FOR CONTRACT NO.: 06-442611

INFORMATION HANDOUT

MATERIALS INFORMATION

FOUNDATION REPORT BIOLA JUNCTION OH (WIDEN)

FOUNDATION REPORT HERNDON CANAL BRIDGE (CENTER WIDENING)

EXISTING RETAINNING WALL PILE COMPRESSION CAPACITY ASSESSMENT

FINAL HYDRAULIC REPORT

ROUTE: 06-Fre-99 PM 26.7/30.6

State of California – Department of Transportation
Division of Engineering Services
Office of Design & Technical Services

FINAL HYDRAULIC REPORT

Herndon Canal Bridge (Widen)

Bridge No. 42-0129R/L
6-Fre-99-PM 28.40
EA 06-442601

Prepared by:

Juan Jauregui, PE
Structure Hydraulics & Scour Mitigation
January 30, 2009

REGISTERED PROFESSIONAL ENGINEER
JUAN J. JAUREGUI
C 63939
EXP. 9/30/2010
CIVIL
STATE OF CALIFORNIA

General:

A median bridge widening is proposed for the Herndon Canal Bridge located on State Route 99 in Fresno County. The existing Herndon Canal Bridge consists of a left and right structure originally constructed in 1959. The right structure was widened in 1997. Both structures are two-span, continuously reinforced concrete slabs. They are supported by a pier wall and open-end diaphragm abutments, all founded on concrete piles. The pier wall supports a 3-ft wide "foot" bridge, both left and right structures, and a local frontage road bridge. A debris nose is located on the upstream end of the pier wall.

The proposed median bridge widening will consist of a prestressed-precast composite deck over the existing pier wall. The existing pier wall will be raised approximately 1'-5½" to match the new bridge soffit.

This report is based on the General Plans provided by Structure Design at the Type Selection Meeting held on January 8, 2009. This report also references information provided by Felix Vaquilar with the Fresno Irrigation District at (559) 233-7161, ext. 344.

All elevations indicated in this report are based on Vertical Datum NGVD 1988. Elevations obtained for this report are based on Preliminary Investigation's field survey data (performed 1/27/2009) and As-Built information. Elevations obtained from As-Builts were vertically transformed to vertical datum NGVD88 from NGVD29 by adding 2.1 feet to the As-Built elevations.

Herndon Canal:

Herndon Canal is operated by the Fresno Irrigation District (FID) and is part of a network of canals that not only convey irrigation water for agricultural uses but also provide storm water drainage and groundwater recharge. The source of the irrigation water conveyed by Herndon Canal is from Kings River located east of Fresno. Unused flows ultimately discharge into "banking" facilities used for groundwater recharge.

According to FID, the irrigation season typically runs from mid-February to October, but dates may vary. During non-irrigation season the canal conveys storm water and other FID operational flows. The majority of the storm water runoff entering the canal is pumped from detention basins operated by the Fresno Metropolitan Flood Control District.

Discharges:

Flows in Herndon Canal are regulated by FID. According to FID, the maximum flow conveyed at the bridge site location is 550 cfs. The Flood Insurance Study prepared by the Federal Emergency Management Agency (FEMA) lists the maximum discharge as 600 cfs. For this study, the higher discharge value reported by FEMA is used.

Stage/Velocity:

Based on a site review performed January 27, 2009, the high-water mark was observed on the pier wall to be at an elevation of 302 feet. A hydraulic analysis was performed to evaluate the water surface elevation based on FID's maximum flow and FEMA's maximum discharge value. A Manning's roughness coefficient of 0.025 and a slope of 0.07% were used in the analysis. Calculated water surface elevations from the hydraulic analysis were lower than the observed high-water mark.

FID acknowledges that downstream weirs and control structures exist which may potentially raise the water surface level, but they have stated that the water surface elevation will not be allowed to exceed the observed high-water mark at the bridge site.

The existing bridge has a minimum of 1.7 feet of available freeboard from the high-water elevation to the existing bridge soffit. The proposed median bridge widening will not result in a lower soffit elevation and will maintain the existing available freeboard.

Table 1 below provides a summary of the hydraulic analysis.

Table 1: Hydraulic Summary*
Herndon Canal Bridge (Br. No. 42-0129R/L)
Existing Bridge Soffit Elevation = 303.7 ft

Description	Discharge (cfs)	Average Velocity (fps)	Water Surface Elevation (ft)	Available Freeboard (ft)
Fresno Irrigation District's Maximum Flow	550	3.4	300.7	3.0
Q100 per FEMA's Flood Insurance Study	600	3.5	300.9	2.8
High Water Mark	---	---	302.0	1.7

Elevations are based on NGVD88
** For informational purposes only. The hydraulic analysis assumed no tailwater conditions resulting from downstream control structures.*

Streambed:

Note: References to left and right bank are based on facing in the downstream direction (i.e., left bank runs along Abutment 1 and the right bank runs along Abutment 3).

At the bridge site the canal flows in a southwesterly direction. Upstream of the bridge site the left bank of the canal is concrete lined and the right bank is an unlined earthen bank. Both banks are concrete lined beginning from the upstream side of the right bridge and continuing to the downstream side of the left bridge. Between the left bridge and the local frontage road bridge the banks are unlined. At the frontage road bridge both banks are concrete lined. Downstream of the frontage road bridge the left bank is concrete lined and the right bank is unlined.

The channel invert appears to be a natural silty-sand bed. The channel bottom was probed to a depth of 2 feet (to approximately an elevation of 295 feet). No evidence of a concrete lined bottom was found. FID has indicated that they do not believe the channel bottom is concrete lined immediately upstream nor downstream of the State's right-of-way. FID also indicated that they do not maintain the channel in the State's right-of-way.

Scour:

A scour analysis was performed assuming a silty-sand channel bottom. The potential scour is estimated at 2.3 feet and results in a scour elevation of 294.1 feet, which is at the top of the existing pile cap (i.e., pier wall from 1997 bridge widening). The maximum potential pier scour would occur at the upstream end of the pier wall at the debris nose. The resulting scour hole would typically decrease in depth downstream of the leading edge of the debris nose. A conservative rule-of-thumb is to assume that the scour hole will decrease 1 foot in depth for every 6 feet downstream of the leading edge of the pier. Therefore, no scour would be anticipated approximately 27 feet downstream of the leading edge of the pier nose where the top of the pile cap elevation is 296.1 feet (i.e., original 1959 pier wall).

Potential scour depths are summarized below in Table 2.

