# FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: **Structure Design**
1. Design
2. R.E. Pending File
3. Specifications & Estimates
4. File

**Geotechnical Services**
1. GD - North ; South ; West
2. GS File Room

Date: 8/30/02

Carmel Mountain Rd. OC
Structure Name

11 - SD - 15 - 33.0
District County Route km Post

**District** Project Development
District Project Engineer

11-080901 57-1125
E.A. Number Structure Number

Foundation Report By: H. Valencia

Dated: 8/13/02

Reviewed By: S. Trouve (SD) R. Price (GS)

General Plan Dated: 8/7/02

Foundation Plan Dated: 8/14/02

☒ No changes. ☐ The following changes are necessary.

**FOUNDATION CHECKLIST**

- ✓ Pile Types and Design Loads
- ✓ Pile Lengths
- ✓ Predrilling
- ✓ Pile Load Test
- ✓ Substitution of H Piles For Concrete Piles ☐ Yes ☐ No
- ✓ Footing Elevations, Design Loads, and Locations
- ✓ Seismic Data
- ✓ Location of Adjacent Structures and Utilities
- ✓ Stability of Cuts or Fills
- ✓ Fill Time Delay
- ✓ Effect of Fills on Abutments and Bents
- __ Fill Surcharge
- ✓ Approach Paving Slabs
- __ Scour
- __ Ground Water
- ✓ Tremie Seals/Type D Excavation

[signature] 12
Structure Design Bridge Design Branch No.

[signature]
Geotechnical Services

Rev. 06/02

State of California

Business, Transportation and Housing Agency

# Memorandum

*Flex your power!*
*Be energy efficient!*

To: MR. KEVIN ROSS
Structures Design
Office of Bridge Design South
Bridge Design Branch 12
MS - 9 - 3/3G

Attention: Kenny Kwong

Date: August 13, 2002

File: 11-SD-15-KP 31.3
11-080901
Carmel Mountain
Overcrossing (Replace)
Br. No. 57-1125

From: **DEPARTMENT OF TRANSPORTATION**
**DIVISION OF ENGINEERING SERVICES**
**Geotechnical Services**
**Office of Geotechnical Design - South MS #5**

Subject: Foundation Recommendations

## Introduction

This report presents the foundation recommendations for the proposed Carmel Mountain Overcrossing, Br. No. 57-1125, which will replace the existing Carmel Mountain Overcrossing, Br. No. 57-0576. The Structure Foundations 2, Branch F (SF2BF) of the Office of Geotechnical Design South (OGDS) completed a foundation investigation pursuant to the March 8, 2002 request by the Office of Bridge Design South (OBDS) for a foundation investigation and recommendations for the proposed widening.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (March/April 2002) along with a review of the previous foundation reports, "As-Built" records and Log of Test Borings (LOTB) for the existing bridge. With regards to the current foundation recommendations given in this report, elevations are based on NAVD 88 vertical datum and horizontal coordinates are based on CCS 83 horizontal datum, unless noted otherwise.

## Project / Site Description

The existing structure site is located in the Rancho Penasquitos area where Carmel Mountain Road and Route 15 intersect. At this location, Carmel Mountain Overcrossing presently consists of a 4-lane road, which crosses over Route 15 and consists of an 8 lane divided highway. The existing bridge two span bridge was constructed in 1983 and is a cast-in-place, pre-stressed concrete box girder type structure supported on driven steel "H" pile foundations.

The proposed bridge, which measures approximately 89.1 m in length and 37.0m in width, will replace the existing bridge. The new bridge will consist of a three span, cast-in-place, pre-stressed concrete box girder type structure, which will accommodate the proposed Route 15 managed lanes. The layout of the proposed widened structure is shown on the Carmel Mountain Overcrossing, General Plan No. 1 & 2, provided by OBDS and dated April 9, 2002.

**Geology**

The recent foundation investigation performed for the proposed widening consisted of nine mud-rotary, sampled borings advanced with wireline-diamond coring methods extending down to a maximum depth of 33.5 m (110.0 ft). The March 2002 foundation investigation revealed that earth materials encountered at the site can be generally separated into three units.

At the Abutment 1 location, the northwest corner of the bridge was constructed partially within a cut in the existing hillside. The upper unit is a thin layer of fill material consisting of clayey sand. These fill soils extend from the ground surface to a maximum depth of 1.2 meters (elevation 198.5 m) at the Boring B-5-02 location. The fill soils are underlain by sedimentary rock that consists of decomposed, very soft to soft sandstone and claystone. This sedimentary rock was encountered to a depth of 9.7 m (elevation 190.0 m). The lowest unit is an altered igneous rock that is typically decomposed to very intensely weathered, soft to moderately soft with localized moderately hard zones. Typically, the altered igneous rock when mechanically broken down has the physical properties of a clayey silt with minor fine sand. At the Abutment 1 location, this unit was encountered in all borings drilled in March 2002 to the maximum explored depth 19.6 m (elev. 180.1 m). The 1982-1983 As-Built LOTB sheet identifies the top of igneous rock varied across the existing Abutment 1 location from elevation 191.1 m (Boring B-1) to elevation 187.5m (Boring B-8).

At the Bent locations, the existing Route 15 is partially located within a cut. Minor variations in the depth to top of rock and the type of rock were encountered. The upper unit is a thin layer of fill material consisting of silty/clayey sand, sandy clay and lean clay. These fill soils extend from the ground surface to a maximum depth of 4.6 meters (elevation 185.9 m) at the Boring B-3-02 location. The fill soils are underlain by sedimentary rock at some boring locations. At these locations, the sedimentary rock consists of decomposed, soft claystone. This sedimentary rock was encountered to a maximum depth of 2.82 m (elevation 186.1 m). The lowest unit is an altered igneous rock that is typically decomposed, soft to moderately soft with localized moderately hard zones. This unit was encountered in all borings to the maximum explored depth 33.5 m (elev. 156.6 m).

