FOR CONTRACT NO.: 01-262004

INFORMATION HANDOUT - PART 2

FOUNDATION REPORTS

2-1. Foundation Report for the Baechtel Creek Retaining Wall (New), Bridge No. 10E0009, dated January 15, 2009
2-2. Revised Foundation Report for the Baechtel Creek Retaining Wall (New), Bridge No. 10E0009, dated July 3, 2009
2-3. Foundation Report for the A1RW, Bridge No. 10E0001, dated December 7, 2006
2-4. Foundation Report for the UPP Creek Bridge, Bridge No. 10-0305, dated March 11, 2009
2-5. Foundation Report for the UPP Creek Bridge, Bridge No. 10-0174K, dated March 11, 2009
2-6. Foundation Report for the UPP Creek (SB Off-Ramp), Bridge No. 10-0174K, dated March 11, 2009
2-7. Foundation Report for the UPP Creek (NB On-Ramp), Bridge No. 10-0174S, dated March 11, 2009
2-8. Foundation Report for U4 Retaining Wall Bridge No. 10E0002, dated December 8, 2006.
2-9. Foundation Report for the S101-W20 Connector Bridge No. 10-0129F, dated July 6, 2009.
2-10. Foundation Report for the Rte 101/20 Separation, Bridge No. 10-0128R/L, dated October 20, 2006.
2-11. Foundation Report for the Quail Meadows OH, Bridge No. 10-0171, dated January 15, 2009.
2-12. Revised Foundation Report for the Quail Meadows OH, Bridge No. 10-0171, dated June 19, 2009.
2-13. Foundation Report for the Haehl Creek Bridge, Bridge No. 10-0129R/L, dated November 9, 2006.
2-14. Foundation Report for the Haehl Creek Bridge, Bridge No. 10-0159, dated June 18, 2008.
2-15. Foundation Report for the Floodway Viaduct, Bridge No. 10-0165, dated February 23, 2009.
2-16. Foundation Report for the East Hill Road UC, Bridge No. 10-0157, dated June 18, 2008.
2-17. Foundation Report for the E20-N101 Connector Bridge, Bridge No. 10-0129G, dated October 27, 2006.

ROUTE: 01-MEN-101-R69.4/78.9

Foundation Reports

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: January 15, 2009

File: 01-MEN-101-KP R74.430/PM R46.25
EA#01-262001
Baechtel Creek Ret Wall (NEW)
Br. No. 10E0009

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Baechtel Creek Retaining Wall (Br. No. 10E0009). The Office of Geotechnical Design North, Branch A completed a subsurface investigation during August 2006 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (August 2008) along with a review of the available General Plan dated November 3, 2008 and Foundation Plan dated October 30, 2008. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Baechtel Creek Retaining Wall (Br. No. 10E0009) will be incorporated as part of the Willits Bypass Project located within the city limits of Willits in Mendocino County (Figures 1 and 2). The proposed new retaining wall will be located on the west shoulder of the new proposed 101 Freeway approximately 167 meters south of the new proposed Floodway Viaduct (Br. No. 10-0165). In this report, the Stations are referenced from the "BCRW" LOL and the Offsets are referenced from the "A" Line shown on the General and Foundation Plans dated November 3, 2008 and October 30, 2008, respectively. The proposed Baechtel Creek Retaining Wall (BCRW) will be supported on driven steel H-Piles. Table 1 summarizes the beginning and ending stations of the proposed retaining wall with corresponding heights.

Table 1. Proposed Baechtel Creek Retaining Wall (Br. No. 10E0009)

Retaining Wall Number	Station "BCRW" LOL		Offset from "A" Line		Length (m)	Height (mm)
	Begin (m)	End (m)	Begin (m)	End (m)		
BCRW	146+00.00	147+10.00	17.60	17.60	110.00	4800-6100

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the center of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary deposits) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Quatenary alluvium. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation during August 2006.

The subsurface investigation consisted of two mud rotary borings (Nos. B-70 and B-71). The mud rotary borings were advanced using a self-casing wireline-diamond coring method extending down to a maximum depth of 37.03 m (121.5 ft). The equipment used to drill borings B-70 and B-71 consisted of a CS2000 Drill Rig (CME) drill rig equipped with a Diedrich automatic hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler at 1.5-meter intervals. Selected soil samples were bagged for laboratory testing.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed retaining wall (Br. No. 10E0009) generally consist of approximately 18.6 m to 19.2 m (61 ft. to 63 ft) of very soft to hard clay and clay with sand overlying medium dense to dense clayey gravel and well graded gravel with silt, clay and sand. The foundation materials were encountered down to the maximum depth explored of 37.03 m (121.5 ft), an elevation of 376.85 m (1236.4 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water was encountered at elevation 410.14 m (1345.6 ft) in boring B-71 during the August 2006 subsurface investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Corrosion Evaluation

Composite soil samples were collected in one of the borings during the August 2006 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for Baechtel Creek Retaining Wall (Br. No. 10E0009)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639901	B-70	1.52	6.87	3700	N/A	N/A
C639870	B-70	6.40	7.41	1800	N/A	N/A
C639905	B-71	1.52-2.44	6.14	5600	N/A	N/A
C639905	B-71	5.03-5.49	6.73	4800	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Moisture Content, Specific Gravity, Consolidation, Unconfined Compression Testing, Triaxial (Unconsolidated-Undrained) and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 0.6 kilometer southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Fill Settlement

Due to the presence of compressible clay soils beneath the proposed retaining wall, it is recommended that the roadway alignment adjacent to the proposed Baechtel Creek Retaining Wall be preloaded. Preloading is to be achieved by placement of roadway and retaining wall fill prior to construction of the proposed retaining wall foundation. Settlement will take place during and after the construction of the roadway and retaining wall fill. It has been estimated that the maximum total settlement of the proposed roadway and retaining wall fill will be about 260 millimeters.

It has been calculated that 90% of the settlement of the roadway and wall fill will take place approximately 270 days after placement of the fill is completed. If it is necessary to accelerate the settlement, placement of additional surcharge fill and/or the use of wick drains may be recommended. Details of these measures will be included in the Geotechnical Design Report to be prepared by this Office.

Regardless of the target settlement or the estimated settlement time period, 260 mm and 270 days, respectively, the rate of settlement obtained during the settlement monitoring will determine if the majority of the settlement in the fill is complete. That is if the settlement rate is noticeably reduced sooner than the end of the estimated settlement time period, then it would be acceptable for construction of the retaining wall foundations to commence. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period until the settlement rate diminishes.

Foundation Recommendations

The following foundation recommendations are for the new proposed Baechtel Creek Retaining Wall (Br. No. 10E0009). At this location, it is recommended driven steel HP 250X62 "H" piles be used to support the proposed Type 1 Retaining Wall. The specified pile tip elevations, shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed Baechtel Creek Retaining Wall, Type 1 Retaining Wall (Br. No. 10E0009).

Retaining Wall Station along "BCRW" LOL (Approximate)	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
146+00.00 to 146+37.00	4800	HP 250X62	400	800	0	412.575 (1353.59)	391.0 (1) (1283.0)	386.0 (1283.0)
146+37.00 to 146+77.00	5500	HP 250X62	400	800	0	412.575 (1353.59)	391.0 (1) (1283.0)	386.0 (1283.0)
146+77.00 to 147+10.00	6100	HP 250X62	400	800	0	412.575 (1353.59)	391.0 (1) (1283.0)	386.0 (1283.0)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Structural Foundations shall be contacted for further recommendations.

3. Specified pile tip elevations shown in Table 2 above are based on preloading of compressible clay soils achieved by placement of roadway/retaining wall fill prior to construction of the proposed retaining wall foundations. If preloading is not performed, the Office of Geotechnical Design-North is to be contacted for revised foundation recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. Settlement shall be monitored. Monitoring devices shall consist of settlement platforms placed at the bottom of the roadway/retaining wall fill adjacent to the proposed Baechtel Creek Retaining Wall. A minimum of three settlement devices shall be placed beneath the centerline of the roadway embankment adjacent to the Baechtel Creek Retaining Wall at locations to be determined by the Engineer.

3. Due to the presence of compressible clay soils beneath the proposed retaining wall, it is recommended that the roadway alignment adjacent to the proposed Baechtel Creek Retaining Wall be preloaded. Preloading shall be achieved by placement of roadway and retaining wall fill prior to construction of the Baechtel Creek Retaining Wall. A 270-day waiting period is required prior to installation of piles and construction of the wall. The fill settlement shall be monitored during the waiting period. If it is determined that the settlement has ceased prior to the end of the 270-day waiting period, with the written approval of the Engineer, the waiting period may be terminated at that time. In the opposite case, if the settlement does not change by the time the estimated settlement period is over, monitoring shall continue, extending the settlement period until the settlement rate diminishes.

4. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035, or Reid Buell (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard Special Provisions 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:

A. Log of Test Borings for Baechtel Creek Retaining Wall, Bridge No. 10E0009.

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

A. Foundation Report for Baechtel Creek Retaining Wall, Bridge No. 10E0009, dated January 15, 2009.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North

Report by:

REZA MAHALLATI
Senior Materials & Research Engineer
Office of Geotechnical Design-North

Supervised by:

REID BUELL, C.E.G. NO. 1481
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STATE OF CALIFORNIA

cc: OGDSN
GS File Room
Reid Buell
R.E. Pending
Structure OE

REFERENCES

Federal Highway Administration (FHWA), Design and Construction of Driven Pile Foundations: Workshop Manual – Volume 1 (Pub. No. FHWA HI-97-013) published January 1997, Revised November1998.

Harden, Deborah R., 1998, California Geology, Prentice Hall, 252 pp.

Jennings, Charles W. and Strand, Rudolph G., 1960, Geologic Map of California [Ukiah Sheet] and Index to Geologic Mapping, California Division of Mines, scale 1:250,000, 2 sheets.

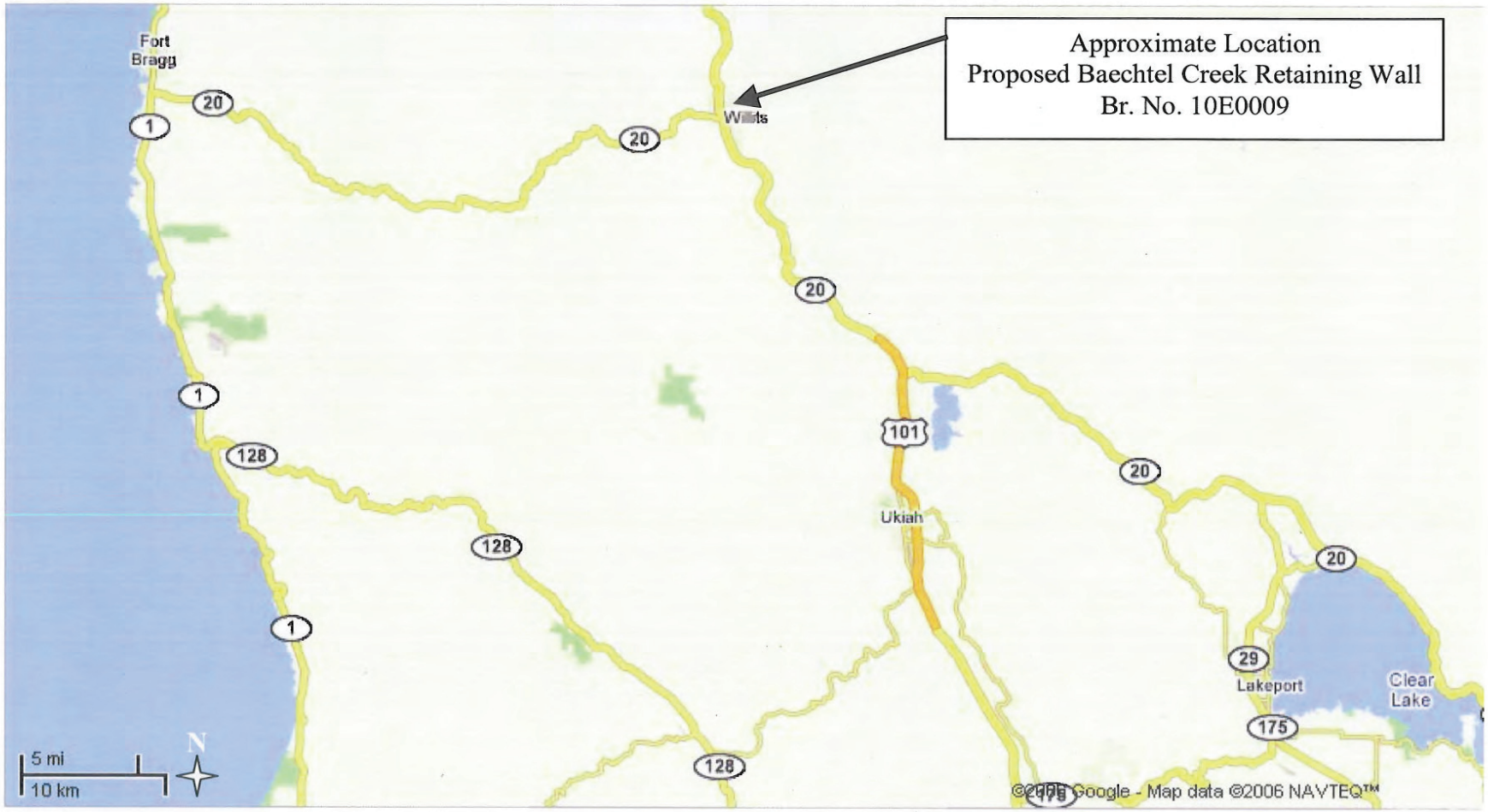
Norris, Robert M., and Webb, Robert M., 1990, Geology of California, Second Edition, John Wiley & Sons, Inc., pp 57, 359, 363.

Seismic Design Criteria (2006), California Department of Transportation, Version 1.4.

State of California, Department of Transportation (Caltrans):

- Standard Plans, July 2004 and May 2006
- Standard Specifications, July 2004 and May 2006
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 January 15, 2009	Location Map
	01-MEN-101 KP R74.43/PM R46.25 **Baechtel Creek Retaining Wall, Br. No. 10-0E0009**	Figure 1

Caltrans

Division of Engineering Services
Geotechnical Services
Geotechnical Design – North

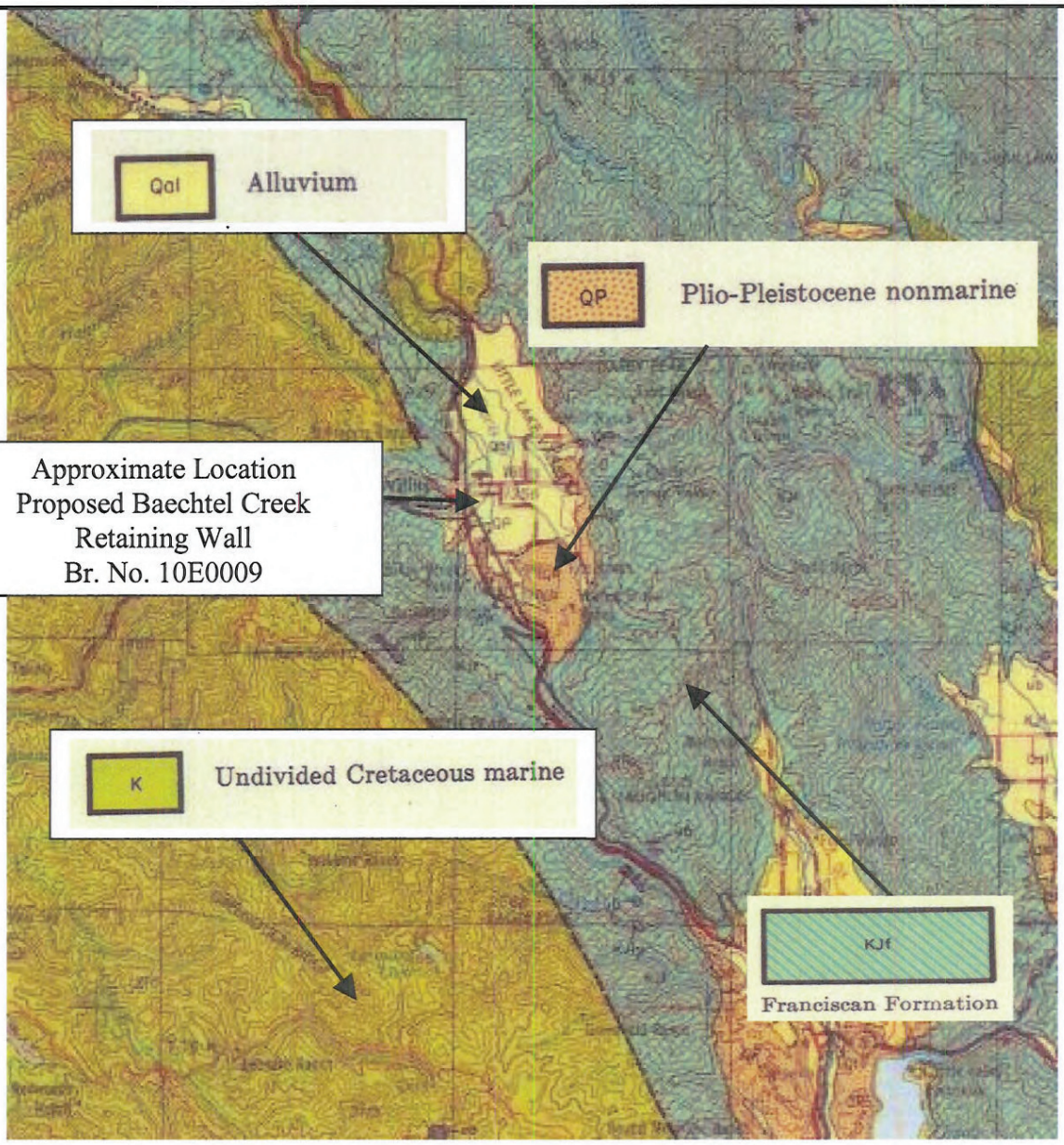
EA: 01-262001

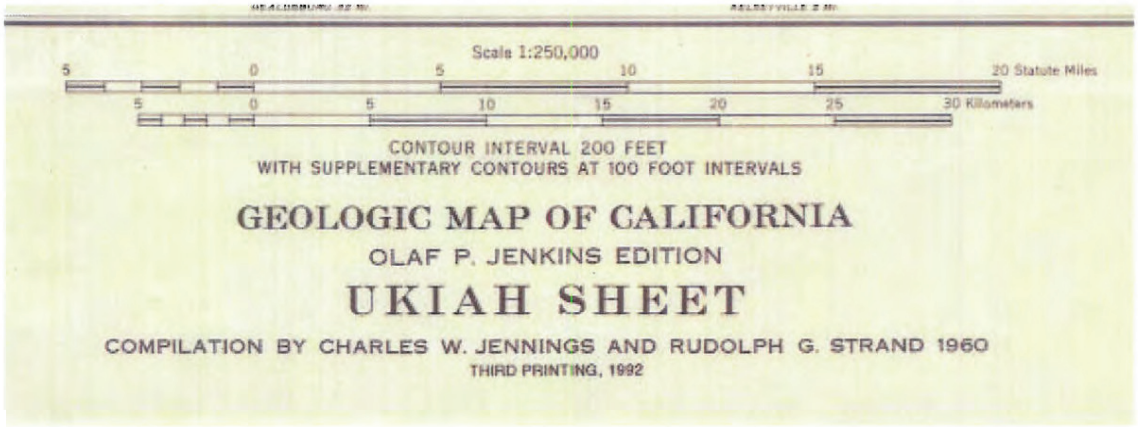
January 15, 2009

Location Map

01-MEN-101 KP R74.43/PM R46.25
Baechtel Creek Retaining Wall, Br. No. 10-0E0009

Figure 2





Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 January 15, 2009	Geologic Map
	01-MEN-101 KP R74.43/PM R46.25 **Baechtel Creek Retaining Wall,** **Br. No. 10E0009**	Figure 3

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: July 3, 2009

File: 01-MEN-101-KP R74.430
EA#01-262001
Baechtel Creek Ret Wall (NEW)
Br. No. 10E0009

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Revised Foundation Report

Introduction/Scope of Work

The following recommendations are in response to a request for revised foundation recommendations made by the Office of Bridge Design North – Branch 2 for the proposed Baechtel Creek Retaining Wall, Bridge No. 10E0009. The following report supersedes our original Baechtel Creek Ret Wall (New) report dated January 15, 2009. This revised foundation report includes revised foundation recommendations which account for potential downdrag loads due to the elimination of the settlement waiting period recommended in our previous report.

This report presents the foundation recommendations for the new proposed Baechtel Creek Retaining Wall (Br. No. 10E0009). The Office of Geotechnical Design North, Branch A completed a subsurface investigation during August 2006 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (August 2006) along with a review of the available General Plan dated November 3, 2008 and Foundation Plan dated October 30, 2008. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

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in Mendocino County (Figures 1 and 2). The proposed new retaining wall will be located on the west shoulder of the new proposed 101 Freeway approximately 167 meters south of the new proposed Floodway Viaduct (Br. No. 10-0165). In this report, the Stations are referenced from the "BCRW" LOL and the Offsets are referenced from the "A" Line shown on the General and Foundation Plans dated November 3, 2008 and October 30, 2008, respectively. The proposed Baechtel Creek Retaining Wall (BCRW) will be supported on driven steel H-Piles. Table 1 summarizes the beginning and ending stations of the proposed retaining wall with corresponding heights.

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Composite soil samples were collected in one of the borings during the August 2006 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for Baechtel Creek Retaining Wall (Br. No. 10E0009)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639901	B-70	1.52	6.87	3700	N/A	N/A
C639870	B-70	6.40	7.41	1800	N/A	N/A
C639905	B-71	1.52-2.44	6.14	5600	N/A	N/A
C639905	B-71	5.03-5.49	6.73	4800	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Moisture Content, Specific Gravity, Consolidation, Unconfined Compression Testing, Triaxial (Unconsolidated-Undrained) and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results are available upon request.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 0.6 kilometers southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Foundation Recommendations

The following foundation recommendations are for the new proposed Baechtel Creek Retaining Wall (Br. No. 10E0009). At this location, it is recommended driven steel HP 360X132 "H" piles be used to support the proposed Type 1 Retaining Wall. The specified pile tip elevations, shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed Baechtel Creek Retaining Wall, Type 1 Retaining Wall (Br. No. 10E0009).

Retaining Wall Station along "BCRW" LOL (Approximate)	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)	Nominal Driving Resistance Required kN
				Compression kN	Tension kN				
146+00.00 to 146+37.00	4800	HP 360X132	400	800	0	412.575 (1353.59)	396.5 (1) (1300.9) 388.0 (2) (1273.0)	388.0 (1273.0)	1850
146+37.00 to 146+77.00	5500	HP 360X132	400	800	0	412.575 (1353.59)	396.5 (1) (1300.9) 388.0 (2) (1273.0)	388.0 (1273.0)	1850
146+77.00 to 147+10.00	6100	HP 360X132	400	800	0	412.575 (1353.59)	396.5 (1) (1300.9) 388.0 (2) (1273.0)	388.0 (1273.0)	1850

Note: Design Pile Tip Elevation is controlled by the following demands: (1) Compression (2) Downdrag; due to downdrag potential, the nominal resistance value required for pile acceptance criteria for all driven steel HP 360x132 piles shall be 1850 kN.

Downdrag (negative skin friction) potential is estimated to exist to elevation 402.4 m (1320 ft.). Due to the potential for the upper soils to consolidate in response to the weight of the retaining wall backfill, skin friction force in this zone may act downward and thus become an added load to piles instead of providing resistance. The additional load due to downdrag is estimated to be 525 kN.

Geotechnical resistance, but not the overburden pressure, of the soil from the original ground surface to the bottom of the estimated compressible layer (elevation 402.4 m) was neglected in the pile capacity analyses. However, this geotechnical resistance will act on the pile during pile driving. During pile driving, the geotechnical resistance will be higher than the nominal resistance when the pile reaches specified tip elevation. Therefore, the nominal driving resistance required shall be1850 kN.

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Structural Foundations shall be contacted for further recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. Due to downdrag potential, the nominal driving resistance required for pile acceptance criteria for all driven steel HP 360x132 piles shall be 1850 kN.

3. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035, or Reid Buell (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard special Provisions 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:

A. Log of Test Borings for Baechtel Creek Retaining Wall, Bridge No. 10E0009.

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

A. Foundation Report for Baechtel Creek Retaining Wall, Bridge No. 10E0009, dated July 3, 2009.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North

Report by:

REZA MAHALLATI
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REFERENCES

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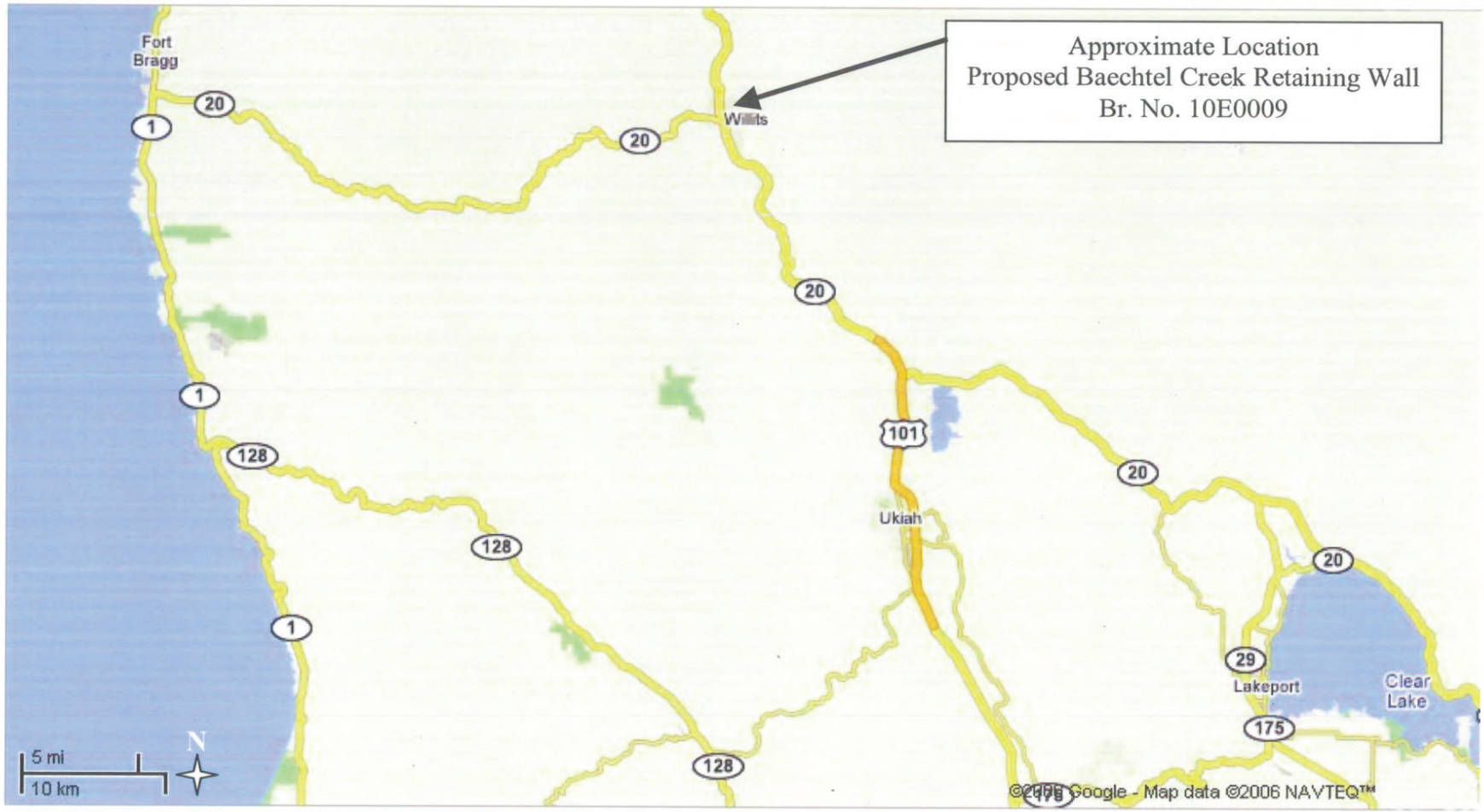
State of California, Department of Transportation (Caltrans):

- Standard Plans, July 2004 and May 2006
- Standard Specifications, July 2004 and May 2006
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 July 3, 2009	Location Map
	01-MEN-101 KP R74.43/PM R46.25 **Baechtel Creek Retaining Wall, Br. No. 10-0E0009**	Figure 1

Caltrans

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Geotechnical Services
Geotechnical Design – North

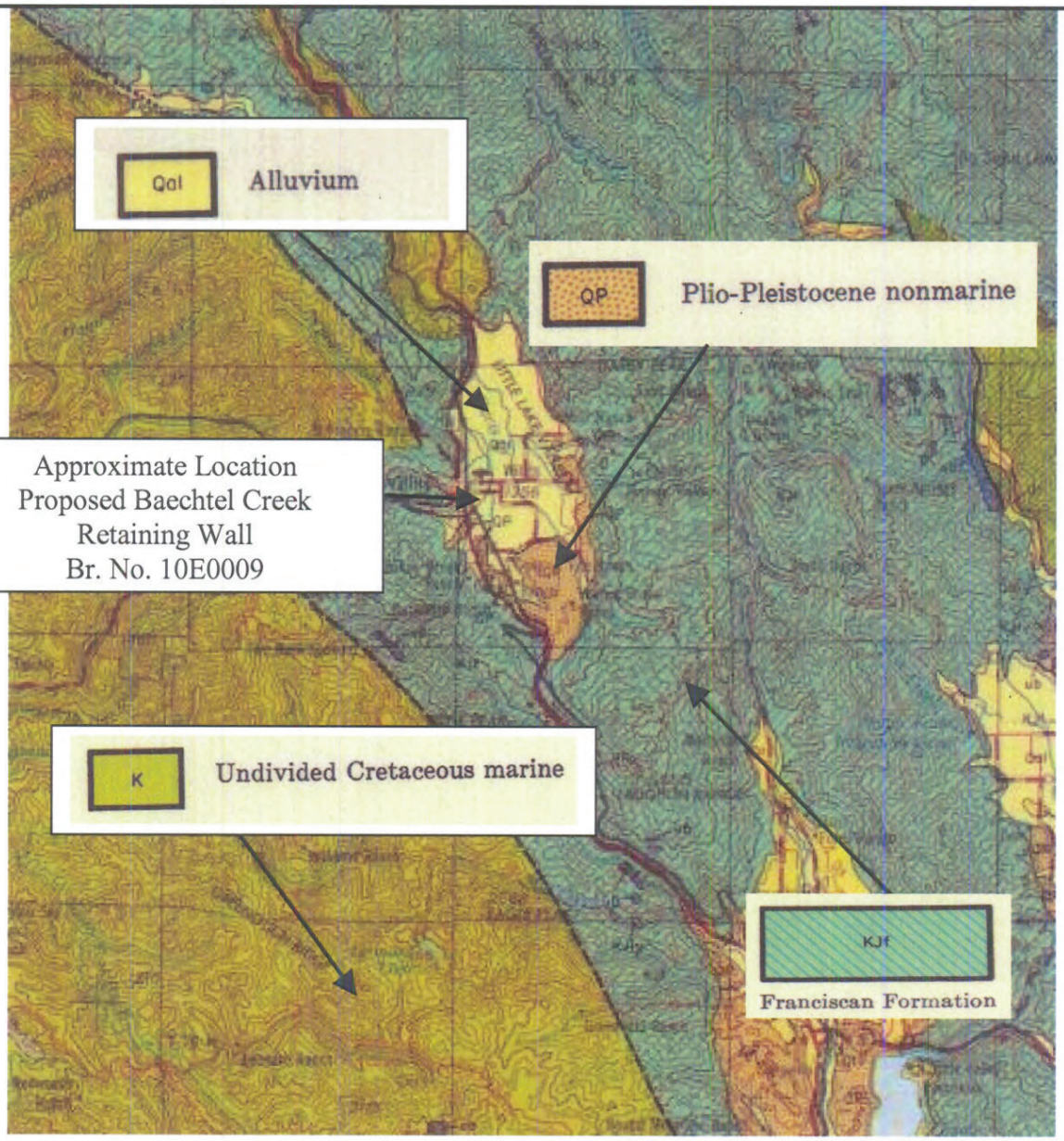
EA: 01-262001

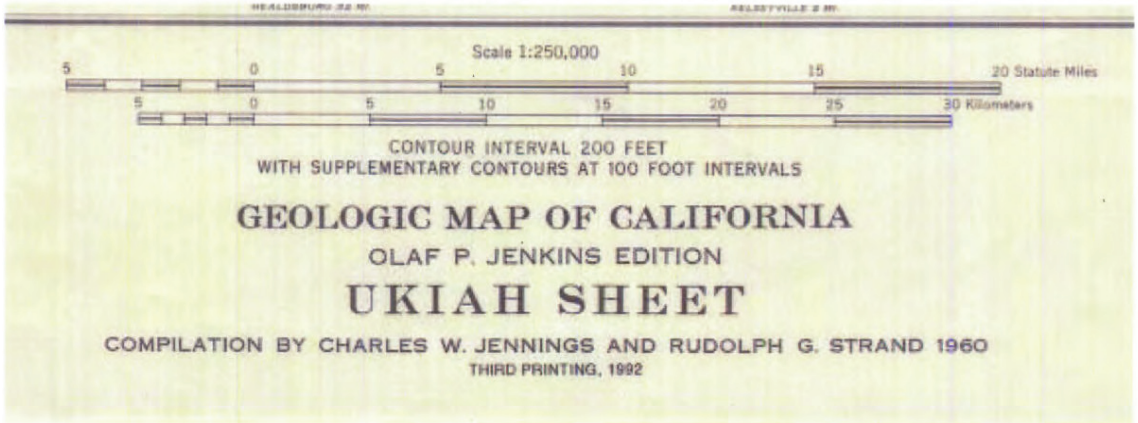
July 3, 2009

Location Map

01-MEN-101 KP R74.43/PM R46.25
Baechtel Creek Retaining Wall, Br. No. 10-0E0009

Figure 2





Division of Engineering Services Geotechnical Services Geotechnical Design – North	EA: 01-262001 July 3, 2009	Geologic Map
	01-MEN-101 KP R74.43/PM R46.25 **Baechtel Creek Retaining Wall,** **Br. No. 10E0009**	Figure 3

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: December 7, 2006

File: 01-MEN-101-KP R70.46/PM R43.78
EA#01-262001
A1 Retaining Wall (NEW)
Br. No. 10E0001

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed A1 Retaining Wall (Br. No. 10E0001). The Office of Geotechnical Design North, Branch A completed a subsurface investigation during May 2004 to June 2005 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (May 2004 to June 2005) along with a review of the available General Plan dated February 28, 2006 and Foundation Plan dated February 28, 2006. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new A1 Retaining Wall (Br. No. 10E0001) will be incorporated as part of the Willits Bypass Project located within the city limits of Willits in Mendocino County (Figures 1 and 2). The proposed new retaining wall will be located on the east shoulder of the new proposed northbound 101 Freeway between the new proposed Route 101/20 Separation Right Bridge (Br. No. 10-0128R) and the new proposed Haehl Creek Right Bridge (Br. No. 10-0129R). In this report, the Stations are referenced from the "A1RW" LOL and the Offsets are referenced from the "A" Line shown on the General and Foundation Plans dated February 28, 2006. The proposed A1 Retaining Wall (A1RW)

will be supported on driven steel H-Piles. Table 1 summarizes the beginning and ending stations of the proposed retaining wall with corresponding heights.

Table 1. Proposed A1 Retaining Wall (Br. No. 10E0001).

Retaining Wall Number	Station "A1RW" LOL		Offset from "A" Line		Length (m)	Height (mm)
	Begin (m)	End (m)	Begin (m)	End (m)		
A1RW	106+30.00	107+12.96	23.69	23.69	82.96	3000-6700

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the south end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary deposits) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Plio-Pleistocene nonmarine sedimentary deposits identified as the Cache Formation. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation from May 2004 to June 2005.

The subsurface investigation consisted of three mud rotary borings (Nos. B-1, B-24, and B-4) and three cone penetrometer soundings (No. C-2, C-3, and C-4). The mud rotary borings were advanced using a self-casing wireline-diamond coring method extending down to a maximum depth of 37.2 m (122.0 ft). The equipment used to drill borings B-1, B-24 and B-4 consisted of a Central Mine Equipment (CME) drill rig equipped with a CME automatic hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler at 1.5-meter intervals. Selected soil samples were bagged for laboratory testing. The cone penetromer soundings were used to supplement the subsurface investigation.

The subsurface investigation revealed the foundation material encountered at the location of the proposed retaining wall (Br. No. 10E0001) generally consist of approximately 12.5 m (41.0 ft) of firm to stiff sandy clay with a few interbeds of medium dense to dense silty sand and sand with gravel overlying very stiff to hard clay and dense to very dense poorly graded sand with gravel and silty sand. The foundation material was encountered down to the maximum depth explored of 37.2 m (122.0 ft), an elevation of 414.5 m (1360.1 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels varied from elevation 443.6 m (1455.4 ft) in boring B-4 and 446.0 m (1463.3ft) in Boring B-1 during the June 2004 subsurface investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Scour Evaluation

A Final Hydraulic Report (dated March 22, 2005) for the Haehl Creek structures was completed by the Office of Structure Maintenance and Investigations, Structure Hydraulics Branch. The report provided information for the Haehl Creek Bridge, right bridge (Br. No. 10-0129R) that is located near the location of the proposed A1RW. This information was used to evaluate the scour potential at the proposed A1RW due to the proximity of this nearby structure to the proposed retaining wall location.

According to this report, there are no local scour concerns for the nearby Haehl Creek Bridge, right bridge (Br. No. 10-0129R) since it is a single span structure. The report recommends the abutment foundations be designed assuming no ground support (lateral and vertical) as a result of soil loss due to possible degradation or lateral stream migration down to the current stream thalwag elevation (lowest elevation in channel). The approximate thalwag elevation provided for the proposed Haehl Creek Bridge, right bridge (Br. No. 10-0129R) is 442.5 m (1451.8 ft).

It is recommended that Rock Slope Protection (RSP) be designed by the District to mitigate migration of Haehl Creek towards the retaining wall.

For further information including site-specific scour assessment and mitigation measures, the Structures Hydraulics Branch should be contacted.

Corrosion Evaluation

Composite soil samples were collected during the subsurface investigation. The results will be forwarded once completed.

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Moisture Content, Specific Gravity, Consolidation, Unconfined Compression Testing, Triaxial (Unconsolidated-Undrained) and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request once the results are received.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 1.0 kilometers (0.6 miles) southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Foundation Recommendations

The following foundation recommendations are for the new proposed A1 Retaining Wall (Br. No. 10E0001). At this location, it is recommended driven steel HP 250X85 "H" piles be used to support the proposed Type 1 Retaining Wall. The specified pile tip elevations, shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed A1RW, Type 1 Retaining Wall (Br. No. 10E0001).

Retaining Wall Station along "A1RW" LOL (Approximate)	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
106+30.00 to 106+32.44	3000	HP 250X85	400	800	0	451.6 (1481.6)	432.8 (1, 2) (1420.1)	432.8 (1420.1)
106+32.44 to 106+34.88	4800	HP 250X85	400	800	0	449.9 (1476.0)	432.8 (1,2) (1420.1)	432.8 (1420.1)
106+34.88 to 106+39.76	5500	HP 250X85	400	800	0	448.8 (1472.4)	432.8 (1,2) (1420.1)	432.8 (1420.1)
106+39.76 to 106+49.52	6700	HP 250X85	400	800	0	447.2 (1467.2)	429.5 (1,2) (1409.0)	429.5 (1409.0)
106+49.52 to 106+78.80	6700	HP 250X85	400	800	0	446.6 (1465.2)	429.5 (1,2) (1409.0)	429.5 (1409.0)
106+78.80 to 107+08.08	6100	HP 250X85	400	800	0	446.3 (1464.2)	429.5 (1,2) (1409.0)	429.5 (1409.0)
107+08.08 to 107+10.52	5500	HP 250X85	400	800	0	447.0 (1466.5)	429.5 (1,2) (1409.0)	429.5 (1409.0)
107+10.52 to 107+12.96	4200	HP 250X85	400	800	0	448.2 (1470.5)	429.5 (1,2) (1409.0)	429.5 (1409.0)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 442.5 m (1451.8 ft) at A1 Type 1 Retaining Wall.*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Structural Foundations shall be contacted for further recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. The steel "H" pile section below the scour elevation has been designed to develop the required nominal resistance at all support locations.

3. Specialty equipment may be required for installation of the battered steel "H" piles. The contractor should be prepared to alternate from battered to vertical installation methods of the steel "H" piles.

4. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

5. The contractor should anticipate hard and erratic driving of the steel "H" piles due to the presence of very dense foundation material. The contractor should anticipate field cutting and splicing of the steel "H" piles. Refer to the LOTB sheets for details.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-7260, Jacqueline Martin at (916) 227-5503 or Reid Buell (916) 227-7190, of the Office of Geotechnical Design-North, Branch A.

