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Memorandum

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To: MR. BARTT GUNTER
Branch Chief
Structure Design Branch 19
Office of Bridge Design South 2

Date: November 3, 2014

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Watson Wash Bridge-Left (Replace)
Br. #54-1282 L

Attention: Mrs. Rui Wang

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2
Design Branch B

Subject: Foundation Report for Watson Wash Bridge - Left

In a memorandum dated May 23, 2014, Structures Design, Office of Bridge Design South 2, Bridge Design Branch 19 requested a Foundation Report (FR) for the proposed replacement of Watson Wash left bridge (Br. No. 54-1282L). This FR supersedes all previously generated Preliminary Foundation Reports for this structure. The following recommendations are based on subsurface information gathered during the 2013 subsurface investigation performed at the project site, a review of "As-Builts" Log of Test Boring (LOTB) data from the 1969 subsurface investigation performed for the left and right bridges and 2000/2001 subsurface investigation performed for the right bridge. With regards to the current foundation recommendations, all elevations referenced within this report and shown on the recent Log of Test Boring sheets are based on the NGVD 1929 vertical datum.

Project Description

The Watson Wash Left Bridge (Br. No. 54-1282L) is located in San Bernardino County on State Route 40 approximately 33 miles west of Needles, California. The proposed left bridge replacement structure will consist of a seventeen-span, pre-cast, pre-stressed, concrete slabs structure on seat type abutments.

Geology

The "Geologic Map of the San Bernardino Quadrangle, San Bernardino County, California (Bortugno and Spittler, Revised 1998)" indicates that the site is located on Quaternary Alluvium which consists of dissected and undifferentiated alluvium deposits, colluvium and fan conglomerate.

The 2013 subsurface investigation consisted of 8 mud-rotary soil borings. The soil borings revealed the site is underlain mainly by layers of predominantly dense to very dense poorly-graded sands with silt and gravel, and silty sands, to the maximum explored depth of 74.4 feet (Elev. 1906.9 feet). For more detail, please refer to the Log of Test Borings.

Ground Water

No ground water was encountered in any of the subsurface investigations. Due to the Watson Wash bridge being located in an extremely arid environment, it is anticipated that surface water would only be present in the wash during brief wet periods.

Scour Potential

Structure Hydraulics and Hydrology has provided a Final Hydraulic Report in a memorandum dated September 5, 2014, which states that Watson Wash has a potential for scour. The scour data presented in the report is shown in Table 1 below. Please refer to that memorandum for more specific information.

Table 1 - Scour Data Watson Wash Bridge - Left (54-1282 L)

Support Location	Long Term Scour (ft)		Short Term Scour (Local) Depth (ft)	Scour Elevation (ft)
	Degradation	Contraction		
Abutment 1	0	0	N/A	N/A
Piers 2 to 6	3	0	7.5	1967.5
Piers 7 to 12	3	0	7.5	1969.0
Piers 13 to 17	3	0	7.5	1970.5
Abutment 18	0	0	N/A	N/A

Corrosion

Corrosion test results are shown below in Table 2. The tested soil sample was taken from soil boring RC-13-001. Test results indicate the soil sample is considered non-corrosive by current Caltrans standards.

Table 2 - Corrosion Test Summary

Location	SIC Number	pH	Minimum Resistivity (Ohm-cm)	Sulfate Content (ppm)	Chloride Content (ppm)
RC-13-001 0' - 40' (Elev. 1994.9 ft–1954.9 ft)	C637006	8.33	3092	NA	NA

Note: Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 2000 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,000 ohm-cm.

Fault and Seismic Data

The structure site is potentially subject to ground motions from nearby earthquake sources during the design life of the new structure. For the deterministic procedure, the controlling fault for the site is the San Andreas (Coachella) fault zone (Fault ID 372). It is a right-lateral strike-slip (RLSS) fault with a maximum credible earthquake Mw=7.9, located approximately 92.2 miles southwest of the bridge site. Based on the 2013 Seismic Design Procedure, a minimum deterministic response spectrum for a vertical strike-slip fault of $M_{max} = 6.5$ at a distance of 7.5 mile (12 km) should be used in the design. The corresponding peak ground acceleration (PGA) is estimated to be 0.23g. The office of Geotechnical Design has provided Seismic Design Recommendations in a memorandum dated July 31, 2014. Please refer to that memorandum for more specific seismic recommendations.

Liquefaction/Settlement

The Seismic Design Recommendations state that due to the dense nature of the underlying soils and deep groundwater, the potential for soil to liquefy at the site will be low. The amount of seismic settlement due to strong ground shaking is considered less than one inch.

Surface Rupture Potential

The site does not fall within Fault Rupture Hazard Zones in California (Alquist-Priolo Earthquake Fault Maps). The surface rupture potential at the bridge site is considered low.