Table 2: Scour Summary
Herndon Canal Bridge (Br. No. 42-0129R/L)
Thalweg Elevation = 296.4 ft

Scour Type	Scour Depth	Comments
Degradation	N/A	Based on historical cross-sections there is no evidence to indicate degradation.
General Scour	N/A	No evidence to indicate contraction scour or other forms of general scour.
Local Pier Scour	2.3 ft	Assuming a silty-sand channel bed.
Local Abutment Scour	N/A	Abutments are protected by concrete lining.
Total Potential Scour Depth	2.3 ft	The potential scour elevation is at the top of the existing pile cap.
Total Potential Scour Elevation	294.1 ft	
Elevations are based on NGVD88		

Drift:

Some drift (i.e., floating debris) accumulation was observed during the site visit. The debris nose appeared to be effective as no drift was observed at the upstream end of the pier wall where scour potential is greatest.

Bank Protection:

Concrete lining in reasonably good condition exists along the banks of the existing bridge abutments and at the proposed abutment locations for the median bridge widening. According to Structure Design, the existing canal will not be disturbed by the proposed bridge widening.

Summary & Recommendations:

The existing structure meets hydraulic requirements and is not scour critical. The proposed median bridge widening will not adversely affect the hydraulic capacity or the current scour rating of the bridge.

A hydrologic summary of the bridge site is provided in Table 3 below.

Table 3: Hydrologic Summary
Herndon Canal Bridge (Br. No. 42-0129R/L)
Drainage Area: N/A(1)

	Design Flood	**Base Flood**	**Overtopping Flood / Flood of Record**
Frequency	---(2)	*N/A*	*N/A*
Discharge	600 cfs	*N/A*	*N/A*
Water Surface Elevation at Bridge	302.0 ft (3)	*N/A*	*N/A*

Elevations are based on NGVD88

Notes: (1) *Distributive canal.*
(2) *Regulated flows.*
(3) *High-water elevation governs due to potential hydraulic impacts from downstream control structures*

Flood plain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested or affected parties should make their own investigation.

Memorandum

Flex your power!
Be energy efficient!

To: RODNEY SIMMONS
Bridge Design Branch 17
Structure Design
Division of Engineering Services

Attn: Ramon Reyes

Date: April 16, 2009

File: 06-FRE-99 PM28.4
06-442611
North Fresno 6-Lane
Herndon Canal Bridge
(Center Widening)
Br. 42-0129

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES – MS 5

Subject: Foundation Report

1.0 INTRODUCTION

Per your request dated January 23, 2009, the Office of Geotechnical Design - North (OGDN) has prepared this Foundation Report (FR) for the proposed center widening of the Herndon Canal Bridge (Bridge No. 42-0129) located on State Route 99 at post mile (PM) 28.4 in Fresno County.

2.0 SCOPE

The scope of this report includes:

1. Review of "As-Built" information of the existing bridge and site reconnaissance,
2. Review of available published information related to the site including site geology and seismicity,
3. Work with District design project engineers and biologist, Drilling Services, and Fresno Irrigation District (owner of the canal) in pursuit of the necessary permits to perform the field investigation,
4. Conducting filed investigation including two soil test borings,
5. Generating and submitting draft log of borings for LOTB,
6. Review of findings of field investigation,

7. Performing laboratory tests on the soil samples collected from field investigation,
8. Discussion of the project with Structure Design project engineer,
9. Performing engineering analysis, calculations, and developing recommendations, and
10. Completing this report.

In preparing this report, the following documents were reviewed:

- Type Selection Recommendations, State Route 99, "North Fresno 6-Lane", 06-FRE-180 PM26.7/30.6, Dist-EA 06-442601, Design Branch 17, January 8, 2009,
- "North Fresno 6-Lane" Type Selection Meeting Minutes, 06-FRE-180 PM26.7/30.6, Dist-EA 06-442601, January 8, 2009,
- General Plan, Herndon Canal Center (Widen), Design Branch 17, January 8, 2009,
- Preliminary Geologic Recommendations and Resource Estimate for Advance Planning Study, Herndon Canal Bridge (Widening), Bridge No. 42-0129R/L, November 17, 2000,
- Foundation Plan, Herndon Canal Bridge (Widen), Bridge No. 42-129R/L, Post Mile 28.4, Sheet 2 of 14, November 6, 1996,
- Logs of Test Borings No. 1, Herndon Canal Bridge (Widen), Bridge No. 42-129R, Post Mile 28.4, Sheet 13 of 14, November 6, 1995,
- Logs of Test Borings, Herndon Canal, Bridge 42-129, December 9, 1958,
- A foundation installation observation letter for Herndon Canal Bridge, Bridge No. 42-129, R. E. Haverkamp, Bridge Department, Division of Highway, December 10, 1958,
- A letter of Foundation Data for Herndon Canal Bridge, Bridge No. 42-129, R. W. Reynolds, T.L. Sommers, May 1, 1957,
- Foundation Investigation, Herndon Canal Br. (Widen), Bridge No. 42-0129 R/L, July 19, 1994,
- Supplemental Foundation Recommendations, Herndon Canal Br. (Widen), Bridge No. 42-0129 R/L, August 31, 1994,
- Supplemental Foundation Recommendations, Herndon Canal Br. (Widen), Bridge No. 42-0129 R/L, September 8, 1994,
- Supplemental Foundation Recommendations, Herndon Canal Bridge (Widen), Bridge No. 42-0129 R/L, October 19, 1994,

- Foundation Plan No. 1, Herndon Canal Bridge, Bridge No. 42-0129LR, Post Mile 28.4, April 15, 2009

3.0 PROJECT DESCRIPTION

The existing Herndon Canal Bridge consists of two individual bridges, one for each direction on Route 99. The median (space between the two bridges) is discrete from the bridges, exposing the Herndon Canal. The bridge on the northbound lanes was widened on its east side in 1994. The original bridges are supported on 16-inch diameter Cast-in-Drilled-Hole (CIDH) piles. The bridge widening is supported on 12-inch driven concrete piles.

The proposed project will construct an additional bridge in the median connecting the two existing bridges, and consequently widen Route 99 at the project location. It is desired to re-utilize the existing CIDH piles located beneath the center of the canal to support the proposed bridge bent.

4.0 FIELD INVESTIGATION

Two soil test borings were performed at the site on March 10, 2009. Boring R-09-001 was performed about 19 feet north of the canal in the median adjacent to the northbound lanes of Route 99, and extended to a depth of 101.5 feet below the existing ground surface. Boring R-09-003 was performed about 38 feet south of the canal in the medium adjacent to the southbound lanes of Route 99, and extended to a depth of 81.5 feet below the existing ground surface. The borings were advanced using the rotary wash method coupled with the Standard Penetration Testing (SPT) and the standard split spoon sampling. Upon the completion of boring, piezometers were installed into the boreholes. The boreholes were backfilled with clean sands and sealed with bentonite.

Sheets of Logs of Test Boring (LOTB) for R-09-001 and R-09-003 as well as the previous As-Built LOTBs, which are to be incorporated in the project plans, are being prepared by Geotechnical Services, Office of Geotechnical Support Branch D – Contracts, Graphics & Records, and will be forwarded when completed. Mrs. Irma Gamarra-Remmen of the Contracts, Graphics, & Records branch may be contacted directly for information on the LOTB.

