State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. KEVIN ROSS
Structure Design MS - 9
Office of Bridge Design-South
Bridge Design Branch 12

Attention: Mr. Dan Texler

Date: September 20, 2001

File: 11-SD-56-KP 8.70
EA 11-172821

McGonigle Creek Bridge
Bridge No. 57-1082 R/L

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Geotechnical Services - MS 5
Office of Geotechnical Design - South
Structure Foundations - South Branch

Subject Foundation Recommendations

This report presents the foundation recommendations for the proposed McGonigle Creek Bridges (Bridge No. 57-1082 R/L). The Office of Geotechnical Services - South, Structure Foundations - South Branch completed a foundation investigation pursuant to the July 24, 2000 request by Structure Design, Office of Bridge Design - South for a foundation investigation and recommendations for the proposed structures.

The following foundation recommendations are based on subsurface information gathered during the recent foundation investigation (Nov. 2000 - April 2001) performed by Caltrans along with a review of subsurface information used to develop the Draft Type Selection Report for the proposed structures, prepared by Boyle Engineering Corporation (BEC), dated January 28, 1999. With regards to the current foundation recommendations, all elevations referenced within this report and shown on the Log of Test Boring Sheets are based on the NAVD 88 vertical datum.

Project Description

The project site is located within the Carmel Valley area within San Diego County, and is located along the proposed alignment of State Route 56 where it crosses McGonigle Creek. To the north of the proposed structures site, there are residential subdivisions, and other subdivisions are in the process of being constructed to the south of the site. The terrain around the project site generally consists of low hills that are dissected by natural drainage paths. The localized area bordering McGonigle Creek is considered to be a wetland area, and environmentally sensitive.

The proposed new bridges are to consist of three span, cast-in-place, pre-stressed, concrete box girder type structures with wingwalls at the abutments. The proposed right bridge is to have double column piers, and the proposed left bridge is to have single column piers.

Geology

The foundation investigation performed from November 2000 to April 2001 consisted of seven mud rotary borings drilled with a Mobile Drill B-47 drill rig. The investigation revealed that the soils encountered at the proposed bridge site can be generally separated into two units. The upper unit soils are described as an alluvium consisting of a silty, clayey sand with gravels and cobbles, and a sandy clayey silt with scattered gravels. The alluvium ranged from 1.82 m deep (elev. 70.0 m) in boring B-00-1 at the right bridge Abutment 1 location, to 3.40 meters deep (elev. 70.3 m) in boring B-00-2 at the left bridge Abutment 4 location. The upper unit soils at the site are underlain by poorly indurated sandstone (La Jolla Group) consisting of a dense to very dense silty sand with occasional gravels and cobbles, with localized lenses of weak to strong cementation, as well as interbedded weakly to strongly cemented siltstone and claystone layers. The maximum depth explored in the 2000/2001 Caltrans investigation was 61.5 m (elev. 7.9 m) in boring B-01-2 at the right bridge Pier 3 location. Refer to the Log of Test Borings for site-specific soils data.

The subsurface exploration completed by BEC, in December 1998, revealed similar soil conditions as described above with minor differences. The upper unit is described by BEC as a medium dense to dense clayey sand to sandy clay with scattered gravels and cobbles to a depth of 5.6 m (elev. 77.2 m) in boring MCB-HSA-1, to 5.2 m (elev. 65.0 m) in boring MCB-HSA-2, to 3.8 m (elev. 64.8 m) in boring MCB-HSA-3, and to a depth of 2.6 m (elev. 71.9 m) in boring MCB-HSA-4. In boring MCB-HSA-1, the upper unit was described as a possible landslide deposit. The upper unit soils at the site are underlain by a weakly cemented sandstone characterized as a very dense silty to clayey sand and sandy clay with localized lenses of strong cementation, as well as scattered interbedded gravel and cobble layers. The maximum depth explored in the BEC 1998 investigation was 18.5 m (elev. 64.3 m) in boring MCB-HSA-1, which is at the proposed right bridge Abutment 4 location.

