State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. KEVIN ROSS
Division of Structure Design
Bridge Design Branch 15

Date: September 8, 1999

File: 04-CC-80-Var.
04-013011
Carquinez Bridge 28-0352L
Replacement Project

Caltrans Metric

Crockett Viaduct
Bridge No. 28-0367L

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

Subject: Supplemental Foundation Recommendations for the new Crockett Interchange Viaduct Structure

INTRODUCTION

These supplemental foundation recommendations are due to a request from the Division of Structure Design, dated August 30, 1999, to transfer "D" Line Bent 7 from the Interchange Project (EA 013051) to the Main Span and North Approach Project (EA 013011). These supplemental foundation recommendations supercede the sections Construction Considerations, Nos. 13 and 16, which address pile load testing and pile dynamic monitoring contained in our original report for the new Crockett Interchange Viaduct Structure, Bridge No. 28-0367L, dated June 10, 1999. The remaining portions of that report have remained unchanged.

Supplemental Construction Considerations (EA 013051)

13. Pile load testing is required at the following locations. Pile load testing is to be performed in both compression and tension. In addition to the load test pile, the anchor piles are to be dynamically monitored. The load test and anchor piles may be production piles and incorporated into the footing after testing has been completed and piles have been accepted. Load test locations and corresponding control locations are as follows.

Bridge Number	Load Test Pile Location	Control Location
28-0367L	Bent 5	Bent 5
	Bent 6	Bent 6

16. In addition to the load test and anchor piles, the first pile installed at Abutment 1 and Bents 2, 3, and 4, locations shall be dynamically monitored. Dynamic monitoring is to be performed for the last 7 meters.

Construction Considerations (EA013011)

Pile load testing is required at the following location. Pile load testing is to be performed in both compression and tension. In addition to the load test pile, the anchor piles are to be dynamically monitored. The load test and anchor piles may be production piles and incorporated into the footing after testing has been completed and piles have been accepted. Load test location and corresponding control locations are as follows.

Bridge Number	Load Test Pile Location	Control Location
28-0367L	Bent 7	Bent 7

Before the remaining piles at the control locations listed in the table above are driven, cut to length, or cast, load testing of each load test pile shown on the plans for the corresponding control locations shall be completed.

The pile load test should be performed on the steel shell after cleanout for the reinforcement and prior to placing reinforcement and concrete therein.

Any questions regarding the above recommendations should be directed to the attention of Michael Huyette (916) 227-7228 (CALNET 498-7228) or Mark DeSalvatore (916) 227-7056 (CALNET 498-7056), Division of Materials and Foundations, Office of Structure Foundations.

Supplemental Report by

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State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. JOSEPH E. DOWNING
Office of Structure Design
Design Branch A

Attention Mr. Mike Marquez

Date: June 10, 1999

File: 04-CC-80-Var.
04-013051
Crockett Interchange
New Carquinez Bridge
South Approach

Caltrans Metric

Crockett Viaduct
Bridge No. 28-0367L

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

Subject: Foundation Recommendations for the new Crockett Interchange Viaduct Structure

INTRODUCTION

This report presents the results of the foundation investigation for the new Crockett Interchange Viaduct structure. The Structure Foundations Branch of the Office of Materials and Foundations (OMF) performed this investigation pursuant to the request for seismic, geologic and foundation information, dated March 13, 1998, by the Office of Structure Design (OSD). The new Crockett Interchange will replace the existing interchange structures as part of the seismic retrofit (replacement) of the existing westbound (1927) Carquinez Bridge. A Preliminary Foundation Report (PFR) for this project was previously submitted on April 9, 1998. Preliminary (dated July 23, 1998) and Final (dated January 14, 1999) p-y, t-z, q-z curves and ARS data for this project was transmitted to OSD by the Geotechnical Earthquake Engineering Section. The foundation recommendations presented in this report supercede any previous foundation recommendations for this project unless specifically noted otherwise. The Log of Test Boring (LOTB) sheets will be submitted when finalized.

The layout of the planned Viaduct structure is shown on the Crockett Viaduct Foundation Plan, latest revision dated 3/2/99. The new Viaduct structure will connect at its northern end with the new replacement westbound Carquinez Toll Bridge, and will merge with existing westbound Interstate 80 (I-80) at its south abutment.

INVESTIGATION

The Structure Foundations Branch of OMF performed a geologic reconnaissance and a subsurface investigation of the project site from July 1998 through March 1999, following reviews of pertinent geologic literature and As-Built documents of the existing structures.

