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-R32.72
11203 030111

Carmel Valley
Creek (Replace)
Bridge No. 57-0590

**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 (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 421.5 foot long, concrete slab bridge will replace the existing triple-box culvert. The existing fill will be removed to elevation 12. 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, fossiliferrous, 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 flow 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 (Vs greater than 2,500 feet per second) varies from 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 (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, 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 $(N_1)_{60} + \blacktriangle(N_1)_{60}$ 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, 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 approach approach embankments to mitigate liquefaction-induced settlement. Less than five feet of new fill will be placed at each abutment.

## 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 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 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

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.

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:** A 30 day settlement period after the placement of the five feet of new fill is advised before continuing abutment construction.

Corrosion Protection

The samples tested in the area are generally non corrosive; 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 soil 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 of 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.

## REFERENCES CITED

Federal Highway Administration, 1986, Manual on Design and Construction of Driven Pile Foundations: U S Department of Transportation, Washington, D. C., p. 684.

Hart, E. W., 1990, Fault-rupture hazard zones in California: California Division of Mines & Geology Special Publication 42.

Ishihara, K., 1985, Stability of natural deposits during earthquakes; Proceedings of 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, California, v. 1, 321-376.

Ishihara, K., *in press*, Evaluation of Liquefaction potential and consequent deformations in sand fills.

Mualchin, L. & Jones, A. L., 1991, Peak acceleration from maximum credible earthquakes in California: California Division of Mines & Geology Open File Report 87 & Map Sheet 45.

National Research Council, 1985, Liquefaction of soils during earthquakes; National Academy Press, Washington, D.C.; Report No. PB86-163110, pp. 240.

Power, M. S., Dawson, A. W., Streiff, D. W., Perman, R. C. & Berger, V., 1982, Evaluation of liquefaction susceptibility in the San Diego, California urban area; v.1, U. S. Geological Survey Technical Report No. 14-08-0001-19110.

Reichle, M. S. and others, 1990, Planning Scenario for a Major Earthquake, San Diego-Tijuana Metropolitan Area; California Division of Mines and Geology Special Publication 100, pp. 181.

Seed, H. B. and Idriss, I. M., 1982, *Ground Motion and Soil Liquefaction during Earthquakes*; Earthquake Engineering Research Institute Monograph, v. 5, pp. 134.

Tokimatsu, K. and Seed, H. R., 1987, Evaluation of settlements in sands due to earthquake shaking: Journal of Geotechnical Engineering, v. 113, no. 8, 861-868.

Tokimatsu, K. and Yoshimi, Y., 1983, Empirical correlatin of soil liquefaction based upon SPT N-value and fines content: Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, v. 23, n. 4, p. 56-74.

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

Report by

Jeffrey R. Knott

JEFFREY R. KNOTT
Associate Engineering Geologist
Office of Engineering Geology

Reviewed by:

W. C. Cain

W. C. CAIN, C.E.G. 732
Senior Engineering Geologist
Office of Engineering Geology

REGISTERED GEOLOGIST
W.C. CAIN
No. 732
Exp. 6-30-94
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

cc: NTM&R: W. C. Cain (2)
E. Leivas
R. Prysock
File
District Materials
District Design
PISouth
R. E. Pending

Enclosures: Figures 1-5

57-0590
JK;hs

State of California | Business, Transportation and Housing Agency

# Memorandum

**To:** MR. TOM POLLOCK, Chief
Office of Structures Design

Attention: Mr. Bob Anderson
Design Section 59-234

**Date:** September 18, 1992

**File:** 11-SD-05-32.88
11203 030111

Carmel Valley Rd. UC #4
Bridge No. 57-0991G

**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 Road Undercrossing (Bridge No. 57-0991G) on Route 5, San Diego, CA. The investigation was a joint effort with District 11 Materials and consisted of drilling ten rotary borings, two electric cone penetrometer (CPT) soundings, reviewing the site conditions and the available records. Our investigation was based upon conversations with Design Section 10, the Foundation Plan received September 28, 1989 and the General Plan received August 19, 1991. The field investigation is a compilation of work completed by this office and District 11 Materials.

The proposed bridge will be a multi-span, prestressed-concrete box girder. Spans lengths range from 91.5 to 125 feet. Bents 2, 3 and 5 will be single-column supports and Bent 4 a multi-column support. Approach fills will be placed at each abutment to a maximum height of approximately 20 feet. The bridge will be designed for 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 should 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 Road UC is east of the existing Carmel Valley Road Overcrossing on Route 5. The area of the proposed bridge is an undeveloped shoulder within the existing State right-of-way. The ground surface is relatively level south of Carmel Valley Road, ascending at a 2:1 gradient north from Carmel Valley Road. Carmel Valley Creek, an east-west flowing tributary of the Soledad Valley estuary, is south of the proposed bridge. In this area the creek is a sinuous, perennial stream that shows no incision below the flood plain. The site is covered with native and ornamental plants and grasses.