Project Information

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

A. Foundation Report for A1RW, Bridge No. 10E0001, dated December 7, 2006.

Report by:

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Engineering Geologist
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Report by:

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R.E. Pending
Structure OE

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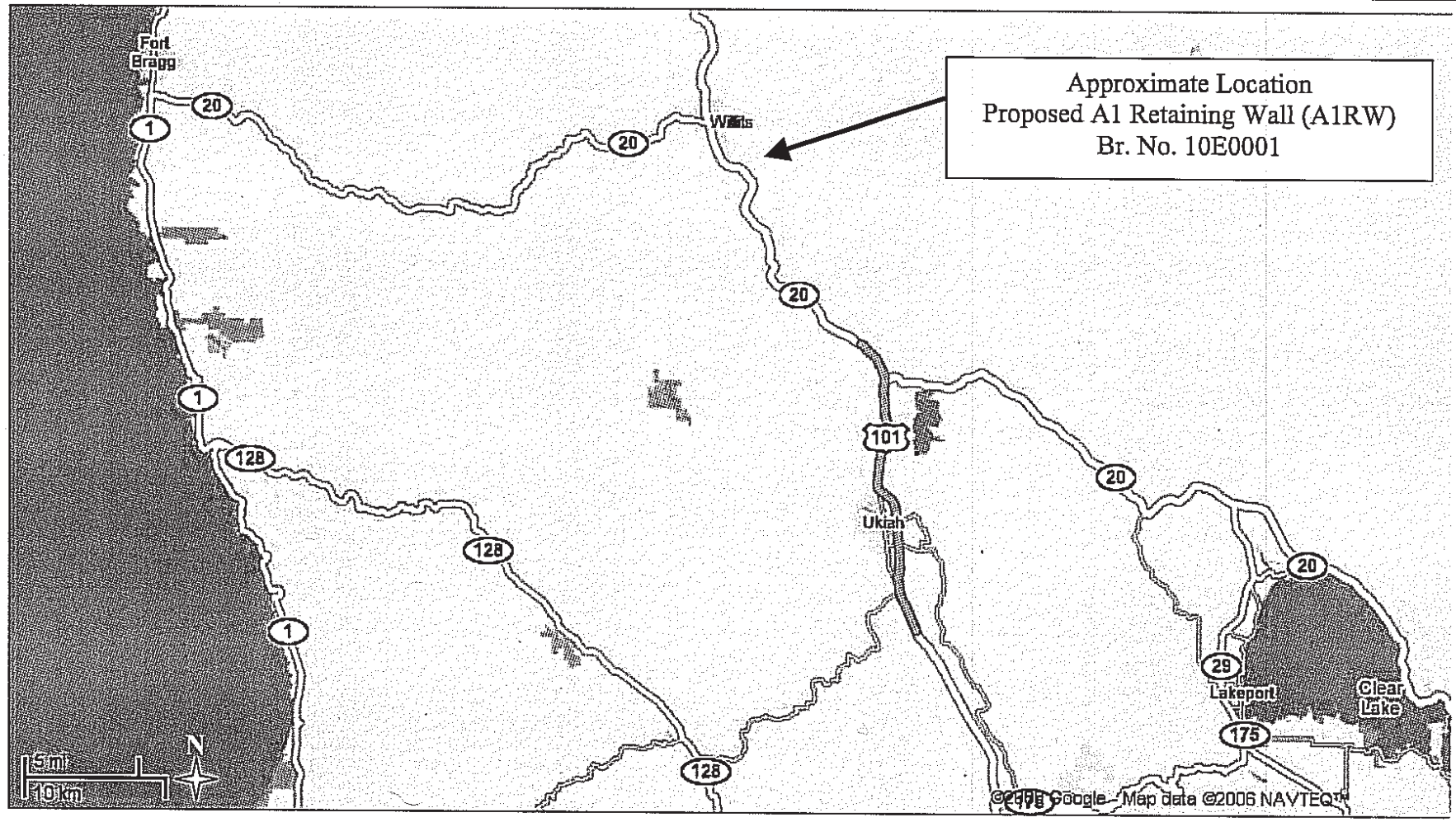
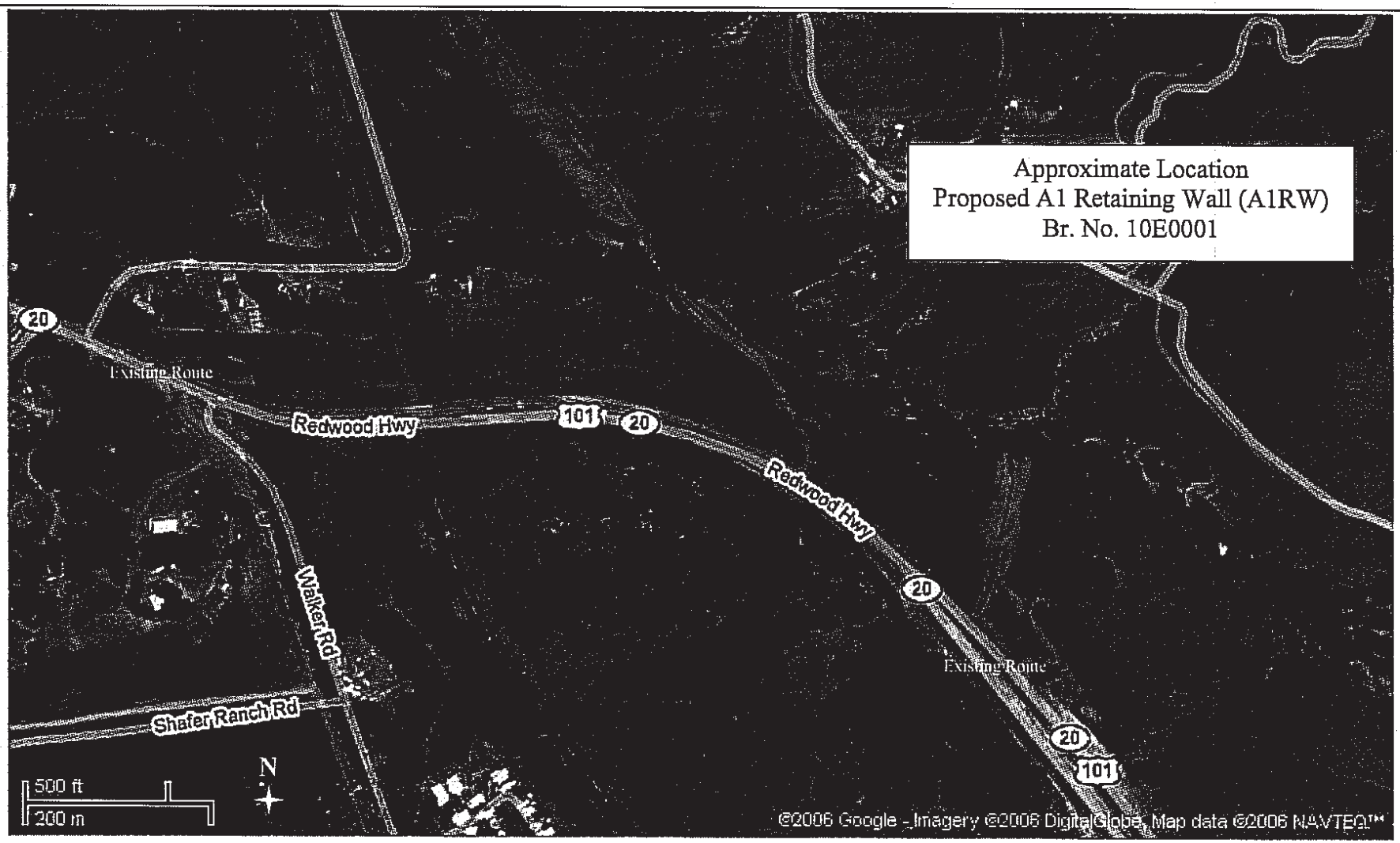
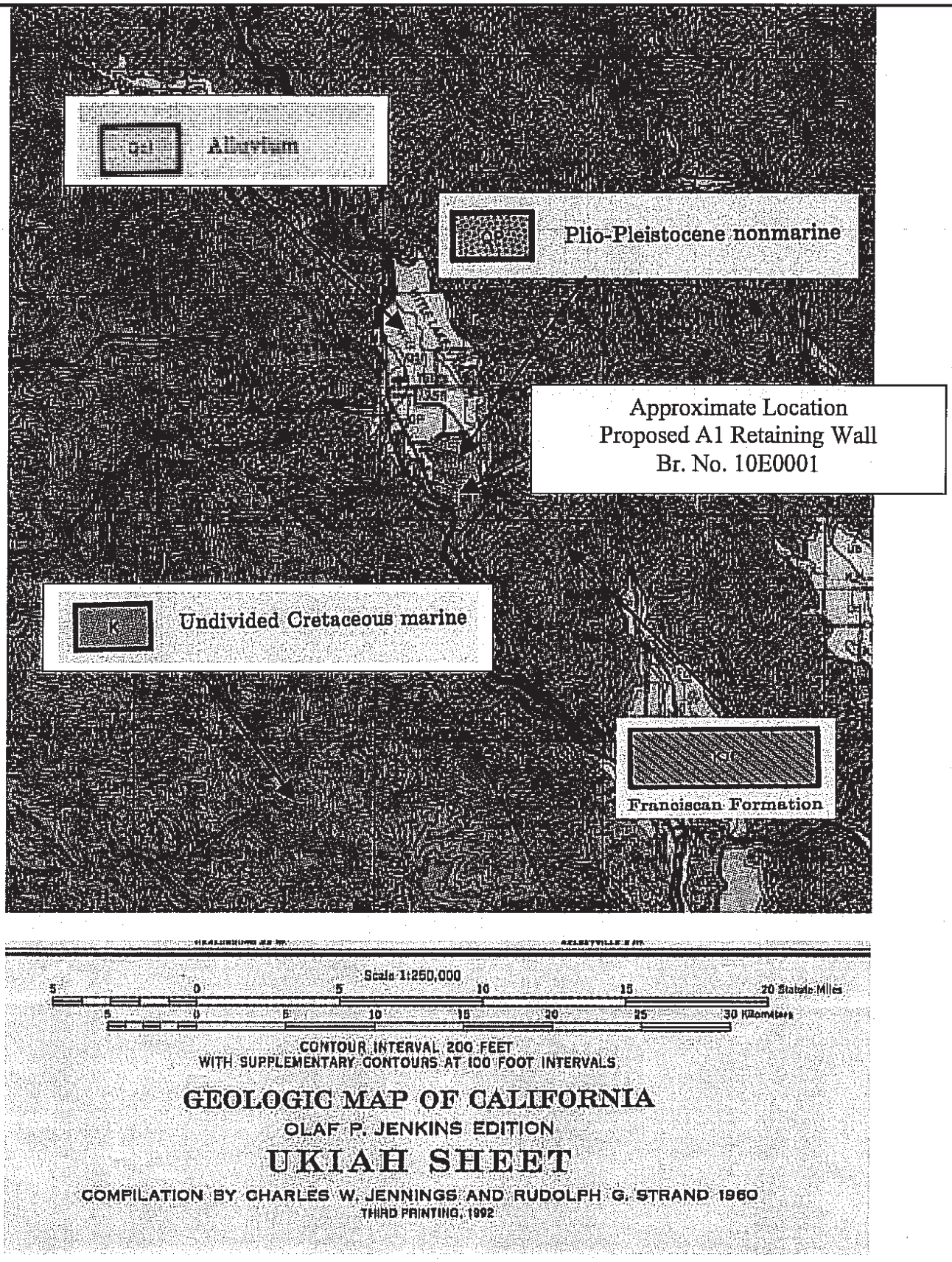


Caltrans Division of Engineering Services Geotechnical Services Geotechnical Design – North	EA: 01-262001 December 7, 2006	Location Map
	01-MEN-101 KP R70.46/PM R43.78 A1 Retaining Wall, Br. No. 10E0001	Figure 1



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 December 7, 2006	Location Map
	01-MEN-101 KP R70.46/PM R43.78 **A1 Retaining Wall, Br. No. 10E0001**	Figure 2



Caltrans Division of Engineering Services Geotechnical Services Geotechnical Design – North	EA: 01-262001 December 7, 2006	Geologic Map
	01-MEN-101 KP R70.46/PM R43.78 A1 Retaining Wall, Br. No. 10E0001	Figure 3

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Date: March 11, 2009

File: 01-MEN-101-KP R78.90/PM R49.03
EA#01-262001
UPP CREEK BRIDGE
Br. No. 10-0305

Attention: Mr. Marc Friedheim

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Upp Creek Bridge (Br. No. 10-0305). The Office of Geotechnical Design North, Branch A completed a subsurface investigation during October 2008 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (October 2008) along with a review of the available General Plan dated January 20, 2009 and the Foundation Plan dated January 17, 2009. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Upp Creek Bridge (Br. No. 10-0305) will consist of one new structure and will be incorporated as part of the Willits Bypass. The structure will consist of a single span prestressed cast in place concrete voided slab structure supported on driven steel piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-

sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the north end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Quaternary alluvium or the Plio-Pleistocene nonmarine. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation during October 2008.

The subsurface investigation consisted of two mud rotary borings (Nos. B-98 and B-99). The mud rotary borings were advanced using a self-casing wireline drilling method extending down to a maximum depth of 24.84 m (81.5 ft). The equipment used to drill borings B-98 and B-99 consisted of an Acker MPCA drill rig equipped with a Longyear automatic hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT). Selected soil samples were bagged for laboratory testing.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed bridge (Br. No. 10-0305) generally consist of interbedded medium dense to very dense clayey gravel, well graded gravel with clay/sand and very soft to hard clay, sandy clay and gravelly clay. The foundation materials extend from the ground surface to the maximum depth of the field investigation, 24.84 m (81.5 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels varied from elevation 404.37 m (1326.7 ft) in Boring B-99 to 404.99 m (1328.7 ft) in Boring B-98 during the October 2008 subsurface investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Corrosion Evaluation

Corrosion testing performed in Boring B-84 located approximately 100 meters east of this site. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for Upp Creek Bridge (Br. No. 10-0305)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639920	B-84	0-1.52	6.96	5214	N/A	N/A
C639921	B-84	1.52-3.05	6.87	5214	N/A	N/A

Laboratory Testing

No laboratory testing was performed from this subsurface investigation.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 0.5 kilometer southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Based on the boring logs a final Caltrans Seismic Design Criteria (CSDC) Acceleration Response Spectrum (ARS) Curve corresponding to soil profile Type D is recommended for design. Please note that due to the close proximity of this structure to the fault, we have performed a second modification to the CSDC ARS curve (see Figure 1). The modification is such that there is no increase in spectral accelerations (SA) for periods less than 0.5 seconds, and a 20% increase in SA for periods greater than 1 second. Between the periods of 0.5 and 1 second, a linear interpolation was used to estimate the SA.

Foundation Recommendations

The following foundation recommendations are for the new proposed Upp Creek Bridge (Br. No. 10-0305).

At Abutment 1 and 2 support locations, driven steel HP 250x85 "H" piles are recommended for support. The specified pile tip elevations (SPTE), shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed Upp Creek Bridge (Br. No. 10-0305) for Abutment 1 and 2 support locations.

Location	Pile Type	Nominal Resistance		Bottom of Pile Cap Elevation	Design Pile Tip Elevation	Specified Pile Tip Elevation
		Compression	Tension			
Abutment 1	HP 250x85	1250 kN	0	407.33 m (1336.4 ft.)	393.0 m (1,2) (1289.4 ft.)	393.0 m (1289.4 ft.)
Abutment 2	HP 250x85	1250 kN	0	407.33 m (1336.4 ft.)	393.0 m (1,2) (1289.4 ft.)	393.0 m (1289.4 ft.)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 406.88 m (1334.9 ft)*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Branch A shall be contacted for further recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. At the Engineer's option, any steel piles driven within 3.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035 or Reid Buell at (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard Special Provisions 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:

A. Log of Test Borings for Upp Creek Bridge, Bridge No. 10-0305.

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

A. Foundation Report for the Upp Creek Bridge, Bridge No. 10-0305, dated March 11, 2009.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North

Report by:

REZA MAHALLATI
Senior Materials & Research Engineer
Office of Geotechnical Design-North

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Senior Engineering Geologist
Office of Geotechnical Design-North

cc: OGDSN
GS File Room
Reid Buell
R.E. Pending
Structure OE

REFERENCES

Federal Highway Administration (FHWA), Design and Construction of Driven Pile Foundations: Workshop Manual – Volume 1 (Pub. No. FHWA HI-97-013) published January 1997, Revised November1998.

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Norris, Robert M., and Webb, Robert M., 1990, Geology of California, Second Edition, John Wiley & Sons, Inc., pp 57, 359, 363.

Seismic Design Criteria (2006), California Department of Transportation, Version 1.4.

State of California, Department of Transportation (Caltrans):

- Standard Plans, July 2004 and May 2006.
- Standard Specifications, July 2004 and May 2006.
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

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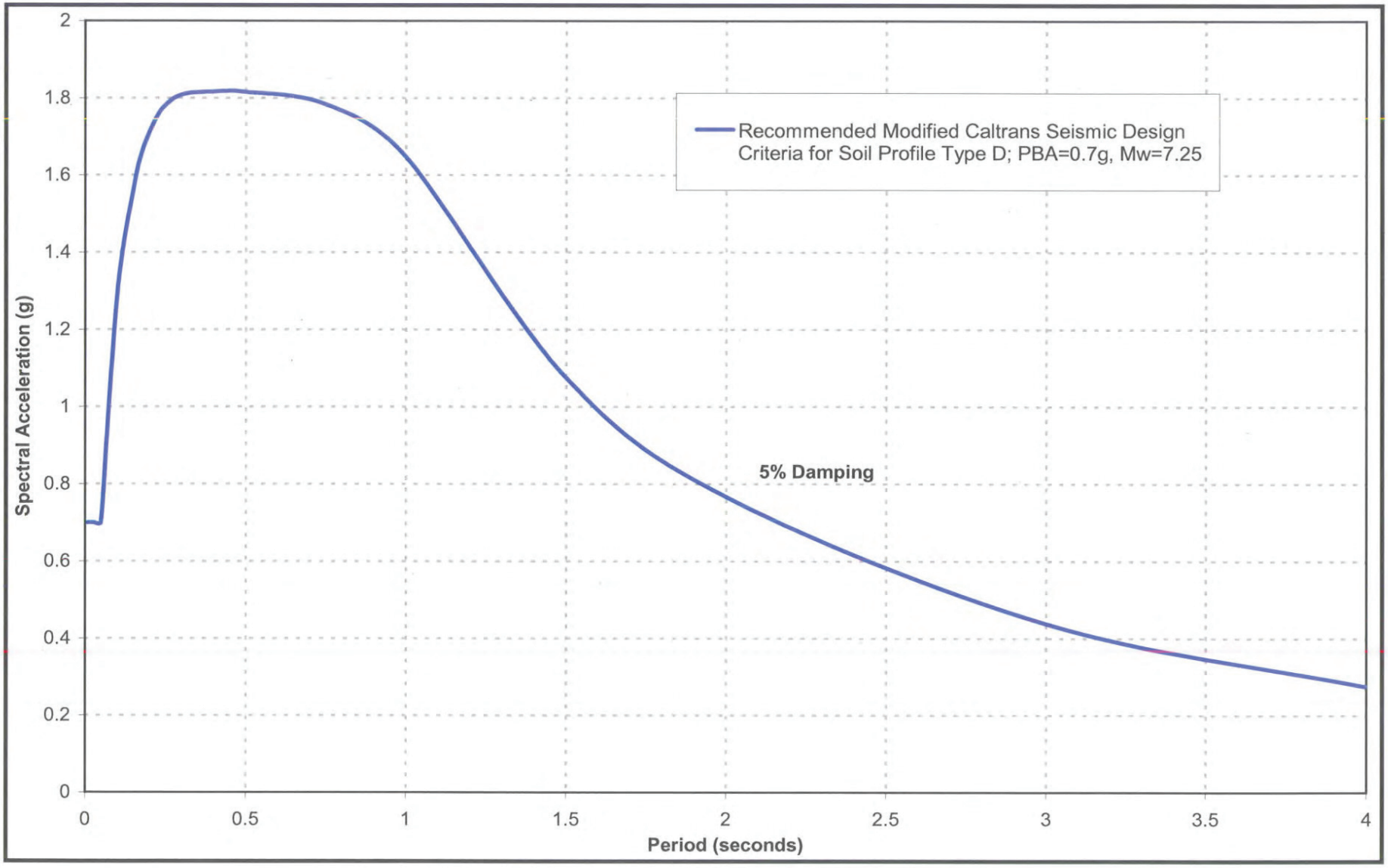

Figure 1. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
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To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Date: March 11, 2009

File: 01-MEN-101-KP R77.701/PM R48.28
EA#01-262001
UPP CREEK BRIDGE
Br. No. 10-0174

Attention: Mr. Marc Friedheim

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Upp Creek Bridge (Br. No. 10-0174). The Office of Geotechnical Design North, Branch A completed a subsurface investigation from September 2007 to October 2008 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (September 2007 to October 2008) along with a review of the available General Plan dated October 21, 2008 and the Foundation Plan dated January 31, 2009. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Upp Creek Bridge (Br. No. 10-0174) will consist of one new structure and will be incorporated as part of the Willits Bypass. The structure will consist of a single span prestressed cast in place concrete box girder (4 cell) structure with open end seat abutments with wingwalls supported on driven steel piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains

dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the north end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Quaternary alluvium. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation from September 2007 to October 2008.

The subsurface investigation consisted of four mud rotary borings (Nos. B-82, B-87, B-97 and B-100). The mud rotary borings were advanced using a self-casing wireline drilling method extending down to a maximum depth of 34.0 m (111.5 ft). The equipment used to drill borings B-82, B-87, B-97 and B-100 consisted of an Acker MPCA drill rig equipped with a Longyear

automatic hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT). Selected soil samples were bagged for laboratory testing.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed bridge (Br. No. 10-0174) generally consist of interbedded very soft to hard clay, loose to dense clayey sand, clayey gravel and gravel with clay. The foundation materials extend from the ground surface to the maximum depth of the field investigation, 34.0 m (111.5 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels varied from elevation 403.62 m (1324.2 ft) in Boring B-82 to 404.68 m (1327.7 ft) in Boring B-87 during the September 2007 to October 2008 subsurface investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Corrosion Evaluation

Composite soil samples were collected in one of the borings during the September 2007 to October 2008 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for Upp Creek Bridge (Br. No. 10-0174)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639917	B-87	1.52-3.05	6.39	3800	N/A	N/A
C639918	B-87	4.57-5.18	8.07	5988	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Moisture Content, Specific Gravity, Consolidation, Triaxial (unconsolidated-undrained), and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request once the results are received.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 0.5 kilometer southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Based on the boring logs a final Caltrans Seismic Design Criteria (CSDC) Acceleration Response Spectrum (ARS) Curve corresponding to soil profile Type D is recommended for design. Please note that due to the close proximity of this structure to the fault, we have performed a second modification to the CSDC ARS curve (see Figure 1). The modification is such that there is no increase in spectral accelerations (SA) for periods less than 0.5 seconds, and a 20% increase in SA for periods greater than 1 second. Between the periods of 0.5 and 1 second, a linear interpolation was used to estimate the SA.

Fill Settlement

Due to the presence of compressible clay soils beneath the proposed approach fills at Abutment 1 and Abutment 2, a 270 day waiting period is to be required after completion of the approach fills and before installing the steel pipe piles for the abutments. Settlement will take place during and after the construction of the approach fills. It has been estimated that the maximum total settlement of the approach fills will be about 600 millimeters.

It has been calculated that 90% of the settlement of the approach fills will take place approximately 270 days after placement of the fill is completed. If it is necessary to accelerate the settlement, mitigation measures such as placement of additional surcharge fill and/or the use of wick drains may be recommended. Details of these measures will be included in the Geotechnical Design Report to be prepared by this Office.

Regardless of the target settlement or the estimated settlement time period, 600 mm and 270 days, respectively, the rate of settlement obtained during the settlement monitoring will determine if the majority of the settlement in the fill is complete. That is, if the settlement rate is noticeably reduced sooner than the end of the estimated settlement time period, then it would be acceptable for construction of the abutment foundations to commence. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

Foundation Recommendations

The following foundation recommendations are for the new proposed Upp Creek Bridge (Br. No. 10-0174).

At Abutment 1 and 2 support locations, driven steel HP 360x132 "H" piles are recommended for support. The specified pile tip elevations (SPTE), shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed Upp Creek Bridge (Br. No. 10-0174) for Abutment 1 and 2 support locations.

Location	Pile Type	Nominal Resistance		Bottom of Pile Cap Elevation	Design Pile Tip Elevation	Specified Pile Tip Elevation
		Compression	Tension			
Abutment 1	HP 360x132	1500 kN	0	410.2 m (1345.8 ft.)	379.5 m (1,2) (1245.1 ft.)	379.5 m (1245.1 ft.)
Abutment 2	HP 360x132	1500 kN	0	409.7 m (1344.2ft.)	379.5 m (1,2) (1245.1 ft.)	379.5 m (1245.1 ft.)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 404.80 m (1328.1 ft)*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Branch A shall be contacted for further recommendations.

3. Specified pile tip elevations shown in Table 2 above are based on preloading of compressible clay soils achieved by placement of approach fills prior to construction of the proposed abutment foundations. If preloading is not performed, the Office of Geotechnical Design North is to be contacted for revised foundation recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. Settlement shall be monitored. Monitoring devices shall consist of settlement platforms placed at the bottom of the approach fill embankments at Abutments 1 and 2. A minimum of six settlement devises (three or more beneath each approach fill) shall be placed beneath the embankments at Abutments 1 and 2 at locations to be determined by the Engineer.

3. Due to presence of compressible clay soils beneath the proposed abutments, the abutment locations shall be preloaded. Preloading shall be achieved by placement of approach fill embankments prior to construction of the Upp Creek Bridge. A 270-day waiting period is required prior to installation of piles at Abutments 1 and 2. The fill settlement shall be monitored during the waiting period. If it is determined that the settlement has ceased prior to the end of the 270-day waiting period, with the written approval of the Engineer, the waiting period may be terminated at that time. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

4. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

5. At Abutment 1 and 2 locations, driven steel HP 360X132 "H" piles shall be driven through the new fill embankment in oversized pre-drilled holes that conform to the Standard Specifications in Section 49-1.06, "Pre-drilled Holes." The predrilled holes are not to extend below the elevations shown in Table 3.

Table 3. Predrilled Elevations

Location	Predrilled Elevation
Abutment 1	406.0 m (1332.0 ft)
Abutment 2	406.0 m (1332.0 ft)

The recommendations contained in this report are based on specific project information regarding design loads and structure locations provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035 or Reid Buell at (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard Special Provisions 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:

A. Log of Test Borings for Upp Creek Bridge, Bridge No. 10-0174.

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

A. Foundation Report for the Upp Creek Bridge, Bridge No. 10-0174, dated March 11, 2009.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North

Report by:

REZA MAHALLATI
Senior Materials & Research Engineer
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REFERENCES

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Norris, Robert M., and Webb, Robert M., 1990, Geology of California, Second Edition, John Wiley & Sons, Inc., pp 57, 359, 363.

Seismic Design Criteria (2006), California Department of Transportation, Version 1.4.

State of California, Department of Transportation (Caltrans):

- Standard Plans, July 2004 and May 2006.
- Standard Specifications, July 2004 and May 2006.
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.

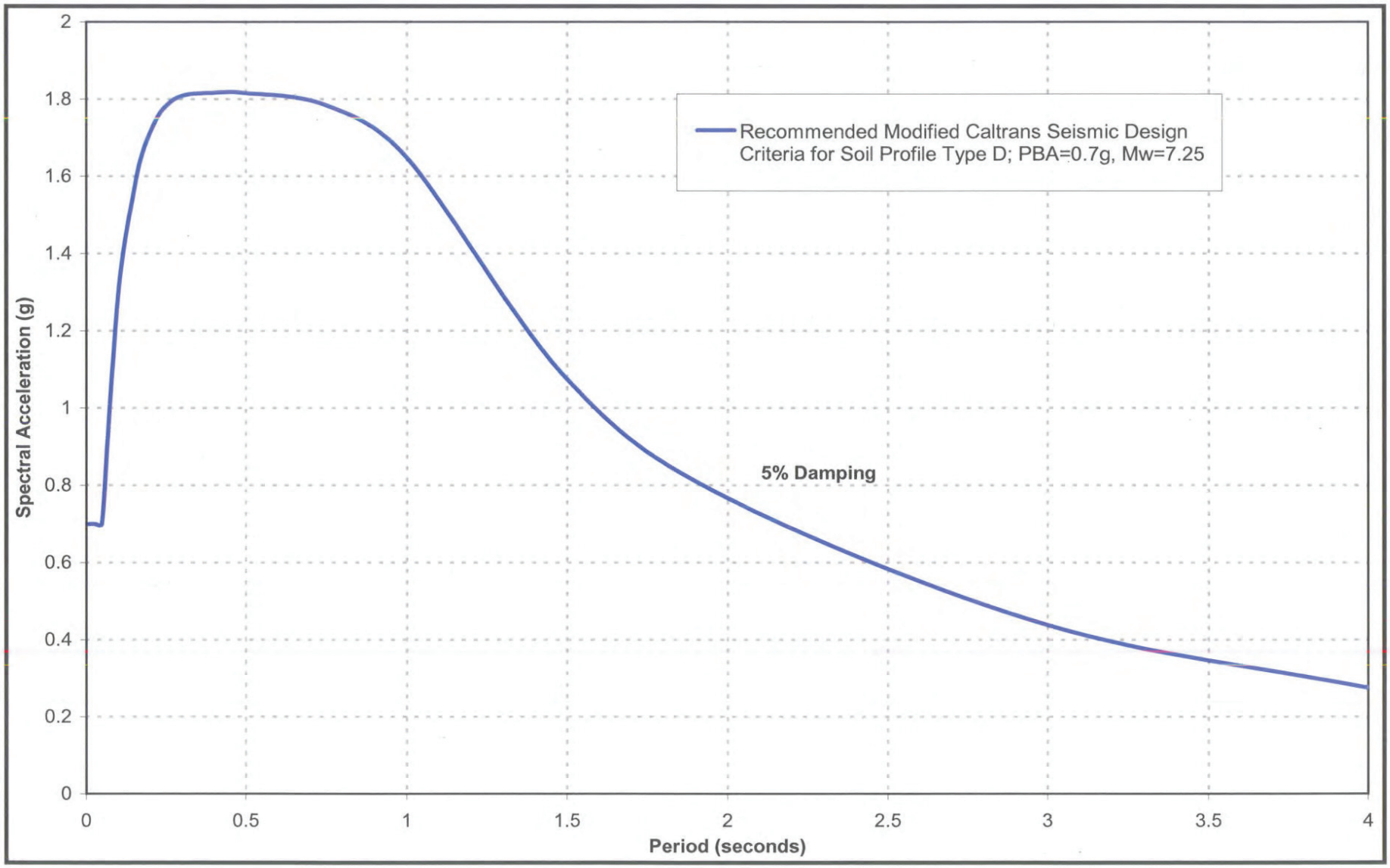

Figure 1. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Date: March 11, 2009

File: 01-MEN-101-KP R77.68/PM R48.27
EA#01-262001
UPP CREEK (SB Off-Ramp)
Br. No. 10-0174K

Attention: Mr. Marc Friedheim

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Upp Creek (SB Off-Ramp) (Br. No. 10-0174K). The Office of Geotechnical Design North, Branch A completed a subsurface investigation from October 2007 to October 2008 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (October 2007 to October 2008) along with a review of the available General Plan dated August 27, 2008 and the Foundation Plan dated January 26, 2009. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Upp Creek (SB Off-Ramp) (Br. No. 10-0174K) will consist of one new structure and will be incorporated as part of the Willits Bypass. The structure will consist of a single span prestressed cast in place concrete box girder (3 cell) structure with open end seat abutments with wingwalls supported on driven steel piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains

dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the north end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Quaternary alluvium. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation from October 2007 to October 2008.

The subsurface investigation consisted of three mud rotary borings (Nos. B-84, B-97 and B-100). The mud rotary borings were advanced using a self-casing wireline drilling method extending down to a maximum depth of 34.0 m (111.5 ft). The equipment used to drill borings B-84, B-97 and B-100 consisted of an Acker MPCA drill rig equipped with a Longyear automatic hammer.

Sampling was achieved by utilizing the Standard Penetration Test (SPT). Selected soil samples were bagged for laboratory testing.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed bridge (Br. No. 10-0174K) generally consist of interbedded very soft to hard clay, loose to dense clayey sand, clayey gravel and gravel with clay. The foundation materials extend from the ground surface to the maximum depth of the field investigation, 34.0 m (111.5 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels varied from elevation 403.46 m (1323.7 ft) in Boring B-84 to 404.68 m (1327.7 ft) in Boring B-100 during the October 2007 to October 2008 subsurface investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Corrosion Evaluation

Composite soil samples were collected in one of the borings during the October 2007 to October 2008 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for Upp Creek (SB Off-Ramp) (Br. No. 10-0174K)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639920	B-84	0-1.52	6.96	5214	N/A	N/A
C639921	B-84	1.52-3.05	6.87	5214	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Moisture Content, Specific Gravity, Consolidation, Triaxial (unconsolidated-undrained), and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request once the results are received.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 0.5 kilometer southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Based on the boring logs a final Caltrans Seismic Design Criteria (CSDC) Acceleration Response Spectrum (ARS) Curve corresponding to soil profile Type D is recommended for design. Please note that due to the close proximity of this structure to the fault, we have performed a second modification to the CSDC ARS curve (see Figure 1). The modification is such that there is no increase in spectral accelerations (SA) for periods less than 0.5 seconds, and a 20% increase in SA for periods greater than 1 second. Between the periods of 0.5 and 1 second, a linear interpolation was used to estimate the SA.

Fill Settlement

Due to the presence of compressible clay soils beneath the proposed approach fills at Abutment 1 and Abutment 2, a 270 day waiting period is to be required after completion of the approach fills and before installing the steel pipe piles for the abutments. Settlement will take place during and after the construction of the approach fills. It has been estimated that the maximum total settlement of the approach fills will be about 600 millimeters.

It has been calculated that 90% if the settlement of the approach fills will take place approximately 270 days after placement of the fill is completed. If it is necessary to accelerate the settlement, mitigation measures such as placement of additional surcharge fill and/or the use of wick drains may be recommended. Details of these measures will be included in the Geotechnical Design Report to be prepared by this Office.

Regardless of the target settlement or the estimated settlement time period, 600 mm and 270 days, respectively, the rate of settlement obtained during the settlement monitoring will determine if the majority of the settlement in the fill is complete. That is, if the settlement rate is noticeably reduced sooner than the end of the estimated settlement time period, then it would be acceptable for construction of the abutment foundations to commence. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

Foundation Recommendations

The following foundation recommendations are for the new proposed Upp Creek (SB Off-Ramp) (Br. No. 10-0174K).

At Abutment 1 and 2 support locations, driven steel HP 250x85 "H" piles are recommended for support. The specified pile tip elevations (SPTE), shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed Upp Creek (SB Off-Ramp) (Br. No. 10-0174K) for Abutment 1 and 2 support locations.

Location	Pile Type	Nominal Resistance		Bottom of Pile Cap Elevation	Design Pile Tip Elevation	Specified Pile Tip Elevation
		Compression	Tension			
Abutment 1	HP 250x85	1200 kN	0	406.9 m (1335.0 ft.)	376.5 m (1,2) (1235.2 ft.)	376.5 m (1235.2 ft.)
Abutment 2	HP 250x85	1200 kN	0	407.3 m (1336.3 ft.)	376.5 m (1,2) (1235.2 ft.)	376.5 m (1235.2 ft.)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 405.17 m (1329.3 ft)*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Branch A shall be contacted for further recommendations.

3. Specified pile tip elevations shown in Table 2 above are based on preloading of compressible clay soils achieved by placement of approach fills prior to construction of the proposed abutment foundations. If preloading is not performed, the Office of Geotechnical Design North is to be contacted for revised foundation recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. Settlement shall be monitored. Monitoring devices shall consist of settlement platforms placed at the bottom of the approach fill embankments at Abutments 1 and 2. A minimum of six settlement devises (three or more beneath each approach fill) shall be placed beneath the embankments at Abutments 1 and 2 at locations to be determined by the Engineer.

3. Due to presence of compressible clay soils beneath the proposed abutments, the abutment locations shall be preloaded. Preloading shall be achieved by placement of approach fill embankments prior to construction of the Upp Creek (SB Off-Ramp). A 270-day waiting period is required prior to installation of piles at Abutments 1 and 2. The fill settlement shall be monitored during the waiting period. If it is determined that the settlement has ceased prior to the end of the 270-day waiting period, with the written approval of the Engineer, the waiting period may be terminated at that time. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

4. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035 or Reid Buell at (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard Special Provisions 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:

A. Log of Test Borings for Upp Creek (SB Off-Ramp), Bridge No. 10-0174K.

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

A. Foundation Report for the Upp Creek (SB Off-Ramp), Bridge No. 10-0174K, dated March 11, 2009.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North

Report by:

REZA MAHALLATI
Senior Materials & Research Engineer
Office of Geotechnical Design-North

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cc: OGDSN
GS File Room
Reid Buell
R.E. Pending
Structure OE

REFERENCES

Federal Highway Administration (FHWA), Design and Construction of Driven Pile Foundations: Workshop Manual – Volume 1 (Pub. No. FHWA HI-97-013) published January 1997, Revised November1998.

Harden, Deborah R., 1998, California Geology, Prentice Hall, 252 pp.

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State of California, Department of Transportation (Caltrans):

- Standard Plans, July 2004 and May 2006.
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- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
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Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.

Upp Creek (SB Off-Ramp)
Br. No. 10-0174K
01-262001

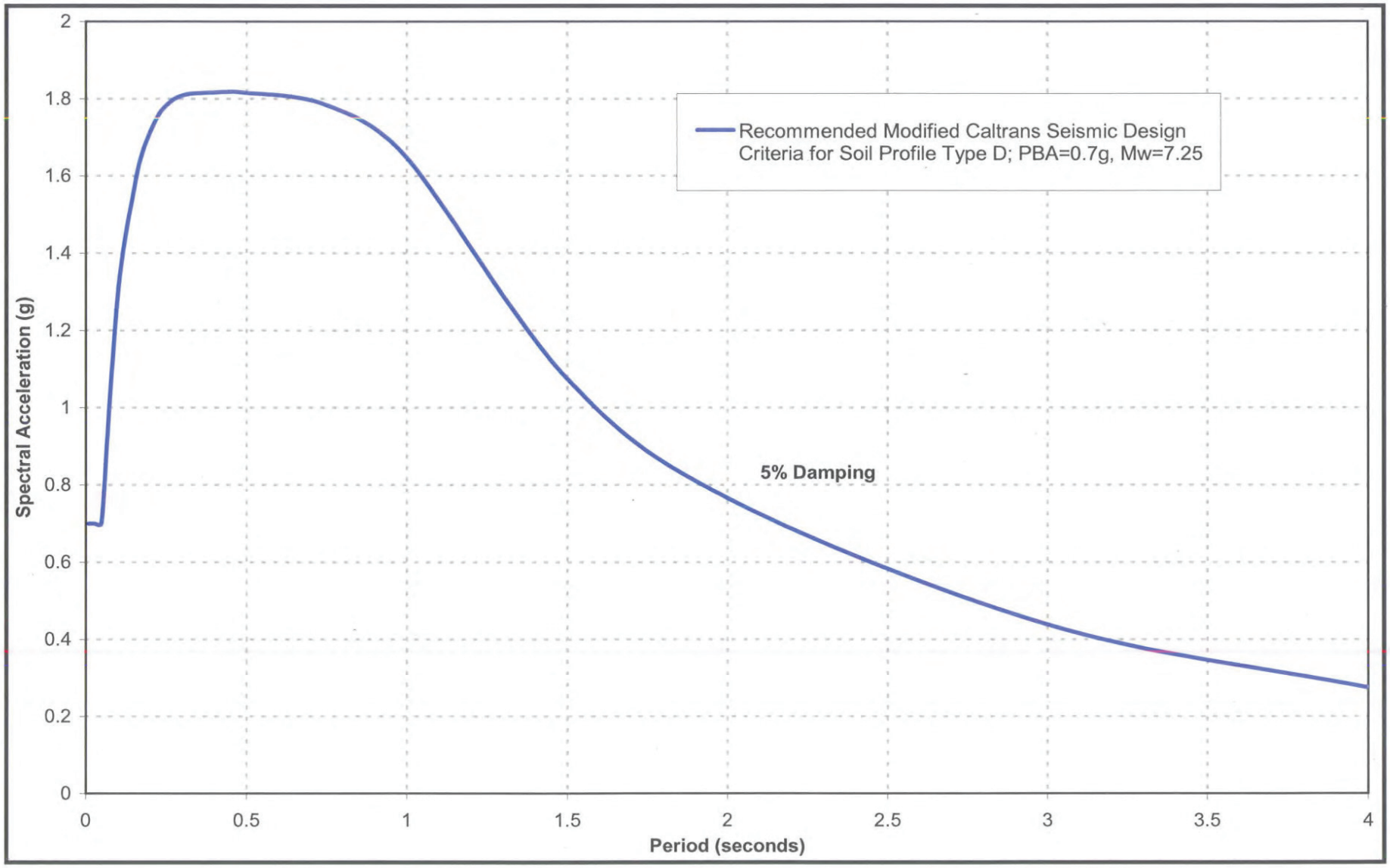

Figure 1. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Date: March 11, 2009

File: 01-MEN-101-KP R77.701/PM R48.28
EA#01-262001
UPP CREEK (NB ON-RAMP)
Br. No. 10-0174S

Attention: Mr. Marc Friedheim

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Upp Creek (NB On-Ramp) (Br. No. 10-0174S). The Office of Geotechnical Design North, Branch A completed a subsurface investigation from September 2007 to October 2007 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (September 2007 to October 2007) along with a review of the available General Plan dated August 29, 2008 and the Foundation Plan dated January 12, 2009. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Upp Creek (NB On Ramp) (Br. No. 10-0174S) will consist of one new structure and will be incorporated as part of the Willits Bypass. The structure will consist of a single span prestressed cast in place concrete box girder (3 cell) structure with open end seat abutments with wingwalls supported on driven steel piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains

dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the north end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Quaternary alluvium. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation from September 2007 to October 2007.

The subsurface investigation consisted of two mud rotary borings (Nos. B-82 and B-87). The mud rotary borings were advanced using a self-casing wireline drilling method extending down to a maximum depth of 28.2 m (92.5 ft). The equipment used to drill borings B-82 and B-87 consisted of an Acker MPCA drill rig equipped with a Longyear automatic hammer. Sampling

was achieved by utilizing the Standard Penetration Test (SPT). Selected soil samples were bagged for laboratory testing.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed bridge (Br. No. 10-0174S) generally consist of interbedded very soft to very stiff clay, loose to dense clayey sand, clayey gravel and gravel with clay. The foundation materials extend from the ground surface to the maximum depth of the field investigation, 28.2 m (92.5 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels varied from elevation 403.62 m (1324.2 ft) in Boring B-82 to 404.7 m (1327.68 ft) in Boring B-87 during the September 2007 to October 2007 subsurface investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Corrosion Evaluation

Composite soil samples were collected in one of the borings during the September 2007 to October 2007 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for Upp Creek (NB On-Ramp) (Br. No. 10-0174S)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639917	B-87	1.52-3.05	6.39	3800	N/A	N/A
C639918	B-87	4.57-5.18	8.07	5988	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Moisture Content, Specific Gravity, Consolidation, Triaxial (unconsolidated-undrained), and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request once the results are received.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 0.5 kilometer southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Based on the boring logs a final Caltrans Seismic Design Criteria (CSDC) Acceleration Response Spectrum (ARS) Curve corresponding to soil profile Type D is recommended for design. Please note that due to the close proximity of this structure to the fault, we have performed a second modification to the CSDC ARS curve (see Figure 1). The modification is such that there is no increase in spectral accelerations (SA) for periods less than 0.5 seconds, and a 20% increase in SA for periods greater than 1 second. Between the periods of 0.5 and 1 second, a linear interpolation was used to estimate the SA.

Fill Settlement

Due to the presence of compressible clay soils beneath the proposed approach fills at Abutment 1 and Abutment 2, a 270 day waiting period is to be required after completion of the approach fills and before installing the steel pipe piles for the abutments. Settlement will take place during and after the construction of the approach fills. It has been estimated that the maximum total settlement of the approach fills will be about 600 millimeters.

It has been calculated that 90% if the settlement of the approach fills will take place approximately 270 days after placement of the fill is completed. If it is necessary to accelerate the settlement, mitigation measures such as placement of additional surcharge fill and/or the use of wick drains may be recommended. Details of these measures will be included in the Geotechnical Design Report to be prepared by this Office.

Regardless of the target settlement or the estimated settlement time period, 600 mm and 270 days days, respectively, the rate of settlement obtained during the settlement monitoring will determine if the majority of the settlement in the fill is complete. That is, if the settlement rate is noticeably reduced sooner than the end of the estimated settlement time period, then it would be acceptable for construction of the abutment foundations to commence. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

Foundation Recommendations

The following foundation recommendations are for the new proposed Upp Creek (NB On-Ramp) (Br. No. 10-0174S).

At Abutment 1 and 2 support locations, driven steel HP 250x62 "H" piles are recommended for support. The specified pile tip elevations (SPTE), shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed Upp Creek (NB On-Ramp) (Br. No. 10-0174S) for Abutment 1 and 2 support locations.

Location	Pile Type	Nominal Resistance		Bottom of Pile Cap Elevation	Design Pile Tip Elevation	Specified Pile Tip Elevation
		Compression	Tension			
Abutment 1	HP 250x62	800 kN	0	407.5 m (1336.9 ft.)	383.0 m (1,2) (1256.6 ft.)	383.0 m (1256.6 ft.)
Abutment 2	HP 250x62	800 kN	0	407.9 m (1338.3 ft.)	383.0 m (1,2) (1256.6 ft.)	383.0 m (1256.6 ft.)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 404.80 m (1328.1 ft)*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Branch A shall be contacted for further recommendations.

3. Specified pile tip elevations shown in Table 2 above are based on preloading of compressible clay soils achieved by placement of approach fills prior to construction of the proposed abutment foundations. If preloading is not performed, the Office of Geotechnical Design North is to be contacted for revised foundation recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. Settlement shall be monitored. Monitoring devices shall consist of settlement platforms placed at the bottom of the approach fill embankments at Abutments 1 and 2. A minimum of six settlement devises (three or more beneath each approach fill) shall be placed beneath the embankments at Abutments 1 and 2 at locations to be determined by the Engineer.

3. Due to presence of compressible clay soils beneath the proposed abutments, the abutment locations shall be preloaded. Preloading shall be achieved by placement of approach fill embankments prior to construction of the Upp Creek (NB On-Ramp). A 270-day waiting period is required prior to installation of piles at Abutments 1 and 2. The fill settlement shall be monitored during the waiting period. If it is determined that the settlement has ceased prior to the end of the 270-day waiting period, with the written approval of the Engineer, the waiting period may be terminated at that time. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

4. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

5. At Abutment 1 and 2 locations, driven steel HP 250X62 "H" piles shall be driven through the new fill embankment in oversized pre-drilled holes that conform to the Standard Specifications in Section 49-1.06, "Pre-drilled Holes." The predrilled holes are not to extend below the elevations shown in Table 3.

Table 3. Predrilled Elevations

Location	Predrilled Elevation
Abutment 1	406.0 m (1332.0 ft)
Abutment 2	406.0 m (1332.0 ft)

The recommendations contained in this report are based on specific project information regarding design loads and structure locations provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035 or Reid Buell at (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard Special Provisions 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:

A. Log of Test Borings for Upp Creek (NB On-Ramp), Bridge No. 10-0174S.

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

A. Foundation Report for the Upp Creek (NB On-Ramp), Bridge No. 10-0174S, dated March 11, 2009.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North

Report by:

REZA MAHALLATI
Senior Materials & Research Engineer
Office of Geotechnical Design-North

Supervised by:

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Reid Buell
R.E. Pending
Structure OE

REFERENCES

Federal Highway Administration (FHWA), Design and Construction of Driven Pile Foundations: Workshop Manual – Volume 1 (Pub. No. FHWA HI-97-013) published January 1997, Revised November1998.

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Norris, Robert M., and Webb, Robert M., 1990, Geology of California, Second Edition, John Wiley & Sons, Inc., pp 57, 359, 363.

Seismic Design Criteria (2006), California Department of Transportation, Version 1.4.

State of California, Department of Transportation (Caltrans):

- Standard Plans, July 2004 and May 2006.
- Standard Specifications, July 2004 and May 2006.
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.