Ground Water

Ground water was encountered during the Caltrans subsurface investigation at a depth of 3.8 m (elev. 68.0 m) in boring B-00-1 and at 4.5 m deep (elev. 69.2 m) in B-00-2 in November 2000, as well as at 1.63 m deep (elev. 67.8 m) in boring B-01-2 in April 2001. Ground water was identified in two of the exploratory borings performed by BEC, in December 1998, at a depth of 1.7 m (elev. 68.5 m) in boring MCB-HSA-2, and at a depth of 2.2 m (elev. 66.4 m) in boring MCB-HSA-3. Ground water will be encountered during the construction of the pile caps and CIDH piles at the Piers 2 and 3 locations of the left and right bridges.

Scour Potential

The memorandum provided by the Office of Structure Maintenance and Investigations, dated July 23, 2001, states that the maximum potential scour at the bridge site extends to elevation 67.0 meters. During the 2000/2001 Caltrans subsurface investigation, potentially scourable material was encountered to elevation 68.1 m in boring B-00-3 at the left bridge Pier 2 location, to elevation 67.6 m in boring B-00-5 at the left bridge Pier 3 location, to elevation 66.1 m in boring B-00-4a at the right bridge Pier 2 location, and to elevation 66.0 m in boring B-01-2 at the right bridge Pier 3 location.

Corrosion

Corrosion test results for soil samples collected from borings B-00-1, B-00-2, B-00-3, and B-01-2 are shown below in Table 1, and indicate that all of the soil samples tested have a minimum resistivity less than 1000 ohm-cm which indicates that they are corrosive. The most recent corrosion recommendations for this structure have been issued by the Office of Testing and Technology Services, Corrosion Technology Branch in a memorandum dated September 6, 2001. A copy of this memorandum has been sent to Structure Design, Office of Bridge Design - South. For specific corrosion recommendations, refer to the above-mentioned memorandum.

Table 1
Corrosion Test Summary

Location	Corrosion Test Number	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
Boring B-00-1 (Elev. 65.38 m)	01-0017	6.6	770	187	135
Boring B-00-1 (Elev. 58.51 m)	01-0018	5.7	460	1423	151
Boring B-00-2 (Elev. 67.6 m)	01-0019	7.8	765	310	103
Boring B-00-2 (Elev. 55.04 m)	01-0021	7.8	650	303	54
Boring B-00-2 (Elev. 50.15 m)	01-0020	4.6	555	2007	45
Boring B-00-3 (Elev. 68.58 m)	01-0065	7.3	460	334	598
Boring B-00-3 (Elev. 63.24 m)	01-0066	5.5	410	4423	300
Boring B-00-3 (Elev. 55.29 m)	01-0067	5.0	870	4162	126
Boring B-00-3 (Elev. 19.44 m)	01-0022	6.5	710	550	134
Boring B-01-2 (Elev. 64.07 m)	01-0253	8.0	730	240	220
Boring B-01-2 (Elev. 57.45 m)	01-0254	5.9	690	560	88
Boring B-01-2 (Elev. 49.13 m)	01-0255	5.1	760	2700	55
Boring B-01-2 (Elev. 38.46 m)	01-0256	8.3	820	230	29
Boring B-01-2 (Elev. 28.53 m)	01-0471	4.2	700	1400	31
Boring B-01-2 (Elev. 16.12 m)	01-0472	7.0	600	870	96
Boring B-01-2 (Elev. 8.62 m)	01-0484	9.1	610	100	82

Note: Caltrans currently defines a corrosive environment as an area where the soil contains more than 500 ppm of chlorides, or more than 2000 ppm of sulfates, or has a minimum resistivity of less than 1000 ohm-cm, or has a pH of 5.5 or less.

Fault and Seismic Data

The proposed structure site is potentially subject to strong ground motions from nearby earthquake sources during the design life of the new structure. The Office of Geotechnical Earthquake Engineering (OGEE) has provided Final Seismic Design Recommendations for the site in the memorandum dated September 17, 2001. The controlling fault for the site is the Newport-Inglewood-Rose Canyon fault with a maximum credible earthquake M_w=7.0 located approximately 13.5 kilometers southwest of the site. The corresponding Peak Bedrock Acceleration is estimated to be 0.3g. The above-mentioned memorandum states the potential for liquefaction at the site is considered to be very low.