Exploratory borings reveal the subsurface at the bridge site consists of Holocene-age estuary and alluvial deposits (Power and others, 1982) overlying Eocene bedrock and mantled by artificial fill. The artificial fills at the surface consist of loose, slightly moist to moist, brown, silty to clayey sands. The estuary deposits (Qhe) are very loose to loose dark, fossiliferrous gray silty sands to micaceous silts and clays. These silts and clays are interbedded with gray to light gray to brown, slightly compact to very dense fluvial sands to silty sands (Qhfl). The base of the fluvial deposits is well defined in most areas by dense to very dense, gravelly to cobbly sands to cobbles that overlies the Delmar Formation. The elevation of the top of the Delmar Formation (Td) beneath the bridge varies from elevation -38.1 to -76.4. The Delmar Formation consists of poorly to moderately cemented, green to brown, Eocene mudstones and sandstones.

Ground Water

Ground water was measured between elevations 13.9 and 18.7 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.

Corrosivity

The following table lists the results from soil samples taken in borings near the bridge site and tested for corrosivity (California Test 643).

| Sample | Sample Type | pH | Min. Resistivity (Ohm-cm) | Soluble Sulfates (ppm) | Soluble Chlorides (ppm) | Years |
|---|---|---|---|---|---|---|
| B-2L @ 20 | SM | 8.1 | 1026 | - | - | 25 |
| B-4L @ 30 | SM/OL | 7.7 | 958 | 595 | 102 | 25 |
| B-4L @ 65 | SM | 7.6 | 1436 | - | - | 29 |
| B-5L @ 70 | Td | 7.8 | 821 | 144 | 60 | 23 |
| B-5L @ 75 | Td | 7.8 | 821 | 144 | 60 | 23 |
| R-22 @ 45 | SM | 7.6 | 1915 | - | - | 33 |
| R-22 @ 65 | SM | 7.4 | 2257 | - | - | 35 |
| R-22 @ 75 | Td | 7.7 | 2599 | - | - | 37 |
| R-22 @ 100 | Td | 8.1 | 889 | 129 | 83 | 24 |
| B-17L @ 30 | SM/ML | 7.5 | 1642 | - | - | 31 |
| B-18L @ 55 | SM/ML | 7.4 | 1402 | - | - | 29 |

The CALTRANS Corrosion Unit classifies sulfates in excess of 2,000 ppm and chlorides in excess of 500 ppm as corrosive. The number of years represents the length of time to perforate an 18 gage galvanized steel culvert. The limited testing indicates that the deposits in the area of the Carmel Valley Road UC are not corrosive to steel and concrete.

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.

Plasticity

The results of testing for Atterberg's Limits (California Test 204) are shown in the table below. Figure 1 is a plot of the Plasticity Index vs. Liquid Limit from the alluvial soils listed below compared to the limits of liquefiable soils (after Tokimatsu & Yoshimi, 1983).

| Sample (Boring # and depth) | Plasticity Index | Plastic Limit | Liquid Limit | Soil Type* |
|---|---|---|---|---|
| B-2L @ 30(A) | 13 | 28 | 41 | ML |
| B-2L @ 120 | 12 | 36 | 48 | Td |
| B-2L @ 85 | 4 | 20 | 24 | ML |
| B-4L @ 60(A) | np** | np | np | SM/SC |
| B-5L @ 25 | np | np | np | SM/SC |
| B-5L @ 80 | 24 | 23 | 47 | Td |
| B-5L @ 85 | 4 | 20 | 24 | Td |
| B-5L @ 120 | 7 | 36 | 43 | Td |
| B-6L @ 20 | np | np | np | SM |
| B-6L @ 90 | 9 | 32 | 41 | Td |
| B-8L @ 80 | 7 | 27 | 34 | Td |

*Unified Soil Classification or Formation abbreviation

** np=nonplastic

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) varies from 62 to 103 feet below existing ground. 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 just 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 potential of experiencing ground failure due to liquefaction during an earthquake on the Silver Strand fault in Mission Bay.

Figures 2-9 are illustrations of the liquefaction susceptibility of the deposits underlying the Carmel Valley Road Undercrossing and nearby locations. A figure for B-4L was not prepared because it is adjacent to B-5L. 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 2-9 show cyclic stress ratio versus normalized blow counts (adjusted for fines content) for various depths below present ground surface within each boring. The points plotted on the figures were determined by the method described 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*). Samples with less than 20% clay (0.005 mm) were considered liquefiable (Seed and Idriss, 1982) and $(N_1)_{60}$ was then converted to $(N_1)_{60} + \blacktriangle (N_1)_{60}$ using equation (12) from Ishihara (*in press*). This number $[(N_1)_{60} + \blacktriangle (N_1)_{60}]$ 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.

Cyclic stress ratio was determined by the method presented in NRC (1985). Where $a_{max}$ is the peak horizontal bedrock acceleration as determined from Mualchin & Jones (1987), $r_d$ is the stress reduction factor of 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-24L. These soil densities compare favorably to typical values of soil unit weight determined by Powers and others (1982).