5.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Geology

Regionally, the Herndon Canal Bridge is located within the San Joaquin Valley, a two-hundred-mile long, seventy-mile wide structural trough that forms the southern portion of the Great Valley geomorphic province. The valley is bound on the north by the Sacramento Valley, on the south by the Tehachapi Mountains, on the east by the Sierra Nevada Mountains, and on the west by the Coast Ranges. The valley is filled with several hundred feet of non-marine sedimentary deposits underlain by older marine sediments. It is estimated the total thickness of these sedimentary deposits exceeds 30,000 feet.

Locally, the Herndon Canal Bridge is located approximately 3 miles south of the San Joaquin River. The floodplain deposits of the river generally consist of clays, silts, sands, and organic materials.

5.2 Seismicity

Based on the Caltrans California Seismic Hazard Map 1996, the Herndon Canal Bridge is located within a distance of less than 62 miles (100 kilometers) from two active faults. The Prairie Creek-Spenceville-Dentman (PSD) fault is located approximately 34 miles north-northwest of the bridge. The Coast-Ranges Sierran Block (CSB) Boundary Zone fault is located approximately 40 miles west of the bridge. The maximum credible earthquake (MCE) moment magnitudes are 6.5 and 7.0 for PSD and CSB, respectively. Based on the attenuation equation by Geomatrix 97, the ground motion is controlled by CSB with an estimated peak bedrock acceleration (PBA) of 0.2g.

A Caltrans Seismic Design Criteria (CSDC) Acceleration Response Spectrum (ARS) curve corresponding to soil profile Type D is recommended for design of the proposed bridge. The recommended ARS Curve is provided in Figure 1 in the Appendix.

5.3 Surface Rupture and Liquefaction

There is no known fault projecting toward or passing directly through the project site. Therefore, the potential for surface rupture at the site due to fault movement is considered insignificant.

Liquefaction Potential is considered to be low at the site due to low ground motion as well as the high apparent densities and elevated fine contents of the subsurface materials.

5.4 Subsurface Condition

Based on the soil test borings and the above referenced As-Built LOTBs, the subsurface materials consist primarily of medium dense to very dense sands and very stiff to hard silts. The SPT values recorded in the sands and silts ranged from 17 blows per foot (bpf) to 50 blows for 4 inches of penetration.

6.0 GROUNDWATER

The above-referenced As-Built LOTBs (1958) indicate that groundwater was not encountered within 55 feet of the ground surface at the bridge site in 1957.

During our current field investigation (2009), groundwater was measured in the installed piezometers after the completion of drilling. Table 1 below summarizes the measured groundwater data. The groundwater depths measured in the piezometers (2009) are consistent with the findings documented in the previous investigation (1957).

Table 1. Measure Groundwater Data

Borehole	Ground Surface Elevation (ft)	Measured Groundwater Depth (feet)	Measured Groundwater Elevation (ft)	Measuring Date
R-09-001	306	58	248	March 10, 2009
		58	248	March 25, 2009
R-09-003	304.9	N/A[1]	N/A[1]	March 10, 2009
		N/A[1]	N/A[1]	March 25, 2009

Note:
1. Borehole/piezometer abandoned due to clogging.

Herndon Canal is an irrigation canal with peak discharge of 550 to 600 cubic feet per second (cfs). The high water elevation in the canal is estimated to be at 300 feet above mean sea level (MSL). At the time of our current exploration (2009), the water level in the canal was measured at about 6 feet below the adjacent ground surfaces (tops of the canal embankments in the median of Route 99).

We understand that both sidewalls of the canal were concrete lined and the liner extends to a minimum 2 feet below the flow line (bottom of the canal). The bottom of the canal is not lined. By gravity, water in the canal could seep into the subsurface soils. We anticipate that the seepage would predominantly be in a downward vertical direction. It is unlikely that water would seep over a significant horizontal distance. Therefore, if seepage occurs, it would primarily be underneath the canal and would not impact the abutments.

7.0 SCOUR

The anticipated scour depth in the canal is provided at elevation of approximately 294 feet, which is 2 feet above the bottom elevation of the existing pile footing of 292 feet. Therefore, scour is not anticipated to significantly affect the existing CIDH piles located below the center of the canal.

8.0 CORROSION

Laboratory tests on soil corrosivity are in progress. Actual condition will be verified once test results become available. Based on the tests previously performed for the existing bridges and other bridges in the project area, soil is anticipated to be non-corrosive at the site.

9.0 EXISTING BRIDGE FOUNDATIONS

The existing Herndon Canal Bridge structures were designed and built by the California Department of Public Works, Division of Highways in 1958. The bridges are 65-foot long, two-span bridges built with reinforced concrete slabs, concrete wall bents, and open-end diaphragm abutments.

The original foundations of the existing bridges consist of 16-inch diameter CIDH piles. The design load of the CIDH piles is 90 kips. The tip elevation of the original CIDH piles is about 274.5 feet.

The bridge located on northbound lanes was widened at its east side in 1994. The foundation for the 1994-widening portion of the bridge consist of 12-inch Class 140 precast, prestressed driven concrete piles. The specified tip elevations of the driven piles were 268 feet and 258 feet for the piles supporting the abutments and bent, respectively. Class 90 precast, prestressed driven concrete piles were used to support the bent nose extension with a specified pile tip elevation of 268 feet.

The following Table 2 provides a summary of the existing bridge foundations.

Table 2. Existing Bridge Foundations

Bridge No.	Location	Year Built	Foundation	
			Pile Type	Pile Length[1] (ft)
42-0429 R	Route 99 Southbound	1958	16" CIDH	17'
42-0429 L	Route 99 Northbound	1958	16" CIDH	17'
	Route 99 Northbound (Widening)	1994	12" Driven Concrete Pile	34'

Note:
1. Pile length is interpreted based on the specified pile tip elevation and the original ground surface elevation of 300 feet.

10.0 FOUNDATION RECOMMENDATIONS

10.1 CIDH Foundations for Abutments 1 and 3

CIDH piles with a diameter of 24 inches were selected to support the proposed center-widening bridge Abutments 1 and 3. Pile data recommendations are presented in the following Tables 3 and 4.

Table 3. Foundation Recommendations For Abutments 1 and 3

Support No.	Pile Type	Cut-off Elevation (ft)	LRFD Service-1 Limit State Load (kips) Per Support		LRFD Service-1 Limits State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elevations (ft)	Specified Tip Elevation (ft)
			Total	Permanent				
Abut 1	24" CIDH	301.49	402.66	265.74	100.66	202	(a) 261 (b) 286	261
Abut 3	24" CIDH	302.34	402.66	265.74	100.66	202	(a) 262 (b) 287	262

Notes:

1. Recommendations are based on Load Resistance Factor Design (LRFD) and load data provided by Structure Design (SD).
2. The Design Tip Elevation recommended herein is controlled by: (a) Compression (Service Limit) and (b) Settlement.
3. The Design Tip Elevation controlled by lateral load is typically provided by SD.
4. The Specified Tip Elevation recommended herein shall not be raised if controlled by lateral load.
5. Skin friction is used to determine the Design Tip Elevation controlled by compression and settlement.

