Geotechnical Investigation

Ground Improvement

Mirabel Fish Passage and Seismic Improvement Project

Forestville, California

SONOMA COUNTY WATER AGENCY

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1.0 Introduction

This report presents HDR Engineering Inc.'s (HDR) geotechnical investigation, engineering analysis and recommendations for the proposed ground improvement program at the Mirabel Station a diversion point for groundwater recharge including an inflatable rubber dam and diversion facility. The proposed ground improvement project is part of the Sonoma County Water Agency (the Water Agency) proposed Mirabel fish passage improvements and seismic upgrade of the existing diversion pumping facilities. Specifically, the Water Agency is planning to replace the existing water diversion intake/fish screen and fishway at this location with a new facility. The ground improvement program is required to improve seismic stability of the diversion facility and riverbank adjacent to the new facility. The new facility is proposed to enhance fish passage around the existing inflatable dam, and allow for visitors to have an interactive experience with migrating fish. To support these goals the proposed facility will include a new river diversion intake and fish screen system, a new fishway, an Americans with Disabilities Act accessible viewing gallery, access roadways, and associated support structures. As shown on Figure 1, the site is located along the western bank of the Russian River approximately 0.5 miles south of the Wohler Bridge near Forestville, California.

2.0 Background

Currently, the project site consists of a fish passage (ladder system), River Diversion Structure (RDS), and an inflatable dam supported by a man made embankment along the western bank of the Russian River. The RDS includes an intake structure with a fish screen that acquires water from the Russian River. The water is then pumped to the other side of the embankment through a 72-inch corrugated metal pipe (CMP) to a desilting pond. The relative locations of these systems are illustrated on Figure 2. The Water Agency constructed these structures in 1971. As-built drawings and photographs of the construction activities indicate that relatively modern equipment was used to construct the structures and place fill associated with the embankment. Photographs illustrate that the contractor at the time excavated large portions of the embankment to foundation levels of the various structures by benching and laying back temporary slopes. Sheet pile shoring and dewatering wells were utilized at elevations that were below the groundwater and within the river. It is unclear from the available documentation the method by which the backfill soils were placed and compacted. However, based on the equipment and means available at the time it has been assumed that backfill materials consisted of the excavated embankment and river materials placed in relatively horizontal continuous lifts using mechanical means (e.g. dozer or loader).

More recently, the Water Agency has been evaluating the wells and delivery systems along the Russian River for seismic risk and potential upgrades. Investigations at the project site have been performed by MMI/Geosyntec and Brunsing Associates Inc. MMI/Geosyntec performed a global study for the Water Agency as part of the development of the Local Hazard Mitigation Plan (LHMP). Brunsing Associates Inc. was subcontracted by HDR to determine the geotechnical feasibility and design of the proposed new fish passage improvements and included seismic parameters specific to the fish passage foundation improvements. These investigations concluded that the potential deformations resulting from seismic events occurring at the site is moderate to high. Subsequently, the Water Agency requested that HDR provide an expanded scope of services in support of the fish passage project and provide liquefaction mitigation recommendations for potential risks to the proposed new facility identified by the previous efforts.

This report is a result of HDRs site specific geotechnical investigation of the embankment adjacent to the proposed fish passage project to 1) evaluate the potential risk posed by liquefaction and associated lateral spreading on the facilities, 2) develop potential site remediation approaches, and 3) provide design recommendations of a selected system(s) to lower the potential risk to the existing RDS.

3.0 Scope of Services

A scope of work was developed for the Water Agency as part of contract addendum dated April 16, 2013. The overall project as described above includes several phases. The scope of work associated with the ground improvement phase includes review of previous geotechnical investigations at the site by others; development and performance of a targeted field investigation program; analysis of data specific to ground improvement design; preparation of a geotechnical report summarizing HDR's findings, conclusions and recommendations; and development of plans and specifications for construction bid purposes. HDR has developed civil design drawings and specifications for the ground improvement project that are being provided as a separate submittal to the Water Agency.

HDRs original scope for this effort included liquefaction mitigation in support of the new fish passage facility and the existing RDS structure. Through meetings and coordination with the Water Agency and their representatives, the scope of work and goals of the project were modified to only providing protection for the proposed new fish passage and as much as possible for the RDS within the Water Agency's operational constraints. As a result of The Water Agency operational constraints, the current scope of work includes the liquefaction mitigation in support of the new fish passage facility; and, the assessment and reduction of lateral spreading deformations to existing RDS structure.

4.0 Site Description

The site is located approximately 1.5 miles north of Forestville, California and approximately 0.5 miles south of Wohler Bridge along the Russian River. The site is located at the Water Agency RDS facility adjacent to the west bank of the river. The RDS consists of a seasonal inflatable rubber dam; a fish ladder; a water intake structure at the river; an outlet structure to desilting ponds; and the RDS pump station. Constructed in 1971, the RDS pump station is founded on a 17 foot diameter concrete caisson located within an existing earthen embankment constructed along the west bank of the Russian River. The RDS structure is located approximately 1 mile from Westside Road along the embankment. The pump house floor is approximately 12 feet above the top of the embankment and the supporting caisson is founded at Elevation[1] +18 feet (47 foot embedment depth). A 72-inch-diameter CMP connects laterally the reinforced concrete intake structure to the RDS structure from the river and the pumps discharge into the adjacent desilting ponds. Structural cross section S_1 shown on Figure 3, illustrates the RDS and associated pipeline configuration.

The embankment geometry varies along its length to accommodate several structures and changes in site topography. Within the limits of this project, the embankment has a crown elevation of approximately +65 feet, and toe elevations of approximately +27 feet and +39 feet on the river side (east) and desilting pond (west), respectively. Embankment slopes vary from approximately 2:1 (horizontal:vertical) to 3:1 (horizontal:vertical) along the west side and 1.3:1 (horizontal:vertical) to 4:1 (horizontal:vertical) along the east side. These slopes are interrupted by a series ramps for vehicle access. Figure 2 illustrates the varying topography across the project site. The site is generally vegetated with trees and bushes typically found along the river.

[1] Project Elevations based on NGVD 29 Datum.

5.0 Field Investigation

Field investigations at the project site included the current field investigation performed by HDR as well as those performed by MMI/Geosyntec and Brunsing Associated, Inc. HDR's field investigation included visits to the site, advancement of Cone Penetration Tests (CPT), and borings, in addition to measurements of groundwater wells. Boring and CPT locations are presented on Figure 2.

Exploration logs for HDR's and explorations by others are presented in Appendix A and B respectively. Geotechnical laboratory test results for HDR's investigation are presented in Appendix C.

5.1 Current Subsurface Investigation

The subsurface investigation for this scope of work included:

- A total of 3 onsite meetings with the Water Agency to determine the limits of the project and subsequent survey data needs, as-built utility locations and utility clearance with NorCal Geophysical Surveys were conducted.
- Conducting 7 CPTs
- Conducting two hollow stem auger borings, and
- Limited laboratory testing
- An HDR field representative was onsite fulltime observing and documenting the field activities.

Prior to the advancement of CPTs and borings HDR made three site visits to meet with the Water Agency in order to develop acceptable exploration methods (CPTs and Hollow Stem Auger Borings) and locate underground and overhead utilities. Furthermore, Underground Service Alert (USA) was notified at least 48 hours prior to beginning the field operations. Site visits were made on March 1, 2013, April 18, 2013 and April 19, 2013 to meet with the design team, walk the site and locate utilities. Utilities were located by SWCA on April 18, 2013 based on as–built drawings. The utility locations were confirmed by NorCal Geophysical Consultants on April 19, 2013 using instrumentation. Based on HDR's site walks, review of the previous studies and the located utility layouts, boring and CPT locations were staked in the field.