Foundation Recommendations

Bridge Foundations

The following recommendations are for the proposed McGonigle Creek Bridge (Br. No. 57-1082 R/L), as shown on the General Plan dated August 1, 2001. At Abutments 1 and 4 support locations of the right and left bridges it is recommended to use 625 kN design load HP 250x85 steel "H" Piles for support. At Piers 2 and 3 support locations, of the right and left bridges, it is possible to utilize 600 mm Cast-In-Drilled-Hole (CIDH) piles for support. The specified pile tip elevations are listed below in Tables 2 and 3. 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 and tension listed below in Tables 2 and 3.

Table 2:
Left Bridge Pile Data (Br. No. 57-1082 L)

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension			
Abutment 1	HP 250x85 "H" Piles	625 kN	1250 kN	-0-	83.25 m	69.0 m (1)	69.0 m
Pier 2	600 mm CIDH	N/A	2250 kN	450 kN	65.95 m	52.2 m (1) 58.6 m (2)	52.2 m
Pier 3	600 mm CIDH	N/A	2250 kN	450 kN	65.95 m	52.2 m (1) 58.6 m (2)	52.2 m
Abutment 4	HP 250x85 "H" Piles	625 kN	1250 kN	-0-	89.10 m	67.0 m (1)	67.0 m

Note: Design tip elevation is controlled by the following demands: (1) Compression, (2) Tension

Table 3:
Right Bridge Pile Data (Br. No. 57-1082 R)

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension			
Abutment 1	HP 250x85 "H" Piles	625 kN	1250 kN	-0-	82.90 m	65.0 m (1)	65.0 m
Pier 2	600 mm CIDH	N/A	2250 kN	-0-	65.10 m	51.4 m (1)	51.4 m
Pier 3	600 mm CIDH	N/A	2250 kN	-0-	65.10 m	51.4 m (1)	51.4 m
Abutment 4	HP 250x85 "H" Piles	625 kN	1250 kN	-0-	88.70 m	74.0 m (1)	74.0 m

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

Retaining Wall Foundations

For support of the proposed Type 1 retaining walls at the Abutment 1 and 4 locations of the left and right bridges, two options are presented below:

Option #1
One option is to use spread footings founded on the engineered fill material for support of the proposed retaining walls. The bottom of footing elevations and gross allowable soil bearing

pressures are shown below in Tables 4 and 5. All abutment retaining wall footings shall be positioned such that there will be a minimum horizontal distance of 1.22 meters from the near face/top of the footing to the face of the finished slope (Bridge Design Specifications 4.4.2.1). The finished slope is not to exceed a 1:1.5 (vertical to horizontal) ratio.

Table 4
Left Bridge
Spread Footing Data: Abutments 1 and 4 Type 1 Retaining Walls (Br. No. 57-1082 L)