Foundation Recommendations

The following recommendations are for the proposed Watson Wash Bridge-Left (Br. #54-1282L), as shown on the General Plan dated November 13, 2013. Abutments 1 and 18 and Piers 2 to 17 may be supported on spread footings.

Abutment Location

Abutments 1 and 18 can be supported on spread footings in the existing embankment fill. The Spread Footing Design Data, provided by Structure Design, is presented in Tables 3 and 4 below.

Table 3 - Spread Footing Design Data

Support Location	Design Method	Finished Grade Elevation (ft)	Bottom of Footing Elevation (ft)	Footing Size (ft)		Permissible Settlement under Service Load (in)
				B	L	
Abutment 1	WSD	1985.00	1978.00	12.00	53.67	1
Abutment 18	WSD	1989.00	1982.00	12.00	53.67	1

Table 4 - Spread Footing Design Data – Service I Limit State Loads

Support Location	Total Load				Permanent Load		
	Vertical Load (kips)	Effective Dimensions (ft)		Horizontal Load in Longitudinal Direction (kips)	Vertical Load (kips)	Effective Dimensions (ft)	
		B'	L'			B'	L'
Abutments 1 and 18	1390.0	11.91	53.67	225.0	1234.0	11.89	53.67

The recommended Permissible Gross Contact Stress, Allowable Gross Bearing Capacities and Bottom of Footing Elevations, for Abutments 1 and 18, are listed in Table 5 below.

Table 5 - Foundation Design Recommendations for Abutments 1 and 18

Support Location	Footing Size (ft)		Bottom of Footing Elevation (ft)	Minimum Horizontal Footing Embedment Depth (ft)	Total Permissible Support Settlement (in)	WSD (LRFD Service Limit State Load Combination)	
	B	L				Permissible Gross Contact Stress (ksf)	Allowable Gross Bearing Capacity (ksf)
Abut 1	12.0	53.67	1978.0	4.0	1	5.8	3.0
Abut 18	12.0	53.67	1982.0	4.0	1	5.8	3.0

In Table 5 above, the recommended Permissible Gross Contact Stress (q_{pg}) and Allowable Gross Bearing Capacity to be used for design, are based on the following design criteria:

1) The final designed spread footing will have an effective width (B') that will produce an equivalent Gross Uniform Bearing Stress (q_o), which does not exceed the Allowable Gross Bearing Capacity (q_{all}).
2) The Allowable Gross Bearing Capacity (q_{all}) was calculated with a maximum slope of 1.5H:1V in front of the abutment.
3) The spread footings are to be constructed at or below the recommended elevations shown in Table 5.

Contact the Office of Geotechnical Design-South 2, Branch B for re-evaluation if any of the following change:

- The footing size (B) is reduced.
- The loading conditions change.
- The bottom of footing elevation is raised.
- The minimum vertical or horizontal footing embedment depths are reduced.

Pier Locations

Each Pier location will consist of two (4) support columns on individual spread footings. Table 6 below, presents the Pier Spread Footing Design Data provided by Structure Design.

Table 6 - Pier Spread Footing Design Data

Support Location	Design Method	Finished Grade Elevation (ft)	Bottom of Footing Elevation (ft)	Footing Size (ft)		Permissible Settlement under Service Load (in)
				B	L	
Piers 2 to 5 per column	LRFD	1980.0	1965.0	8.0	8.0	1
Pier 6 per column	LRFD	1981.0	1965.0	8.0	8.0	1
Pier 7 per column	LRFD	1981.0	1966.5	8.0	8.0	1
Pier 8 per column	LRFD	1982.0	1966.5	8.0	8.0	1
Pier 9 per column	LRFD	1983.0	1966.5	8.0	8.0	1
Pier 10 per column	LRFD	1982.0	1966.5	8.0	8.0	1
Pier 11 and 12 per column	LRFD	1982.0	1966.5	8.0	8.0	1
Pier 13 per column	LRFD	1982.0	1968.0	8.0	8.0	1
Pier 14 per column	LRFD	1983.0	1968.0	8.0	8.0	1
Pier 15 and 16 per column	LRFD	1984.0	1968.0	8.0	8.0	1
Pier 17 per column	LRFD	1985.0	1968.0	8.0	8.0	1

Tables 7 and 8 below, present the LRFD Service, Strength, and Extreme Limit State Design Data provided by Structure Design.

Table 7 - LRFD Service-I Limit State Spread Footing Design Data

Support Location	Total Load			Permanent Load		
	Vertical Load (kips)	Effective Dimensions (ft)		Vertical Load (kips)	Effective Dimensions (ft)	
		B’	L’		B’	L’
Piers 2 to 17 per column	380.0	7.3	7.1	250.0	7.3	7.1

Table 8 - LRFD Strength and Extreme Event Limit States

Support Location	Strength Limit State (Controlling Group)			Extreme Event Limit State (Control Group)		
	Vertical Load (kip)	Effective Dimensions (ft)		Vertical Load (kip)	Effective Dimensions (ft)	
		B'	L'		B'	L'
Piers 2 to 17 per column	594.0	7.2	7.2	439.0	3.9	3.9

Foundation design recommendations for Piers 2 to 17, based on the spread footing design loading and approximate footing geometry provided by Structure Design, are presented in Table 9.