5.1.1 Cone Penetration Tests

On April 22 and 23, 2013, Gregg Drilling advanced seven CPTs using a truck-mounted 20-ton cone apparatus in general accordance with ASTM D5778. The piezocone assembly used for this project has a cone tip with a 60-degree apex, and a tip area of 10 square centimeters (cm^2). The friction sleeve has an area of 150 cm^2. The piezocone had a pore pressure (u_2) transducer near the base (shoulder) of the cone tip. The CPT probe was hydraulically pushed into the ground at a rate of 2 centimeters per second (cm/s). As the cone advanced, additional cone rods were added such that a "string" of rods continuously advanced through the subsurface soils.

Tip resistance (q_t), sleeve friction (f_s), dynamic u_2, and tip inclination were recorded at 0.02-meter intervals. Information collected during each push was stored digitally as ASCII formatted data on an on-board computer. The log of each CPT sounding is presented in Appendix A. The CPT log provides graphical plots of q_t, f_s, u_2, and friction ratio (f_s/q_t). The logs also include interpreted Soil Behavior Type based on the Robertson (1990) classification systems.

5.1.2 Hollow Stem Auger Borings

Borings were advanced on April 30, 2013 by Clear Heart Drilling. As a means of addressing the Water Agency concerns while collecting accurate blow counts and quality samples, boring methods employed consisted of using a truck mounted hollow stem auger, where the augers were plugged and filled with water to have a positive head pressure. Blows per foot and samples were obtained by driving a Standard Penetration Test (SPT) sampler using an automatic 140 pound hammer with a free fall of 30-inches. The SPT sampler had a 2.0-inch outside diameter and a 1.5-inch inside diameter. No liners were used during sampling. The depth of collected drill cutting samples and driven soil samples were recorded on boring logs by an HDR engineer in the field. Boring logs are presented in Appendix A.

5.2 Previous Investigation

Previous investigations by MMI/GeoSyntec, and Brunsing Associates were performed at the project site under various scopes of work. These investigations included hollow stem auger borings, monitoring well installation and Seismic CPTs. These investigations focused on seismic hazards along the Russian River and separately, the proposed fish passage improvements structures. Soil testing was included in these previous investigations. Soil testing included moisture content, densities, gradation, and unconfined compression triaxial testing. The boring and CPT logs along with laboratory results have been included in Appendix B.

5.3 Laboratory Testing

Soil samples collected during the current field investigation program were visually classified and recorded on the boring logs. Select samples were transported to a soils laboratory for -200 testing. Tests were performed in general accordance with ASTM standards. Test results were reviewed, used in HDR's analysis, and presented on the boring logs as well in Appendix C.

6.0 Regional Geology

The site is situated in the active floodplain of the lower Russian River in Sonoma County, California. The river alluvium extends to depths in excess of 80 feet below ground surface, and is composed primarily of silty to clean poorly-graded sand, with occasional layers of gravel and cobbles. It is possible that the deeper portions of this alluvium are older than earliest Holocene (approximately 11,400 years before present). Based on geologic mapping by the California Geological Survey (CGS) (Delatte, M.P., and Koehler, R.D., 2009), the Holocene alluvium of the active river channel is flanked to the east and west by Quaternary stream terrace deposits. Man-made fills have been placed along the banks of the Russian River in the form of earthen embankments as well as grading related to construction of above and below grade facilities that resulted in fills that range in thickness of 3 to 47 feet. A geology map of the region is presented on Figure 4.

These materials are in turn likely underlain in part, and are flanked by Tertiary (65.5 to 1.8 million years before present) marine sediments attributed to the Wilson Grove Formation of late Pliocene to late Miocene age (Delatte and Koehler, 2009). At depth metamorphic rocks of the Franciscan Complex of Cretaceous age (65.5 to 145.5 million years before present) underlie the area of the site (Koenig, J.B., 1963). The metamorphic rocks comprise a wide array of rock types, including volcanic, metavolcanic, and metasedimentary rocks.

7.0 Subsurface Conditions

The collective CPT and boring data indicate that the site is underlain by fill and river alluvium that consist of loose to dense sand and silty sand, and medium dense to dense sandy gravels.

Fill was identified in all borings and CPTs conducted at the site. The lateral limits of deep fills were confirmed with as-built drawings and construction photos provided by the Water Agency. Given embankment geometry and topography of the site the fill thickness varies from 3 feet to 47 feet. The fill materials encountered in the explorations generally consisted of medium dense to dense silty sand, silty sand with clay, stiff sandy silt and stiff clayey silt.

Groundwater elevations at the site vary seasonally and depend on whether the Water Agency is diverting water from the river into the desilting ponds and if the inflatable dam has been raised. At the time of this investigation, the Water Agency had been diverting water from the river into the desilting ponds for approximately 2 weeks and the dam was not raised. The water elevation in the desilting ponds was being maintained at an approximate Elevation of +48 feet. Existing wells within the project limits were measured with a water sounder at the time of the latest CPTs on April 22, 2013. Measurements at that time recorded water Elevations at +27 feet and +30 feet for wells M-3 and M-4, respectively. The Russian River was observed to be at approximate elevation of +32 feet during this field investigation. However, the elevation does fluctuate throughout the year as a result of the inflatable dam usage and seasonal storms. River elevations are generally between +32 and + 45 feet.

Two geologic cross sections (sections A-A’ and B-B’) were developed from the data obtained during the subsurface investigation. Cross section alignments were selected at locations that represent locations of interest for slope stability and project impact. The alignments are presented on Figure 2. These cross sections were used to develop the general substrata conditions and perform subsequent analysis. Cross sections are presented as Figures 5 and 6.

8.0 Geologic Hazards

The project site is within a seismically active region of California. HDR evaluated the potential for earthquake induced geologic hazards including ground shaking, liquefaction, lateral spreading, differential dynamic settlements and ground rupture. The results of HDR's evaluation regarding seismic considerations for the project site are presented in the following sections.

8.1 Faulting

Jennings, C.W., and Bryant, W.A., (2010) show Quaternary faults, defined as faults that have been active within the last 1.8 million years as defined by CGS Note 31, trending northwest both west and east of the site. The nearest of these, the potentially-active Mount Jackson Fault, is located approximately 0.8 kilometers (km) south-southwest of the site. The potentially active Trenton Fault has two mapped traces; one trace is approximately 1.6 km east of the site, and the other trace is approximately 0.8 km to the north-northwest. If the two traces are connected beneath the river alluvium (this is likely), the location of the fault plane nearest the site is approximately 0.8 km north of the site.

The active fault (active is defined as having movement within the last 11,400 years per CGS Note 31) nearest the site is the Rodgers Creek Fault, located approximately 11.5 km to the northeast at its nearest point. The active San Andreas Fault is located approximately 22.1 km southwest of the site. The site is not situated within a State of California Earthquake Fault (Alquist-Priolo) Special Studies Zone.

8.2 Regional Seismicity

The major active faults in the area include Rodgers Creek, Hayward, Maacama-Garberville and San Andreas (1906 Rupture). These faults are illustrated on Figure 6 of this report. The estimated Moment Magnitude[2] for each of the faults within 50 km of the site is presented in Table 1.