Upp Creek (NB On-Ramp)
Br. No. 10-0174S
01-262001

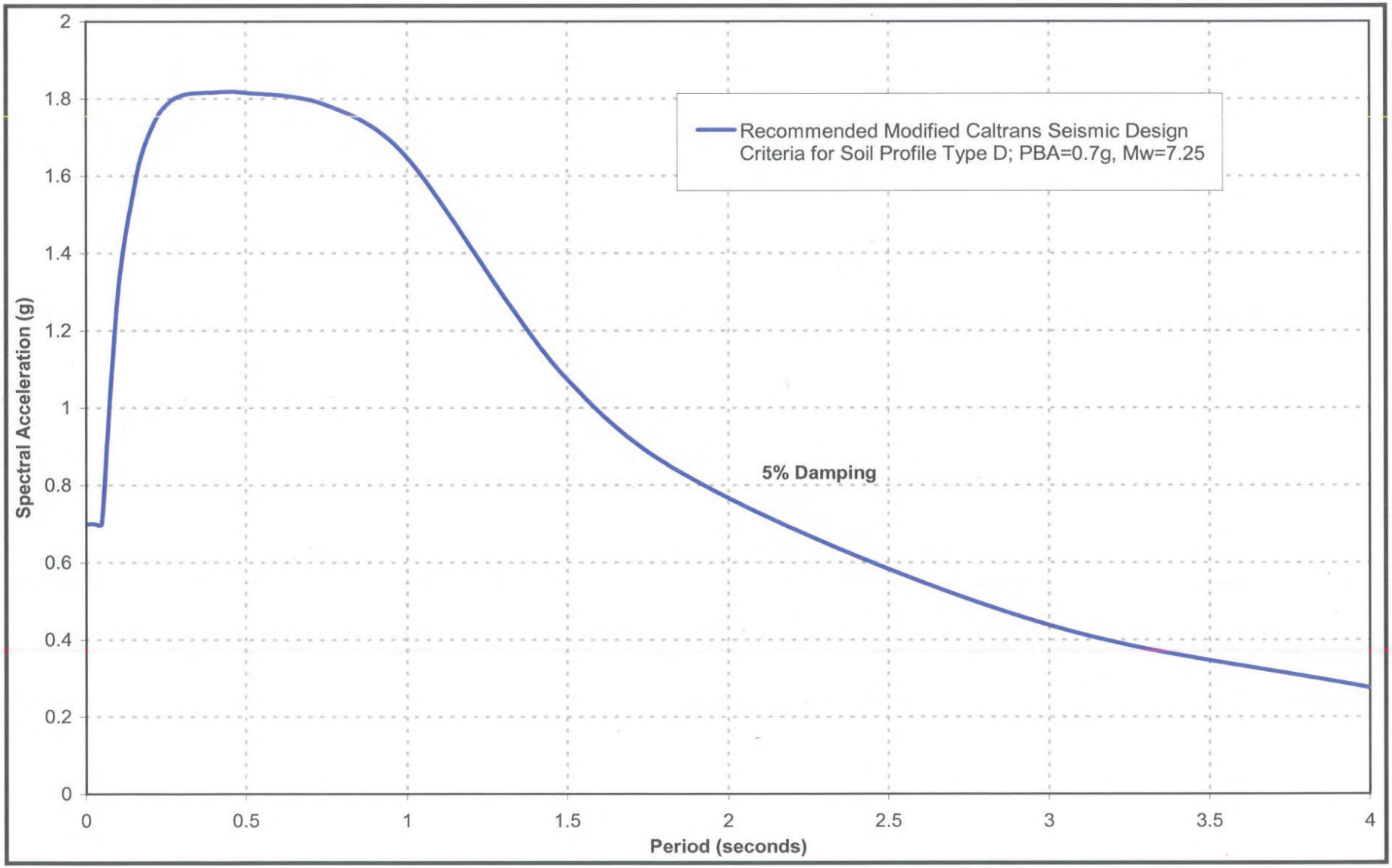

Figure 1. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: December 8, 2006

File: 01-MEN-101-KP R70.53/PM R 43.83
EA#01-262001
U4 Retaining Wall (NEW)
Br. No. 10E0002

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed U4 Retaining Wall (Br. No. 10E0002). The Office of Geotechnical Design North, Branch A completed a subsurface investigation during April 2004 to July 2004 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (April 2004 to July 2004) along with a review of the available General Plan dated February 28, 2006 and Foundation Plan dated February 28, 2006. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new U4 Retaining Wall (Br. No. 10E0002) will be incorporated as part of the Willits Bypass Project located within the city limits of Willits in Mendocino County (Figures 1 and 2). The proposed new retaining wall will be located at the west shoulder of the new proposed northbound 101 Connector, approximately 100 meters north of the new proposed E20-N101 Connector Bridge (Br. No. 10-0129G). In this report, the Stations are referenced from the "U4RW" LOL and the Offsets are referenced from the "U4" Line shown on the General and Foundation Plans dated February 28, 2006. The proposed U4 Retaining Wall (U4RW) will be supported on driven steel H-Piles.
Table 1 summarizes the beginning and ending stations of the proposed retaining wall with corresponding heights.

Table 1. Proposed U4 Retaining Wall (Br. No. 10E0002).

Retaining Wall Number	Station "A1RW" LOL		Offset from "A" Line		Length (m)	Height (mm)
	Begin (m)	End (m)	Begin (m)	End (m)		
U4RW	107+00.00	107+50.00	1.69	1.69	50.00	3000-4200

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the south end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary deposits) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits

bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Plio-Pleistocene nonmarine sedimentary deposits identified as the Cache Formation. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation in April 2004 to July 2004.

The subsurface investigation consisted of one mud rotary boring (No. B-10) and three cone penetrometer soundings (No. C-9, C-10, and C-11). The mud rotary boring was advanced using a self-casing wireline-diamond coring method extending down to a maximum depth of 24.8 m (81.5 ft). The equipment used to drill boring B-10 consisted of a Mobile B-47 drill rig equipped with a safety hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler. Selected soil samples were bagged for laboratory testing. The cone penetromer soundings were used to supplement the subsurface investigation.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed retaining wall (Br. No. 10E0002) generally consist of approximately 22.6 m (74.3 ft) of very stiff, firm to hard clay, sandy clay and silty clay with gravel overlying dense to very dense clayey sand and poorly graded sand. The foundation materials were encountered down to the maximum depth explored of 24.8 m (81.5 ft), an elevation of 422.9 m (1387.5 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

The ground water level was measured at an approximate elevation 443.8 m (1456.0 ft) in Boring B-10 during the April 2004 to July 2004 subsurface investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Scour Evaluation

A Final Hydraulic Report (dated March 22, 2005) for the Haehl Creek structures was completed by the Office of Structure Maintenance and Investigations, Structure Hydraulics Branch. The report provided information for the Haehl Creek Bridge, right bridge (Br. No. 10-0129R) that is located near the location of the proposed U4RW. This information was used to evaluate the scour potential of the proposed U4RW due to the proximity of this nearby structure to the proposed retaining wall location.

According to this report, there are no local scour concerns for the nearby Haehl Creek Bridge, right bridge (Br. No. 10-0129R) since it is a single span structure. The report recommends the abutment foundations be designed assuming no ground support (lateral and vertical) as a result of soil loss due to possible degradation or lateral stream migration down to the current stream thalwag elevation (lowest elevation in channel). The approximate thalwag elevation provided for the proposed Haehl Creek Bridge, right bridge (Br. No. 10-0129R) is 442.5 m (1451.8 ft).

It is recommended that Rock Slope Protection (RSP) be designed by the District to protect against the migration of the Haehl Creek towards the retaining wall location.

For further information including site-specific scour assessment and mitigation measures, the Structures Hydraulics Branch should be contacted.

Corrosion Evaluation

Composite soil samples were collected during the subsurface investigation. The results will be forwarded once completed.

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request once the results are received.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 1.0 kilometers (0.6 miles) southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Foundation Recommendations

The following foundation recommendations are for the new proposed U4 Retaining Wall (Br. No. 10E0002). At this location, it is recommended driven steel HP 250X85 "H" piles be used to support the proposed Type 1 Retaining Wall. The specified pile tip elevations, shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed U4RW, Type 1 Retaining Wall (Br. No. 10E0002).

Retaining Wall Station along "U4RW" LOL (Approximate)	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Approximate Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
107+00.00 to 107+04.88	3000	HP 250X85	400	800	0	447.2-447.1 (1467.2-1466.9)	429.5 (1,2) (1409.0)	429.5 (1409.0)
107+04.88 to 107+21.95	3600	HP 250X85	400	800	0	447.1-446.74 (1467.2-1465.7)	429.5 (1,2) (1409.0)	429.5 (1409.0)
107+21.95 to 107+50.00	4200	HP 250X85	400	800	0	446.69-446.3 (1465.5-1464.2)	429.5 (1,2) (1409.0)	429.5 (1409.0)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 442.5 m (1451.8 ft) at U4 Type 1 Retaining Wall.*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Branch A shall be contacted for further recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. The steel "H" pile section below the scour elevation has been designed to develop the required nominal resistance at all support locations.

3. Specialty equipment may be required for installation of the battered steel "H" piles. The contractor should be prepared to alternate from battered to vertical installation methods of the steel "H" piles.

4. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

5. The contractor should anticipate hard and erratic driving of the steel "H" piles due to the presence of very dense foundation material. The contractor should anticipate field cutting and splicing of the steel "H" piles. Refer to the LOTB sheets for details.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-7260, Jacqueline Martin at (916) 227-5503 or Reid Buell at (916) 227-7190, of the Office of Geotechnical Design-North, Branch A.

Project Information

Project Information

Standard special Provisions 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:

A. Log of Test Borings for U4RW, Bridge No. 10E0002.

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

A. Foundation Report for U4RW, Bridge No. 10E0002, dated December 8, 2006.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North

Report by:

Jacqueline Martin

JACQUELINE MARTIN
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R.E. Pending
Structure OE

REFERENCES

Federal Highway Administration (FHWA), Design and Construction of Driven Pile Foundations: Workshop Manual – Volume 1 (Pub. No. FHWA HI-97-013) published January 1997, Revised November1998.

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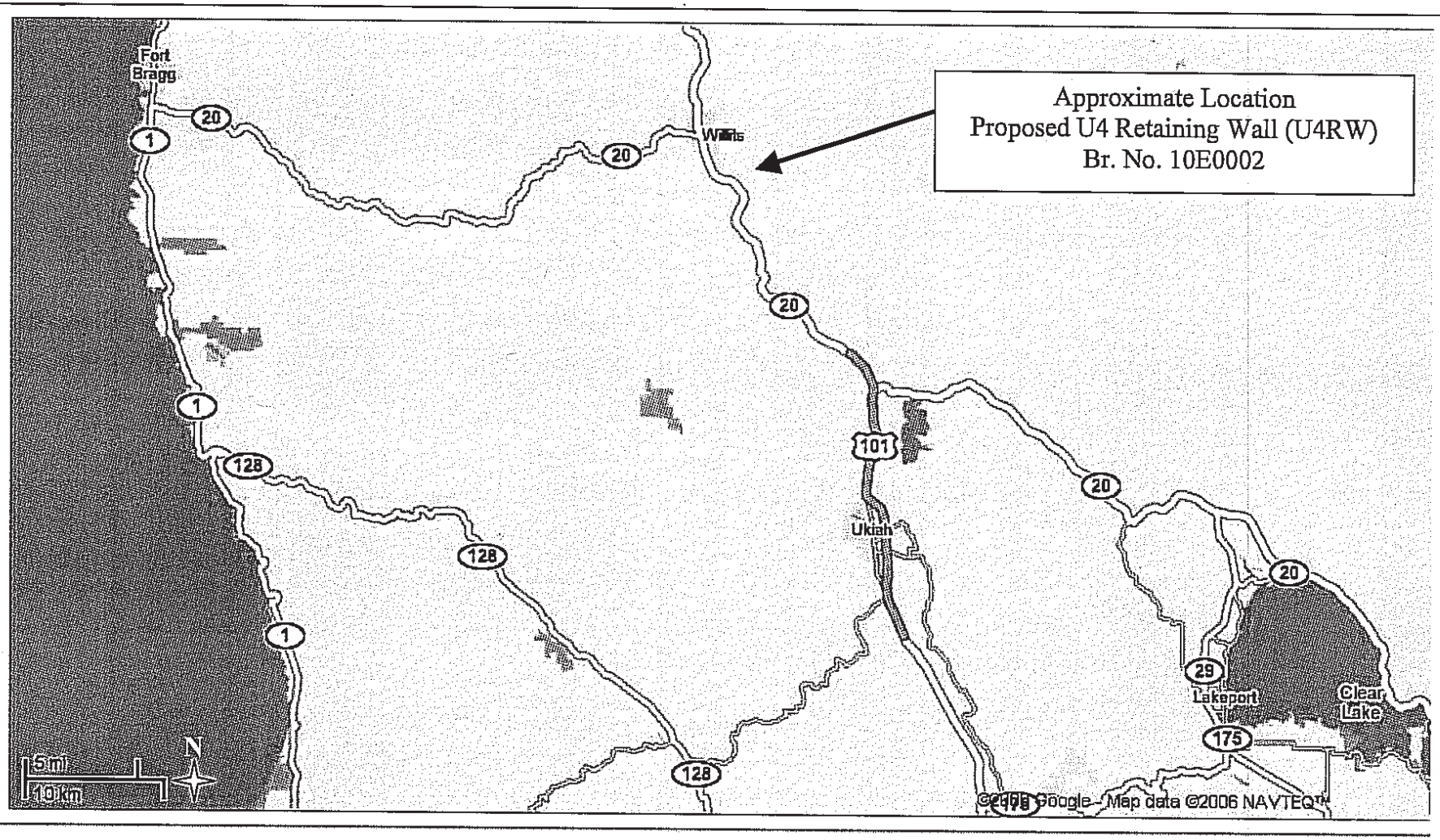
Seismic Design Criteria (2006), California Department of Transportation, Version 1.4.

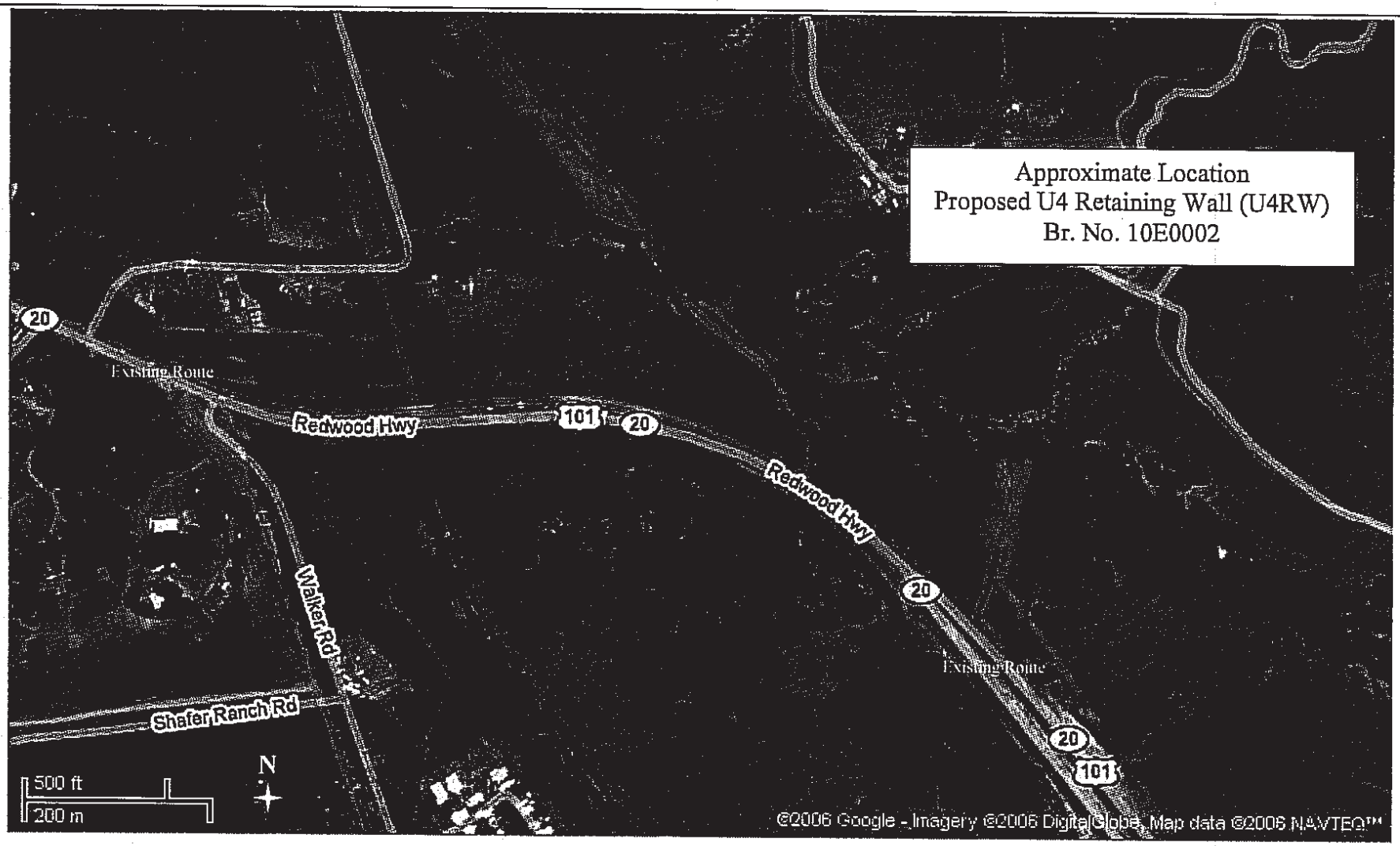
State of California, Department of Transportation (Caltrans):

- Standard Plans, July 2004 and May 2006
- Standard Specifications, July 2004 and May 2006
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

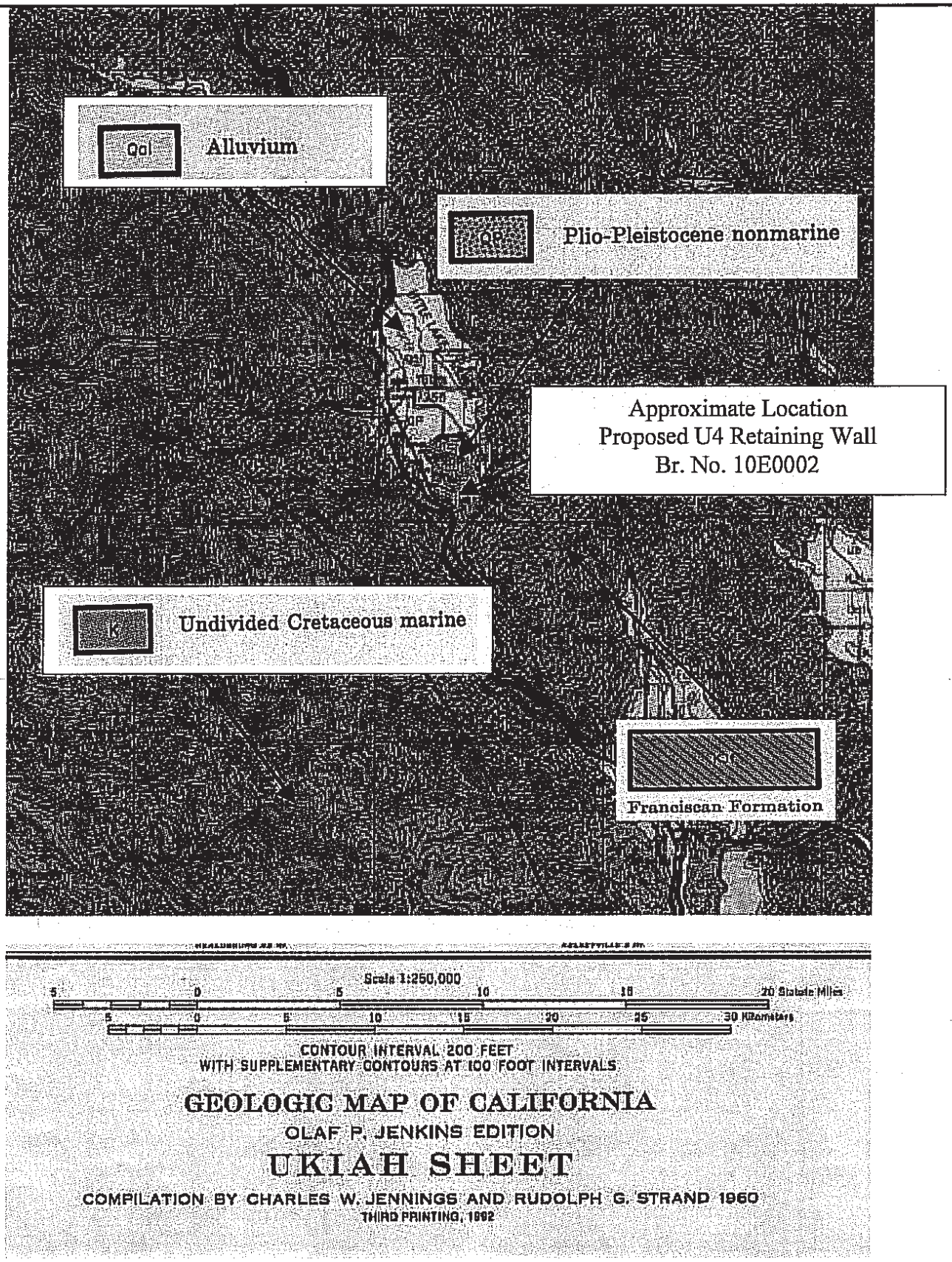
Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.



Caltrans Division of Engineering Services Geotechnical Services Geotechnical Design – North	EA: 01-262001 December 8, 2006	Location Map
	01-MEN-101 KP R70.53/PM R43.83 U4 Retaining Wall, Br. No. 10E0002	Figure 1



Caltrans Division of Engineering Services Geotechnical Services Geotechnical Design – North	EA: 01-262001 December 8, 2006	Location Map
	01-MEN-101 KP R70.53/PM R43.83 U4 Retaining Wall, Br. No. 10E0002	Figure 2



Caltrans Division of Engineering Services Geotechnical Services Geotechnical Design – North	EA: 01-262001 December 8, 2006	Geologic Map
	01-MEN-101 KP R70.53/PM R43.83 U4 Retaining Wall, Br. No. 10E0002	Figure 3

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: July 6, 2009

File: 01-MEN-101-KP R70.62/PM R43.88
EA#01-262001
S101-W20 Connector Bridge (NEW)
Br. No. 10-0129F

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed S101-W20 Connector Bridge (Br. No. 10-0129F). The Office of Geotechnical Design North, Branch A completed an intermittent subsurface investigation from April 2004 to October 2006 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (April 2004 to October 2006) along with a review of the available General Plan dated May 6, 2009 and Foundation Plan dated May 6, 2009. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new S101-W20 Connector Bridge (Br. No. 10-0129F) will be incorporated as part of the Willits Bypass Project. The proposed new bridge will consist of one new structure and will be located within the city limits of Willits in Mendocino County (Figures 1 and 2). The structure will consist of a single span with cast-in-place/ reinforced concrete box girder (2 cell) type structure with diaphragm abutments and wingwalls supported on driven steel H-Piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and

narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The chain contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the south end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from through flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1960 (Third Printing, 1992), California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary deposits) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Plio-Pleistocene nonmarine sedimentary deposits identified as the Cache Formation. Refer to the field investigation section of this report for more detailed information.

Landslide

Our investigation has determined that Abutment 1 is located within the slope of a pre-existing shallow landslide. The landslide is roughly defined with a principal scarp at the head of the slide and a center hummocky surface that includes relatively young vegetation within the debris slope as shown on the aerial photo (Figure 4). The landslide is roughly 45 meters in length, with an average width of approximately 100 meters, and has an elevation differential of approximately

20 meters from the scarp to the toe. The toe of the landslide extends into Haehl Creek where a potential slide plane was observed near the creek elevation. The potential slide plane consists of black and dark bluish gray elastic silt and fat clay with some organics dipping approximately 19 to 21 degrees towards the north (towards Haehl Creek). Remedial work will be required to stabilize the landslide prior to construction and installation of the new proposed foundations. The remedial work will be included in a separate report to be prepared by Jim Morris from the Office of Geotechnical Design North.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted an intermittent subsurface investigation from April 2004 to October 2006.

The subsurface investigation included the drilling of exploratory borings, a cone penetrometer sounding, the installation of a slope inclinometer (an instrument for measuring movement of the landslide) and the installation of a piezometer and observation well to monitor groundwater conditions. Upon completion of the subsurface investigation, laboratory testing of selected samples was performed along with monitoring of the instrumentation. Multiple site reconnaissances were performed prior to and following the subsurface investigation.

The subsurface investigation consisted of three mud rotary borings (Nos. B-5, B-37 and B-40), an auger boring (No. B-78) and one cone penetrometer sounding (No. C-7). The mud rotary borings were advanced using a self-casing wireline drilling method extending down to a maximum depth of 46.2 m (151.5 ft). The equipment used to drill Borings B-37 and B-40 consisted of a Mobile B-80 drill rig equipped with a safety hammer. The equipment used to drill Boring B-5 consisted of an Acker drill rig equipped with a safety hammer. The equipment used to drill Boring B-78 consisted of a Mobile B-47 drill rig equipped with 6-inch augers. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler at 1.5-meter intervals except for Borings B-40 and B-78. Select soil samples were collected and bagged for laboratory testing. Samples were not collected in Borings B-40 and B-78. Slope inclinometer (SI) piping was installed and grouted in Boring B-40 for monitoring slope movement. The cone penetrometer sounding was used to supplement the subsurface investigation.

The subsurface investigation revealed the foundation materials encountered at the project site generally consist of 5.2 m (17 ft.) to 16.2 m (53 ft.) of stiff to hard clay and dense to very dense silt, silty gravel, silty sand and clayey sand. Underlying this material is 4.1 m (13.5 ft.) to 5.2 m (17 ft.) of hard fat clay and elastic silt with organics. Below the hard fat clay and elastic silt is interbedded hard clay and dense to very dense sand, clayey sand and silt. The foundation materials were encountered down to a maximum depth explored of 46.2 m (151.5 ft), an approximate elevation of 403.2 m (1322.7 ft).

The slope inclinometer has been monitored from December 14, 2005 thru April 29, 2009. It appears the landslide is dormant except for the upper 1.5 m (5.0 ft). This movement may be interpreted as creep that was observed during a field visit on February 6, 2007. The creep appeared to be occurring within the hummocky surface of landslide material in the reddish brown and yellowish brown silty sand and sandy clay with gravel.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions.

Ground Water

Ground water was measured in mud rotary Boring B-37, auger Boring B-78 and cone penetrometer sounding C-7 during and after the subsurface investigation. Ground water levels varied from elevation 447.46 m (1468.0 ft) in Boring B-37 to elevation 441.49 m (1448.5 ft) in sounding C-7. Due to the large variation in the ground water elevation, it appears artesian conditions may exist at the site.

Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Scour Evaluation

A Final Hydraulic Report (dated March 22, 2005) for the Haehl Creek structures was completed by the Office of Structure Maintenance and Investigations, Structure Hydraulics Branch. According to this report, there are no local scour concerns for this structure since it is a single span structure. The report recommends the abutment foundations be designed assuming no ground support (lateral and vertical) as a result of soil loss due to possible degradation or lateral stream migration down to the current stream thalwag elevation (lowest elevation in channel). The approximate thalwag elevation provided for the proposed S101-W20 Connector Bridge (Br. No. 10-0129F) is 440.25 m (1444.4 ft).

According to the Final Hydraulic Report, Rock Slope Protection (RSP) was mentioned and will be designed by the District to protect the roadway approach fills, if required.

For further information including site-specific scour assessment and mitigation measures, the Structures Hydraulics Branch should be contacted.

Corrosion Evaluation

Composite soil samples were collected from Boring B-37 during the field investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not considered to be corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for S101-W20 Connector Bridge (Br. No. 10-0129F)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639872	B-37	0.0-1.5	8.00	1800	N/A	N/A
C639873	B-37	5.0-6.1	7.24	1407	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Triaxial (UU) and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located less than one kilometer west of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g. The soil profile at the site may be classified as Type D, as defined in the Department's Seismic Design Criteria (SDC, 2006, Version 1.4). The recommended design Acceleration Response Spectrum (ARS) curve shown in Figure 5 was obtained by modifying the ARS curve in Figure B.8 of the SDC corresponding to a PBA of 0.7g. These modifications were introduced to account for the proximity of the controlling seismic source and included a 20% increase in spectral accelerations for periods greater than or equal to 1.0 second. No modifications were introduced for periods less than or equal to 0.5 second. The spectral accelerations for periods between 0.5 and 1.0 second were obtained by linear interpolation.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Foundation Recommendations

The following foundation recommendations are for the new proposed S101-W20 Connector Bridge (Br. No. 10-129F).

At Abutment 1 and Abutment 2 support locations, driven steel HP 360x174 "H" piles are recommended for support. The specified pile tip elevations (SPTE), shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed S101-W20 Connector Bridge (Br. No. 10-0129F).

Location	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
			Compression kN	Tension kN			
Abutment 1	HP 360X174	625	1250	0	445.00 (1460.0)	431.0 (1,2) (1414.0)	431.0 (1414.0)
Abutment 2	HP 360X174	625	1250	0	443.50 (1455.1)	431.0 (1,2) (1414.0)	431.0 (1414.0)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 440.25 m (1444.4 ft) at Abutments 1 and 2.*

Retaining Wall Foundations

The following foundation recommendations are for the proposed Type 1 Retaining Wall structures located at Abutment 1 and Abutment 2 locations as shown on the Foundation Plan dated May 6, 2009, Abutment Details No. 1 sheet dated June 30, 2009, and Abutment Details No. 2 sheet dated June 30, 2009. At these locations, it is recommended driven steel piles and spread footings be used to support these retaining wall segments.

Abutment 1-Retaining Wall Type 1

Driven steel HP 250X62 "H" piles are recommended for support of the 4800 mm and 3600 mm retaining wall height segments and are shown below in Table 3. The specified pile tip elevations will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 3. Pile Data Table for the new proposed Type 1 Retaining Wall Segment – Abutment 1 (Br. No. 10-0129F).

Location	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 1 RW Type 1	4800	HP 250X62	400	800	0	446.80 (1465.9)	431.5 (1,2) (1415.7)	431.5 (1415.7)
Abutment 1 RW Type 1	3600	HP 250X62	400	800	0	448.00 (1469.8)	431.5 (1,2) (1415.7)	431.5 (1415.7)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 440.25 m (1444.4 ft) at Abutment 1 Type 1 Retaining Wall Segment.*

Spread footings are recommended for support of the 2400 mm and 1800 mm retaining wall height segments and are shown below in Table 4. The ultimate soil bearing capacity will meet or exceed three times the gross allowable soil bearing pressure. This recommendation is based on the minimum footing widths and bottom of footing elevations listed below.

Table 4. Spread Footing Data Table for the new proposed Type 1 Retaining Wall Segment – Abutment 1 (Br. No. 10-0129F).

Support Location	Retaining Wall Height mm	Support Type	Minimum Footing Width mm	Bottom of Footing Elevation m	Recommended Soil Bearing Pressures	
					ASD[1]	LFD[2]
					Gross Allowable Soil Bearing Pressure (q_{all}) kPa	Ultimate Soil Bearing Pressure (q_{ult}*) kPa
Abutment 1 RW Type 1	2400	Spread Footing	1600	449.00	105	N/A
Abutment 1 RW Type 1	1800	Spread Footing	1300	450.00	90	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}*).*

Abutment 2-Retaining Wall Type 1

Driven steel HP 250X62 "H" piles are recommended for support of the 4800 mm, 3600 mm and 2400 mm retaining wall height segments and are shown below in Table 5. The specified pile tip elevations will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 5. Pile Data Table for the new proposed Type 1 Retaining Wall Segment – Abutment 2 (Br. No. 10-0129F).

Location	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 2 RW Type 1	4800	HP 250X62	400	800	0	444.80 (1459.3)	431.0 (1,2) (1414.0)	431.0 (1414.0)
Abutment 2 RW Type 1	3600	HP 250X62	400	800	0	445.50 (1461.6))	431.0 (1,2) (1414.0)	431.0 (1414.0)
Abutment 2 RW Type 1	2400	HP 250X62	400	800	0	446.50 (1464.9)	431.0 (1,2) (1414.0)	431.0 (1414.0)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 440.25 m (1444.4 ft) at Abutment 2 Type 1 Retaining Wall Segment.*

Spread footings are recommended for support of the 1200 mm retaining wall height segments and are shown below in Table 6. The ultimate soil bearing capacity will meet or exceed three times the gross allowable soil bearing pressure. This recommendation is based on the minimum footing widths and bottom of footing elevations listed below.

Table 6. Spread Footing Data Table for the new proposed Type 1 Retaining Wall Segment – Abutment 2 (Br. No. 10-0129F).

Support Location	Retaining Wall Height mm	Support Type	Minimum Footing Width mm	Bottom of Footing Elevation m	Recommended Soil Bearing Pressures	
					ASD[1]	LFD[2]
					Gross Allowable Soil Bearing Pressure (q_{all}) kPa	Ultimate Soil Bearing Pressure (q_{ult}*) kPa
Abutment 2 RW Type 1	1200	Spread Footing	1000	448.0	80	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}*).*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Branch A shall be contacted for further recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. The steel "H" piles sections below the scour elevation have been designed to develop the required nominal resistance.

3. The Contractor should anticipate hard and erratic driving of the steel "H" piles due to the presence of very dense material. The Contractor should anticipate field cutting and splicing of the steel "H" piles. Refer to the LOTB sheets for details.

4. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

5. Scour is predicted to extend to elevation 440.25 m (1444.4 ft) at abutment support locations. In order to achieve the specified pile tip elevation, predrilling is recommended for the installation of the steel "H" piles. The HP 360X174 piles and HP 250x62 piles (including battered) shall be driven in pre-drilled holes that conform to the Standard Specifications in Section 49-1.06, "Pre-drilled Holes." The predrilled holes are not to extend below the elevations shown in Table 5.

Table 5. Predrilled Elevations

Location	Predrilled Elevation
Abutment 1	440.25 m (1444.4 ft.)
Abutment 1-Retaining Wall Type 1	440.25 m (1444.4 ft.)
Abutment 2	440.25 m (1444.4 ft.)
Abutment 2-Retaining Wall Type 1	440.25 m (1444.4 ft.)

6. Specialty equipment may be required for installation of the battered steel "H" piles. The Contractor should be prepared to alternate from battered to vertical installation methods of the steel "H" piles.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035, or Reid Buell (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard special Provisions 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:

A. Log of Test Borings for S101-W20 Connector Bridge, Bridge No. 10-0129F.

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

A. Foundation Report for S101-W20 Connector Bridge, Bridge No. 10-0129F, dated July 6, 2009.

Data and information available for inspection at the District Office:

A. None

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

A. None

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Structure OE

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State of California, Department of Transportation (Caltrans):

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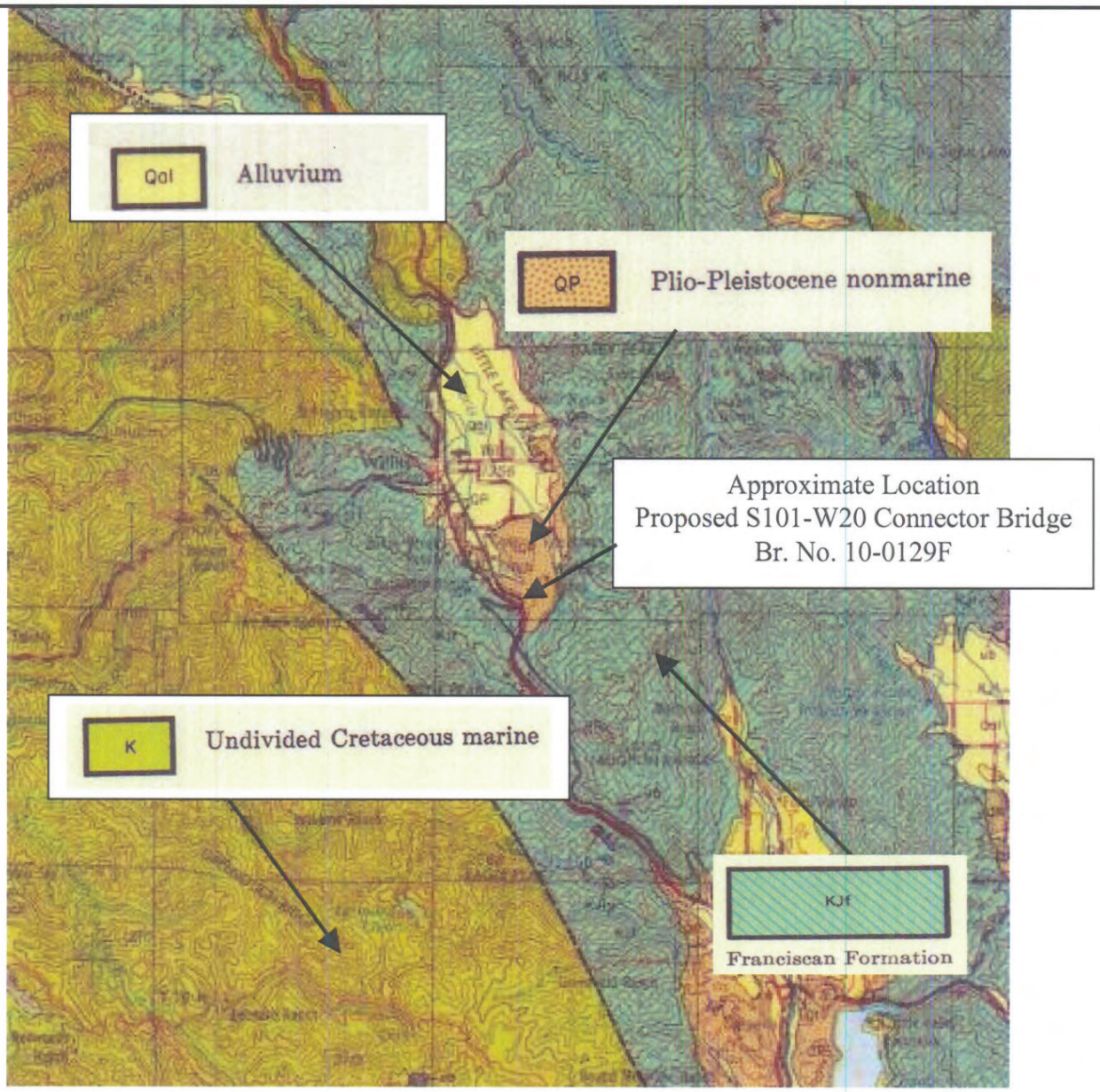
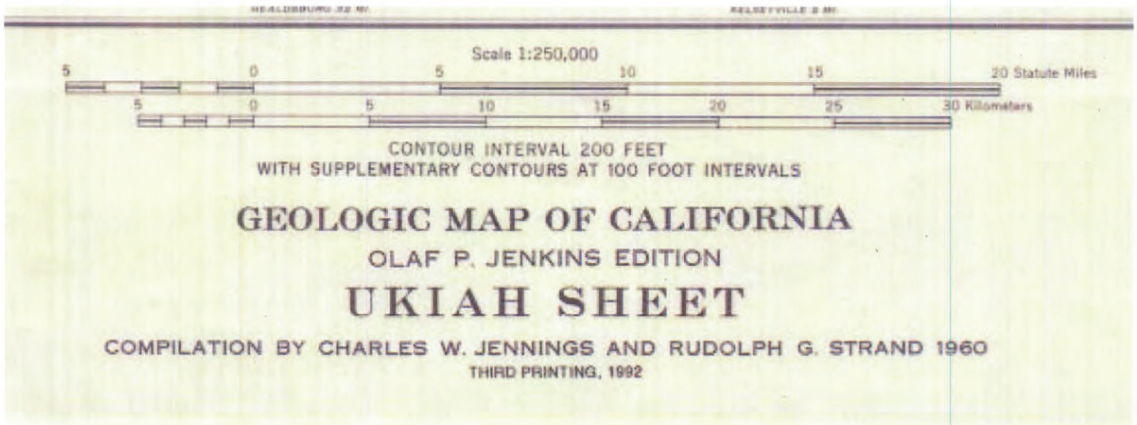
- Standard Specifications, May 2006
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.

Caltrans **Division of Engineering Services** Geotechnical Services Office of Geotechnical Design – North	EA: 01-262001 July 6, 2009	Location Map
	01-MEN-101 KP R70.62/PM R43.9 **S101-W20 Connector Bridge, Br. No. 10-0129F**	Figure 1

Caltrans **Division of Engineering Services** Geotechnical Services Office of Geotechnical Design – North	EA: 01-262001 July 6, 2009	Location Map
	01-MEN-101 KP R70.62/PM R43.88 **S101-W20 Connector Bridge, Br. No. 10-0129F**	Figure 2





Division of Engineering Services Geotechnical Services Office of Geotechnical Design – North	EA: 01-262001 July 6, 2009	Geologic Map
	01-MEN-101 KP R70.62/PM R43.88 **S101-W20 Connector Bridge, Br. No. 10-0129F**	Figure 3

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Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – North

EA: 01-262001	Landslide Map
July 6, 2009	
01-MEN-101 KP R70.62/PM R43.88 **S101-W20 Connector Bridge, Br. No. 10-0129F**	Figure 4

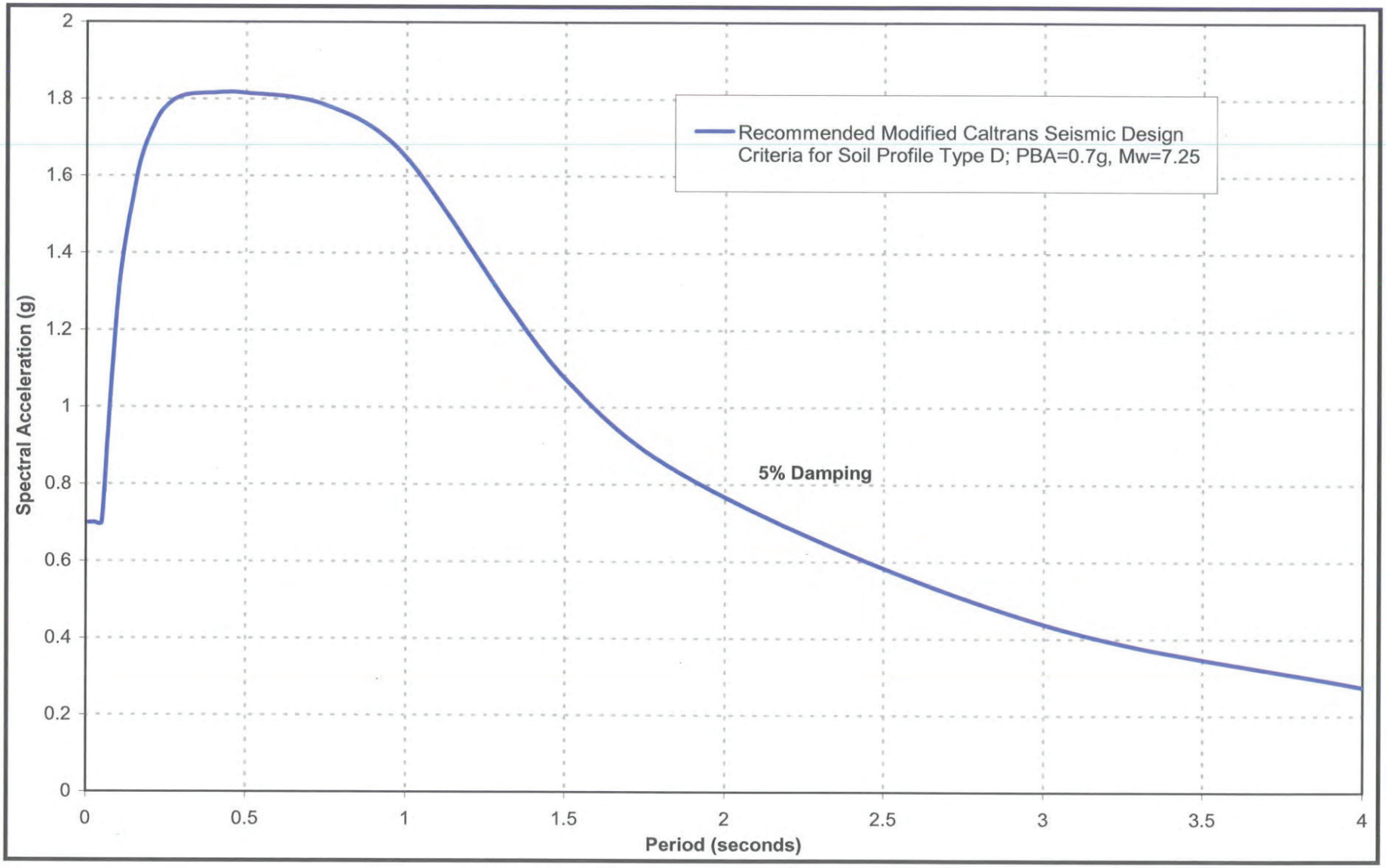

Figure 5. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: October 20, 2006

File: 01-MEN-101-KP R70.41
EA#01-262001
Route 101/20 Separation (NEW)
Br. No. 10-0128R/L

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Route 101/20 Separation (Br. No. 10-0128R/L). The Office of Geotechnical Design North, Structure Foundations Branch completed a subsurface investigation from May 2004 to September 2005 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (May 2004 to September 2005) along with a review of the available General Plan dated April 4, 2006 and Foundation Plan dated April 3, 2006. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new 101/20 Separation (Br. No. 10-0128R/L) will be incorporated as part of the Willits Bypass Project. The proposed new separation will consist of two new separate structures and will be located within the city limits of Willits in Mendocino County (Figures 1 and 2). The left and right structures will consist of single spans with prestressed cast-in-place box girder (3 cell) type structure with seat type abutments and wingwalls supported on driven steel H-Piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Range province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The chain contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the south end of the Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from through flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary deposits) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Plio-Pleistocene nonmarine sedimentary deposits identified as the Cache Formation. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation in May 2004 to September 2005.

