State of California | Business, Transportation and Housing Agency

Memorandum

To: MR. TOM POLLOCK, Chief
Office of Structure Design

Attention: Mr. Bob Anderson
Design Section 59-234

Date: September 18, 1992

File: 11-SD-5-R32.88
11203 030111

Carmel Valley
Creek Connector
Bridge No. 57-0990G
#6

From: DEPARTMENT OF TRANSPORTATION
Division of New Technology, Materials & Research
Office of Engineering Geology - South

Subject: FOUNDATION RECOMMENDATIONS

INTRODUCTION

This office has completed a subsurface investigation at the proposed Carmel Valley Creek Connector (Bridge No. 57-0990G) on Route 5, San Diego, CA. The investigation was a joint effort with District 11 Materials and consisted of drilling eight rotary borings, seven electric cone penetrometer (CPT) soundings, reviewing the site conditions and available records. Our investigation was based upon conversations with Design Section 10, the Foundation Plan received January 22, 1990 and the General Plan received December 13, 1991.

The proposed 1180 foot long, concrete prestressed box girder bridge will be constructed east of the existing Route 5 and connect northbound Route 5 traffic to eastbound Route 56. Approach fills will be placed to an approximate maximum height of 40 feet. The bridge will be designed using 100 ton (compressive load) driven piles with tension capacities of 30 tons.

SUMMARY OF FINDINGS

Our findings are presented within this report and the Log of Test Borings (LTB). The LTB will be transmitted at a later date and is to be included in the contract plans. The layout sheet shows all borings drilled in the area, those borings not shown on the profile will be available through the Office of Geotechnical Engineering.

The site of the proposed Carmel Valley Creek Connector is southeast of the Carmel Valley Road Overcrossing along Route 5. Carmel Valley Creek, a tributary of the Soledad Valley estuary, flows east to west southeast of the proposed structure. In this area Carmel Valley Creek is a sinuous, perennial stream that shows no incision below the active flood plain. At its western end, the bridge will be constructed over the existing El Camino Real and restaurant. El Camino Real will be re-routed and the restaurant structure removed. At the easterly end of the proposed structure, the surrounding ground is undeveloped.

Exploratory borings reveal the bridge site is thinnly mantled by artificial fill that is underlain by Holocene-age, interbedded estuary and alluvial deposits (Power and others, 1982), that overlie Eocene bedrock. The slightly compact to compact artificial fill is located in the area of El Camino Real and the restaurant (Sta. 1097 to 1103). The estuary deposits (Qhe) are very loose to loose, dark gray to blue to black, fossiliferrous, silty sands to micaceous silts and highly plastic clays that are highly organic with root traces. Interbedded with the estuary deposits is brown to gray fluvial deposits (Qhfl) that are slightly compact to compact silty sand to sand with lenses of clay, cobbles and boulders. At the base of the fluvial deposits and are varying thicknesses of very dense, cobbly to bouldery sands to sandy boulders. Between 69 and 115 feet below the present ground surface, the Holocene deposits overlie moderately-cemented, green to brown, Eocene mudstones and sandstones of the Delmar Formation (Td).

Ground Water

Ground water was estimated between 12.0 and 20.0 feet above sea level. The elevation of the ground water surface is highly dependent upon the seasonal rainfall. In the undeveloped areas, from December to late April, ground water is at or near the ground surface.

Surface Water

Minor flows were noted in the stream between the months of November and May. Flow was confined to the channel with sheet flows occurring only in the road and restaurant area during heavy rains.

Corrosivity

Corrosion tests performed in the area indicate that the soils are, in general, noncorrosive. The CALTRANS Corrosion Unit classifies sulfates in excess of 2,000 ppm and chlorides in excess of 500 ppm as corrosive. The presence of an organic odor and identification of roots within the samples indicates a high organic content (10-20%) in the earth materials.

Plasticity

A number of samples were selected and submitted for testing for Atterberg's Limits (California Test 204) and are plotted on Figure 1. Additional Atterberg's Limits by District 11 are shown on the LTB.

Sample (Boring # and depth)	Plasticity Index	Plastic Limit	Liquid Limit	Soil Type*
B-2L @ 30	13	18	41	ML/OL
B-2L @ 85	4	20	24	ML/OL
B-20L @ 25	5	17	22	CL
B-21L @ 10	3	19	22	CL
B-21L @ 35	4	19	23	CL
B-21L @ 50	8	15	23	CL
B-22L @ 30	np**	np	np	SM/SC
B-22L @ 35	7	19	26	ML/OL
B-22L @ 50	7	19	26	ML/OL
B-23L @ 25	np	np	np	SM/SC
B-23L @ 30	9	22	31	CL
B-23L @ 35	np	np	np	SM/SC
B-23L @ 40	np	np	np	SM/SC
B-23L @ 75	np	np	np	SM/SC
B-23L @ 80	np	np	np	SM/SC
B-23L @ 90	np	np	np	SM/SC

*Unified Soil Classification

** np=nonplastic

Sieve Analysis

A number of samples were selected and submitted for testing for grain-size distribution (California Test 202). The soils were found to be predominantly sandy silts and silty clays.