At the Abutment 4 location, the upper unit is a thick layer of fill material consisting of silty/clayey sand, sandy clay and lean clay with scattered hard cobbles and boulders. These fill soils extend from the ground surface to a maximum depth of 8.22 meters (elevation 187.3 m) at Boring B-9-02 location. The fill soils are underlain by alluvium that consists of stiff to very stiff, lean and fat clay. The lowest unit is an altered igneous rock that is typically decomposed to very intensely weathered, soft to moderately soft with localized moderately hard zones. At Abutment 4 location, this unit was encountered to the maximum explored depth 22.9 m (elev. 170.2 m). The 1982-1983 As-Built LOTB sheet shows that top of igneous rock varied across the existing Abutment 3 location from elevation 183.5 m to elevation 182.0 m.

The 1983 field investigation for the existing structure, revealed similar earth materials at the site with the addition of boulders, which were identified at both the existing Abutment 1 and 3 locations.

## Groundwater

The 1982-1983 LOTB sheet indicated that groundwater was not encountered at the site. However, during the 2002 field investigation, groundwater was periodically measured at approximately elevation 180.5 m and is shown on the recent LOTB.

## Corrosion

Soil samples collected during the 2002 foundation investigation were combined from three borings (B-2-02, B-3-02 & B-4-02) to make composite samples of native earth materials at depth. The Office of Testing and Technology Services, Corrosive Technology Branch (CTB) tested the composite sample for corrosive potential. The results of the laboratory tests determined that the composite sample was corrosive. Results from Refer to Table 1 below for specific test results.

**Table 1: Corrosion Test Summary-Composite Samples for Boring B-3-02**

| Support Location/ Corrosion Number | Sample Depth (m) | pH | Minimum Resistivity (Ohm-Cm) | Sulfate Content (PPM)* | Chloride Content (PPM)* | Years To Perforation 18 ga. Galv. Steel Culvert |
|---|---|---|---|---|---|---|
| Abut 1 / 02-0370 | 0 to 1.5 | 8.60 | 715 | 116 | 42 | 22 |
| Abut 1 / 02-0369 | 3.1 to 6.2 | 8.62 | 490 | 182 | 52 | 19 |
| Abut 1 / 02-0371 | 10.5 to 18.3 | 7.82 | 325 | 186 | 516 | 16 |
| Bent 2 / 02-0251 | 0 to 1.5 | 6.92 | 540 | 31 | <30 | 14 |
| Bent 2 / 02-0252 | 1.5 to 3.2 | 8.41 | 900 | <30 | <30 | 24 |
| Bent 2 / 02-0253 | 3.2 to 6.2 | 8.37 | 800 | <30 | <30 | 23 |
| Bent 3 / 02-0210 | 0 to 1.5 | 8.53 | 350 | 48 | 650 | 16 |
| Bent 3 / 02-0211 | 5.6 to 9.1 | 8.95 | 750 | 69 | 38 | 22 |
| Abut 4 / 02-0367 | 0 to 4.6 | 8.17 | 590 | 149 | 90 | 14 |
| Abut 4 / 02-0368 | 4.6 to 10.7 | 7.68 | 430 | 88 | 333 | 18 |

*The Corrosion Technology Branch policy states that if the minimum resistivity is greater than 1000ohm-cm the sample is considered to be non-corrosive and testing to determine sulfate and chloride contents are not performed.

Corrosion recommendations for the proposed Carmel Mountain Overcrossing (Br. No. 57-1125) were developed by Mike Piepoli of the Corrosion Technology Branch. Specific questions concerning the corrosion recommendations should be directed to Mr. Douglas Parks at 916-227-7068.

**Seismic Data**

The site is potentially subject to strong ground motions from nearby earthquake sources during the design life of the new structure. The Newport-Inglewood Rose Canyon Fault/E (NIE, Strike-Slip) fault located approximately 18 km southwest from the site is the controlling fault for this site with a maximum credible earthquake of Mw=7. The Peak Bedrock Acceleration at this site, based on the Caltrans California Seismic Hazard Map, is estimated to be 0.3g. At this site, the liquefaction potential is considered to be minimal.

For site specific seismic data and design recommendations, refer to the memorandum concerning final seismic design recommendations dated July 18, 2002, by Asef Wardak of the Office of Geotechnical Earthquake Engineering.

**"As-Built" Information**

The original foundation report (1977) and As-Built records (1982-1983) indicate that the bridge foundations for the existing structure consist of driven steel "H" piles at the abutments and Spread footing foundations at Bent 2. At the Abutment 1 and 4 locations, driven steel "H" piles with a design load of 625 kN (70 tons) were used for support. At the abutments, predrilled holes extended to elevation 195.1 m (640 ft: NGVD29) and elevation 185.0 (607 ft; NGVD29) for Abutment 1 and 4, respectively. The As-Built pile tip elevations and As-Built spread footing information for the bridge are listed below in Table 2 and Table 3.

**Table 2. "As-built" Driven Steel 250X85 "H" Piles with 625 kN (70 ton) Design Load**

| Location | Specified Pile Tip Elevation | Minimum "As-built" Pile Tip Elevation | Average "As-built" Pile Tip Elevation | Maximum "As-built" Pile Tip Elevation |
|---|---|---|---|---|
| Abutment 1 | 187.5 m (615.0 ft) | 187.3 m (614.4 ft) | 187.6 m (615.4 ft) | 188.8 m (619.4 ft) |
| Abutment 3 | 181.4 m (595.0 ft) | 178.4 m (585.3 ft) | 180.7 m (592.8 ft) | 182.1 m (597.4 ft) |

Note: As-Built Elevations shown above are based on the NAVD29 vertical datum.