Table 4. Abutments 1 and 3 Pile Data Table

Support No.	Pile Type	Nominal Resistance (kips)		Design Tip Elevations (ft)	Specified Tip Elevation (ft)
		Compression	Tension		
Abut 1	24" CIDH	202	N/A	(a) 261 (b) 286	261
Abut 3	24" CIDH	202	N/A	(a) 262 (b) 287	262

Notes:

1. The Design Tip Elevation is controlled by: (a) Compression (Service Limit) and (b) Settlement.
2. The Specified Tip Elevation recommended herein shall not be raised.
3. Skin friction is used to determine the Design Tip Elevation controlled by compression and settlement.

10.2 CIDH Foundations for Bent 2

At the center of the canal, the existing CIDH piles are to be re-utilized to support the proposed center-widening bridge Bent 2. Based on the above referenced previous reports, plans, and LOTBs, it is our understanding that the existing piles are CIDH piles with a design diameter of 16 inches and a tip elevation of 275 feet. The design load of these CIDH piles was 90 kips.

CIDH piles develop their load-carry capacity through skin friction and end bearing. Due primarily to the difficulty in bottom cleaning of the drilled shaft, current Caltrans practice typically does not consider end bearing for the capacity of the CIDH pile when the tip of the pile is embedded in soils. Based on the LOTBs, the existing CIDH piles are tipped in soils. By using only skin friction and with a factor of safety (FS) of 2.0, the allowable axial capacity of the existing CIDH piles would be about 44 kips, which is less then the original design load of 90 kips.

It is our assessment that the existing CIDH piles for this structure were designed using both skin friction and end bearing. Based on the above referenced LOTBs and the results of current soil test borings, at elevations of 275 feet and immediately below, the soils are medium dense to very dense sands and very stiff to hard silts. The above referenced foundation installation observation letter documented "Examination of the material removed form the drilled holes showed about the same results as the soil borings taken by the preliminary investigation crew." and "Material at the bottom of the holes was compact gray-green silt and fine sand. None of the holes caved in to any great extend while drilling." As such for all these CIDH piles constructed, end bearing could develop and contribute to the axial pile capacity with minimal settlement. Based on these subsurface conditions and an assumption of clean bottom of drilled shaft, the allowable end bearing capacity was estimated to be on the order of 46 kips. As a result, the existing CIDH piles would have a combined allowable axial capacity of 90 kips. The assumption of clean bottom of drilled shaft is critical for development of end bearing capacity of the CIDH piles with minimum settlement.

With the bottom of the canal being un-lined and with passage of time, potential of seepage, predominantly in a downward vertical direction as discussed above, could produce localized perched water condition, which may influence the development of the load carrying capacity of the existing CIDH piles and induce additional settlement. Therefore, we recommend that the bottom of the canal be concrete lined covering the sections underneath the bridges and extending a minimum of 50 feet at both upstream and downstream directions beyond the bridges. The concrete liner shall be constructed as soon as the District receives permit for working in the canal.

In conclusion,

- The existing CIDH piles may be used to support the proposed Bent 2 of the center-widening bridge. We recommend that a design load of no greater than 90 kips or a required nominal resistance of no greater than 180 kips be used for these existing CIDH piles.

- Settlement monitoring for the existing CIDH piles at Bent 2. The monitoring should be performed on daily basis during construction and in the first week after the completion of bridge construction, then on weekly basis for the next three months, and on monthly basis for the following year. If excessive settlement (more than 1 inch) occurs, the capacity of the existing CIDH piles should be re-evaluated. Depending on the results of the monitoring, additional foundation supports, such as micropiles, may be implemented if required.

11.0 CONSTRUCTION CONSIDERATIONS

Localized loose materials containing silts, sands, and gravels are present at the site. Caving of the drilled shafts may be expected. Temporary casing may be used to stabilize the drilled shafts. If used, temporary casing shall be retrieved from drilled shafts. All necessary precautions should be exercised to prevent pile anomalies.

Localized groundwater condition may be encountered, which may necessitate the use of "wet method" to construct the CIDH piles.

12.0 PROJECT INFORMATION

Standard Special Provision S5-280, "Project Information", discloses to bidders and contractors a list of pertinent information available for their inspection prior to bid opening. The following is an excerpt from SSP S5-280 disclosing information originating from Geotechnical Services. Items listed to be included in the Information Handout will be provided in Acrobat (.pdf) format to the addressee(s) of this report via electronic mail.

Data and information attached with the project plans are:

An As-Built Log of Test Borings No. 2, Herndon Canal, Bridge 42-129

An As-Built Log of Test Boring No. 1, Herndon Canal Bridge (Widen), Bridge No. 42-129R

A Log of Test Boring, Herndon Canal Bridge (Center Widening), 42-0129

Data and information included in the Information Handout provided to the bidders and contractors are:

Foundation Report for EA 06-442611, dated April 16, 2009.

Data and information available for inspection at the District Office:

None.

Data and information available for inspection at the Transportation Laboratory are:

None.

If any changes to the structure are proposed during the final project design, this office, Office of Geotechnical Design North (OGDN) should review those changes to determine if the foundation recommendation herein still applies.

If you have any questions or comments, please call Carolyn Zhen at (916) 227-1055, Thomas Song at (916) 227-1054, or John Huang at (916) 227-1037.