Support Location	Stationing From CL Proposed "A" Line	Design Height of Wall	Minimum Footing Width	Bottom of Footing Elevation	Recommended Soil Bearing Pressures	
					ASD[1]	LFD[2]
					Gross Allowable Soil Bearing Pressure (q_{all})	Ultimate Soil Bearing Pressure (q_{ult}^*)
Abutment 1 (Left Wall)	23.5 m Lt. Sta. 96+06.587 to 23.5 m Lt. Sta. 96+09.487	1800 mm	1300 mm	87.325 m	90 kPa (1.9 ksf)	N/A
Abutment 1 (Left Wall)	23.5 m Lt. Sta. 96+09.487 to 23.5 m Lt. Sta. 96+11.887	3000 mm	1900 mm	86.125 m	120 kPa (2.5 ksf)	N/A
Abutment 1 (Left Wall)	23.5 m Lt. Sta. 96+11.887 to 23.5 m Lt. Sta. 96+14.287	4200 mm	2450 mm	84.875 m	160 kPa (3.3 ksf)	N/A
Abutment 1 (Right Wall)	11.25 m Lt. Sta. 95+99.690 to 11.25 m Lt. Sta. 96+03.990	1800 mm	1300 mm	87.325 m	90 kPa (1.9 ksf)	N/A
Abutment 1 (Right Wall)	11.25 m Lt. Sta. 96+03.990 to 11.25 m Lt. Sta. 96+06.390	3000 mm	1900 mm	86.125 m	120 kPa (2.5 ksf)	N/A
Abutment 1 (Right Wall)	11.25 m Lt. Sta. 96+06.390 to 11.25 m Lt. Sta. 96+08.790	4200 mm	2450 mm	84.875 m	160 kPa (3.3 ksf)	N/A
Abutment 4 (Left Wall)	23.50 m Lt. Sta. 97+99.836 to 23.50 m Lt. Sta. 98+02.236	4200 mm	2450 mm	90.725 m	160 kPa (3.3 ksf)	N/A
Abutment 4 (Left Wall)	23.50 m Lt. Sta. 98+02.236 to 23.50 m Lt. Sta. 98+04.636	3000 mm	1900 mm	91.975 m	120 kPa (2.5 ksf)	N/A
Abutment 4 (Left Wall)	23.50 m Lt. Sta. 98+04.636 to 23.50 m Lt. Sta. 98+09.636	2400 mm	1600 mm	93.175 m	105 kPa (2.2 ksf)	N/A
Abutment 4 (Right Wall)	11.25 m Lt. Sta. 97+94.340 to 11.25 m Lt. Sta. 97+96.740	4200 mm	2450 mm	90.725 m	160 kPa (3.3 ksf)	N/A
Abutment 4 (Right Wall)	11.25 m Lt. Sta. 97+96.740 to 11.25 m Lt. Sta. 97+99.140	3000 mm	1900 mm	91.975 m	120 kPa (2.5 ksf)	N/A
Abutment 4 (Right Wall)	11.25 m Lt. Sta. 97+99.140 to 11.25 m Lt. Sta. 98+03.840	2400 mm	1600 mm	93.175 m	105 kPa (2.2 ksf)	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, (ϕ), 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}^*).

Table 5
Right Bridge
Spread Footing Data: Abutments 1 and 4 Type 1 Retaining Walls (Br. No. 57-1082 R)

Support Location	Stationing From CL Proposed "A" Line	Design Height of Wall	Minimum Footing Width	Bottom of Footing 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 (Left Wall)	11.25 m Rt. Sta. 95+87.875 to 11.25 m Rt. Sta. 95+91.575	1800 mm	1300 mm	86.975 m	90 kPa (1.9 ksf)	N/A
Abutment 1 (Left Wall)	11.25 m Rt. Sta. 95+91.575 to 11.25 m Rt. Sta. 95+93.975	3000 mm	1900 mm	85.775 m	120 kPa (2.5 ksf)	N/A
Abutment 1 (Left Wall)	11.25 m Rt. Sta. 95+93.975 to 11.25 m Rt. Sta. 95+96.375	4200 mm	2450 mm	84.525 m	160 kPa (3.3 ksf)	N/A
Abutment 1 (Right Wall)	28.50 m Rt. Sta. 95+80.020 to 28.50 m Rt. Sta. 95+83.620	1800 mm	1300 mm	86.875 m	90 kPa (1.9 ksf)	N/A
Abutment 1 (Right Wall)	28.50 m Rt. Sta. 95+83.620 to 28.50 m Rt. Sta. 95+86.020	2400 mm	1600 mm	85.775 m	105 kPa (2.2 ksf)	N/A
Abutment 1 (Right Wall)	28.50 m Rt. Sta. 95+86.020 to 28.50 m Rt. Sta. 95+88.420	3600 mm	2200 mm	84.575 m	135 kPa (2.8 ksf)	N/A
Abutment 4 (Left Wall)	11.25 m Rt. Sta. 97+81.925 to 11.25 m Rt. Sta. 97+84.325	4200 mm	2450 mm	90.325 m	160 kPa (3.3 ksf)	N/A
Abutment 4 (Left Wall)	11.25 m Rt. Sta. 97+84.325 to 11.25 m Rt. Sta. 97+86.725	3600 mm	2200 mm	91.575 m	135 kPa (2.8 ksf)	N/A
Abutment 4 (Left Wall)	11.25 m Rt. Sta. 97+86.725 to 11.25 m Rt. Sta. 97+92.325	2400 mm	1600 mm	92.775 m	105 kPa (2.2 ksf)	N/A
Abutment 4 (Right Wall)	28.50 m Rt. Sta. 97+73.970 to 28.50 m Rt. Sta. 97+76.370	3600 mm	2200 mm	90.375 m	135 kPa (2.8 ksf)	N/A
Abutment 4 (Right Wall)	28.50 m Rt. Sta. 97+76.370 to 28.50 m Rt. Sta. 97+78.770	2400 mm	1600 mm	91.575 m	105 kPa (2.2 ksf)	N/A
Abutment 4 (Right Wall)	28.50 m Rt. Sta. 97+78.770 to 28.50 m Rt. Sta. 97+82.670	1800 mm	1300 mm	92.775 m	90 kPa (1.9 ksf)	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, (ϕ), 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}^{*}).