**Table 3. "As-built" Spread Footing Table**

| Location | Allowable Soil Pressure | Bottom of Footing Elevation (Max) |
|---|---|---|
| Bent 2 | 383 kPa (4.0 tsf) | 184.4 m (605 ft) |

Note: As-Built Elevations shown above are based on the NAVD29 vertical datum.

**Foundation Recommendations**

The following recommendations are for the proposed Carmel Mountain Overcrossing (Br. No. 57-1125), as shown on the General Plan No. 1 & 2, dated April 9, 2002. At the Abutment 1 location, a spread footing foundation is recommended for support. At the Bent 2 and 3 locations, large diameter CIDH shafts will be used for support. At the Abutment 4 location and all retaining wall locations, driven steel "H" piles will be used for support.

Bridge Foundations

At the Abutment 1 location, subsurface information indicates the proposed bottom of footing elevation will have the footing partially situated on near surface fill and weathered igneous rock. In order to eliminate differential settlement across the proposed Abutment 1, sub-excavation of earth materials below the proposed Abutment 1 bottom of footing elevation down to elevation 186.2 m and replacement with Class 4 concrete is recommended. The Gross Allowable Soil Bearing Pressure to be used for design is listed below in Table 4.

**Table 4: Spread Footing Data Table (Bridge No. 57-1125)**

| Support Location | Minimum Footing Width | Bottom of Footing Elevation | Sub-Excavation Elevation | Recommended Soil Bearing Pressures: ASD[1] Gross Allowable Soil Bearing Pressure ($q_{all}$) | Recommended Soil Bearing Pressures: LFD[2] Ultimate Soil Bearing Pressure ($q_{ult}^*$) |
|---|---|---|---|---|---|
| Abutment 1 | 5.35 m | 187.5 m | 186.2 m | 383 kPa (4 tsf) | N/A |

Notes: 1) Allowable Stress Design, (ASD). The Maximum Contact Pressure, ($q_{max}$), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, ($q_{all}$). The Ultimate Soil Bearing Capacity, ($q_{ult}$), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, ($q_{all}$).
2) Load Factor Design, (LFD). The Maximum Contact Pressure, ($q_{max}$), divided by the Strength Reduction Factor, ($\phi$), is not to exceed the recommended Ultimate Soil Bearing Pressure, ($q_{ult}^*$). The Ultimate Soil Bearing Capacity, ($q_{ult}$), will equal or exceed the recommended Ultimate Soil Bearing Pressure, ($q_{ult}^*$).

The recommended pressures to be used for design for the proposed structure support spread footings, listed above in Table 4, are based on the following criteria:

- At Abutment 1, the footing shall be supported on 1.3 meters of Class 4 concrete extending down to elevation 186.2 m. The limits of sub-excavation and replacement with Class 4 concrete shall conforms to the same limits required for relative compaction of engineered fill below retaining wall footings without piles as defined in section 19-5.03 of the Standard Specifications.
- Support footings shall have a minimum footing width of 5.35 meters at Abutment 1.
- All footings are to be constructed at or below the recommended bottom of footing elevations provided above in Table 4.

If any of the above minimum footing widths or limits of sub-excavation are reduced, the Office of Geotechnical Design South, Structure Foundations 2 - Branch F shall be contacted for reevaluation.

At Bents 2 and 3 support locations, it is possible to utilize 2.1-m Cast-In-Drilled-Hole (CIDH) shafts for support. The specified pile tip elevations, listed below in Tables 5 and 6, were developed using information received from OBDS via email on May 23, 2002. At the Abutment 4 support location, it is recommended to use 625 kN design load, HP250X85 steel "H" piles for support. The ultimate geotechnical pile capacity is two-times (2x) the specified design load for driven steel "H" piles with 625 kN design load. The ultimate geotechnical pile capacity for the CIDH piles will meet or exceed the required nominal resistance in compression listed below in Tables 5.

**Table 5: Pile Data: Proposed Carmel Mountain Overcrossing (Br. No. 57-1125)**

| Support Location | Pile Type | Design Loading | Nominal Resistance | | Pile Cut-Off Elevation (m) | Design Tip Elevation (m) | Specified Tip Elevation (m) |
|---|---|---|---|---|---|---|---|
| | | | Compression | Tension | | | |
| Bent 2 | 1.8 m CIDH | N/A | 17800 kN | 0 kN | 188.0 | 159.4 m (1) | 159.4 m |
| Bent 3 | 1.8 m CIDH | N/A | 17800 kN | 0 kN | 188.1 | 156.9 m (1) | 156.9 m |
| Abutment 4 | HP 250X85 "H" Piles | 625 kN | 1250 kN | 0 kN | N/A | 178.3 m (1) | 178.3 m |

Note: Design tip elevation is controlled by the following demands: (1) Compression

## Retaining Wall Foundations

The following foundation recommendations are for the proposed retaining wall structures located at both bridge abutment locations as shown on the foundation plan dated 7-19-02. The proposed Type 1 Retaining Wall structures at the bridge abutment locations may all be supported with 400 kN design load HP 250x63 steel "H" Piles. The specified pile tip elevations are listed below in Table 5. The ultimate geotechnical pile capacity is two-times (2x) the specified design load for driven steel "H" piles with 400 kN design load. The retaining wall locations referenced below in Table 6 were provided by OBDS on 7-9-02.