Table 1. Regional Faults and Seismicity

Fault Name	Distance (km)	Direction from Site	Moment Magnitude*
Rodgers Creek	11.5	East	6.98
Hayward-Rodgers Creek	11.5	East	7.26
Maacama-Garberville	18.3	Northeast	6.90
San Andreas - 1906 Rupture	22.1	Southwest	7.90
San Andreas- North Coast South	22.1	Southwest	7.45
Collayomi	35.8	Northeast	6.50
West Napa	46.1	East	6.50
Point Reyes	46.9	Southwest	6.80
Hunting Creek-Berryessa	51.3	East	6.90

* Maximum Moment Magnitudes were obtained from

[2] Moment magnitude (Mw) is an energy-based scale and provides a physically meaningful measure of the size a faulting event. Moment magnitude is related to the physical size of fault rupture and movement across the fault.

Since 1800, major earthquakes have been recorded on the Hayward-Rodgers Creek and San Andreas faults. The last significant earthquake event on the Hayward fault was on October 21, 1868 when the area was sparsely populated. The U.S. Geologic Survey (USGS) developed a shaking intensity map of the 1868 Hayward earthquake of VI on the Modified Mercalli Intensity (MMI) scale from reliable reports of the event. In 1836 an event with estimated MMI of VII occurred east of Monterey Bay (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this event is approximately 6.25. In 1838, an earthquake occurred with an estimated intensity of approximately VIII-IX (MMI), with a corresponding M_w estimated at 7.5. The San Francisco Earthquake of 1906 created the most damage to infrastructure along the San Andreas Fault. This event had a maximum intensity of XI (MMI), and a corresponding M_w of approximately 7.9. Most recently, the Loma Prieta earthquake in 1989 occurred with an M_w of 6.9. The most recent significant earthquake in the area occurred on October 1, 1969, when two moderate shocks (Richter Magnitude 5.6 and 5.7) caused localized damage to structures.

In 2008, the Working Group on California Earthquake Probabilities (WGCEP) at the USGS predicted a 63 percent probability of a magnitude 6.7 or greater earthquake in the Bay Area. Table 2 provides specific probabilities for faults associated with this project. It is possible that earthquakes could occur on these faults during the lifetime of the proposed structure.

Table 2. WGCEP (2008) Estimates of 30-Year Probability (before 2031) of a Magnitude 6.7 or Greater Earthquake.

Fault	Probability (Percent)
Hayward-Rodgers Creek	31
San Andreas	21

8.3 Liquefaction, Differential Dynamic Settlement, and Lateral Spreading

During a major earthquake event along a segment of one of the nearby active faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in soil strength loss and associated vertical and lateral deformations due to soil liquefaction, differential compaction and lateral spreading. Soils most susceptible to these phenomena is loose, clean, uniformly graded sands and low plasticity silts that are relatively free of clay.

Liquefaction is the transformation of a soil to a liquefied state. The soil, which is saturated and not drained, temporarily loses strength as the soil particles lose particle-to-particle contact and become suspended as a result of excess pore pressure build up. Pore pressures build up during strong cyclic loading from earthquakes, pile driving, etc. Settlement then occurs as the excess pore pressures dissipate. Deformations are driven by static shear stresses after the cyclic stresses have triggered the liquefaction. Similarly, cyclic mobility is another type of deformation resulting from cyclic loading. However, cyclic mobility occurs when the static shear stress is less than shear strength of the liquefaction soil. These deformations develop incrementally during cyclic loading and are the resultant of shearing soft clays and plastic silts.

Lateral spreading is a phenomenon in which the surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Mobilization of the shear zone(s) enables the surficial block to move downslope toward a free face by cyclic loading and gravitational forces. Heterogeneity in the subsurface materials influences the distribution of and magnitude of liquefaction-induced ground deformations.

Differential dynamic settlement occurs in loose relatively dry (non-saturated), cohesionless soil. During cyclic loading these materials densify as voids are reduced and the soils moves towards a critical state. Settlements can be the resultant of the densified materials. Section 8.1.2 discusses the results of HDRs analysis further.

8.4 Ground Rupture

In general, ground rupture closely follows the trace of geologically young faults. The site is not within an earthquake zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act. Furthermore, as discussed above in Section 8.1 no known active faults cross the project site. Therefore, HDR concludes that the risk of ground rupture to be very low.

9.0 Discussions and Recommendations

As discussed above, the intent of the ground improvements are to mitigate site deformations resulting from seismic loading at the project site. This includes mitigation of potential lateral seismic loads for the proposed fish passage project and reduction of deformations for the existing RDS structure. The new fish passage and intake structures are planned to be immediately adjacent to an existing inflatable dam and supported on a pile foundation and sheet pile walls. Based on review of the 65 percent fish passage improvements drawings and several schematic renderings, the currently proposed project includes a partially subterranean viewing gallery, a fish ladder, intake structure and roadways. The viewing gallery will be structurally connected to the fish passage facility and is proposed to be separated from the soil embankment on the west by a retaining structure. Currently, the retaining structure is proposed to consist of sheet piles with free space (void area) offset from the gallery walls of 18 inches and deadmen. The proposed sheet pile wall will share the eastern boundary of the ground improvements. The design displacements along the eastern embankment slope considered in this report are based in part on the maximum allowed displacement of the sheet pile wall of 12 inches. Post ground improvement geotechnical design parameters for the fish passage foundation are being provided by Brunsing and, therefore not included in this report. As discussed above, the RDS structure was constructed in 1971 and is founded in loose to medium dense fill materials. The RDS caisson and pump station may be susceptible to toppling and shearing as a result of lateral spreading, liquefaction induced slope instability and liquefaction induced settlements.

The following sections discuss the design approach and recommendations for the ground improvements to mitigate deformations associated with geologic hazards.

9.1 Design Criteria

The basis of design for the ground improvement project was developed in conjunction with HDRs structural engineering group and the intended use of the proposed and existing facilities on site. To develop the basis for a design earthquake event, HDR reviewed the site geologic setting, the regional fault characteristics, the Water Agency's Local Hazard Mitigation Plan (2012 Update version) and the proposed Fish Passage structural design. The basis of design is founded on the allowable lateral deformations of the fish ladder structure and the adjacent sheet pile walls. The fish ladder is positioned to encompass the western abutment of the existing inflatable dam. A space will be constructed between the rubber dam and the fish ladder providing the fish ladder with an allowable lateral deformation of ½-inch at the foundation level. The allowable lateral deformation is associated with an earthquake event of 10 percent in 50 years as developed for life safety using the California Building Code (CBC)/ASCE 7. In order to accommodate the lateral deformations due to anticipated lateral pressures developed during a large seismic event, the proposed sheet pile wall separating the embankment from the fish passage structure will have an 18 inch clear setback. The sheet pile wall is proposed to be located along the western edge of the fish passage project footprint. The wall will range for 10 to 30 feet tall and will gail lateral support with the used of tie rods and deadmen. The deadmen are designed to be embedded into the improved embankment materials as shown in Figure 2.

Performance criteria for the improved soils is to buttress the embankment against catastrophic failure and reduce retaining wall deformations so as not to exceed the allowable lateral deformations used in the fish passage structural design. Operational displacements of the fish passage structure are on the order of ½ inch; specifically the designed lateral displacements at the fish passage sheet pile wall are a maximum of 12-inches.

Liquefaction and differential compaction densification were reviewed for the most critical groundwater elevations. Critical groundwater elevations were determined using the high groundwater elevation of +45 feet and low groundwater +32 feet for typical winter and summer elevations, respectively. Groundwater elevations that resulted in the largest deformations were used to develop the recommendations herein.