Table 9 - Foundation Design Recommendations for Piers 2 to 17

Support Location	Footing Size (ft)		Bottom of Footing Elevation (ft)	Minimum [1] Footing Embedment Depth (ft)	Total Permissible Support Settlement (in)	Service Limit State	Strength Limit State $\varphi = 0.45$	Extreme Limit State $\varphi = 1.0$
	L	B				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
Piers 2 to 6 per column	8.0	8.0	1965.0	4.0	1	14.7	18.5	53.0
Piers 7 to 12 per column	8.0	8.0	1966.5	4.0	1	14.7	18.5	53.0
Piers 13 to 17 per column	8.0	8.0	1968.0	4.0	1	14.7	18.5	53.0

[1] Minimum footing embedment depth starts below the combined degradation, contraction and local scour depths.

In Table 9 above, the recommended Permissible Net Contact Stress (q_{pn}) and Factored Gross Nominal Bearing Resistances (q_R) to be used for design, are based on the following design criteria:

1) The final designed spread footing will have an effective width (B') such that:
   - The equivalent Net Uniform Bearing Stress ($q_{n,u}$), does not exceed Permissible Net Contact Stress (q_{pn}) for Service-I Limit State.
   - The Gross Uniform Bearing Stress ($q_{g,u}$) does not exceed the recommended design values for the Factored Gross Nominal Bearing Resistances (q_R) for Strength and Extreme Limit States.

2) The spread footings are to be constructed at or below the recommended bottom of footing elevations shown in Table 9.

Contact the Office of Geotechnical Design-South 2, Branch B for re-evaluation if any of the following change:

- The footing size (B) is reduced.
- The loading conditions change.
- The bottom of footing elevation is raised.
- The minimum vertical footing embedment depths are reduced.

The Spread Footing Data table for Abutment and Pier supports is listed in Table 10, below.

Table 10 – Spread Footing Data Table

Support Location	Working Stress Design (WSD)		Load Resistance Factor Design (LRFD)		
	Permissible Gross Contact Stress (Settlement) (ksf)	Allowable Gross Bearing Capacity (ksf)	Service Permissible Net Contact Stress (Settlement) (ksf)	Strength Factored Gross Nominal Bearing Resistance $\varphi = 0.45$ (ksf)	Extreme Event Factored Gross Nominal Bearing Resistance $\varphi = 1.00$ (ksf)
Abutment 1	5.8	3.0	N/A	N/A	N/A
Piers 2 to 17 per column	N/A	N/A	14.7	18.5	53.0
Abutment 18	5.8	3.0	N/A	N/A	N/A

Construction Considerations:

1) At Abutments 1 and 18 support locations, the bottom of footings are to be constructed on existing fill. Concrete for the support footings shall be placed neat against the undisturbed material at the bottom of the footing excavation. Should the bottom of the footing excavation be disturbed then the bottom of the footing excavation is to be re-compacted or replaced with structural backfill compacted to 95% relative compaction, prior to placement of steel and concrete for the structure support footings.

2) At Piers 2 to 17 support locations, the bottom of the footing shall be constructed on dense native soil. Concrete for the support footings shall be placed neat against the undisturbed material at the bottom of the footing excavation. Should the bottom of the footing excavation be disturbed then the bottom of the footing excavation shall be compacted at 95% relative compaction, prior to placement of steel and concrete for the structure support footings.

3) The excavations at all support locations are classified as Structure Excavation (Bridge). No seal coarse is required.

4) At all support locations, the excavations are to be inspected and approved by a representative of the Office of Geotechnical Design-South 2, Branch B, prior to placing any concrete. The required inspection is to verify that the soil exposed at the bottom of the excavation complies with recommendations included in this report. Once the excavation has been completed to the specified elevations, the contractor is to allow the Office of

Geotechnical Design-South 2, Branch B, seven (7) calendar days to perform the inspection. The structures representative is to provide the Office of Geotechnical Design-South 2, Branch B, a one-week notification prior to beginning the seven-day contractor waiting period.

This Foundation Report is based on specific project information regarding structure type and location that have been provided by the Office of Bridge Design South 2. Once the project plans are available, the Office of Geotechnical Design-South 2, Design Branch B should review the information to determine if this FR is still applicable. Any questions regarding the above recommendations should be directed to the attention of Fernando De Haro, (916) 227-4556 or Mark DeSalvatore, (916) 227-5391, at the Office of Geotechnical Design-South 2, Branch B.

Prepared by: Date: 11/3/2014

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