**Table 6. Pile Data: Abutment 1 and 4 Type 1 Retaining Walls (Br. No. 57-1130)**

| Location | Stationing From CL Proposed "NA" Line | Pile Type | Design Loading | Nominal Resistance | | Design Tip Elevation | Specified Tip Elevation |
|---|---|---|---|---|---|---|---|
| | | | | Compression | Tension | | |
| Abutment 1 (Left Wall) | 15.2 m Lt. Sta. 29+63.7 to 15.2 m Lt. Sta. 29+53.1 | HP 250X63 "H" Piles | 400 kN | 800 kN | -0- | 184.0 m (1) | 184.0 m |
| Abutment 1 (Right Wall) | 21.8 m Rt. Sta. 29+45.7 to 21.8 m Rt. Sta. 29+33.0 | HP 250X63 "H" Piles | 400 kN | 800 kN | -0- | 182.5 m (1) | 182.5 m |
| Abutment 4 (Left Wall) | 15.2 m Lt. Sta. 30+59.7 to 15.2 m Lt. Sta. 30+68.2 | HP 250X63 "H" Piles | 400 kN | 800 kN | -0- | 180.0 m (1) | 180.0 m |
| Abutment 4 (Right Wall) | 21.8 m Rt. Sta. 30+41.5 to 21.8 m Rt. Sta. 30+50.0 | HP 250X63 "H" Piles | 400 kN | 800 kN | -0- | 180.0 m (1) | 180.0 m |

Note: Design tip elevation is controlled by the following demands: (1) Compression

## General Notes

1. All 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.

2. 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. If the specified pile tip elevation required to meet lateral load demands exceed the specified pile tip elevation given within this report, the Office of Geotechnical Design South, Structure Foundations 2 Branch F should be contacted for further recommendations.

## Construction Considerations

1. A 30-day waiting period is required at the Abutment 1 and Abutment 4 locations, where new fill is being placed, prior to beginning any pile driving at this location.

2. At Abutment 1 location, support footing excavations are to be inspected and approved by a representative of the Office of Geotechnical Design South, Structure Foundations 2 Branch F. The inspections are to be made after the excavation has been completed to the specified sub-excavation elevation listed above in Table 4 and prior to placing Class 4 concrete in the excavation. The contractor is to allow 4 working days for inspection personnel to travel to the site and inspect the excavation.

3. At the Abutment 1 location, shear keys shall be incorporated into the foundation footing design.

4. At Abutment 1 location, it is recommended that a shear key, with adequate dimensions to accommodate the proposed shear key as shown in the abutment detail sheets, be formed in the top of the Class 4 concrete.

5. Pile acceptance at Abutment 4 location will be assessed by the ENR equation. At the abutment support location, any pile achieving two times (2x) the required design loading shown as shown on the contract plans and above in Table 5, within 3 meters of the specified tip elevation, may be considered satisfactory and cut off with the Engineers written approval. Two times the required design loading is 1250 kN.

6. Pile acceptance at all Type 1 Retaining Wall locations will be assessed by the ENR equation. At the retaining wall support locations, any pile achieving two times (2x) the required design loading shown on the contract plans and above in Table 6, within 2 meters of the specified tip elevation, may be considered satisfactory and cut off with the Engineers written approval. Two times the required design loading is 800 kN.

7. Piles at Abutments 4 location and all Retaining Wall locations shall be driven in pre-drilled holes in conformance with the provisions in Section 49-1.06, "Predrilled Holes," of the Standard Specifications, to the corresponding bottom of hole elevations listed below.

| Structure Support Location | Retaining Wall Location | Bottom of Pre-drilled Holes Elevation |
|---|---|---|
| Abutment 4 (Structure Support) | | 185.5 m |
| | Abutment 4 (Left Ret. Wall) | 185.5 m |
| | Abutment 4 (Right Ret. Wall) | 185.5 m |

8. Difficult drilling for "Predrilled Holes" should be anticipated due to the presence of very dense soils with scattered cobbles and boulders as identified in the abutment embankments. Please refer to the Log of Test Borings sheets.

9. Hard driving of the "H" piles should be anticipated. Due to the presence of hard cobbles, boulders and due to the variations in the rock weathering and the top of igneous rock elevations across the site, field cutting and splicing of all "H" piles should be anticipated.

10. At the Contractors option, driven "H" piles may have lugs installed to aid in achieving the required nominal resistance at the specified tip elevation and to avoid field splicing. Lugs shall be installed as specified in the Bridge Construction Records and Procedures Manual, Bridge Construction Memo 130-5.0.

11. Groundwater was encountered during the 2002 field investigation and it is anticipated that groundwater will be encountered during CIDH pile construction. Groundwater surface elevation is subject to seasonal fluctuations and may occur higher or lower depending on the conditions and time of construction.

12. The calculated geotechnical capacity of the CIDH shafts is based upon Skin Friction only. The geotechnical capacity of the shaft was calculated from one diameter below the pile cut-off elevation extending down to the specified tip elevation.

13. Caving conditions may be encountered during CIDH pile construction. Temporary casing may be necessary to control caving during construction. All temporary casing is to be removed during concrete placement.

14. Difficult pile installation and drilling is anticipated due to the presence of variably weathered, variable hard igneous rock described in the geology section. Hard rock drilling should be anticipated to advance the shaft excavations to the specified pile tip elevations.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by OBDS. If any conceptual changes are made during final project design, the Office of Geotechnical Design South, Structure Foundations 2 Branch F 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 Hector Valencia (916) 227-4555 (CALNET 498-4555) or Mark DeSalvatore (916) 227-5391 (CALNET 498-5391), Office of Geotechnical Design South, Structure Foundations 2 Branch F.

