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CA PFH 112-1(2)

Final Foundation Design Report

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SECTION ONE - INTRODUCTION

1.1 BACKGROUND AND LOCATION

The CA PFH 112-1(2) project is to replace two deficient bridge structures along the South Fork Smith River Road within the Six River National Forest, Del Norte County, CA. The bridges are known as the Steven Memorial Bridge and the Hurdy Gurdy Creek Bridge. The bridge sites are located approximately 10 miles due east of Crescent City, CA. Small communities that exist within the local vicinity of the project include Smith River, Hiouchi, Gasquet, and Big Flat. A project vicinity map and a project location map are presented in Figure A1 of Appendix A.

The project site is primarily forested and mountainous. In the Smith River area, summers are relatively dry and mild, and winters are cool and wet. The project site is located within 12 miles of the Pacific coast, allowing marine air flows to moderate temperatures and increase humidity. Average temperatures range between 36° F and 72° F, with extreme temperatures ranging between 5° F and 110° F.

Precipitation in this area is seasonal, with about 90 percent of the mean annual percent occurring in the rainy season between October and April. The mean annual precipitation at the project site is estimated to be approximately 100 inches. During the rainy season, heavy storms move through the area from the Pacific Ocean and remain for several days depositing several inches of rain in a short period. Distribution of precipitation is influenced by the interaction of air flow and topography. Precipitation increases with elevation but also decreases with distance from the coast.

The existing Steven Memorial Bridge is a 14-foot wide, three-span structure that is approximately 330-feet long. It will be replaced with a 32-foot wide three-span bridge that is 470-foot long located on the downstream side of the existing structure. The existing Hurdy Gurdy Creek Bridge is a 14-foot wide, single-span structure approximately 170-feet long. It will be replaced with a 32-foot wide, single-span, 190-foot long bridge located on the upstream side of the existing structure. Staged construction is not anticipated at either bridge site.

This report presents the findings of the subsurface investigation programs for both bridge sites and provides geotechnical recommendations for the Steven Memorial Bridge and the Hurdy Gurdy Creek Bridge along the South Fork Smith River road.

SECTION TWO – GEOLOGY AND SEISMICITY

2.1 REGIONAL GEOLOGY

The South Fork Smith River Project site lies within the western foothills of the Klamath Mountains. Uplift of the Klamath Mountains is related to the movement of tectonic plates within the Cascadia Subduction Zone, as the ocean floor is sliding beneath the North American Plate. Various geologic materials on the oceanic plate have been broken off, bent, and added to the edge of the continent as a result of this sliding. This accumulation includes marine sedimentary and volcanic rocks from the ocean floor that have been metamorphosed to varying degrees. One of the geologic materials that has been added to the continental margin during subduction is peridotite. During the subduction process, the peridotite has been chemically weathered to varying degrees to produce serpentinized peridotite. Above the peridotite, is the Josephine Ophiolite, which lies on the South Fork Mountain thrust fault, above the younger Franciscan complex. The ophiolite mainly consists of basalt and gabbro.

The project location is mainly within ultramafic rocks of the western Jurassic belt, some of which are partially serpentinized. Rocks along the belt are generally volcanic in origin and also include peridotite, gabbro, and diabase. The surrounding area is dominated by classic Jurassic marine sedimentary rocks and greenstone of the Galice formation, as shown in Figure 2A of Appendix A.

The region is characterized by mountains with rounded ridges, steep sides, and narrow canyons. There are narrow floodplains and high terraces along the Smith River and its tributaries. Landslides, frequently caused by over saturated soils and hydrostatic forces, are a common feature of the landscape. Mass wasting, slope movement, and fluvial erosion are the main geomorphic processes.

2.2 SEISMICITY

The Klamath Mountains are traversed by many faults, including two major thrust faults that extend roughly north and south, and dip to the east. Numerous additional faults are located offshore within 14 miles of the Pacific coast. The Trinidad Fault and the Little Salmon Fault, south of the project site, may also affect the Klamath Mountains during seismic activity.

2.3 SEISMIC DESIGN PARAMETERS

Recommended seismic response parameters for use in design are based on the *"2008 Interim Revisions of the AASHTO LRFD Bridge Design Specifications, 4th Edition."* Design recommendations represent ground motion corresponding to an exceedance probability of 7% in

75 years for an earthquake with an approximate 1000-year return period. The main principles behind the AASHTO seismic LRFD specifications are: 1) small to moderate earthquakes should be resisted without significant damage, 2) realistic seismic ground motion intensities and forces should be used in the design, and 3) exposure to shaking from large earthquakes may cause damage to bridges, but the probability of collapse will be low and the damage will be limited to areas of the structure that are accessible for inspection and repair. The 1000-year return period data for the bridge site located at 41.75º N latitude and -123.99º W longitude, was obtained in accordance with the AASHTO ground motion maps for the probabilistic horizontal acceleration values corresponding to specific peak ground acceleration (PGA) and the spectral coefficients, namely the short- and long- period ground acceleration (S_s and S_1 respectively) for a certain soil profile at the bridge site.

The site soils are classified as ***"Class B"*** in accordance with the site class definitions specified in Table 3.10.3.1-1 of the 2008 Interim Revisions of the AASHTO LRFD Bridge Design Specifications, 4th Edition. Although the shear wave velocities of the soils were not measured, the top 100 feet of soils are mainly hard bedrock overlain with gravelly sand and silt materials. The recommended acceleration coefficient values for design with a return period of 1000-years were calculated using the program provided with the AASHTO LRFD Bridge Design manual and are summarized in Table 2.1.

TABLE 2.1:- Summary of Seismic Parameters for Earthquake with 1000-Year Return Period.

Site Soil Classification (Based on Soil Type and Profile)	**Class B**
Horizontal Peak Ground Acceleration, (PGA)	0.388g
Horizontal Response Spectral Acceleration at Period of 0.2 sec, (S_s)	0.910g
Horizontal Response Spectral Acceleration at Period of 1.0 sec, (S_1)	0.397g
Site Factor at Zero-Period of Acceleration Spectrum, (F_{pga})	1.00
Site Factor at Short-Period Range of Acceleration Spectrum, (F_a)	1.00
Site Factor at Long-Period Range of Acceleration Spectrum, (F_v)	1.00
Seismic Zone	**Zone 3**

The 5% damped design response spectrum for a total period of 2.000 seconds was calculated (Table 2.2) and is plotted in Figure 2.1. This spectrum is calculated using spectral acceleration coefficients listed in Table 2.2.

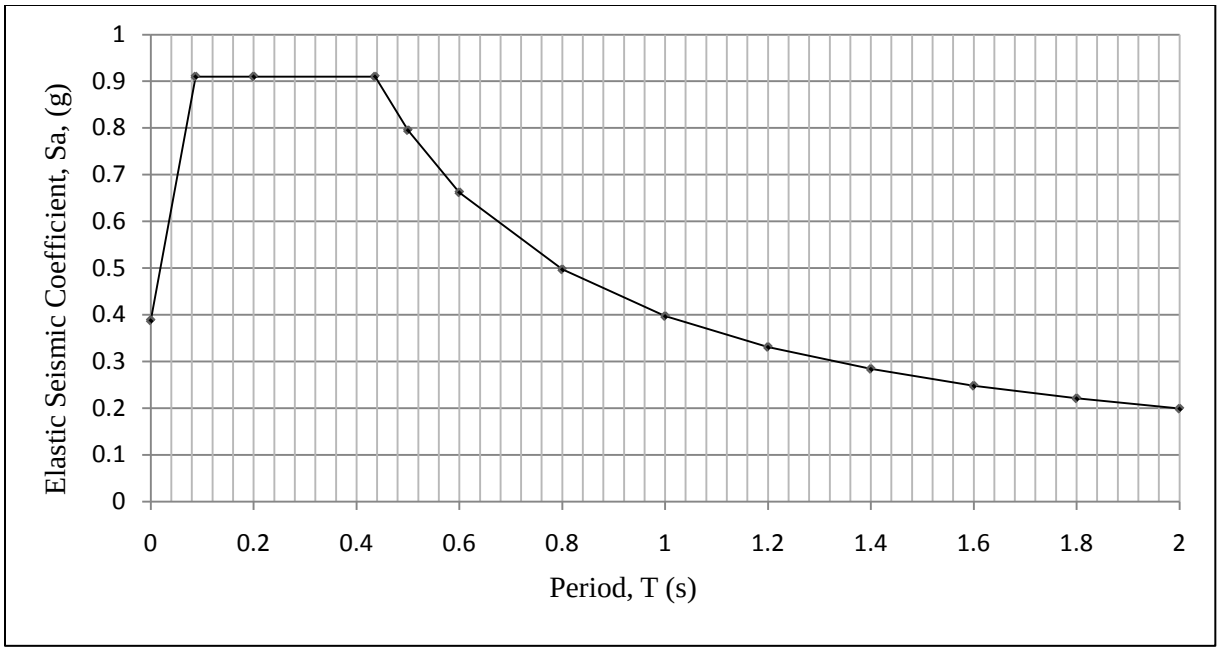


FIGURE 2.1:- Five-Percent-Damped Design Response Spectrum.

TABLE 2.2:- Summary of Design Response Parameters.

Time (sec)	Sa (g)	Sd (in)	Remarks
0.000	0.388	0.000	**T = 0.000, Sa = PGA**
0.087	0.910	0.068	**T =To, Sa = Ss**
0.200	0.910	0.356	**T = 0.200, Sa = Ss**
0.436	0.910	1.694	**T =Ts, Sa = Ss**
0.500	0.795	1.941	
0.600	0.662	2.329	
0.800	0.497	3.106	
1.000	0.397	3.882	**T = 1.0, Sa = S1** *Used for SD_1 and Zone Class Calculations*
1.200	0.331	4.658	
1.400	0.284	5.435	
1.600	0.248	6.211	
1.800	0.221	6.987	
2.000	0.199	7.764	

Based on an acceleration coefficient (SD_1, the seismic response motion acceleration at 1.000 second) value of 0.397g calculated as $F_v.S_1$, the bridge is assigned a seismic performance ***Zone 3*** in accordance with Table 3.10.6-1 in the AASHTO LRFD Bridge Design Specifications ($0.30 \leq F_v.S_1 \geq 0.50$). Seismic zones reflect the variation in seismic risk in different parts of the country and should be used for permitting different requirements for foundation design.

2.4 GEOLOGICAL HAZARDS

Ground motions caused by an earthquake are influenced not only by the distance from the fault planes, but also by the geology and soils found at the site. Amplified ground motions are expected in areas underlain by thick sand layers or alluvial materials within the Smith River watershed. Materials prone to liquefaction or amplified ground motions are not expected to be encountered at either of the bridge sites. However, intense ground motions may trigger landslides or rock falls especially following heavy rains when soils are saturated. Several slopes in the vicinity of the project sites have experienced massive landslides, but it is not known if they were primarily caused by ground motions or loss of soil shear strength caused by heavy rains.

Except for the gentle coastal plain, most of the Smith River Basin is inherently landslide prone (California Department of Fish and Game, 1980). Areas composed of weak rock, such as the Galice and Franciscan Formations, are especially subject to erosion, mass wasting, and sediment production.

In January 1970, following heavy rains, a large land mass slide from the northeast face of Rattlesnake Mountain, *near the project sites* blocked the South Fork Smith River. This slide, known as the Rattlesnake Slide, covered nearly a square mile and moved about 2 million cubic yards of materials (California Department of Fish and Game, 1980). The slide blocked the South Fork for several hours, creating a lake two miles long and 50 feet deep. When the river breached this blockage, a large wave of water was released, but caused little damage downstream. Roadway construction coupled with heavy rain likely caused the landslide.

There are over a thousand landslides in the Smith River Basin, including hundreds over 200-feet wide (California Department of Fish and Game, 1980). Landslide events, such as the Rattlesnake Slide on the South Fork and various smaller events are likely to continue to occur along the South Fork, especially during periods of high precipitation and as mass wasting and erosion continue along the canyon walls.

SECTION THREE – SUBSURFACE INVESTIGATION

3.1 FIELD EXPLORATION

Multi-phase subsurface investigation programs were conducted at both bridge sites to evaluate subsurface conditions required for foundation design. The field program mainly consisted of core drilling near pier and abutment locations of the proposed Steven Memorial and Hurdy Gurdy Creek Bridges. The initial subsurface investigation phase was conducted in September 2005 by Crux Subsurface Inc. using a Burley 5500-1 track mounted drill rig and logged on-site by a geologist from Central Federal Lands (CFL). Helicopter assistance was required to place the rig at the proposed pier locations of the Steven Memorial Bridge. During this field investigation, four borings, labeled B-SM1 through B-SM4, were advanced at the Steven Memorial Bridge site. Two additional borings, labeled B-SM5 and B-SM6, were advanced at the proposed abutment locations of the Steven Memorial Bridge in March 2010 by Diamond Core Company, using the Mobile B-80 drilling rig.

Six borings, labeled B-HG1 through B-HG6, were also advance at the Hurdy Gurdy Creek Bridge site during October, 2005 by Crux subsurface Inc., and logged on-site by Kleinfelder personnel. Borings were advanced with a Burley 5500-1 track mounted drill rig.

All borings conducted at both bridge sites were advanced to appropriate depths for deep foundations using a casing advancer system in overburden soils and either HQ-3 or NQ-3 triple tube coring systems in bedrock. Standard Penetration Tests (SPT) were performed using an automatic hammer of 80% efficiency in accordance with AASHTO T200-87. Samples were obtained in the overburden soils at 5-foot intervals using a standard 2-inch outside diameter split spoon sampler. The number of blows required to drive the split spoon sampler for three consecutive 6-inch intervals, or a total of 1.5 feet, was recorded on the test hole logs. Representative portions of recovered samples were preserved for laboratory testing. The sampling sequence within the borings is summarized on the final boring logs attached in Appendix B. Percent core recovery and rock quality designation (RQD) were also determined for each core run to provide a quantitative basis of the rock mass conditions. Core recovery is the ratio of the length of recovered material to the total length of the core run, presented as a percentage. The RQD is the ratio of the cumulative length of rock core pieces that are 4 inches in length or greater to the total length of the core run, per ASTM D-6032. Percent core recovery and RQD provide a qualitative indication as to the competency and structural integrity of the investigated rock mass. Boring location plans are provided in Appendix A. Photos of the drill rig set up at each boring location and recovered core samples are located in Appendix E.

3.2 SUMMARY OF SUBSURFACE EXPLORATION

Field test results were recorded on field logs and appropriate data sheets at the time of the

investigation. The data sheets and logs contain information concerning the boring methods; samples attempted and recovered; indications of the presence of various materials such as gravel, pebbles, organic matter, and groundwater. They also include laboratory test results and interpretations by the field personnel of the conditions based on the performance of the drilling equipment and cuttings brought to the surface.

Specific site descriptions are based on field observations, subsurface characterization information, and may vary in material type and strength with distance away from test boring locations. This variability is due to topography and land forming process.

Groundwater elevations were not measured during the subsurface investigation due to the use of water in the wire-line coring process. Fluctuations in the ground water level due to seasonal and climatic effects are, however, expected. The following is a detailed description of the materials encountered at each bridge site:

I. THE STEVEN MEMORIAL BRIDGE SITE:

A total of six borings, labeled B-SM1 through B-SM6, were completed near the proposed bridge abutments and pier locations. Table 3.1 provides a summary of the boring information for the Steven Memorial Bridge.

TABLE 3.1:- Summary of Borings for the Steven Memorial Bridge Site.

Boring No.	Foundation	Boring Depth, ft	Station	Offset**	Ground Elevation, ft	Bedrock Elevation, ft
B-SM1	Abutment 1*	39	608+40	Centerline	568.0	559.6
B- SM2	Pier 1	30	609+40	Centerline	535.0	535.0
B- SM3	Pier 2	31	611+20	3-ft RT	532.0	520.2
B- SM4	Abutment 2	70	612+25	10-ft RT	580.0	573.5
B- SM5	Abutment 1*	62.5	608+00	5-ft RT	576.0	562.4
B- SM6	Abutment 2*	71.5	612+60	5-ft LT	588.0	571.0

* Abutment locations were moved following the initial subsurface investigation of the site.
** Offset is from proposed bridge centerline.

Abutment 1 (Station 607+90 and Elevation 587.85) - Borings B-SM1 and B-SM5: Boring B-SM1, located at Station 608+40 near centerline of proposed west abutment, was advanced using the HQ-3 wire line rock core system to a total depth of 39 feet from a starting elevation of 568.0 feet. The subsurface materials encountered were 8.4 feet of dense, slightly moist, red brown and grey clays and silty sand, gravels and cobbles overlaying white green Serpentinite. An SPT N value of 47 was recorded in the overburden soil layer. The coring in the bedrock consistently produced 100 percent recoveries and an RQD that ranged between 48 and 100 with an average of 83. Unconfined compressive strength (UCS) of bedrock was

measured in the laboratory on three core samples obtained between 20 and 30 feet deep were 18,540, 20,390, and 21,230 psi with an average UCS of 20,053 psi.

Boring B-SM5, located at Station 608+00 about 5 feet to the right of the proposed west abutment centerline, was advanced using a casing advancer system to a depth of 3 feet and then cored using the NQ-3 wire line rock core system for the total depth of the boring of 62.5 feet from a starting elevation of 576.0 feet. The subsurface materials encountered were 13.6 feet of dense, red brown and grey clayey and silty sand, gravels, cobbles, and boulders overlaying white green Serpentine. Percent recovery in bedrock generally varied between 82 and 100 percent and the RQD ranged between 16 and 96 with an average of 59, generally increasing with depth. A 3-foot thick, highly fractured zone (36 percent recovery and 0 RQD) was encountered at an elevation of approximately 556.0 feet.

Pier 1 (Station 609+35, Elevation 587.13) – Boring B-SM2: Boring B-SM2, located at Station 609+40 near centerline of proposed bridge pier 1 location, was advanced using the HQ-3 wire line rock core system for the total depth of the boring of 30.0 feet from a starting elevation of 535.0 feet. The subsurface materials encountered were white green Serpentinite with Basalt intrusion. Percent recovery in bedrock was consistently measured at 100 percent with the exception of the first 5 feet which measured at 94 percent and the RQD ranged between 60 and 84 with an average of 72, generally increasing with depth. Unconfined compressive strength (UCS) of bedrock was measured in the laboratory on two core samples obtained between 6 and 14 feet deep were 9,280 and 11,590 psi with an average UCS of 10,435 psi

Pier 2 (Station 611+15, Elevation 586.23) – Boring B-SM3: Boring B-SM3, located at Station 611+20 approximately 3-feet right of proposed bridge pier 2 location, was advanced using the HQ-3 wire line rock core system to a total depth of 31 feet from a starting elevation of 532.0 feet. The subsurface materials encountered were 11.8 feet of dense, grey and silty sand with gravel and cobbles overlaying white green Serpentinite. An SPT N value of 43 was recorded in the overburden soil layer and encountered refusal at 11.8 feet. The coring in the bedrock consistently produced 100 percent recoveries and an RQD that ranged between 90 and 100 with an average of 94.

Abutment 2 (Station 612+60, Elevation 585.5) – Borings B-SM4 and B-SM6: Boring B-SM4, located at Station 612+25 10-feet right of centerline at proposed east abutment, was advanced using the HQ-3 wire line rock core system to a total depth of 70 feet from a starting elevation of 580.0 feet. The subsurface materials encountered were 6.5 feet of medium dense to dense, slightly moist, yellow brown clayey sand, with cobbles and boulders, with an SPT N-value of 22, overlaying a 23.5-foot layer of decomposed to moderately weathered green Serpentine with high percent recovery ranging between 75 and 100 percent with very low

RQD values (RQD = 0 to 18). A more competent, less weathered Serpentine layer was encountered at elevation 550.0 between 30 feet and 70 feet. Percent recovery in bedrock generally varied between 20 and 100 percent and the RQD ranged between 0 and 78 with an average of 38. A highly fracture zone (0 RQD) was also encountered at elevation 530.0 between approximately 50 and 60 feet in depth.

