

Memorandum**To:** MR. TOM POLLOCK, Chief
Office of Structure Design**Date:** September 18, 1992Attention: Mr. Bob Anderson
Design Section 59-234**File:** 11-SD-5-R32.72
11203 030111**From:** DEPARTMENT OF TRANSPORTATION
Division of New Technology, Materials & Research
Office of Engineering Geology - SouthCarmel Valley
Creek Bridge Connector
Bridge No. 57-0590G #8**Subject:** FOUNDATION RECOMMENDATIONS**INTRODUCTION**

This office has completed a subsurface investigation at the proposed Carmel Valley Creek (Replace) (Bridge No. 57-0590) on Route 5, San Diego, CA. The investigation was a joint effort with District 11 Materials and consisted of drilling five rotary borings, six 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 25, 1990 and the General Plan received December 13, 1991.

The proposed 437.5 foot long, concrete slab bridge will be constructed just east of the proposed Carmel Valley Creek Bridge (Br. No. 57-0590) and connect northbound Route 5 traffic to eastbound Route 56. Approach fills will be placed to an approximate height of 15 feet. The bridge will be designed using 45 ton (compressive load) driven piles at the abutments and 70 ton piles at bent locations with no tension load.

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 Bridge is 0.17 miles south of Carmel Valley Road Overcrossing along Route 5. Carmel Valley Creek flows east to west beneath the eight-lane highway through the existing triple-box culvert. Carmel Valley Creek is a tributary of the Soledad Valley estuary. The surrounding ground on either end of the culvert is undeveloped. In this area Carmel Valley Creek is a sinuous, perennial stream that shows no incision below the active flood plain.

Exploratory borings reveal the bridge site is mantled by artificial fill that is underlain by Holocene estuary and alluvial deposits (Power and others, 1982), that overlie Eocene bedrock. The slightly compact to compact artificial fill supports the existing roadbed and varies in thickness from 12 to 18 feet. The estuary deposits (Qhe) are very loose to loose, dark grey to blue to black, fossiliferous, silty sands to micaceous silts and highly plastic clays that are highly organic with abundant root traces. Underlying the estuary deposits is 60 to 70 feet of brown to gray

fluvial deposits (Qhfl) that are slightly compact to compact silty sand to sand with lenses of clay . At the base of the fluvial deposits are varying thicknesses of very dense, cobbly to bouldery sands to sandy boulders. Between elevations -72 and -99 feet in borings 24L, 26L and 27L, the Holocene deposits overlie moderately-cemented, green to brown, Eocene mudstones and sandstones of the Delmar Formation (Td).

Ground Water

Ground water was estimated at 10.0 feet above sea level. The elevation of the ground water surface is highly dependent upon the seasonal rainfall. In general, 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 during heavy rains.

Scour

Evidence of scour was not found in the exploratory borings. Oblique and vertical aerial photographs of the area indicate that deposition or aggradation is occurring at the site since the installation of a sewer line to the west.

Corrosivity

Corrosion tests performed in the area indicate that the soils are, in general, non-corrosive. 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-24L @ 20	9	22	31	CL
B-24L @ 25	8	37	43	ML/OL
B-24L @ 30	np**	np	np	SM/SC
B-24L @ 35	np	np	np	SM/SC
B-24L @ 40	8	26	34	ML/OL
B-24L @ 45	4	32	36	ML/OL
B-24L @ 50	np	np	np	SM/SC
B-26L @ 20	8	20	28	CL
B-26L @ 30	np	np	np	SM/SC
B-26L @ 37	4	23	27	ML/OL
B-26L @ 40	np	np	np	SM/SC

*Unified Soil Classification

** np=non plastic

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 (V_s greater than 2,500 feet per second) is 96 to 122 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 (Q_{hfl}) and Holocene estuarine (Q_{he}) 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, 3 and 4 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, 3 and 4 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 $(N_1)_{60} + \Delta(N_1)_{60}$ using equation (12) from Ishihara (*in press*). $(N_1)_{60} + \Delta(N_1)_{60}$ 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, 3 and 4).

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, at the center of Carmel Valley (B-26L) along Carmel Valley Creek, the sediments are liquefiable between the depths of 45 to 80 feet and will not produce liquefaction-induced ground damage, but may produce settlement during a seismic event (Figure 5). Away from Carmel Valley Creek (B-24L & B-27L), the sediments ranging between the depths of 20 to 80 feet are liquefiable and will produce surface manifestations.

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 1.07 feet near Carmel Valley Creek 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 using the Hough Method.

RECOMMENDATIONS

Additional Studies

The Office of Geotechnical Engineering should review and comment on the liquefaction susceptibility at the site and conduct additional studies as they deem necessary. The tip elevations of the stone columns should be specified at each location because of the varying depths of liquefiable material.

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 90 to 120 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 alternatives.

As required by Design Section 10, the bridge will be supported by 45 ton piling at bridge abutments and 70 ton piles at the bents. The heavier than normal pile sections are required for drivability and seating into bedrock. 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	SPECIFIED TIP ELEVATION*	PILE LENGTH (FEET)**	ULTIMATE COMPRESSIVE LOAD (TONS)	ULTIMATE TENSION LOAD FOR SEISMIC DESIGN (TONS)
Abut 1	-80	102	200	90
Bent 2	-80	102	200	90
Bent 3	-80	102	200	90
Bent 4	-80	102	200	90
Bent 5	-80	102	200	90
Bent 6	-90	112	200	90
Bent 7	-90	112	200	90
Bent 8	-95	117	200	90
Bent 9	-95	117	200	90
Bent 10	-95	117	200	90
Bent 11	-95	117	200	90
Abut 12	-90	112	200	90

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

** measured from bridge soffit

File 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.

Scour

Scour is not a hazard to the bridge and no special mitigative measures are required.

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: At abutment 1, the plans show that less than 5 feet of fill will be placed. A 30 day stabilization period is recommended.

After embankment fill have been placed to full height at butment 12, an additional ten (10) foot high surcharge is recommended on the 150 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 feet of settlement; 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 be corrosive.

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 +24. 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 initial 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 1-5

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