Prepared by: Date:

Supervised by: Date:

Hector Valencia
Associate Engineering Geologist
Office of Geotechnical Design – South
Structure Foundations 2 - Branch F

Mark DeSalvatore, RCE# 39499
Senior Materials and Research Engineer
Office of Geotechnical Design - South
Structure Foundations 2 - Branch F

cc: R.E. Pending File
John Stayton – Specs & Estimates
Tom Ruckman – Specs & Estimates
Tony Marquez – Project Mgmt
Dave Pajouhesh – PCE
Lawrence Carr – District 11
Marcelo Peinado – District 11
John Ehsan – OGDS
Geology – North
Geology - South
RGES 30

State of California

Business, Transportation and Housing Agency

# Memorandum

*Flex your power!*
*Be energy efficient!*

To: KENNY KWONG, SENIOR
Task Order Manager
Office of Structure Contract Management
MS - 12

Date: March 18, 2003

File: 11-SD-15-KP 33.10
11-080901
Carmel Mountain
Overcrossing (Replace)
Br. No. 57-1125

From: **DEPARTMENT OF TRANSPORTATION**
**DIVISION OF ENGINEERING SERVICES**
**GEOTECHNICAL SERVICES**
**OFFICE OF GEOTECHNICAL DESIGN – SOUTH 2 (MS #5)**

Subject: Revised Foundation Recommendations

## Introduction

The following revised foundation recommendations are in response to a memorandum from the Office of Structure Contract Management (OSCM) dated February 3, 2003, requesting revised foundation recommendations to prior foundation recommendations for the consultant redesigned Carmel Mountain Overcrossing replacement, Br. No. 57-1125, which will replace the existing Carmel Mountain Overcrossing, Br. No. 57-0576. This report supercedes the original foundation recommendations for the proposed Carmel Mountain Overcrossing replacement (dated August 13, 2002), Br. No. 57-1125, which was originally prepared for the Office of Bridge Design South, Branch 12.

The Office of Geotechnical Design South 2, Branch B (OGDS2B) completed a foundation investigation pursuant to the March 8, 2002, original request by the Office of Bridge Design South (OBDS) for recommendations for the originally proposed structure. The redesigned Carmel Mountain Overcrossing replacement is being designed by the consulting firm of Dokken Engineering under Contract No. 11A0713.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (March/April 2002) along with a review of the previous foundation reports, "As-Built" records and Log of Test Borings (LOTB) for the existing bridge. With regards to the current foundation recommendations given in this report, elevations are based on NAVD 88 vertical datum and horizontal coordinates are based on CCS 83 horizontal datum, unless noted otherwise.

**Project / Site Description**

The existing structure site is located in the Rancho Penasquitos area where Carmel Mountain Road and Route 15 intersect. At this location, Carmel Mountain Overcrossing presently consists of a 4-lane road, which crosses over Route 15 and consists of an 8 lane divided highway. The existing two span bridge was constructed in 1983 and is a cast-in-place, pre-stressed concrete box girder type structure supported on driven steel "H" pile and spread footing foundations.

The proposed bridge, which measures approximately 89.1 m in length and 44.3 m in width, will replace the existing bridge. The new bridge will consist of a three span, cast-in-place, pre-stressed concrete box girder type structure, which will accommodate the proposed Route 15 managed lanes passing underneath. The layout of the proposed replacement structure is shown on the Carmel Mountain Overcrossing, General Plan No. 1, 2 and 3, provided by Dokken Engineering and dated February 25, 2003.

**Geology**

The 2002 foundation investigation performed for the proposed structure consisted of nine mud-rotary, sampled borings advanced with wireline-diamond coring methods extending down to a maximum depth of 33.5 m (110.0 ft.). The 2002 foundation investigation revealed that earth materials encountered at the site can be generally separated into three units.

At the Abutment 1 location, the northwest corner of the bridge was constructed partially within a cut in the existing hillside. The upper unit is a thin layer of fill material consisting of clayey sand. These fill soils extend from the ground surface to a maximum depth of 1.2 meters (elevation 198.5 m) at the Boring B-5-02 location. The fill soils are underlain by sedimentary rock that consists of decomposed, very soft to soft sandstone and claystone with soil like physical properties. This sedimentary rock was encountered to a depth of 9.7 m (elevation 190.0 m). The lowest unit is an altered igneous rock that is typically decomposed to very intensely weathered, soft to moderately soft with localized moderately hard zones. Typically, the altered igneous rock when mechanically broken down has the physical properties of a clayey silt with minor fine sand. At the Abutment 1 location, this unit was encountered in all borings drilled in March 2002 to the maximum explored depth 19.6 m (elev. 180.1 m).

At the Bent locations, the existing Route 15 is partially located within a cut. Minor variations in the depth to top of rock and the type of rock were encountered. The upper unit is a thin layer of fill material consisting of silty/clayey sand, sandy clay and lean clay. These fill soils extend from the ground surface to a maximum depth of 4.6 meters (elevation 185.9 m) at the Boring B-3-02 location. The fill soils are underlain by a thin layer of sedimentary rock at some boring locations. At these locations, the sedimentary rock consists of decomposed, soft claystone. This sedimentary rock was encountered to a maximum depth of 2.82 m (elevation 186.1 m). The lowest unit is an altered igneous rock that is typically decomposed, soft to moderately soft with localized moderately hard zones. The altered igneous rock when mechanically broken down has the physical properties of a clayey silt with minor fine sand. This unit was encountered in all borings to the maximum explored depth 33.5 m (elev. 156.6 m).

At the Abutment 4 location, the upper unit is a thick layer of fill material consisting of silty/clayey sand, sandy clay and lean clay with scattered hard cobbles and boulders. These fill soils extend from the ground surface to a maximum depth of 8.22 meters (elevation 187.3 m) at Boring B-9-02 location. The fill soils are underlain by alluvium that consists of stiff to very stiff, lean and fat clay. The lowest unit is an altered igneous rock that is typically decomposed to very intensely weathered, soft to moderately soft with localized moderately hard zones. The altered igneous rock when mechanically broken down has the physical properties of a clayey silt with minor fine sand. At Abutment 4 location, this unit was encountered to the maximum explored depth 22.9 m (elev. 170.2 m).

The 1982-1983 As-Built LOTB sheet identifies the top of igneous rock varied across the existing Abutment 1 location from elevation 191.1 m (Boring B-1) to elevation 187.5m (Boring B-8). The As-Built LOTB sheet also shows that top of igneous rock varied across the existing Abutment 3 location from elevation 183.5 m to elevation 182.0 m. The As-Built LOTB sheet for the existing structure, revealed similar earth materials at the site with the addition of boulders, which were identified at both the existing Abutment 1 and 3 locations.