Boring B-SM6, located at Station 612+60 approximately 5 feet to the left of the proposed east abutment centerline, was advanced a casing advancer system to a depth of 5 feet and then cored using the NQ-3 wire line rock core system for the total depth of the boring of 71.5 feet from a starting elevation of 588.0 feet. The subsurface materials encountered were 17.0 feet of red brown to grey clay sand, gravels, cobbles, and boulders overlaying a highly fractured layer of dark green Serpentine to a depth of 35.3 feet. The percent recovery within this layer was very low with several runs producing no recovery and RQD of 0. Green weathered Serpentine bedrock layer was encountered between 35.3 feet and 71.5 feet. Percent recovery in bedrock generally varied between 17 and 100 percent and the RQD ranged between 0 and 50 with an average of 17.

II. THE HURDY GURDY CREEK BRIDGE SITE:

A total of six borings, labeled B-HG1 through B-HG6, were completed near the proposed bridge abutments. Table 3.2 provide a summary of the boring information for the Hurdy Gurdy Creek Bridge.

TABLE 3.2:- Summary of Borings for the Hurdy Gurdy Bridge Site.

Boring No.	Foundation	Boring Depth, ft	Station	Offset *	Ground Elevation, ft	Bedrock Elevation, ft
B-HG1	Abutment 1	34	808+10	Centerline	621.0	612.0
B-HG2	Abutment 1	47	808+40	Centerline	612.5	590.5
B-HG3	Abutment 2	30.5	809+90	17-ft RT	625.0	603.0
B-HG4	Abutment 2	27	809+90	15-ft LT	616.0	601.0
B-HG5	Abutment 2	21	810+15	15-ft LT	625.0	612.5
B-HG6	Abutment 1	28.4	808+32	17-ft RT	614.0	593.0

* Offset is from proposed bridge centerline

Abutment 1 (Station 808+25, Elevation 635.25) – Borings B-HG1, B-HG2, and B-HG6:
Boring B-HG1, located at Station 808+10 near centerline of proposed west abutment, was advanced using the HQ-3 wire line rock core system to a total depth of 34 feet from a starting elevation of 621.0 feet. The subsurface materials encountered were 9 feet of brown silty sand with rock fragments, classified by the Unified Soil Classification System (USCS) as SM in the laboratory, overlaying highly fractured (along a 45 degree cleavage planes) dark gray Slate . An SPT N value of 16 was recorded in the overburden soil layer. The coring in the bedrock

consistently produced high recovery percentages, ranging between 60 and 100 percent and zero RQD.

Boring B-HG2, located at Station 808+40 near centerline of proposed west abutment was advanced using the HQ-3 wire line rock core system to a total depth of 34 feet from a starting elevation of 612.5 feet. The subsurface materials encountered were 22 feet of moist dark brown silty sand with rock fragments, classified by the USCS as SM in the laboratory, overlaying highly fractured (along a 45 degree cleavage planes) dark gray to black Slate . An average SPT N value of 19 was recorded in the overburden soil layer. The coring in the bedrock produced recoveries ranging between 35 and 100 percent and zero RQD.

Boring B-HG6, located at Station 808+32 approximately 17 feet to the left of the proposed west abutment centerline, was advanced using the HQ-3 wire line rock core system to a total depth of 28.4 feet from a starting elevation of 614.0 feet. The subsurface materials encountered were 21 feet of brown silty sand with rock fragments, classified by the USCS as SC-SM in the laboratory, overlaying highly fractured (along a 45 degree planes) dark gray Slate. Bedrock coring consistently produced high percent recoveries ranging between 71 and 92 percent and zero RQD.

Abutment 2 (Station 610+32, Elevation 637.15) – Borings B-HG3 B-HG4, and B-HG5:
Boring B-HG3, located at Station 809+90 approximately 17 feet to the right of the proposed east abutment centerline was advanced using the HQ-3 wire line rock core system to a total depth of 30.5 feet from a starting elevation of 625.0 feet. The subsurface materials encountered were 22 feet of brown silty sand with rock fragments, classified by the USCS as SM in the laboratory, overlaying highly fractured (along a 45 degree planes) dark gray Slate. An average SPT N value of 22 was recorded in the overburden soil layer. The coring in the bedrock consistently produced high percent recoveries, ranging between 88 and 100 percent and zero RQD.

Boring B-HG4, located at Station 809+90 approximately 15 feet to the left of the proposed east abutment centerline was advanced using the HQ-3 wire line rock core system to a total depth of 27 feet from a starting elevation of 616.0 feet. The subsurface materials encountered were 15 feet of dark brown silty sand with rock fragments, classified by USCS as SM in the laboratory, overlaying highly fractured (along a 45 degree cleavage planes) dark gray to black Slate . A SPT N value of 16 was recorded in the overburden soil layer. The coring in the bedrock consistently produced high percent recoveries ranging between 89 and 100 percent and zero RQD.

Boring B-HG5, located at Station 810+15 approximately 15 feet to the left of the proposed east abutment centerline was advanced to a depth of 12.5 feet and then cored using the HQ-3 wire

line system to a total depth of 21 feet from a starting elevation of 625.0 feet. The subsurface materials encountered were 12.5 feet of brown silty sand with rock fragments, classified by the USCS as SM in the laboratory, overlaying highly fractured (along a 45 degree planes) dark gray Slate. A SPT N-value of 14 was recorded in the overburden layer. Bedrock coring consistently produced high percent recoveries, ranging between 92 and 94 percent, and zero RQD.

3.3 LABORATORY TESTING

Laboratory testing on recovered soil samples included gradation and classification in accordance with ASTM D 2487 and AASHTO M-145, and Atterberg limits in accordance with AASHTO T-89, T-90. Classification in this manner provides an indication of the soil's mechanical properties. A summary of the soil test results is shown in Table 3.3. Detailed laboratory test results are presented in Appendix C.

TABLE 3.3:- Summary of Laboratory Index Tests of Retrieved Soil Samples.

Boring Number	Station	SPT Sample No.	Sample Depth, ft	Percent Passing 200	LL	PI	USCS	AASHTO
B-SM1	608+40	SPT 1	4.0 to 5.5	24	36	8	SM	A-2-4
B-SM3	611+20	SPT 1 & 2	6.0 to 11.5.5	5.1	*	*	*	*
B-SM4	612+25	SPT 1	5.0 to 6.5	39	24	6	SC-SM	A-4
B-SM4	612+25	SPT 2	10.0 to 10.5	15	20	3	SM	A-1-b
B-HG2	808+40	SPT 1, 2 & 3	9.0-20.5	15	27	4	SM	A-1-b
B-HG3	809+90	SPT 1 &2	3.5-10.0	15	29	6	SM	A-1-a
B-HG4	809+90	SPT 1	3.5-5.0	24	25	3	SM	A-1-b
B-HG5	810+15	SPT 1	8.5-10.0	17	27	5	SM	A-1-b
B-HG6	808+32	SPT 1 & 2	9.0-15.5	17	26	5	SC-SM	A-1-b

* Indicates insufficient amount of material for completing the laboratory test.

Rock cores were obtained and representative rock core were selected and tested in the laboratory to determine the unconfined compressive strength (UCS) in accordance with ASTM D 2938 for use in bridge foundation design. During the test, an axial load is continuously applied and increased until a peak load and failure are observed. Each test was conducted on HQ size rock core, measuring 2.4 inches in diameter. A summary of the test results is presented in Table 3.4. Unconfined compressive strength (UCS) test results are contained in Appendix C.

TABLE 3.4:- Unconfined Compression Strength Test Results.

Boring Number	Station	Sample Depth, ft	Rock Type	UCS*, psi
B-SM1	608+40	20.00 to 20.80	Serpentine	18,540
B-SM1	608+40	22.00 to 23.00	Serpentine	20,390
B-SM1	608+40	29.00 to 30.00	Serpentine	21,230
B-SM2	609+40	6.70 to 8.00	Serpentine	11,590
B-SM2	609+40	12.50 to 14.00	Serpentine	9,280

*Uniaxial Compressive Strength

SECTION FOUR – ANALYSIS AND RECOMMENDATIONS

The site investigation program suggests that the general subsurface profile at the proposed bridge site abutment and pier locations can be characterized for *design purposes* as follows:

I. THE STEVEN MEMORIAL BRIDGE SITE:

Borings advanced at the proposed bridge abutment and pier locations encountered overburden soils consisting of silty sand with cobbles and boulders, ranging in thickness from 0 to 17 feet. Bedrock was encountered beneath the overburden soils. For design purposes, the elevation of bedrock was assumed to be 559.0 and 571.0 for Abutment 1 and 2, respectively, and 535.0 and 520.0 for Piers 1 and 2, respectively. Bedrock was encountered at the ground surface at the proposed Pier 1 location. Core percent recoveries within bedrock were consistently high, averaging over 90 percent with an average RQD of 57, indicative of *"fair"* quality bedrock.

For design purposes, the encountered overburden material was estimated to have a conservative unit weight (γ) of 120 pounds per cubic foot, a friction angle (φ) of 30 degrees, and no cohesion. A total of five unconfined compressive strength (UCS) tests were conducted on recovered core samples from borings at the proposed Steven Memorial Bridge location. Uniaxial compressive strengths of the Gneiss samples ranged from 9,280 to 21,230 psi, with an average UCS of approximately 16,200 psi.

II. THE HURDY GURDY CREEK BRIDGE SITE:

Borings advanced at the proposed bridge abutment locations encountered silty sand overburden with boulders, ranging in thickness between 9 and 22 feet. Bedrock was encountered beneath the overburden materials. At the proposed Abutment 1 location, bedrock was encountered at elevations ranging between 590.5 and 601.0 feet. At the proposed Abutment 2 location, bedrock was encountered at elevations ranging between 612.5 and 601.0 feet. Bedrock core recovery was consistently high, averaging over 90 percent. RQD's were zero for all completed core runs, due to the frequent cleavage planes persistent throughout the rock indicating *"poor"* rock quality.

For design purposes, the encountered overburden material was estimated to have a conservative unit weight (γ) of 120 pounds per cubic foot, a friction angle (φ) of 30 degrees, and no cohesion. Due to the fractured nature of the bedrock samples recovered at the proposed Hurdy Gurdy Creek Bridge abutment locations, neither unconfined compressive strength tests nor point load tests were completed. Due to the fractured and fissile nature of

the encountered bedrock, a presumptive unconfined compressive strength value of 5,000 psi is recommended.

4.1 FOUNDATION DESIGN RECOMMENDATIONS

Due to the relative depth of competent bearing material, environmental and aesthetic considerations, drilled shafts are recommended as bridge foundations at the abutment and pier locations for both bridge sites.

The drilled shaft analysis for each foundation unit is based on the lower shaft portion socketed into bedrock. The side resistance provided by overburden soils and by the first 5 feet of the bedrock socket at Abutments 1 and 2 for the Steven Memorial Bridge site (relatively low RQD) is neglected in the shaft capacity analysis. Based on load and resistance factor design (LRFD), the applied strength limit state resistance factor for base resistance in bedrock (for both bridge sites), per the Canadian Geotechnical Society, is 0.50. The applied strength limit state resistance factor for side resistance in bedrock (for both bridge sites), per Horvath and Kenney is 0.55. Because the UCS of the encountered bedrock is significantly higher than the compressive strength of concrete used in the shaft (4,000 psi), the compressive strength of the concrete governs and will be used in the drilled shaft design.

Since the rock socket is in *"fair"* to *"poor"* quality bedrock at the proposed abutments of the Steven Memorial Bridge and the Hurdy Gurdy Creek Bridge, the shaft side friction will be mobilized with a maximum vertical movement of 0.5 inches, which allows both the side friction and end bearing to be accounted for in the determining the geotechnical capacity of the drilled shaft. A minimum socket depth of 1.5 shaft diameters into bedrock is recommended. This embedment depth may be increased based on lateral and seismic capacity requirements. Table 4.1 shows the factored side resistance for various socket geometries based on a nominal side friction of 157 psi and a resistance factor of 0.55.

TABLE 4.1:- Factored Side Resistance for Various Rock Socket Geometries.

Rock Socket Diameter, in	Factored Side Resistance, kips		
	5-ft Rock Socket	10-ft Rock Socket	15-ft rock Socket
12	190	390	580
18	290	580	870
24	390	780	1170
30	480	970	1460
36	580	1170	1750
42	680	1360	2040
48	780	1560	2340
54	870	1750	2630
60	970	1950	2920
66	1070	2140	3220
72	1170	2340	3510

78	1260	2530	3800
84	1360	2730	4090

The shaft end bearing capacity was calculated using a nominal (unfactored) rock-mass unit base resistance (q_{BN}) of the bearing strata, as represented in the following equation:

$$q_{BN} = N_c.q_u$$

Where q_u is the compressive strength of the concrete, 4000 psi, used in the drilled shafts and N_c is an empirical bearing capacity factor for bedrock based on rock type. An N_c value 2.5 is recommended by AASHTO if the bedrock beneath the drilled shaft base to a depth of twice the shaft diameter (2.0B) is competent or tightly fractured with no compressible materials or gouge filled fractures, as is the case at the investigated sites. A minimum socket depth of 1.5 times the shaft diameter (1.5B) and a clean base is achieved during construction.

Table 4.2 shows the factored end resistance for various socket geometries based on a nominal end resistance of 10,000 psi and a resistance factor of 0.5.

TABLE 4.2:- Factored End Resistance for Various Rock Socket Diameters.

Rock Socket Diameter, in	Factored End Resistance, kips
12	560
18	1270
24	2260
30	3530
36	5080
42	6920
48	9040
54	11450
60	14130
66	17100
72	20350
78	23890
84	27700

The base resistance provided by the rock socket in Pier 1 and 2 of the Steven Memorial Bridge was accounted for because of the relatively large size (greater than 5 feet in diameter) of the rock socket. In this case the side resistance in overburden soils and in the rock socket was neglected. Reasons for neglecting the side resistance of the rock socket includes; (1) the possibility of strain-softening behavior of the sidewall interface, (2) the possibility of the degradation of the material at the borehole wall and, (3) the uncertainty in the roughness of the socket sidewall. The values in Table 4.2 can be used to determine the geotechnical axial capacity of the drilled shafts for the proposed piers at the Steven Memorial Bridge.

For both the service and extreme limit states, a resistance factor of 1.0 is recommended. Elastic settlements in bedrock will occur immediately and be essentially complete at the end of construction and are estimated to be less than ½ inch. Differential settlements are not anticipated.

4.1.1 Group Effects on Axial Resistance

The resistance of a shaft group to the applied axial loads is not necessarily the sum of the axial resistance of individual shafts within the group. The zone of influence from an individual pile in a pile group may intersect with other piles, depending on the pile spacing. Historically the efficiency of groups of drilled shafts in rock has not been a concern as long as the center-to-center spacing between shafts is greater than four times the shaft diameter (4.0B) to avoid interference between adjacent piles during construction. An efficiency factor (η) of 1.0 for the drilled shaft group can be used for a minimum center-to-center spacing of 4-shaft diameters. The capacity of a shaft group is the sum of the resistance of all individual drilled shafts in the group. If the center-to-center spacing is 2.5B an efficiency factor of 0.65 should be applied. A linear interpolation between 0.65 and 1.0 can be used for any other drilled shaft spacing.

Besides the effect of overlapping zones of influence, effects of construction on ground conditions in and around the group can be significant. Excavated deep foundation elements such as drilled shafts in cohesionless soils tend to decrease the effective stress of the surrounding soils. Poorly controlled shaft construction methods can result in soil loosening during drilling and adversely reduce the lateral stress around other shafts within the group. Casing driven in advance of excavation may increase the relative density and effective stress of the surrounding soil.

Tables 4.3 contains the maximum elevation for the top of the drilled shaft rock sockets at each foundation structure. The drilled shafts at the proposed Abutment 1 and Abutment 2 location of the Steven Memorial Bridge should not bear between elevations 556.0 and 553.0 feet, and 554.0 and 539.0 feet, respectively. This is due to the presence of a decomposed bedrock layer.

TABLE 4.3:- Maximum Elevation for the Top of the Drilled Shaft Rock Socket at Each Structure.

STEVEN MEMORIAL BRIDGE	
Foundation	**Elevation, ft**
Abutment 1	554.0
Pier 1	527.0
Pier 2	520.0
Abutment 2	566.0
HURDY GURDY CREEK BRIDGE	
Abutment 1	590.0
Abutment 2	601.0

4.1.2 Lateral Load Analysis

It is recommended that the lateral load analysis be conducted by the structural engineer on a single shaft from each substructure unit using the soil parameters listed in Tables 4.4, 4.5, and 4.6. Tables 4.4 and 4.5 detail the lateral load response parameters for the portion of the shafts embedded in rock for each foundation structure. Table 4.6 details the lateral load response parameters for the portion of the shafts embedded in overburden materials or structural backfill.

TABLE 4.4:- Lateral Load Parameters for the Rock Socket at Steven Memorial Bridge.

Material properties	Abutment 1 and Pier 1 and 2	Abutment 2
Material Type	Rock	Weak Rock
Effective Unit Weight	0.08 pci	0.1 pci
Elastic Modulus	$5x10^6$ psi	$0.25x10^6$ psi
Unconfined Compresive Strength	9,000 psi	4,000 psi
RQD (Average)	85	27
k_{rm}	0.0005	0.00005

TABLE 4.5:- Lateral Load Parameters for the Rock Socket at Hurdy Gurdy Creek Bridge.

Material Properties	Abutments 1 and 2
Material Type	Rock
Effective Unit Weight	0.08 pci
Elastic Modulus	$0.1x10^6$ psi
Unconfined Comp. Strength	5,000 psi
RQD (Average)	0
k_{rm}	0.00005

TABLE 4.6:- Lateral Load Parameters for the Overburden Materials for Both Bridges.

Material Type	Sand
Unit Weight	0.07 pci
Effective Unit Weight	0.035 pci
K value -above water	90 pci
K value - below water	60 pci
Internal friction angle	30 deg.

To prevent wedge failures induced by high lateral loads and extreme event loading at Pier 1 of the Steven Memorial Bridge, it is recommended that the drilled shaft be over-reamed to a depth of one-shaft diameter from the ground surface and filled with unconsolidated sand.

4.2 ABUTMENT DESIGN RECOMMENDATIONS

Abutments and wing walls should be designed to resist lateral earth pressures and other applicable lateral loads in accordance with the AASHTO Specifications. Lateral earth pressure is influenced by the strength of the abutment backfill, the presence of water, and the ability of the abutment or wall to move in response to lateral loads. Other loads, such as live loads, construction loads, and soil compaction loads should also be considered.

Groundwater behind an abutment or wall adds significant pressure and should be avoided by using permeable backfill (FP-03, Section 703.04) against backfilled structures and assuring a free draining gravity outlet for captured water. Remaining backfill should consist of select granular backfill (FP-03, Section 704.10).

The coefficient of at-rest earth pressure (K_o) should be used for design if the wall is so restrained that it cannot be expected to rotate (deflect at the top) 0.002 times the wall height. The recommended coefficient of at-rest earth pressure is 0.44, which corresponds to an equivalent fluid pressure of 55 pcf. Where the wall can be expected to rotate, a coefficient of active earth pressure (Ka) should be used for wall design. The recommended coefficient of active earth pressure is 0.28, which corresponds to an equivalent fluid pressure of 35 pcf.