9.2 Seismic Hazards

9.2.1 Seismic Design

Applicable data from the aforementioned geotechnical investigations for this site were used to develop the site specific parameters. Using the shear wave velocities recorded in the CPTs by GeoSyntec, as well as, CPT data from this investigation and standard penetration test blow count results from the boring logs, a site classification of Site Class F per 2010 CBC/ASCE 7 was determined for unimproved site conditions. The site classification is a result of the potentially liquefiable layers in the substrata. HDR proposes herein to improve the substrata insitu densities. Therefore a site classification D should be used for future structural designs within the limits of the ground improvement project.

Site specific information was used to develop accelerations used in the development of the recommendations presented herein. Several deterministic and probabilistic styles were incorporated as part of the development. These included Next Generation Attenuation models, Deaggregation tools and code methodologies. Further detailed discussion regarding analysis using Next Generation Attenuation models and USGS Hazard Tools to support the acceleration selected for design are included in Appendix D.

HDR anticipates that the Rodgers Creek Fault, approximately 11.5 km from the site, will have the most significant influence on the sites seismic risk level. Based in review of the site conditions and the level of structural performance for this project the selected operational and design parameters for this project are PGAs of 0.44g and 0.69g, respectively, with an M_w of 7.1. These values are similar to those values reported for River Diversion Structure/Inflatable dam in Table 4 of the Water Agency's 2012 LHMP.

9.2.2 Soil Liquefaction and Associated Hazards

9.2.2.1 Liquefaction

As detailed in Section 8.3, liquefaction is a phenomenon in which the strength and stiffness of a saturated soil is reduced by earthquake shaking or other rapid loading where excess pore pressure are generated.

Generalized substrata conditions are illustrated in cross sections taken along two locations, as shown on Figure 2, that were determined to best represent the critical topography and influence on the overall project. The cross sections are presented in Figures 5 and 6. The cross sections assume that the fill materials used to construct the embankment were placed in relatively planar lifts by dozers and backhoes. Furthermore, the cross sections were constructed using both CPT and borings. However, CPT data was considered to provide more reliable results than the borings. This is due to the methodology described in Section 5.1.2. Also, an assumed high groundwater elevation of +45 feet was used for this assessment.

Liquefaction triggering was calculated using the methods presented by Idriss and Boulanger (2008). Available data was corrected for fine contents and used to calculate Cyclic Stress Ratios (CSR) and Cyclic Resistance Ratios (CRR). Fine contents were obtained from laboratory testing performed on soil samples collected from nearby borings.

The current estimates of liquefaction induced vertical deformations of the embankment vary between 2.5 and 7.7 inches Calculated vertical deformation for each CPT are summarized in Table 3 below and individual plots are presented in Appendix E.CPTs H-CPT-4 and 5 were advanced in the vicinity of the existing RDS and Well Fields Control structures. Liquefaction estimates using the data obtained in these two locations indicate vertical deformations on the order of 4 ½ inches. Estimated deformations along the river side of the project within the proposed footprint of the fish passage project are estimated to be 6 inches vertical.

Table 3. Vertical Deformations Due to Liquefaction

CPT ID	No. of Layers	Limits		Vertical Deformation (inches)
		Upper Elevation (feet)	Lower Elevation (feet)	
H-CPT-01	10	36	-5	5.5
H-CPT-02	11	38	-1	4.2
H-CPT-03	4	27	5	2.5
H-CPT-04	9	40	2	4.4
H-CPT-05	12	45	-10	4.2
H-CPT-06	9	43	-3	7.7
H-CPT-07	9	40	-12	4.8

9.2.2.2 Lateral Spreading and Slope Stability

Lateral spreading, as discussed in Section 8.3, may be triggered along a low strength (shear) zone that has formed within an underlying liquefied layer. The mobilization of the shear zone enables the surficial block to move downslope toward a free face by earthquake and gravitational forces. As discussed above, liquefaction is likely to occur at the site during the design event. Given the embankment topography and potential risk of liquefaction at the project site, HDR has determined that the subsequent risk of lateral spreading is high. Lateral

deformations for existing site conditions were calculated using Ishihara (1985) using the computer program NovaCPT by NovaTech. NovaCPT results varied between 18 to 95 inches. Given the sites embankment topography, lateral deformations were also analyzed using block sliding method presented by Bray and Travasarou (2001).

Review of the CPTs, boring logs and surface conditions used to develop Cross Sections A-A' and B-B' indicated the potential loss of support along the slope toes as a result of liquefaction. This was condition was reviewed during HDRs slope stability analysis. As shown in Appendix E, slope stability runs, the critical slope failures were not deep enough to loose toe support through liquefaction of materials beyond the slope toes. The subsurface conditions along the river side that would appear to fail out into the river would require a failure surface to shear through deeper, denser materials with higher fines content than shallower strata's. Thereby, forcing the critical shear zones at above elevations -15 feet. The topography on along the eastern slope contains features that provide natural buttresses. Liquefaction in the lowest areas would have to occur and would be confined by the other features. Also, like the river side, the deeper failures would be in denser, siltier materials, making the shallower failure more critical.

After the areas of improvement were determined, HDR conducted slope stability analysis using the GeoStudios SlopeW (version 2012) software to evaluate the location of failure surface that have a static slope stability factors of safety equal to 1.0 or less assuming reduced strengths within the susceptible sand layers (show in Figures 5 and 6). Figures 5 and 6 do not show all of the layers as contiguous however these layers were modeled in SlopeW as being contiguous to be conservative. These layers (strata) were identified in the triggering analysis (Appendix E) as strata that required liquefaction and differential compaction densification triggering mitigation. Soil properties developed for potentially liquefiable soils are presented Tables 4 and 5 and in Appendix F. Slope stability deformations were estimated using the Bray and Travasarou procedure and were calculated to vary between 18 to 60 inches.

9.2.3 Differential Compaction Densification

Differential compaction densification occurs in relatively dry, loose clean sands. The field investigation and construction documents indicate that the site conditions present a moderate risk level of localized deformations in the substrata materials above Elevation +32 feet. An elevation of +30 feet was used as the representative low groundwater condition to analyze differential compaction densification to be conservative. This condition provides that the site has a moderate susceptibility to differential compaction densification. Differential compaction densification was analyzed using Tokimatsu and H.B. Seed (1984). Current soil condition deformations vary between 2 and 6 inches.

9.2.4 Ground Improvement

Based upon the project site and operational constraint (discussed in Section 9.2), HDR judged that improving the insitu soils in-place was the most feasible option to mitigate liquefaction within the project limits. After the aforementioned liquefaction triggering analysis was

completed, analysis was conducted to evaluate the level of ground improvement (densification) needed to achieve a minimum factor of safety of 1.2 and 1.5 for liquefaction and slope stability, respectively. As stated in the original scope of work, analysis was performed to mitigate the entire project site. As the project progressed the Water Agency's operational restrictions resulted in a phased approach to the ground improvements. The current phase of ground improvements includes the areas illustrated on Figure 2. Available areas were restricted due to topographic conditions and presence of existing site improvements. The centerline of the embankment contains critical Water Agency utilities that are not subject to shut down for this project (Water Agency communication in a team meeting on April 28, 2013). Therefore, the ground improvement limits were offset from the top of embankment utility corridor by at least 10 feet.