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Geotechnical Design - North

c: R. E. Pending
Structure OE
PCE (E-copy)
DME (E-copy)
GDN File
GS File Room

Appendix

Herndon Canal Bridge (Center Widen)
Bridge No. 42-0129
06-442601

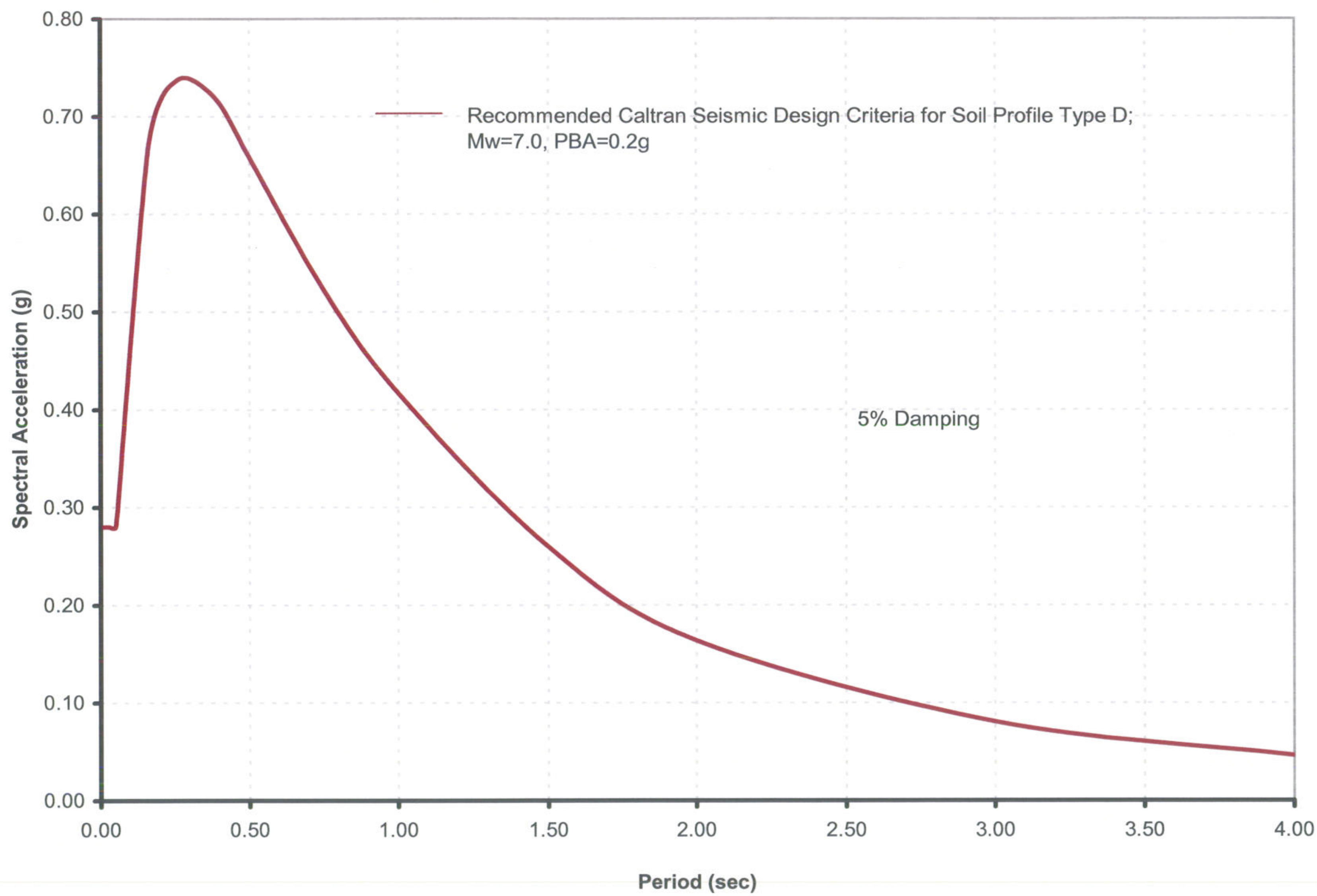

Figure 1. Acceleration Response Spectrum Recommended for Design

Memorandum

Flex your power!
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To: RODNEY SIMMONS
Bridge Design Branch 17
Structure Design
Division of Engineering Services

Attn: Ramon Reyes

Date: February 5, 2009

File: 06-FRE-99 PM30.5
06-442601
North Fresno 6-Lane
Grantland Ave. UC (Widen)
Br. 42-0127L/R

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES – MS 5

Subject: Existing Retaining Wall Piles Compression Capacity Assessment

Introduction

Per your request dated January 23, 2009, an Existing Retaining Wall Piles Compression Capacity Assessment is prepared for the proposed widening of the Grantland Ave. UC (Bridge No. 42-0127L/R). This bridge is located within Fresno County, on Highway 99, at PM 30.5. The existing structure consists of two single span bridges (left and right) with retaining walls between the bridges in the median area. All abutments and retaining walls are founded on 45-ton design load, 16-inch Cast-in-Drilled-Hole (CIDH) piles constructed in 1958. The project is proposed to widen the bridge in the median. We understand that those existing retaining walls and existing retaining wall piles would be used to support the proposed widening bridge abutments.

Pertinent Reports and Investigations

In preparing this assessment, the following documents were reviewed:

- Preliminary Geologic Recommendations and Resource Estimate for Advance Planning Study, dated November 2000.
- Preliminary Seismic Design Recommendations, dated December 5, 2000.
- As-built Log of Test Boring for Grantland Ave UC, dated December 9, 1958.

- Foundation Data dated June 14, 1957.
- A Memo documenting CIDH construction dated December 10, 1958

Geology

The Grantland Avenue UC widening project is located within the San Joaquin Valley, a two hundred miles long, seventy miles wide structural trough that forms the southern portion of the Great Valley geomorphic province. The valley is bound on the north by the Sacramento Valley, on the south by the Tehachapi Mountains, the east by the Sierra Nevada Mountains, and on the west by the Coast Ranges. The structural trough is filled with several hundred feet of nonmarine sedimentary deposits, which are in turn underlain by much thicker deposits of older marine sediments. It is estimated the total thickness of sedimentary deposits within the Great Valley exceeds 30,000 feet.

The Grantland Ave Bridge is located approximately 1 mile south of the San Joaquin River. The river's floodplain deposits generally consist of gray and brown clay, silt, sand, and organic materials. A review of the 1958 Log of Test Borings for the Grantland Ave. UC indicates the underlying sedimentary deposits consist of interbedded fine to medium coarse sand and cemented silty sand/sandy silt, with limonite staining and small concretions (duripan). Densities varied from slightly compact to very dense.

Seismicity

Based on the Caltrans California Seismic Hazard Map 1996, the bridge site is located between two active faults. The Prairie Creek-Spencevill-Dentman (PSD, normal faulting) fault located about 37 miles north northwest of the bridge, and the Coast-Ranges Sierran Block fault (CSB, reverse including thrust fault mechanism) located about 43 miles southwest of the proposed project. The maximum credible earthquake moment magnitude for PSD and CSB faults are 6.5 and 7.0, respectively. Based on the attenuation equation by Geomatrix 97, the ground motion is controlled by the CSB fault with an estimated peak bedrock acceleration of 0.2g.

Based on the As-Built Log of Test Borings a Caltrans Seismic Design Criteria Acceleration Response Spectrum curve corresponding to soil profile Type D is recommended for design. The recommended ARS Curve is attached in Figure 1.

Groundwater

Groundwater was not encountered within 45 feet of the ground surface during field investigation in February of 1957. However, increased demand for groundwater since that time suggests that current groundwater levels will have further declined. Based on available DWR water well data in the area, the groundwater table is at least 55 feet below existing ground surface.

Surface Rupture

The potential for surface rupture at the site due to fault movement is considered insignificant since there are no known faults projecting towards or passing directly through the project site.

