.0)
Bent 7/CIDH 2.4 m (8 ft)				**16,050 (1800)**		9.45 (31.0)	+10.00 (+32.8)	-7.92 (-26.0)	-17.37(1) (-57.0)(1)	-17.37 (-57.0)
Abut 8/CIDH 1.2 m (4 ft)	N/A			**2900 (325)**	**0**	5.33 (17.5)	+22.00 (+72.2)	-8.08 (-26.5)	-13.41(1) (-44.0)(1)	-13.41 (-44.0)

Notes: Design tip elevation is controlled by the following demands: (1) Compression; (2) Tension; (3) Lateral Loads

Sorrento Viaduct (Left Bridge), Br. No. 57-0513R - right side widen

Support Location/ Type & Diameter	Design Loading Compression kN (tons)	Design Loading Tension kN (tons)	Design Loading Lateral kN (tons)	Nominal Resistance Compression kN (tons)	Nominal Resistance Tension kN (tons)	Intended Length of Rock Socket m (ft)	Bottom of Pile Footing/Cutoff Elevation m (ft)	Permanent Casing Specified Tip Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
Abut 1/CIDH 1.2 m (4 ft)	N/A			**2250 (250)**	**0**	3.96 (13.0)	+26.00 (+85.3)	+7.62 (+25.0)	+3.66(1) (+12.0)(1)	+3.66 (+12.0)
Bent 2/CIDH 2.4 m (8 ft)				**16,050 (1800)**		9.14 (30.0)	+13.00 (+42.7)	+5.79 (+19.0)	-3.35(1) (-11.0)(1)	-3.35 (-11.0)
Bent 3/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.36 (34.0)	+10.00 (+32.8)	+3.05 (+10.0)	-7.32(1) (-24.0)(1)	-7.32 (-24.0)
Bent 4/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.52 (34.5)	+8.00 (+26.2)	-0.76 (-2.5)	-11.28(1) (-37.0)(1)	-11.28 (-37.0)
Bent 5/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.67 (35.0)	+9.00 (+29.5)	-7.92 (-26.0)	-18.59(1) (-61.0)(1)	-18.59 (-61.0)
Bent 6/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.06 (33.0)	+8.00 (+26.2)	-8.53 (-28.0)	-18.59(1) (-61.0)(1)	-18.59 (-61.0)
Bent 7/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.06 (33.0)	+8.00 (+26.2)	-8.53 (-28.0)	-18.59(1) (-61.0)(1)	-18.59 (-61.0)
Bent 8/CIDH 2.4 m (8 ft)				**16,050 (1800)**		9.75 (32.0)	+8.00 (+26.2)	-7.62 (-25.0)	-17.37(1) (-57.0)(1)	-17.37 (-57.0)
Abut 9/CIDH 1.2 m (4 ft)	N/A			**2850 (320)**	**0**	3.96 (13.0)	+19.00 (+62.3)	-7.92 (-26.0)	-11.89(1) (-39.0)(1)	-11.89 (-39.0)

Notes: Design tip elevation is controlled by the following demands: (1) Compression; (2) Tension; (3) Lateral Loads

Sorrento Viaduct (Left Bridge), Br. No. 57-0513R - left side widen:

Support Location/ Type & Diameter	Design Loading			Nominal Resistance		Intended Length of Rock Socket m (ft)	Bottom of Pile Footing/Cutoff Elevation m (ft)	Permanent Casing Specified Tip Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
	Compression kN (tons)	Tension kN (tons)	Lateral kN (tons)	Compression kN (tons)	Tension kN (tons)					
Abut 1/CIDH 1.2 m (4 ft)	N/A			**2250 (250)**	**0**	3.96 (13.0)	+26.00 (+85.3)	+9.75 (+32.0)	+5.49(1) (+18.0)(1)	+5.49 (+18.0)
Bent 2/CIDH 2.4 m (8 ft)				**16,050 (1800)**		9.45 (31.0)	+13.00 (+42.7)	+11.58 (+38.0)	+2.13(1) (+7.0)(1)	+2.13 (+7.0)
Bent 3/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.97 (36.0)	+10.00 (+32.8)	+4.27 (+14.0)	-6.71(1) (-22.0)(1)	-6.71 (-22.0)
Bent 4/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.36 (34.0)	+8.00 (+26.2)	-0.61 (-2.0)	-10.97(1) (-36.0)(1)	-10.97 (-36.0)
Bent 5/CIDH 2.4 m (8 ft)				**16,050 (1800)**		11.58 (38.0)	+9.00 (+29.5)	-7.01 (-23.0)	-18.59(1) (-61.0)(1)	-18.59 (-61.0)
Bent 6/CIDH 2.4 m (8 ft)				**16,050 (1800)**		11.58 (38.0)	+8.00 (+26.2)	-8.53 (-28.0)	-20.12(1) (-66.0)(1)	-20.12 (-66.0)
Bent 7/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.67 (35.0)	+8.00 (+26.2)	-0.61 (-2.0)	-11.28(1) (-37.0)(1)	-11.28 (-37.0)
Bent 8/CIDH 2.4 m (8 ft)				**16,050 (1800)**		10.97 (36.0)	+8.00 (+26.2)	-0.61 (-2.0)	-11.58(1) (-38.0)(1)	-11.58 (-38.0)
Abut 9/CIDH 1.2 m (4 ft)	N/A			**2850 (320)**	**0**	3.96 (13.0)	+19.00 (+62.3)	-7.62 (-25.0)	-11.58(1) (-38.0)(1)	-11.58 (-38.0)