The subsurface investigation consisted of four mud rotary borings (Nos. B-1, B-3, B-43 and B-51) and two cone penetrometer soundings (Nos. C-1 and C-2). The mud rotary borings were advanced using a self-casing wireline-diamond coring method extending down to a maximum depth of 46.2 m (151.5 ft). The equipment consisted of a CME drill rig equipped with a CME automatic hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler. Selected soil samples were bagged for laboratory testing. The cone penetromer soundings were used to supplement the subsurface investigation.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed right bridge (Br. No. 10-0128R) generally consist of approximately 14.1 m (46.5 ft) to 18.3 m (60.0 ft) of alternating layers of firm to very stiff and hard clay, medium dense sand and silty sand overlying medium dense to very dense gravel and silty sand and hard clay. The foundation material was encountered down to the maximum depth explored of 37.2 m (122.0 ft), an elevation of 414.5 m (1360.1 ft).

The subsurface investigation revealed the foundation materials encountered at the location of the proposed left bridge (Br. No. 10-0128L) generally consist of approximately 11.6 m (38.1 ft) to 15.9 m (52.2 ft) of dense to very dense silt, sand, silty sand, clayey sand and gravel with trace hard cobbles overlying very stiff to hard clay and sandy clay. The foundation material was encountered down to the maximum depth explored of 46.2 m (151.5 ft), an elevation of 411.6 (1350.6 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water was measured in two of the four mud rotary borings (B-1 and B-51) drilled during the subsurface investigation. Ground water was not measured in borings B-3 and B-43 and the borings were immediately backfilled. Ground water levels varied from elevation 446.0 m (1463.3 ft) in boring B-1 to elevation 447.3 m (1467.6 ft) in Boring B-51. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Corrosion Evaluation

Composite soil samples were collected during the subsurface investigation. The results will be forwarded once completed.

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request once the results are received.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 1.1 kilometers (0.7 miles) west of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g. The soil profile at the site may be classified as Type D, as defined in the Department's Seismic Design Criteria (SDC, 2006, Version 1.4). The recommended design Acceleration Response Spectrum (ARS) curve shown in Figure 4 was obtained by modifying the ARS in Figure B.8 of the SDC corresponding to a PBA of 0.7g. These modifications were introduced to account for the close proximity of the controlling seismic source and included a 20% increase in spectral accelerations for periods greater than or equal to 1.0 second. No modifications were introduced for periods less than or equal to 0.5 second. The spectral accelerations for periods between 0.5 and 1.0 second were obtained by linear interpolation.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Foundation Recommendations

The following foundation recommendations are for the new proposed Rte. 101/20 Separation (Br. No. 10-128R/L).

Rte. 101/20 Separation (Br. No. 10-0128R)

At all structure support locations, driven steel HP 250X85 "H" piles are recommended for support at the right bridge. The specified pile tip elevations (SPTE), shown below in Table 1, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 1. Pile Data Table for the new proposed Rte. 101/20 Separation (Br. No. 10-0128R).

Location	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
			Compression kN	Tension kN			
Abutment 1	HP 250X85	400	800	0	452.7 (1485.2)	436.5 (1432.2)	436.5 (1432.2)
Abutment 2	HP 250X85	400	800	0	451.3 (1480.6)	434.5 (1425.6)	434.5 (1425.6)

Note: Design Pile Tip Elevation is controlled by the following demand: (1) *Compression.*

Rte. 101/20 Separation (Br. No. 10-0128L)

At all structure support locations, driven steel HP 360X132 "H" piles are recommended for support at the left bridge. The specified pile tip elevations (SPTE), shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed Rte. 101/20 Separation (Br. No. 10-0128L).

Location	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
			Compression kN	Tension kN			
Abutment 1	HP 360X132	400	800	0	452.3 (1483.9)	437.1 (1433.9)	437.1 (1433.9)
Abutment 2	HP 360X132	400	800	0	450.9 (1479.3)	434.1 (1424.3)	434.1 (1424.3)

Note: Design Pile Tip Elevation is controlled by the following demand: (1) *Compression.*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Structural Foundations shall be contacted for further recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions and time of construction. Refer to the Log of Test Boring Sheets for details.

2. Hard cobbles were encountered in Boring B-51 from an approximate elevation of 449.8 m (1475.6 ft) extending to an approximate elevation of 448.1 m (1470.1 ft) and may exist across the site. The contractor should anticipate encountering hard cobbles and having to drill through the cobbles to reach the predrilled elevations in Table 3 below.

3. At the left bridge location, the Abutment 1 and 2 footings will be located within the slope of the existing hillside. It is anticipated that hard driving will occur due to the very dense material encountered during the subsurface investigation for the left bridge. Therefore, predrilling is recommended for the installation of the steel "H" piles at the left bridge support locations. It is recommended the HP 360X132 piles (including battered) shall be driven in pre-drilled holes that conform to the Standard Specifications in Section 49-1.06, "Pre-drilled Holes." The predrilled holes are not to extend below the elevations shown in Table 3.

Table 3. Predrilled Elevations

Location	Predrilled Elevation
Abutment 1L	445.9 m (1463.1 ft)
Abutment 2L	445.8 m (1462.6 ft)

4. At the right bridge location, the Abutment 1 and 2 footings will be partially located within the slope of the existing hillside. It is anticipated that hard driving may exist during installation for some of the piles and that predrilling may be necessary. However, predrilling shall not be performed at the right bridge without prior authorization from this office.

5. Specialty equipment may be required for installation/predrilling of the battered steel "H" piles. The contractor should be prepared to alternate from battered to vertical installation methods of the steel "H" piles.

6. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

7. The contractor should anticipate hard and erratic driving of the steel "H" piles due to the presence of hard cobbles and very dense foundation material. The contractor should anticipate field cutting and splicing of the steel "H" piles. Refer to the LOTB sheets for details.

8. Due to anticipated hard driving at the left bridge, HP 360X132 "H" piles are to be used in place of 250X85 "H" piles.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Structural Foundations Branch 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 Tim Alderman at (916) 227-7260, or Reid Buell (916) 227-7190, of the Office of Geotechnical Design-North, Structural Foundations Branch.

Project Information

Standard special Provisions 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:

A. Log of Test Borings for Rte. 101/20 Separation, Bridge No. 10-0128R/L.

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

A. Foundation Report for Rte. 101/20 Separation, Bridge No. 10-0128R/L, dated October 20, 2006.

Report by:

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Engineering Geologist
Office of Geotechnical Design-North
Structure Foundations Branch

Report by:

Jacqueline Martin

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Engineering Geologist
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cc: OGDSN
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Reid Buell
R.E. Pending
Structure OE
Eskinder Taddese – PCE
Joe Peterson – DME

REFERENCES

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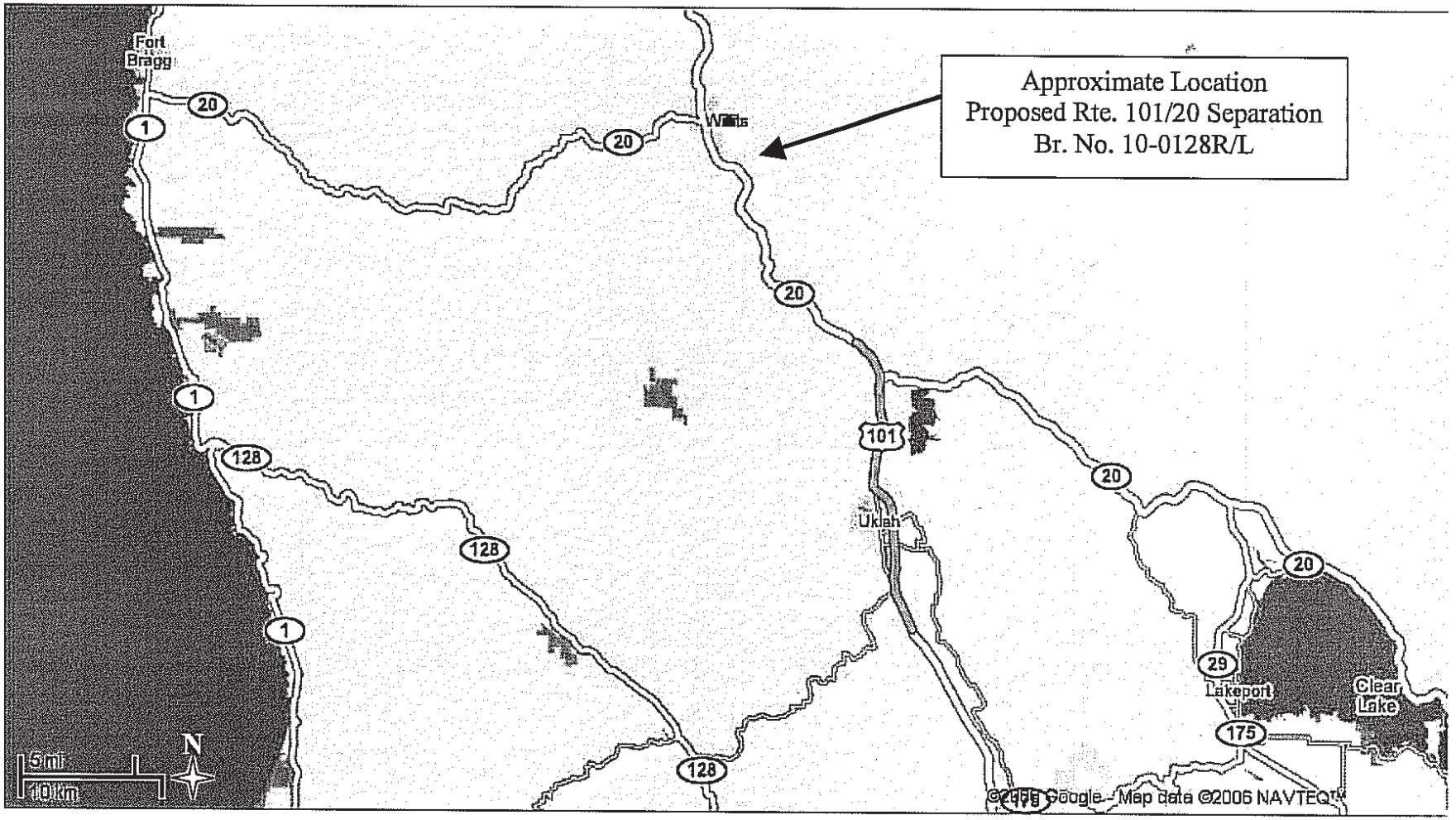
Norris, Robert M., and Webb, Robert M., 1990, Geology of California, Second Edition, John Wiley & Sons, Inc., pp 57, 359, 363.

Seismic Design Criteria (2006), California Department of Transportation, Version 1.4.

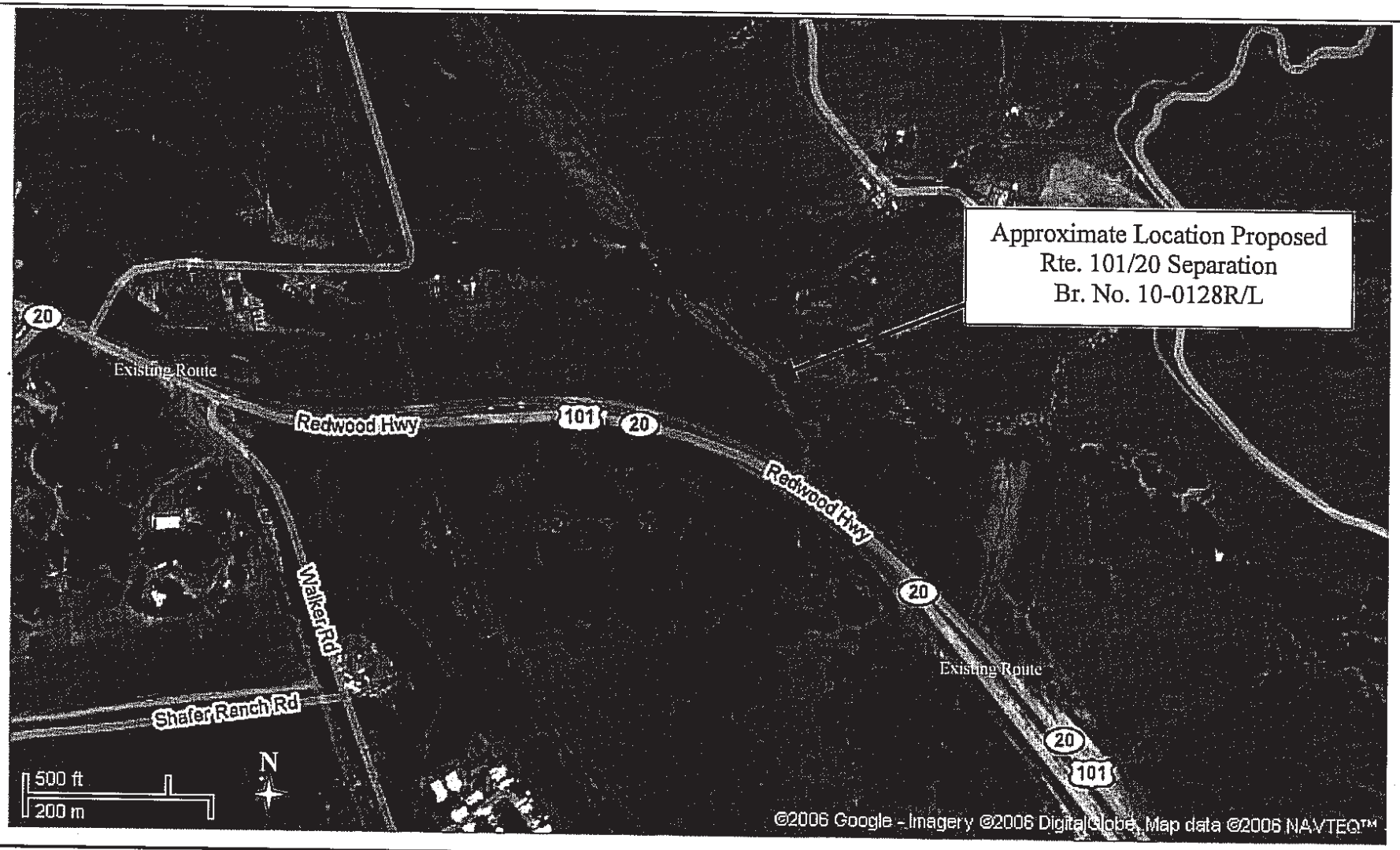
State of California, Department of Transportation (Caltrans):

- Standard Plans, May 2006
- Standard Specifications, May 2006
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.



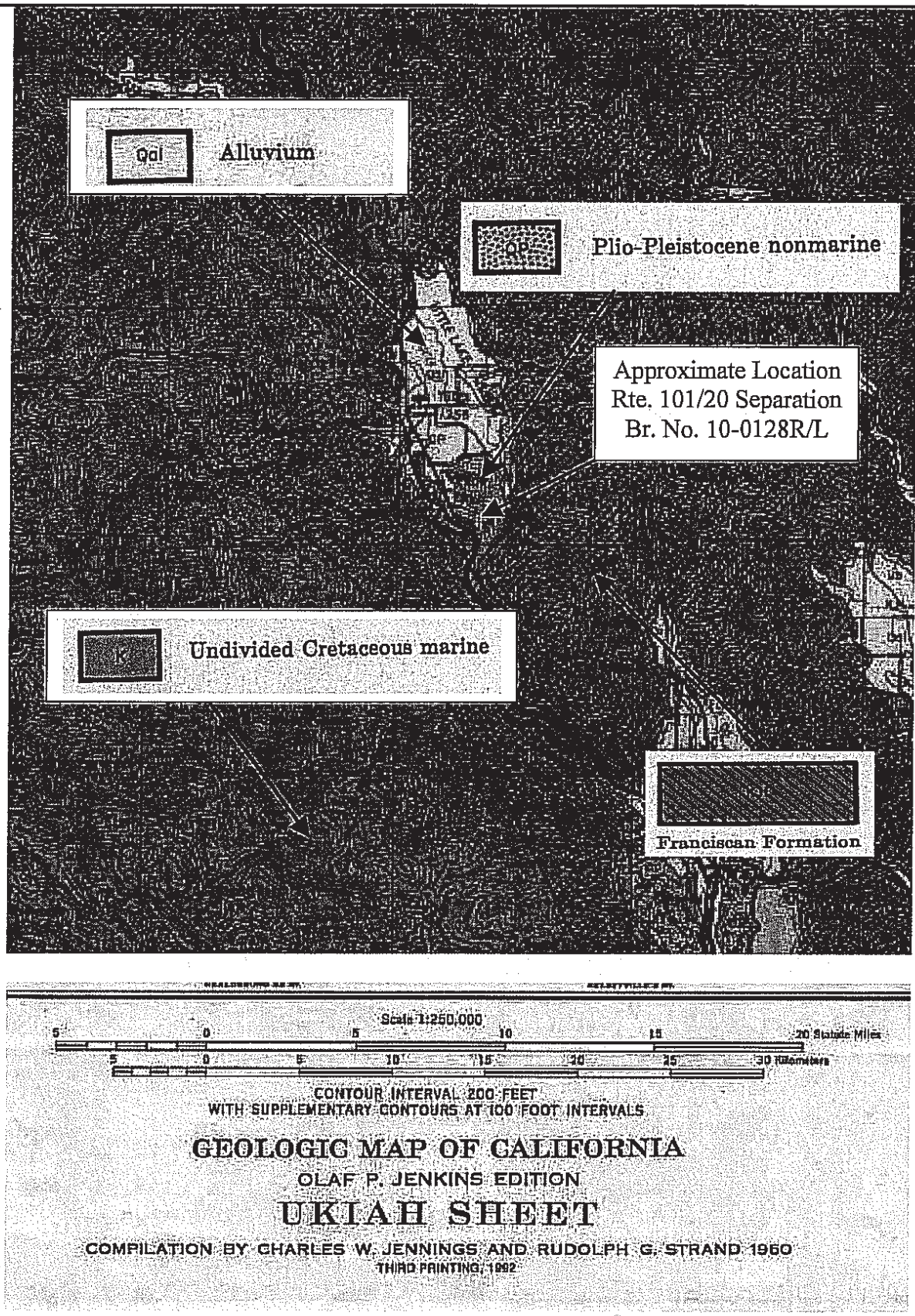
Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North Structure Foundations Branch	EA: 01-262001 October 20, 2006	Location Map
	01-MEN-101 KP R70.41/PM R43.75 **Rte. 101/20 Separation, Br. No. 10-0128R/L**	Figure 1



Caltrans

Division of Engineering Services
Geotechnical Services
Geotechnical Design – North
Structure Foundations Branch

EA: 01-262001	Location Map
October 20, 2006	
01-MEN-101 KP R70.41/PM R43.75 **Rte. 101/20 Separation, Br. No. 10-0128R/L**	Figure 2



Caltrans Division of Engineering Services Geotechnical Services Geotechnical Design – North Structure Foundations Branch	EA: 01-262001 October 20, 2006	Geologic Map
	01-MEN-101 KP R70.41/PM R43.75 Rte. 101/20 Separation, Br. No. 10-0128R/L	Figure 3

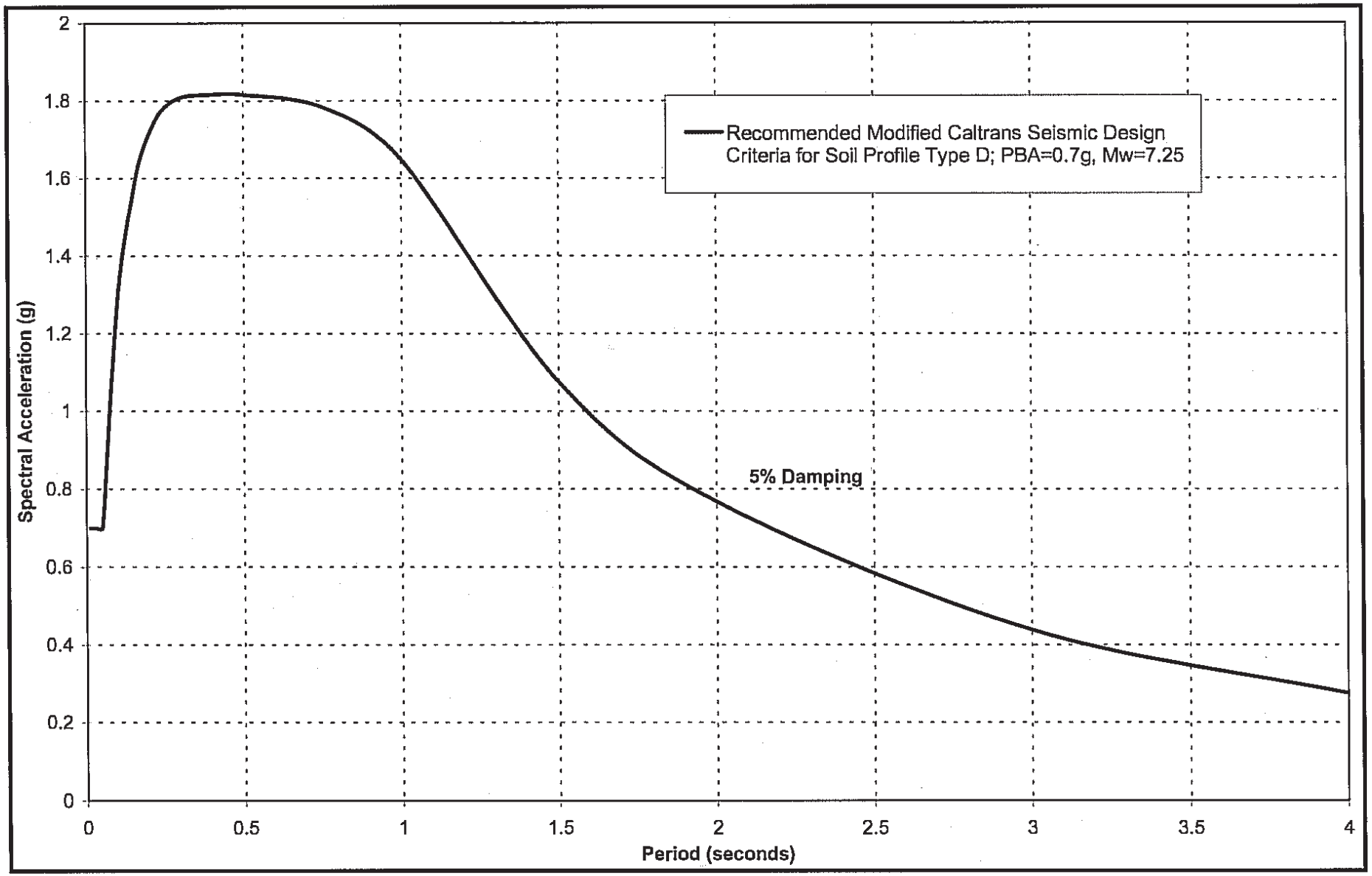


Figure 4. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: January 15, 2009

File: 01-MEN-101-KP R77.541/PM R48.18
EA#01-262001
Quail Meadows UC
Br. No. 10-0173

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Quail Meadows Undercrossing (Br. No. 10-0173). The Office of Geotechnical Design North, Branch A completed a subsurface investigation during September 2007 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (September 2007) along with a review of the available General Plan dated October 3, 2008. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Quail Meadows Undercrossing (Br. No. 10-0173) will consist of one new structure and will be incorporated as part of the Willits Bypass (Figures 1 and 2). The structure will be a single span prestressed cast-in-place box girder (3 cell) type structure with seat type abutments and wingwalls supported on driven steel H-Piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate

ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the north end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium, recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary deposits) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Quatenary alluvium. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation during September 2007.

The subsurface investigation consisted of two mud rotary borings (Nos. B-80 and B-81). The mud rotary borings were advanced using a self-casing wireline-diamond coring method extending down to a maximum depth of 46.48 m (152.5 ft). The equipment used to drill borings B-80 and B-81 consisted of a Acker MPCA drill rig equipped with a Longyear automatic hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler at 1.5-meter intervals. Selected soil samples were bagged for laboratory testing.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed undercrossing (Br. No. 10-0173) generally consist of soft to very stiff clay and gravelly clay interbedded with loose to dense clayey gravel, clayey sand, sand with clay and gravel with clay. The foundation materials were encountered down to the maximum depth explored of 46.48 m (152.5 ft), an elevation of 359.38 m (1179.1 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels were measured during the September 2007 field investigation at elevations 403.64 m (1324.28 ft.) and 403.63 m (1324.25 ft.) in Borings B-80 and B-81, respectively. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Corrosion Evaluation

Composite soil samples were collected in one of the borings during the September 2007 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the

possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for Quail Meadows UC (Br. No. 10-0173)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639919	B-80	1.52	5.82	4729	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Moisture Content, Specific Gravity, Consolidation, Triaxial (Unconsolidated-Undrained) and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results are available upon request.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 0.5 kilometer southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Based on the boring logs a final Caltrans Seismic Design Criteria (CSDC) Acceleration Response Spectrum (ARS) Curve corresponding to soil profile Type D is recommended for design. Please note that due to the close proximity of this structure to the fault, we have performed a second modification to the CSDC ARS curve (see Figure 4). The modification is such that there is no increase in spectral accelerations (SA) for periods less than 0.5 second, and a 20% increase in SA for periods greater than 1 second. Between the periods of 0.5 and 1 second, a linear interpolation was used to estimate the SA.

Fill Settlement

Due to the presence of compressible clay soils beneath the proposed approach fills at Abutment 1 and Abutment 2, a 270 day waiting period is to be required after completion of the approach fills and before installing the steel "H" piles. Settlement will take place during and after the construction of the approach fills. It has been estimated that the maximum total settlement of the approach fills will be about 500 millimeters.

It has been calculated that 90% of the settlement of the approach fills will take place approximately 270 days after placement of the fill is completed. If it is necessary to accelerate the settlement, placement of additional surcharge fill and/or the use of wick drains may be recommended. Details of these measures will be included in the Geotechnical Design Report to be prepared by this Office.

Regardless of the target settlement or the estimated settlement time period, 500 mm and 270 days, respectively, the rate of settlement obtained during the settlement monitoring will determine if the majority of the settlement in the fill is complete. That is if the settlement rate is noticeably reduced sooner than the end of the estimated settlement time period, then it would be acceptable for construction of the abutment foundations to commence. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

Foundation Recommendations

The following foundation recommendations are for the new proposed Quail Meadows UC (Br. No. 10-0173). At this location, it is recommended driven steel HP 250X85 piles be used at the Abutment 1 and 2 support locations. The specified pile tip elevations,

shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed Quail Meadows UC, (Br. No. 10-0173) at Abutment 1 and 2 locations.

Location	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
			Compression kN	Tension kN			
Abutment 1	HP 250X85	575	1150	0	411.400 (1331.56)	380.5 (1) (1248.4)	380.5 (1248.4)
Abutment 2	HP 250X85	575	1150	0	411.400 (1331.56)	380.5 (1) (1248.4)	386.0 (1248.4)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Structural Foundations shall be contacted for further recommendations.

3. Specified pile tip elevations shown in Table 2 above are based on preloading of compressible clay soils achieved by placement of approach fills prior to construction of the proposed abutment foundations. If preloading is not performed, the Office of Geotechnical Design-North is to be contacted for revised foundation recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. Settlement shall be monitored. Monitoring devices shall consist of settlement platforms placed at the bottom of the approach fill embankments at Abutments 1 and 2. A minimum of six settlement devices (three or more beneath each approach fill) shall be placed beneath the centerline of embankments at Abutments 1 and 2 at locations to be determined by the Engineer.

3. Due to the presence of compressible clay soils beneath the proposed abutments, the abutment locations shall be preloaded. Preloading shall be achieved by placement of approach fills prior to construction of the Quail Meadows Undercrossing. A 270-day waiting period is required prior to installation of piles at Abutments 1 and 2. The fill settlement shall be monitored during the waiting period. If it is determined that the settlement has ceased prior to the end of the 270-day waiting period, with the written approval of the Engineer, the waiting period may be terminated at that time. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

4. Piles to be driven through the new fill embankment at Abutments 1 and 2 are to be placed in predrilled holes to elevation 406.0 meters per Caltrans Specifications 49-1.06

5. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035, or Reid Buell (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard Special Provisions 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:

A. Log of Test Borings for Quail Meadows UC, Bridge No. 10-0173.

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

A. Foundation Report for Quail Meadows UC, Bridge No. 10-0173, dated January 15, 2009.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North
Structure Foundations Branch

Report by:

REZA MAHALLATI
Senior Materials & Research Engineer
Office of Geotechnical Design-North

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cc: OGDSN
GS File Room
Reid Buell
R.E. Pending
Structure OE

REFERENCES

Federal Highway Administration (FHWA), Design and Construction of Driven Pile Foundations: Workshop Manual – Volume 1 (Pub. No. FHWA HI-97-013) published January 1997, Revised November1998.

Harden, Deborah R., 1998, California Geology, Prentice Hall, 252 pp.

Jennings, Charles W. and Strand, Rudolph G., 1960, Geologic Map of California [Ukiah Sheet] and Index to Geologic Mapping, California Division of Mines, scale 1:250,000, 2 sheets.

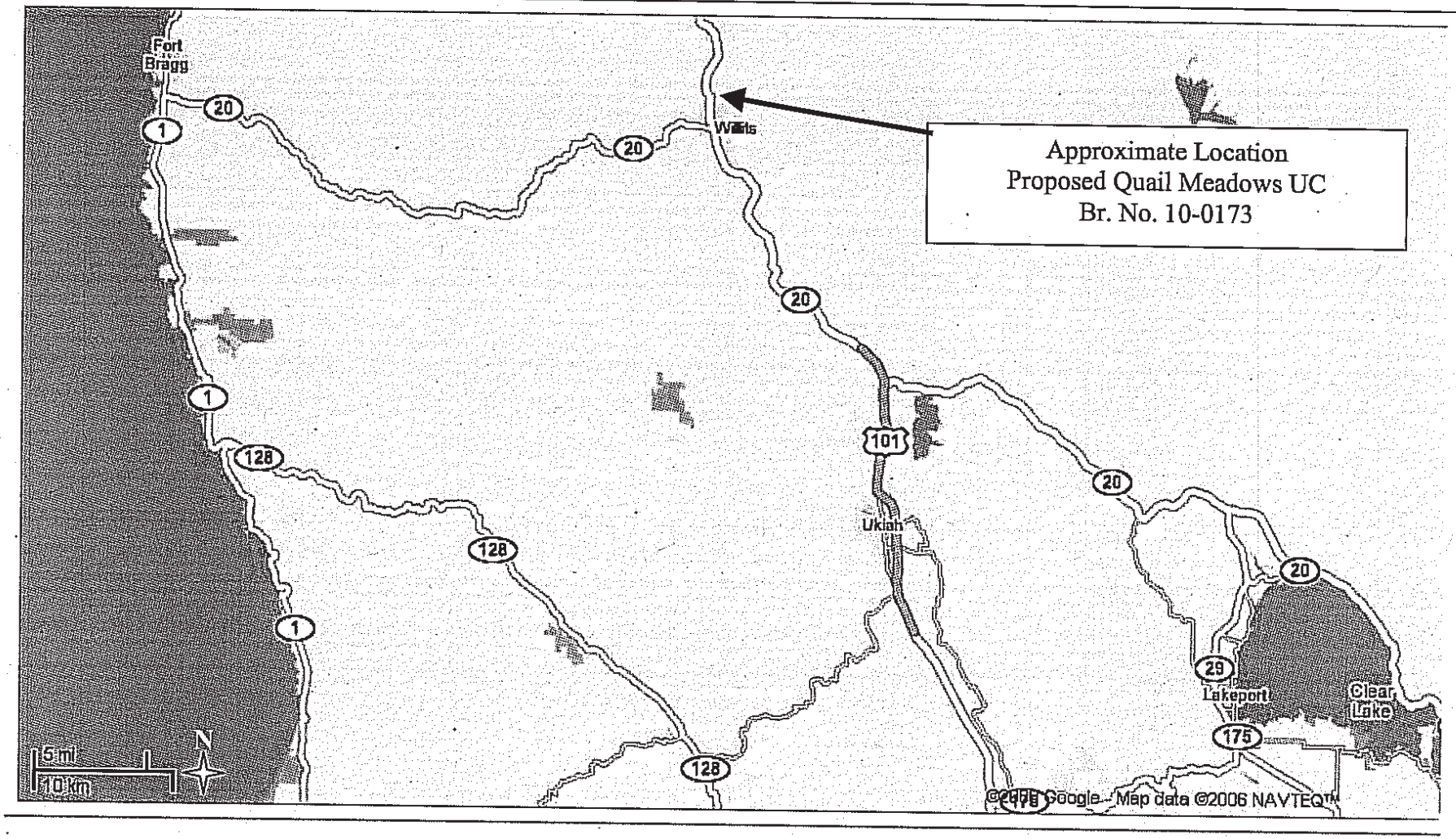
Norris, Robert M., and Webb, Robert M., 1990, Geology of California, Second Edition, John Wiley & Sons, Inc., pp 57, 359, 363.

Seismic Design Criteria (2006), California Department of Transportation, Version 1.4.

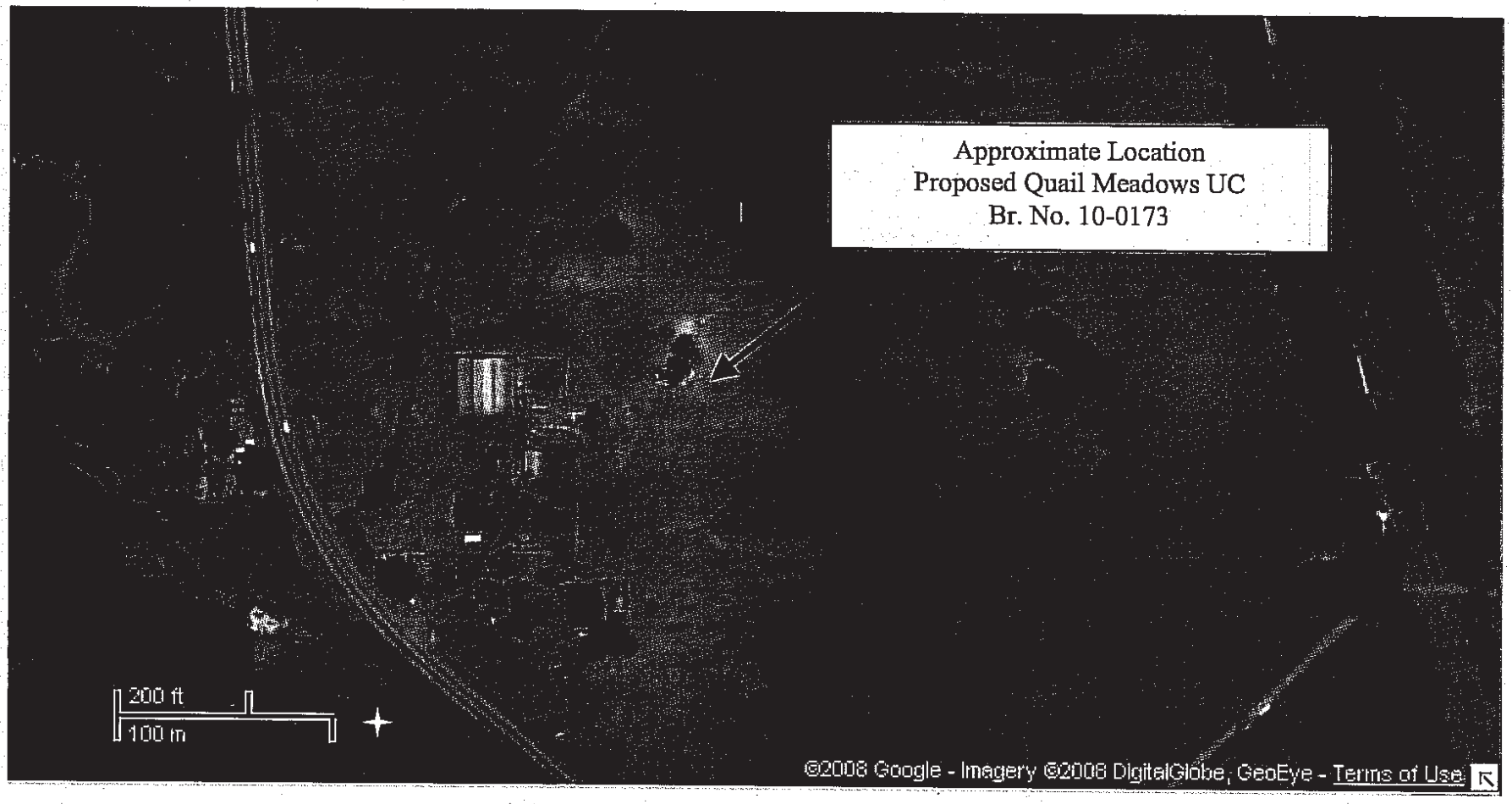
State of California, Department of Transportation (Caltrans):

- Standard Plans, July 2004 and May 2006
- Standard Specifications, July 2004 and May 2006
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

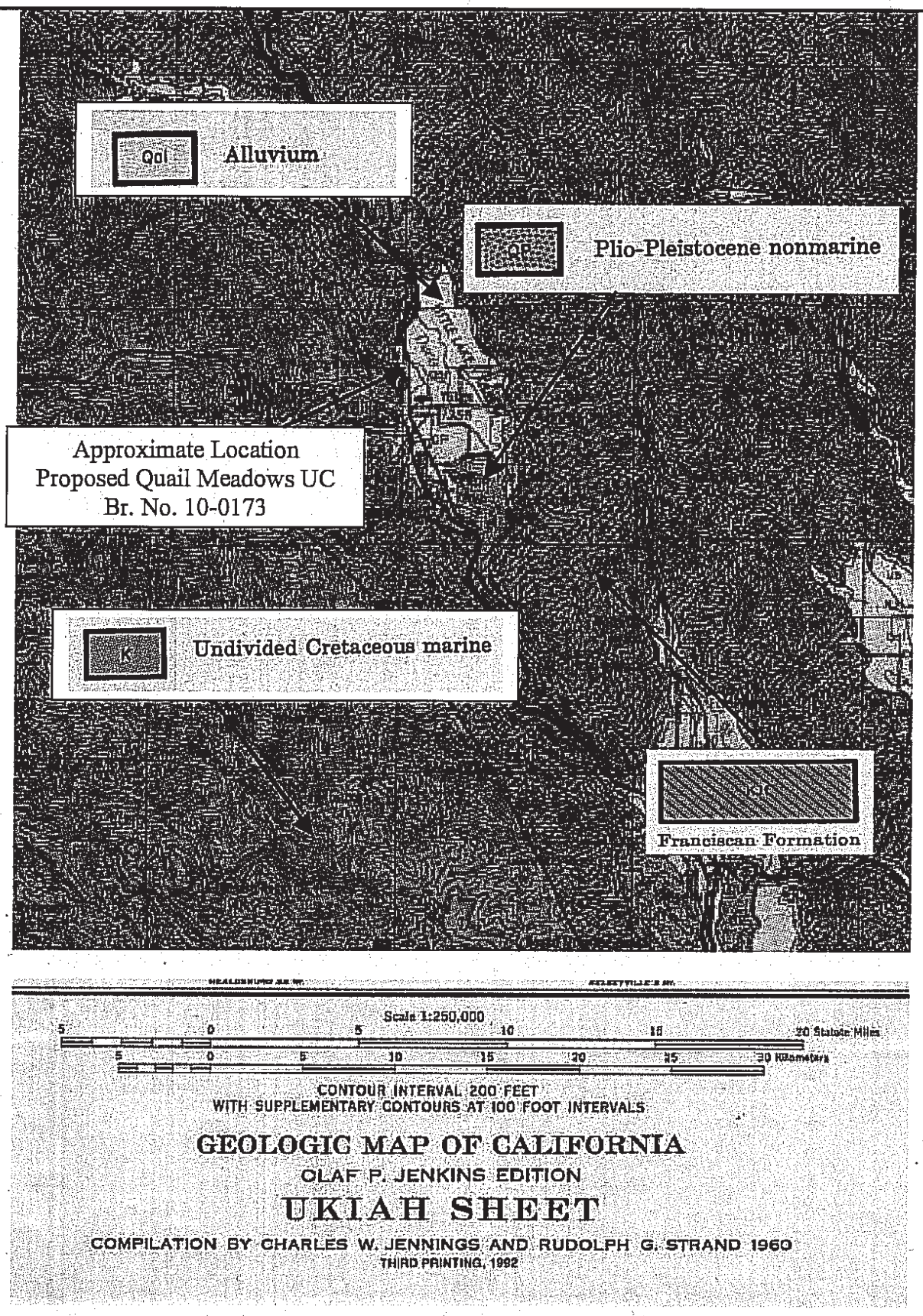
Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 January 15, 2009	Location Map
	01-MEN-101 KP R77.54/PM R48.18 **Quail Meadows UC, Br. No. 10-0173**	Figure 1



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 January 15, 2009	Location Map
	01-MEN-101 KP R77.54/PM R48.18 **Quail Meadows UC, Br. No. 10-0173**	Figure 2



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 January 15, 2009	Geologic Map
	01-MEN-101 KP R77.54/PM R48.18 **Quail Meadows UC, Br. No. 10-0173**	Figure 3

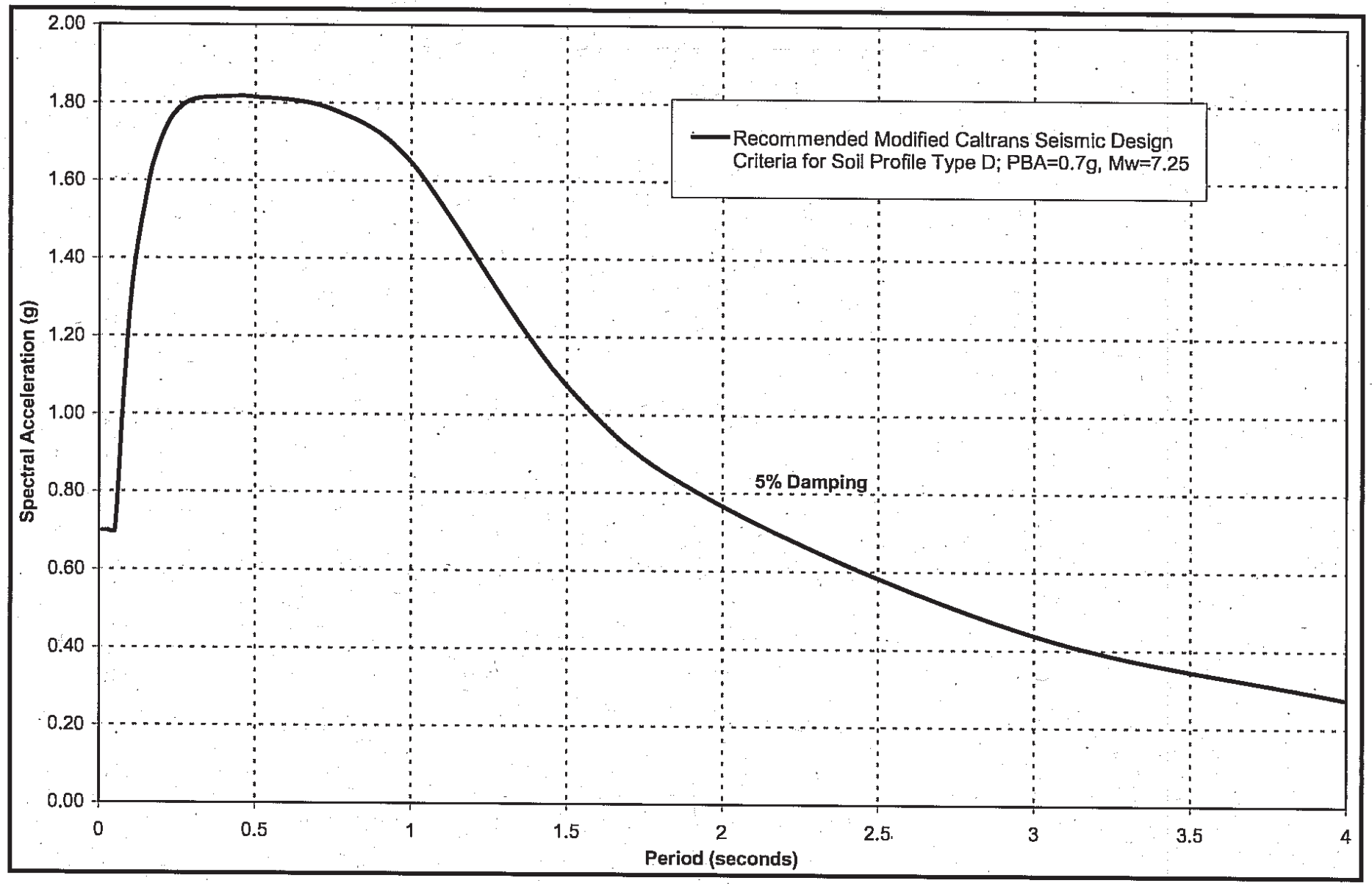


Figure 4. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: January 15, 2009

File: 01-MEN-101-KP R76.807/PM R47.73
EA#01-262001
Quail Meadows OH
Br. No. 10-0171

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Quail Meadows Overhead (Br. No. 10-0171). The Office of Geotechnical Design North, Branch A completed a subsurface investigation from August 2004 to September 2006 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (August 2004 to September 2006) along with a review of the available General Plan dated February 19, 2008. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Quail Meadows Overhead (Br. No. 10-0171) will consist of one new structure and will be incorporated as part of the Willits Bypass (Figures 1 and 2). The structure will be a single span prestressed cast-in-place box girder (4 cell) type structure with seat type abutments and wingwalls supported on driven steel H-Piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate

ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the north end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium, recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary deposits) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Quatenary alluvium. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation from August 2004 to September 2006.