Liquefaction Potential

Due to lack of groundwater in shallow depth and relative small peak ground acceleration (PGA), the potential for liquefaction is considered to be very low.

Existing Retaining Wall Piles Compression Capacity Assessment

It is understood that the proposed widening abutments will be at current retaining wall locations, and would use current retaining wall foundation piles to support the proposed abutments. The required design load of the proposed widening abutment piles is 88 kips/pile according to Structure Design.

Per LOTB and information provided by Structure Design, the existing retaining wall piles are 16" CIDH (45-ton design load) with a pile length of approximately 18 feet.

Due to the concerns of bottom clean-up, Caltrans current practice does not typically account for end bearing when CIDH is tipped in soils. Based on current design practice of using skin resistance only, the existing piles would have about 22 tons allowable capacity with FS=2, less than the original design load requirement of 45 tons.

It is our assessment that the existing piles were designed using both end bearing and skin friction. A memo, dated December 10, 1958, documenting CIDH construction mentioned "Materials at the bottom of the holes was dense gray or light brown silty sand with some clay present. The holes caved in very little during drilling operations". If this was the case for every pile constructed, all the piles would have end bearing capacity with minimal settlement. The estimated end bearing capacity would be in the order of 23 tons allowable based on existing boring log and clean pile hole bottom. If all the piles had clean bottoms as described in the 1958 memo, the piles would be able to support design load of 45 tons and ultimate load of 90 tons, in terms of geotechnical capacity.

Groundwater level is another issue. For sandy soils, groundwater can reduce the soil effective stress by roughly half. This can reduce the CIDH piles capacity. However, based on LOTB and available groundwater data in the area, groundwater level is at least 55 feet below existing ground surface. Therefore, groundwater issue may not be a concern.

The existing bridge abutments on both sides of the proposed abutments are supported on the same type of piles, design load, and tip elevation as the current retaining wall piles. No foundation problems have been recorded based on information provided by Structure Design. However, according to Structure Design, some wall lateral deflections (less than 0.5") have occurred at the current retaining wall locations, but no foundation settlements were reported.

In conclusion, it is our opinion that the current retaining wall piles may be used to support the proposed widening abutment design load of 88 kips per pile, provided that a settlement monitor be installed at each abutment. Readings shall be taken every day during construction and the first week after the completion of the bridge construction, then once a week for the first three months, and once a month for two years. Any excessive movement (more than 0.5 " of settlement) shall be brought to Structure Engineer and Geotechnical Engineer/Geologist for immediate attention. Evaluation and remediation measures shall be applied.

Project Information

Standard Special Provision S5-280, "Project Information", discloses to bidders and contractors a list of pertinent information available for their inspection prior to bid opening. The following is an excerpt from SSP S5-280 disclosing information originating from Geotechnical Services. Items listed to be included in the Information Handout will be provided in Acrobat (.pdf) format to the addressee(s) of this report via electronic mail.

Data and information attached with the project plans are:

None

Data and information included in the Information Handout provided to the bidders and contractors are:

None

Data and information available for inspection at the District Office:

None.

Data and information available for inspection at the Transportation Laboratory are:

-Existing Retaining Wall Piles Compression Capacity Assessment, dated February 5, 2009

-As Built LOTB with pile tips dated December 9, 1958

-Foundation Data for EA 06-442601, Br # 42-127L/R, dated June 14, 1957.

If any changes to the structure are proposed during the final project design, the Office of Geotechnical Design – North should review those changes to determine if the foundation recommendation herein still applies.

If you have any questions or comments, please call Carolyn Zhen at (916) 227-1055 or John Huang at (916) 227-1037.

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c: R. E. Pending
Structure OE
PCE (E-copy)
DME (E-copy)
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Grantland Ave UC (Widen)
Bridge No. 42-0127
06-442601

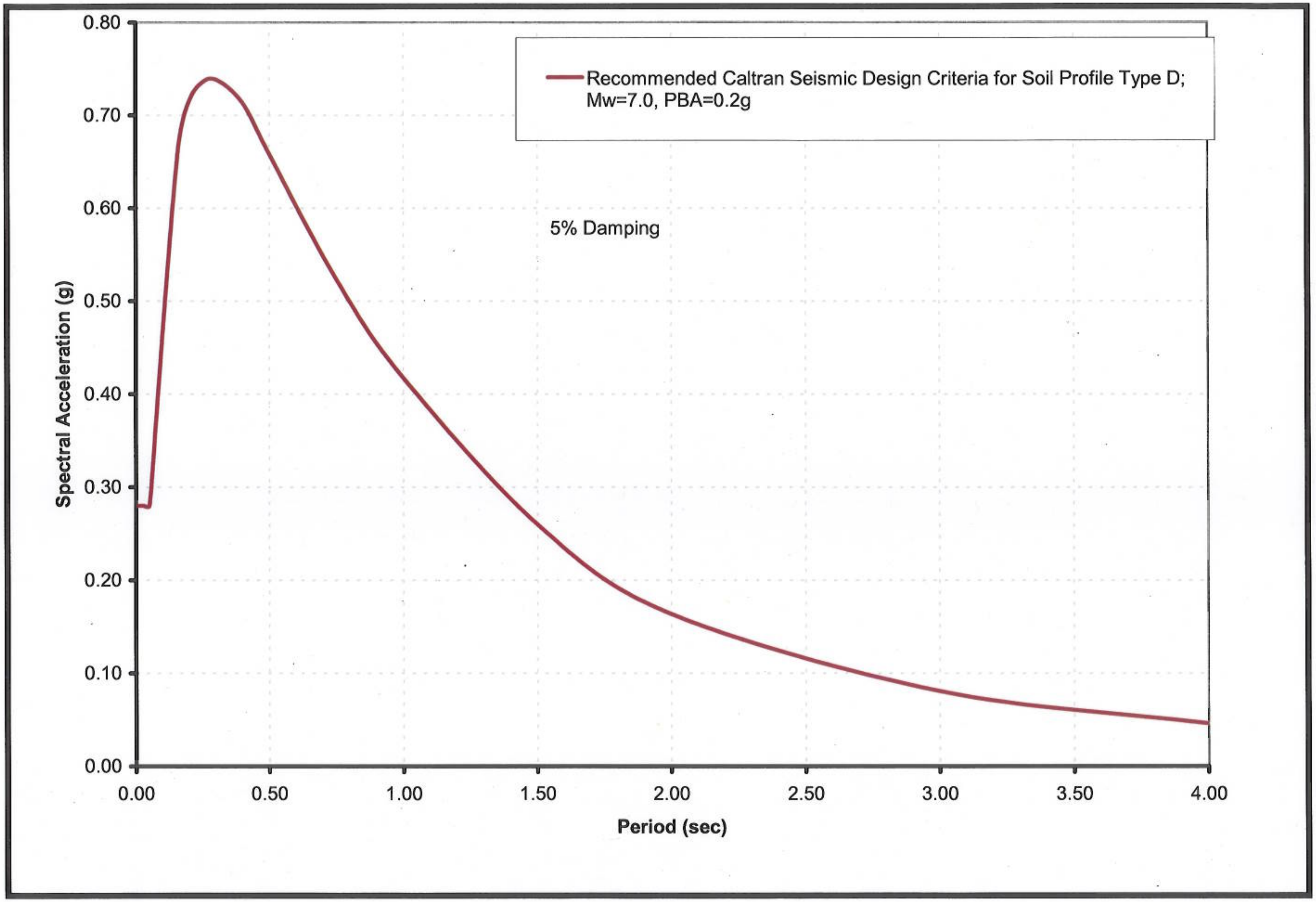

Figure 1. Acceleration Response Spectrum Recommended for Design

Memorandum

Flex your power!
Be energy efficient!