4.3 ROCKERY RECOMMENDATIONS

A change in design caused the Hurdy Gurdy Creek Bridge to be shifted along the previous alignment and therefore a retaining wall is required to retain the fill at Abutment 1of the bridge. Geotechnical field investigations (Boring BHG-1) indicate that the materials at the wall location consist mainly of Slate and Gabbro rock fragments in dense gravelly silty sand matrix overlaying the bedrock.

To retain the materials around Abutment 1 of the proposed Hurdy Gurdy Creek Bridge, a single or double tiered rockery is recommended. The rockery should be installed at a maximum distance of 30 feet from the center line of the proposed roadway/bridge alignment and wrap around the abutment beneath the bridge. The beginning station of the rockery is 807+90 and the maximum expected height is 10 feet. The wall should be designed to catch a 1V:1.5H soil slope, which extends from adjacent wingwalls and abutment walls. If the rockery height exceeds 10 feet, a double tier rockery should be installed. The recommended minimum horizontal separation between tiers (measured between the front of the base of each tier is 12 feet and the maximum slope ration between tiers is 1V:4H.

Rockeries consist of earth retaining structures comprised of interlocking, dry-stacked rocks without mortar or steel reinforcement. These structures rely on the weight, size, shape, and interface friction of the individual rocks to resist earth pressures and provide overall stability.

CFL has recently utilized rockeries in many projects as context sensitive solutions preserving scenic, aesthetic, historic and environmental resources, while maintaining safety and mobility.

Rockeries were designed in accordance with guidelines provided in the Central Federal Lands publication *"Rockery Design and Construction Guidelines"*. Although a short segment of wall approaches the maximum height of 10 feet, rockeries were designed for the maximum height as gravity retaining structures without lateral reinforcements. This provides a higher safety factor than required within the lower segments of the wall since the average wall height is about 8 feet. Conservative soil strength properties for both the retained and foundation soils were assumed and passive pressure was neglected in the design. Both static and seismic lateral earth pressures were applied to the back of the rockery and the design was checked for adequate factors of safety against sliding, overturning, and bearing capacity failures. Recommended factors of safety for static and seismic rockery design are presented in the table below:

Table 4.7:- Recommended Static and Seismic Factors of Safety.

Mode of Failure	Recommended Static FS	Recommended Seismic FS
Sliding	1.5	1.1
Overturning	2.0	1.5
Bearing	3.0	2.0

Material properties, rockery geometry and the calculated rockery design factors of safety for wall analysis are shown in Tables 4.8 and 4.9: The following figure provides clarifications to the symbols used in the tables.

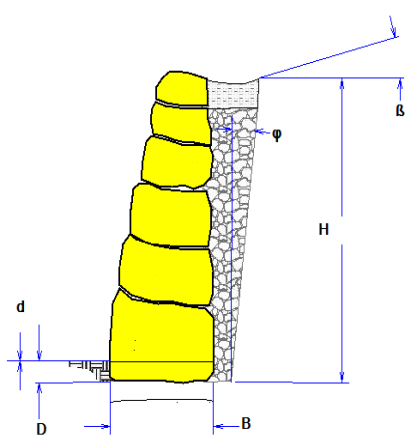


TABLE 4.8:- Material Properties for Rockery Design

ϕ: Retained soil friction angle	32º
γ_s: Retained soil unit weight	120 pcf
c: Soil Cohesion	0 psf
μ:Sliding friction angle	0.67
γ_r: Rockery face unit weight	145 pcf
K_h: Horizontal seismic coefficient	0.67
K_v: Vertical seismic coefficient	0

TABLE 4.9:- Rockery Geometry

B: Base rock width	4.5 ft
H: Rockery height	10 ft
D: Depth at toe	1.5 ft
ß: Backslope angle	15º
Face Batter (V:H)	4:1
Backcut inclination(V:H)	8:1

TABLE 4.10:- Rockery Design Results

Factors of Safety FS	**Static**	**Seismic**
Sliding FS	2.1	1.3
Overturning FS	2.3	1.3
Maximum bearing capacity	12 ksf	4 ksf
Bearing Capacity FS	4.7	2.9

The results indicate a minimum rockery base width (B) of 4.5 feet for a maximum wall height (H) of 10 feet embedded (D) a minimum of 1.5 feet is suitable for the retaining wall to provide a safe design. The standard rockery plan sheets and specifications will work for this project.

The use of a rockery to retain the fill at this location will result in substantial savings, especially if all the rocks and the crushed rock back-drain materials for the rockeries are obtained on site.

A non-woven geotextile should be used as a filtration/separation fabric between the native soils and the back-drain materials. This material should conform to the requirements of FP-03 Section 714, Type I-B geotextile. Also, a 12-inch (minimum) layer of foundation fill should be installed for leveling below the base rock.

4.4 DISCLAIMER/LIMITATIONS CLAUSE

The subsurface explorations and tests described in the Subsurface Investigation Section on Procedures and Results have been conducted in accordance with standard practices and procedures (except as specifically noted). The results of these explorations and tests represent conditions at the specific locations indicated. Subsurface conditions between these locations may vary. The Analysis and Recommendations Section in this report include interpretations and recommendations developed by the Government in the process of preparing the design. These interpretations are not intended as a substitute for the personal investigation, independent interpretation, and judgment of the Contractor.

SECTION FIVE - REFERENCES

American Association of State Highway and Transportation Officials (AASTHO). Standard Specifications for Highway Bridges, 17th Edition, 2002.

2010 AASHTO LRFD Bridge Design Specifications, 5th Edition

2008 Interim Revisions of the AASHTO LRFD Bridge Design Specifications, 4th Edition.

California Department of Fish and Game. Smith River draft waterway management plan, 1980

California Department of Transportation. Corrosion Guidelines, Version 1.0, 2003.

Federal Lands Highway. Project Development and Design Guidelines (PDDM), 2008.

APPENDIX A

PROJECT LOCATION MAPS,
GEOLOGICAL MAPS,
SITE PHOTOGRAPHS

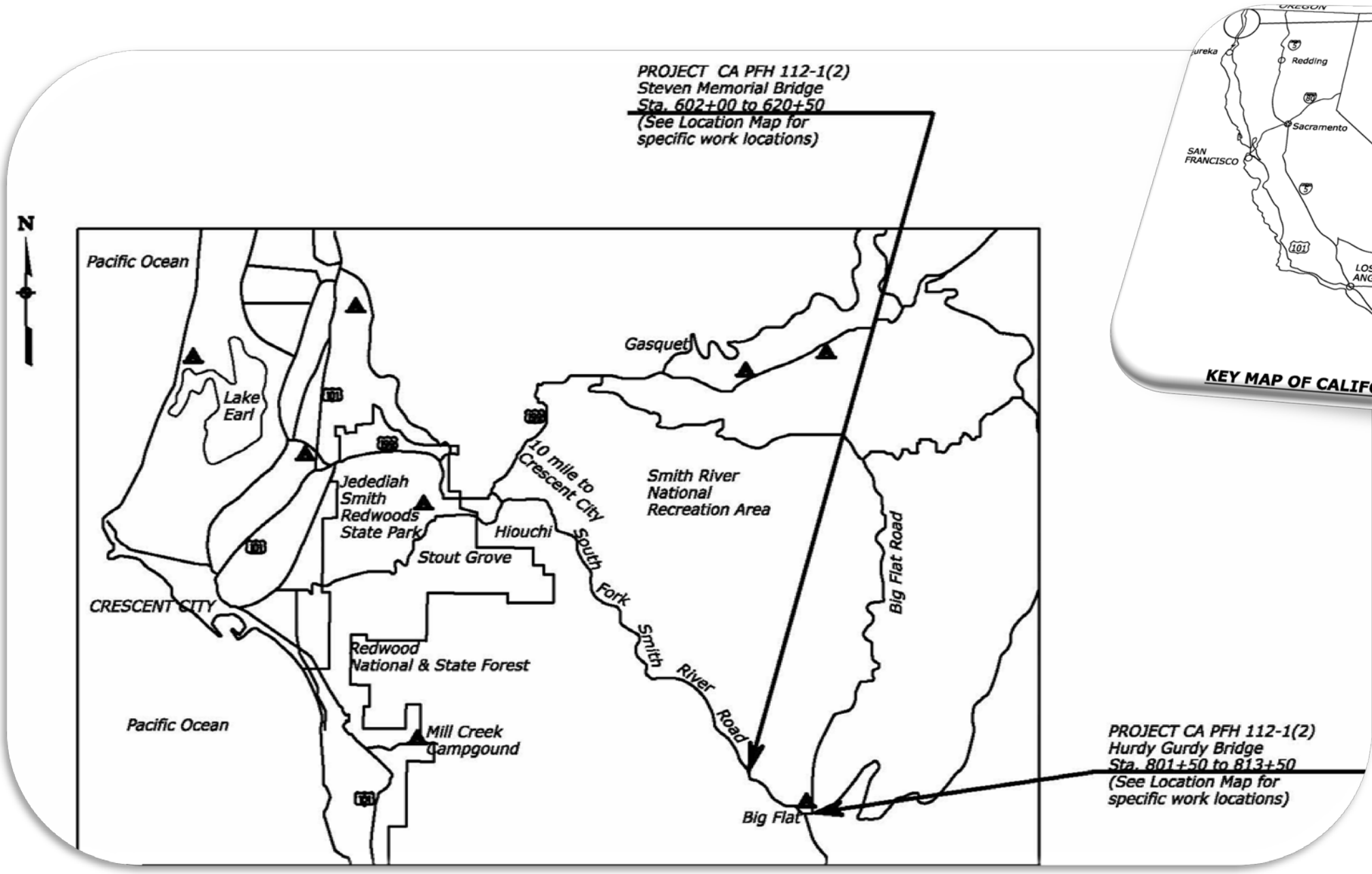


FIGURE A1:- Map Showing General Bridge Locations along South Fork Smith River Road.

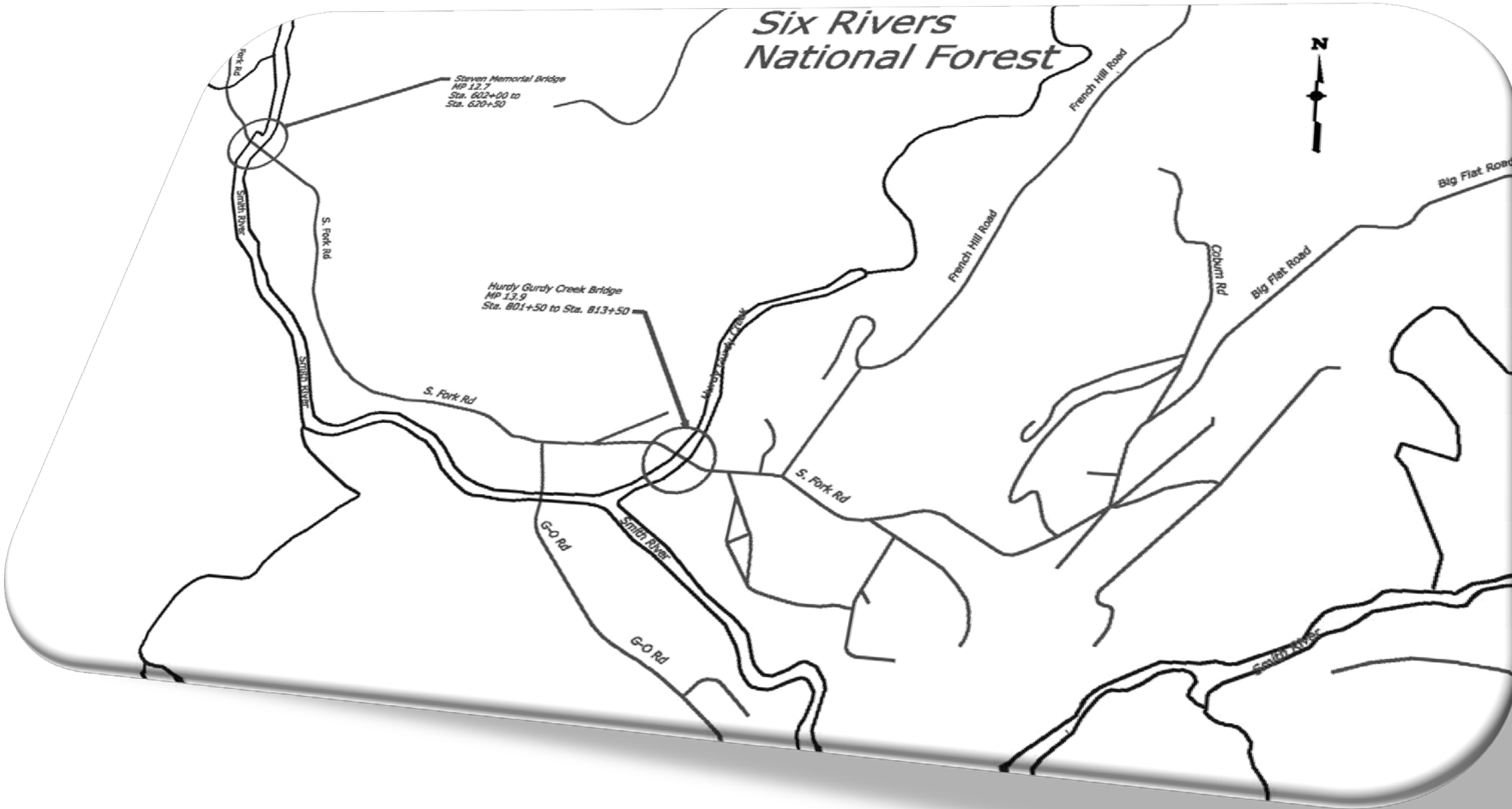


FIGURE A2. Site Map Showing Bridge Locations along South Fork Smith River Road.

South Fork Smith River Road
Six Rivers National Forest
CA PFH 112-1(2)

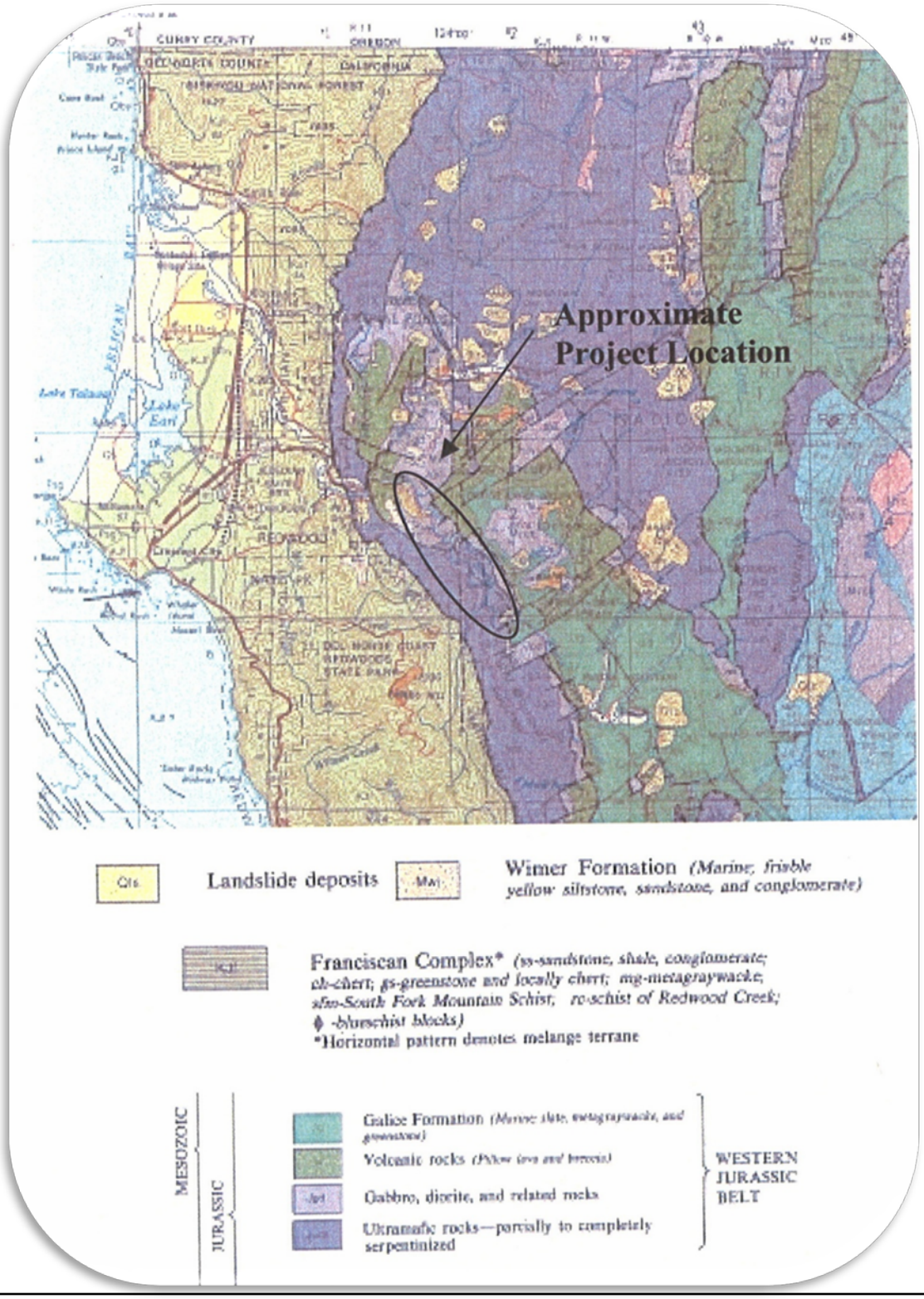


Figure A3:-. Geologic Map of the Project Site.

South Fork Smith River Road
Six Rivers National Forest
CA PFH 112-1(2)

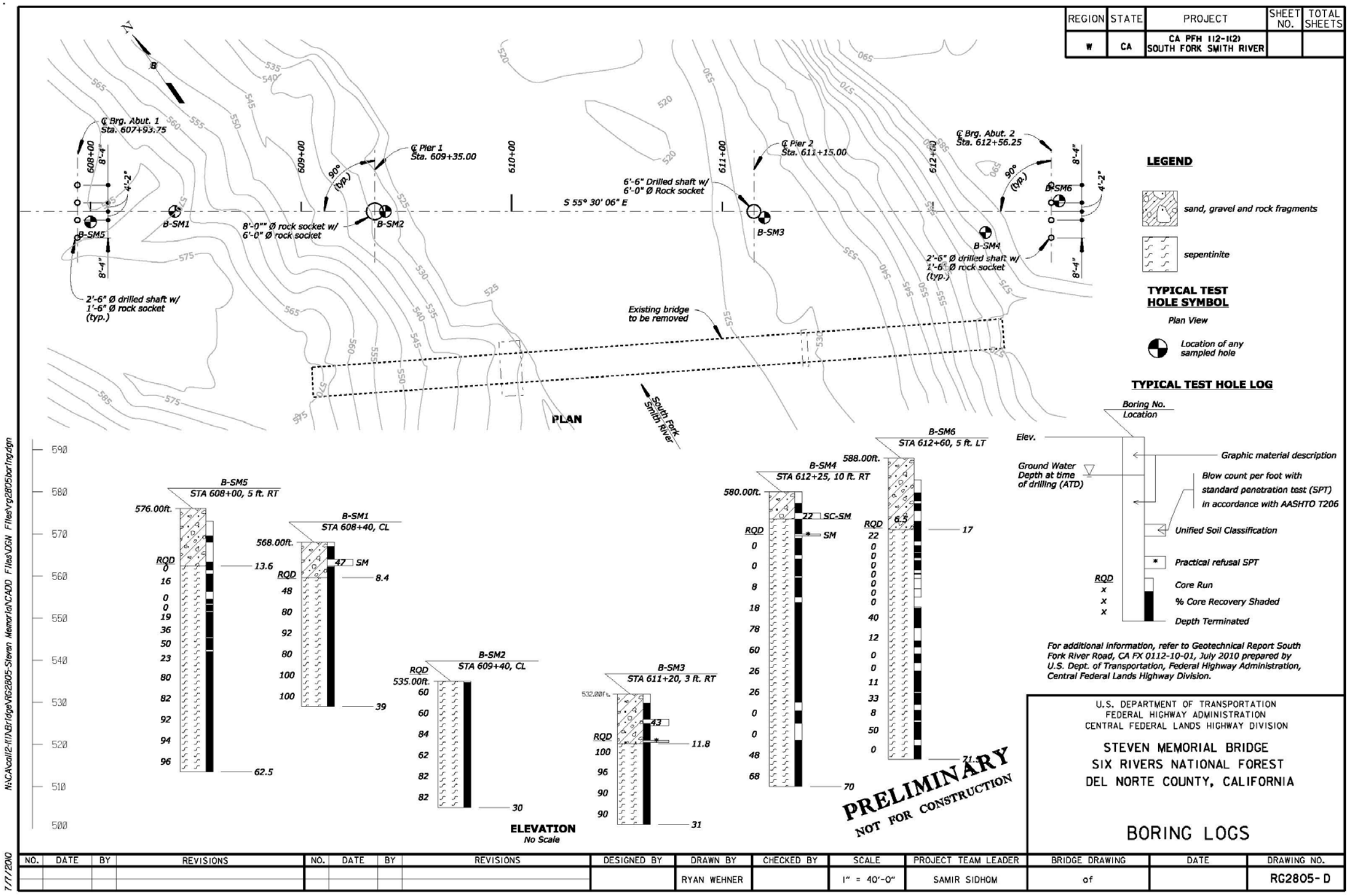


FIGURE A4 :-Boring Location Map for proposed Steven Memorial Bridge.