The subsurface investigation consisted of seven mud rotary borings (Nos. B-15, B-16, B-20, B-57, B-73, B-75 and B-76) and two cone penetrometer soundings (C-14 & C-19). In addition, borehole geophysical measurements were acquired in Boring B-73. Geophysical measurements taken in B-73 included natural gamma, formation conductivity and resistivity, and P-S Suspension log records. The mud rotary borings were advanced using a self-casing wireline-diamond coring method extending down to a maximum depth of 70.10 m (230.0 ft). The equipment used to drill borings B-15 consisted of a CS500 drill rig (CME) equipped with a safety hammer. The equipment used to drill borings B-16 and B-57 consisted of a Mobile B-80 drill rig equipped with a safety hammer. The equipment used to drill boring B-20 consisisted of a Mobile B-47 drill rig equipped with a safety hammer. The equipment used to drill borings B-73, B-75 and B-76 consisted of a CS2000 Drill Rig (CME) drill rig equipped with a Diedrich automatic hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler at 1.5-meter intervals. The cone penetrometer sounding was used to supplement the subsurface investigation. Selected soil samples were bagged for laboratory testing. It should be noted that several of the borings and soundings listed above were intended for use in the design of three other bridge structures which were subsequently eliminated from this project.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed overhead (Br. No. 10-0171) generally consist of 22.3 m (73 ft.) to 25.3 m (83 ft.) of very soft to very stiff clay, loose to medium dense clayey sand and gravel with clay underlain by stiff to very stiff clay and medium dense to very dense fine sand, clayey sand & clayey gravel. The foundation materials were encountered down to the maximum depth explored of 70.10 m (230.0 ft.), an elevation of 337.15 m (1106.1 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels were measured during the August 2004 to September 2006 field investigation at elevation 402.95 m (1322.02 ft.) in cone penetrometer sounding C-19 and elevations 404.02 m (1325.51 ft.), 403.28 m (1323.10 ft.) and 404.45 m (1326.94 ft.) in Borings B-15, B-57 and B-75, respectively. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Corrosion Evaluation

Composite soil samples were collected in two of the borings during the August 2004 to September 2006 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for Quail Meadows OC (Br. No. 10-0171)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639860	B-15	0.30 - 1.52	6.00	3900	N/A	N/A
C639861	B-15	4.57 - 6.10	7.40	2400	N/A	N/A
C639910	B-75	3.05 – 4.57	6.25	3000	N/A	N/A
C639911	B-75	6.55 - 7.32	7.44	6900	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Moisture Content, Specific Gravity, Consolidation, Triaxial (Unconsolidated-Undrained) and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results are available upon request.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 0.6 kilometer southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Based on the boring logs along with availability of shear wave velocity, undrained shear, and Atterberg limits, a final Caltrans Seismic Design Criteria (CSDC) Acceleration Response Spectrum (ARS) Curve corresponding to soil profile Type D is recommended for design. Please note that due to the close proximity of this structure to the fault, we have performed a second modification to the CSDC ARS curve (see Figure 4). The modification is such that there is no increase in spectral accelerations (SA) for periods less than 0.5 second, and a 20% increase in SA for periods greater than 1 second. Between the periods of 0.5 and 1 second, a linear interpolation was used to estimate the SA.

Fill Settlement

Due to the presence of compressible clay soils beneath the proposed approach fills at Abutment 1 and Abutment 2, an 825 day waiting period is to be required after completion of the approach fills and before installing the steel "H" piles for either the abutments or

associated abutment retaining walls. Settlement will take place during and after the construction of the approach fills. It has been estimated that the maximum total settlement of the approach fills will be about 500 millimeters.

It has been calculated that 90% of the settlement of the approach fills will take place approximately 825 days after placement of the fill is completed. If it is necessary to accelerate the settlement, mitigation measures such as placement of additional surcharge fill and/or the use of wick drains may be recommended. Details of these measures will be included in the Geotechnical Design Report to be prepared by this Office.

Regardless of the target settlement or the estimated settlement time period, 500 mm and 825 days, respectively, the rate of settlement obtained during the settlement monitoring will determine if the majority of the settlement in the fill is complete. That is if the settlement rate is noticeably reduced sooner than the end of the estimated settlement time period, then it would be acceptable for construction of the abutment and retaining wall foundations to commence. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

Foundation Recommendations

The following foundation recommendations are for the new proposed Quail Meadows OH (Br. No. 10-0171).

At Abutment 1 and Abutment 2 support locations, driven steel HP 360X132 "H" piles are recommended for support. The specified pile tip elevations, shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed Quail Meadows OH, (Br. No. 10-0171) at Abutment 1 and 2 locations.

Location	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
			Compression kN	Tension kN			
Abutment 1	HP 360X132	625	1250	0	405.620 (1330.77)	380.0 (1) (1246.7)	380.0 (1246.7)
Abutment 2	HP 360X132	625	1250	0	405.880 (1331.62)	379.25 (1) (1244.3)	379.25 (1244.3)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

Retaining Wall Foundations

The following foundation recommendations are for the proposed Type 1 Retaining Wall structures located at Abutment 1 and Abutment 2 as shown on Abutment 1 Right Retaining Wall Layout 1 and Abutment 1 Left Retaining Wall Layout 1 sheets dated December 19, 2008 and December 16, 2008, respectively. At these locations, it is recommended driven steel HP 250X62 "H" piles be used to support the retaining wall segments. The specified pile tip elevations, shown below in Tables 3, 4, 5 and 6 will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 3. Pile Data Table for the new proposed Type 1 Retaining Wall Segment – Abutment 1 Right (Br. No. 10-0171).

Location	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 1	10300	HP 250X62	400	800	0	407.400 (1336.61)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	9100	HP 250X62	400	800	0	408.975 (1341.78)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	7900	HP 250X62	400	800	0	410.395 (1346.44)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	6700	HP 250X62	400	800	0	411.760 (1350.92)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	5500	HP 250X62	400	800	0	413.030 (1355.09)	380.50 (1) (1248.36)	380.50 (1248.36)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

Table 4. Pile Data Table for the new proposed Type 1 Retaining Wall Segment – Abutment 1 Left (Br. No. 10-0171).

Location	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 1	10900	HP 250X62	400	800	0	406.860 (1334.84)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	9700	HP 250X62	400	800	0	408.280 (1339.50)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	8500	HP 250X62	400	800	0	409.850 (1344.65)	380.50 (1) (1248.36)	380.50 (1248.36))
Abutment 1	7300	HP 250X62	400	800	0	411.270 (1349.31)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	6100	HP 250X62	400	800	0	412.590 (1353.64)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	4800	HP 250X62	400	800	0	413.810 (1357.64)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	3600	HP 250X62	400	800	0	415.080 (1361.81)	380.50 (1) (1248.36)	380.50 (1248.36)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

Table 5. Pile Data Table for the new proposed Type 1 Retaining Wall Segment – Abutment 2 Right (Br. No. 10-0171).

Location	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 2	10900	HP 250X62	400	800	0	406.420 (1333.40)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 2	9700	HP 250X62	400	800	0	407.785 (1337.88)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 2	8500	HP 250X62	400	800	0	409.360 (1343.04)	379.25 (1) (1244.25)	379.25 (1244.25))
Abutment 2	7300	HP 250X62	400	800	0	410.775 (1347.69)	379.25 (1) (1244.25)	379.25 (1244.25))
Abutment 2	6100	HP 250X62	400	800	0	412.095 (1352.02)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 2	4800	HP 250X62	400	800	0	413.315 (1356.02)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 2	3600	HP 250X62	400	800	0	414.585 (1360.19)	379.25 (1) (1244.25)	379.25 (1244.25)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

Table 6. Pile Data Table for the new proposed Type 1 Retaining Wall Segment – Abutment 2 Left (Br. No. 10-0171).

Location	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 1	10300	HP 250X62	400	800	0	407.690 (1337.57)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 1	9100	HP 250X62	400	800	0	409.265 (1342.73)	379.25 (1) (1244.25)	379.2 5 (1244.25)
Abutment 1	7900	HP 250X62	400	800	0	410.685 (1347.39)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 1	6700	HP 250X62	400	800	0	412.050 (1351.87)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 1	5500	HP 250X62	400	800	0	413.320 (1356.04)	379.25 (1) (1244.25)	379.25 (1244.25)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to

meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Structural Foundations shall be contacted for further recommendations.

3. Specified pile tip elevations shown in Tables 2, 3, 4, 5, and 6 above are based on preloading of compressible clay soils achieved by placement of approach/retaining wall fills prior to construction of the proposed abutment foundations. If preloading is not performed, the Office of Geotechnical Design-North is to be contacted for revised foundation recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. Settlement shall be monitored. Monitoring devices shall consist of settlement platforms placed at the bottom of the approach/retaining wall fill embankments at Abutments 1 and 2. Settlement shall be monitored. Monitoring devices shall consist of settlement platforms placed at the bottom of the approach fill embankments at Abutments 1 and 2. A minimum of six settlement devices (three or more beneath each approach fill) shall be placed beneath the centerline of embankments at Abutments 1 and 2 at locations to be determined by the Engineer.

3. Due to the presence of compressible clay soils beneath the proposed abutments and retaining walls, the abutment and retaining wall locations shall be preloaded. Preloading shall be achieved by placement of approach/retaining wall fill embankments prior to construction of the Quail Meadows Overcrossing. An 825-day waiting period is required prior to installation of piles at Abutments 1 and 2 and prior to installation of piles at Abutment 1 and 2 retaining walls. The fill settlement shall be monitored during the waiting period. If it is determined that the settlement has ceased prior to the end of the 825-day waiting period, with the written approval of the Engineer, the waiting period may be terminated at that time. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

4. At Abutment 1 Left and Right Retaining Wall locations, driven steel HP 250X62 "H" piles shall be driven through the new fill embankment in oversized pre-drilled

holes (where applicable) that conform to the Standard Specifications in Section 49-1.06, "Predrilled Holes." The predrilled holes are not to extend below the elevation 407.75 m (1337.8 ft).

5. At Abutment 2 Left and Right Retaining Wall locations, driven steel HP 250X62 "H" piles shall be driven through the new fill embankment in oversized pre-drilled holes (where applicable) that conform to the Standard Specifications in Section 49-1.06, "Predrilled Holes." The predrilled holes are not to extend below the elevation 407.00 m (1335.3 ft).

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035, or Reid Buell (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard Special Provisions 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:

A. Log of Test Borings for Quail Meadows OH, Bridge No. 10-0171.

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

A. Foundation Report for Quail Meadows OH, Bridge No. 10-0171, dated January 15, 2009.

Report by:

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Engineering Geologist
Office of Geotechnical Design-North
Structure Foundations Branch

Report by:

REZA MAHALLATI
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Reid Buell
R.E. Pending
Structure OE

REFERENCES

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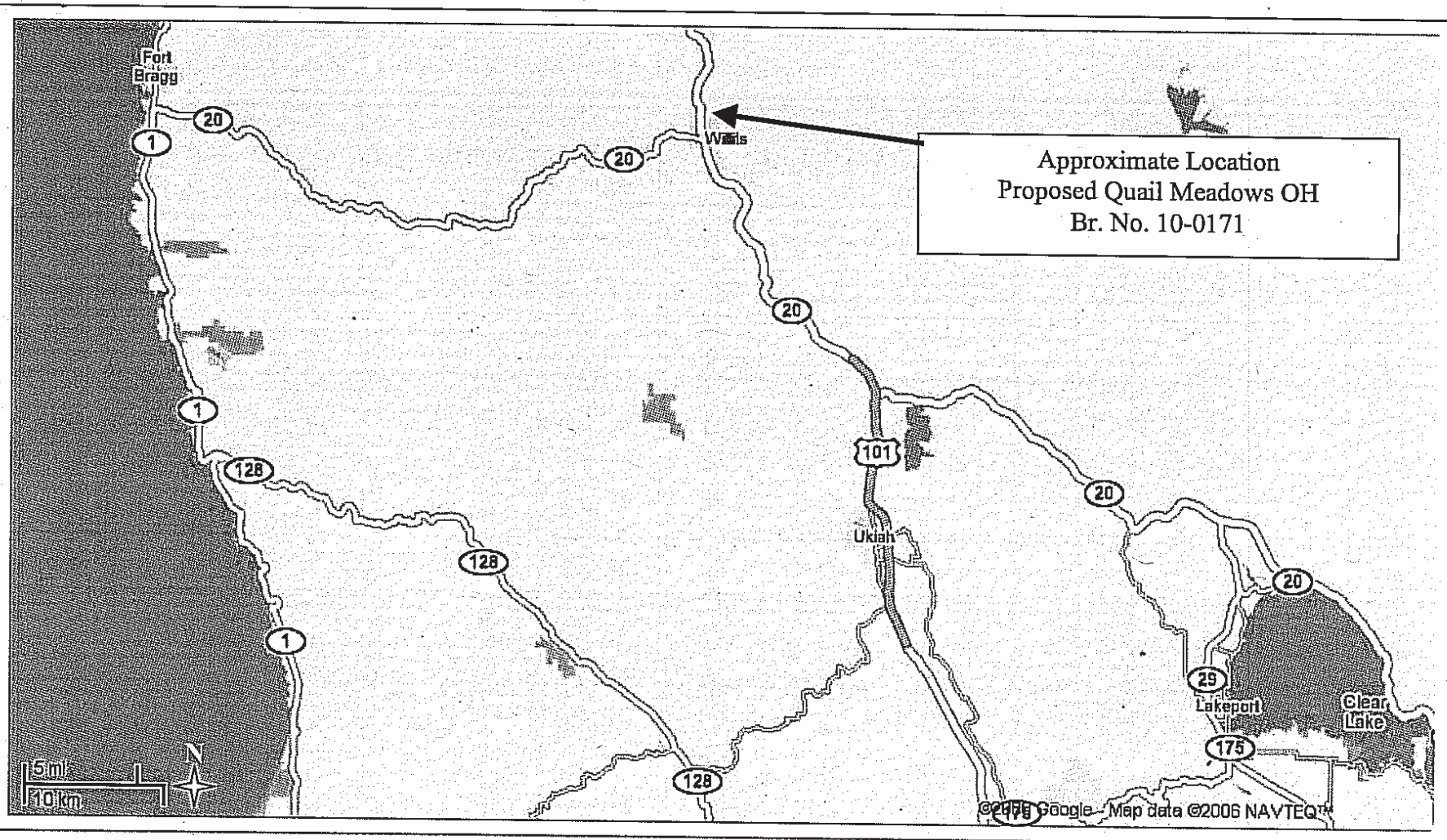
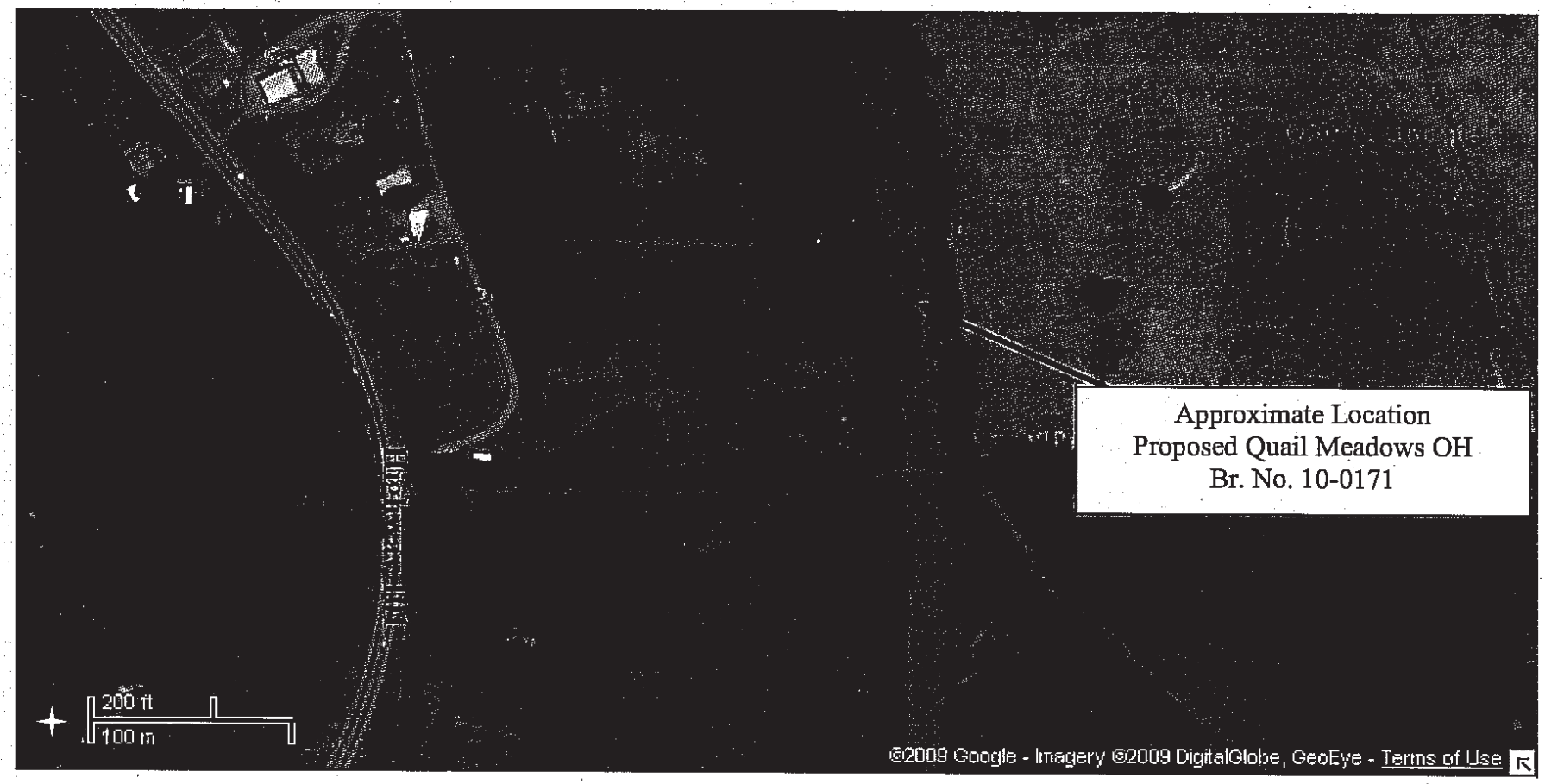
State of California, Department of Transportation (Caltrans):

- Standard Plans, July 2004 and May 2006
- Standard Specifications, July 2004 and May 2006
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

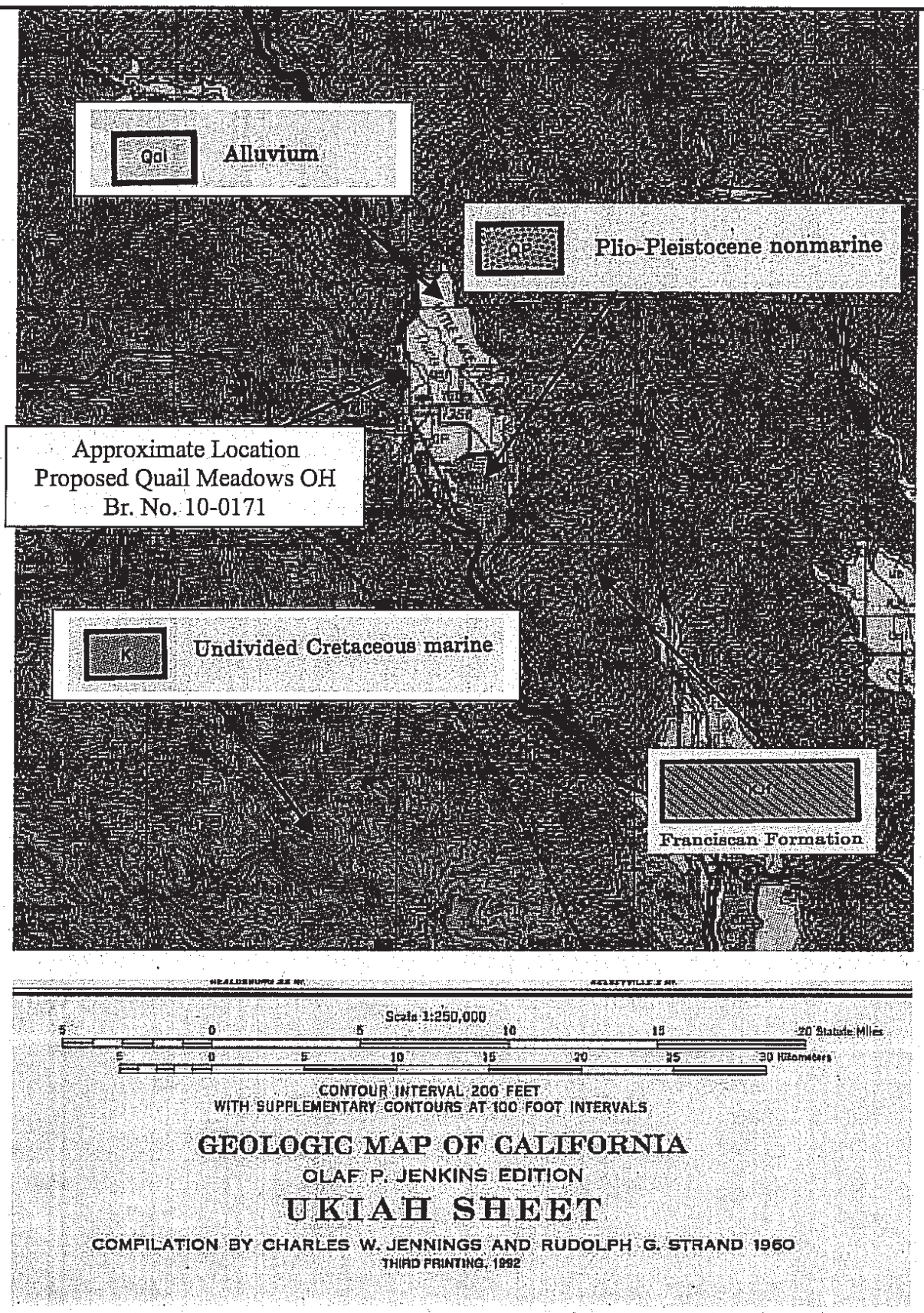
Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 January 15, 2009	Location Map
	01-MEN-101 KP R76.81/PM R47.73 **Quail Meadows OH, Br. No. 10-0171**	Figure 1



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 January 15, 2009	Location Map
	01-MEN-101 KP R76.81/PM R47.73 **Quail Meadows OH, Br. No. 10-0171**	Figure 2



Caltrans

Division of Engineering Services
Geotechnical Services
Geotechnical Design – North

EA: 01-262001	Geologic Map
January 15, 2009	
01-MEN-101 KP R76.81/PM R47.73 **Quail Meadows OH, Br. No. 10-0171**	Figure 3

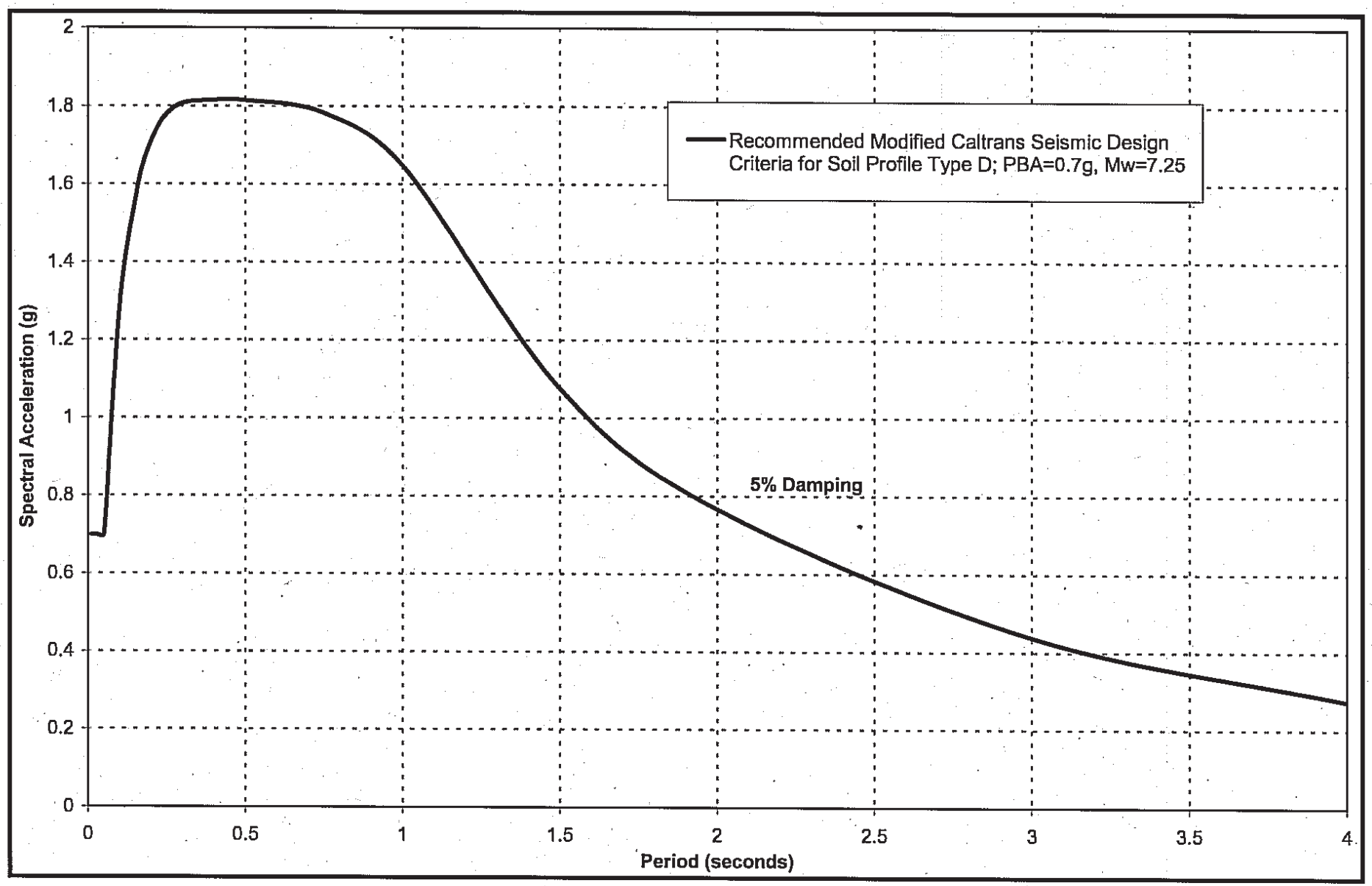


Figure 4. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: June 19, 2009

File: 01-MEN-101-KP R76.807/PM R47.73
EA#01-262001
Quail Meadows OH
Br. No. 10-0171

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Revised Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Quail Meadows Overhead (Br. No. 10-0171). Following the completion of the Foundation Report dated January 15, 2009, modifications were made to the bridge and retaining walls. This report supercedes the previously sent report.

The Office of Geotechnical Design North, Branch A completed a subsurface investigation from August 2004 to September 2006 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (August 2004 to September 2006) along with a review of the available General Plan dated February 19, 2008. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Quail Meadows Overhead (Br. No. 10-0171) will consist of one new structure and will be incorporated as part of the Willits Bypass (Figures 1 and 2). The structure will be a single span prestressed cast-in-place box girder (4 cell) type structure with seat type abutments and wingwalls supported on driven steel H-Piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the north end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium, recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary deposits) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Quatenary alluvium. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation from August 2004 to September 2006.

The subsurface investigation consisted of seven mud rotary borings (Nos. B-15, B-16, B-20, B-57, B-73, B-75 and B-76) and two cone penetrometer soundings (C-14 & C-19). In addition, borehole geophysical measurements were acquired in Boring B-73. Geophysical measurements taken in B-73 included natural gamma, formation conductivity and resistivity, and P-S Suspension log records. The mud rotary borings were advanced using a self-casing wireline drilling method extending down to a maximum depth of 70.10 m (230.0 ft). The equipment used to drill borings B-15 consisted of a CS500 drill rig (CME) equipped with a safety hammer. The equipment used to drill borings B-16 and B-57 consisted of a Mobile B-80 drill rig equipped with a safety hammer. The equipment used to drill boring B-20 consisisted of a Mobile B-47 drill rig equipped with a safety hammer. The equipment used to drill borings B-73, B-75 and B-76 consisted of a CS2000 Drill Rig (CME) drill rig equipped with a Diedrich automatic hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler at 1.5-meter intervals. The cone penetrometer sounding was used to supplement the subsurface investigation. Selected soil samples were bagged for laboratory testing. It should be noted that several of the borings and soundings listed above were intended for use in the design of three other bridge structures which were subsequently eliminated from this project.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed overhead (Br. No. 10-0171) generally consist of 22.3 m (73 ft.) to 25.3 m (83 ft.) of very soft to very stiff clay, loose to medium dense clayey sand and gravel with clay underlain by stiff to very stiff clay and medium dense to very dense fine sand, clayey sand & clayey gravel. The foundation materials were encountered down to the maximum depth explored of 70.10 m (230.0 ft.), an elevation of 337.15 m (1106.1 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels varied from elevation 402.95 m (1322.02 ft.) in cone penetrometer sounding C-19 to elevation 405.57 m (1330.61 ft.) in Boring B-20 during the August 2004 to September 2006 field investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Corrosion Evaluation

Composite soil samples were collected in six of the borings during the August 2004 to September 2006 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for Quail Meadows OC (Br. No. 10-0171)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639860	B-15	0.30 - 1.52	6.00	3900	N/A	N/A
C639861	B-15	4.57 - 6.10	7.40	2400	N/A	N/A
C639910	B-75	3.05 – 4.57	6.25	3000	N/A	N/A
C639911	B-75	6.55 - 7.32	7.44	6900	N/A	N/A
C640630	B-16	3.05 – 4.57	5.68	1900	N/A	N/A
C639855	B-20	1.52 – 3.05	6.73	8600	N/A	N/A
C639854	B-20	0 – 1.52	6.01	4500	N/A	N/A
C639909	B-73	0.61 – 1.52	5.61	4900	N/A	N/A
C639912	B-76	1.52 – 3.05	5.88	2800	N/A	N/A
C639913	B-76	8.23 – 9.14	7.90	5100	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Moisture Content, Specific Gravity, Consolidation, Triaxial (Unconsolidated-Undrained) and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results are available upon request.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault

is located approximately 0.6 kilometer southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Based on the boring logs along with availability of shear wave velocity, undrained shear, and Atterberg limits, a final Caltrans Seismic Design Criteria (CSDC) Acceleration Response Spectrum (ARS) Curve corresponding to soil profile Type D is recommended for design. Please note that due to the close proximity of this structure to the fault, we have performed a second modification to the CSDC ARS curve (see Figure 4). The modification is such that there is no increase in spectral accelerations (SA) for periods less than 0.5 second, and a 20% increase in SA for periods greater than 1 second. Between the periods of 0.5 and 1 second, a linear interpolation was used to estimate the SA.

Fill Settlement

Due to the presence of compressible clay soils beneath the proposed approach fills at Abutment 1 and Abutment 2, a settlement waiting period is to be required after completion of the approach fills and before installing the steel "H" piles for either the abutments or associated abutment retaining walls. Settlement will take place during and after the construction of the approach fills. Settlement on the order of 500 mm is anticipated.

Due to the extent of compressible clay soils which may result in excessive settlement beneath the proposed approach fills at Abutment 1 and Abutment 2, it is anticipated that additional mitigation measures such as placement of additional surcharge and/or the use of wick drains will be required. Without these measures, the duration of the predicted settlement is estimated to be greater than 2 years. The intent of the surcharge and/or use of wick drains is to decrease the settlement waiting period to less than one-year. Details of these measures will be included in the Geotechnical Design Report to be prepared by this Office. Any questions regarding fill settlement, settlement waiting periods and/or proposed mitigation measures should be directed to John Huang at (916) 227-1037 or Ben Barnes at (916) 227-1039, of the Office of Geotechnical Design-North, Branch E.

Regardless of the target settlement or the estimated settlement time period, the rate of settlement obtained during the settlement monitoring will determine if primary consolidation settlement is complete. That is, if the settlement rate is noticeably reduced sooner than the end of the estimated settlement time period, then it would be acceptable for construction of the abutment foundations to commence. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

Foundation Recommendations

The following foundation recommendations are for the new proposed Quail Meadows OH (Br. No. 10-0171) as shown on the Foundation Plan sheet dated February 18, 2009.

At Abutment 1 and Abutment 2 support locations, driven steel HP 360X132 "H" piles are recommended for support. The specified pile tip elevations, shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed Quail Meadows OH, (Br. No. 10-0171) at Abutment 1 and 2 locations.

Location	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
			Compression kN	Tension kN			
Abutment 1	HP 360X132	625	1250	0	405.910 (1331.73)	380.0 (1) (1246.7)	380.0 (1246.7)
Abutment 2	HP 360X132	625	1250	0	406.165 (1332.56)	379.25 (1) (1244.3)	379.25 (1244.3)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

Retaining Wall Foundations

The following foundation recommendations are for the proposed Type 1 Retaining Wall structures located at Abutment 1 and Abutment 2 as shown on Retaining Wall 0171A Layout sheet dated March 11, 2009, Retaining Wall 0171B Layout sheet dated March 11, 2009 and the Foundation Plan sheet dated February 18, 2009. At these locations, it is recommended driven steel HP 250X62 "H" piles be used to support the retaining wall segments. The specified pile tip elevations, shown below in Tables 3, 4, 5 and 6 will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 3. Pile Data Table for the new proposed Type 1 Retaining Wall Segment (0171A) – Abutment 1 Right (Br. No. 10-0171).

Location	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 1	10300	HP 250X62	400	800	0	407.165 (1335.84)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	9100	HP 250X62	400	800	0	408.635 (1340.67)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	7900	HP 250X62	400	800	0	410.054 (1345.32)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	6700	HP 250X62	400	800	0	411.422 (1349.81)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	5500	HP 250X62	400	800	0	412.692 (1353.98)	380.50 (1) (1248.36)	380.50 (1248.36)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

Table 4. Pile Data Table for the new proposed Type 1 Retaining Wall Segment (0171B) – Abutment 1 Left (Br. No. 10-0171).

Location	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 1	10900	HP 250X62	400	800	0	406.665 (1334.20)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	9700	HP 250X62	400	800	0	408.365 (1339.78)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	8500	HP 250X62	400	800	0	409.835 (1344.60)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	7300	HP 250X62	400	800	0	411.255 (1349.26)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	6100	HP 250X62	400	800	0	412.575 (1353.59)	380.50 (1) (1248.36)	380.50 (1248.36)
Abutment 1	4800	HP 250X62	400	800	0	413.795 (1357.60)	380.50 (1) (1248.36)	380.50 (1248.36)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

Table 5. Pile Data Table for the new proposed Type 1 Retaining Wall Segment (0171C) – Abutment 2 Right (Br. No. 10-0171).

Location	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 2	10900	HP 250X62	400	800	0	406.150 (1332.51)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 2	9700	HP 250X62	400	800	0	407.850 (1338.09)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 2	8500	HP 250X62	400	800	0	409.320 (1342.91)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 2	7300	HP 250X62	400	800	0	410.740 (1347.57)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 2	6100	HP 250X62	400	800	0	412.060 (1351.90)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 2	4800	HP 250X62	400	800	0	413.280 (1355.91)	379.25 (1) (1244.25)	379.25 (1244.25)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

Table 6. Pile Data Table for the new proposed Type 1 Retaining Wall Segment (0171D) – Abutment 2 Left (Br. No. 10-0171).

Location	Retaining Wall Height mm	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 1	10300	HP 250X62	400	800	0	407.655 (1337.45)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 1	9100	HP 250X62	400	800	0	409.091 (1342.16)	379.25 (1) (1244.25)	379.2 5 (1244.25)
Abutment 1	7900	HP 250X62	400	800	0	410.543 (1346.93)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 1	6700	HP 250X62	400	800	0	411.913 (1351.42)	379.25 (1) (1244.25)	379.25 (1244.25)
Abutment 1	5500	HP 250X62	400	800	0	413.182 (1355.58)	379.25 (1) (1244.25)	379.25 (1244.25)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Structural Foundations shall be contacted for further recommendations.

3. Specified pile tip elevations shown in Tables 2, 3, 4, 5, and 6 above are based on preloading of compressible clay soils achieved by placement of approach/retaining wall fills prior to construction of the proposed abutment foundations. If preloading is not performed, the Office of Geotechnical Design-North is to be contacted for revised foundation recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. Settlement shall be monitored. Monitoring devices shall consist of settlement platforms placed at the bottom of the approach/retaining wall fill embankments at Abutments 1 and 2. A minimum of six settlement devices (three or more beneath each approach fill) shall be placed beneath the centerline of embankments at Abutments 1 and 2 at locations to be determined by the Engineer. Abutment construction shall not start until primary consolidation is completed per field monitoring.

3. Due to the presence of compressible clay soils beneath the proposed abutment and retaining wall locations, the abutment and retaining wall locations shall be preloaded. Preloading shall be achieved by placement of approach/retaining wall fill embankments prior to construction of the Quail Meadows Overcrossing. A waiting period will be required prior to installation of piles at Abutments 1 and 2 and associated Abutment 1 and 2 retaining walls. Additional mitigation measures such as placement of additional surcharge and/or the use of wick drains may be required. Details of these measures can be found in the Geotechnical Design Report to be prepared by this Office. The fill settlement shall be monitored during the waiting period. If it is determined that the settlement has ceased prior to the end of the recommended waiting period, with the written approval of the Engineer, the waiting period may be terminated at that time. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes. Any questions regarding fill settlement, settlement waiting periods and/or proposed mitigation measures should be directed to John Huang at (916) 227-1037 or Ben Barnes at (916) 227-1039, of the Office of Geotechnical Design-North, Branch E.

4. At Abutment 1 Left and Right Retaining Wall locations, driven steel HP 250X62 "H" piles shall be driven through the new fill embankment in oversized pre-drilled holes (where applicable) that conform to the Standard Specifications in Section 49-1.06, "Predrilled Holes." The predrilled holes are not to extend below the elevation 407.75 m (1337.8 ft).

5. At Abutment 2 Left and Right Retaining Wall locations, driven steel HP 250X62 "H" piles shall be driven through the new fill embankment in oversized pre-drilled holes (where applicable) that conform to the Standard Specifications in Section 49-1.06, "Predrilled Holes." The predrilled holes are not to extend below the elevation 407.00 m (1335.3 ft).

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035, or Reid Buell (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard Special Provisions 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:

A. Log of Test Borings for Quail Meadows OH, Bridge No. 10-0171.

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

A. Revised Foundation Report for Quail Meadows OH, Bridge No. 10-0171, dated June 19, 2009.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North
Structure Foundations Branch

Report by:

REZA MAHALLATI
Senior Materials & Research Engineer
Office of Geotechnical Design-North

REGISTERED PROFESSIONAL ENGINEER
REZA MAHALLATI
No. 49374
Exp. 9/30/10
CIVIL
STATE OF CALIFORNIA

Supervised by:

REID BUELL, C.E.G. NO. 1481
Senior Engineering Geologist
Office of Geotechnical Design-North
Structure Foundations Branch

REGISTERED GEOLOGIST
REID L. BUELL
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CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

cc: OGDSN
GS File Room
Reid Buell
R.E. Pending
Structure OE

REFERENCES

Federal Highway Administration (FHWA), Design and Construction of Driven Pile Foundations: Workshop Manual – Volume 1 (Pub. No. FHWA HI-97-013) published January 1997, Revised November1998.

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State of California, Department of Transportation (Caltrans):

- Standard Plans, July 2004 and May 2006
- Standard Specifications, July 2004 and May 2006
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

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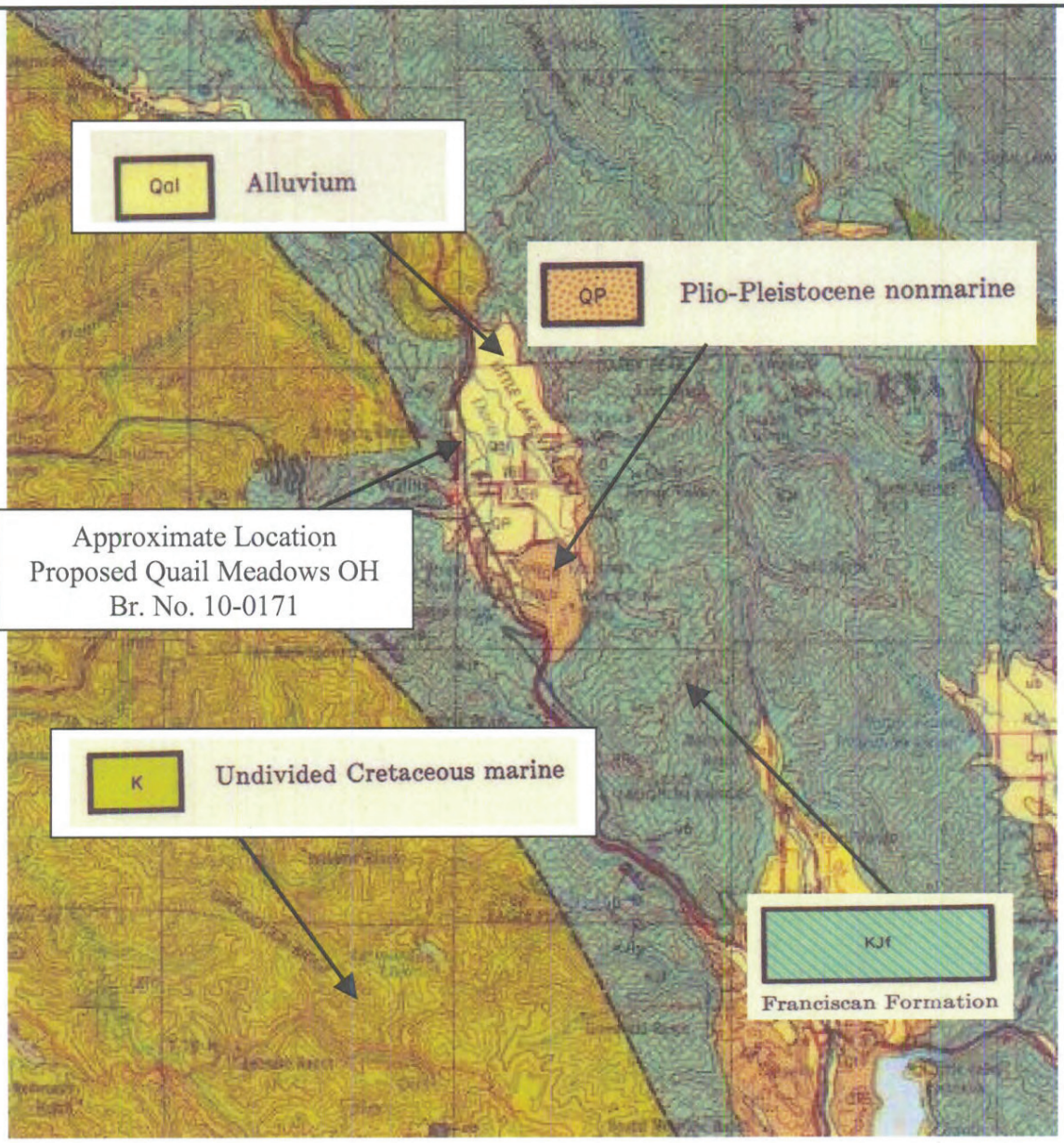
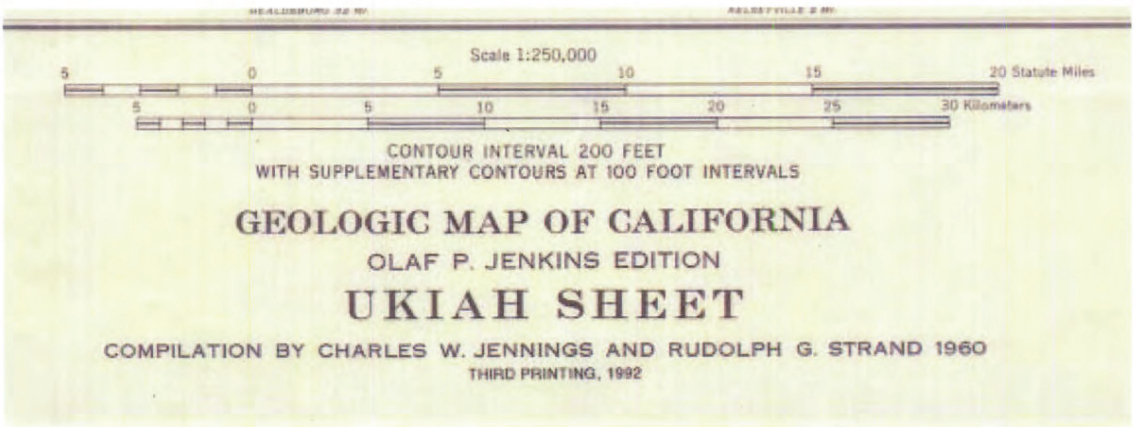
Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 June 19, 2009	Location Map
	01-MEN-101 KP R76.81/PM R47.73 **Quail Meadows OH, Br. No. 10-0171**	Figure 1

Caltrans

Division of Engineering Services
Geotechnical Services
Geotechnical Design – North

EA: 01-262001	Location Map
June 19, 2009	
01-MEN-101 KP R76.81/PM R47.73 **Quail Meadows OH, Br. No. 10-0171**	Figure 2





Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 June 19, 2009	Geologic Map
	01-MEN-101 KP R76.81/PM R47.73 **Quail Meadows OH, Br. No. 10-0171**	Figure 3

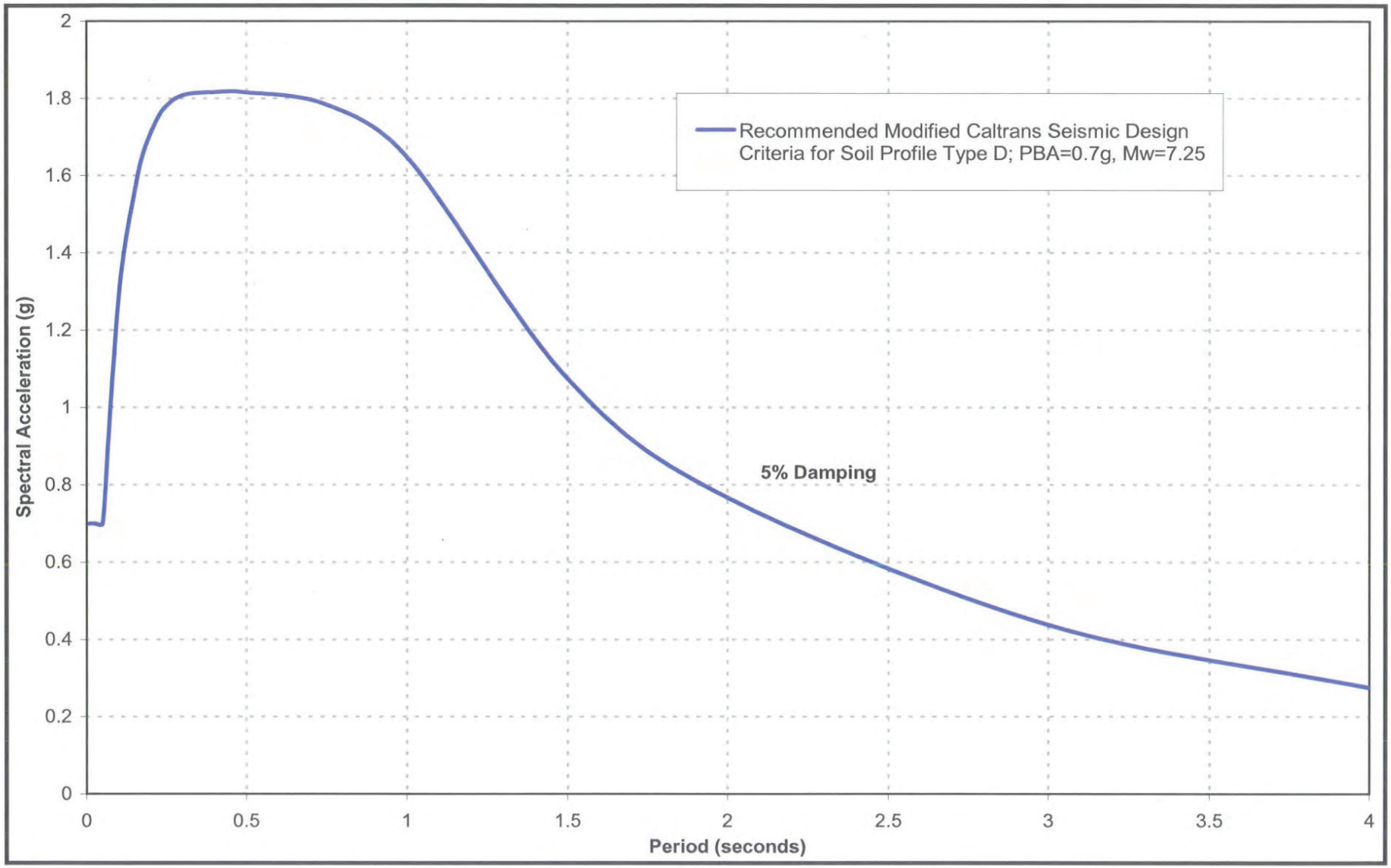


Figure 4. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: January 15, 2009

File: 01-MEN-101-KP R77.541/PM R48.18
EA#01-262001
Quail Meadows UC
Br. No. 10-0173

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Quail Meadows Undercrossing (Br. No. 10-0173). The Office of Geotechnical Design North, Branch A completed a subsurface investigation during September 2007 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (September 2007) along with a review of the available General Plan dated October 3, 2008. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Quail Meadows Undercrossing (Br. No. 10-0173) will consist of one new structure and will be incorporated as part of the Willits Bypass (Figures 1 and 2). The structure will be a single span prestressed cast-in-place box girder (3 cell) type structure with seat type abutments and wingwalls supported on driven steel H-Piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: November 9, 2006

File: 01-MEN-101-KP R70.58
EA#01-262001
Haehl Creek (NEW)
Br. No. 10-0129R/L

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Haehl Creek Bridge (Br. No. 10-0129R/L). The Office of Geotechnical Design North, Branch A completed a subsurface investigation between April and June 2004 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (April through June 2004) along with a review of the available General Plan dated November 7, 2005, Foundation Plan dated January 31, 2006 and the Abutment Details No. 1 through 5 dated December 7 and 9, 2005. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Haehl Creek Bridge (Br. No. 10-0129R/L) will be incorporated as part of the Willits Bypass Project. The proposed new bridge will consist of two separate new structures and will be located within the city limits of Willits in Mendocino County (Figures 1 and 2). The left and right structures will consist of single spans with prestressed cast-in-place box girder (3 cell) type structure with seat type abutments and wingwalls supported on driven steel H-Piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the south end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary deposits) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Plio-Pleistocene nonmarine sedimentary deposits identified as the Cache Formation. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation between April and June 2004.