The geologic reconnaissance consisted of observing soils and surface water exposed at the planned new structure locations and local vicinity and observing deformation of cultural features (such as streets, power poles, curbs, sidewalks, fences etc.) for evidence of past slope instability. Our reconnaissance was aided by studying historic (1956, 67, 70, 78, 83 and 88) and more recent (1995) aerial photographs enlarged to a scale of 1 cm equals approximately 30 m (1 inch equals approximately 250 feet).

The subsurface investigation for the proposed Crockett Interchange structures consisted of drilling twenty three 77 mm to 94 mm diameter "mud" rotary NX and HQ size core test holes ranging from 3.7 m to 90.8 m deep, drilling two 152 mm hollow- stem auger test borings approximately 6.8 m deep, twenty 2.54 cm (1-inch) driven test probes ranging from about 2 m to 6.8 m deep, and performing 13 Electronic Cone Penetration Soundings (CPT's).

Borehole geophysical measurements were performed in select borings and consisted of seismic velocities (P- and S-wave), as well as formation conductivity / resistivity, natural gamma, and borehole caliper measurements. The results of the borehole geophysical measurements were presented in a report dated December 12, 1998, prepared by Geophysics Section of the Geotechnical Support Branch of OMF and the plots are included on the Log of Test Borings (LOTB's).

Pressuremeter testing was conducted in borings 98-104 and 98-119, at Abutment 1 and Bent 7 respectively, to provide information on the stress-strain behavior of the formational bedrock at the site. Pressuremeter test results were presented in a report dated November 30, 1998 prepared by the Testing and Instrumentation Section of the Geotechnical Support Branch of OMF. The pressuremeter test data was inconclusive (limit pressure of the formational bedrock was not reached) and therefore, the data was not used in analysis and foundation design for this project.

The 13 electronic Cone Penetration Soundings were located in the vicinity of borings 98-119 and 98-120 to help define the extent of a suspected liquefiable sand layer encountered in those test borings, and to better define the slope of the bedrock surface at those locations. The plots of the CPT data are shown on the LOTB's.

Laboratory testing for classification and strength was performed on selected soil and rock samples recovered from the test borings by the Geotechnical Laboratory Section of the Geotechnical Support Branch of OMF.

PRESENT SITE CONDITIONS

Topography at the planned Viaduct location consists of a north-facing hillside and northerly adjacent reclaimed mud flats. Elevations vary along the proposed Viaduct from elevation 48 meters at the south end (Abutment 1) to about elevation 3 meters at the north end (Bent 7) of the planned Viaduct structure. The hillside portion of the site (Abutment 1 through Bent 4) is developed with residential streets and houses and numerous above ground and underground utilities. The streets that traverse the hillside were constructed on gently sloping cut and fill benches within a broad canyon fill area. The slopes between the hillside streets are mostly steep to very steep. The nearly level (mud flat fill) portion of the structure site (Bents 5 through 7) is presently developed with C&H maintenance/storage yard facilities and County maintained roads which cross under the existing interchange structures.

Underground utilities for the existing developments within the planned Viaduct area include PG&E natural gas and hot oil lines, County sewer and storm drainage lines, fiber optic communications lines and East Bay MUD water lines. Additionally, sewer collection and out-fall pipes and high voltage electric lines cross the C&H yard to the southwesterly adjacent sewage treatment plant. Overhead utilities consist primarily of PG&E electric, private cable and PAC Bell telephone lines on utility poles.

SITE GEOLOGY AND SOILS

Formational bedrock materials were encountered within all of our borings. The bedrock is composed primarily of marine claystone, with minor lenses of marine siltstone, sandstone and shale of the Late Cretaceous Panoche Formation, a part of the Lower Cretaceous to Upper Jurassic Great Valley Sequence. The structure of the rock formation is chaotic, and is discontinuous both laterally and with depth due to the mode of initial deposition (typical of relatively deep-water submarine-fan complexes) and the tectonic shearing effects associated with the nearby Franklin thrust fault, located about 1/2 km (1/3 mile) south of the Viaduct structure. The chaotic bedrock structure results in discontinuous, cobble to boulder sized blocks of more competent formational rock within a highly sheared, very deeply weathered to decomposed rock material resulting in a soil-like matrix.