Seismicity

The Rose Canyon fault is mapped 5 miles west of the site (Reichle and others, 1990). The site is not within the Alquist-Priolo Special Study Zone (Hart, 1990). Mualchin & Jones (1991) proffer the following information for design of structures in the area:

Maximum Credible Earthquake Magnitude	7.0
Peak Horizontal Bedrock Acceleration	0.5 gravity

The depth to "rock-like" material (Vs greater than 2,500 feet per second) is 70-115 feet. The duration of strong-ground motion should be on the order of 15-20 seconds. The bridge site has not experienced ground shaking greater than 0.1 gravity in nearly 200 years (Reichle and others, 1990; Figure 2 & Table II).

Secondary Seismic Effects

Power and others (1982) performed a regional evaluation of liquefaction susceptibility in the San Diego Metropolitan area south of Carmel Valley. Their Table 1-1 indicates that the Holocene fluvial (Qhfl) and Holocene estuarine (Qhe) deposits, similar to those found in our borings, have a moderate to high susceptibility to liquefy during seismic events. They found that the estuarine and fluvial deposits have a mean blow count of 16 and recommended that site specific liquefaction studies be performed in areas where these deposits occur. Reichle and others (1990) hypothesized that Carmel Valley is not an area with high to very high potential of experiencing ground failure due to liquefaction during an earthquake on the Silver Strand fault in Mission Bay.

Figures 2-6 illustrate the liquefaction susceptibility of the deposits underlying Carmel Valley Creek. The analysis performed for this report utilized the method outlined by the National Research Council (1985), after Seed and Idriss (1982), and supplemented by Ishihara (*in press*). Figures 2-6 show cyclic stress ratio versus normalized blow counts (adjusted for fines content) for those samples that are below the ground water table at the time of the investigation, have less than 20% clay (0.005 mm) and blow counts less than 30 per foot. The remaining samples not plotted contained greater than 20% clay (0.005 mm) or blow counts greater than 30 and are not liquefiable (see below).

Blow counts (abscissa) were determined using the method outlined by the National Research Council [NRC] (1985) and supplemented by Ishihara (*in press*). First, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586 incorporating the recommendations contained within NRC (1985; Tables 4-3 & 4-4). Secondly, the measured blow count (N) was normalized to one ton of overburden at 60% energy transfer or $(N_1)_{60}$ using the method outlined in NRC (1985). Third, sieve analysis (California Test 202) was performed to determine the influence of fines content (percentage of materials passing through the #200 sieve) as outlined by Ishihara (*in press*). At this point, any sample with greater than 20% clay was considered not liquefiable and eliminated. The $(N_1)_{60}$ of samples with less than 20% clay was then converted to $(N_1)_{60} + \blacktriangle(N_1)_{60}$ using equation (12) from Ishihara (*in press*). $(N_1)_{60} + \blacktriangle(N_1)_{60}$ is plotted versus cyclic stress ratio (ordinate) to determine susceptibility to liquefaction for samples with a clay content less than 20 percent during a M=7.0 earthquake (Figures 2-6).

Cyclic stress ratio was determined by the methods presented in NRC (1985). Where a_{max} is the peak horizontal bedrock acceleration as determined from Mualchin & Jones (1991), r_d is the stress reduction factor that ranges from 1 at the surface to 0.9 at or below 35 feet. Total overburden and effective overburden were determined using saturated densities of 110 pcf and 130 pcf for estuary and fluvial deposits based upon samples taken near B-23L. These soil densities compare favorably to typical values of soil unit weight determined by Powers and others (1982).

In summary, the sediments are liquefiable from the ground surface to bedrock. There are occasional lenses of non liquefiable soils especially in the cobbly to bouldery sediments. Liquefaction will produce both lateral spreading and settlement. To mitigate the effects of liquefaction, the Office of Geotechnical Engineering has recommended that stone columns be placed to a minimum depth of 50 feet in the area of the proposed bridge. The stone columns will be placed around the bents and abutments in order to reduce the potential for lateral spreading.

Settlement

Foundations: Calculations provided by the Office of Geotechnical Engineering indicate that dynamic settlement due to liquefaction can be as great as 1.21 feet near the east end of Carmel Valley Creek Connector and foundations should be designed against downdrag forces along the pile.

Embankments: The Office of Geotechnical Engineering has recommended that stone columns be placed beneath the proposed approach embankments to mitigate liquefaction-induced settlement. Settlement due to placement of the embankment fills is estimated to be 0.8 feet at Abutment 1 and 3.0 feet at Abutment 9 using the Hough Method.

RECOMMENDATIONS

Additional Studies

The Office of Geotechnical Engineering should review and comment on the liquefaction susceptiblity at the site and conduct additional studies as they deem necessary. The tip elevations of the stone columns should be specified by the Office of Geotechnical Engineering because of the varying thicknesses of liquefiable materials.