**Groundwater**

The 1982-1983 LOTB sheet indicated that groundwater was not encountered at the site. However, during the 2002 field investigation, groundwater was periodically measured at approximately elevation 180.5 m and is shown on the recent LOTB sheets.

**Corrosion**

Soil samples collected during the 2002 foundation investigation were combined from three borings (B-2-02, B-3-02 & B-4-02) to make composite samples of native earth materials at depth. The Office of Testing and Technology Services, Corrosive Technology Branch (CTB) tested the composite sample for corrosive potential. The results of the laboratory tests determined that the composite sample was corrosive. Please refer to Table 1 below for specific test results.

**Table 1: Corrosion Test Summary-Composite Samples**

| Support Location/ Corrosion Number | Sample Depth (m) | pH | Minimum Resistivity (Ohm-Cm) | Sulfate Content (PPM)* | Chloride Content (PPM)* | Years To Perforation 18 ga. Galv. Steel Culvert |
|---|---|---|---|---|---|---|
| Abut 1 / 02-0370 | 0 to 1.5 | 8.60 | 715 | 116 | 42 | 22 |
| Abut 1 / 02-0369 | 3.1 to 6.2 | 8.62 | 490 | 182 | 52 | 19 |
| Abut 1 / 02-0371 | 10.5 to 18.3 | 7.82 | 325 | 186 | 516 | 16 |
| Bent 2 / 02-0251 | 0 to 1.5 | 6.92 | 540 | 31 | <30 | 14 |
| Bent 2 / 02-0252 | 1.5 to 3.2 | 8.41 | 900 | <30 | <30 | 24 |
| Bent 2 / 02-0253 | 3.2 to 6.2 | 8.37 | 800 | <30 | <30 | 23 |
| Bent 3 / 02-0210 | 0 to 1.5 | 8.53 | 350 | 48 | 650 | 16 |
| Bent 3 / 02-0211 | 5.6 to 9.1 | 8.95 | 750 | 69 | 38 | 22 |
| Abut 4 / 02-0367 | 0 to 4.6 | 8.17 | 590 | 149 | 90 | 14 |
| Abut 4 / 02-0368 | 4.6 to 10.7 | 7.68 | 430 | 88 | 333 | 18 |

Corrosion recommendations for the proposed Carmel Mountain Overcrossing (Br. No. 57-1125) were developed by Mike Piepoli of the Corrosion Technology Branch. Specific questions concerning the corrosion recommendations should be directed to Mr. Douglas Parks at 916-227-7007.

## Seismic Data

The site is potentially subject to strong ground motions from nearby earthquake sources during the design life of the new structure. The Newport-Inglewood Rose Canyon Fault/E (NIE, Strike-Slip) fault located approximately 18 km southwest from the site is the controlling fault for this site with a maximum credible earthquake of Mw=7. The Peak Bedrock Acceleration at this site, based on the Caltrans California Seismic Hazard Map, is estimated to be 0.3g. At this site, the liquefaction potential is considered to be minimal.

For site specific seismic data and design recommendations, refer to the memorandum concerning final seismic design recommendations dated July 18, 2002, by Asef Wardak of the former Office of Geotechnical Earthquake Engineering.

**"As-Built" Information**

The original foundation report (1977) and As-Built records (1982-1983) indicate that the bridge foundations for the existing structure consist of driven steel "H" piles at the abutments and spread footing foundations at Bent 2. At the Abutment 1 and 3 locations, driven steel "H" piles with a design load of 625 kN (70 tons) were used for support. At the abutments, predrilled holes extended to elevation 195.1 m (640 ft: NGVD29) and elevation 185.0 (607 ft; NGVD29) for Abutment 1 and 3, respectively. The As-Built pile tip elevations and As-Built spread footing information for the existing bridge are listed below in Table 2 and Table 3.

**Table 2. "As-built" Driven Steel 250X85 "H" Piles with 625 kN (70 ton) Design Load**

| Location | Specified Pile Tip Elevation | Minimum "As-built" Pile Tip Elevation | Average "As-built" Pile Tip Elevation | Maximum "As-built" Pile Tip Elevation |
|---|---|---|---|---|
| Abutment 1 | 187.5 m (615.0 ft) | 187.3 m (614.4 ft) | 187.6 m (615.4 ft) | 188.8 m (619.4 ft) |
| Abutment 3 | 181.4 m (595.0 ft) | 178.4 m (585.3 ft) | 180.7 m (592.8 ft) | 182.1 m (597.4 ft) |

Note: As-Built Elevations shown above are based on the NAVD29 vertical datum.

**Table 3. "As-built" Spread Footing Table**

| Location | Allowable Soil Pressure | Bottom of Footing Elevation (Max) |
|---|---|---|
| Bent 2 | 383 kPa (4.0 tsf) | 184.4 m (605 ft) |

Note: As-Built Elevations shown above are based on the NAVD29 vertical datum.

**Foundation Recommendations**

The following recommendations are for the proposed Carmel Mountain Overcrossing (Br. No. 57-1125), as shown on the General Plan No. 1, 2 and 3, dated February 25, 2003. At the Abutment 1 location, a spread footing foundation is recommended for support. At the Bent 2 and 3 locations, large diameter CIDH shafts will be used for support. At the Abutment 4 location and all retaining wall locations, driven steel "H" piles will be used for support. The following foundations recommendations were developed using deign information provided by Dokken Engineering (dated 1-31-03).

Bridge Foundations

At the Abutment 1 location, subsurface information indicates the proposed bottom of footing elevation will have the footing partially situated on near surface fill and weathered igneous rock. In order to eliminate differential settlement across the proposed Abutment 1, sub-excavation of earth materials below the proposed Abutment 1 bottom of footing elevation down to elevation 186.2 m and replacement with Class 3 concrete is recommended. The Gross Allowable Soil Bearing Pressure to be used for design is listed below in Table 4.