The triggering analyses results indicate that stability is controlled by potentially liquefiable layers located between Elevation +20 feet and +30 feet along the western slopes and Elevation -10 feet and -30 feet along the eastern slopes. As discussed above, these layers appear to be relatively uniform and continuous across the project site. Shallower layers appear to be non-continuous in a north south trend. Deeper layers appear to have more silt content and larger Qtn and blows per foot values making layers below elevation -30 feet. Therefore, layers between +20 feet and +30 feet; and -10 feet and -30 feet appear to be the critical layers and the most likely to fail first. Localized vertical and horizontal deformations should be anticipated along the centerline of the embankment strata between Elevations +20 feet and +45 feet where improvements are not implemented. Furthermore, relatively shallow, localized failures are anticipated along unprotected portions of the embankment faces.

Slope stability analysis of the improved ground was performed as for the unimproved conditions. As described above, unimproved deformations were calculated on the order of 10 inches vertical and 95 inches lateral. The increase in $N_1(60)$ and Qtn required to increase the factors of safety against liquefaction triggering was calculated. Representative mohr-coulomb strength parameters were then developed for input in to the SlopeW stability models, summarized in Tables 4 and 5. Both circular and block failures were conducted for pseudo static slope stability analysis for the improved ground condition. Slope stability was evaluated for two river water surface elevations +32 and +45 feet, to analyze the potential for the groundwater to vary across the site. The river site Factors of Safety were calculated to be more critical (lowest) with a river water surface elevation of +32 feet. However, the results for the pond side slopes were more critical with a river elevation of+45 feet. Graphical results of the improved ground slope stability analysis are provided in Appendix F.

Table 4. Cross Section A-A' Slope Stability Input Parameters

Soil Type	Unit Wt (pcf)	ϕ' (degrees)	Cohesion (psf)	Residual Strength[1] (Sr) [psf]
Silt (ML)	115	28	200	
Silty Sand/Sand with Silt (SM/SP-SM)	110	32	0	
Silty Sand/Silty Gravel (SM/GM)	127	32	0	
Sand (SP)	100			200
Sand (SP)	100			500
Sand (SP)	100			400
Improved Soils	120	38		
Proposed Piles	120	42		

[1] Residual Strength calculated from CPT data using NovoCPT software by NovoTech

Table 5. Cross Section B-B' Slope Stability Input Parameters

Soil Type	Unit Wt (pcf)	ϕ' (degrees)	Cohesion (psf)	Residual Strength[1] (Sr) [psf]
Silt (ML)	115	28	200	
Silty Sand (SM)	127	32	0	
Silty Sand (SM)	100			200
Silty Sand (SM)	100			300
Improved Soils	120	38		

[1] Residual Strength calculated from CPT data using NovoCPT software by NovoTech

9.3 Recommendations

For the purposes of the Mirabel Ground Improvement Project, HDR recommends that a performance based design approach be used to develop ground improvement designs. Based on the results of the analysis described above, post ground improvement, CPTs should have an improved ground condition corresponding to Qtn of at least 200 tons per square foot (psf). Furthermore, the strata should have improved densities that correspond to a minimum SPT blow count of $N_1(60) = 25$ blow per foot.

Prior to mitigation the embankment is calculated to experience up to 10 inches of lateral movements and 8 inches of vertical settlements along the top of the embankment. Areas that remain unimproved will continue to have a high potential of liquefaction induced settlements. These areas are anticipated to have vertical settlements manifested at the surface on the order of 4 to 6 inches. Post ground improvements to the aforementioned densities are calculated to have lateral displacements on the order of 2 inches with a probability of exceedance of 50 percent. Liquefaction mitigation within the improved areas has a calculated factor of safety of at least 1.2 with localized areas of vertical deformations of 1 to 2 inches. These localized areas are anticipated to be on the perimeters of the improved areas as these are the areas transitioning into unimproved zones.

Post improvement slope stability deformations were estimated using the Bray and Travasarou procedure and were calculated to vary between 0.6 and 1.5 inches. Estimated slope lateral deformations are presented in Table 6 below.

Table 6. Slope Lateral Deformations Post Improvements

Location	Ky	Lateral Deformation (inches)
Cross Section A-A' River Side	0.39	0.6
Cross Section A-A' Desilting Pond Side	0.31	0.8
Cross Section B-B' River Side	0.34	0.6
Cross Section B-B' Desilting Pond Side	0.25	1.5

For design purposes, HDR recommends that lateral deformations of 3 inches along the proposed sheet pile walls be used for design within improved areas.

9.4 Ground Improvements

Ground improvements are recommended as means of lowering the risk level associated with the current seismic hazards. HDR reviewed several options with differing abilities to lower risk levels. The following options were reviewed:

- Compaction Grouting – displaces and densifies loose granular soils, reinforces fine grained soils, by the staged injection of low-slump, low mobility aggregate grout. The low mobility grout bulbs displace surrounding soils and increase the surrounding soils density, friction angle and stiffness in granular soils. This provides an increase in relative densities and lowers the risk for settlements.
- Permeation Grouting – is the injection of a grout fluid into granular pore spaces filling voids. This decreases the void spacing and lowers the risk for settlements.
- Stone Columns – a method in which a down-hole vibratory probe compacts aggregate into a column and displace surrounding soils and increases the surrounding soils density, friction angle and stiffness in granular soils. This provides an increase in relative densities and lowers the risk for settlements.

9.4.1 Compaction Grouting and Permeation Grouting

Compaction Grouting and Permeation Grouting would allow for a contractor to improve the substrata adjacent the RDS caisson and pile foundation. Further advantages include the potential for working around existing utility corridors, relatively low impact to the existing grades, and the ability to work in relatively tight places. However, in discussions with The Water Agency any type of ground improvement that relies on chemicals or may produce significant turbidity should be avoided. It is our understanding that there may be an adverse effect in the nearby drinking water wells should turbidity of any kind arise in nearby wells. Therefore, these two methods were not analyzed beyond conceptual discussions with the design

team including The Water Agency representatives. These methods may be considered in future phases should one or both methods be determined to produce acceptable levels of turbidity.

9.4.2 Stone Columns

Stone columns may be used to densify the potentially liquefiable substrata. Using aggregate with little to no fines, the turbidity would be generated from the disturbance of the insitu fine grained soils and will be dependent on whether the dry or wet method is used by the contractor. However, given the laboratory test data and the distances between the project site and the drinking water wells, an increase in turbidity is anticipated to be relatively low, addressing the Water Agency's turbidity considerations. A monitoring program should be developed with the Water Agency's input to measure the amount of turbidity in the river and within nearby wells that will be generated during construction. The methodology of stone column installation limits the construction to relatively open spaced areas away from structures and overhead power lines. Furthermore, since the insertion of the vibratory probe greatly disturbs the existing grades and generally causes damage to nearby utility corridors, stone column remediation cannot be performed directly below utilities or structures. As discussed above, The Water Agency has communicated to the HDR team that existing utilities located along the centerline of the embankment cannot be turned off and/or temporarily relocated, which limits area of remediation. Therefore, stone column ground improvement is limited to the areas of the project site as shown on Figure 2. Using stone columns to densify the underlying the loose foundation materials will lower the risk of the embankment slopes failing resulting from liquefaction triggered lateral spreading but will not lower the risk of liquefaction along the centerline of the embankment.

Densification by means of stone columns is considered to be suitable for lowering the seismic hazard risk level to the proposed fish passage project and the existing RDS structure. Furthermore, the installation of stone columns appears to address the Water Agency's turbidity considerations. Therefore, for this project HDR recommends that stone columns be constructed as a means lowering the seismic hazard risk level.