To: RODNEY SIMMONS
Bridge Design Branch 17
Structure Design
Division of Engineering Services

Attn: Ramon Reyes

Date: February 5, 2009

File: 06-FRE-99 PM27.3
06-442611
North Fresno 6-Lane
Biola Junction OH (Widen)
Br. 42-0131

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES – MS 5

Subject: Foundation Report

Introduction

Per your request, dated January 23, 2009, a Foundation Report (FR) has been prepared to provide the foundation recommendations for the widening of the Biola Junction OH (Bridge No. 42-0131). This bridge is located within Fresno County on Highway 99 at PM 27.3. The proposed project is to widen the bridge in the median. The existing structures are three-span bridges whose abutments are founded on Cast-in-Drilled-Hole (CIDH) piles with bents founded on spread footings.

Pertinent Reports and Investigations

In preparing this report, the following documents were reviewed:

- Preliminary Geologic Recommendations and Resource Estimate for Advance Planning Study, dated November 17, 2000.
- Preliminary Seismic Design Recommendations, dated December 5, 2000.
- As-built Log of Test Boring for Biola Junction OH, dated August 1, 1958.

Site Geology

The Biola Junction OH bridge widening project is located within the San Joaquin Valley, a two hundred miles long, seventy miles wide structural trough that forms the southern portion of the Great Valley geomorphic province. The valley is bounded on the north by the Sacramento Valley, on the south by the Tehachapi Mountains, the east by the Sierra Nevada Mountains, and on the west by the Coast Ranges. It is filled with several hundred feet of nonmarine sedimentary deposits, which are in turn underlain by much thicker deposits of older marine sediments. It is estimated the total thickness of these sedimentary deposits exceeds 30,000 feet.

A review of the 1959 Log of Test Borings for Biola Junction OH indicates the underlying alluvial deposits consist of interbedded silt, fine to coarse sand, and gravels. Near surface layers include duripan caused by long term exposure over time. These are described in the LOTB as very dense and very hard, gray green silt interbedded with partly cemented silt with limonite staining and concretions.

Seismicity

Based on the Caltrans California Seismic Hazard Map 1996, the bridge site is located between two active faults. The Prairie Creek-Spencevill-Dentman (PSD, normal faulting) fault located about 37 miles north northwest of the bridge, and the Coast-Ranges Sierran Block fault (CSB, reverse including thrust fault mechanism) located about 43 miles southwest of the proposed project. The maximum credible earthquake moment magnitude for PSD and CSB faults are 6.5 and 7.0, respectively. Based on the attenuation equation by Geomatrix 97, the ground motion is controlled by the CSB fault with an estimated peak bedrock acceleration of 0.2g.

Based on the As-Built Log of Test Borings, a final Caltrans Seismic Design Criteria Acceleration Response Spectrum curve corresponding to soil profile Type D is recommended for design. The recommended ARS Curve is attached in Figure 1.

Groundwater

California Department of Water Resources (DWR) has monitored groundwater levels in the project vicinity for many years. The DWR monitoring wells show groundwater levels ranging from 70 to 100 feet below the existing ground surface. According to As-Built LOTB of Biola Junction OH, groundwater was encountered at the depth of 55 feet (elevation 240 feet) in February 14, 1957. However, increased demand for groundwater since 1957 suggests that current groundwater levels will have declined significantly.

Liquefaction Potential

Due to the relative density of the foundation materials and deep ground water elevation (about 55 feet below original ground), the potential for liquefaction is considered to be very low.

Surface Fault Rupture Hazard

The potential for surface rupture at the site due to fault movement is considered insignificant since there are no known faults projecting towards or passing directly through the project site.

Corrosion

Laboratory tests on soil corrosivity are in progress. According to existing corrosion tests done for nearby bridges, project area soil is not anticipated to be corrosive. Actual conditions can be verified once test results are available.

Geotechnical Recommendations

CIDH piles are recommended for the abutments, and spread footings are recommended for the bents to support the proposed widening structure. Geotechnical recommendations are presented in the following tables.

Table 1. Foundation Recommendation for Abutments - CIDH

Support	Pile	Cut-off Elevation (ft)	LRFD Service-I Limit State Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Required Nominal Resistance (kips)	Design Tip Elevation (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kips)
			Total	Permanent					
Abut 1	24" CIDH	321.30	889	533	90	180	277(a)	277	N/A
Abut 4	24" CIDH	320.65	889	534	90	180	277(a)	277	N/A

Notes:

1. Recommendations are based on Load Resistance Factor Design (LRFD) and load data provided by Structure Design (SD) on the Foundation Design Data Sheet.
2. See MTD 3-1 for definitions and applications of the recommended design parameters.
3. Design tip elevation is controlled by (a) Compression.

Table 2. Foundation Recommendation for Bents - Spread Footings

Support	Footing Size (ft)		Bottom of Footing Elevation (ft)	Minimum Footing Embedment Depth (ft)	LRFD		
					Service	Strength φ_b =0.45	Extreme Event φ_b = 1.00
	B	L			Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
Bent 2	12	12	286	8	6.1	17.1	38.0
Bent 3	12	12	286	8	6.1	17.1	38.0

Notes:

4. Recommendations are based on Load Resistance Factor Design (LRFD) and load data provided by Structure Design (SD) on the Foundation Design Data Sheet.
5. See MTD 4-1 for definitions and applications of the recommended design parameters.
6. A resistance factor of 0.45 is used to calculate the available geotechnical resistance in Strength Limit State. A resistance factor of 1.0 is used to calculate the available geotechnical resistance in Extreme Limit State.
7. Permissible net contact stress is based on 1-inch permissible settlement.

For contract Plans:

Table 3. Pile Data Table for Abutments

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	24" CIDH	180	0	277(a)	277	N/A
Abut 4	24" CIDH	180	0	277(a)	277	N/A

Notes:

8. Recommendations are based on Load Resistance Factor Design (LRFD) and load data provided by Structure Design (SD) on the Foundation Design Data Sheet.
9. See MTD 3-1 for definitions and applications of the recommended design parameters.
10. Design tip elevation is controlled by (a) Compression.