South Fork Smith River Road
Six Rivers National Forest
CA PFH 112-1(2)

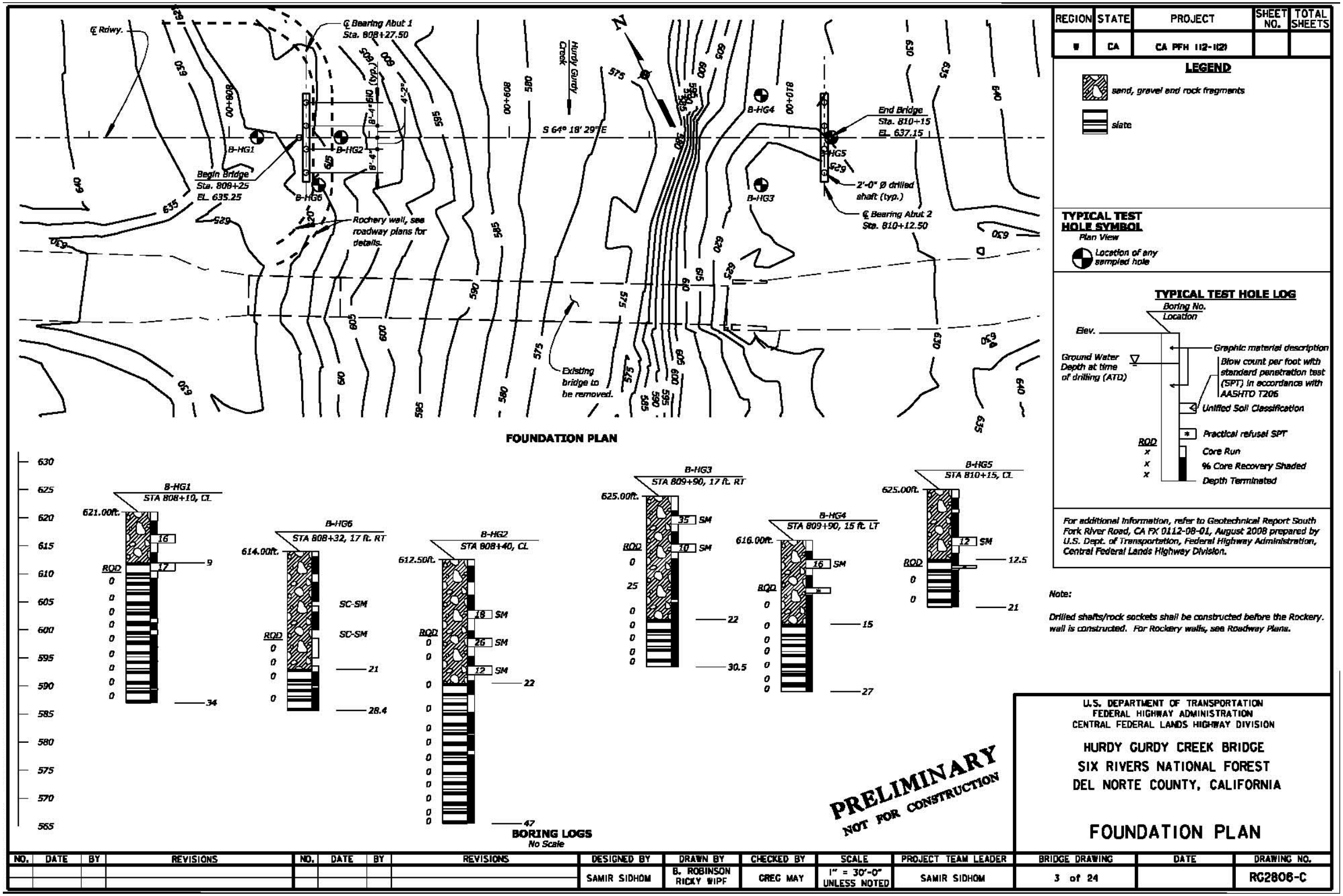


FIGURE A5:- Boring Location Map for Proposed Hurdy Gurdy Creek Bridge.

South Fork Smith River Road
Six Rivers National Forest
CA PFH 112-1(2)

Figure A6:- Existing Steven Memorial Bridge

Figure A7:- Existing Steven Memorial Bridge

Figure A8:- Existing Steven Memorial Bridge

Figure A9:- Existing Steven Memorial Bridge

APPENDIX B

BOREHOLE LOGS

Boring Log Legend and Terminology

1. Color.
2. Soil Density or consistency.
3. Primary Constituent – Major Grain Size (IN CAPS)
4. Secondary Constituent:
   - Coarse-grained — 30% to 50% of secondary coarse-grained constituent (i.e. sandy, gravely)
   - Fine-grained — Based on plasticity and determined by Atterberg limit tests (i.e. silty, clayey)
5. Additional Constituents:
   - Coarse-grained — 'with some' = 10% to 30%; 'trace of' = less than 10%
   - Fine-grained — 'with sone' = 5% to 12%; 'trace of' = less than 5%
6. Moisture Content — Dry (D), Moist (M), Wet(W)

* Index tests and/or plasticity tests are performed to determine whether the terms "silt" or "clay" are used.

Soil Density and Consistency Related To SPT

GRANULAR SOIL		COHESIVE SOIL	
BLOWS	**DENSITY**	**BLOWS**	**CONSISTENCY**
0 – 4	Very Loose	0 - 1	Very Soft
5 – 10	Loose	2 – 4	Soft
11 – 24	Medium Dense	5 – 8	Medium Stiff
25 – 50	Dense	9 – 15	Stiff
Over 50	Very Dense	16- 30	Very Stiff
		31-60	Hard
		Over 60	Very Hard

United Soil Classification System (USCS)

Major divisions			Group symbol	Group name
Coarse grained soils more than 50% retained on No. 200 (0.075 mm) sieve	gravel > 50% of coarse fraction retained on No. 4 (4.75 mm) sieve	clean gravel <5% smaller than #200 Sieve	**GW**	well graded gravel, fine to coarse gravel
			GP	poorly graded gravel
		gravel with >12% fines	**GM**	silty gravel
			GC	clayey gravel
	sand ≥ 50% of coarse fraction passes No.4 sieve	clean sand	**SW**	well graded sand, fine to coarse sand
			SP	poorly-graded sand
		sand with >12% fines	**SM**	silty sand
			SC	clayey sand
Fine grained soils more than 50% passes No.200 sieve	silt and clay liquid limit < 50	inorganic	**ML**	silt
			CL	clay
		organic	**OL**	organic silt, organic clay
	silt and clay liquid limit ≥ 50	inorganic	**MH**	silt of high plasticity, elastic silt
			CH	clay of high plasticity, fat clay
		organic	**OH**	organic clay, organic silt
Highly organic soils			**Pt**	peat

ASTM D 2487 Terminology

The basic reference for the Unified Soil Classification System is ASTM D 2487. Terms include:

Coarse-Grained Soils	More than 50 percent retained on a 0.075 mm (No. 200) sieve
Fine-Grained Soils	50 percent or more passes a 0.075 mm (No. 200) sieve
Gravel	Material passing a 75-mm (3-inch) sieve and retained on a 4.75-mm (No. 4) sieve.
Coarse Gravel	Material passing a 75-mm (3-inch) sieve and retained on a 19.0-mm (3/4-inch) sieve.
Fine Gravel	Material passing a 19.0-mm (3/4-inch) sieve and retained on a 4.75-mm (No. 4) sieve.
Sand	Material passing a 4.75-mm sieve (No. 4) and retained on a 0.075-mm (No. 200) sieve.
Coarse Sand	Material passing a 4.75-mm sieve (No. 4) and retained on a 2.00-mm (No. 10) sieve.
Medium Sand	Material passing a 2.00-mm sieve (No. 10) and retained on a 0.475-mm (No. 40) sieve.
Fine Sand	Material passing a 0.475-mm (No. 40) sieve and retained on a 0.075-mm (No. 200) sieve
Clay	Material passing a 0.075-mm (No. 200) that exhibits plasticity, and strength when dry (PI ³ 4).
Silt	Material passing a 0.075-mm (No. 200) that is non-plastic, and has little strength when dry (PI < 4).
Peat	Soil of vegetable matter.

SOIL DESCRIPTIVE TERMINOLOGY

CONSISTENCY OF COHESIVE SOILS

SPT N-Value (blows/ft)	Consistency	Field Identification
0 - 1	Very Soft	Easily penetrated several inches by first
2 - 4	Soft	Easily penetrated several inches by thumb
5 - 8	Firm	Moderate effort required to penetrate several inches by thumb
9 - 15	Stiff	Readily indented by thumb but penetrated only with great effort
16 - 30	Very Stiff	Readily indented by thumbnail
> 30	Hard	Indented with difficulty by thumbnail

RELATIVE DENSITY OF COHESIONLESS SOILS

SPT N-Value (blows/ft)	Relative Density
0 - 4	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
> 50	Very Dense

PARTICLE SIZE

Term	Sieve Size
Boulder	> 12” (>300mm)
Cobble	3” - 12” (75mm - 300mm)
Gravel – coarse	¾” - 3” (19mm - 75mm)
Gravel – fine	#4 - ¾” (4.75mm - 19mm)
Sand – coarse	#10 - #4 (2.00mm - 4.75mm)
Sand – medium	#40 - #10 (0.425mm - 2.00mm)
Sand – fine	#200 - #40 (0.075mm - 0.425mm)
Silt & Clay	< #200 (<0.075mm)

MOISTURE CONTENT

Dry	Dusty, dry to the touch
Moist	Damp, but no visible free water
Wet	Visible free water, saturated, over optimum

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			Group Symbols (†)	Typical Names	Field Identification Procedures (excluding particles larger than 75 mm and basing fractions on estimated weights)		
1	2		3	4	5		
Coarse grained Soils: More than half of material is larger than No. 200 (‡) (75 μm) sieve size. The No. 200 sieve size is about the smallest particle visible to the naked eye.	Gravels: More than half of gravel fraction is larger than No. 4 sieve size. (4.75 mm) (for visual classification, 5 mm may be used as equivalent to the No. 4 sieve size)	Clean Gravels (little or no fines)	GW	Well-graded gravels, gravel sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.		
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		
		Gravels with Fines (appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).		
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below).		
	Sands: More than half of coarse fraction is smaller than No. 4 sieve size. (4.75 mm)	Clean Sands (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.		
			SP	Poorly graded sands, gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		
		Sands with Fines (appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).		
			SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see CL below).		
Fine grained Soils: More than half of material is smaller than No. 200 (75 μm) sieve size.					Identification Procedures on Fraction Smaller than No. 40 Sieve Size		
					Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near PL)
	Silts and Clays: Liquid limit less than 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium
			OL	Organic silts and organic silty clays of low plasticity.	Slight to medium	Slow	Slight
	Silts and Clays: Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium
			CH	Inorganic clays of high plasticity, fat clays.	High to very high	None	High
			OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium
Highly Organic Soils			Pt	Peat and other highly organic soils.	Readily identified by color, odor, spongy feel, and frequently by fibrous texture.		

Laboratory Classification Criteria (6)

Use grain size curve in identifying the fractions as given under field identification.

Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows:

- Less than 5%: GW, GP, SW, SP
- More than 12%: GM, GC, SM, SC.
- 5% to 12%: Borderline cases requiring use of dual symbols.

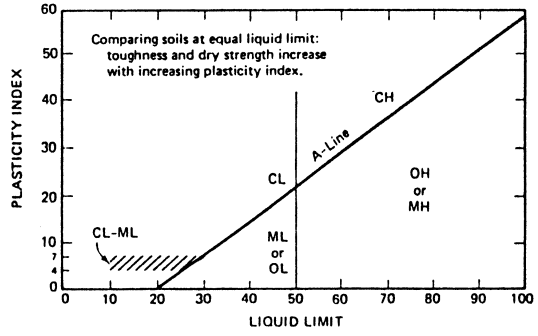
GW: $C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 (See Sec. 2-5)

- Not meeting all gradation requirements for GW
- Atterberg limits below A-line, or PI less than 4
- Atterberg limits above A-line with PI greater than 7
- Above A-line with PI between 4 and 7 are borderline cases requiring use of dual symbols.

SW: $C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 (See Sec. 2-5)

- Not meeting all gradation requirements for SW
- Atterberg limits below A-line, or PI less than 4
- Atterberg limits above A-line with PI greater than 7
- Limits plotting in hatched zone with PI between 4 and 7 are borderline cases requiring use of dual symbols.

Plasticity Chart
For laboratory classification of fine-grained soils



† Boundary classifications: soils possessing characteristics of two groups are designated by combinations of group symbols. For example: GW-GC, well-graded gravel sand mixture with clay binder.

‡ All sieve sizes on this chart are U.S. Standard.

ROCK DESCRIPTIVE TERMS

ROCK WEATHERING

Field Identification	Term
Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.	Fresh
Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric.	Slightly Weathered
Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.	Moderately Weathered
Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.	Highly Weathered
Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.	Decomposed

DISCONTINUITY SPACING

Spacing (in)	Spacing Term
< 2.5	Very Close
2.5 – 8	Close
8 – 24	Moderately close
24 – 80	Wide
> 80	Very Wide

ROCK QUALITY DESIGNATION (RQD)

RQD %	Designation
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

(RQD is the percent of a core run with intact lengths greater than 4 inches excluding breaks caused by drilling.)

INTACT ROCK HARDNESS

Field Identification	UCS (psi)	Hardness Term
Indented by thumbnail.	40 - 150	Extremely Weak (R0)
Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife.	150 - 730	Very Weak (R1)
Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with geological hammer.	730 – 3,600	Weak (R2)
Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow of geological hammer.	3,600 – 7,300	Medium Strong (R3)
Specimen requires more than one blow of geological hammer to fracture it.	7,300 – 15,000	Strong (R4)
Specimen requires many blows of geological hammer to fracture it.	15,000 – 36,000	Very Strong (R5)
Specimen can only be chipped with geological hammer.	> 36,000	Extremely Strong (R6)

(UCS – Unconfined Compressive Strength)

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)		Boring No. B-SM1	Date: June 2010	Sheet 1 of 2
Boring Location: STA 608+40, CL		Type of Boring: Wireline Core		
Coordinates: N 40 E 0		Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track	Driller: Crux Subsurface Inc.	Boring Began: 9/28/05	Completed: 9/28/05	
Field Logged By: C. Martinez		Ground Elev: 568.0 ft.	Weather: Sunny	
Revisions/Final By: C. Martinez		Water Depth:		
		Date:		
		Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 1				3.00 75%			0.0 - 8.4 ft. Red brown to gray clayey and silty sand, gravels, cobbles and boulders. RCH 1 Rec. 3 ft. of red to gray clayey sand and cobbles. 9 minute run.
SPT 1	5		SM	1.33 89%		19/22/25	SPT 1 Rec. 1.33 ft. of gray silty sand and gravels, sl. moist.
RCH 2	*559.6*			3.20 91%			RCH 2 Rec. 3.2 ft. of red to gray clayey sand and cobbles and white green massive serpentinite at 8.4'. 9 minute run. 8.4 - 39.0 ft. White green massive **SERPENTINITE**
RCH 3	10			5.00 100%	48		RCH 3 Rec. 5 ft. of white green massive serpentinite. 9½ minute run.
RCH 4	15			5.00 100%	80		RCH 4 Rec. 5 ft. of white green massive serpentinite. 10½ minute run.
RCH 5	20			5.00 100%	92		RCH 5 Rec. 5 ft. of white green massive serpentinite. Unconfined compressive strength = 18.5 and 20.4 ksi and unit weight = 189.4 and 188.6 pcf for (2) sample from 20' to 20.8' and 22' to 23'. 9 minute run.
RCH 6	25			5.00 100%	80		RCH 6 Rec. 5 ft. of white green massive serpentinite. 11 minute run.
RCH 7	30			5.00 100%	100		RCH 7 Rec. 5 ft. of white green massive serpentinite. Unconfined compressive strength = 21.2 ksi and unit weight = 189.9 pcf for sample from 29' to 30'. 10 minute run.
	35						RCH 8 Rec. 5 ft. of white green massive serpentinite. 10

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-SM1	Date: June 2010	Sheet 2 of 2
Boring Location: STA 608+40, CL	Type of Boring: Wireline Core		
Coordinates: N 40 E 0	Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track / Driller: Crux Subsurface Inc.	Boring Began: 9/28/05	Completed: 9/28/05	
Field Logged By: C. Martinez	Ground Elev: 568.0 ft.	Weather: Sunny	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 8	*529*			5.00 100%	100		minute run.
	40						BHT at 39.0 ft.
	45						
	50						
	55						
	60						
	65						
	70						

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-SM2	Date: June 2010	Sheet 1 of 1
Boring Location: STA 609+40, CL	Type of Boring: Wireline Core		
Coordinates: N 140 E 0	Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track / Driller: Crux Subsurface Inc.	Boring Began: 9/30/05	Completed: 9/30/05	
Field Logged By: C. Martinez	Ground Elev: 535.0 ft.	Weather: Sunny	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 1				4.70 94%	60		0.0 - 30.0 ft. White and green massive **SERPENTINITE**with dark gray basalt dikes RCH 1 Rec. 4.7 ft. of white green massive serpentinite. 13 minute run.
RCH 2	5			5.00 100%	60		RCH 2 Rec. 5 ft. of white green massive serpentinite. Unconfined compressive strength = 11.6 ksi and unit weight = 188.0 pcf for sample from 6.7' to 8.0'. 12 minute run.
RCH 3	10			5.00 100%	84		RCH 3 Rec. 5 ft. of white green massive serpentinite. Unconfined compressive strength = 9.3 ksi and unit weight = 187.5 pcf for sample from 12.5' to 14.0'. 13 minute run.
RCH 4	15			5.00 100%	62		RCH 4 Rec. 5 ft. of white green massive serpentinite with dark gray basalt dikes. 13 minute run.
RCH 5	20			5.00 100%	82		RCH 5 Rec. 5 ft. of dark gray basalt dike with bands of white green massive serpentinite. 12 minute run.
RCH 6	25			5.00 100%	82		RCH 6 Rec. 5 ft. of white green massive serpentinite. 11 minute run.
	505 30						BHT at 30.0 ft.
	35						

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)		Boring No. B-SM3	Date: June 2010 / Sheet 1 of 1
Boring Location: STA 611+20, 3 ft. RT		Type of Boring: Wireline Core	
Coordinates: N 220 E 0		Casing Used: HQ3	Size: 4" I.D.
Drill: Burley 5500-1 Track	Driller: Crux Subsurface Inc.	Boring Began: 9/29/05	Completed: 9/29/05
Field Logged By: C. Martinez		Ground Elev: 532.0 ft.	Weather: Sunny
Revisions/Final By: C. Martinez		Water Depth:	
		Date:	
		Time:	

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 1	5			3.80 63%			0.0 - 11.8 ft. Brown clayey sand with cobbles and boulders RCH 1 Rec. 3.8 ft. of gravels, cobbles and gray coarse sand. 16½ minute run.
SPT 1				0.42 28%		20/32/11	SPT 1 Rec. 0.42 ft. of gray sl. silty sand and gravels.
RCH 2	10			4.00 114%			RCH 2 Rec. 4 ft. of mostly cobbles and sl. silty sand and gravels. 13 minute run.
SPT 2	*520.2*			0.25 50%		50-6"	SPT 2 Rec. 0.25 ft. of gray sl. silty sand and gravels.
RCH 3	15			4.50 100%	100		RCH 3 Rec. 4.5 ft. of gravels and cobbles (0.3') and 4.2' of white and gray gneiss. 10½ minute run. 11.8 - 31.0 ft. White green massive **SERPENTINITE**
RCH 4	20			5.00 100%	96		RCH 4 Rec. 5 ft. of white green massive serpentinite. 9½ minute run.
RCH 5	25			5.00 100%	90		RCH 5 Rec. 5 ft. of white green massive serpentinite. 8 minute run.
RCH 6	30			5.00 100%	90		RCH 6 Rec. 5 ft. of white green massive serpentinite. 8 minute run.
	501 35						BHT at 31.0 ft.