Option #2

A second option is to utilize 400 kN design load HP 250x63 steel "H" Piles for support of the proposed retaining walls. The specified pile tip elevations are listed below in Table 6 and 7. The ultimate geotechnical pile capacity is two-times (2x) the specified design load for driven steel "H" piles with 400 kN design load.

Table 6
Left Bridge
Pile Data: Abutment 1 and 4 Type 1 Retaining Walls (Br. No. 57-1082 L)

Location	Stationing From CL Proposed "A" Line	Pile Type	Design Loading	Nominal Resistance		Design Tip Elevation	Specified Tip Elevation
				Compression	Tension		
Abutment 1 (Left Wall)	23.5 m Lt. Sta. 96+06.587 to 23.5 m Lt. Sta. 96+14.287	HP 250x63 "H" Piles	400 kN	800 kN	-0-	71.3 m (1)	71.3 m
Abutment 1 (Right Wall)	11.25 m Lt. Sta.95+99.690 to 11.25 m Lt. Sta.96+08.790	HP 250x63 "H" Piles	400 kN	800 kN	-0-	70.5 m (1)	70.5 m
Abutment 4 (Left Wall)	23.50 m Lt. Sta.97+99.836 to 23.50 m Lt. Sta.98+09.636	HP 250x63 "H" Piles	400 kN	800 kN	-0-	67.8 m (1)	67.8 m
Abutment 4 (Right Wall)	11.25 m Lt. Sta.97+94.340 to 11.25 m Lt. Sta.98+03.840	HP 250x63 "H" Piles	400 kN	800 kN	-0-	69.4 m (1)	69.4 m

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

Table 7
Right Bridge
Pile Data: Abutment 1 and 4 Type 1 Retaining Walls (Br. No. 57-1082 R)

Location	Stationing From CL Proposed "A" Line	Pile Type	Design Loading	Nominal Resistance		Design Tip Elevation	Specified Tip Elevation
				Compression	Tension		
Abutment 1 (Left Wall)	11.25 m Rt. Sta. 95+87.875 to 11.25 m Rt. Sta.95+96.375	HP 250x63 "H" Piles	400 kN	800 kN	-0-	68.8 m (1)	68.8 m
Abutment 1 (Right Wall)	28.50 m Rt. Sta.95+80.020 to 28.50 m Rt. Sta.95+88.420	HP 250x63 "H" Piles	400 kN	800 kN	-0-	66.3 m (1)	66.3 m
Abutment 4 (Left Wall)	11.25 m Rt. Sta.97+81.925 to 11.25 m Rt. Sta.97+92.325	HP 250x63 "H" Piles	400 kN	800 kN	-0-	76.6 m (1)	76.6 m
Abutment 4 (Right Wall)	28.50 m Rt. Sta.97+73.970 to 28.50 m Rt. Sta.97+82.670	HP 250x63 "H" Piles	400 kN	800 kN	-0-	78.4 m (1)	78.4 m

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

General Notes:

1. When applicable, 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. Should the specified pile tip elevation required to meet lateral load demands exceed the specified pile tip elevations given within this report, the Office of Geotechnical Design - South, Structure Foundations - South Branch shall be contacted for further recommendations.