The subsurface investigation consisted of four mud rotary borings (Nos. B-4, B-6, B-8, and B-9) and four cone penetrometer soundings (No. C-4, C-5, C-6, and C-8). The mud rotary borings were advanced using a self-casing wireline-diamond coring method extending down to a maximum depth of 30.9 m (101.5 ft). The equipment used to drill B-4 consisted of a Central Mine Equipment (CME) drill rig equipped with a CME automatic hammer. The equipment used to drill B-6 and B-8 consisted of an Acker drill rig equipped with a CME automatic hammer. The equipment used to drill B-9 consisted of a Mobile B-47 drill rig equipped with a safety hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler at 1.5-meter intervals. Selected soil samples were bagged for laboratory testing. The cone penetromer soundings were used to supplement the subsurface investigation.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed bridge (Br. No. 10-0129RL) generally consist of approximately 4.6 m (15.0 ft) to 13.1 m (43.0) of loose to very dense sand, clayey sand, silty sand and firm to hard clay overlying medium dense to very dense sand, sandy gravel, clayey sand and hard clay. The foundation materials extend from the ground surface to a maximum depth of 30.9 m (101.5 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels varied from elevation 443.6 m (1455.4 ft) in Boring B-4 and 441.6 m (1448.8 ft) in Boring B-8 during the April to June 2004 subsurface investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Scour Evaluation

A Final Hydraulic Report (dated March 22, 2005) for the Haehl Creek structures was completed by the Office of Structure Maintenance and Investigations, Structure Hydraulics Branch. According to this report, there are no local scour concerns for these structures since they are single span structures. The report recommends the abutment foundations be designed assuming no ground support (lateral and vertical) as a result of soil loss due to possible degradation or lateral stream migration down to the current stream thalwag elevation (lowest elevation in channel). The approximate thalwag elevation provided for the proposed right bridge (Br. No. 10-0129R) is 442.5 m (1451.8 ft). The approximate thalwag elevation provided for the proposed left bridge (Br. No. 10-0129L) is 438.5 m (1438.6 ft).

Rock Slope Protection (RSP) was mentioned and will be designed by the District to protect the roadway approach fills, if required.

For further information including site-specific scour assessment and mitigation measures, the Structures Hydraulics Branch should be contacted.

Corrosion Evaluation

Composite soil samples were collected during the subsurface investigation. The results will be forwarded once completed.

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request once the results are received.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault

is located approximately 1.0 kilometers (0.6 miles) southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g. The soil profile at the site may be classified as Type D, as defined in the Department's Seismic Design Criteria (SDC, 2006, Version 1.4). The recommended design Acceleration Response Spectrum (ARS) curve shown in Figure 4 was obtained by modifying the ARS curve in Figure B.8 of the SDC corresponding to a PBA of 0.7g. These modifications were introduced to account for the proximity of the controlling seismic source and included a 20% increase in spectral accelerations for periods greater than or equal to 1.0 second. No modifications were introduced for periods less than or equal to 0.5 second. The spectral accelerations for periods between 0.5 and 1.0 second were obtained by linear interpolation.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Foundation Recommendations

The following foundation recommendations are for the new proposed Haehl Creek Bridge (Br. No. 10-129R/L).

Haehl Creek Bridge (Br. No. 10-0129R)

At all structure support locations, driven steel HP 360X132 "H" piles are recommended for support at the right bridge. The specified pile tip elevations (SPTE), shown below in Table 1, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 1. Pile Data Table for the new proposed Haehl Creek Bridge (Br. No. 10-0129R).

Location	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
			Compression kN	Tension kN			
Abutment 1	HP 360X132	625	1250	0	445.5 (1461.6)	428.7 (1,2) (1406.6)	428.7 (1406.6)
Abutment 2	HP 360X132	625	1250	0	445.7 (1462.4)	428.7 (1,2) (1406.6)	428.7 (1406.6)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 442.5 m (1451.8 ft) at Abutments 1R and 2R.*

Wingwall Pedestal Foundation

The following foundation recommendations are for the proposed wingwall structure located at Abutment 1, right bridge location as shown on the foundation plan dated January 31, 2006. At this location, it is recommended driven steel HP 250X63 "H" piles be used to support this pedestal. The specified pile tip elevations, shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed Wingwall Pedestal – Abutment 1, Right Bridge (Br. No. 10-0129R).

Location	Wingwall Height mm (ft)	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 1R	1200	HP 250X63	400	800	0	453.0 (1486.2)	433.1(1) (1421.0)	433.1 (1421.0)

Note: Design Pile Tip Elevation is controlled by the following demand: (1) *Compression.*

Haehl Creek Bridge (Br. No. 10-0129L)

At all structure support locations, driven steel HP 360X132 "H" piles are recommended for support at the left bridge. The specified pile tip elevations (SPTE), shown below in Table 3, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 3. Pile Data Table for the new proposed Haehl Creek Bridge (Br. No. 10-0129L).

Location	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
			Compression kN	Tension kN			
Abutment 1	HP 360X132	625	1250	0	446.5 (1464.9)	426.7 (1,2) (1400.0)	426.7 (1400.0)
Abutment 2	HP 360X132	625	1250	0	444.0 (1456.7)	428.7 (1,2) (1406.7)	428.7 (1406.7)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 438.5 m (1438.6 ft) at Abutments 1L and 2L.*

Retaining Wall Foundation

The following foundation recommendations are for the proposed Type 1 Retaining Wall structure located at Abutment 2, left bridge location as shown on the Foundation Plan dated January 31, 2006. At this location, it is recommended driven steel HP 250X63 "H" piles be used to support these retaining wall segments. The specified pile tip elevations, shown below in Table 4, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 4. Pile Data Table for the new proposed Type 1 Retaining Wall Segment – Abutment 2, Left Bridge (Br. No. 10-0129L).

Location	Retaining Wall Height mm (ft)	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 2L	6700	HP 250X63	400	800	0	445.1 (1460.3)	431.0 (1,2) (1414.0)	431.0 (1414.0)
Abutment 2L	5500	HP 250X63	400	800	0	446.3 (1464.2)	431.0 (1,2) (1414.0)	431.0 (1414.0)
Abutment 2L	4200	HP 250X63	400	800	0	447.3 (1467.5)	431.0 (1,2) (1414.0)	431.0 (1414.0)
Abutment 2L	3000	HP 250X63	400	800	0	448.3 (1470.8)	431.0 (1,2) (1414.0)	431.0 (1414.0)
Abutment 2L	2400	HP 250X63	400	800	0	449.5 (1474.7)	431.0 (1,2) (1414.0)	431.0 (1414.0)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 438.5 m (1438.6 ft) at Abutment 2L Type 1 Retaining Wall Segment.*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Branch A shall be contacted for further recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. At the left bridge location, scour is predicted to extend to elevation 438.5 m (1438.6 ft). In order to achieve the specified pile tip elevation, predrilling is recommended for the installation of the steel "H" piles at the left bridge support locations. It is recommended the HP 360X132 piles (including battered) shall be driven in pre-drilled holes that conform to the Standard Specifications in Section 49-1.06, "Pre-drilled Holes." The predrilled holes are not to extend below the elevations shown in Table 5.

Table 5. Predrilled Elevations

Location	Predrilled Elevation
Abutment 1L	438.5 m (1438.6 ft)
Abutment 2L	438.5 m (1438.6 ft)

3. At the right bridge location, scour is predicted to extend to elevation 442.5 m (1451.8 ft). In order to achieve the specified pile tip elevation, predrilling may be necessary for the installation of the steel "H" piles at the left bridge support locations. It is recommended the HP 360X132 piles (including battered) shall be driven in pre-drilled holes that conform to the Standard Specifications in Section 49-1.06, "Pre-drilled Holes." The predrilled holes are not to extend below the elevations shown in Table 6.

Table 6. Predrilled Elevations

Location	Predrilled Elevation
Abutment 1R	442.5 m (1451.8 ft)
Abutment 2R	442.5 m (1451.8 ft)

4. The steel "H" pile section below the scour elevation has been designed to develop the required nominal resistance at all support locations.

5. Specialty equipment may be required for installation of the battered steel "H" piles. The contractor should be prepared to alternate from battered to vertical installation methods of the steel "H" piles.

6. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

7. The contractor should anticipate hard and erratic driving of the steel "H" piles due to the presence of very dense foundation material. The contractor should anticipate field cutting and splicing of the steel "H" piles. Refer to the LOTB sheets for details.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-7260, or Reid Buell (916) 227-7190, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard special Provisions 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:

A. Log of Test Borings for Haehl Creek Bridge, Bridge No. 10-0129R/L.

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

A. Foundation Report for Haehl Creek Bridge, Bridge No. 10-0129R/L, dated November 9, 2006.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North

Report by:

JACQUELINE MARTIN
Engineering Geologist
Office of Geotechnical Design-North

Supervised by:

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cc: OGDSN
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R.E. Pending
Structure OE

REFERENCES

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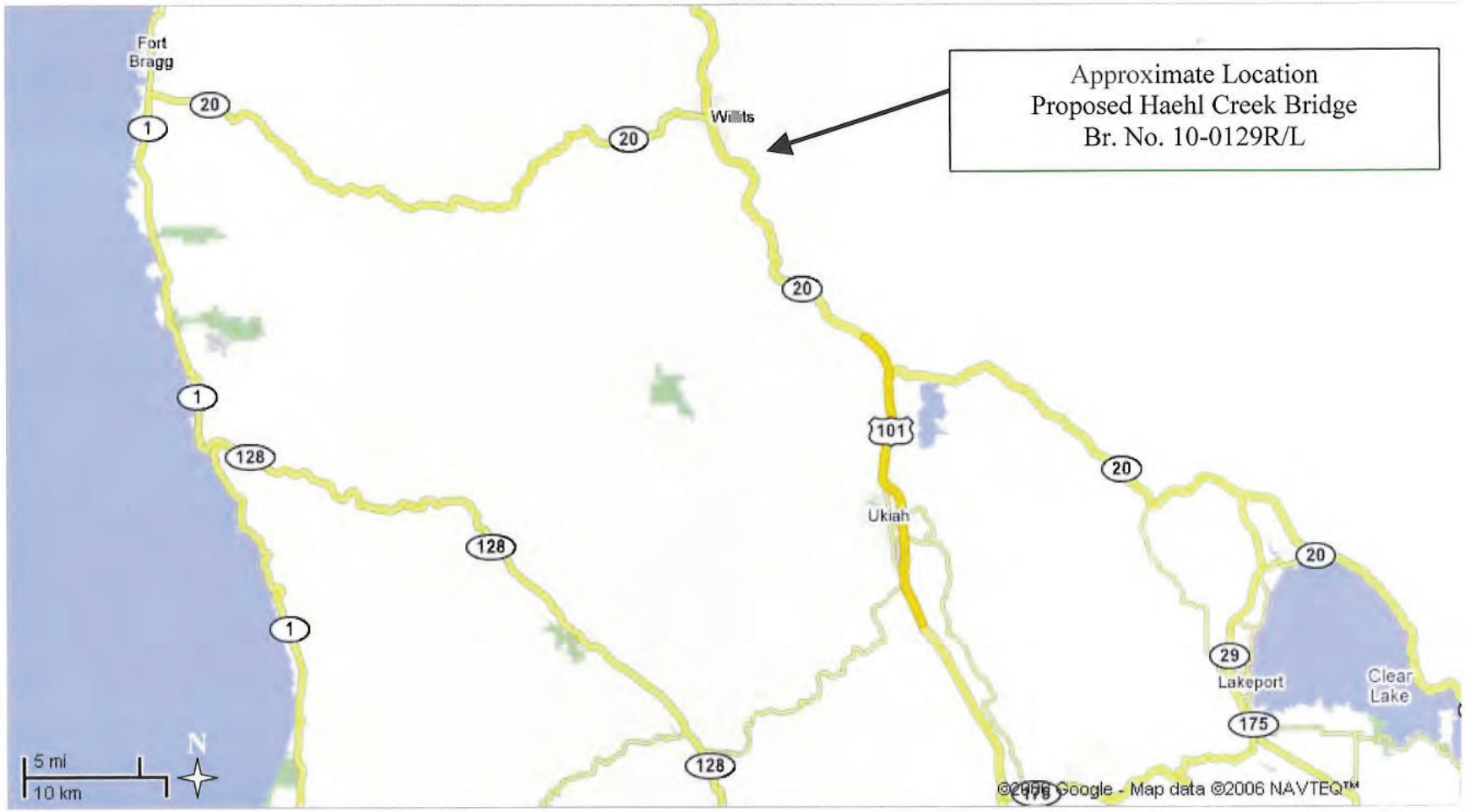
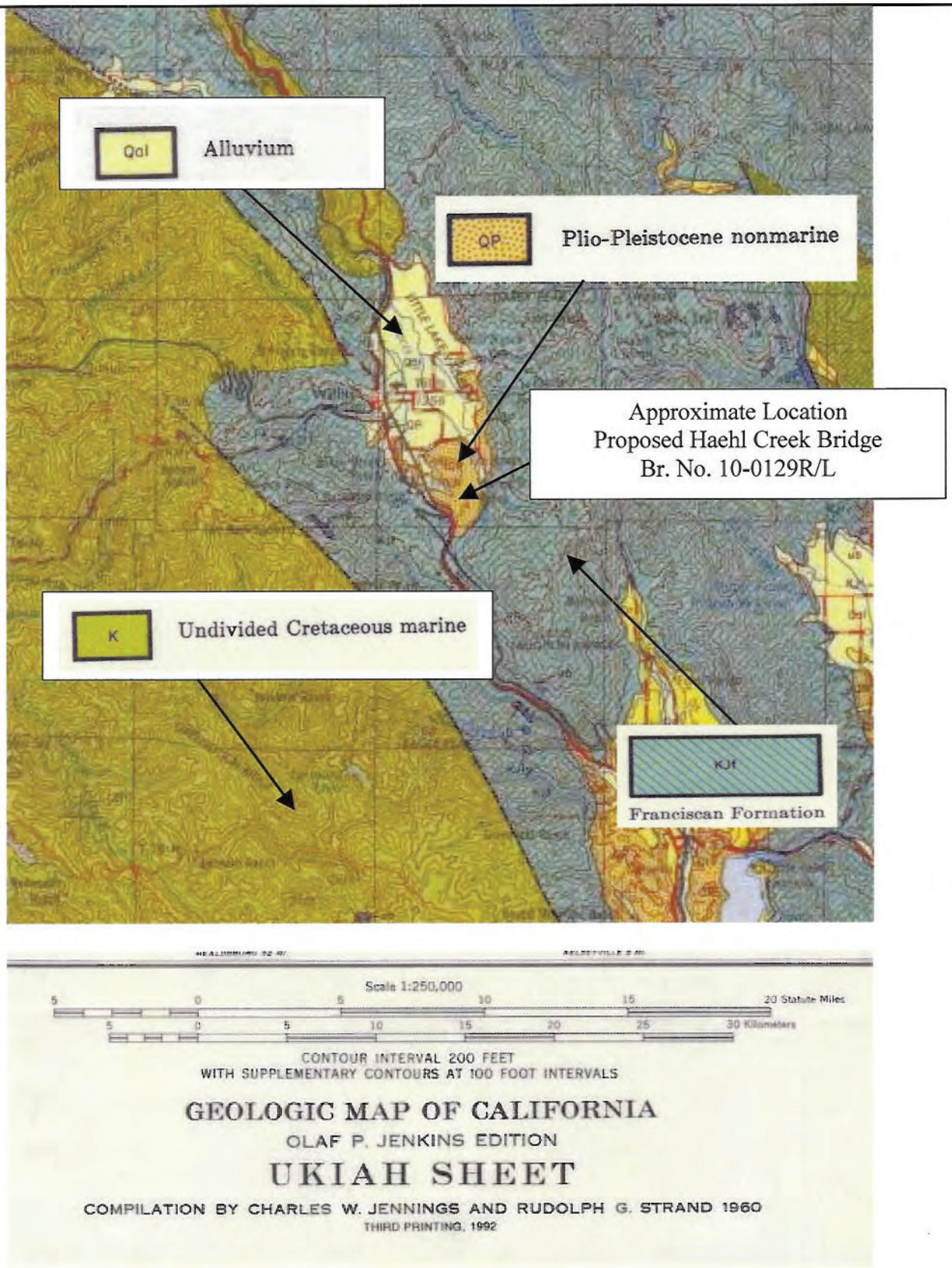


Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 November 9, 2006	Location Map
	01-MEN-101 KP R70.58/PM R43.86 **Haehl Creek Bridge, Br. No. 10-0129R/L**	Figure 1

Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 November 9, 2006	Location Map
	01-MEN-101 KP R70.58/PM R43.86 **Haehl Creek Bridge, Br. No. 10-0129R/L**	Figure 2



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 November 9, 2006	Geologic Map
	01-MEN-101 KP R70.58/PM R43.86 **Haehl Creek Bridge, Br. No. 10-0129R/L**	Figure 3

Willits Bypass
Haehl Creek Bridge (NEW)
Br. No. 10-0129R/L
01-262001

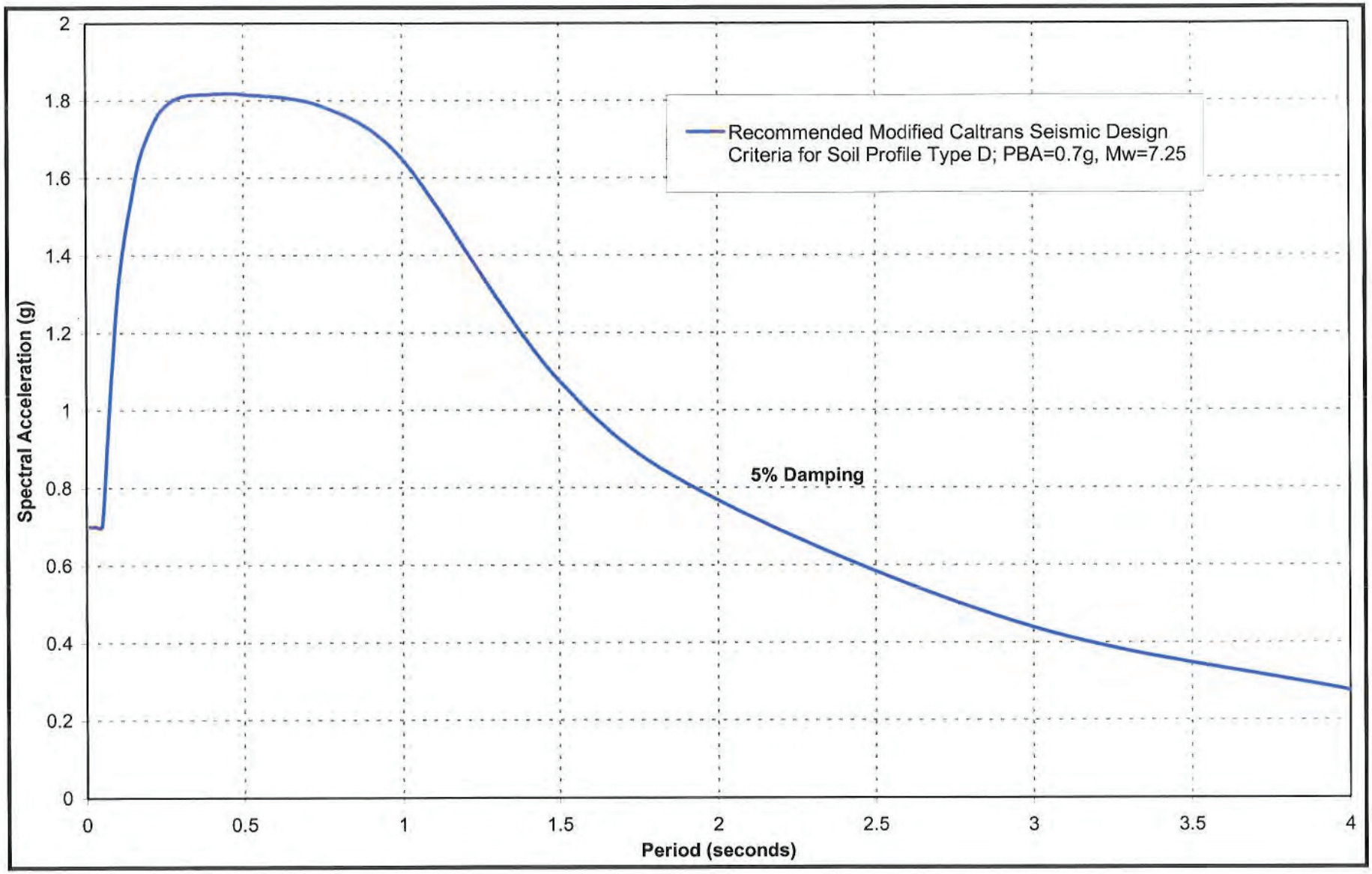


Figure 4. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: June 18, 2008

File: 01-MEN-101-KP R73.23/PM R45.50
EA#01-262001
Haehl Creek Bridge (NEW)
Br. No. 10-0159

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Haehl Creek Bridge (Br. No. 10-0159). The Office of Geotechnical Design North, Branch A completed a subsurface investigation in July 2005 to October 2006 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (July 2005 to October 2006) along with a review of the General Plan dated March 2, 2006 and a Foundation Plan dated February 21, 2006. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Haehl Creek Bridge (Br. No. 10-0159) will be incorporated as part of the Willits Bypass Project. The proposed new bridge will consist of one new structure and will be located within the city limits of Willits in Mendocino County (Figures 1 and 2). The structure will consist of a single span prestressed cast-in-place box girder (3 cell) type structure with seat type abutments and a wingwall at Abutment 1 and a Type 1 Retaining Wall at Abutment 2. The abutments, wingwall and Type 1 Retaining Wall will be supported on driven steel piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the south end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Quaternary alluvium. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation during July 2005 to October 2006.

The subsurface investigation consisted of three mud rotary borings (Nos. B-42, B-58 and B-77). The mud rotary borings were advanced using a self-casing wireline drilling method extending down to a maximum depth of 61.4 m (201.5 ft). The equipment used to drill the borings consisted of a Mobile B-80 drill rig equipped with a safety hammer and a CS200 drill rig equipped with a Diedrich automatic hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler at 1.5-meter intervals. Selected soil samples were bagged for laboratory testing.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed bridge (Br. No. 10-0159) generally consist of approximately 16.0 m (52.5 ft) to 22.3 m (73.0 ft) of firm to very stiff clay and clayey sand with gravel interbedded with medium dense gravel with clay and sand overlying hard sandy clay along with very dense sand and gravel with clay and sand with a trace of hard cobbles. The foundation materials extend from the ground surface to a maximum depth of 61.4 m (201.5 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels varied in elevation and were encountered at elevation 417.5 m (1369.8 ft) in Boring B-42, elevation 414.7 m (1360.5 ft) in Boring B-58, and 415.3 m (1362.5 ft) in Boring B-77 during the July 2005 to October 2006 subsurface investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Scour Evaluation

A Final Hydraulic Report (dated September 15, 2005) for the Haehl Creek Bridge was completed by the Office of Structure Maintenance and Investigations, Structure Hydraulics Branch. According to this report, there are no local scour concerns for this structure since it is a single span structure. The report recommends the abutment foundations be designed assuming no ground support (lateral and vertical) as a result of soil loss due to possible degradation or lateral stream migration down to the current stream thalwag elevation (lowest elevation in channel). The approximate thalwag elevation provided for the proposed structure (Br. No. 10-0159) is 414.3 m (1359.3 ft).

Rock Slope Protection (RSP) was mentioned and will be designed by the District to protect the roadway approach fills, if required.

For further information including site-specific scour assessment and mitigation measures, the Structures Hydraulics Branch should be contacted.

Corrosion Evaluation

Composite soil samples were collected in one of the borings during the 2006 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for Haehl Creek Bridge (Br. No. 10-0159)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639871	B-58	0.91-1.52	5.95	8000	N/A	N/A
C639925	B-77	3.66-4.11	6.48	4130	N/A	N/A
C639924	B-77	13.72-14.48	7.49	1534	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: Mechanical Analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Consolidation, Triaxial (Unconsolidated-Undrained) and Soil Corrosion Testing (pH, Sulfate, Chloride, and Resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request once the results are received.

Seismic Data and Evaluation

Based on the Caltrans California Seismic Hazard Map 1996, the controlling fault is the Maacama-Brush with a maximum credible earthquake moment magnitude of Mw=7.25, and is located about 0.2 kilometers southwest of the site. The Peak Bedrock Acceleration, based on attenuation equation by Geomatrix'97 is about 0.7g.

A field visit by the Engineering Geologist Martha Merriam, of the Office of Geotechnical Support, and several other staff from the Office of Geotechnical Design –North, concluded that 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.

Based on the foundation investigation dated September 2005, the subsurface geology generally consists of soft to hard sandy clay, and medium to very dense sandy and gravelly soil with clay. Liquefaction potential at this site is considered minimal.

Based on the boring logs a final Caltrans Seismic Design Criteria (CSDC) Acceleration Response Spectrum (ARS) corresponding to soil profile Type D is recommended for

design. Please note that due to the close proximity of this structure to the fault, we have performed a second modification to the CSDC ARS curve (see Figure 4). The modification is such that there is no increase in spectral accelerations (SA) for periods less than 0.5 second, and a 20% increase in SA for periods greater than 1 second. Between the periods of 0.5 and 1 second, a linear interpolation was used to estimate the SA.

Foundation Recommendations

The following foundation recommendations are for the new proposed Haehl Creek Bridge (Br. No. 10-0159).

At the Abutment 1 and 2 support locations, Class 625 Alternative "W" piles are recommended for support. The specified pile tip elevations (SPTE), shown below in Table 1, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 1. Pile Data Table for the new proposed Haehl Creek Bridge (Br. No. 10-0159) at Abutment 1 and 2 locations.

Location	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
			Compression kN	Tension kN			
Abutment 1	Class 625 Alt. "W"	625	1250	0	419.5 (1376.3)	396.0 (1,2) (1299.2)	396.0 (1299.2)
Abutment 2	Class 625 Alt. "W"	625	1250	0	419.0 (1374.7)	396.0 (1,2) (1299.2)	396.0 (1299.2)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 414.3 m (1359.3 ft)*

Retaining Wall Foundation

The following foundation recommendations are for the proposed Type 1 Retaining Wall structure located at Abutment 2, bridge location as shown on the Foundation Plan dated February 21, 2006. At this location, it is recommended driven steel HP 250X62 "H" piles be used to support these retaining wall segments. The specified pile tip elevations, shown below in Table 4, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 4. Pile Data Table for the new proposed Type 1 Retaining Wall Segment – Abutment 2 (Br. No. 10-0159).

Location	Retaining Wall Height mm (ft)	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
				Compression kN	Tension kN			
Abutment 2	4800	HP 250X62	400	800	0	418.7 (1373.7 ft)	396.0 (1,2) (1299.2)	396.0 (1299.2)
Abutment 2	3600	HP 250X62	400	800	0	419.9 (1377.7)	396.0 (1,2) (1299.2)	396.0 (1299.2)
Abutment 2	2400	HP 250X62	400	800	0	421.1 (1381.7)	396.0 (1,2) (1299.2)	396.0 (1299.2)
Abutment 2	1800	HP 250X62	400	800	0	422.4 (1385.8)	396.0 (1,2) (1299.2)	396.0 (1299.2)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 414.3m (1359.3 ft) at Abutment 2 Type 1 Retaining Wall Segment.*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Branch A shall be contacted for further recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. At the Engineer's option, any Class 625, Alt. "W" pile or steel "H" pile driven to within 3.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

3. The Contractor should anticipate hard and erratic driving of the Class 625, Alt. "W" piles and the steel "H" piles due to the presence of hard cobbles and very dense foundation material. The Contractor should anticipate field cutting and splicing of the Class 625, Alt. "W" piles and the steel "H" piles. Refer to the LOTB sheets for details.

4. At Abutment 2 Retaining Wall location, driven steel HP 250X62 "H" piles shall be driven through the new fill embankment in oversized pre-drilled holes (where applicable) that conform to the Standard Specifications in Section 49-1.06, "Pre-drilled Holes." The predrilled holes are not to extend below the elevation 419.0 m (1374.7 ft).

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-7260, Jacqueline Martin at (916) 227-5503, Reza Mahallati at (916) 227-1033 or Reid Buell at (916) 227-7190, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard Special Provisions 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:

A. Log of Test Borings for Haehl Creek Bridge, Bridge No. 10-0159.

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

A. Foundation Report for Haehl Creek Bridge, Bridge No. 10-0159, dated June 18, 2008.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North

Report by:

FOR JACQUELINE MARTIN

JACQUELINE MARTIN
Engineering Geologist
Office of Geotechnical Design-North

Supervised by:

REGISTERED PROFESSIONAL ENGINEER
REZA MAHALLATI
No. 49374
Exp. 9/30/08
CIVIL
STATE OF CALIFORNIA

REID BUELL, C.E.G. NO. 1481
Senior Engineering Geologist
Office of Geotechnical Design-North

REZA MAHALLATI
Senior Materials & Research Engineer
Office of Geotechnical Design-North

REGISTERED GEOLOGIST
REID L. BUELL
No. 1481
Exp. 4/30/09
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

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

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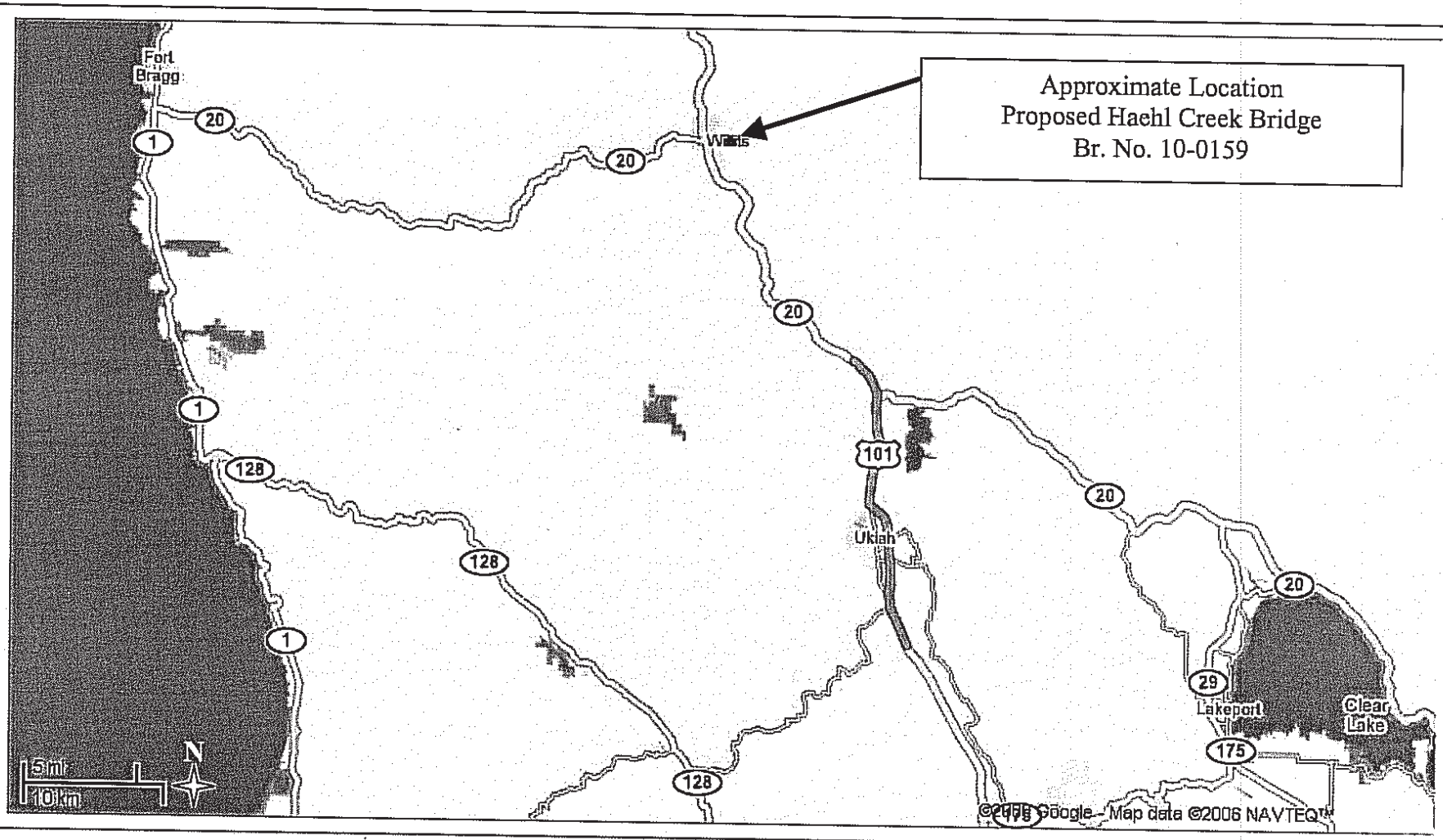
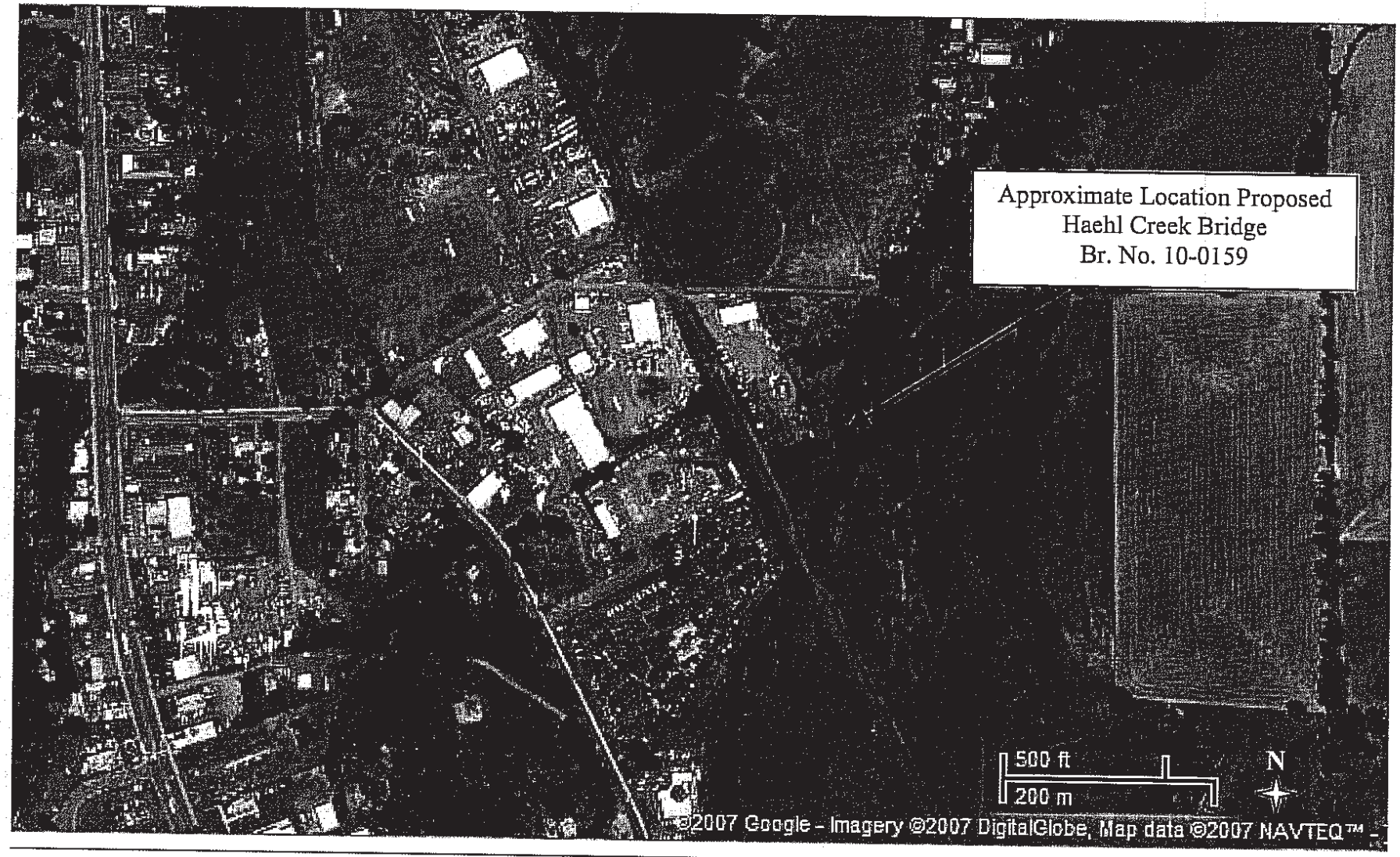


Caltrans

Division of Engineering Services
Geotechnical Services
Geotechnical Design – North

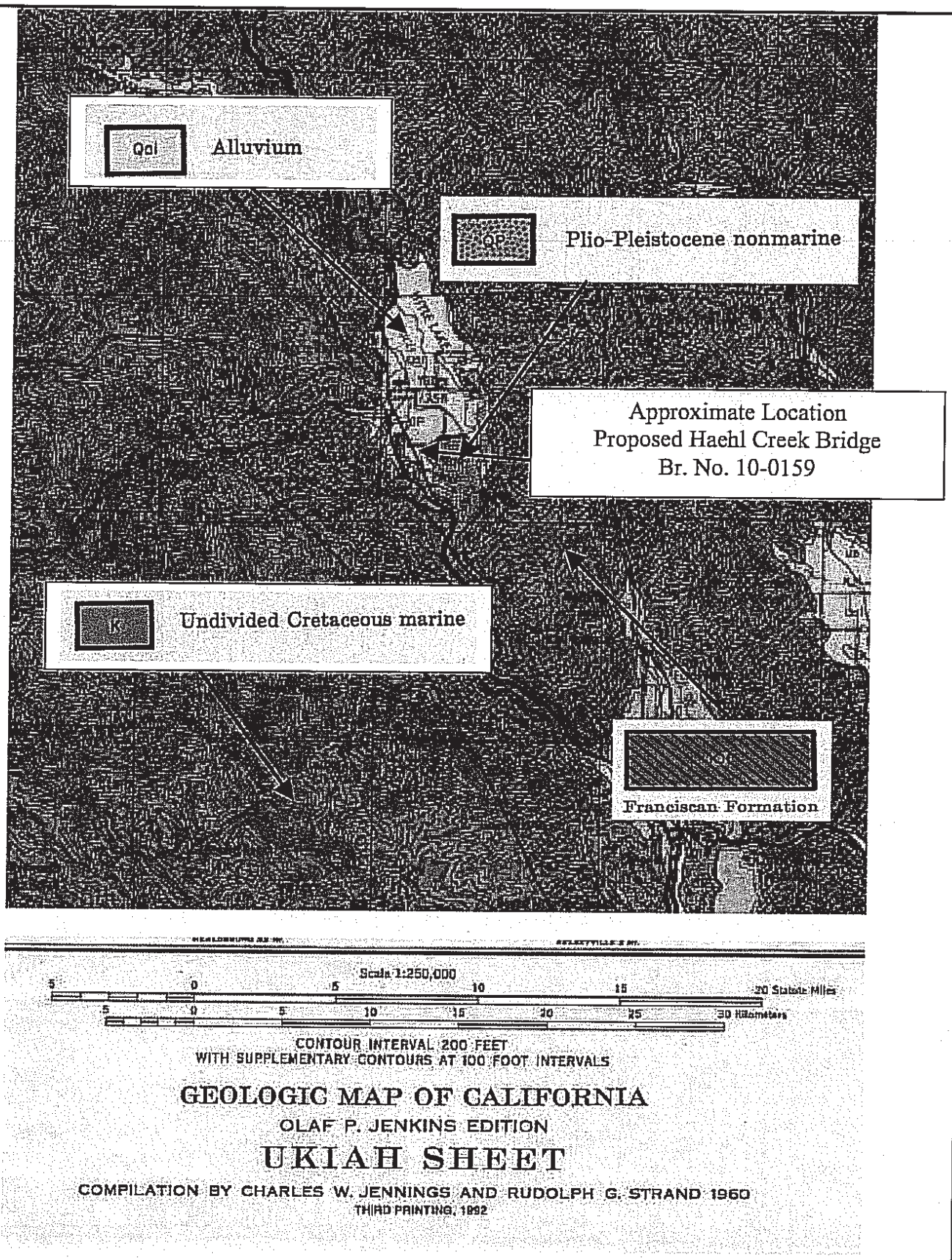
EA: 01-262001	Location Map
June 18, 2008	
01-MEN-101 KP R73.23/PM R45.50 Haehl Creek Bridge, Br. No. 10-0159	Figure 1



Caltrans

Division of Engineering Services
Geotechnical Services
Geotechnical Design – North

EA: 01-262001	Location Map
June 18, 2008	
01-MEN-101 KP R73.23/PM R45.50 **Haehl Creek Bridge, Br. No. 10-0159**	Figure 2



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 June 18, 2008	Geologic Map
	01-MEN-101 KP R73.23/PM R45.50 **Haehl Creek Bridge, Br. No. 10-0159**	Figure 3

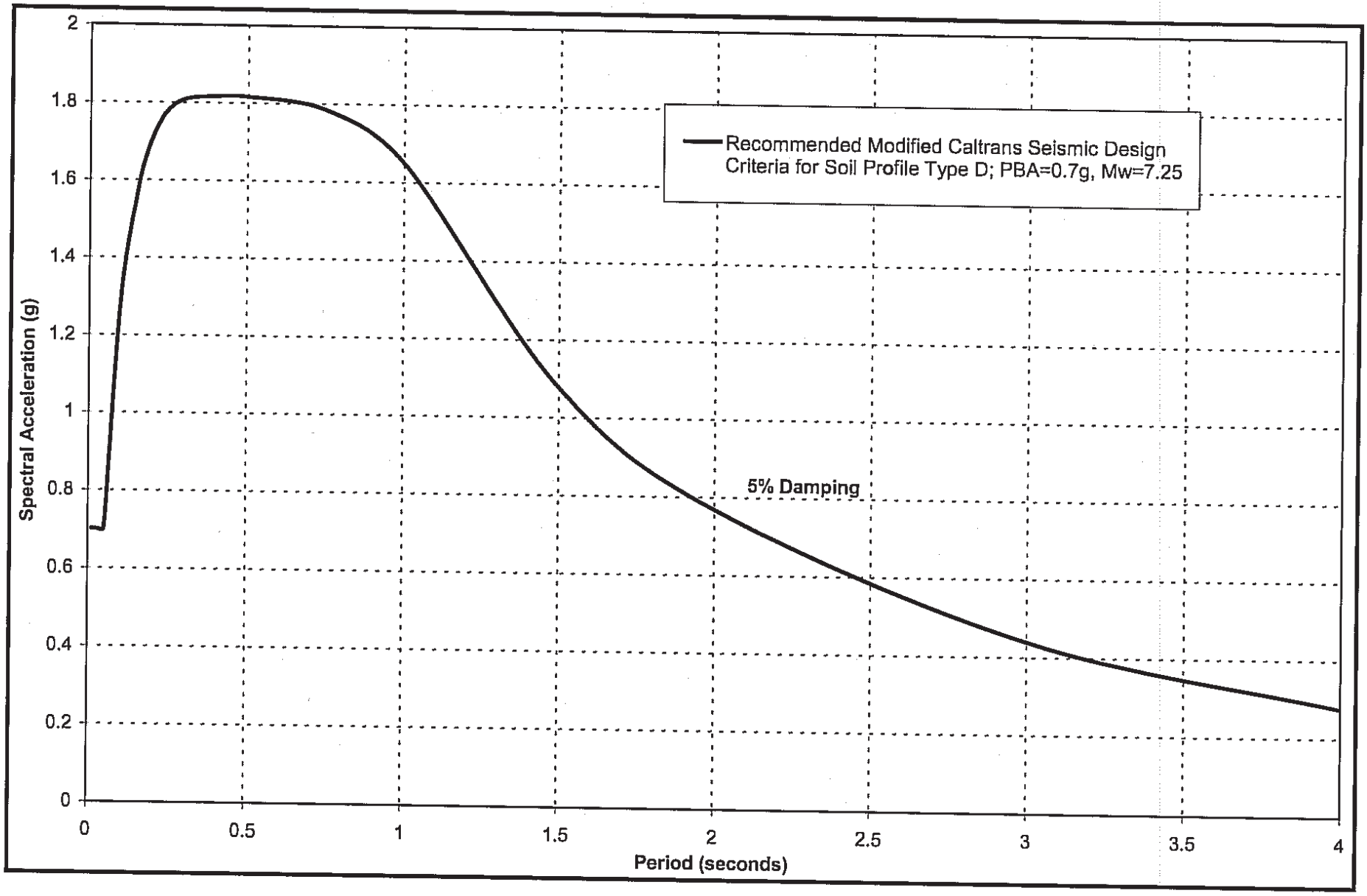


Figure 4. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Date: February 23, 2009

File: 01-MEN-101-KP R74.37/PM R46.21
EA#01-262001
FLOODWAY VIADUCT (NEW)
Br. No. 10-0165

Attention: Mr. Marc Friedheim

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed Floodway Viaduct (Br. No. 10-0165). The Office of Geotechnical Design North, Branch A completed a subsurface investigation from May 2004 to September 2008 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (May 2004 to September 2008) along with a review of the available General Plan dated May 17, 2008. No Foundation Plan was available at the time of this report preparation. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new Floodway Viaduct (Br. No. 10-0165) will consist of one new structure and will be incorporated as part of the Willits Bypass (Figures 1 and 2). The structure will consist of a continuous thirty-three span prestressed cast in place concrete box girder (3 cell) structure with open end seat abutments with wingwalls and two column bents, supported on driven steel "H" piles at the abutments and driven steel pipe piles at the bents.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a

somewhat more northerly trend than do the ridges and valleys. The province contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the south end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

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At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Quaternary alluvium. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation from May 2004 to September 2008.