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-SM4	Date: June 2010	Sheet 1 of 2
Boring Location: STA 612+25, 10 ft. RT	Type of Boring: Wireline Core		
Coordinates: N 325 E 0	Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track / Driller: Crux Subsurface Inc.	Boring Began: 9/27/05	Completed: 9/27/05	
Field Logged By: C. Martinez	Ground Elev: 580.0 ft.	Weather: Sunny	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 1				2.30 46%			0.0 - 6.5 ft. Brown clayey sand with cobbles and boulders RCH 1 Rec. 2.3 ft. of yellow brown clayey sand, cobbles and boulders (boulder on surface). 14½ minute run.
SPT 1	5 573.5		SC-SM	0.58 39%		7/10/12	SPT 1 Rec. 0.58 ft. of yellow brown clayey sand, sl. moist.
RCH 2				3.50 100%			6.5 - 70.0 ft. Green Serpentinite RCH 2 Rec. 3.5 ft. of yellow brown decomposed serpentinite. 12½ minute run.
SPT 2	10		SM	0.50 100%		30/10-0	SPT 2 Rec. 0.5 ft. of yellow brown decomposed serpentinite.
RCH 3				4.00 89%	0		RCH 3 Rec. 4 ft. of green decomposed to moderately weathered serpentinite. 17½ minute run.
RCH 4	15			4.10 82%	0		RCH 4 Rec. 4.1 ft. of green decomposed to moderately weathered serpentinite. 12 minute run.
RCH 5	20			4.60 92%	8		RCH 5 Rec. 4.6 ft. of green decomposed to moderately weathered serpentinite. 12½ minute run.
RCH 6	25			3.75 75%	18		RCH 6 Rec. 3.75 ft. of green decomposed to moderately weathered serpentinite. 15½ minute run.
RCH 7	30			5.00 100%	78		RCH 7 Rec. 5 ft. of green moderately weathered serpentinite. 9½ minute run.
	35						

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-SM4	Date: June 2010	Sheet 2 of 2
Boring Location: STA 612+25, 10 ft. RT	Type of Boring: Wireline Core		
Coordinates: N 325 E 0	Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track / Driller: Crux Subsurface Inc.	Boring Began: 9/27/05	Completed: 9/27/05	
Field Logged By: C. Martinez	Ground Elev: 580.0 ft.	Weather: Sunny	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 8				5.00 100%	60		RCH 8 Rec. 5 ft. of green moderately weathered serpentinite. 11 minute run.
	40						
RCH 9				5.00 100%	26		RCH 9 Rec. 5 ft. of green moderately weathered serpentinite. 10½ minute run.
	45						
RCH 10				5.00 100%	26		RCH 10 Rec. 5 ft. of green moderately weathered serpentinite. 10 minute run.
	50						
RCH 11				3.00 60%	0		RCH 11 Rec. 3 ft. of green moderately weathered serpentinite. 10½ minute run.
	55						
RCH 12				1.00 20%	0		RCH 12 Rec. 1 ft. of green moderately weathered serpentinite. 11 minute run.
	60						
RCH 13				5.00 100%	48		RCH 13 Rec. 5 ft. of green moderately weathered serpentinite. 14 minute run.
	65						
RCH 14				5.00 100%	68		RCH 14 Rec. 5 ft. of green moderately weathered serpentinite. 14½ minute run.
	510 70						BHT at 70.0 ft.

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-SM5	Date: June 2010	Sheet 1 of 2
Boring Location: STA 608+00, 5 ft. RT	Type of Boring: Casing Advancer/Wireline Core		
Coordinates: N 0 E 0	Casing Used: H size	Size: 3.75"	
Drill: Mobile B-80 / Driller: Diamond Core	Boring Began: 3/16/10	Completed: 3/16/10	
Field Logged By: C. Martinez	Ground Elev: 576.0 ft.	Weather: Cloudy, showers	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RUN 1							0.0 - 13.6 ft. Red brown to gray clay **SAND, GRAVELS, COBBLES and BOULDERS** RUN 1 Drove casing advancer through red brown to gray clayey sand, gravels, cobbles and boulders.
RCN 1	5			1.50 30%			RCN 1 Rec. 1.5 ft. of boulder fragment and gravels. 15 minute run.
RCN 2	10			1.00 18%			RCN 2 Rec. 1 ft. of gravels and cobbles. 10 minute run.
RCN 3	*562.4*			0.90 90%	0		13.6 - 62.5 ft. White green **SERPENTINITE** RCN 3 Rec. 0.9 ft. of white green serpentinite. 5 minute run.
RCN 4	15			4.20 82%	16		RCN 4 Rec. 4.2 ft. of white green serpentinite. 20 minute run.
RCN 5	20			1.00 36%	0		RCN 5 Rec. 1 ft. of white green serpentinite. Blocked off at 22.5'. 11 minute run.
RCN 6				1.50 83%	0		RCN 6 Rec. 1.5 ft. of white green serpentinite. Blocked off at 24.3'. 7 minute run.
RCN 7	25			2.40 89%	19		RCN 7 Rec. 2.4 ft. of white green serpentinite. Blocked off at 27'. 10 minute run.
RCN 8	30			3.50 100%	36		RCN 8 Rec. 3.5 ft. of white green serpentinite. Blocked off at 30.5'. 13 minute run.
RCN 9				2.80 93%	50		RCN 9 Rec. 2.8 ft. of white green serpentinite. Blocked off at 33.5'. 14 minute run.
	35						RCN 10 Rec. 3.5 ft. of white green serpentinite. Blocked off at 37.5'. 9 minute run.

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-SM5	Date: June 2010	Sheet 2 of 2
Boring Location: STA 608+00, 5 ft. RT	Type of Boring: Casing Advancer/Wireline Core		
Coordinates: N 0 E 0	Casing Used: H size	Size: 3.75"	
Drill: Mobile B-80 / Driller: Diamond Core	Boring Began: 3/16/10	Completed: 3/16/10	
Field Logged By: C. Martinez	Ground Elev: 576.0 ft.	Weather: Cloudy, showers	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCN 10				3.50 88%	23		
RCN 11	40			5.00 100%	80		RCN 11 Rec. 5 ft. of white green serpentinite. 15 minute run.
RCN 12	45			5.00 100%	82		RCN 12 Rec. 5 ft. of white green serpentinite. 22 minute run.
RCN 13	50			5.00 100%	92		RCN 13 Rec. 5 ft. of white green serpentinite. 20 minute run.
RCN 14	55			5.00 100%	94		RCN 14 Rec. 5 ft. of white green serpentinite. 20 minute run.
RCN 15	60			5.00 100%	96		RCN 15 Rec. 5 ft. of white green serpentinite. 18 minute run.
	513.5						BHT at 62.5 ft.
	65						
	70						

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)		Boring No. B-SM6	Date: June 2010	Sheet 1 of 3
Boring Location: STA 612+60, 5 ft. LT		Type of Boring: Casing Advancer/Wireline Core		
Coordinates: N 360 E 0		Casing Used: H size	Size: 3.75"	
Drill: Mobile B-80	Driller: Diamond Core	Boring Began: 3/17/10	Completed: 3/18/10	
Field Logged By: C. Martinez		Ground Elev: 588.0 ft.	Weather: Sunny	
Revisions/Final By: C. Martinez		Water Depth:		
		Date:		
		Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RUN 1							0.0 - 17.0 ft. Red brown to gray clay **SAND, GRAVELS, COBBLES and BOULDERS** RUN 1 Drove casing advancer through red brown to gray clayey sand with gravels and cobbles.
	5						
RCN 1				2.00 40%			RCN 1 Rec. 2 ft. of gravels and cobbles. Lost water return. 25 minute run.
	10						
RCN 2				1.70 74%			RCN 2 Rec. 1.7 ft. of boulder fragment (1.4') and gravels (0.3'). Lost water return. 10 minute run.
RCN 3	15			2.00 44%			RCN 3 Rec. 2 ft. of gravels and cobbles. Lost water return. 17 minute run. Drove casing advancer from 5.2' to 17'.
	571						
RCN 4				2.70 100%	22		17.0 - 71.5 ft. Dark green **SERPENTINITE** RCN 4 Rec. 2.7 ft. of very fractured dark green serpentinite. 50% lost water return. Blocked off at 19.7'. 13 minute run.
	20						
RCN 5				1.50 59%	0		RCN 5 Rec. 1.5 ft. of very fractured dark green serpentinite. 50% lost water return. Blocked off at 22.25'. 10 minute run.
RCN 6				1.25 86%	0		RCN 6 Rec. 1.25 ft. of very fractured dark green serpentinite. 50% lost water return. Blocked off at 23.7'. 8 minute run.
RCN 7	25			2.30 79%	0		RCN 7 Rec. 2.3 ft. of very fractured dark green serpentinite. 50% lost water return. Blocked off at 26.6'. 11 minute run.
RCN 8				0.30 30%	0		RCN 8 Rec. 0.3 ft. of very fractured dark green serpentinite. 50% lost water return. Blocked off at 27.6'. 8 minute run.
RCN 9				0.00 0%	0		RCN 9 No recovery. 50% lost water return. Blocked off at 28.6'. 7 minute run.
RCN 10	30			0.00 0%	0		RCN 10 No recovery. 50% lost water return. Blocked off at 31'. 14 minute run.
RCN 11				0.00 0%	0		RCN 11 No recovery. 50% lost water return. Blocked off at 33'. 7 minute run.
RUN 2	35						RUN 2 Drove casing advancer from 17' to 35.3'.

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-SM6	Date: June 2010	Sheet 2 of 3
Boring Location: STA 612+60, 5 ft. LT	Type of Boring: Casing Advancer/Wireline Core		
Coordinates: N 360 E 0	Casing Used: H size	Size: 3.75"	
Drill: Mobile B-80 / Driller: Diamond Core	Boring Began: 3/17/10	Completed: 3/18/10	
Field Logged By: C. Martinez	Ground Elev: 588.0 ft.	Weather: Sunny	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCN 12	40			4.70 94%	40		RCN 12 Rec. 4.7 ft. of dark green serpentinite. Good water return for remainder of boring. 18 minute run.
RCN 13	45			1.50 33%	12		RCN 13 Rec. 1.5 ft. of dark green serpentinite. Last 2" of recovery very weathered. 12 minute run.
RCN 14	50			2.10 57%	0		RCN 14 Rec. 2.1 ft. of fractured weathered dark green serpentinite. Blocked off at 48.6'. 13 minute run.
RCN 15	55			0.40 17%	0		RCN 15 Rec. 0.4 ft. of fractured dark green serpentinite. Blocked off at 50.9'. 8 minute run.
RCN 16	60			4.60 100%	11		RCN 16 Rec. 4.6 ft. of dark green serpentinite. Blocked off at 55.5'. 20 minute run.
RCN 17	65			2.70 90%	33		RCN 17 Rec. 2.7 ft. of dark green serpentinite. Blocked off at 58.5'. 15 minute run.
RCN 18	70			2.40 63%	8		RCN 18 Rec. 2.4 ft. of dark green serpentinite. 20 minute run.
RCN 19				4.20 93%	50		RCN 19 Rec. 4.2 ft. of dark green serpentinite. 17 minute run.
RCN 20				3.20 68%	0		RCN 20 Rec. 3.2 ft. of dark green serpentinite. 20 minute run.

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-SM6	Date: June 2010	Sheet 3 of 3
Boring Location: STA 612+60, 5 ft. LT	Type of Boring: Casing Advancer/Wireline Core		
Coordinates: N 360 E 0	Casing Used: H size	Size: 3.75"	
Drill: Mobile B-80 / Driller: Diamond Core	Boring Began: 3/17/10	Completed: 3/18/10	
Field Logged By: C. Martinez	Ground Elev: 588.0 ft.	Weather: Sunny	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
	516.5						
							BHT at 71.5 ft.
	75						
	80						
	85						
	90						
	95						
	100						
	105						

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-HG1	Date: June 2010	Sheet 1 of 1
Boring Location: STA 808+10, CL	Type of Boring: Wireline Core		
Coordinates:	Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track / Driller: Crux Subsurface	Boring Began: 10/15/05	Completed: 10/15/05	
Field Logged By: J. Hunyadi (Kleinfelder)	Ground Elev: 621.0 ft.	Weather: cloudy, cool, rain	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 1				2.40 60%			0.0 - 9.0 ft. Brown silty sand and rock fragments RCH 1 Rec. 2.4 ft. of boulder (gabbro) and brown silty sand.
SPT 1	5			0.42 28%		7/9/7	SPT 1 Rec. 0.42 ft. of brown silty sand with slate and gabbro fragments.
RCH 2	*612*			1.80 51%			RCH 2 Rec. 1.8 ft. of boulders (gabbro) and light brown silty sand.
SPT 2	10			0.92 61%		5/8/9	9.0 - 34.0 ft. Black slate SPT 2 Rec. 0.92 ft. of broken (pulverized) slate, flaky, dry.
RCH 3				2.25 64%	0		RCH 3 Rec. 2.25 ft. of dark gray slate, fractured along 45° planes. Iron oxide staining along cleavage planes.
RCH 4	15			2.50 100%	0		RCH 4 Rec. 2.5 ft. of dark gray slate, fractured along 45° planes. Iron oxide staining along cleavage planes.
RCH 5				2.50 100%	0		RCH 5 Rec. 2.5 ft. of dark gray slate, fractured along 45° planes. Iron oxide staining along cleavage planes.
RCH 6	20			1.20 60%	0		RCH 6 Rec. 1.2 ft. of dark gray slate, fractured along 45° planes. Iron oxide staining along cleavage planes.
RCH 7				2.13 71%	0		RCH 7 Rec. 2.13 ft. of dark gray slate, fractured along 45° planes. Iron oxide staining along cleavage planes.
RCH 8	25			2.30 92%	0		RCH 8 Rec. 2.3 ft. of dark gray slate, fractured along 45° planes. Iron oxide staining along cleavage planes.
RCH 9				2.30 92%	0		RCH 9 Rec. 2.3 ft. of dark gray slate, fractured along 45° planes. Iron oxide staining along cleavage planes.
RCH 10	30			2.50 100%	0		RCH 10 Rec. 2.5 ft. of dark gray slate, fractured along 45° planes. Iron oxide staining along cleavage planes.
RCH 11	*587*			2.30 92%	0		RCH 11 Rec. 2.3 ft. of dark gray slate, fractured along 45° planes. Iron oxide staining along cleavage planes.
	35						BHT at 34.0 ft.

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-HG2	Date: June 2010	Sheet 1 of 2
Boring Location: STA 808+40, CL	Type of Boring: Wireline Core		
Coordinates:	Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track / Driller: Crux Subsurface	Boring Began: 10/15/05	Completed: 10/15/05	
Field Logged By: J. Hunyadi (Kleinfelder)	Ground Elev: 612.5 ft.	Weather: cloudy, cool, rain	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 1				2.83 71%			0.0 - 22.0 ft. Brown silty sand and rock fragments RCH 1 Rec. 2.83 ft. of cobble/boulder (gabbro) and dark brown silty sand, moist.
RCH 2	5			2.67 53%			RCH 2 Rec. 2.67 ft. of small gravels and silty sand, moist.
SPT 1	10		SM	0.67 45%		12/10/8	SPT 1 Rec. 0.67 ft. of gabbro rock fragments and light brown silty sand.
RCH 3				1.33 38%			RCH 3 Rec. 1.33 ft. of gravels and cobbles.
SPT 2	15		SM	0.75 50%	0	5/13/13	SPT 2 Rec. 0.75 ft. of gravel and cobbles mixed with silty sand.
RCH 4				1.75 50%	0		RCH 4 Rec. 1.75 ft. of rock fragments (weathered gabbro) mixed with brown silty sand.
SPT 3	20		SM	0.42 28%		5/8/4	SPT 3 Rec. 0.42 ft. of gravels (gabbro) mixed brown silty sand.
RCH 5	*590.5*			2.50 71%	0		RCH 5 Rec. 2.5 ft. of gravel and cobbles (gabbro) mixed with dark gray to light brown silty sand from 20.5' to 24.0' and black slate with near horizontal cleavage planes from 22.0' to 24.0'. 22.0 - 47.0 ft. Black slate
RCH 6	25			1.75 35%	0		RCH 6 Rec. 1.75 ft. of black slate with 45° cleavage planes.
RCH 7	30			2.00 100%	0		RCH 7 Rec. 2 ft. of black slate with 45° cleavage planes.
RCH 8				2.67 89%	0		RCH 8 Rec. 2.67 ft. of black slate with 45° cleavage planes.
	35						RCH 9 Rec. 1.75 ft. of black slate (same as RCH #8).