9.5 Construction Considerations

The existing pump station should be photographed and surveyed before, during and after the construction of the proposed ground improvements and grading for settlement. Monitoring should include photographs, survey of the structures floor both interior and exterior, crack monitors, wet utility pressures and dry utility functions. A monitoring program should be developed with the contractor's schedule of work. Furthermore, a baseline condition should be documented prior to the contractors mobilizing to the site. Regular monitoring intervals should be set for the duration of the project.

9.5.1 Site Preparation

9.5.1.1 Clearing and Grubbing

The site is broken into work limits as illustrated on Figure 2. Trees, bushes and grass should be removed within the work limits shown. All vegetation should be removed and disposed of within permitting guidelines and take into account raptors and any other environmental concerns.

9.5.1.2 Groundwater

Groundwater was measured to be between an Elevation of +27 and +30 feet when the inflatable dam was down. However, groundwater and river elevations should be anticipated to be at an Elevation of +45 feet across the site and have a potential high river elevation of 65 feet for design and construction purposes.

9.5.2 Ground Improvement Baseline

Based on the results from HDR's analysis, stone columns should be installed along the slopes of the embankment in the areas shown on Figure 2. HDR recommends that stone columns be advanced to a minimum Elevation of +15 feet along the west slopes. In addition, HDR recommends that stone columns be advanced to a minimum of Elevation of -15 feet along the east slopes. Using the Federal Highway Administrations Design and Construction of Stone Columns (FHWA/RD-83/026) and supplemented with Preibe (1995) a minimum stone column spacing and diameter estimate was developed. At a minimum, stone columns should be constructed with a maximum spacing of 9 feet on center in a square pattern; column diameters of at least 4 feet; and with a replacement area (Ar) of at least 0.25.

Stone columns are anticipated to be constructed below the groundwater table and possibly in free standing water in the desilting ponds. The free standing water will depend on the time of year that the Water Agency is diverting water.

The contractor is responsible for the development of a design and performance of a quality control test program. The test program should include testing at a representative location within the project limits to demonstrate that their design meets or exceeds the performance requirements recommended in Section 9.1.3. Minimum requirements for such a testing program are provided in the following paragraph. The contractors design should provide a spacing, configuration, and Ar of stone columns that their equipment is capable of achieving the densification\ recommendations. The contractor should also construct a test section based upon its design and conduct CPT and/or SPT testing to demonstrate that their design meets the minimum recommended densification requirements. The same spacing and Ar used in the test section and as accepted by the Water Agency's project engineer of record should be used during production.

The contractor should develop quality control testing program including a baseline and post improvement subsurface investigation (study) to provide a detailed level of current ground conditions and document the level of improved conditions. Both baseline and confirmation

should consist of CPTs and borings. CPT's shall be performed in conformance with ASTM D5778. Borings shall employ SPTs in conformance with ASTM D1586. SPTs shall use a sampler with space for a liner. No liners shall be used. Boring methods shall further be in conformance with the Water Agency drilling requirements. The quality control testing program should be provided to the Water Agency as a submittal for review and approval as part of the Contractor's design submittal.

9.5.3 Grading

It is anticipated that the contractor will need to excavate and create temporary working benches and access roads along portions of the project site to provide working pads and temporary roads to facilitate the installation of ground improvements. The contractor shall reconstruct the site to the preconstruction grades as shown in the project plans and as specified in the project specification. Bench elevations should be not lower than Elevation 44 feet along the riverside (east). The site will be regraded to restore the site grades. To reduce the active soils loads imposed on the proposed fish ladder sheet pile walls, geogrid reinforcement should be placed in the fill. Recommendations for geogrid reinforcement are provided below.

9.5.3.1 Excavations

The onsite soil conditions tend to ravel and slough easily when exposed to low overburden conditions and allowed to dry out as a result of being exposed to weather. Therefore, unshored temporary cut slopes should not be any steeper than 2:1 (horizontal:vertical). Utility trenches should be either laid back or shored. All shoring should be designed by a licensed civil engineer specializing in shoring and in accordance with CalOSHA requirements.

9.5.3.2 Fill Placement

On site soils are generally considered suitable of use as engineered fill for the purposes of this project. Engineered fill and subgrade soils should be compacted to at least 95 percent relative compaction for cohesive soils and cohesionless soils, respectively. Fill materials should be moisture conditioned to 2 to 3 percent above optimum moisture content per ASTM D 1557. Fill materials should be spread and compacted in lifts no greater than 8 inches in uncompacted thickness. Field density testing on subgrade and fill material should be performed at a minimum of one test per 500 square feet per lift. Furthermore, for fills that are not associated with utilities and will require at least ten feet of compacted fill a layer of geogrid (Tensar BX1200 or equivalent) shall be incorporated with the backfill. These areas are anticipated to be confined to the riverside of the project.

Geogrid materials used in fills shall be placed every 24-inches vertical lift starting at the subgrade level. The final layer of grid shall be overlain by at least 24 inches of compacted soil. The limits of the geogrid backfill shall stay within the limits of construction as shown on Figure 2. All geogrid seems shall have at least a 24 inch overlap and shall be placed flat and without folds and wrinkles.

All areas to be improved shall have a 5 foot cap of low pervious material. This material shall be compacted to a minimum of 90 percent compaction relative to ASTM D 1557. Fill materials should be moisture conditioned to 2 to 3 percent above optimum moisture content per ASTM D 1557.

Field density tests to determine the in-place densities of compacted fill shall be obtained in accordance with ASTM D 1556 when calibrated with ASTM D 6938. Check the calibration prior to the initial use in the field and on each type of material to be used. Periodic performance testing should be done with ASTM D 6938 to confirm ASTM D 1556 on a ratio of one sand cone for six nuclear tests.

Utilities trenches should be backfilled with fill placed in lifts of approximately 8 inches in uncompacted thickness. Backfill should be placed by mechanical means only. Jetting shall not be permitted. Trench backfill should be compacted to at least 90 percent relative compaction. Imported sand trench backfill should be compacted to at least 95 percent relative compaction and sufficient water should be added during backfilling operations to reduce the potential for soil “bulking” during compaction.

10.0 Geotechnical Services during Construction

HDR should review the final project plans and specifications to confirm that they are in general conformance with the intent of the recommendations presented herein. Furthermore, HDR should review the contractor's stone column submittal for continuity with the recommendation presented herein. During construction, an HDR field engineer should provide on-site observation during site preparation, during excavation and installation of stone columns, overexcavation and recompaction efforts. These observations will allow HDR to compare actual with anticipated soil conditions, provide field engineering where needed and to check that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

11.0 Limitations

Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, placement of fill, or excavation). Such unexpected variations in soil conditions often require that additional expenditures be made to attain a properly constructed project. Therefore, a contingency fund is suggested to accommodate such potential extra costs.

Historical explorations and testing were not performed by HDR and HDR cannot vouch for the accuracy of data and information obtained by others. The historical explorations and other subsurface data by others presented in this report were provided by the Water Agency and are presented herein in an attempt to include data relevant to this project in one document. Data by others should not be relied upon unless the originator of that data is available to confirm its accuracy.

This geotechnical study did not include an investigation regarding the existence, location, or type of possible hazardous materials. If any hazardous materials are encountered during construction of the project, the proper regulatory officials should be notified immediately.