Seismic Hazards

Ground rupture is not a hazard at the site and, therefore, no special mitigative measures are required. Preliminary design of the bridge should be completed using a peak horizontal bedrock acceleration of 0.5 gravity and a depth to "rock-like" material is 70 to 115 feet. Final design should be based upon the site specific acceleration response study from the Office of Geotechnical Engineering.

Foundations

Foundations for the proposed bridge should be driven HP14 x 89 steel H-sections or 13 5/8 inch diameter, 1/2-inch thick wall pipe piles (open or closed end). The conical shaped tip is required for the pipe piles; the flat plate end is not an option. For both the open ended pipe and the H-section pile, tip protection is required. Concrete piles are not considered a viable alternative.

As required by Design Section 10, the allowable compressive capacity of the piles is 100 tons with a tension capacity of 30 tons. Pile capacities were calculated using the SPT method outlined by the FHWA and a minimum factor of safety of 2.0. Piles may be designed using the following table.

SUPPORT LOCATION	BOTTOM OF FOOTING ELEVATION	SPECIFIED TIP ELEVATION*	PILE LENGTH (FEET)	ULTIMATE COMPRESSIVE LOAD (TONS)	ULTIMATE TENSION LOAD FOR SEISMIC DESIGN (TONS)
Abut 1	+26.0	-90.0	116	220	45
Bent 2	+14.0	-87.0	101	220	45
Bent 3	+15.0	-76.0	91	220	45
Bent 4	+16.0	-70.0	86	220	45
Bent 5	+7.0	-50.0	57	220	45
Bent 6	+24.0	-49.0	75	220	45
Bent 7	+25.0	-52.0	77	220	45
Bent 8	+22.0	-53.0	75	220	45
Abut 9	+45 to 48	-33	78 to 81	220	45

*Probable Tip Elevations are estimated to be within 5 feet of specified tip.

Pile Load Tests

Dynamic and static (compressive and tension) pile load tests are planned for three locations within the interchange under the direction of the Office of Geotechnical Engineering. It is recommended that at least one of the load tests be performed in an area where the ground has been improved with stone columns and another in an area where no ground improvement has been done. Static load tests should be performed on the same day as driving to reduce the effects of soil set up. These tests should be performed prior to the driving of production piles for the bridge so that additional recommendations regarding the pile driving or construction sequence may be made if necessary. The location, specifications and layout for the pile load tests will be provided by the Office of Geotechnical Engineering.

Settlement

Foundations: Static and dynamic settlement of the foundations should be negligible because piles will be founded into the underlying bedrock. Piles founded into the bedrock will resist downdrag (FHWA, 1986).

Embankments: After embankment fills have been placed to full height, an additional ten (10) foot high surcharge is recommended on the 100 feet of embankment closest the bridge. Settlement platforms should be installed and monitored by the Resident Engineer. A minimum settlement period of at least 120 days should be observed to allow for the approximately 0.8 to 3.0 feet of settlement at Abutments 1 & 9, respectively; however, this settlement period may be accelerated by the installation of the stone columns. The settlement is complete when the rate of settlement is less than 1/4 inch over 10 consecutive days. The actual settlement period shall be determined by the engineer in the field.

Corrosion Protection

The samples tested in the area are generally noncorrosive; however, this does not preclude the possibility of corrosive layers unidentified by our testing. Considering the depositional environment, concrete below ground should be resistant to sulfates and organics. The heavier than normal steel piling will compensate for the limited areas that may exhibit corrosive properties.

Approach Slabs

Seismic approach slabs will be required at both abutment locations.

Construction Specifications

The construction sequence should be as follows:

1. **Stone columns installed.**
2. **Embankments placed to full height with surcharge and settlement platforms installed.**
3. **Settlement period observed.**
4. **Piles driven.**

This sequence is recommended for all support locations, including bents, to increase ground stability and access during pile driving.

Predrilling may be required through the embankments fills to elevation +21. Hard driving (in excess on 150 ton ENR bearing) may be anticipated to attain specified tip elevation. The Special Provisions should state that if difficult driving is encountered, this office should be contacted prior to submission of pile driving alternatives (i.e.- jetting or predrilling) to the contractor.

The Special Provisions should state that the conical tip, or equivalent, is the only type of tip allowed for the closed end pipe piles. The Structure Representative should monitor initail pile installation efforts to evaluate the effect of the closed end on the driving. It is the option of the Structure Representative to remove the tip after consulting with this office.

Ground and surface water will effect construction. The contractor may be required to mitigate the effects of surface water in order to work. District 11 Environmental Planning should provide recommendations regarding restrictions on the work area.

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If you have any further questions, please do not hesitate to call (213) 620-3780 (ATSS-640-3780).

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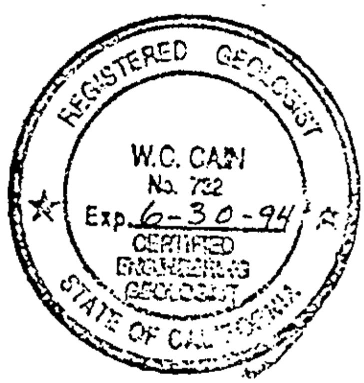

Enclosures: Figures 1-6

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