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-HG2	Date: June 2010	Sheet 2 of 2
Boring Location: STA 808+40, CL	Type of Boring: Wireline Core		
Coordinates:	Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track / Driller: Crux Subsurface	Boring Began: 10/15/05	Completed: 10/15/05	
Field Logged By: J. Hunyadi (Kleinfelder)	Ground Elev: 612.5 ft.	Weather: cloudy, cool, rain	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 9				1.75 70%	0		
RCH 10				2.25 90%	0		RCH 10 Rec. 2.25 ft. of black slate (same as RCH #8).
RCH 11	40			2.00 100%	0		RCH 11 Rec. 2 ft. of black slate (same as RCH #8).
RCH 12				2.92 97%	0		RCH 12 Rec. 2.92 ft. of black slate (same as RCH #8).
RCH 13	45			1.92 96%	0		RCH 13 Rec. 1.92 ft. of black slate (same as RCH #8 with quartz seam 46').
RCH 14	*565.5*			1.00 100%	0		RCH 14 Rec. 1 ft. of black slate (same as RCH #8).
	50						BHT at 47.0 ft.
	55						
	60						
	65						
	70						

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-HG3	Date: June 2010	Sheet 1 of 1
Boring Location: STA 809+90, 17 ft. RT	Type of Boring: Wireline Core		
Coordinates:	Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track / Driller: Crux Subsurface	Boring Began: 10/16/05	Completed: 10/16/05	
Field Logged By: J. Hunyadi (Kleinfelder)	Ground Elev: 625.0 ft.	Weather: Sunny	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 1				0.92 26%			0.0 - 22.0 ft. Brown silty sand and rock fragments RCH 1 Rec. 0.92 ft. of gravels and cobbles (gabbro). 7 minute run.
SPT 1			SM	0.75 50%		8/15/20	SPT 1 Rec. 0.75 ft. of brown silty sand, dry to sl. moist.
	5						
RCH 2				2.17 62%			RCH 2 Rec. 2.17 ft. of gravels and cobbles (gabbro) mixed with dark brown sandy silt. 8 minute run.
SPT 2			SM	0.58 39%		6/6/4	SPT 2 Rec. 0.58 ft. of brown silty sand with rock (gabbro) fragments.
	10						
RCH 3				2.33 67%	0		RCH 3 Rec. 2.33 ft. of broken boulders (gabbro and serpentinite) and brown silty sand. 12 minute run.
	15						
RCH 4				4.75 95%	25		RCH 4 Rec. 4.75 ft. of dark gray gabbro to 15.5, decomposed and weathered gabbro mixed with gray silty sand from 15.5' to 18.5'.
	20						
RCH 5				3.58 90%	0		RCH 5 Rec. 3.58 ft. of dark gray gabbro boulders mixed with gray silty sand to 22' and black slate with 45° cleavage planes from 22' to 22.5'.
	603						22.0 - 30.5 ft. Black slate
RCH 6				0.88 88%	0		RCH 6 Rec. 0.87 ft. of black slate
RCH 7				3.50 100%	0		RCH 7 Rec. 3.5 ft. of black slate (same as RCH #6).
	25						
RCH 8				1.50 100%	0		RCH 8 Rec. 1.5 ft. of black slate (same as RCH #6).
RCH 9				2.00 100%	0		RCH 9 Rec. 2 ft. of black slate (same as RCH #6).
	594.5						
	30						BHT at 30.5 ft.
	35						

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-HG4	Date: June 2010	Sheet 1 of 1
Boring Location: STA 809+90, 15 ft. LT	Type of Boring: Wireline Core		
Coordinates:	Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track / Driller: Crux Subsurface	Boring Began: 10/17/05	Completed: 10/17/05	
Field Logged By: J. Hunyadi (Kleinfelder)	Ground Elev: 616.0 ft.	Weather: clear, cool	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 1				1.58 45%			0.0 - 15.0 ft. Brown silty sand and rock fragments RCH 1 Rec. 1.58 ft. of gravels and cobbles (gabbro) mixed with dark brown silty sand, moist.
SPT 1			SM	1.33 89%		3/3/13	SPT 1 Rec. 1.33 ft. of light brown silty sand, dry to sl. moist.
	5						
RCH 2				2.17 62%			RCH 2 Rec. 2.17 ft. of boulders (gabbro) mixed with brown silty sand between boulders.
SPT 2				0.63 63%		23/50-6"	SPT 2 Rec. 0.62 ft. of gravels and cobbles (gabbro) with gray silty sand.
	10						
RCH 3				3.67 92%	0		RCH 3 Rec. 3.67 ft. of boulders (gabbro) with gray silty sand and serpentinite staining between boulders.
RCH 4	*601*			3.50 100%	0		RCH 4 Rec. 3.5 ft. of boulders (gabbro) with gray silty sand and black slate at 15'. 15.0 - 27.0 ft. Black slate
	15						
RCH 5				1.33 89%	0		RCH 5 Rec. 1.33 ft. of black slate with 45° cleavage planes.
RCH 6				2.27 91%	0		RCH 6 Rec. 2.27 ft. of black slate with 45° cleavage planes (same as RCH #5).
	20						
RCH 7				2.33 93%	0		RCH 7 Rec. 2.33 ft. of black slate with 45° cleavage planes (same as RCH #5).
RCH 8				2.50 100%	0		RCH 8 Rec. 2.5 ft. of black slate with 45° cleavage planes (same as RCH #5).
	25						
RCH 9	*589*			1.00 100%	0		RCH 9 Rec. 1 ft. of black slate with 45° cleavage planes (same as RCH #5). BHT at 27.0 ft.
	30						
	35						

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-HG5	Date: June 2010	Sheet 1 of 1
Boring Location: STA 810+15, CL	Type of Boring: Wireline Core		
Coordinates:	Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track / Driller: Crux Subsurface	Boring Began: 10/17/05	Completed: 10/17/05	
Field Logged By: J. Hunyadi (Kleinfelder)	Ground Elev: 625.0 ft.	Weather: sunny, warm	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 1				2.00 57%			0.0 - 12.5 ft. Brown silty sand and rock fragments RCH 1 Rec. 2 ft. of boulders (gabbro) mixed with poorly graded brown silty sand.
RCH 2	5			4.34 87%			RCH 2 Rec. 4.34 ft. of boulders (gabbro) from 3.5' to 4.5' and gravels mixed with poorly graded brown silty sand from 4.5' to 8.5'.
SPT 1			SM	0.58 39%		6/5/7	SPT 1 Rec. 0.58 ft. of brown silty sand, dry to sl. moist.
RCH 3	10 *612.5*			2.17 62%			RCH 3 Rec. 2.17 ft. of boulders (gabbro) mixed with brown silty sand from 10' to 12.5' and black slate, intensely fractured along 45° cleavage planes. 12.5 - 21.0 ft. Black slate
SPT 2				0.17 34%		50-6"	SPT 2 Rec. 0.17 ft. of black slate, broken up by SPT spoon.
RCH 4	15			3.67 92%	0		RCH 4 Rec. 3.67 ft. of Black slate, 45° cleavage planes with iron oxide staining.
RCH 5	20 *604*			2.83 94%	0		RCH 5 Rec. 2.83 ft. of black slate (same as RCH #4).
	25 30 35						BHT at 21.0 ft.

BORING LOG

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

Project Name: South Fork Smith River Road, CA PFH 112-1(2)	Boring No. B-HG6	Date: June 2010	Sheet 1 of 1
Boring Location: STA 808+32, 17 ft. LT	Type of Boring: Wireline Core		
Coordinates:	Casing Used: HQ3	Size: 4" I.D.	
Drill: Burley 5500-1 Track / Driller: Crux Subsurface	Boring Began: 10/20/05	Completed: 10/20/05	
Field Logged By: J. Hunyadi (Kleinfelder)	Ground Elev: 614.0 ft.	Weather: foggy, cool	
Revisions/Final By: C. Martinez	Water Depth:		
	Date:		
	Time:		

Run/Samp No.	Depth *Elevation* (feet)	Graphic Log	U.S.C.S.	Length Recov. feet ----- % Rec.	RQD	SPT Blows per 6 in	Description: (Density, Color, Type, Moisture, Other)
RCH 1				3.00 75%			0.0 - 21.0 ft. Brown silty sand and rock fragments RCH 1 Rec. 3 ft. of boulders (gabbro) mixed with dark brown silty sand.
RCH 2	5			3.50 70%			RCH 2 Rec. 3.5 ft. of boulders (gabbro) mixed with dark brown silty sand.
SPT 1			SC-SM	0.33 44%			SPT 1 Rec. 0.33 ft. of brown silty sand.
RCH 3	10			3.17 75%			RCH 3 Rec. 3.17 ft. of boulders (gabbro) mixed with dark brown silty sand (same as RCH #2).
SPT 2	15		SC-SM	0.58 39%			SPT 2 Rec. 0.58 ft. of brown silty sand (same as SPT #1).
RCH 4				0.00 0%	0		RCH 4 No recovery. No recovery.
SPT 3	20			0.00 0%	0		SPT 3 No recovery. No recovery.
RCH 5	*593*			2.50 71%	0		RCH 5 Rec. 2.5 ft. of black slate with 45° cleavage planes and green serpentinite. 21.0 - 28.4 ft. Black slate and green serpentinite
RCH 6	25			4.04 92%	0		RCH 6 Rec. 4.04 ft. of light green serpentinite and black slate with 45° cleavage planes.
	585.6						BHT at 28.4 ft.
	30						
	35						

APPENDIX C

LABORATORY DATA

U.S. Department of Transportation
Federal Highway Administration

Central Federal Lands Highway Division Laboratory

An AASHTO and ISO Accredited Laboratory

AASHTO R18 ISO/IEC 17025

Report of Soil or Aggregate Tests

Project: California PFH 112-1 (1) South Fork Smith River Road

Submitted By: Daniel Alzamora

Date Reported: 12/29/2005

Sample Number	Lab Number		05-1481-SB	05-1482-SB	05-1483-SB	05-1485-SB	05-1487-SB
	Hole Number		B-A1	BA2	B-RC1	B-BC1	B-HG2
	Field Number		SPT # 3 & 4	SPT # 1,2,3	SPT # 1 & 2	SPT # 3 & 4	SPT # 1,2,3
Sample Location	Station or Location		107+00	109+00	207+75	408+20	808+40
	Offset				CL	CL	
	Depth	Feet	13-19.5	2.5-19	8-14.5	13-18.3	9-20.5
AASHTO T 11, T 27 & T 88 Washed Sieve Analysis % Passing	3"	75.0 mm					
	1 1/2"	37.5 mm	100	100	100		100
	1"	25.0 mm	86	72	91	100	91
	3/4"	19.0 mm	75	61	81	93	91
	1/2"	12.5 mm	58	56	72	89	86
	3/8"	9.5 mm	50	47	64	77	75
	#4	4.75 mm	35	33	52	58	60
	#8	2.36 mm					
	#10	2.00 mm	25	24	42	41	44
	#16	1.18 mm	21	21	37	32	37
	#30	600 µm					
	#40	425 µm	15	16	30	17	27
	#50	300 µm					
	#100	150 µm	10	12	23	11	19
	#200	75 µm	7.9	9.5	18	7.6	15
		20 µm					
		2 µm					
		1 µm					
AASHTO T 255	Moisture, %						
AASHTO T 89 & T 90	Liquid Limit			22			27
	Plasticity Index			5			4
Soil Classification	AASHTO M 145			A-1-a (0)			A-1-a (0)
	ASTM D 2487			GP-GC			SM
AASHTO T 190	R –Value						
AASHTO T 288	Min. Resistivity, ohm-cm						
AASHTO T 289	pH						
AASHTO Method	Optimum Moisture, %						
	Maximum Dry Density, pcf						

Distribution:

Num. / Project File	
Laboratory	Darrell Harding
Geotechnical	Daniel Alzamora
Materials	1 Copy

Remarks:
Where no liquid limit / plasticity index and classification appear, an insufficient amount of material was provided.

For all the samples, less than 10 % of the required quantity was furnished for gradation determination.

Reported By:

Darrell Harding

Form FHWA 1702 Rev. 12/01

U.S. Department of Transportation
Federal Highway Administration

Central Federal Lands Highway Division Laboratory

An AASHTO and ISO Accredited Laboratory

AASHTO R18 ISO/IEC 17025

Report of Soil or Aggregate Tests

Project: California PFH 112-1 (1) South Fork Smith River Road

Submitted By: Daniel Alzamora

Date Reported: 12/29/2005

Sample Number	Lab Number		05-1488-SB	05-1489-SB	05-1490-SB	05-1491-SB	05-1492-SB
	Hole Number		B-HG3	B-HG4	B-HG5	B-HG6	B-C5
	Field Number		SPT # 1 & 2	SPT # 1	SPT # 1	SPT # 1 & 2	SPT # 1
Sample Location	Station or Location		809+90	809+90	810+15	808+32	510+50
	Offset		17' Right	15' Left	CL	17' Left	4' Left
	Depth	Feet	3.5-10	3.5-5	8.5-10	9-15.5	15.5-17
AASHTO T 11, T 27 & T 88 Washed Sieve Analysis % Passing	3"	75.0 mm					
	1 1/2"	37.5 mm					100
	1"	25.0 mm	100			100	93
	3/4"	19.0 mm	87	100		86	88
	1/2"	12.5 mm	78	98	100	81	86
	3/8"	9.5 mm	74	95	90	74	82
	#4	4.75 mm	64	86	82	63	77
	#8	2.36 mm					
	#10	2.00 mm	49	74	67	48	67
	#16	1.18 mm	42	65	56	40	62
	#30	600 µm					
	#40	425 µm	30	46	37	29	52
	#50	300 µm					
	#100	150 µm	20	31	23	21	42
	#200	75 µm	15	24	17	17	35
		20 µm					
		2 µm					
		1 µm					
AASHTO T 255	Moisture, %						
AASHTO T 89 & T 90	Liquid Limit		29	25	27	26	28
	Plasticity Index		6	3	5	5	13
Soil Classification	AASHTO M 145		A-1-a (0)	A-1-b (0)	A-1-b (0)	A-1-b (0)	A-2-6 (1)
	ASTM D 2487		SM	SM	SM	SC-SM	SC
AASHTO T 190	R –Value						
AASHTO T 288	Min. Resistivity, ohm-cm						
AASHTO T 289	pH						
AASHTO Method	Optimum Moisture, %						
	Maximum Dry Density, pcf						

Distribution:

Laboratory	Num. / Project File
	Darrell Harding
Geotechnical	Daniel Alzamora
Materials	1 Copy

Remarks:
For all the samples, less than 10 % of the required quantity was furnished for gradation determination.

Reported By:

Darrell Harding

Form FHWA 1702 Rev. 12/01

U.S. Department of Transportation
Federal Highway Administration

Central Federal Lands Highway Division Laboratory

An AASHTO and ISO Accredited Laboratory

AASHTO R18 ISO/IEC 17025

Report of Soil or Aggregate Tests

Project: California PFH 112-1 (1) South Fork Smith River Road

Submitted By: Daniel Alzamora

Date Reported: 12/29/2005

Sample Number	Lab Number		05-1493-SB	05-1494-SB	05-1495-SB	05-1496-SB	05-1497-SB
	Hole Number		B-C6	B-C6	B-C7	SI-C2	SI-C3
	Field Number		SPT# 2,3,4,5	SPT # 7 & 9	SPT#1,3,4,6	SPT# 3,4,5,6,7	SPT # 1 & 2
Sample Location	Station or Location		519+30	519+30	523+40	519+80	520+20
	Offset		CL	CL	CL		
	Depth	Feet	8.8-25.3	33.8-45.3	4-30.5	15.5-37	4-10.5
AASHTO T 11, T 27 & T 88 Washed Sieve Analysis % Passing	3"	75.0 mm					
	1 1/2"	37.5 mm	100			100	100
	1"	25.0 mm	84	100	100	95	87
	3/4"	19.0 mm	81	80	91	91	72
	1/2"	12.5 mm	64	60	73	77	59
	3/8"	9.5 mm	56	55	67	67	52
	#4	4.75 mm	40	41	49	46	37
	#8	2.36 mm					
	#10	2.00 mm	30	29	37	28	28
	#16	1.18 mm	25	24	33	22	24
	#30	600 µm					
	#40	425 µm	19	18	26	15	19
	#50	300 µm					
	#100	150 µm	14	13	21	10	14
	#200	75 µm	11	10	17	7.8	11
		20 µm					
		2 µm					
		1 µm					
AASHTO T 255	Moisture, %						
AASHTO T 89 & T 90	Liquid Limit		20		24	18	
	Plasticity Index		4		3	1	
Soil Classification	AASHTO M 145		A-1-a (0)		A-1-b (0)	A-1-a (0)	
	ASTM D 2487		GP-GC		GM	GP-GM	
AASHTO T 190	R –Value						
AASHTO T 288	Min. Resistivity, ohm-cm						
AASHTO T 289	pH						
AASHTO Method	Optimum Moisture, %						
	Maximum Dry Density, pcf						

Distribution:

Laboratory	Num. / Project File
	Darrell Harding
Geotechnical	Daniel Alzamora
Materials	1 Copy

Remarks:
Where no liquid limit / plasticity index and classification appear, an insufficient amount of material was provided.

For all the samples, less than 10 % of the required quantity was furnished for gradation determination.

Reported By:

Darrell Harding

Form FHWA 1702 Rev. 12/01

U.S. Department of Transportation
Federal Highway Administration

Central Federal Lands Highway Division Laboratory

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AAP SM
AASHTO R18 ISO/IEC 17025

Report of Soil or Aggregate Tests

Project: California PFH 112-1 (1) South Fork Smith River Road
Submitted By: Daniel Alzamora

Date Reported: 12/29/2005

Sample Number	Lab Number		05-1498-SB	05-1499-SB		05-1503-SB	
	Hole Number		SI-C4	SI-C5		B-BC1/B-BC2	
	Field Number		SPT# 1 & 2	SPT # 4,5,6		SPT# 1 & 2	
Sample Location	Station or Location		521+45	521+85		408+20/409+25	
	Offset					CL	
	Depth	Feet	5.5-17	19-30.5		3-9.5	
AASHTO T 11, T 27 & T 88 Washed Sieve Analysis % Passing	3"	75.0 mm					
	1 1/2"	37.5 mm		100			
	1"	25.0 mm	100	79		100	
	3/4"	19.0 mm	78	65		89	
	1/2"	12.5 mm	63	53		85	
	3/8"	9.5 mm	53	47		81	
	#4	4.75 mm	34	37		75	
	#8	2.36 mm					
	#10	2.00 mm	24	26		69	
	#16	1.18 mm	21	22		66	
	#30	600 µm					
	#40	425 µm	16	16		61	
	#50	300 µm					
	#100	150 µm	13	11		42	
	#200	75 µm	10	8.8		28	
		20 µm					
		2 µm					
		1 µm					
AASHTO T 255	Moisture, %						
AASHTO T 89 & T 90	Liquid Limit					26	
	Plasticity Index					3	
Soil Classification	AASHTO M 145					A-2-4 (0)	
	ASTM D 2487					SM	
AASHTO T 190	R –Value						
AASHTO T 288	Min. Resistivity, ohm-cm						
AASHTO T 289	pH						
AASHTO Method	Optimum Moisture, %						
	Maximum Dry Density, pcf						

Distribution:

Laboratory	Num. / Project File
	Darrell Harding
Geotechnical	Daniel Alzamora
Materials	1 Copy

Remarks:
Where no liquid limit / plasticity index and classification appear, an insufficient amount of material was provided.

For all the samples, less than 10% of the required quantity was furnished for gradation determination.

After the transmittal was received, Daniel Alzamora requested B-BC 1 & B-BC 2 1&2 be combined before testing.

Reported By:

Darrell Harding

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Report of Soil or Aggregate Tests

Project: California PFH 112-1 (1) South Fork Smith River Road

Submitted By: Charlie Martinez

Date Reported: 11/14/2005

Sample Number	Lab Number		05-1380-SB	05-1381-SB	05-1382-SB	05-1383-SB	
	Hole Number		B-SM 1	B-SM 3	B-SM 4	B-SM 4	
	Field Number		SPT #1	SPT #1 & 2	SPT #1	SPT #2	
Sample Location	Station or Location		Abut 1	Pier 2	Abut 2	Abut 2	
	Offset						
	Depth	Feet	0-4.5*	6-11.5	5-6.5	10-10.5	
AASHTO T 11, T 27 & T 88 Washed Sieve Analysis % Passing	3"	75.0 mm					
	1 1/2"	37.5 mm	100	100			
	1"	25.0 mm	92	90	100		
	3/4"	19.0 mm	89	80	89		
	1/2"	12.5 mm	85	66	87	100	
	3/8"	9.5 mm	79	55	84	98	
	#4	4.75 mm	70	43	80	85	
	#8	2.36 mm					
	#10	2.00 mm	59	28	76	56	
	#16	1.18 mm	53	20	73	44	
	#30	600 µm					
	#40	425 µm	41	12	63	29	
	#50	300 µm					
	#100	150 µm	31	7	49	20	
	#200	75 µm	24	5.1	39	15	
		20 µm					
		2 µm					
		1 µm					
AASHTO T 255	Moisture, %						
AASHTO T 89 & T 90	Liquid Limit		36	**	24	20	
	Plasticity Index		8	**	6	3	
Soil Classification	AASHTO M 145		A-2-4 (0)	**	A-4 (0)	A-1-b (0)	
	ASTM D 2487		SM	**	SC-SM	SM	
AASHTO T 190	R –Value						
AASHTO T 288	Min. Resistivity, ohm-cm						
AASHTO T 289	pH						
AASHTO Method	Optimum Moisture, %						
	Maximum Dry Density, pcf						

Distribution:

Num. / Project File	
Laboratory	Darrell Harding
Geotechnical	Charlie Martinez
Materials	1 Copy

Remarks: * Baggie marked as 4' – 5.5' sample depth.