This report is a data document that has been prepared in accordance the Scope of Work and with generally accepted geotechnical engineering practices for the exclusive use of the Water Agency and their consultants for specific application to this project in Forestville, California. It is the responsibility of the Water Agency to transmit the information in this report to the contractor for column layout and column diameter design and construction of the project. HDR will not be responsible for the alteration or misinterpretation of the information provided in this report.

12.0 References

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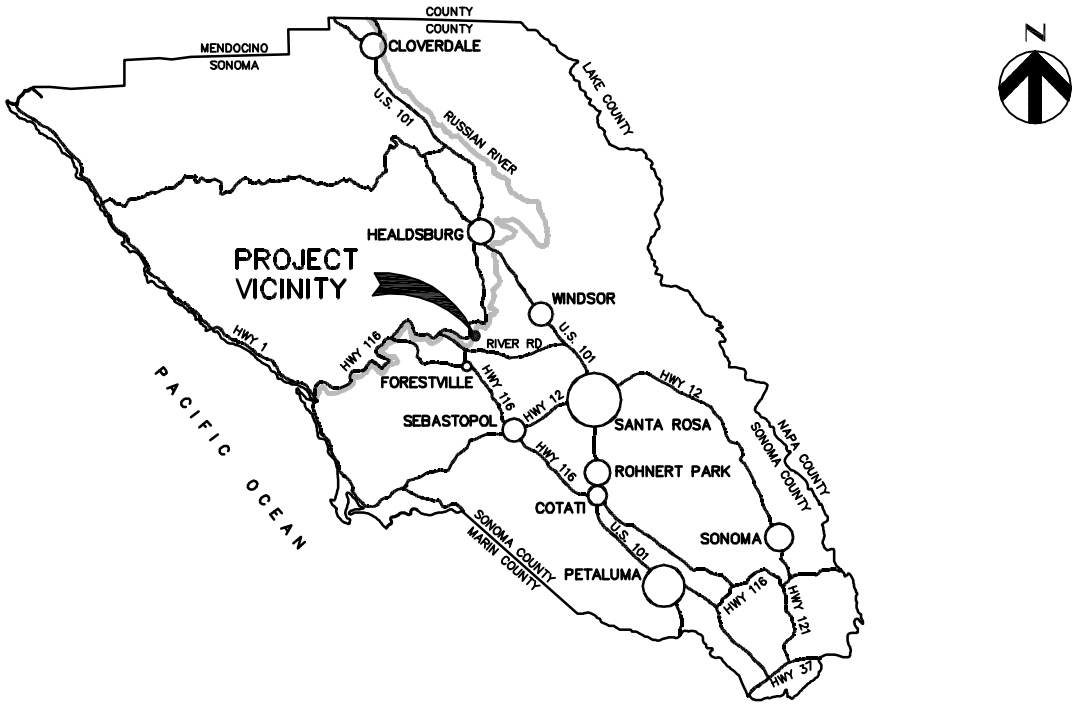
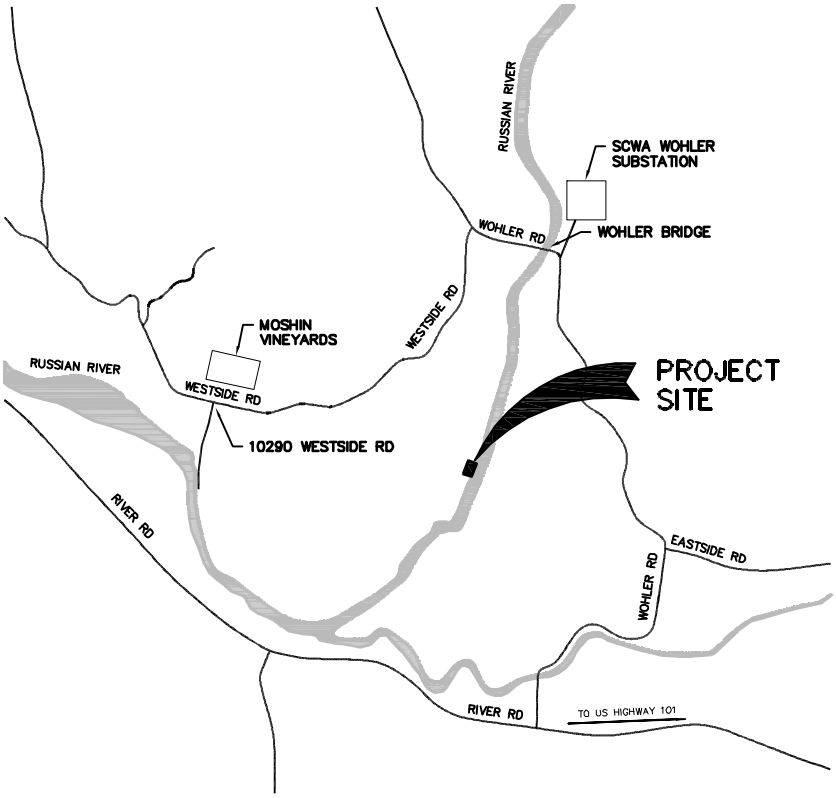
Figures



VICINITY MAP



PROJECT LOCATION

SONOMA COUNTY WATER AGENCY	SCALE: N/A	DATE: 05/23/13	MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT
	DRAWN: PVM		SITE LOCATION MAP
	REVIEWED:		
	FILE NAME: 164479_gtr_g01.dwg CONTRACT NUMBER: 164479		DRAWING NUMBER: Figure 1

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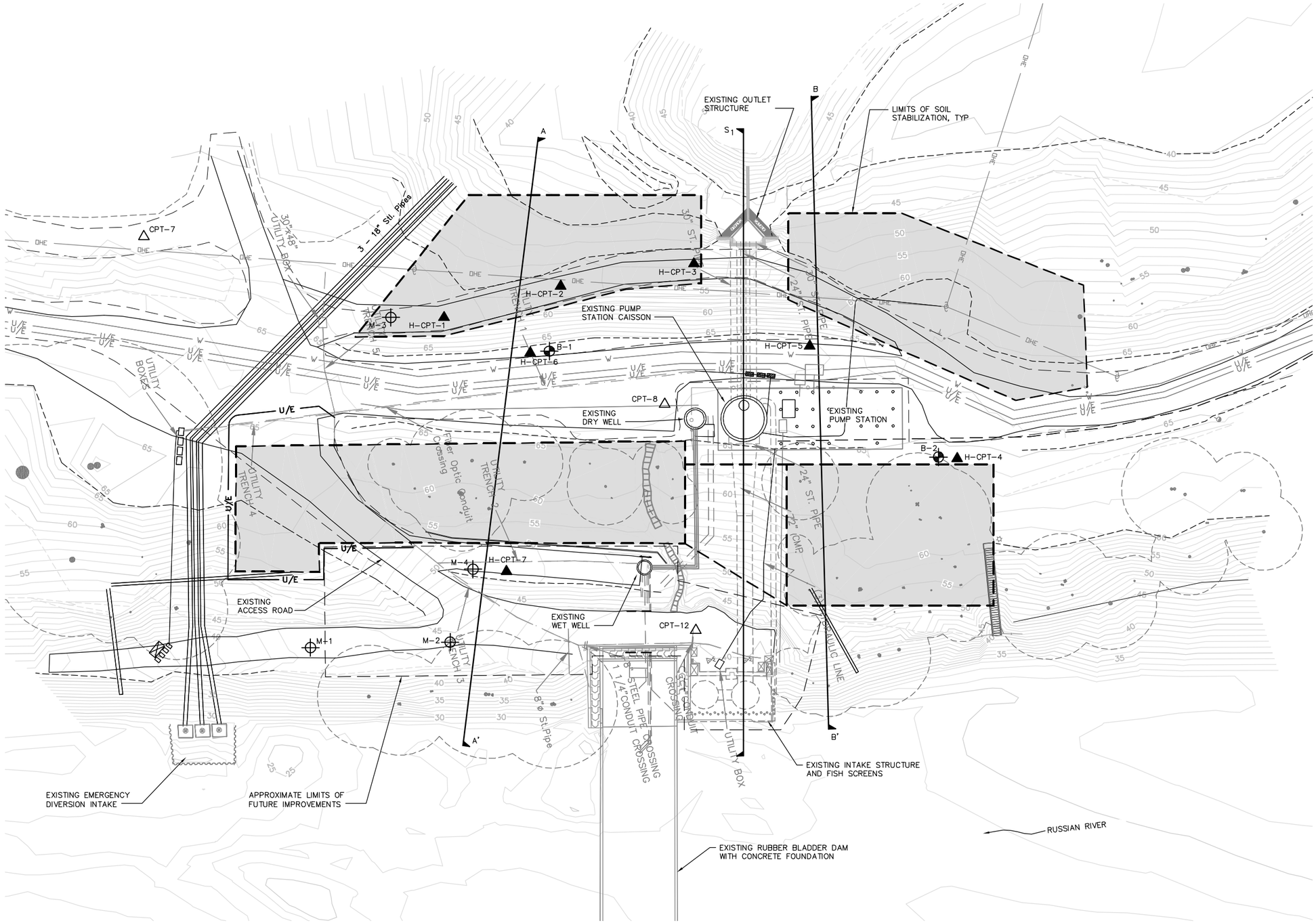