Table 4. Spread Footing Table for Bents

Location	Service	Strength	Extreme Event
	Permissible Net Contact Stress (Settlement) (ksf)	Factored Gross Nominal Bearing Resistance $\varphi b = 0.45$ (ksf)	Factored Gross Nominal Bearing Resistance $\varphi b = 1.00$ (ksf)
Bent 2	6.1	17.1	38
Bent 3	6.1	17.1	38

Notes:

11. Recommendations are based on Load Resistance Factor Design (LRFD) and load data provided by Structure Design (SD) on the Foundation Design Data Sheet.
12. See MTD 4-1 for definitions and applications of the recommended design parameters.
13. A resistance factor of 0.45 is used to calculate the available geotechnical resistance in Strength Limit State. A resistance Factor of 1.0 is used to calculate the available geotechnical resistance in Extreme Limit State.
14. Design tip elevation is controlled by (a) Compression.

Construction Consideration

All earthworks shall follow Section 19 of Caltrans Standard Specifications. Based on a Construction memo, dated April 2, 1959, documenting the original bridge foundation CIDH pile installation, no caving-in of the holes was observed. However, due to sandy nature of the on-site soils, temporary casing may be used if caving occurs.

If unsuitable native material is found below the bottom of the recommended spread footing elevations, these materials shall be removed and replaced by sub-excavating and replacing with engineered fill that is compacted to 95% relative compaction.

Project Information

Standard Special Provision S5-280, "Project Information", discloses to bidders and contractors a list of pertinent information available for their inspection prior to bid opening. The following is an excerpt from SSP S5-280 disclosing information originating from Geotechnical Services. Items listed to be included in the Information Handout will be provided in Acrobat (.pdf) format to the addressee(s) of this report via electronic mail.

Data and information attached with the project plans are:

None

Data and information included in the Information Handout provided to the bidders and contractors are:

Foundation Report for EA 06-442601, Br # 42-0131, dated February 5, 2009.
As-Built LOTB for Biola Junction OH, dated August 4, 1958.

Data and information available for inspection at the District Office:

None.

Data and information available for inspection at the Transportation Laboratory are:

Foundation Recommendation for Biola Junction Overhead, dated June 17, 1957.
Construction memo documenting foundation construction, dated April 2, 1959.

If any changes to the structure are proposed during the final project design, the Office of Geotechnical Design – North should review those changes to determine if the foundation recommendation herein still applies.

If you have any questions or comments, please call Carolyn Zhen at (916) 227-1055 or John Huang at (916) 227-1037.

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Attachment(s):
Figure 1 – ARS Curve for Biola Junction Overhead (Widen)

c: R. E. Pending
Structure OE
PCE (E-copy)
DME (E-copy)
GDN File
GS File Room

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Biola Junction Overhead (Widen)
Bridge No. 42-0131
06-442601

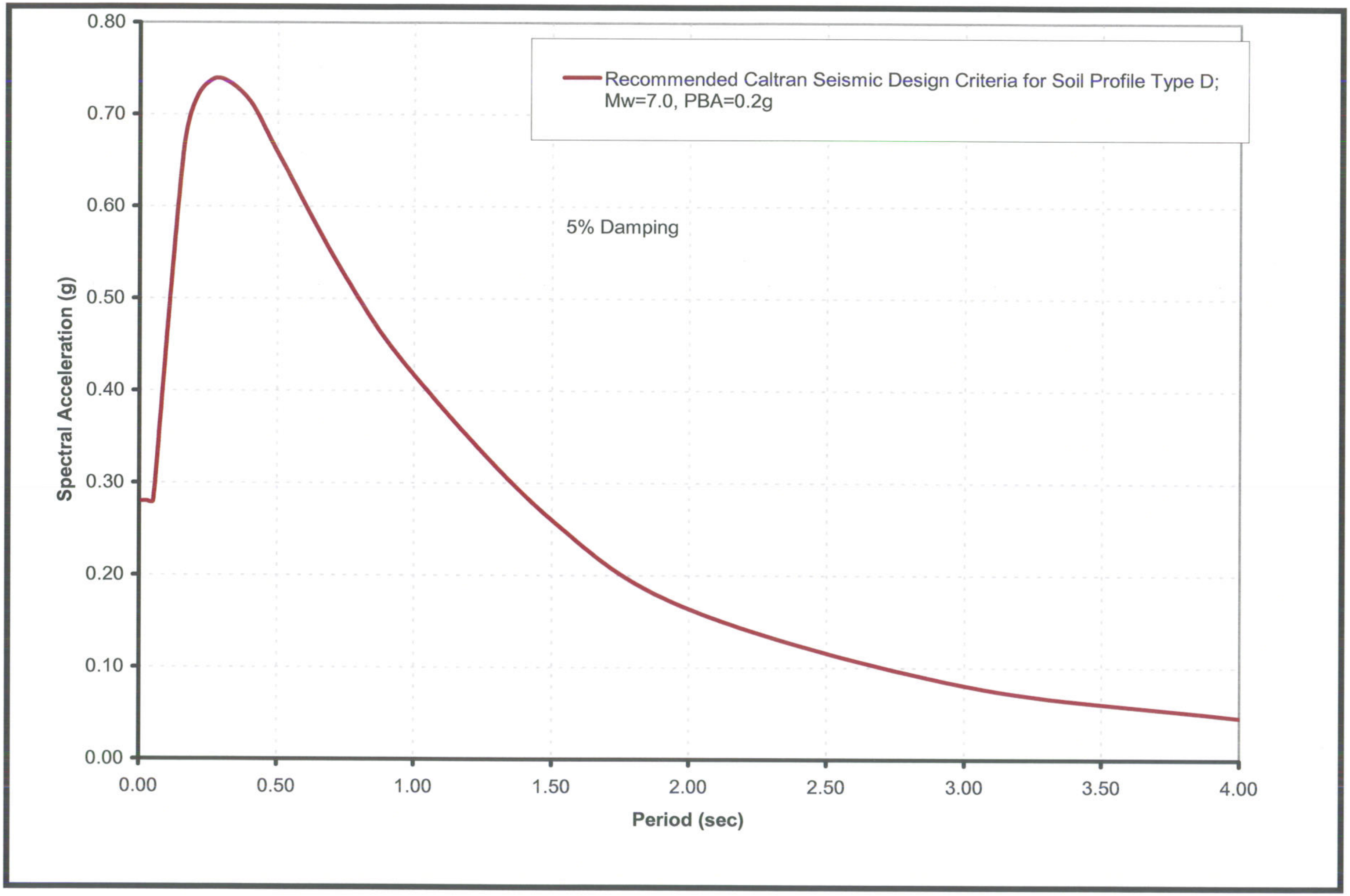

Figure 1. Acceleration Response Spectrum Recommended for Design