** An insufficient amount of material was furnished to perform plasticity and classification testing.

Testing was performed on the entire submitted sample. AASHTO test procedures require approximately 10 to 50 times more mass.

Reported By:

Darrell Harding

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AASHTO R18 ISO/IEC 17025

Report of Unconfined Compressive Strength Tests

Project: California PFH 112-1 (1) South Fork Smith River Road

Laboratory Numbers: See Below

Date Reported: 1/12/2006

Submitted By: Martinez / Alzamora

Material Type: Rock Cores

Material Source: --

Tested For: Unconfined Compressive Strength / Unit Weight

Field Sample Numbers: See Below

Test Results

Laboratory Number	05-1519-C	05-1521-C	05-1522-C	05-1523-C
Field Number	RC1	RC1	RC2	RC1
Boring Number	B-A1	B-B1	B-B1	B-BC1
Station	107+00	308+46	308+46	408+20
Offset (ft)	NA	3.4 Left	3.4 Left	CL
Depth (ft)	24-24.9	4-4.7	5.5-6.1	15.5-16.6
Length (in)	5.53	5.49	5.56	5.51
Diameter (in)	2.41	2.39	2.39	2.42
L/D Ratio	2.3	2.3	2.3	2.3
Total Load (lbf)	224,440	24,646*	189,250	6501**
Compressive Strength (psi)	49,220	5490	41,440	1410
Unit Weight (pcf)	180.4	175.3	177.8	140.7

* This core broke along a vein of light colored mineral material. **This core appears to be rock fragments in a soft matrix. The matrix failed while the aggregate did not fracture.

Laboratory Number	05-1524-C	05-1525-C	05-1527-C	05-1529-C
Field Number	RC1	RC1	RC3	RC1
Boring Number	B-BC1	B-BC2	B-BC2	B-RC1
Station	408+20	409+25	409+25	207+75
Offset (ft)	CL	CL	CL	CL
Depth (ft)	19-19.7	21.8-22.6	36.2-37.8	19-20.3
Length (in)	5.48	5.45	5.52	5.48
Diameter (in)	2.41	2.41	2.41	2.40
L/D Ratio	2.3	2.3	2.3	2.3
Total Load (lbf)	107,420	127,770	101,400	5450¤
Compressive Strength (psi)	23,560	28,020	22,240	1210
Unit Weight (pcf)	166.8	181.4	184.4	156.8

¤ This core contained numerous veins of light colored mineral material. The core failed along these veins.

Distribution:
Laboratory
Geotechnical
Materials

Num. / Project File
Darrell Harding
Daniel Alzamora
1 Copy

Reported By:

For
Darrell Harding

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Central Federal Lands Highway Division Laboratory

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AASHTO R18 ISO/IEC 17025

Report of Unconfined Compressive Strength Tests

Project: California PFH 112-1 (1) South Fork Smith River Road

Laboratory Numbers: See Below

Date Reported: 1/12/2006

Submitted By: Martinez / Alzamora

Material Type: Rock Cores

Material Source: --

Tested For: Unconfined Compressive Strength / Unit Weight

Field Sample Numbers: See Below

Test Results

Laboratory Number	05-1531-C	05-1532-C	05-1533-C	05-1534-C
Field Number	RC3	RC1	RC2	RC1
Boring Number	B-RC1	B-RC2	B-RC2	B-C5
Station	207+75	209+00	209+00	510+50
Offset (ft)	CL	CL	CL	4 Left
Depth (ft)	32-33	5.5-6	29.5-31	23-24.5
Length (in)	5.57	4.99	5.51	5.48
Diameter (in)	2.40	2.40	2.40	2.40
L/D Ratio	2.3	2.1	2.3	2.3
Total Load (lbf)	132,220	187,780	225,550	69,911
Compressive Strength (psi)	29,250	41,540	49,900	15,470
Unit Weight (pcf)	176.8	185.1	186.8	192.3

Laboratory Number	05-1535-C	05-1536-C	05-1537-C	05-1540-C
Field Number	RC1	RC2	RC3	RC2
Boring Number	B-SM1	B-SM1	B-SM1	B-SM2
Station	608+40	608+40	608+40	609+40
Offset (ft)	CL	CL	CL	CL
Depth (ft)	20-20.8	22-23	29-30	6.7-8.0
Length (in)	5.53	5.22	5.60	5.47
Diameter (in)	2.40	2.40	2.40	2.40
L/D Ratio	2.3	2.2	2.3	2.3
Total Load (lbf)	83,806	92,169	95,941	52,395
Compressive Strength (psi)	18,540	20,390	21,230	11,590
Unit Weight (pcf)	189.4	188.6	189.9	188.0

Distribution:
Laboratory
Geotechnical
Materials

Num. / Project File
Darrell Harding
Daniel Alzamora
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Reported By:

For
Darrell Harding

Form FHWA 1742 Rev. 12/01

Report of Miscellaneous Tests

Project: California PFH 112-1 (1) South Fork Smith River Road

Laboratory Numbers: See Below

Date Reported: 1/12/2006

Submitted By: Martinez / Alzamora

Material Type: Rock Cores

Material Source: --

Tested For: Unconfined Compressive Strength / Unit Weight

Field Sample Numbers: See Below

Test Results

Laboratory Number	05-1541-C	05-1546-C	05-1548-C
Field Number	RC3	RC1	RC2
Boring number	B-SM2	SI-C2	SI-C3
Station	609+40	519+80	520+20
Offset (ft)	CL	NA	NA
Depth (ft)	12.5-14	13-14	39-39.7
Length (in)	5.51	5.44	5.44
Diameter (in)	2.40	2.40	2.41
L/D ratio	2.3	2.3	2.3
Total Load (lbf)	41,962	108,940	83,307
Compressive Strength (psi)	9280	24,100	18,270
Unit Weight (pcf)	187.5	191.9	184.5

Testing was not requested on the following samples:

- B-BC2 RC2
- B-BC2 RC4
- B-RC1 RC2
- B-SM1 RC4
- B-SM2 RC1
- B-SM2 RC4

Testing was requested on the following samples; however; due to the fractured nature and/or length of the core they could not be trimmed to meet the required L/D ratio requirement of 2.0 to 2.5:

- BA2 RC1
- B-SM3 RC1
- B-SM4 RC1
- B-SM4 RC2
- SI-C3 RC1
- SI-C4 RC1

Distribution:

Laboratory	Num. / Project File
Geotechnical	Darrell Harding
Materials	Daniel Alzamora
	1 Copy

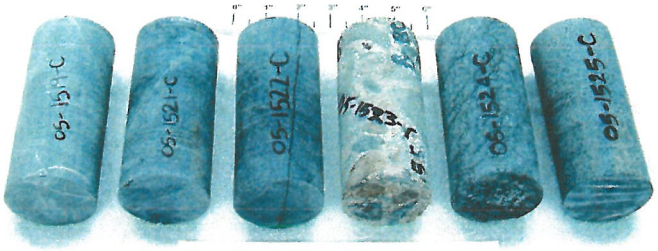
Reported By:

For
Darrell Harding

05-1519-C, 05-1521-C, 05-1522-C, 05-1523-C,
05-1524-C & 05-1525-C

Left to Right

Compressive Strength Samples as Received



05-1519-C, 05-1521-C, 05-1522-C, 05-1523-C,
05-1524-C & 05-1525-C

Left to Right

Trimmed Compressive Strength Samples

05-1519-C, 05-1521-C, 05-1522-C, 05-1523-C,
05-1524-C & 05-1525-C

Left to Right

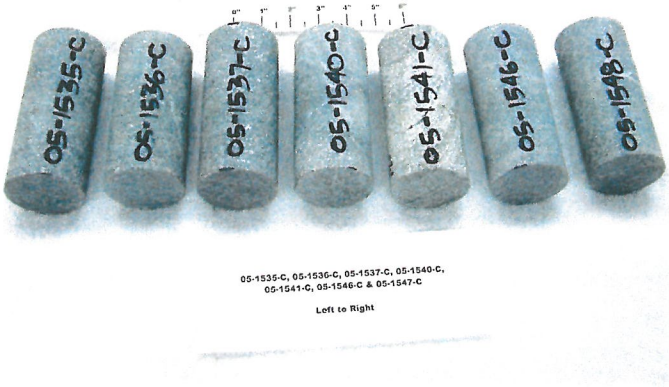
Tested Compressive Strength Samples

Compressive Strength Samples as Received

Trimmed Compressive Strength Samples

Tested Compressive Strength Samples

Compressive Strength Samples as Received



Trimmed Compressive Strength Samples

Tested Compressive Strength Samples

APPENDIX D

SPECIAL CONTRACT REQUIREMENTS

Section 565. — DRILLED SHAFTS

Description

565.01 This work consists of furnishing all necessary materials, services, supervision, labor and equipment to construct and test reinforced concrete drilled shaft foundations.

Material

565.02 Conform to the following Sections and Subsections:

Casings	717.01(a)
Neat cement grout	725.22(f)
Mineral slurry	725.26
Reinforcing steel	709.01
Structural concrete	552

Construction Requirements

565.03 Qualifications and Submittals. Provide an on-site supervisor and drill operator(s) that have worked on at least 5 drilled shaft foundation projects of similar size and scope during the past 3 years. Do not count experience from projects where defects in drilled shaft construction required modification to the design. Use a crosshole sonic logging (CSL) engineer that has completed CSL tests on at least 5 drilled shaft foundation projects of similar size and scope during the past 3 years. At least 20 days prior to beginning drilled shaft construction, submit the following to the CO for review and allow 10 days for response:

a. A summary of the drilled shaft construction and testing personnel qualifying experiences. Identify the on-site supervisor, drill operators and, crosshole sonic logging (CSL) engineer with the above experience to be used on the project. Use only the on-site supervisor, drill operator(s), and CSL engineer(s) listed in the submittal to construct and test the drilled shafts. Do not substitute other personnel for constructing the drilled shafts without providing the requested qualifying information and written approval from the CO.

b. A brief description of each drilled shaft and CSL project including the owning agency's name, contact person, current telephone number, and a sample CSL test report.

c. An installation plan with the following information:

(1) Proposed drilled shaft construction schedule and sequence. Schedule shaft installation to avoid interconnection or damage to shafts in which placed concrete has not achieved final set.

(2) Proposed drilling, hole cleaning/preparation, and reinforcement/concrete placement equipment and procedures for the ground conditions expected to be encountered. Demonstrate an understanding of the subsurface conditions at the site by referencing available subsurface data provided in the contract boring logs. Indicate procedures to minimize disturbance to the construction site or any overlaying or adjacent structure or services. Discuss potential drilling difficulties and indicate methods and tooling that will be used to remove obstructions from the excavation.

(3) Proposed drilled shaft excavation and construction methods used to ensure shaft stability during construction and reinforcement/concrete placement. Include proposed shaft drilling procedures for maintaining required horizontal and vertical shaft alignment and a disposal plan for excavated material. If shaft casing is required, provide casing dimensions and detailed procedures

for permanent casing installation or temporary casing installation and removal

(4) Proposed methods for mixing, circulating, using, maintaining, and disposing of slurry, if used. Provide a detailed slurry mix design and its suitability to the subsurface conditions.

(5) Details of reinforcement placement including bracing, centering, centralizers, and lifting and support methods.

(6) Concrete placement methods including proposed operational procedures for free fall, tremie, or pumping methods.

(7) The method used to form an emergency horizontal construction joint during concrete placement.

565.04 Trial Drilled Shafts. When trial drilled shafts are required, perform the work according to the applicable requirements of Subsection 565.05 and the following.

Before drilling holes for production shafts, demonstrate that the proposed methods and equipment are adequate by drilling a trial drilled shaft adjacent to the production shafts at a location approved by the CO. Make the center-to-center spacing between the trial shaft and production shafts at least 3 shaft diameters or 2 bell diameters, whichever is larger.

Construct the trial drilled shaft to the same size and to the tip elevation of the deepest production shaft shown on the plans. When bells are specified for production shafts, include a bell in the final trial shaft to verify the feasibility of belling in the specified bearing stratum.

If material caves into the drilled hole or the hole deforms sufficient to encroach upon the planned placement of the reinforcement cage with specified minimum concrete cover, case the hole. When casings are used, seat the casing as necessary to prevent caving and to allow dewatering of the hole. Remove all material inside the hole. Keep casing in place a minimum of 4 hours while attempting to remove all water in the hole. Record the rate of groundwater seepage into the hole. After this 4 hour period, fill the hole with saturated sand while the casing is removed to simulate the concreting operation and casing removal for the production drilled shafts. Concrete or reinforcing steel is not required in the trial drilled shaft.

If the trial drilled shaft is determined to be unsatisfactory by the CO per the aforementioned performance requirements, modify the methods and equipment. Submit a new installation plan and drill a new trial shaft at the Contractors' expense.

Once approval is given to construct the production drilled shafts, no changes are permitted in the installation plan without prior approval.

565.05 Shaft Drilling.

A preconstruction meeting will be held by the CO prior to the start of drilled shaft installation to clarify construction requirements, coordinate the construction schedule and activities, and identify contractual relationships and delineation of responsibilities amongst the Contractor and Subcontractor(s).

(a) Drilling. Drill holes according to the approved installation plans. Excavate structural footings supported on drilled shafts and construct fills before initiating shaft drilling.

Provide equipment and tooling with the capability to drill shafts of the specified diameter 20 percent longer than those shown on the plans. Position the drilled shaft within 3 inches (75 millimeters) of the required position in a horizontal plane at the top-of-shaft elevation. Do not allow the constructed alignment of a vertical shaft to vary from the required alignment by more than ¼ inch per foot (6

millimeters per 0.3 meters) of hole depth or the alignment of a battered shaft to vary by more than ½ inch per foot (12 millimeters per 0.3 meters) of hole depth from the required batter.

The hole must be open along the full shaft length to the specified nominal diameter prior to placing reinforcement elements and concrete. If the hole sidewall has degraded due to ground softening, swelling or slurry cake buildup, with approval from the CO, over-ream the sidewall a minimum of ½ inch (12 millimeters) to a maximum of 3 inches (75 millimeters) to reach stable, competent material prior to placing reinforcing elements and concrete. If sidewall material caves into the drilled hole or the hole continues to deform sufficient to encroach upon the planned placement of the reinforcement cage with specified minimum concrete cover, case the hole and/or use mineral slurry during shaft drilling.

Use a qualified geologist or an engineer to maintain a boring log of materials excavated from the drilled shaft. Include the following information:

- Description and approximate top and bottom elevation of each type of soil or rock material encountered and the date and time the soil or rock material was encountered.
- Elevation and approximate rate of any seepage or groundwater encountered.
- Equipment used to drill the shaft, time required to complete the shaft, bit changes, breakdowns, and all other drilling difficulties encountered.
- Remarks.

Upon drilling completion, remove drill cuttings and/or other loose debris from the bottom of the hole. At the time of concrete placement, clean the hole so no more than 50 percent of the bottom of each hole has more than ½ inch (12 millimeters) of sediment and the maximum depth of sediment on the bottom of the hole does not exceed 1½ inches (38 millimeters). For shafts constructed using the dry method, reduce the depth of accumulated water to 3 inches (75 millimeters) or less before placing concrete.

Do not drill additional shafts, allow wheel loads, or allow vibration-inducing equipment or construction activities within 15 feet (4.5 meters) or 3 shaft diameters, whichever is greater, of a newly constructed shaft for at least 20 hours.

Where drilled shafts are located in open water areas, extend exterior casings from above the waterline to a subsurface elevation sufficient to protect against water action during placement and curing of the concrete. Install the exterior casing in a manner that will provide a positive seal at the bottom of the casing to prevent piping of water or entry of other material from the shaft excavation behind the casing.

Use the uncased dry construction method only at sites where the groundwater level and soil conditions are suitable to permit construction of the shaft in a relatively dry excavation and where the sides and bottom of the shaft are stable and may be visually inspected before placing concrete. Use casing and/or the wet construction method for shafts that do not meet the requirements for the dry construction method.

(1) Dry method. This method consists of drilling the shaft, removing accumulated water and loose material from the excavation, placing the reinforcing cage, and concreting the shaft in a relatively dry excavation.

The dry construction method can only be used when the trial shaft excavation meets all of the following:

(a) Less than 12 inches (300 millimeters) of water accumulates above the base of the hole during a 1-hour period when no pumping is permitted;

(b) The sides and bottom of the hole remain stable without detrimental caving, sloughing, or swelling over a 4-hour period immediately following completion of the excavation or CO-approved over-reaming; and

(c) Loose material and water can be satisfactorily removed before inspection and before reinforcement and concrete placement.

(2) Wet method. This method consists of using water or mineral slurry to maintain shaft stability while advancing the excavation to final depth, placing the reinforcing cage, and concreting the shaft. In addition, the wet method requires the following:

(a) Desanding and cleaning drilling slurry;

(b) Final cleaning of the excavation using a bailing bucket, air lift, submersible pump, or other approved devices; and

(c) Placing shaft concrete with a tremie or concrete pump, beginning at the shaft bottom;

If a drill hole is determined unacceptable by the CO per the aforementioned performance requirements and is to be abandoned, fill with lean concrete at no cost to the Government.

(b) Mineral Slurry. Premix mineral slurry with clean fresh water according to the slurry manufacturer's instructions to allow for hydration before introduction into the shaft excavation. Use slurry tanks of adequate capacity for slurry circulation, storage, and treatment. Do not use excavated slurry pits or the shaft excavation to mix the slurry. Do not add mineral component directly into the shaft excavation.

Provide desanding equipment to limit slurry sand content to less than 4 percent by volume at any point within the shaft. Desanding is not required for setting temporary casings, sign posts, or lighting mast foundations.

During drilling, maintain the slurry at least 4 feet (1.2 meters) above the highest expected adjacent piezometric water pressure head and at a level sufficient to prevent caving of the hole.

When there is a sudden loss of slurry from the hole, stop drilling and take corrective measures. Prevent the slurry from setting up in the shaft. If at any time the slurry construction method fails to produce the desired results, discontinue and use an alternative method approved by the CO.

Maintain density, viscosity, and pH of the mineral slurry throughout shaft excavation and concrete placement per Table 565-1. Take slurry samples using a sampling tool approved by the CO. Extract slurry samples from the base of the shaft and 10 feet (3 meters) up from the base of the shaft. Perform 4 sets of tests during the first 8 hours of slurry use. When results are acceptable and consistent, the testing frequency may be decreased to one test set for every 4 hours of slurry use.