The subsurface investigation consisted of twenty-three mud rotary borings (Nos. B-2, B-12, B-17, B-18, B-19, B-22, B-23, B-27, B-30, B-31, B-55, B-66, B-72, B-79, B-88, B-89, B-90, B-91, B-92, B-93, B-94, B-95, and B-96) and one cone penetrometer sounding (C-20). In addition,

borehole geophysical measurements were acquired in Borings B-2 and B-12. The geophysical measurements taken in Borings B-2 and B-12 included natural gamma, formation conductivity and resistivity, and P-S suspension log records. The mud rotary borings were advanced using a self-casing wireline drilling method extending down to a maximum depth of 73.5 m (241.0 ft). Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler. Selected soil samples were bagged for laboratory testing. The cone penetrometer sounding was used to supplement the subsurface investigation. A summary of the borings drilled during the subsurface investigation is included below in Table 1.

Table 1. Subsurface Exploration Summary for the Floodway Viaduct (Br. No. 10-0165)

Boring No.	Completion Date	Drill Rig Type	Hammer Type	Approx. Ground Surface (m)	Boring Depth (m)
B-2	5/28/2004	Acker	Safety	411.6	61.3
B-12	7/15/2004	Mobile B-47	Safety	408.9	61.4
B-17	9/14/2004	CS500	Diedrich Auto	409.8	53.3
B-18	10/14/2004	Mobile B-47	Safety	409.4	46.2
B-19	10/14/2004	Mobile B-47	Safety	410.0	46.2
B-22	1/5/2005	Mobile B-47	Safety	413.3	61.1
B-23	6/16/2005	Mobile B-47	Safety	410.1	69.0
B-27	6/23/2005	Mobile B-47	Safety	409.7	61.4
B-30	7/7/2005	CME850	CME Auto	410.3	46.2
B-31	6/30/2005	Mobile B-47	Safety	410.8	70.6
B-55	10/6/2005	Mobile B-80	Safety	409.5	73.5
B-66	8/16/2006	CS2000	Diedrich Auto	412.3	61.4
B-72	8/29/2006	CS2000	Diedrich Auto	412.8	37.0
B-79	10/19/2006	CME750	CME Auto	411.4	46.2
B-88	10/11/2007	Acker	Longyear Auto	411.1	37.0
B-89	8/6/2008	Acker	Longyear Auto	411.9	34.0
B-90	8/13/2008	Acker	Longyear Auto	411.2	37.0
B-91	8/19/2008	Acker	Longyear Auto	409.5	30.9
B-92	8/21/2008	Acker	Longyear Auto	409.8	37.0
B-93	8/27/2008	Acker	Longyear Auto	409.4	40.1
B-94	8/28/2008	Acker	Longyear Auto	409.0	37.0
B-95	9/4/2008	Acker	Longyear Auto	408.5	40.1
B-96	9/4/2008	CS2000	Diedrich Auto	408.3	37.0

The subsurface investigation revealed the foundation materials encountered at the location of the proposed bridge (Br. No. 10-0165) generally consist of approximately 19.5 m (64 ft) to 33.2 m (109 ft) of very soft to very stiff clay and sandy clay overlying medium dense to very dense well graded gravel with clay, silt and sand; medium dense to very dense clayey sand and clayey gravel along with firm to hard clay.

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels varied from elevation 411.31 m (1349.61 ft) in Boring B-22 to 406.16 m (1332.56 ft) in Boring B-90 during the May 2004 to September 2008 subsurface investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Scour Evaluation

A Final Hydraulic Report (dated March 22, 2005) for the Floodway Viaduct structure was completed by the Office of Structure Maintenance and Investigations, Structure Hydraulics Branch. According to this report, the potential local pier scour for piers located within Baechtel and Broaddus Creeks was calculated to be 3.6 m (11.8 ft.). The report recommends that all new foundations be placed below the anticipated scour elevation. Based on the March 22, 2005 Final Hydraulic Report, the scour elevation is interpreted to be 403.9 m at the Bent 24 support location. No other support locations are located in the creeks.

For further information including site-specific scour assessment and mitigation measures, the Structures Hydraulics Branch should be contacted.

Corrosion Evaluation

Composite soil samples were collected in eight of the borings during the May 2004 to September 2008 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 2 below for specific test results.

Table 2: Corrosion Test Summary-Composite Samples for Floodway Viaduct (Br. No. 10-0165)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C640909	B-12	0.0-1.52	6.47	2600	N/A	N/A
C640810	B-12	8.53-9.14	7.17	2800	N/A	N/A
C640811	B-12	16.15-16.76	6.68	1600	N/A	N/A
C640631	B-17	0.0-1.52	7.74	5100	N/A	N/A
C640632	B-17	5.18-5.49	7.39	2500	N/A	N/A
C639851	B-18	1.83-4.57	5.99	3000	N/A	N/A
C639852	B-18	4.88-7.62	6.74	2400	N/A	N/A
C639853	B-18	7.92-10.67	8.11	2700	N/A	N/A
C639858	B-22	0.91-1.22	6.27	3300	N/A	N/A
C639859	B-22	3.35-4.11	6.78	2700	N/A	N/A
C639863	B-23	3.66-4.57	6.92	2700	N/A	N/A
C639864	B-23	10.06-10.67	7.63	2600	N/A	N/A
C639903	B-66	2.44-2.74	6.43	5300	N/A	N/A
C639904	B-66	5.18-5.49	6.89	2600	N/A	N/A
C639907	B-72	3.05-3.81	6.21	5000	N/A	N/A
C639908	B-72	9.75-10.36	6.91	2700	N/A	N/A
C639916	B-88	0-1.52	6.11	4690	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Moisture Content, Specific Gravity, Consolidation, Triaxial (unconsolidated-undrained), Triaxial (consolidated-undrained), Unconfined

Compression and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results are available upon request.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 0.8 to 1.5 kilometers southwest of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Based on the boring logs along with availability of shear wave velocity, undrained shear, and Atterberg limits, a final Caltrans Seismic Design Criteria (CSDC) Acceleration Response Spectrum (ARS) Curve corresponding to soil profile Type D is recommended for design. Please note that due to the close proximity of this structure to the fault, we have performed a second modification to the CSDC ARS curve (see Figure 4). The modification is such that there is no increase in spectral accelerations (SA) for periods less than 0.5 seconds, and a 20% increase in SA for periods greater than 1 second. Between the periods of 0.5 and 1 second, a linear interpolation was used to estimate the SA.

Fill Settlement

Due to the presence of compressible clay soils beneath the proposed approach fills at Abutment 1 and Abutment 34, a settlement waiting period is to be required after completion of the approach fills and before installing the steel "H" piles for the abutments. Settlement will take place during and after the construction of the approach fills. Settlement on the order of 250 mm to 500 mm is anticipated.

Due to the extent of compressible clay soils which may result in excessive settlement beneath the proposed approach fill at Abutment 34, it is anticipated that additional mitigation measures such as placement of additional surcharge and/or the use of wick drains will be required. Without these measures, the duration of the predicted settlement is estimated to be greater than 10 years. The intent of the surcharge and/or use of wick drains is to decrease the settlement waiting period to less than one-year. Details of these measures will be included in the Geotechnical Design Report to be prepared by this Office. Any questions regarding fill settlement, settlement waiting periods and/or proposed mitigation measures should be directed to John Huang at (916) 227-1037 or Ben Barnes at (916) 227-1039, of the Office of Geotechnical Design-North, Branch E.

Regardless of the target settlement or the estimated settlement time period, the rate of settlement obtained during the settlement monitoring will determine if primary consolidation settlement is complete. That is, if the settlement rate is noticeably reduced sooner than the end of the estimated settlement time period, then it would be acceptable for construction of the abutment foundations to commence. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes.

Foundation Recommendations

The following foundation recommendations are for the new proposed Floodway Viaduct (Br. No. 10-0165).

At Abutment 1 and 34 support locations, driven steel HP 250x85 "H" piles are recommended for support. The specified pile tip elevations (SPTE), shown below in Table 3 will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 3. Pile Data Table for Proposed Floodway Viaduct (Br. No. 10-0165) Abutment 1 and 34 Support Locations.

Location	Pile Type	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m	Specified Pile Tip Elevation m (ft)
		Compression kN	Tension kN			
Abutment 1	HP 250x85	1200	0	416.5 (1366.5)	388.5 (1) (1274.6 ft.)	388.5 (1274.6)
Abutment 34	HP 250x85	1200	0	412.2 (1352.4)	376.5 (1) (1235.2 ft.)	370.5 (1215.6)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression*

At Bents 2 through 33, 610 mm open-ended steel pipe piles are recommended for support. The specified pile tip elevations (SPTE), shown below in Table 4, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression and tension.

Table 4. Pile Data Table for Proposed Floodway Viaduct (Br. No. 10-0165) Bents 2 through 33 Support Locations.

Location	Pile Type	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m	Specified Pile Tip Elevation m (ft)
		Compression kN	Tension kN			
Bent 2	PP 610 x 13	1600	250	411.500 (1350.1)	389.0 (1) 407.5 (2)	389.0 (1277.2)
Bent 3	PP 610 x 13	1450	350	411.900 (1351.4)	389.0 (1) 406.5 (2)	389.0 (1277.2)
Bent 4	PP 610 x 13	1800	150	410.900 (1348.1)	385.5 (1) 406.5 (2)	385.5 (1264.8)
Bent 5	PP 610 x 13	1750	150	410.30 (1346.1)	385.5 (1) 406.5 (2)	385.5 (1264.8)
Bent 6	PP 610 x 13	1750	200	410.100 (1345.5)	385.5 (1) 401.0 (2)	385.5 (1264.8)

Bent 7	PP 610 x 13	1750	250	410.000 (1345.1)	385.0 (1) 400.5 (2)	385.0 (1263.1)
Bent 8	PP 610 x 16	2050	500	409.900 (1344.8)	385.5 (1) 395.5 (2)	385.5 (1264.8)
Bent 9	PP 610 x 13	2100	550	409.000 (1341.9)	385.5 (1) 395.5 (2)	385.5 (1264.8)
Bent 10	PP 610 x 13	1600	200	409.200 (1342.5)	387.5 (1) 400.5 (2)	387.5 (1271.3)
Bent 11	PP 610 x 13	1500	50	409.200 (1342.5)	387.5 (1) 400.0 (2)	387.5 (1271.3)
Bent 12	PP 610 x 13	1500	100	408.900 (1341.5)	387.5 (1) 400.0 (2)	387.5 (1271.3)
Bent 13	PP 610 x 19	2000	400	407.400 (1336.6)	386.5 (1) 398.5 (2)	386.5 (1268.0)
Bent 14	PP 610 x 13	1850	400	408.600 (1340.6)	386.5 (1) 398.5 (2)	386.5 (1268.0)
Bent 15	PP 610 x 13	1500	50	408.400 (1339.9)	387.0 (1) 405.5 (2)	387.0 (1269.7)
Bent 16	PP 610 x 13	1500	50	408.200 (1339.2)	386.5 (1) 405.0 (2)	386.5 (1268.0)
Bent 17	PP 610 x 13	1650	100	408.100 (1338.9)	386.0 (1) 404.5 (2)	386.0 (1266.4)
Bent 18	PP 610 x 19	2500	1150	407.691 (1337.6)	382.5 (1) 388.5 (2)	382.5 (1254.9)
Bent 19	PP 610 x 13	2050	650	407.672 (1337.5)	385.0 (1) 396.5 (2)	385.0 (1263.1)
Bent 20	PP 610 x 13	2150	500	407.326 (1336.4))	384.5 (1) 398.0 (2)	384.5 (1261.5)
Bent 21	PP 610 x 19	2650	1000	407.621 (1337.3)	381.5 (1) 389.0 (2)	381.5 (1251.6)
Bent 22	PP 610 x 19	2800	900	407.351 (1336.5)	381.0 (1) 389.0 (2)	381.0 (1250.0)
Bent 23	PP 610 x 19	3250	1650	408.171 (1339.1)	379.5 (1) 384.5 (2)	379.5 (1245.1)
Bent 24	PP 610 x 19	3400	1650	401.873 (1318.5)	376.0 (1,3) 381.5 (2)	376.0 (1233.6)
Bent 25	PP 610 x 19	3200	1450	407.641 (1337.4)	379.5 (1) 386.0 (2)	379.5 (1245.1)
Bent 26	PP 610 x 19	3000	1100	406.871 (1334.9)	375.0 (1) 388.5 (2)	375.0 (1230.3)
Bent 27	PP 610 x 13	2050	400	407.621 (1337.3)	380.5 (1) 399.0 (2)	380.5 (1248.4)
Bent 28	PP 610 x 13	1700	500	407.871 (1338.2)	381.0 (1) 397.0 (2)	381.0 (1250.0)
Bent 29	PP 610 x 13	1650	500	407.420 (1336.7)	376.0 (1) 393.5 (2)	376.0 (1233.6)
Bent 30	PP 610 x 13	1650	450	407.171 (1335.9)	376.0 (1) 394.5 (2)	376.0 (1233.6)
Bent 31	PP 610 x 13	1600	450	406.511 (1333.7)	376.0 (1) 394.5 (2)	376.0 (1233.6)
Bent 32	PP 610 x 13	1650	400	406.495 (1333.6)	377.0 (1) 394.5 (2)	377.0 (1236.9)
Bent 33	PP 610 x 13	1650	350	405.834 (1331.5)	377.0 (1) 395.0 (2)	377.0 (1236.9)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Tension (3)Scour potential exists to Elev. 403.9 m (1325.1 ft) at Bent 24.*

Notes to Special Provisions

1. Prior to installing driven piling at Bents 2 through 33, the Contractor shall provide a driving system submittal. A submittal shall be made for five control locations. The first control location shall consist of Bents 2 through 7. The second control location shall consist of Bents 8 through 17. The third control location shall consist of Bents 18 through 26. The fourth control location shall consist of Bents 27 through 28. The fifth control location shall consist of Bents 29 through 33. All proposed driving systems (i.e. hammer that may be brought onto the site) shall be included in the submittal.

2. The Contractor shall select a pile from Bent 5 to be dynamically monitored in the first control location, one pile from Bent 11, one pile from Bent 13 and one pile from Bent 14 to be dynamically monitored in the second control location, one pile from Bent 19 and one pile from Bent 23 to be dynamically monitored in the third control location, a pile from Bent 27 to be dynamically monitored in the fourth control location and a pile from Bent 31 to be dynamically monitored in the fifth control location. These piles shall be the first piles driven. No other piles shall be driven until wave equation acceptance criteria has been provided by the Engineer.

3. The first pile driven at Bent 24 shall be an indicator pile to be dynamically monitored.

4. Pile acceptance criteria will be developed using a wave equation analysis in conjunction with dynamic monitoring. Bearing acceptance criteria curves will be provided by the foundation Testing Branch of the Office of Geotechnical Support. Please coordinate with Brian Liebich from the Foundation Testing Branch for this information.

5. Pile restrikes by PDA monitoring shall be required 1 day and 8 days after installation at all control locations. Driving operations shall be suspended a minimum of 1.0 meter above specified tip elevation prior to the 1 day restrike and approximately 0.5 meters above the specified tip elevation prior to the 8 day restrike, as directed by the Engineer. The total pile set period shall be considered 8 days.

6. The Contractor shall provide high-energy hammer blows from the beginning of restrike of piles. For diesel hammers, the Contractor shall provide a hammer that has been warmed up at another location prior to performing the restrike.

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Branch A shall be contacted for further recommendations.

3. Specified pile tip elevations shown in Table 3 above are based on preloading of compressible clay soils achieved by placement of approach fills prior to construction of the proposed abutment foundations. If preloading is not performed, the Office of Geotechnical Design North is to be contacted for revised foundation recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. Settlement shall be monitored. Monitoring devices shall consist of settlement platforms placed at the bottom of the approach fill embankments at Abutments 1 and 34. A minimum of six settlement devises (three or more beneath each approach fill) shall be placed beneath the centerline of embankments at Abutments 1 and 34 at locations to be determined by the Engineer. Abutment construction shall not start until primary consolidation is completed per field monitoring.

3. Due to presence of compressible clay soils beneath the proposed abutments, the abutment locations shall be preloaded. A waiting period will be required prior to installation of piles at Abutments 1 and 34. Additional mitigation measures such as placement of additional surcharge and/or the use of wick drains may be required. Details of these measures can be found in the Geotechnical Design Report to be prepared by this Office. The fill settlement shall be monitored during the waiting period. If it is determined that the settlement has ceased prior to the end of the recommended waiting period, with the written approval of the Engineer, the waiting period may be terminated at that time. In the opposite case, if the rate of settlement does not change by the time the estimated settlement time period is over, monitoring shall continue, extending the settlement time period, until the settlement rate diminishes. Any questions regarding fill settlement, settlement waiting periods and/or proposed mitigation measures should be directed to John Huang at (916) 227-1037 or Ben Barnes at (916) 227-1039, of the Office of Geotechnical Design-North, Branch E.

4. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

5. At Abutment 1 and 34 locations, driven steel HP 250X85 "H" piles shall be driven through the new fill embankment in oversized pre-drilled holes that conform to the Standard Specifications in Section 49-1.06, "Pre-drilled Holes." The predrilled holes are not to extend below the elevations shown in Table 5.

Table 5. Predrilled Elevations

Location	Predrilled Elevation
Abutment 1	413.0 m (1355.0 ft)
Abutment 34	408.5 m (1340.2 ft)

6. The Contractor should anticipate hard and erratic driving of the steel pipe piles at Bents 18 through 26 due to the presence of very dense foundation material.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035 or Reid Buell at (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard special Provisions 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:

A. Log of Test Borings for Floodway Viaduct, Bridge No. 10-0165.

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

A. Foundation Report for the Floodway Viaduct, Bridge No. 10-0165, dated February 23, 2009.

Report by:

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Engineering Geologist
Office of Geotechnical Design-North

Report by:

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Structure OE

REFERENCES

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- Standard Specifications, July 2004 and May 2006.
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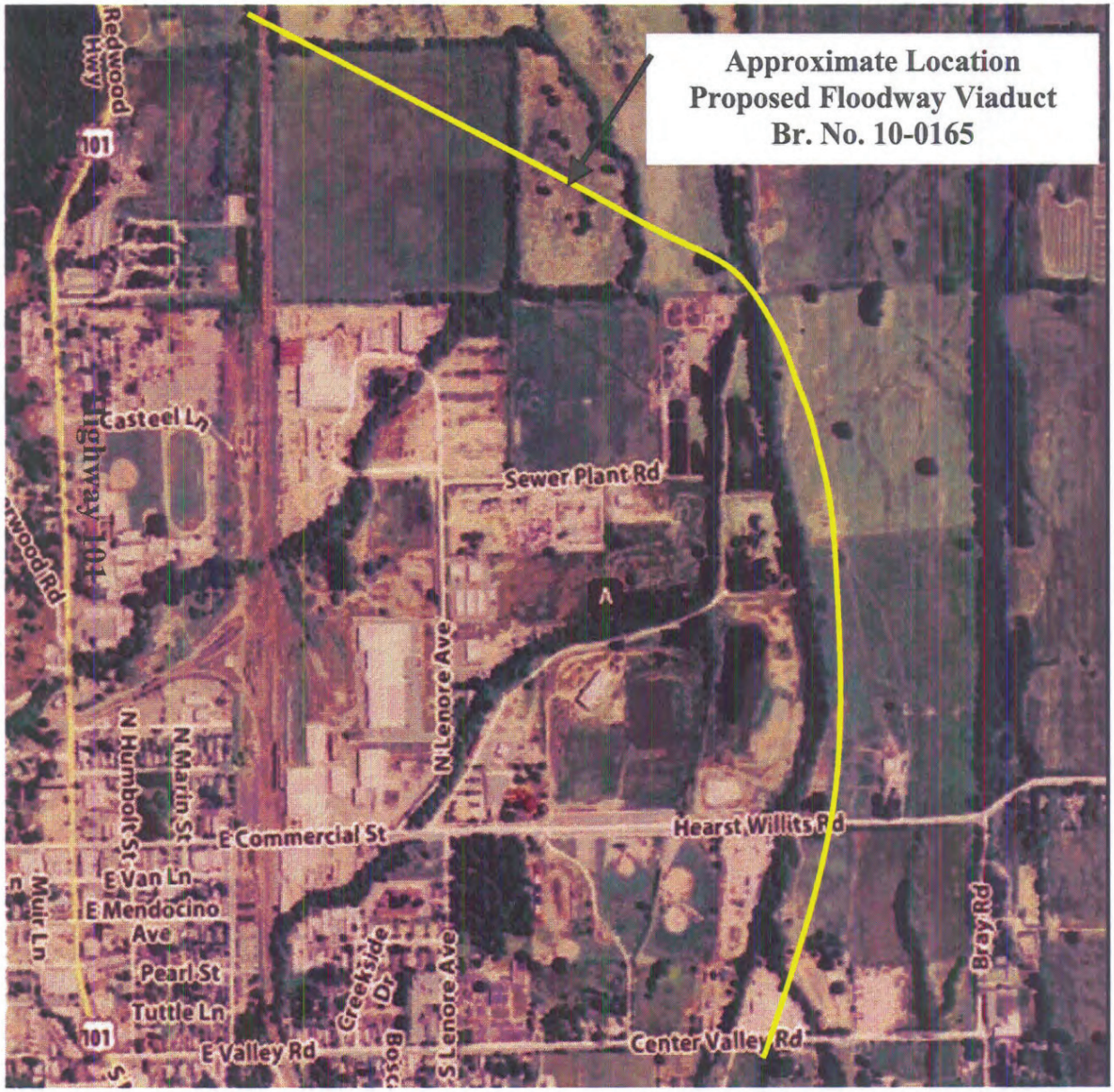
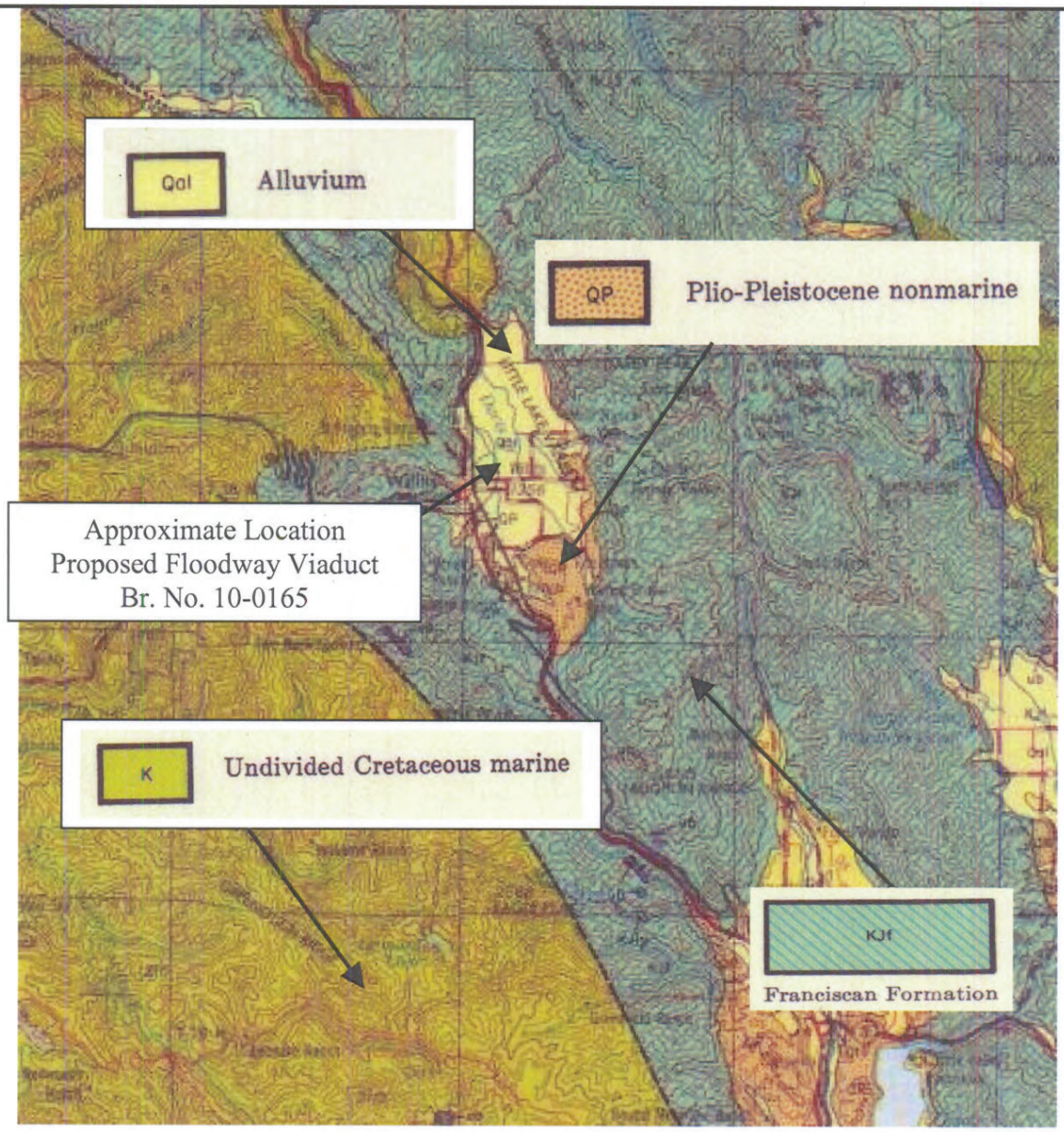
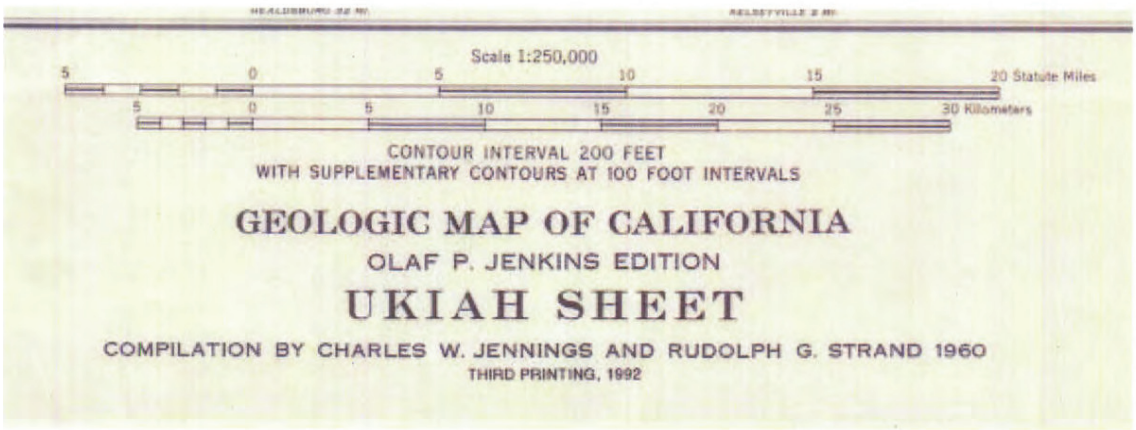
Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 February 23, 2009	Location Map
	01-MEN-101 KP 74.37/PM R46.21 **Floodway Viaduct, Br. No. 10-0165**	Figure 1



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 February 23, 2009	Location Map
	01-MEN-101 KP 74.37/PM R46.21 **Floodway Viaduct, Br. No. 10-0165**	Figure 2





Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 February 23, 2009	Geologic Map
	01-MEN-101 KP 74.37/PM R46.21 **Floodway Viaduct, Br. No. 10-0165**	Figure 3

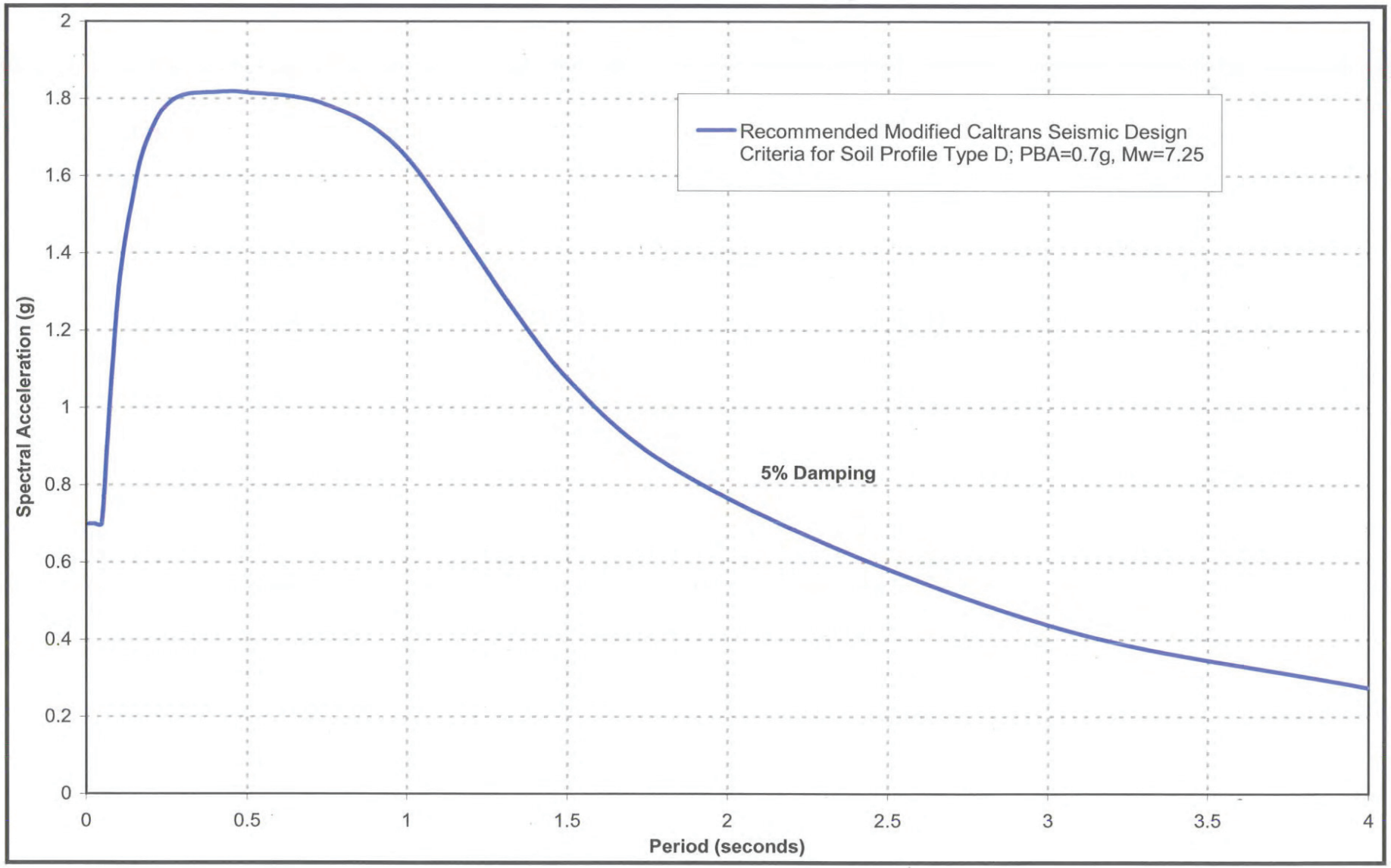


Figure 4. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: June 18, 2008

File: 01-MEN-101-KP R72.39/PM R44.98
EA#01-262001
East Hill Rd UC (NEW)
Br. No. 10-0157

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Revised Foundation Report

Introduction/Scope of Work

The following recommendations are in response to a request for revised foundation recommendations made by the Office of Bridge Design North-Branch 2 for the proposed East Hill Road Undercrossing. The following report supersedes our original East Hill Road UC (New) memo dated December 22, 2006. This report presents the foundation recommendations for the new proposed East Hill Road Undercrossing (Br. No. 10-0157). The Office of Geotechnical Design North, Branch A completed a subsurface investigation in September 2005 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (September 2005) along with a review of the available General Plan dated April 3, 2008 and a Foundation Plan dated February 6, 2006. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new East Hill Road UC (Br. No. 10-0157) will be incorporated as part of the Willits Bypass Project. The proposed new bridge will consist of one new structure and will be located within the city limits of Willits in Mendocino County (Figures 1 and 2). The structure will be a single span prestressed cast-in-place box girder (3 cell) type structure with seat type abutments and wingwalls supported on driven steel H-Piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The province contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the south end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of

sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Quaternary alluvium. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation during September 2005.

The subsurface investigation consisted of two mud rotary borings (Nos. B-52 and B-53). The mud rotary borings were advanced using a self-casing wireline coring method extending down to a maximum depth of 31.2 m (102.5 ft). The equipment used to drill the borings consisted of a Mobile B-80 drill rig equipped with a safety hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler at 1.5-meter intervals. Selected soil samples were bagged for laboratory testing.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed bridge (Br. No. 10-0157) generally consist of approximately 17.7 m (58.0 ft) to 18.9 m (62.0 ft) of firm to very stiff clay and medium dense to very dense clayey sand and gravel with clay and sand overlying hard clay along with very dense sand and gravel with clay and sand. The foundation materials extend from the ground surface to a maximum depth of 31.2 m (102.5 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water levels varied from elevation 422.6 m (1386.5 ft) in Boring B-52 to 421.2 m (1381.9 ft) in Boring B-53 during the September 2005 subsurface investigation. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Corrosion Evaluation

Composite soil samples were collected in one of the borings during the 2006 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that the composite samples collected are not corrosive at this site. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for East Hill Road Undercrossing (Br. No. 10-0157)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C639869	B-52	0.0-1.83	6.22	5800	N/A	N/A
C639870	B-52	6.86-7.92	8.07	1500	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Moisture Content, Specific Gravity, Consolidation, Triaxial (unconsolidated-undrained), and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request.

Seismic Data and Evaluation

Based on the Caltrans California Seismic Hazard Map 1996, the controlling fault is the Maacama-Brush with a maximum credible earthquake moment magnitude of Mw=7.25, and is located about 0.5 kilometers southwest of the site. The Peak Bedrock Acceleration, based on attenuation equation by Geomatrix'97 is about 0.7g.

A field visit by the Engineering Geologist Martha Merriam, of the Office of Geotechnical Support, and several other staff from the Office of Geotechnical Design–North, concluded that 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.

Based on the foundation investigation dated September 2005, the subsurface geology generally consists of soft to hard sandy clay, and medium to very dense sandy and gravelly soil with clay. Liquefaction potential at this site is considered minimal.

Based on the boring logs a final Caltrans Seismic Design Criteria (CSDC) Acceleration Response Spectrum (ARS) corresponding to soil profile Type D is recommended for design. Please note that due to the close proximity of this structure to the fault, we have performed a second modification to the CSDC ARS curve (see Figure 4). The modification is such that there is no increase in spectral accelerations (SA) for periods less than 0.5 second, and a 20% increase in SA for periods greater than 1 second. Between the periods of 0.5 and 1 second, a linear interpolation was used to estimate the SA.

Foundation Recommendations

The following foundation recommendations are for the new proposed East Hill Road Undercrossing (Br. No. 10-0157).

<u>East Hill Road UC (Br. No. 10-0157)</u>

At all structure support locations, driven steel HP 250X85 "H" piles are recommended for support. The specified pile tip elevations (SPTE), shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed East Hill Road UC (Br. No. 10-0157).

Location	Pile Type	Design Load kN	Nominal Resistance		Bottom of Pile Cap Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
			Compression kN	Tension kN			
Abutment 1R	HP 250X85	625	1250	0	427.1 (1401.2)	404.6 (1) (1327.6)	404.6 (1327.6)
Abutment 2R	HP 250X85	625	1250	0	426.8 (1400.3)	402.0 (1) (1318.9)	402.0 (1318.9)

Note: Design Pile Tip Elevation is controlled by the following demand: (1) *Compression*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Branch A shall be contacted for further recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

3. At Abutment 1 and 2 locations, driven steel HP 250X85 "H" piles shall be driven through the new fill embankment in oversized pre-drilled holes that conform to the Standard Specifications in Section 49-1.06, "Pre-drilled Holes." The predrilled holes are not to extend below the elevations shown in Table 4.

Table 4. Predrilled Elevations

Location	Predrilled Elevation
Abutment 1	423.8 m (1390.6 ft)
Abutment 2	423.4 m (1389.4 ft)

4. The Contractor should anticipate hard and erratic driving of the steel "H" piles due to the presence of very dense foundation material. The Contractor should anticipate field cutting and splicing of the steel "H" piles. Refer to the LOTB sheets for details.

5. At the abutment locations, where new fill is being placed, a 120-day waiting period is to be required prior to installing the steel "H" piles. The Contractor may choose to monitor the settlement by the use of settlement monuments. If it is determined that the settlement has ceased prior to the end of the 120-day waiting period, with the written approval of the Engineer, waiting period may be terminated at that time.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-1035, Jacqueline Martin at (916) 227-1051, Reza Mahallati at (916) 227-1033 or Reid Buell at (916) 227-1012, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard Special Provisions 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:

A. Log of Test Borings for East Hill Road UC, Bridge No. 10-0157.

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

A. Revised Foundation Report for East Hill Road UC, Bridge No. 10-0157, dated June 18, 2008.

Report by:

TIM ALDERMAN
Engineering Geologist
Office of Geotechnical Design-North

Report by:

FOR JACQUELINE MARTIN

JACQUELINE MARTIN
Engineering Geologist
Office of Geotechnical Design-North

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REZA MAHALLATI
Senior Materials & Research Engineer
Office of Geotechnical Design-North

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

REFERENCES

Federal Highway Administration (FHWA), Design and Construction of Driven Pile Foundations: Workshop Manual – Volume 1 (Pub. No. FHWA HI-97-013) published January 1997, Revised November1998.

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Jennings, Charles W. and Strand, Rudolph G., 1960, Geologic Map of California [Ukiah Sheet] and Index to Geologic Mapping, California Division of Mines, scale 1:250,000, 2 sheets.

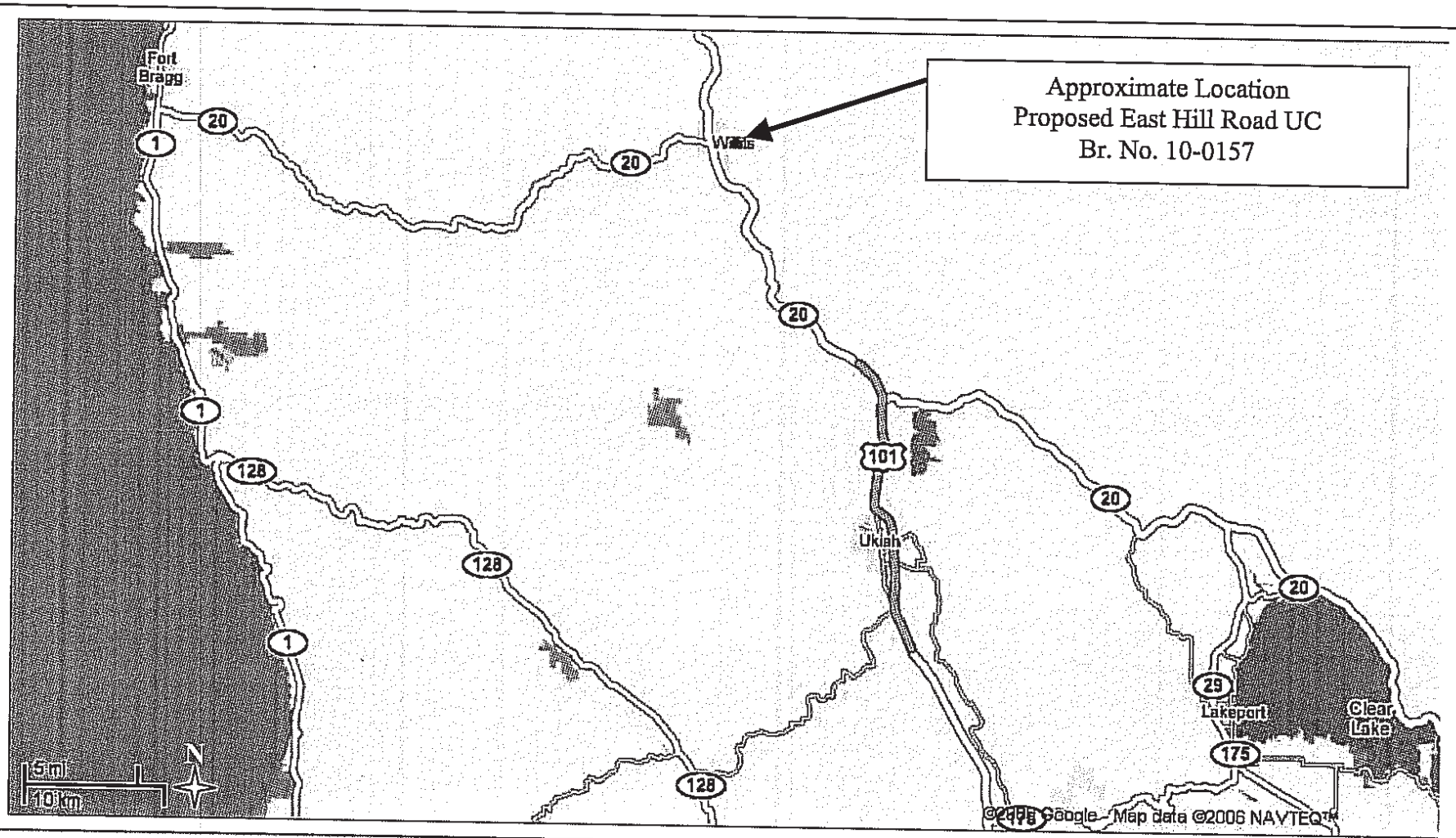
Norris, Robert M., and Webb, Robert M., 1990, Geology of California, Second Edition, John Wiley & Sons, Inc., pp 57, 359, 363.