The claystone encountered within our test borings was generally massive, very intensely weathered or decomposed to a hard clay consistency with varying amounts of gravel and sand sized residual rock fragments. The residual claystone rock fragments are generally intensely to very intensely fractured and moderately hard to soft. The claystone typically swelled vertically into the core barrel during our drilling operations (providing generally greater than 100 percent recovery), indicating expansive bedrock under local stress conditions. The minor lenses of interbedded, generally more competent, siltstone, sandstone and shale are locally very thin to thin bedded, intensely weathered, intensely to very intensely fractured, and moderately hard to soft.

The bedrock surface has been eroded into a northeast trending hillside canyon, located under the existing Viaduct structures, and a generally north trending canyon in the northern (bay flat) portion of the planned new Viaduct structure. Maximum depths to the sloping bedrock surfaces at the structure support locations are approximately 7 m, 6 m, 8 m, 12 m, 9 m, 29 m, and 34 m for Abutment 1 through Bent 7, respectively.

The bedrock on the hillside portion of the project site (Abutment 1 through Bent 4) is covered by colluvial, alluvial and / or residual bedrock (natural) soils between approximately 1 to 3 meters (about 3 to 15 feet) thick. Residual bedrock soils in the northern portion (Bents 5 through 7) of the project site varies from about 3 to 12 meters (about 10 to 40 feet) in thickness, respectively. The natural soils are primarily soft to stiff sandy silts and sandy clays with varying amounts of residual bedrock claystone, siltstone, sandstone and shale fragments.

Overlying the residual bedrock soils in the north portion of the site (Bents 5 through 7) is between 3 to 18 meters (10 to 60 feet) of bay mud consisting of soft clays, silts and loose sands and silty sands with varying amounts of organics (plant matter).

Overlying the hillside colluvium and old bay mud deposit areas are variable density old fill soils. The fill thickness varies from 4 m to 11 m (13 to 36 feet) in thickness at Abutment 1 through Bent 4, respectively. The fill thickness varies from 4 m to 3 m (13 to 10 feet) in thickness at Bent 5 through Bent 7, respectively. The fill soils consist primarily of sandy clays and silts with varying amounts of sands and gravel. A relatively clean gravel layer, about 0.6 m (2 feet) thick was encountered near the hillside toe in our boring 98-121 at Bent 5. The gravel layer is likely the remains of the old railroad ballast fill for Horseshoe Bend. The old fill soils at the site contain scattered and local concentrations of debris consisting of wood, metal, concrete, asphalt, glass and brick. The aerial photographs from 1956 depict tires on the bay mud surface being filled over in the vicinity of Bent 7.

GROUNDWATER

Stabilized drilling fluid was observed in July 1998 at elevation 31.4 m and 15.7 m in our hillside borings at Bents 2 and 3 respectively. Stabilized drill fluid was observed in October and November, 1998 at elevation 2.0 m, 0.4 m, and 0.6 m in the old bay fill area borings at Bents 5, 6 and 7, respectively. Due to the proximity of Bents 5, 6, and 7 with respect to the Carquinez Strait, ground water levels are expected to fluctuate with tidal actions.

Seasonal perched groundwater conditions are expected to occur within the old fill soils at Abutment 1 and at every bent location. Additionally, groundwater was encountered during the easterly adjacent retrofit pile installation where shafts were drilled through buried abandoned utility trenches and / or the abandoned drainage pipes. Where abandoned pipes or utility trenches are encountered during foundation construction, trapped groundwater is expected to be present.

FAULTING AND SEISMICITY

The proposed new Viaduct structure location is within close proximity to several active faults. The site is potentially subject to strong ground motions from these earthquake sources during the design life of the new structure. The Franklin Fault, located less than one kilometer south of the site, is the closest potentially active fault. The active Hayward Fault is located approximately 12 km (7-1/2 mi.) to the southwest and the active San Andreas Fault is located approximately 40 km (25 mi.) west of the site. The Franklin fault is considered having a moment magnitude of 6.5 and could generate a maximum bedrock acceleration of 0.9g at the site. Other nearby active faults are expected to produce peak ground acceleration values less than those produced by the Franklin Fault.

Preliminary Acceleration Response Spectra (ARS) information was previously provided for this project by the Geotechnical Earthquake Engineering Section of OMF, in a memorandum dated June 30, 1998. Recommended design P-Y, T-Z, and Q-Z curves for the new Viaduct structure and revised Acceleration Response Spectra (ARS) for the new Viaduct Bents 6 and 7 were subsequently presented in memorandums dated January 14 and February 4. 1999, respectively, by the Geotechnical Earthquake Engineering Section of OMF.