Table 565-1
Acceptable Range of Values for Mineral Slurry

Property	At Time of Slurry Introduction	In Hole at Time of Test Concreting	Method
Density, pounds per cubic foot (kilograms per cubic meter)	64.3 – 69.1 (1029.5 – 1106.9)	64.3 – 75.0 (1029.5 – 1201.4)	Density balance
Viscosity, seconds per quart (seconds per litter)	28 – 45 (26.5 – 42.5)	28 – 45 (26.5 – 42.5)	Marsh cone
pH	8 – 11	8 – 11	pH paper or meter
Note: Density values shown are for fresh water. Increase density values 2 pounds per cubic foot (23 kilograms per cubic meter) for saltwater. Perform tests when slurry temperature is above 40 °F (4.5 °C). If desanding is required, sand content shall not exceed 4 percent by volume at any point within the shaft, according to the American Petroleum Institute sand content test.			

When a slurry sample is unacceptable per Table 565-1 requirements, make necessary corrections to bring the slurry within specifications. Do not place concrete until the results of the re-sampling and retesting indicate acceptable values.

Dispose of slurry in an environmentally acceptable manner at an approved location.

(c) Casings. Temporary casing is required to prevent sloughing of the top of the shaft excavation, unless it can be satisfactorily demonstrated to the CO that surface casing is not required. Temporary casing is required, regardless of excavation method, when sidewall conditions warrant additional stabilization or mitigation of excessive groundwater infiltration to ensure proper placement of shaft reinforcement and concrete.

Use smooth, clean, watertight, steel casings of sufficient strength to withstand handling and installation stresses and concrete and surrounding earth pressures. When noted in the plans, casing diameters are outside diameters. The diameter of a permanent casing is subject to American Pipe Institute tolerances applicable to regular steel pipe. Make the outside diameter of the casing no less than the specified size of the shaft.

Install casings to produce a positive seal at the bottom of the casing that prevents piping of water or other material into or out of the excavated hole. If it becomes necessary to remove a casing and substitute a longer or larger diameter casing, stabilize the excavation with slurry or backfill before the new casing is installed. Other CO-approved methods may be used to control the stability of the excavation and protect the integrity of the foundation soils. When drilling below the water table, maintain an adequate head of water or slurry inside the casing to prevent piping or sloughing of material at the bottom of the hole.

All subsurface casings are to be considered temporary unless designated in the contract as permanent casing. Remove temporary casing concurrent with concrete placement. During casing extraction, maintain a level of fresh concrete in the casing that is a minimum of 5 feet (1.5 meter) above the surrounding level of water or drilling fluid. During casing removal, maintain an adequate level of concrete within the casing so fluid trapped behind the casing is displaced upward and discharged at the ground surface without contaminating or displacing the shaft concrete.

Temporary casings that have become bound or fouled during shaft construction and cannot be practically removed are considered to be a defect in the drilled shaft. Correct defective shafts using CO-approved methods. Corrective action may consist of, but is not limited to, the following:

(1) Removing the shaft concrete and extending the shaft deeper to compensate for loss of frictional capacity in the cased zone;

(2) Providing straddle shafts to compensate for capacity loss; or

(3) Providing a replacement shaft.

When a casing is designated as permanent, cut the casing off at the required elevation and leave in place.

565.06 Reinforcing Steel and Crosshole Sonic Logging Access Tubes Placement. Perform the reinforcing steel work according to Section 554. Securely wire together contact reinforcing steel lap splices. Tie and support the reinforcing steel so it remains within the required tolerances. Securely tie concrete spacers or other approved spacing devices at fifth points around the cage perimeter and space at intervals not to exceed 10 feet along the length of the cage. Use spacers of approved material at least equal in quality and durability to the shaft concrete.

Install steel CSL access tubes for each drilled shaft in locations shown on the plans and in accordance with ASTM D 6760. Extend access tubes a minimum of 2 feet (0.6 meters) above shaft top and a maximum of 3 inches (75 millimeters) above shaft bottom. Use schedule 40 mild steel standard black pipe conforming to ASTM A 53 (A 53M), Grade A or B, Type E, F or S, 1 ½ inch (38 mm) nominal diameter for CSL access tubes. Provide an end plug at the lower end of the pipe and make all joints watertight. Fill the CSL access

tubes with potable water prior to placing concrete in the drilled shaft. Temporarily cap the top of the tubes to prevent debris or concrete from entering the tubes.

Place the reinforcing steel cage as a unit immediately after the shaft excavation is inspected and accepted and before concrete placement. If concrete is not placed immediately after the cage is installed, the cage may have to be removed before placing the concrete to verify the integrity of the excavated area and to ensure loose material is removed from the bottom of the hole. Handle reinforcing cages in a manner to avoid distortion or racking of the steel.

During concrete placement, provide positive support at the top for the reinforcing steel cage. Maintain the top of the reinforcing steel cage no more than 6 inches (150 millimeters) above and no more than 3 inches (75 millimeters) below the required position. If the reinforcing steel cage is not maintained within tolerances, make acceptable corrections and do not construct additional shafts until the method of reinforcing steel cage support has been approved by the CO.

565.07 Concrete for Drilled Shafts. When the top of shaft is above ground, use a removable form or other approved means to form the shaft to at least 2 feet (0.6 meters) below finished ground. Forms may be removed provided the requirements in Subsection 562.08 are complied with and the shaft concrete has not been exposed to saltwater or moving water for 7 days. Strip the forms without damaging the concrete.

Remove the top portion of the drilled shaft concrete before continuing with column construction when it is determined the concrete has been affected by underwater placement.

Place concrete immediately after all excavation is complete and the reinforcing steel cage with CSL access tubes is in place.

Furnish concrete conforming to Section 552, except as otherwise indicated herein. For shafts constructed without drilling fluid, use class A structural concrete having a slump of 7±1 inches (180±25 millimeters). For shafts constructed with drilling fluid, use class A structural concrete having a slump of 8±1 inches (203±25 millimeters). Use seal concrete for under water placement. Do not use seal concrete above the freeze/thaw or wet/dry zone of the hole. Place underwater concrete according to Subsection 552.11(e), except as modified herein. The method of underwater placement is subject to CO approval.

Adjust approved admixtures for project conditions to ensure that the concrete has the minimum required slump for at least 2 hours. Submit trial mix and slump loss test results for concrete at ambient temperatures appropriate for site conditions.

Place each load of concrete within 2 hours of batching. Longer placement time may be permitted if the concrete mix maintains the minimum required slump for longer than 2 hours. Do not retemper concrete that has developed its initial set.

Place concrete in one continuous operation from bottom to top of the shaft. Continue placing concrete after the shaft excavation is full and until acceptable quality concrete is evident at the top of shaft. Before initial concrete set, consolidate the top 10 feet (3 meters) of the shaft concrete using acceptable vibratory equipment. Finish the top of the shaft to within +1 to -3 inches (+25 millimeters to -75 millimeters) of the required elevation. For wet holes, do not consolidate shaft concrete until all water or slurry above the concrete surface has been removed.

(a) Free-fall concrete placement. Use free-fall placement only in dry holes with a 25-foot (7.5 meters) maximum height of free-fall. The concrete shall fall directly to the shaft base without contacting either the rebar cage or shaft sidewall.

Drop chutes may be used to direct placement of free-fall concrete. Drop chutes consist of a smooth tube of either one-piece construction or sections that can be added and removed. Support the drop chute so that the maximum height of free-fall of the concrete measured from the bottom of the chute is 25 feet (7.5

meters). If concrete placement causes the shaft excavation to cave or slough, or if the concrete strikes the rebar cage or sidewall, reduce the height of free-fall and reduce the rate of concrete flow into the excavation. If placement cannot be satisfactorily accomplished by free-fall, use tremie or pumping to place the concrete.

(b) Tremies. Use tremies for concrete placement in either wet or dry holes. A tremie consists of a tube of sufficient length, mass, and diameter to discharge concrete at the shaft base. Do not use tremies that contain aluminum parts that will come in contact with the concrete. Provide a tremie with clean and smooth inside and outside surfaces, with sufficient wall thickness to prevent crimping or sharp bends, and with an inside diameter at least 6 times the maximum size of aggregate used in the concrete mix and not less than 10 inches (250 millimeters). Use a watertight tremie in accordance with Subsection 552.11(e) for wet holes. Construct the discharge end of the tremie to permit free radial flow of concrete during placement. Place the tremie discharge at the shaft base elevation. Place the concrete in a continuous flow. Keep the tremie discharge immersed at least 5 feet (1.5 meters) below the surface of the fluid concrete. Maintain a positive head of concrete in the tremie at all times. If at any time during the concrete placement, the tremie discharge is removed from the fluid concrete column and discharges concrete above the rising concrete surface into displaced water, remove the reinforcing cage and concrete, complete any necessary sidewall removal as directed, and reconstruct the shaft.

(c) Pumped concrete. Use pumped concrete placement in either wet or dry holes. Use 4-inch (100 millimeters) minimum diameter discharge tubes with watertight joints. Place the discharge tube at the shaft base elevation.

Use a sealed discharge tube according to Subsection 552.11(e) for wet holes. If a plug is used, remove it from the hole or use a plug made from approved material that will prevent a defect in the shaft if not removed.

Place the concrete in a continuous flow. Keep the pump discharge tube immersed at least 5 feet (1.5 meters) below the surface of the fluid concrete. If at any time during the concrete placement, the discharge tube is removed from the fluid concrete column and discharges concrete above the rising concrete surface into displaced water, remove the reinforcing cage and concrete, complete any necessary sidewall removal as directed by the CO, and reconstruct the shaft.

565.08 Integrity Testing.

(a) Testing Perform integrity testing on all production drilled shafts in accordance with ASTM D 6760. Test drilled shafts no earlier than 2 days and no later than 21 days after concrete placement. Before testing, provide information to the CSL subcontractor and to the CO as to drilled shaft bottom and tip elevations, access tube lengths, surveyed tube positions, and date of concrete placement. Perform tests between all tube pairings in the shaft, including adjacent perimeter access tubes and diagonally between tubes.

If any access tube is not acceptable for testing (e.g. tube is not plumb, tube does not retain water, tube is obstructed, tube-concrete debonding has occurred), provide a plumb core hole, drilled to the appropriate depth, and install a fully-grouted replacement tube or propose an alternative integrity test method that is acceptable to the CO. After integrity testing, inspection, and data analysis are completed and accepted by the CO, tremie fill access tubes with neat cement grout in accordance with section 725.22(f).

(b) Test results and reporting. Provide preliminary results to the CO for each shaft tested before CSL test personnel leave the site. Within 5 days of testing, submit a detailed CSL report to the CO. Allow 5 days for the CO to conduct a review of the data before any further construction on the tested shaft and before issuing the final, written report. Include the following in the crosshole sonic logging report:

(1) Project identification and dates of CSL testing;

(2) Table and a schematic showing shafts tested with identification of tube coordinates and collar elevation;
(3) Names of personnel that performed the CSL tests and interpretations and their affiliation(s);
(4) Type of equipment used for testing;
(5) Data logs;
(6) X-Y plots of first arrival times, amplitude, and velocity versus shaft depth; and
(7) Interpretations, analyses, and results.

Identify and provide detailed discussion of each anomalous zone detected by the CSL. Anomalous zones are areas where velocity reduction exceeds 20 percent of the average velocity of properly placed and cured shaft concrete at the time of testing. Within anomalous zones, collect and process additional data sufficient to construct three-dimensional color-coded tomographic-images with two-dimensional cross-sections between tubes.

565.09 Acceptance.

Material for mineral slurry will be evaluated under Subsections 106.02 and 106.03. Concrete will be evaluated under Section 552. Concrete, tremie placed or pumped, will be sampled at point of discharge into the tremie or concrete pump hopper.

Reinforcing steel will be evaluated under Section 554.

Construction of drilled shafts will be evaluated under Subsections 106.02, 106.03, and 106.04 as follows:

(a) Drilled shafts exhibiting velocity reductions less than or equal to 20 percent of the average velocity of properly placed and cured shaft concrete at the time of testing are acceptable.

(b) Where velocity reductions exceed 20 percent of the average velocity of properly placed and cured shaft concrete at the time of testing, furnish additional imaging and other data required in Subsection 565.08(b) to enable further evaluation of the shaft. When required by the CO, drill a minimum of two coreholes to intercept the anomalous zone and obtain core samples from the suspect area. The CO will evaluate the cross-hole sonic logging data, the tomographic imaging data, and the retrieved core data and make a determination as to the presence of substantive defects.

If a shaft is determined to have substantive defects, submit a plan according to Subsection 106.01 to remove and replace, correct, or modify the work. Proposed modifications to the drilled shaft or load transfer mechanisms shall be designed and stamped by a professional engineer. Include drawings stamped by a professional engineer for all foundation elements affected. Do not begin remedial work until the CO has approved the plan.

Measurement

565.10

Measure the Section 565 items listed in the bid schedule according to Subsection 109.02 and the following as applicable.

Measure drilled shafts from the plan top elevation to the approved tip. Do not measure portions of shafts extending deeper than approved.

Do not measure concrete or reinforcing steel.

Measure trial drilled shafts, determined to be satisfactory, from the approved tip elevation to the ground surface at the center of the shaft.

Do not measure drilled shaft testing. All drilled shaft testing is subsidiary to the installation of drilled shafts. Repair or removal of shaft defects or obstructions is subsidiary to the work.

Payment

565.11 The accepted quantities will be paid at the contract price per unit of measurement for the Section 565 pay items listed in the bid schedule except the drilled shaft contract unit bid price will be adjusted according to Subsection 106.05. Payment will be full compensation for the work prescribed in this Section. See Subsection 109.05.

Payment for drilled shafts will be made at a price determined by multiplying the unit bid price by the compressive strength pay factor.

APPENDIX E

PHOTOGRAPHS

Figure E1:- Location of Boring B-SM1

Figure E2:- Core Samples from Boring B-SM1, 0.0-12.0’

Figure E3:- Core Samples from Boring B-SM1, 12.0’-21.0’

Figure E4:- Core Samples from Boring B-SM1, 21.0’-30.3’

Figure E5:- Core Samples from Boring B-SM1, 30.3’-39.0’

Figure E6:- Location of Boring B-SM2

Figure E7:- Core Samples from Boring B-SM2, 0.0-10.0’

Figure E8:- Core Samples from Boring B-SM2, 10.0’-20.0’

Figure E9:- Core Samples from Boring B-SM2, 20.0’-30.0’

Figure E10:- Location of Boring B-SM3

Figure E11:- Core Samples from Boring B-SM3, 0.0-13.0’

Figure E12:- Core Samples from Boring B-SM3, 13.0’-21.5’

Figure E13:- Core Samples from Boring B-SM3, 21.5’-31.0’

Figure E14:- Location of Boring B-SM4

Figure E15:- Core Samples from Boring B-SM4, 0.0-13.5’

Figure E16:- Core Samples from Boring B-SM4, 13.5’-25.0’

Figure E17:- Core Samples from Boring B-SM4, 25.0’-35.0’

Figure E 18:- Core Samples from Boring B-SM4, 35.0’-45.0’

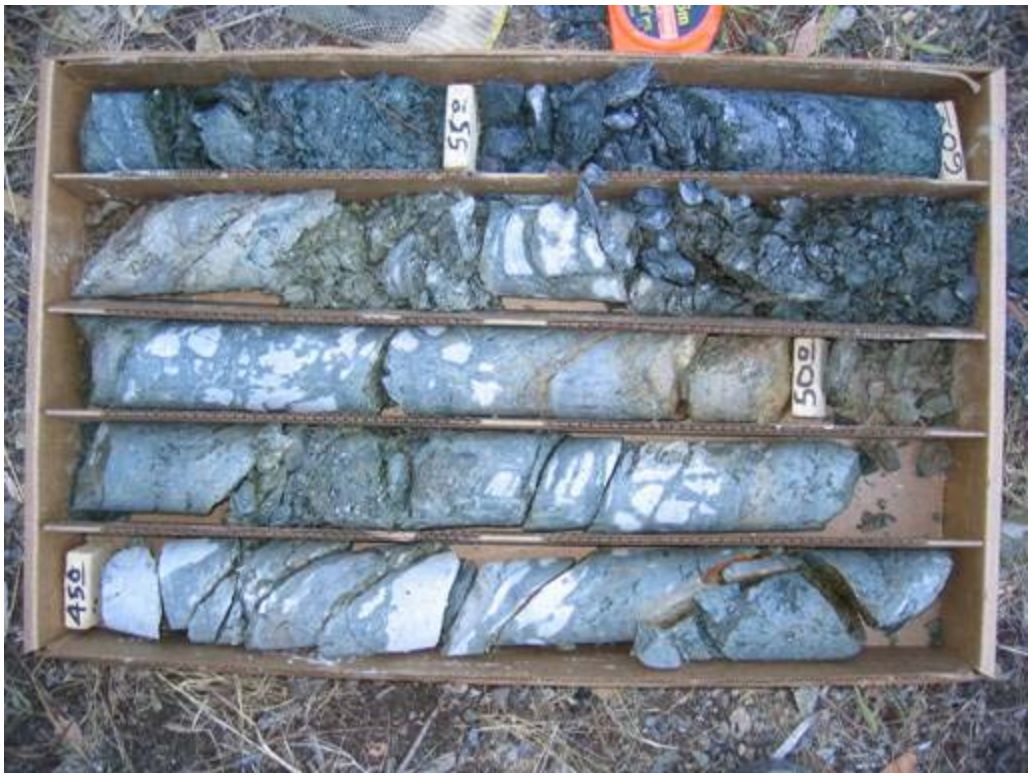


Figure E19:- Core Samples from Boring B-SM4, 45.0’-60.0’

Figure E20:- Core Samples from Boring B-SM4, 60.0’-70.0’

Figure E21:- Location of Boring B-SM5

Figure E22:- Core Samples from Boring B-SM5, 3.8’-24.3’

Figure E23:- Core Samples from Boring B-SM5, 24.3’-34.5

Figure E24:- Core Samples from Boring B-SM5, 34.5-44.0’

Figure E25:- Core Samples from Boring B-SM5, 44.0-52.5’

Figure E26:- Core Samples from Boring B-SM5, 52.5-62.5’

Figure E27:- Location of Boring B-SM6

Figure E28:- Core Samples from Boring B-SM6, 5.2’-22.25’

Figure E29:- Core Samples from Boring B-SM6, 22.25’-44.9’

Figure E30:- Core Samples from Boring B-SM6, 44.9’-62.3’

Figure E31:- Core Sample from Boring B-SM6, 62.3’-71.5’

Figure E32:- Existing Hurdy Gurdy Creek Bridge

Figure E33:- Existing Hurdy Gurdy Creek Bridge

Figure E34:- Location of Boring B-HG1

Figure E35:- Core Samples from Boring B-HG1, 0.0-19.0’

Figure E36:- Core Samples from Boring B-HG1, 19.0’-34.0’

Figure E37:- Location of Boring B-HG2

Figure E38:- Core Samples from Boring B-HG2, 0.0-24.0’

Figure E39:- Core Samples from Boring B-HG2, 24.0’-39.0’

Figure E40:- Core Samples from Boring B-HG2, 39.0’-47.0’

Figure E41:- Location of Boring B-HG3

Figure E42:- Core Samples from Boring B-HG3, 0.0-18.5’

Figure E43:- Core Samples from Boring B-HG3, 18.5’-30.5’

Figure E44:- Location of Boring B-HG4

Figure E45:- Core Samples from Boring B-HG4, 0.0-17.0’

Figure E46:- Core Samples from Boring B-HG4, 17.0’-27.0’

Figure E47:- Location of Boring B-HG5

Figure E48:- Core Samples from Boring B-HG5, 0.0-15.0’

Figure E49:- Core Samples from Boring B-HG5, 15.0’-21.0’

Figure E50:- Location of Boring B-HG6

Figure E51:- Core Samples from Boring B-HG6, 0.0-15.5’

Figure E52:- Core Samples from Boring B-HG6, 20.5’-28.0’