Seismic Design Criteria (2006), California Department of Transportation, Version 1.4.

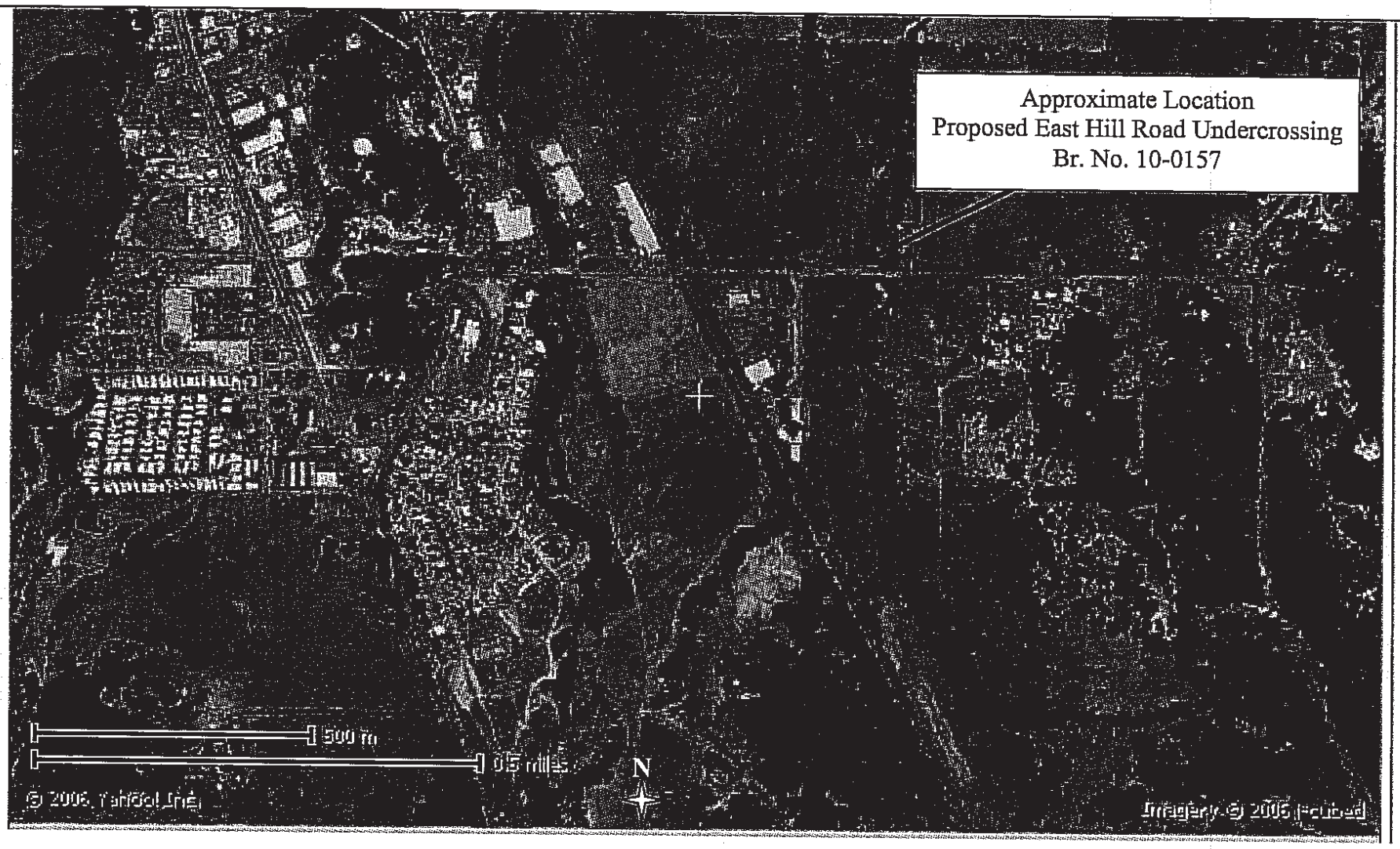
State of California, Department of Transportation (Caltrans):

- Standard Plans, May 2006
- Standard Specifications, May 2006
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

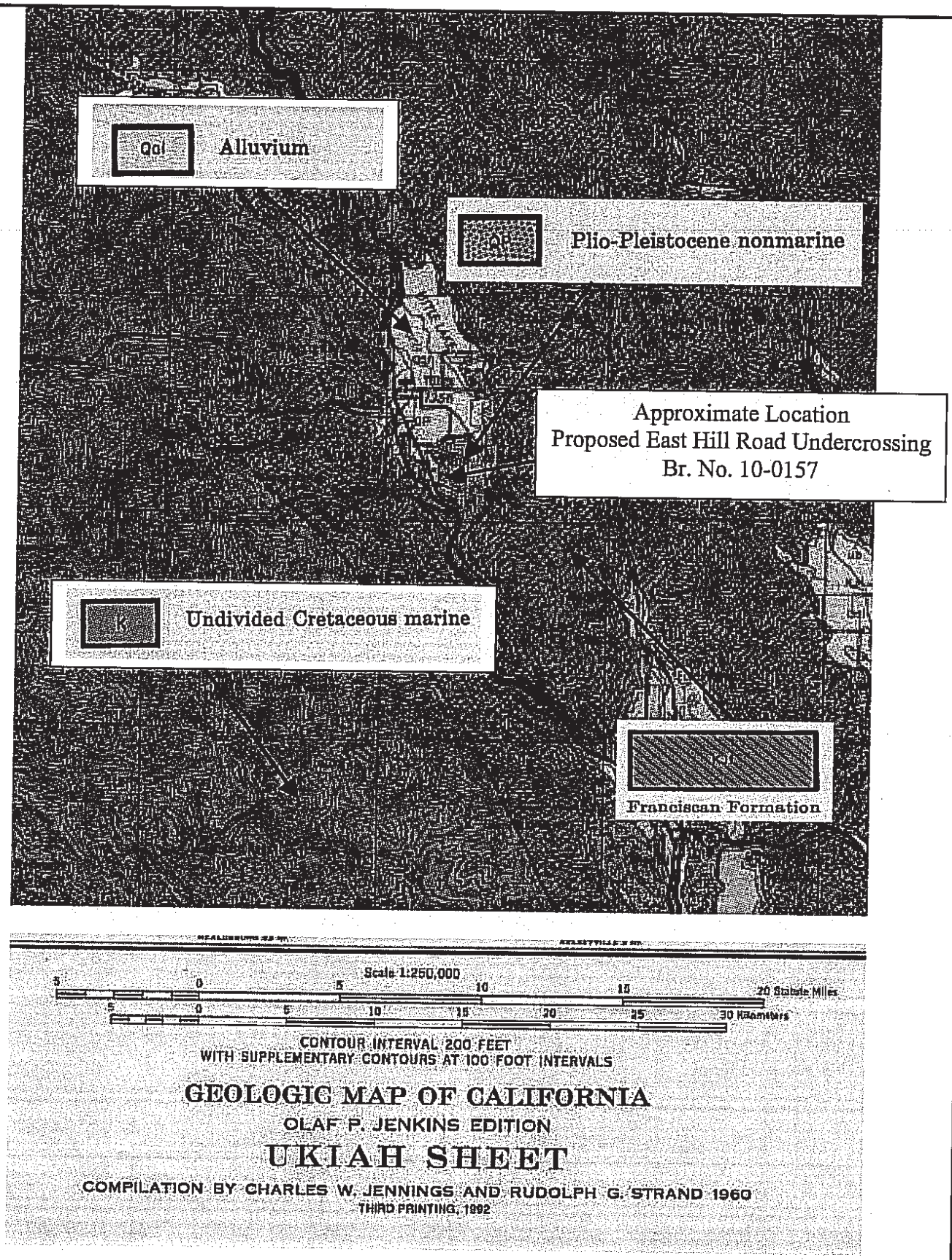
Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 June 18, 2008	Location Map
	01-MEN-101 KP R72.39/PM R44.98 **East Hill Road UC, Br. No. 10-0157**	Figure 1



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 June 18, 2008	Location Map
	01-MEN-101 KP R72.39/PM R44.98 **East Hill Road UC, Br. No. 10-0157**	Figure 2



Caltrans **Division of Engineering Services** Geotechnical Services Geotechnical Design – North	EA: 01-262001 June 18, 2008	Geologic Map
	01-MEN-101 KP R72.39/PM R44.98 **East Hill Road UC, Br. No. 10-0157**	Figure 3

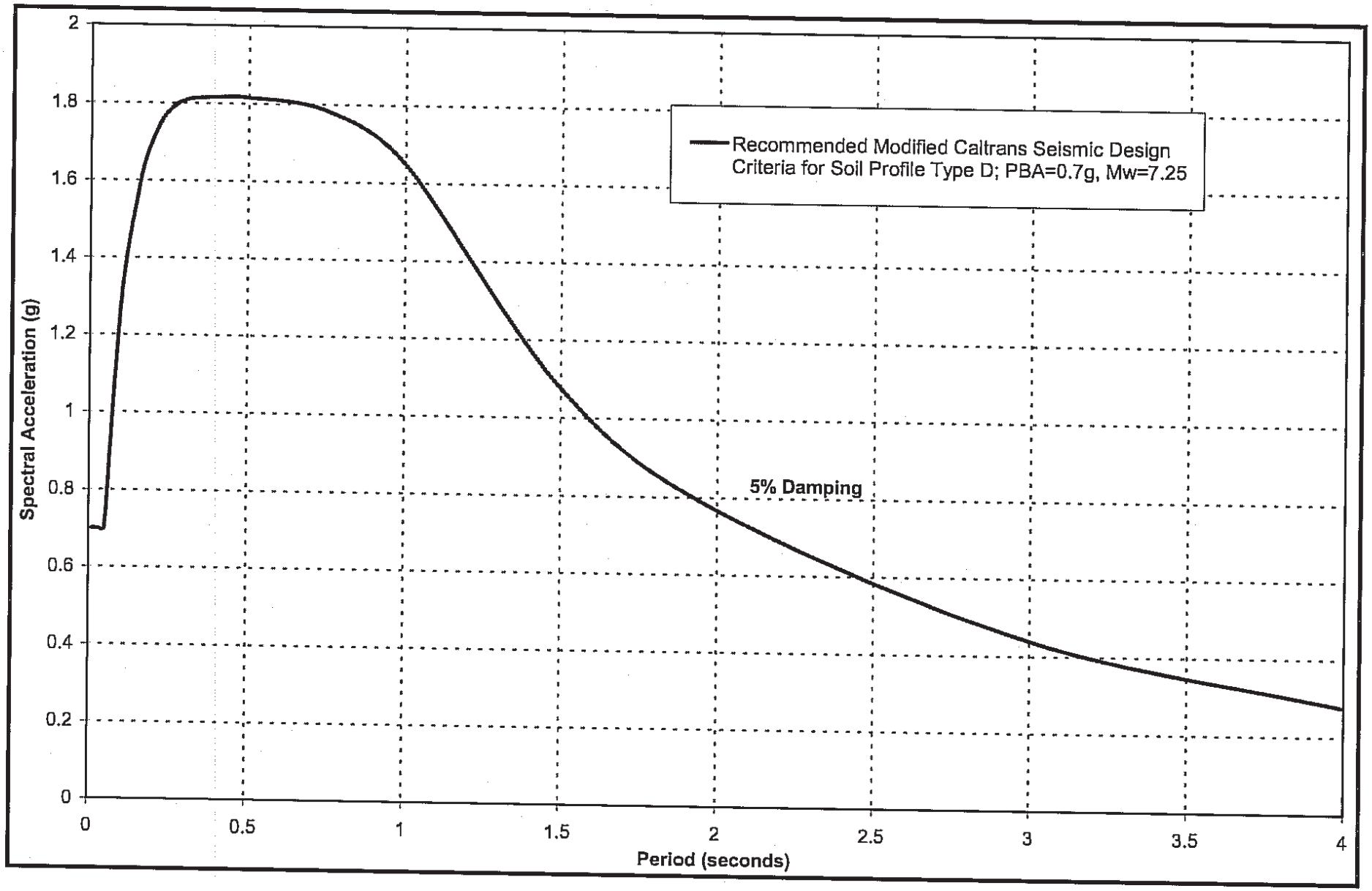


Figure 4. Acceleration Response Spectrum Recommended for Design

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. GUDMUND SETBERG
Senior Bridge Engineer
Division of Structure Design
Office of Bridge Design North
Bridge Design Branch 2

Attention: Mr. Marc Friedheim

Date: October 27, 2006

File: 01-MEN-101-KP R70.62/PM R43.88
EA#01-262001
E20-N101 Connector Bridge (NEW)
Br. No. 10-0129G

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services – MS 5
Office of Geotechnical Design – North

Subject: Foundation Report

Introduction/Scope of Work

This report presents the foundation recommendations for the new proposed E20-N101 Connector Bridge (Br. No. 10-0129G). The Office of Geotechnical Design North, Branch A completed a subsurface investigation from April 2004 to July 2004 following the request by the Office of Bridge Design North (OBDN), Branch 2 for foundation recommendations for the proposed new structure.

The following foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (April 2004 to July 2004) along with a review of the available General Plan dated April 27, 2006 and Foundation Plan dated September 13, 2006. With regards to the current foundation recommendations given in this report, elevations are based on the NGVD 29 vertical datum, unless otherwise noted.

Project Description

The proposed new E20-N101 Connector Bridge (Br. No. 10-0129G) will be incorporated as part of the Willits Bypass Project. The proposed new bridge will consist of one new structure and will be located within the city limits of Willits in Mendocino County (Figures 1 and 2). The structure will consist of a single span with cast-in-place/ reinforced concrete box girder (4 cell) type structure with diaphragm abutments and wingwalls supported on driven steel H-Piles.

Regional Setting and Area Geology

The project is located within the northern section of the Coast Ranges province. West of the Great Valley province, the Coast Ranges province stretches about 960 kilometers from the Oregon border to the Santa Ynez River. The province contains many elongate ranges and narrow valleys that are approximately parallel to the coast, although the coast usually shows a somewhat more northerly trend than do the ridges and valleys. The chain contains dominantly sedimentary rocks underlain by two unlike kinds of basement rocks that are mostly of middle Mesozoic age, the Franciscan Formation and granitics with associated meta-sedimentary rocks. Geologic history of the Coast Ranges is intricately interwoven with the tectonics of the San Andreas and other major faults, particularly those in the western part of the state (Norris & Webb, 1990).

Locally, the project site is located at the south end of Little Lake Valley. Little Lake Valley is an intermontane basin within the northern California Coast Ranges in Mendocino County that contains a record of sedimentation and deformation during the Pleistocene. The basin is approximately 11.3 kilometers in length and up to approximately 4.8 kilometers wide near the middle of the valley. The town of Willits is situated in the west-central portion of the valley. The basin was likely formed by faulting along the Maacama Fault Zone on the southwest and northeast margins of the valley. Sediments within the basin, in places over 140 m thick, consist of fine grained lacustrine and overbank deposits, and coarser grained gravel alluvium from through flowing streams. Geologic investigations of the coarse-grained gravel alluvium suggest the valley formerly drained to the south (Russian River drainage). Currently, the valley drains to the north into the Eel River system. A suggested cause for the stream drainage changes may be from tectonic stresses related to the northward progression of the Mendocino Triple Junction (Woolace, *et al.*, 2005).

The Geologic Map of California, Ukiah Sheet, scale 1:250,000, compiled by Charles W. Jennings and Rudolph G. Strand, 1992, California Division of Mines and Geology (Currently known as the California Geological Survey) indicates the area geology consists of Quaternary alluvium (Qal) deposits overlying Plio-Pleistocene nonmarine sedimentary deposits underlain by the Franciscan Formation (KJf) (Figure 3). According to the map explanation, the Quaternary alluvium deposits consist of alluvium; recent breccia, conglomerate, sand, and valley fill. The Plio-Pleistocene (Pliocene-Pleistocene nonmarine sedimentary deposits) deposits are identified as the Cache Formation that consists of laucustrine and fluvial deposits and unnamed Plio-Pleistocene deposits bordering alluviated valleys in Mendocino County. The Franciscan Formation consists of sandstone, shale, chert, and conglomerate with locally small areas of greenstone, limestone, basalt, glaucophane schist and related metamorphic rocks.

At the project site, the materials encountered during the subsurface investigation are interpreted as deposits of the Plio-Pleistocene nonmarine sedimentary deposits identified as the Cache Formation. Refer to the field investigation section of this report for more detailed information.

Field Investigation and Subsurface Conditions

The Office of Geotechnical Design-North conducted a subsurface investigation in April 2004 to July 2004.

The subsurface investigation consisted of two mud rotary borings (Nos. B-7, and B-11) and one cone penetrometer sounding (No. C-12). The mud rotary borings were advanced using a self-casing wireline-diamond coring method extending down to a maximum depth of 30.9 m (101.5 ft). The equipment used to drill B-7 consisted of an Acker drill rig equipped with a Central Mine Equipment (CME) automatic hammer. The equipment used to drill B-11 consisted of a Mobile B-46 drill rig equipped with a safety hammer. Sampling was achieved by utilizing the Standard Penetration Test (SPT) sampler at 1.5-meter intervals. Selected soil samples were bagged for laboratory testing. The cone penetromer sounding was used to supplement the subsurface investigation.

The subsurface investigation revealed the foundation materials encountered at the location of the proposed bridge (Br. No. 10-0129G) generally consist of approximately 13.4 m (44.0 ft) of alternating layers of stiff to hard clay, very dense sand and clayey sand overlying hard clay and sandy clay. The foundation materials were encountered down to the maximum depth explored of 30.9 m (101.5 ft), an elevation of 408.9 m (1341.6 ft).

The elevations shown on the Log of Test Borings are based on the NGVD 1929 vertical datum.

For subsurface data and boring locations, please refer to the Log of Test Borings for site-specific information and conditions. These sheets will be forwarded once completed.

Ground Water

Ground water was measured in both mud rotary borings (B-7 and B-11) drilled during the subsurface investigation. Ground water levels varied from elevation 443.6 m (1455.4 ft) in boring B-7 to elevation 439.9 m (1443.2 ft) in Boring B-11. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at time of construction. For more details, please refer to the LOTB sheets.

Scour Evaluation

A Final Hydraulic Report (dated March 22, 2005) for the Haehl Creek structures was completed by the Office of Structure Maintenance and Investigations, Structure Hydraulics Branch. According to this report, there are no local scour concerns for this structure since it is a single span structure. The report recommends the abutment foundations be designed assuming no ground support (lateral and vertical) as a result of soil loss due to possible degradation or lateral stream migration down to the current stream thalwag elevation (lowest elevation in channel). The approximate thalwag elevation provided for the proposed E20-N101 Connector Bridge (Br. No. 10-0129G) is 443.7 m (1455.7 ft).

Rock Slope Protection (RSP) was mentioned and will be designed by the District to protect the roadway approach fills, if required.

For further information including site-specific scour assessment and mitigation measures, the Structures Hydraulics Branch should be contacted.

Corrosion Evaluation

Composite soil samples were collected from one of the two borings during the 2006 subsurface investigation. The Office of Testing and Technology Services, Corrosive Technology Branch tested the composite samples for corrosive potential. The Corrosion Technology Branch considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: chloride concentration is 550 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. The minimum resistivity serves only as an indicator parameter for the possible presence of soluble salts and is not included to define a corrosive site. It is the practice of the Corrosion Technology Branch that if the minimum resistivity of the sample is greater than 1000 ohm-cm, the sample is considered to be non-corrosive and testing to determine the sulfate and chloride content is not performed.

The Office of Testing and Technology Services results of the laboratory tests determined that one of the composite samples collected was considered to be corrosive at this site. The designer should consider remedial measures to protect against a corrosive environment at all foundation locations. Refer to Table 1 below for specific test results.

Table 1: Corrosion Test Summary-Composite Samples for E20-N101 Connector Bridge (Br. No. 10-0129G)

SIC Corrosion Number	Boring Number	Sample Depth (m)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (PPM)	Chloride Content (PPM)
C538124	B-11	0.0-1.52	5.31	5500	N/A	N/A
C538125	B-11	9.75-10.36	7.71	2000	N/A	N/A

Laboratory Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained from the subsurface investigation. Tests were performed to determine the corrosion and engineering properties of the subsurface materials for use in the foundation analysis. The tests performed included: mechanical analysis (Sieve and Hydrometer), Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), Unit Weight, Consolidation, Triaxial (UU) and Soil Corrosion Testing (pH, sulfate, chloride, and resistivity). All tests were performed in general accordance with American Society for Testing and Materials (ASTM) standards or California Test Methods (CTM). Laboratory test results will be available upon request once the results are received.

Seismic Data and Evaluation

The project site is not located within any Alquist-Priolo Earthquake Fault Zones (EFZs) as established by the California Geological Survey (CGS, 1997). Based on the Department of Transportation (Caltrans) 1996 Seismic Hazard Map, the controlling fault for the site is the Maacama Brush Mountain (MLM), a strike-slip fault. The MLM fault is located approximately 1.2 kilometers (0.7 miles) west of the site and is capable of generating a Maximum Credible Earthquake (MCE) moment magnitude of M_w=7.25. The corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.7g. The soil profile at the site may be classified as Type D, as defined in the Department's Seismic Design Criteria (SDC, 2006, Version 1.4). The recommended design Acceleration Response Spectrum (ARS) curve shown in Figure 4 was obtained by modifying the ARS curve in Figure B.8 of the SDC corresponding to a PBA of 0.7g. These modifications were introduced to account for the proximity of the controlling seismic source and included a 20% increase in spectral accelerations for periods greater than or equal to 1.0 second. No modifications were introduced for periods less than or equal to 0.5 second. The spectral accelerations for periods between 0.5 and 1.0 second were obtained by linear interpolation.

The potential for ground rupture hazard due to fault movement is considered low since no known fault crosses the project site. The potential for liquefaction to occur is considered to be minimal at the project site.

Foundation Recommendations

The following foundation recommendations are for the new proposed E20-N101 Connector Bridge (Br. No. 10-129G).

At all structure support locations, driven steel HP 250X85 "H" piles are recommended for support. The specified pile tip elevations (SPTE), shown below in Table 2, will provide piles with an ultimate geotechnical capacity that will meet the required nominal resistance in compression.

Table 2. Pile Data Table for the new proposed E20-N101 Connector Bridge (Br. No. 10-0129G).

Location	Pile Type	Design Load kN	Nominal Resistance		Cut-Off Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
			Compression kN	Tension kN			
Abutment 1	HP 250X85	400	800	0	445.4 (1461.3)	431.0 (1,2) (1414.0)	431.0 (1414.0)
Abutment 2	HP 250X85	400	800	0	445.3 (1460.9)	428.0 (1,2) (1404.2)	428.0 (1404.2)

Note: Design Pile Tip Elevation is controlled by the following demands: (1) *Compression (2) Scour potential exists to Elev. 443.7 m (1455.7 ft) at Abutments 1 and 2.*

General Notes to Designer

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

2. If lateral demands exist on the support piles, the structural design engineer shall indicate on the plans, in the pile data table, the design pile tip elevations required to meet the lateral load demands. If the specified pile tip elevations given in the above pile data table are not adequate for lateral load demands; the Office of Geotechnical Design-North, Structural Foundations shall be contacted for further recommendations.

Construction Considerations

1. Ground water was encountered during the subsurface investigation. Ground water surface elevation is subject to seasonal fluctuations and may occur higher or lower than indicated on the Log of Test Boring Sheets (LOTB) depending on the conditions at time of construction. Refer to the Log of Test Boring Sheets for details.

2. The steel "H" pile section below the scour elevation has been designed to develop the required nominal resistance.

3. The contractor should anticipate hard and erratic driving of the steel "H" piles due to the presence of very dense material. The contractor should anticipate field cutting and splicing of the steel "H" piles. Refer to the LOTB sheets for details.

4. It is anticipated that hard driving may exist during installation for some of the piles at Abutment 1 and 2 locations. Drilling to assist driving ("pilot" drill holes not greater than the least dimension of the pile) may be necessary. However, drilling to assist driving shall not be performed without prior authorization from this office.

5. At the Engineer's option, any steel piles driven within 2.0 meters of the specified pile tip elevation may be considered adequate and cut off if two times the required pile acceptance criteria is achieved. Refer to the Caltrans Standard Specifications 49-1.08 (2006) for information concerning the pile driving acceptance criteria.

The recommendations contained in this report are based on specific project information regarding design loads and structure locations that has been provided by the OBDN, Branch 2. If any conceptual changes are made during final project design, the Office of Geotechnical Design - North, Branch A 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 Tim Alderman at (916) 227-7260, or Reid Buell (916) 227-7190, of the Office of Geotechnical Design-North, Branch A.

Project Information

Standard special Provisions 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:

A. Log of Test Borings for E20-N101 Connector Bridge, Bridge No. 10-0129G.

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

A. Foundation Report for E20-N101 Connector Bridge, Bridge No. 10-0129G, dated October 27, 2006.

Report by:

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Office of Geotechnical Design-North

Report by:

Jacqueline Martin

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Reid Buell
R.E. Pending
Structure OE

REFERENCES

Federal Highway Administration (FHWA), Design and Construction of Driven Pile Foundations: Workshop Manual – Volume 1 (Pub. No. FHWA HI-97-013) published January 1997, Revised November1998.

Harden, Deborah R., 1998, California Geology, Prentice Hall, 252 pp.

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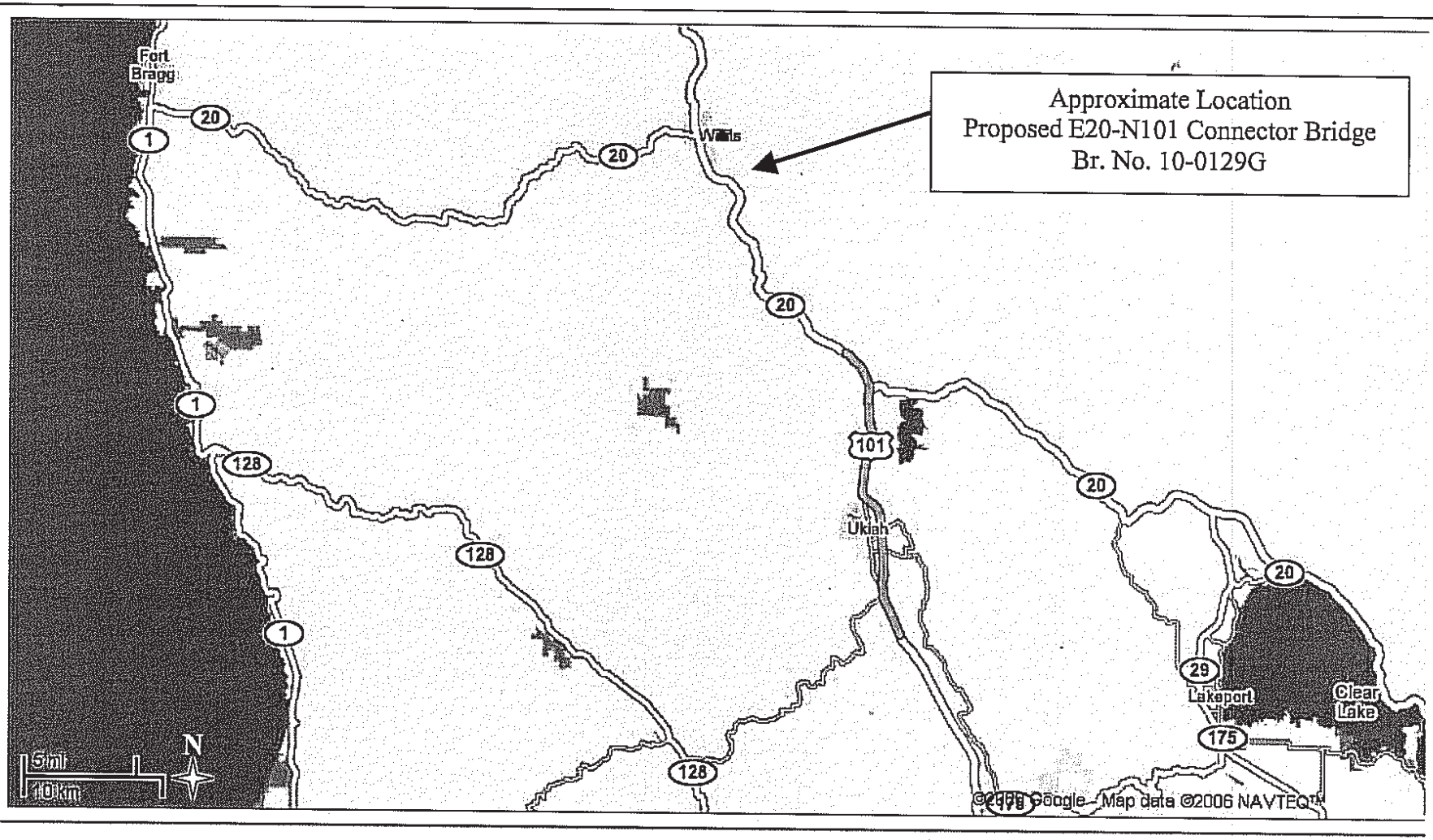
Norris, Robert M., and Webb, Robert M., 1990, Geology of California, Second Edition, John Wiley & Sons, Inc., pp 57, 359, 363.

Seismic Design Criteria (2006), California Department of Transportation, Version 1.4.

State of California, Department of Transportation (Caltrans):

- Standard Plans, May 2006
- Standard Specifications, May 2006
- Bridge Standard Details Sheets, April 2000.
- Memo to Designers, Section 3-1, December 2000.
- CT-Corrosion Guidelines, September 2003, Version 1.0.

Woolace, A. C. *et al.*, 2005, Late Neogene and Quaternary Stratigraphy and Structure of Little Lake (Willits) Valley, Northern Coast Range, California, in Abstracts with Programs, Vol. 37, No. 4, p. 68, Paper No. 25-3, Geological Society of America 101st Annual Meeting, April 29 through May 1, 2005, San Jose, CA.



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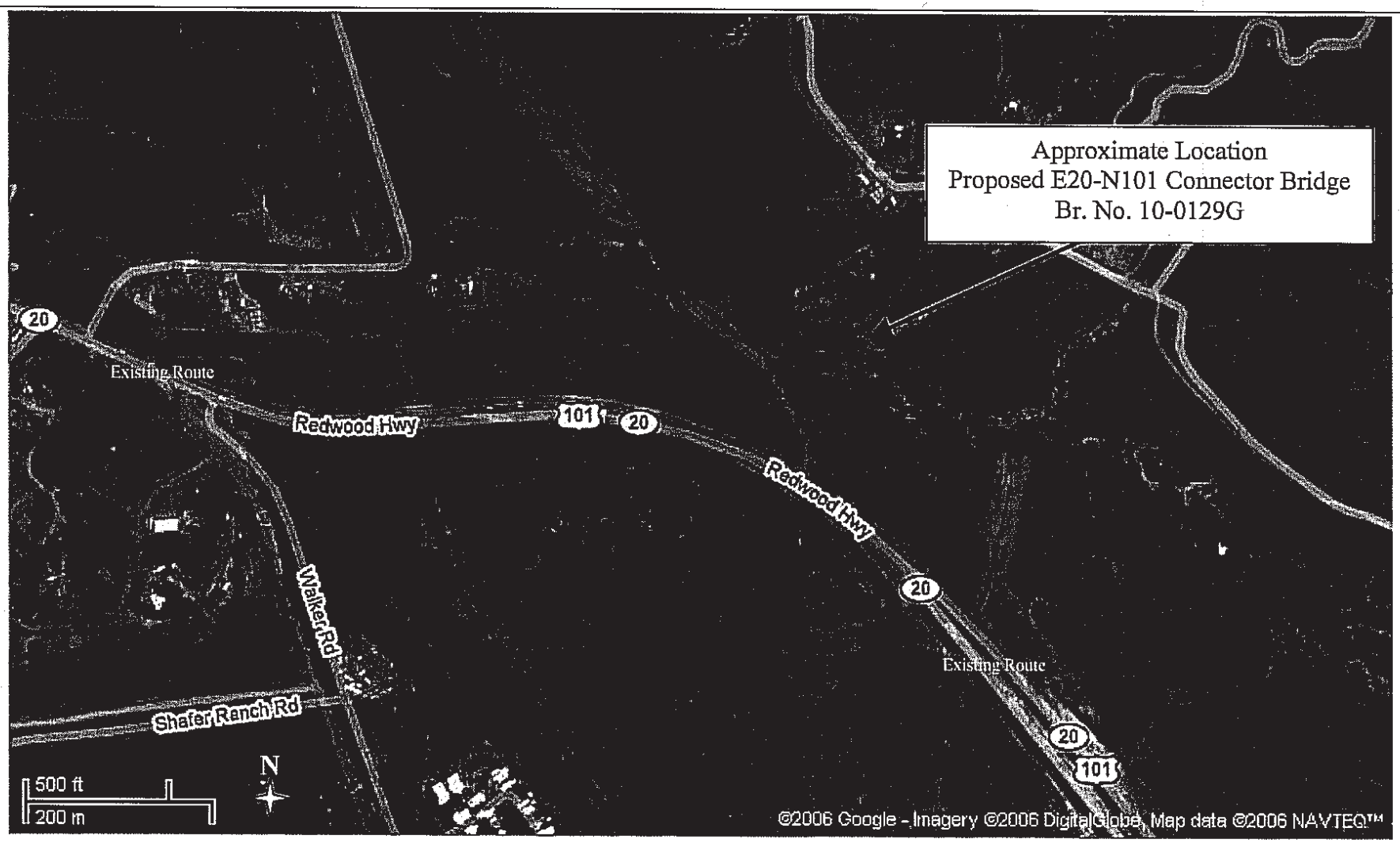
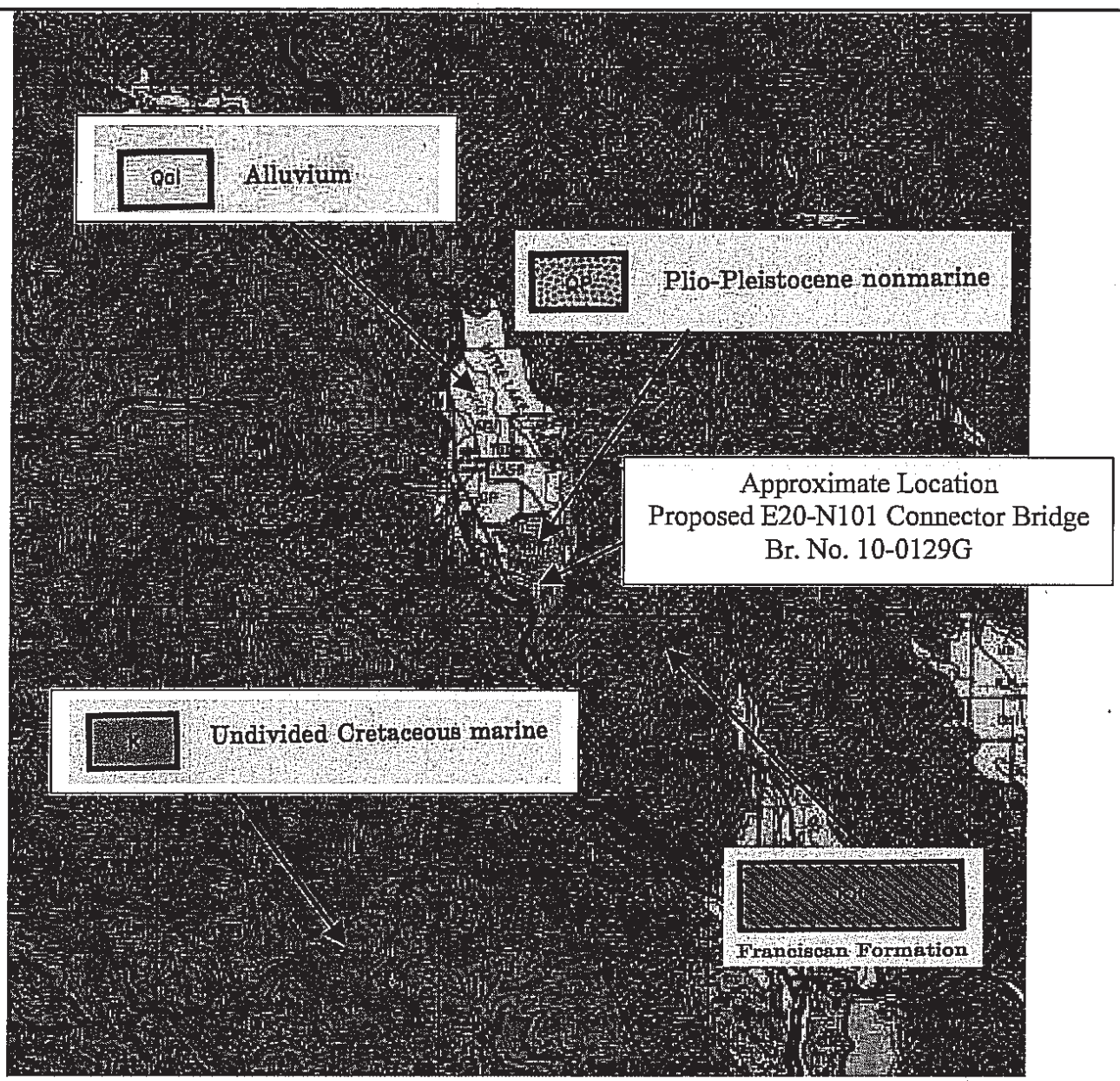
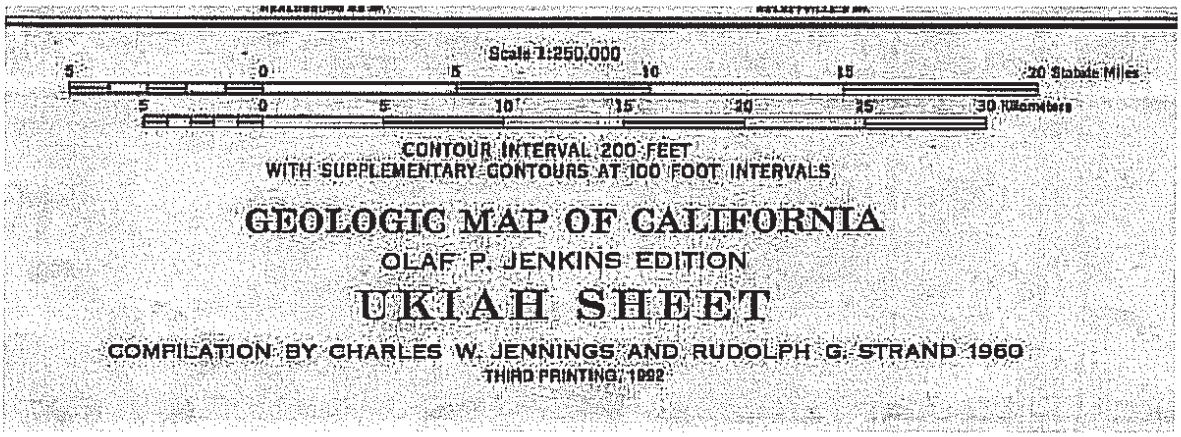
Division of Engineering Services
Geotechnical Services
Geotechnical Design – North

EA: 01-262001	Location Map
October 27, 2006	
01-MEN-101 KP R70.62/PM R43.9 E20-N101 Connector Bridge, Br. No. 10-0129G	Figure 1



Caltrans Division of Engineering Services Geotechnical Services Geotechnical Design – North	EA: 01-262001 October 27, 2006	Location Map
	01-MEN-101 KP R70.62/PM R43.88 E20-N101 Connector Bridge, Br. No. 10-0129G	Figure 2





Caltrans Division of Engineering Services Geotechnical Services Geotechnical Design – North	EA: 01-262001 October 27, 2006	Geologic Map
	01-MEN-101 KP R70.62/PM R43.88 E20-N101 Connector Bridge, Br. No. 10-0129G	Figure 3

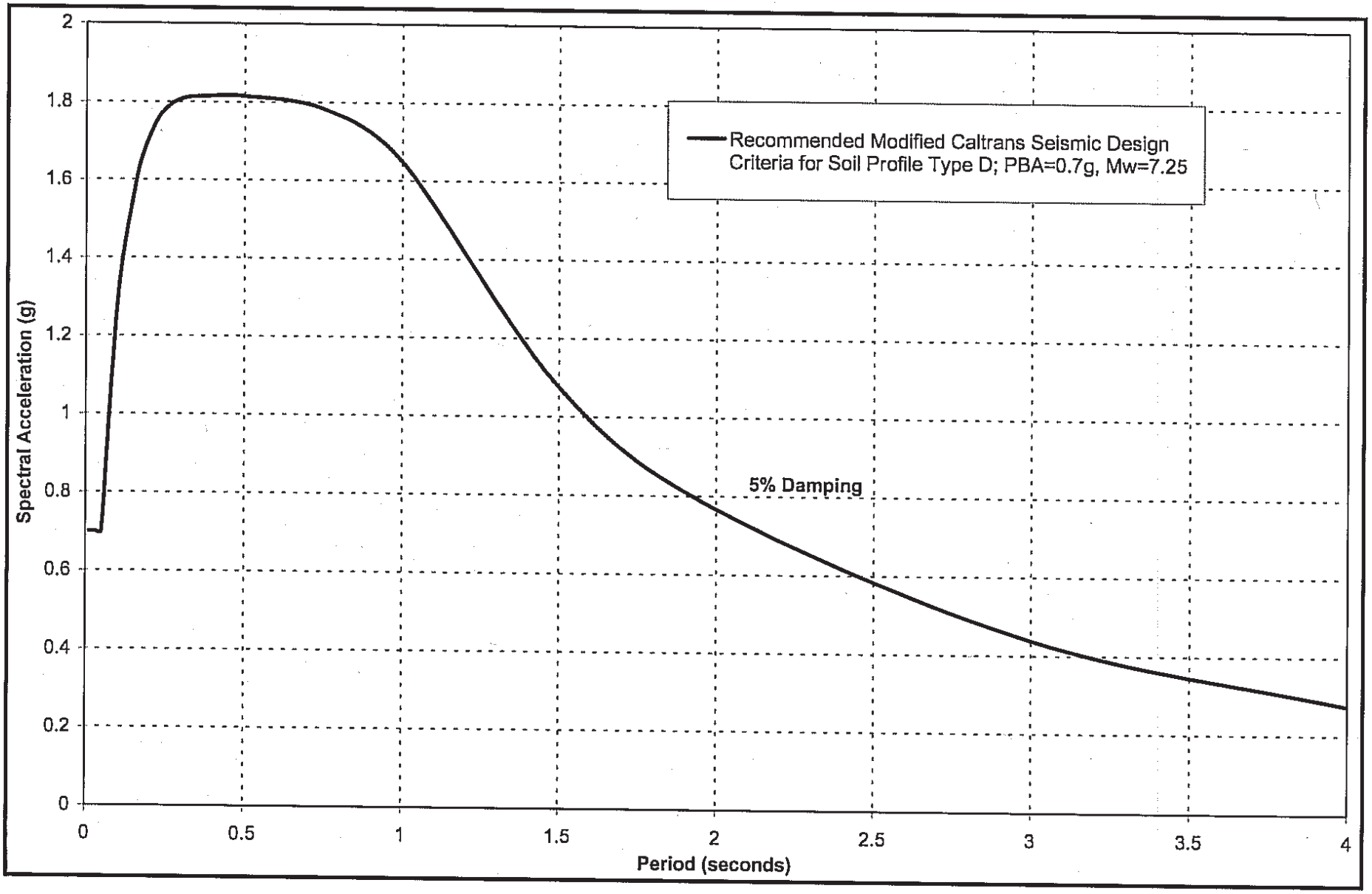


Figure 4. Acceleration Response Spectrum Recommended for Design