Notes: Design tip elevation is controlled by the following demands: (1) Compression; (2) Tension; (3) Lateral Loads

When pile nominal resistance in tension is provided by DSD, OSF can then provide design pile tip elevations in tension. Also, if the pile tip elevation is controlled by lateral demands, the designer is responsible to present correct foundation data, governed by lateral control, on the foundation plans.

Axial compression and tension values noted in the tables below are based on skin friction only within the rock. End bearing was not considered due to working below the water table and the possibility that cleaning out the bottom of pile borings effectively may be rather difficult at depth and may make it difficult to realize substantial end bearing using Caltrans standard pile vertical deflection criteria of 13 mm (0.5 in).

Bedrock topography (top of rock) is often quite variable across short lateral distances. Due to this fact, the pile data tables above include intended length of the rock socket at each support. The intended length of the rock socket should be measured from the bottom of the permanent casing down to the pile specified tip elevation. OSF feels that permanent casing should be seated into rock approximately 0.61 m (2 ft). If the bedrock slope is steeper than expected, the permanent casing may need to be seated slightly deeper to seal out water and potential caving soils.

Constructability

As mentioned above, OSF recommends installation of permanent casing to be placed into bedrock to prevent caving of loose soils and gravel/cobbles lenses into pile borings and help seal off ground water from entering the excavations once seated into rock. OSF and OGEE feel that using a vibratory hammer to place steel casing down to a level close to casing specified tip elevation would facilitate pile construction and effectively reduce creation of voids along the pile length by undesireable caving of loose soils and rounded cobble/gravel material. Drilling ahead of the casing, especially within the upper loose/soft soil zones should be avoided to reduce caving and creation of voids, thus compromising lateral pile capacity. OSF anticipates center relief drilling to facilitate casing advancement. Hard slow drilling [through hard metavolcanic cobble zones (cobbles commonly up to 150 mm diameter), cobble-size mudstone and sandstone rock fragments, and bedrock] is anticipated during installation of permanent casing and CIDH piles (rock sockets). Drilling ahead of casing may be required, in order to advance casing within the lower gravel/cobble lenses and within bedrock. Once casing is seated into bedrock, drilling for the rock sockets can be completed.

Ground water should be anticipated at relatively shallow depths. Static ground water was measured at elevation +7.77 m (+25.5 ft) within Boring 99-1 (drilled near the proposed left widening of the left bridge – Bent 8. The wet method is advised for CIDH pile construction. The bottom of all excavations should be cleaned of loose debris before placing concrete.

Clay mineralogy within formational material appears sensitive to the introduction of fresh water, which could cause swelling of clays and slicking of borehole walls, resulting in reduced pile/soil skin friction capacity. OSF feels that a mud/polymer expert should be consulted and be available to the contractor to advise on proper drilling fluid/slurry chemistry in order to prevent clay swelling. OSF feels that seating permanent casing into the formational mudstones/claystones/siltstones should help seal off ground water from reacting with the formational clays.

Corrosiveness

Laboratory tests of composite soil samples [taken within Boring 99-1 (Left Bridge Widen – Bent 8 Left) and Boring 99-13 (Right Bridge Widen – Bent 6 Left) indicate that fill and native material are somewhat corrosive. Corrosion tests on the above material show pH ranges from 7.67 to 8.41, minimum resistivity ranges from 370 to 678 ohm-cm, sulfate and chloride content were measured at 1730 and 380 ppm, respectively, and the estimated number of years to perforation of 18 gauge galvanized steel culvert ranges from 17 to 21.3 years. OSF feels that the Corrosion Technology Branch should be consulted regarding test results and possible recommendations.

If you have any questions, please call Joe Pratt at (213) 620-2001 or Richard Fox at (916) 227-7085.

Report by:

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c: R.E. Pending File
DBarlow - Specs & Estimates
HBrimhall - Proj Mgmt
District 11 (2)
ELeivas - OSF
RFox - OSF
AAbghari – OGEE
DParks – Corrosion Tech

References

Amiri, S., February 3, 1995, Preliminary Geotechnical/Geological Report For Seismic Retrofit, Sorrento Valley Overhead, Br. Nos. 57-0513R/L, Caltrans Memorandums.

Blakesley, G., March 24, 2000, Final Bottom of Footing/Pile Cutoff Elevations for Sorrento Valley Overhead (Rt. & Lt Widen), Br. No. 57-0513R/L, Caltrans facsimile copy.

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