**Table 4: Spread Footing Data Table**

| Support Location | Minimum Footing Width | Bottom of Footing Elevation | Sub-Excavation Elevation | Recommended Soil Bearing Pressures | |
|---|---|---|---|---|---|
| | | | | ASD[1] | LFD[2] |
| | | | | Gross Allowable Soil Bearing Pressure ($q_{all}$) | Ultimate Soil Bearing Pressure ($q_{ult}^*$) |
| Abutment 1 | 5.35 m | 187.5 m | 186.2 m | 383 kPa (4 tsf) | N/A |

Notes: 1) Allowable Stress Design, (ASD). The Maximum Contact Pressure, ($q_{max}$), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, ($q_{all}$). The Ultimate Soil Bearing Capacity, ($q_{ult}$), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, ($q_{all}$).
2) Load Factor Design, (LFD). The Maximum Contact Pressure, ($q_{max}$), divided by the Strength Reduction Factor, ($\phi$), is not to exceed the recommended Ultimate Soil Bearing Pressure, ($q_{ult}^*$). The Ultimate Soil Bearing Capacity, ($q_{ult}$), will equal or exceed the recommended Ultimate Soil Bearing Pressure, ($q_{ult}^*$).

The recommended pressures to be used for design for the proposed structure support spread footings, listed above in Table 4, are based on the following criteria:

- At Abutment 1, the footing shall be supported on 1.3 meters of Class 3 concrete extending down to elevation 186.2 m. The limits of sub-excavation and replacement with Class 3 concrete shall conforms to the same limits required for relative compaction of engineered fill below retaining wall footings without piles as defined in section 19-5.03 of the Standard Specifications.
- Support footings shall have a minimum footing width of 5.35 meters at Abutment 1.
- All footings are to be constructed at or below the recommended bottom of footing elevations provided above in Table 4.

If any of the above minimum footing widths or limits of sub-excavation are reduced, the Office of Geotechnical Design South 2, Branch B shall be contacted for reevaluation.

At the Bents 2 and 3 (column 1) support locations, it is possible to utilize 2.4-m Cast-In-Drilled-Hole (CIDH) shafts for support. At the Bents 2 and 3 (column 2,3,4,5 & 6) support locations, it is possible to utilize 1.8-m Cast-In-Drilled-Hole (CIDH) shafts for support. The ultimate geotechnical pile capacity for the CIDH piles will meet or exceed the required nominal resistance in compression listed below in Tables 5.

At the Abutment 4 support location, it is recommended to use 625 kN design load, HP250X85 steel "H" piles for support. The specified pile tip elevations are listed below in Table 5. The ultimate geotechnical pile capacity is two-times (2x) the specified 625 kN design load.

**Table 5: Pile Data: Proposed Carmel Mountain Overcrossing**

| Support Location | Pile Type | Design Loading | Nominal Resistance | | Pile Cut-Off Elevation (m) | Design Tip Elevation (m) | Specified Tip Elevation (m) |
|---|---|---|---|---|---|---|---|
| | | | Compression | Tension | | | |
| Bent 2 (Column 1) | 2.4 m CIDH | N/A | 13600 kN | 0 kN | 187.9 | 164.9 m (1) | 164.9 m |
| Bent 3 (Column 1) | 2.4 m CIDH | N/A | 13600 kN | 0 kN | 188.1 | 164.2 m (1) | 164.2 m |
| Bent 2 (Column 2 & 3) | 1.8 m CIDH | N/A | 11500 kN | 0 kN | 187.9 | 166.8 m (1) | 166.8 m |
| Bent 3 (Column 2 & 3) | 1.8 m CIDH | N/A | 11500 kN | 0 kN | 188.1 | 165.8 m (1) | 165.8 m |
| Bent 2 (Column 4, 5, 6) | 1.8 m CIDH | N/A | 11500 kN | 0 kN | 187.9 | 166.8 m (1) | 166.8 m |
| Bent 3 (Column 4, 5, 6) | 1.8 m CIDH | N/A | 11500 kN | 0 kN | 188.1 | 165.8 m (1) | 165.8 m |
| Abutment 4 | HP 250X85 "H" Piles | 625 kN | 1250 kN | 0 kN | N/A | 178.3 m (1) | 178.3 m |

Note 1: Design tip elevation is controlled by the following demands: (1) Compression
Note 2: The column numbering increases from left to right while looking up station of the "N" line.

## Retaining Wall Foundations

The following foundation recommendations are for the proposed retaining wall structures located at both bridge abutment locations as shown on the foundation plan dated February 25, 2003. The proposed Type 1 Retaining Wall structures at the Abutment 1 and Abutment 4 locations may all be supported with 400 kN design load HP 250x62 steel "H" Piles. The specified pile tip elevations are listed below in Table 6. The ultimate geotechnical pile capacity is two-times (2x) the specified 400 kN design load. The retaining wall locations and required pile design loads, referenced below in Table 6, were provided by Dokken Engineering (dated February 25, 2003).

**Table 6. Pile Data: Abutment 1 and Abutment 4, Type 1 Retaining Walls**

| Location | Stationing From CL Proposed "NA" Line | Pile Type | Design Loading | Nominal Resistance | | Design Tip Elevation | Specified Tip Elevation |
|---|---|---|---|---|---|---|---|
| | | | | Compression | Tension | | |
| Abutment 1 (Left Wall) | 22.1 m Lt. Sta. 29+57.2 to 22.1 m Lt. Sta. 29+67.8 | HP 250X62 "H" Piles | 400 kN | 800 kN | -0- | 184.0 m (1) | 184.0 m |
| Abutment 1 (Right Wall) | 22.1 m Rt. Sta. 29+33.5 to 22.1 m Rt. Sta. 29+46.3 | HP 250X62 "H" Piles | 400 kN | 800 kN | -0- | 182.5 m (1) | 182.5 m |
| Abutment 4 (Left Wall) | 22.1 m Lt. Sta. 30+64.0 to 22.1 m Lt. Sta. 30+72.4 | HP 250X62 "H" Piles | 400 kN | 800 kN | -0- | 180.0 m (1) | 180.0 m |
| Abutment 4 (Right Wall) | 22.1 m Rt. Sta. 30+42.3 to 22.1 m Rt. Sta. 30+50.8 | HP 250X62 "H" Piles | 400 kN | 800 kN | -0- | 180.0 m (1) | 180.0 m |

Note: Design tip elevation is controlled by the following demands: (1) Compression

## General Notes

1. All 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.

2. 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. If the specified pile tip elevation required to meet lateral load demands exceed the specified pile tip elevation given within this report, the Office of Geotechnical Design South 2, Branch B should be contacted for further recommendations.