LIQUEFACTION

Potentially liquefiable soil layers were identified within our Bents 6 and 7 borings and encountered at the CPT Sounding locations. As recommended in the January 14, 1999 memorandum by the Earthquake Engineering Section, the axial geotechnical capacity of those layers at Bents 6 and 7 locations was ignored.

FOUNDATION RECOMMENDATIONS

The following recommendations are for the Crockett Viaduct, Br. No. 28-0367L as shown on the "General Plan" dated March 2, 1999.

The following specified pile tip elevations are based on specific project information regarding support locations, pile cut off elevations, design loadings, and required nominal resistances received May 10, 1999 via electronic mail from the Office of Structure Design (OSD). All Bents may be supported by 750 mm OD open ended Cast-In-Steel Shell (CISS) concrete piles as requested by OSD. The specified pile tip elevations (SPTE) are listed below in Table 1. The ultimate geotechnical capacity of the piles will equal or exceed required nominal resistance shown in the table below. Refer to the Design Tip Elevation column in Table 1 to determine which required nominal resistance controls the SPTE per support location. The Foundation Testing and Instrumentation Section of the Office of Materials and Foundations will be requested top perform driveability studies and the results will be forwarded when complete.

Abutment 1 may be supported on either driven steel "H" piles, driven with pile lugs or 600 mm Cast-In-Drilled Hole (CIDH) concrete piles as requested by the Office of Structure Design (OSD).

PILE DATA – TABLE 1

Location	Pile Type	Design Load	Nominal Resistance		Design Tip Elevation * (m)	Specified Tip Elevation (m)
			Compression	Tension		
Abut. 1 (Alt. 1)	Steel "H" piles HP 360 x 174 (14 x 117)	1350 kN	2700 kN	0 kN	+ 17.0 (1)	+ 17.0
Abut. 1 (Alt. 2)	600 mm CIDH	1350 kN	2700 kN	0 kN	24.5 (1)	+ 24.5
Bent 2	CISS Pile PP 750 x 15.88	N/A	4900 kN	0 kN	+ 9.0 (1)	+ 9.0
Bent 3	CISS Pile PP 750 x 15.88	N/A	4900 kN	0 kN	+ 2.0 (1)	+ 2.0
Bent 4	CISS Pile PP 750 x 15.88	N/A	4900 kN	0 kN	- 5.0 (1)	- 5.0
Bent 5	CISS Pile PP 750 X 15.88	N/A	4900 kN	1800 kN	- 24.0 (1) - 15.0 (2)	- 24.0
Bent 6	CISS Pile PP 750 X 15.88	N/A	4900 kN	1800 kN	-33.0 (1) & (3) -25.0 (2) & (3)	- 33.0
Bent 7	CISS Pile PP 750 X 15.88	N/A	4900 kN	1800 kN	-38.0 (1) & (3) -27.0 (2) & (3)	- 38.0

* Design tip elevation is controlled by the following demands: (1) Compression; (2) Tension; (3) Liquefaction potential from elevation – 3.3 m to elevation – 7.6 m at Bent 6 location and from elevation – 4.5 m to elevation – 9.4 m at Bent 7 location.

General Notes

1. The structure engineer shall show on the plans, in the pile data table, the minimum pile tip elevation required to meet the lateral load demands.

2. Should the specified pile tip elevation required to meet lateral load demands exceed the specified pile tip elevations given within this report, the Office of Structural Foundations shall be contacted for further recommendations.

3. Driveability studies will be performed by the Office of Structural Foundations, Foundation Testing and Instrumentation Section, and forwarded when complete. Piles shall conform to the recommendations (min. yield strength, min. wall thickness, and min. hammer size) presented in the Driveability Study.

4. Pile spacing should not be less than 2.5 pile diameters center to center.

5. If the "H" pile option is utilized for support at Abutment 1, all Steel "H" piles are to be driven with Lugs installed. Lug details shall be shown in the Contract Plans.

6. Support locations are to be plotted on the Log of Test Borings, in plan view, as stated in "Memos to Designers" 4-2. The plotting of the support locations should be made prior to the foundation review.

Construction Considerations

1. Ground water was encountered during drilling of test holes and will be encountered during construction and pile installation.. Ground water elevations will vary significantly with seasonal variations and tidal fluctuations. It should be anticipated that groundwater will be encountered during excavation for pile caps at all Bent locations. It should be anticipated that groundwater levels will fluctuate with tidal action within the footing excavations at Bents 5, 6, and 7 locations.