2. All support locations are to be plotted in plan view on the Log of Test Borings as stated in "Memo to Designers" 4-2. The plotting of support locations should be made prior to requesting a final foundation review.

3. Structure excavation Type D is to be shown on the plans at all pier support locations.

4. For driven "H" piles, the lug details shall be provided in the contract plans.

Construction Considerations:

1. A 30-day waiting period is required at the Abutments 1 and 4 support locations, of the left and right bridges, where new fill is being placed, prior to beginning any pile driving at those locations.

2. For Option #1, above, at the proposed retaining wall support locations at the left and right bridges, concrete for the retaining wall support footings shall be placed neat against the undisturbed engineered fill on the bottom of the footing excavation. Should the bottom of the footing excavation be disturbed, then the disturbed soils shall be recompacted to 95% relative compaction prior to placement of concrete for the structure support footings.

3. All driven "H" piles shall have lugs installed to aid in achieving the required nominal resistance at the specified tip elevation. Lugs shall be installed as specified in the Bridge Construction Records and Procedures Manual, Bridge Construction Memo 130-5.0.

4. Pile bearing at the abutment locations will be assessed by the ENR equation. At the abutment support locations, any pile achieving refusal within 1.2 meters of the specified tip elevation may be considered satisfactory and cut off with the Engineers written approval. Refusal shall be considered as two times (2x) the required design loading shown on the contract plans and above in Tables 2 and 3. Two times the required design loading is 1250 kN.

5. For Option #2, above, for the proposed retaining walls, pile bearing will be assessed by the ENR equation. At the retaining wall support locations, any pile achieving refusal within 1.2 meters of the specified tip elevation may be considered satisfactory and cut off with the Engineers written approval. Refusal shall be considered as two times (2x) the required design loading shown on the contract plans and above in Tables 6 and 7. Two times the required design loading is 800 kN.

6. Piles at Abutments 1 and 4 locations, of the left and right bridges, 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.

Support Location	Bottom of Pre-drilled Holes Elevation
Abutment 1 (Left Bridge)	74.75 m
Abutment 4 (Left Bridge)	73.50 m
Abutment 1 (Right Bridge)	70.00 m
Abutment 4 (Right Bridge)	84.00 m

7. Hard driving of the "H" piles should be anticipated. Due to variations of the top of formation elevations across the site, as well as hardness of the formational material, field cutting and splicing of all "H" piles should be anticipated.

8. The load carrying capacity of the CIDH piles is based only on the skin friction developed from approximately two pile diameters below the bottom of footing elevation to the zone within one pile diameter from the specified pile tip elevation. No end bearing was considered.

9. 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.

10. Ground water was encountered during drilling of test borings and ground water will be encountered during CIDH pile construction.

11. Ground water will be encountered during excavation at the pier locations, therefore, structure excavation Type D should be anticipated at all pier support locations during excavation for the pile caps.

12. Difficult pile installation and drilling is anticipated due to the presence of formational layers containing very hard cobbles.

13. Dewatering of drilled pile holes is anticipated to be feasible at all support locations where ground water is encountered, provided the contractor adequately controls the inflow of water from the alluvium above the formational material (i.e. sheet piling). The contractor shall keep drilled excavations dry, by pumping methods, immediately after the boring has reached specified tip elevation until the time concrete is placed for construction of the pile.

14. Pile load tests, one in compression to failure, and one in tension to failure, shall be conducted on separate non-production piles between Piers 2 of the left and right bridges. The contractor shall construct the non-production piles to be used for the pile load tests using the same techniques that will be used for the production piles. The contractor shall not fabricate any pile cages for the production piles until the pile load tests have been completed and approved by a representative of Structure Foundations - South Branch. The controlling zones for the pile load tests will be Piers 2 and 3 locations of the left and right bridges.

The recommendations contained in this report are based on specific project information regarding structure type, location, and design loads that have been provided by the Office of Bridge Design - South. If any conceptual changes are made during final project design, the Office of Geotechnical Design - South, Structure Foundations - South Branch 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 Erich Neupert (916) 227-7145 (CALNET 498-7145) or Mark DeSalvatore (916) 227-7056 (CALNET 498-7056).

Report by: Date: 9/21/01

Erich Neupert

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