Figures 2-9 show that, with a few exceptions, the sediments beneath the bridge site from a depth of 10 feet below the ground surface to the top of the bedrock (as great as 103 feet) are liquefiable. The greatest thickness of liquefiable soils are to the south (Abutment 1) tapering to 40 feet at B-10L north of Abutment 6. Comparison with the CPT testing in the area shows that the general stratigraphy is relatively uniform with a very loose zone approximately 30 to 50 feet thick near the surface underlain by a zone of interbedded liquefiable sands and silts grading to a cobble or gravel zone above the bedrock. The thickness of the interbedded and gravelly zones is variable.

Liquefaction will create surface manifestations, possibly in the form of lateral spreading, ground oscillations and sand boils, in addition, to ground settlement. To mitigate the effects of liquefaction, the Office of Geotechnical Engineering has recommended that stone columns be placed to a depth of 50 feet in the area of the proposed bridge. The stone columns will be placed around the bents and abutments 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 1.1 feet near the Carmel Valley Road UC 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 approach embankments at the Carmel Valley Road UC to support the embankments; however, no construction sequence was provided in the memorandum dated August 17, 1992 or June 22, 1992. Calculations using the Hough Method estimated a ground settlement of 0.5 feet at the approach abutments after surcharge by the approach embankments.

## RECOMMENDATIONS

Additional Studies

The Office of Geotechnical Engineering should review this report and the liquefaction susceptibility at the site and perform additional studies as they deem necessary.

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 62-103 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 minimum 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 | +9.5 | -83.0 | 92.5 | 220 | 45 |
| Bent 3 | +14.5 | -66.0 | 80.5 | 220 | 45 |
| Bent 4 East | +14.5 | -50.0 | 64.5 | 220 | 45 |
| Bent 4 West | +14.5 | -46.0 | 60.5 | 220 | 45 |
| Bent 5 | +12.0 | -69.0 | 81.0 | 220 | 45 |
| Abut 6 | +36.0 | -49.0 | 85.0 | 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 2.1 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 were all non-corrosive; however, this does not preclude the possibility of corrosive layers unidentified by our testing. The heavy H-section and thick walled pipe pile should mitigate the effects of corrosion during the design life of the foundations.

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 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 elevations +20 and +28 at Abutments 1 & 6, respectively. Hard driving (in excess on ENR 100 ton bearing) may be anticipated to attain the specified pile 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.

## REFERENCES CITED

Hart, E. W., 1990, Fault-rupture hazard zones in California: California Division of Mines & Geology Special Publication 42.

Ishihara, K., 1985, Stability of natural deposits during earthquakes; Proceedings of 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, California, v. 1, 321-376.

Ishihara, K., *in press*, Evaluation of Liquefaction potential and consequent deformations in sand fills;

Kennedy, M. P., 1975, Geology of the San Diego Metropolitan area, California; California Division of Mines and Geology Bulletin 200, p. 56.

Mualchin, L. & Jones, A. L., 1991, Peak acceleration from maximum credible earthquakes in California: California Division of Mines & Geology Open File Report 87 & Map Sheet 45.

National Research Council, 1985, Liquefaction of soils during earthquakes; National Academy Press, Washington, D.C.; Report No. PB86-163110, pp. 240.

Power, M. S., Dawson, A. W., Streiff, D. W., Perman, R. C. & Berger, V., 1982, Evaluation of liquefaction susceptibility in the San Diego, California urban area; v.1, U. S. Geological Survey Technical Report No. 14-08-0001-19110.

Reichle, M. S. and others, 1990, Planning Scenario for a Major Earthquake, San Diego-Tijuana Metropolitan Area; California Division of Mines and Geology Special Publication 100, pp. 181.

Seed, H. B. and Idriss, I. M., 1982, *Ground Motion and Soil Liquefaction during Earthquakes*; Earthquake Engineering Research Institute Monograph, v. 5, pp. 134.

Tokimatsu, K. and Seed, H. R., 1987, Evaluation of settlements in sands due to earthquake shaking; Journal of Geotechnical Engineering, v. 113, no. 8, 861-868.

Tokimatsu, K. and Yoshimi, Y., 1983, Empirical correlation of soil liquefaction based upon SPT N-value and fines content; Soils and Foundations, v. 23, no. 4, 56-74.

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

Report by

JEFFREY R. KNOTT
Associate Engineering Geologist
Office of Engineering Geology

Reviewed by:

W. C. CAIN, C.E.G. 732
Senior Engineering Geologist
Office of Engineering Geology

REGISTERED GEOLOGIST
W.C. CAIN
No. 732
Exp. 6-30-94
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

cc: NTM&R: W. C. Cain (2)
E. Leivas
R. Prysock
File
District Materials
District Design
PISouth
R. E. Pending

Enclosures: Figure 1-10

57-0991G
JK;hs