## Construction Considerations

1. A 30-day waiting period is required, where new fill is being placed, at all retaining wall locations and prior to beginning any pile driving at these location.

2. At Abutment 1 location, support footing excavations are to be inspected and approved by a representative of the Office of Geotechnical Design South 2, Branch B. The inspections are to be made after the excavation has been completed to the specified sub-excavation elevation listed above in Table 4 and prior to placing Class 3 concrete in the excavation. The contractor is to allow five (5) working days for the inspection of the excavation to be completed. The structure representative is to provide the Office of Geotechnical Design South 2, Branch B a one-week notification prior to beginning the five-day contractor waiting period.

3. At the Abutment 1 location, shear keys shall be incorporated into the foundation footing design.

4. At Abutment 1 location, it is recommended that a shear key, with adequate dimensions to accommodate the proposed shear key as shown in the abutment detail sheets, be formed in the top of the Class 4 concrete.

5. Pile acceptance at Abutment 4 location will be assessed by the ENR equation (Standard Specifications 49-1.08). At the abutment support location, any pile achieving two times (2x) the required design loading shown on the contract plans and above in Table 5, within 3 meters of the specified tip elevation, may be considered satisfactory and cut off with the Engineers written approval. Two times the required design loading is 1250 kN.

6. Pile acceptance at all Type 1 Retaining Wall locations will be assessed by the ENR equation (Standard Specifications 49-1.08). At the retaining wall support locations, any pile achieving two times (2x) the required design loading shown on the contract plans and above in Table 6, within 2 meters of the specified tip elevation, may be considered satisfactory and cut off with the Engineers written approval. Two times the required design loading is 800 kN.

7. Piles at the Abutments 4 location and the Abutment 4 Retaining Wall locations shall be driven in pre-drilled holes in conformance with the provisions in Section 49-1.06, "Predrilled Holes," of the Standard Specifications, to the corresponding bottom of hole elevations listed below.

| Structure Support Location | Retaining Wall Location | Bottom of Pre-drilled Holes Elevation |
|---|---|---|
| Abutment 4 (Structure Support) | | 185.5 m |
| | Abutment 4 (Left Ret. Wall) | 185.5 m |
| | Abutment 4 (Right Ret. Wall) | 185.5 m |

8. Difficult drilling for "Predrilled Holes" should be anticipated due to the presence of very dense soils with scattered cobbles and boulders as identified in the abutment embankments. Please refer to the Log of Test Borings sheets.

9. Hard driving of the "H" piles should be anticipated. Due to the presence of hard cobbles & boulders, the variations in the top of igneous rock elevations and the variations in the degree of rock weathering across the site, field cutting and splicing of all "H" piles should be anticipated.

10. At the Contractors option, driven "H" piles may be driven with lugs installed to aid in achieving the required nominal resistance at the specified pile tip elevation and to avoid field splicing. Lugs shall be installed as specified in the Bridge Construction Records and Procedures Manual, Bridge Construction Memo 130-5.0.

11. Groundwater was encountered during the 2002 field investigation and it is anticipated that groundwater will be encountered during CIDH pile construction at the Bent 2 & 3 locations. Groundwater surface elevation is subject to seasonal fluctuations and may occur higher or lower depending on the conditions and time of construction.

12. The calculated geotechnical capacity of the CIDH shafts is based upon Skin Friction only. The geotechnical capacity of the shaft was calculated from one pile diameter below the pile cut-off elevation extending down to one pile diameter above the specified tip elevation.

13. Caving conditions may be encountered during CIDH pile construction. Temporary casing may be necessary to control caving during construction. All temporary casing is to be removed during concrete placement.

14. Difficult pile installation and drilling is anticipated due to the presence of variably weathered, variable hard igneous rock described in the geology section. Hard rock drilling should be anticipated to advance the shaft excavations to the specified pile tip elevations.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by Dokken Engineering. If any conceptual changes are made during final project design, the Office of Geotechnical Design South 2, Branch B should review those changes to determine if the foundation recommendations provided in this report are still applicable. Any questions regarding the above recommendations should be directed to the attention of Hector Valencia (916) 227-4555 (CALNET 498-4555) or Mark DeSalvatore (916) 227-5391 (CALNET 498-5391), Office of Geotechnical Design South 2, Branch B.

Prepared by: Date: 3-18-03

Hector Valencia
Associate Engineering Geologist
Office of Geotechnical Design – South 2
Branch B

Supervised by: Date: 3/19/03

Mark DeSalvatore, RCE# 39499
Senior Materials and Research Engineer
Office of Geotechnical Design – South 2
Branch B

REGISTERED PROFESSIONAL ENGINEER
MARK DeSALVATORE
No. CE039499
Exp. 12/31/05
CIVIL
STATE OF CALIFORNIA

cc: R.E. Pending File
John Stayton – Specs & Estimates
Tony Marquez – Project Mgmt
Dave Pajouhesh – PCE
Lawrence Carr – District 11, Project Manager
Marcelo Peinado – District 11, Design Project Manager
Abbas Abghari – OGDS2
Project File
Project File - South