2. Due to the required embedment of piling required to obtain design loads, steel piling is required. Driving tips may be necessary to ensure pile integrity during hard driving.

3. At all support locations, pile bearing will be accessed by the WAVE equation.

4. With the engineers approval in writing, the contractor may use hydraulic hammers to drive support piles.

5. With the engineers approval in writing, the contractor may partially install driven piles with a vibratory hammer at the locations and to the elevations listed below.. Vibratory hammers shall not be used to advance piles at Abutment 1, Bent 2, 3, 4, or Bent 5.

   Bent 6: elev. – 7.6 m
   Bent 7: elev. – 9.4 m

6. The geotechnical capacity of the CISS piles at all Bents locations is based on <u>skin friction and end</u> bearing. A soil plug of at least 5 meters shall remain undisturbed in the bottom of the pile to develop the required end bearing.

7. In addition to driving, drilling through the center of <u>open-ended steel shells</u> (center relief drilling) will be required to obtain the specified penetration. Drilling to assist driving shall not extend more than 1 meter beyond the pile tip during installation. Equipment or methods used for cleaning out steel shells shall not cause quick soil conditions or cause scouring or caving of the hole below the pile tip. Unless approved by the engineer in writing, no drilling to assist driving shall be allowed within 5 meters of the specified pile

tip elevation in order to prevent the piles from running beyond the SPTE and to ensure proper seal at the SPTE for casting concrete in the dry.

8. The specified pile tip elevations given in Table 1 were determined based on the lowest interpreted rock elevation within the footing footprint. Increases or decreases in the length of steel piling including Cast-in-Steel Shell piling may be required.

9. Due to the variability in strength properties of rock at this site, driving will be difficult. To obtain the pile embeddment into rock, increases or decreases in the length for "H" piles and Cast-in-Steel Shell piling may be required.

10. Any driven CISS piles achieving refusal within 5 meters of SPTE during driving may be considered satisfactory and cut off, provided the pile has penetrated to the design tip elevation required to meet lateral demands. Any cut offs are to be performed only after written approval from the Engineer. Refusal shall be defined as a pile achieving two times (2x) the required design loading as shown on the contract plans and above in Table 1. Two times (2x) required design loading shall be 4900 kN (1100 kips) at the Bents.

11. If the steel "H" pile option is used at Abutment 1, steel pile lugs shall be used. The lugs shall be placed in accordance with Bridge Construction Memo #130-5.0. Lug details are to be shown in the Contract Plans.

12. At Abutment 1, any driven steel "H" pile achieving refusal below the SPTE required to meet lateral load demands may be considered satisfactory and cut off. Refusal shall be defined as a pile achieving two times (2x) the required design loading as shown on the Contract Plans. Two times (2x) the required design loading shall be 2700 kN at Abutment 1.

13. Pile load testing is required at the following locations. Pile load testing is to be performed in both compression and tension. In addition to the load test pile, the anchor piles are to be dynamically monitored. The load test and anchor piles may be production piles and incorporated into the footing after testing has been completed and piles have been accepted. Load test locations and corresponding control locations are as follows:

Bridge Number	Load Test Pile Location	Control Location
28-0367L	Bent 5	Bent 5
	Bent 7	Bent 6 and Bent 7

14. Before the remaining piles at the control locations listed in the table above are driven, cut to length, or cast, load testing of each load test pile shown on the plans for the corresponding control locations shall be completed.

15. The pile load test should be performed on the steel shell after cleanout for the reinforcement and prior to placing reinforcement and concrete therein.

16. In addition to the load test and anchor piles, the first pile installed at Abutment 1 and Bent 2, 3, 4, and 6 locations shall be dynamically monitored. Dynamic monitoring is to be performed for the last 7 meters.

The recommendations contained in this report are based on certain specific project information regarding design loads and structure location that has been provided by OSD. If

any conceptual changes are made during final project design, the Structure Foundations Branch of OMF should review those changes to determine if these foundation recommendations are still applicable.

Any questions regarding the above recommendations should be directed to the attention of Michael Huyette (916) 227-7228 (CALNET 498-7228), Tim Crosby (916) 227-7223 (CALNET 498-7223) or Mark DeSalvatore (916) 227-7056 (CALNET 498-7056), Office of Materials and Foundations, Structure Foundations Branch.

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Attachments

c: R.E. Pending File
DBarlow - Specs & Estimates
HBrimhall - Proj Mgmt
District 4 (2)
ELeivas - SFB
DSpeer – GSB
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