EXPLANATION

B-1	APPROXIMATE LOCATION OF BORINGS BY HDR, 2013
H-CPT-1	APPROXIMATE LOCATION OF CONE PENETRATION TEST BY HDR, 2013
M-1	APPROXIMATE LOCATION OF BORING BY BAI, 2011-2012
CPT-12	APPROXIMATE LOCATION OF CONE PENETRATION TEST BY GEOSYNTEC, 2011
	LIMITS OF GROUND IMPROVEMENTS
A A'	APPROXIMATE LOCATION OF CROSS-SECTION

BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

SITE PLAN

0 20 40 60 FT

SCALE: 1"=20'

1 F2 F4

N

SONOMA COUNTY WATER AGENCY

SCALE: AS NOTED	DATE: 8.23.2013
DRAWN: PVM	
REVIEWED:	

MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT

SITE PLAN

FILE NAME: 164479_gtr_g02.dwg
CONTRACT NUMBER: 164479

DRAWING NUMBER: FIGURE 2

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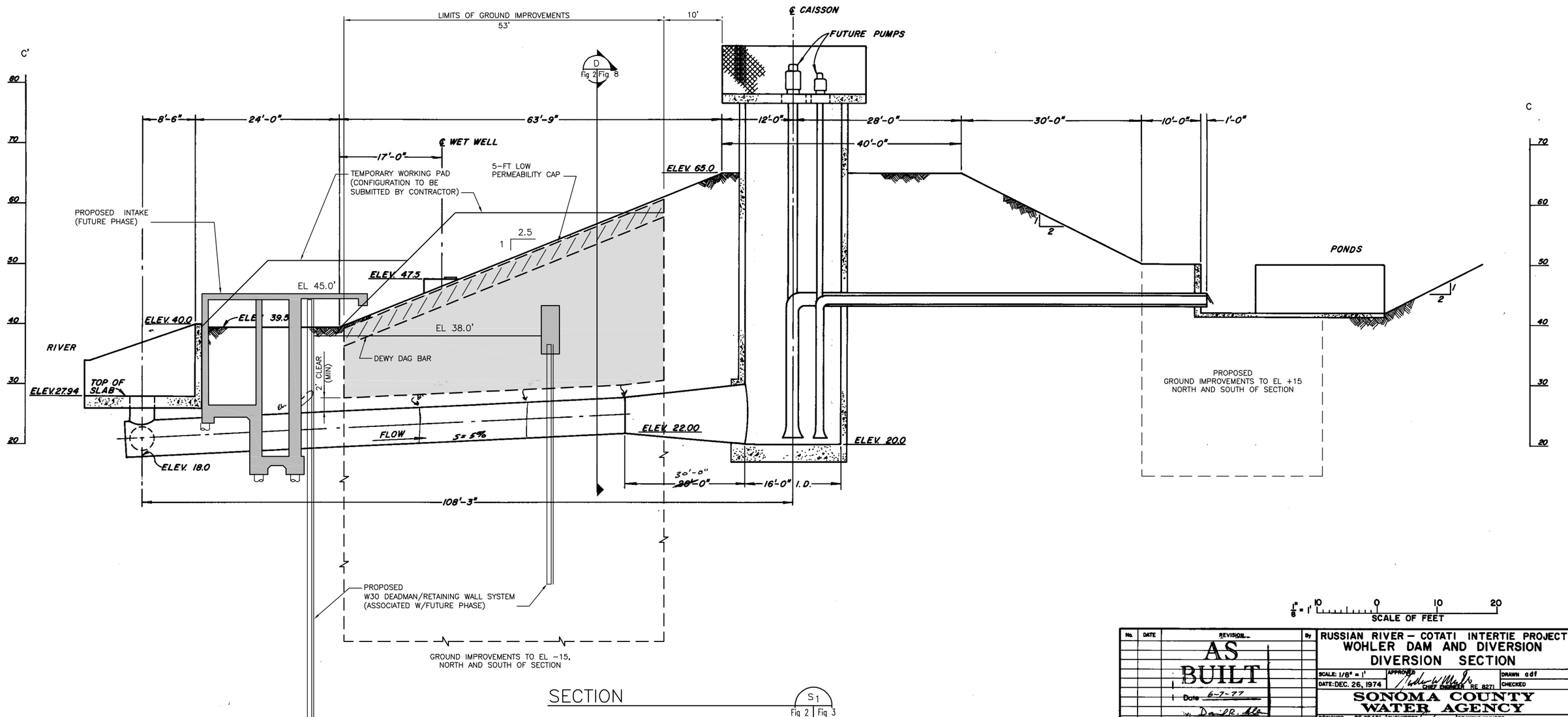

SECTION

S1
Fig 2 | Fig 3

No.	DATE	REVISION	By
		AS BUILT	
		Date 6-7-77	

RUSSIAN RIVER – COTATI INTERTIE PROJECT
WOHLER DAM AND DIVERSION
DIVERSION SECTION

SCALE: 1/8" = 1'
DATE: DEC. 26, 1974
APPROVED: Walter W. Wells Jr. CHIEF ENGINEER RE 8271
DRAWN: adf
CHECKED:

SONOMA COUNTY WATER AGENCY

DESIGNED: RE 20481
SUBMITTED: RE 10196
DRAWING NUMBER: 60-5-102.226

SHEET NO. 88

MICROFILMED

BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

SCALE: AS NOTED
DATE: 8.23.2013
DRAWN: PVM
REVIEWED:

MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT

STRUCTURAL CROSS SECTION S 1

FILE NAME: 164479_fig3.dwg
CONTRACT NUMBER: 164479

DRAWING NUMBER: FIGURE 3

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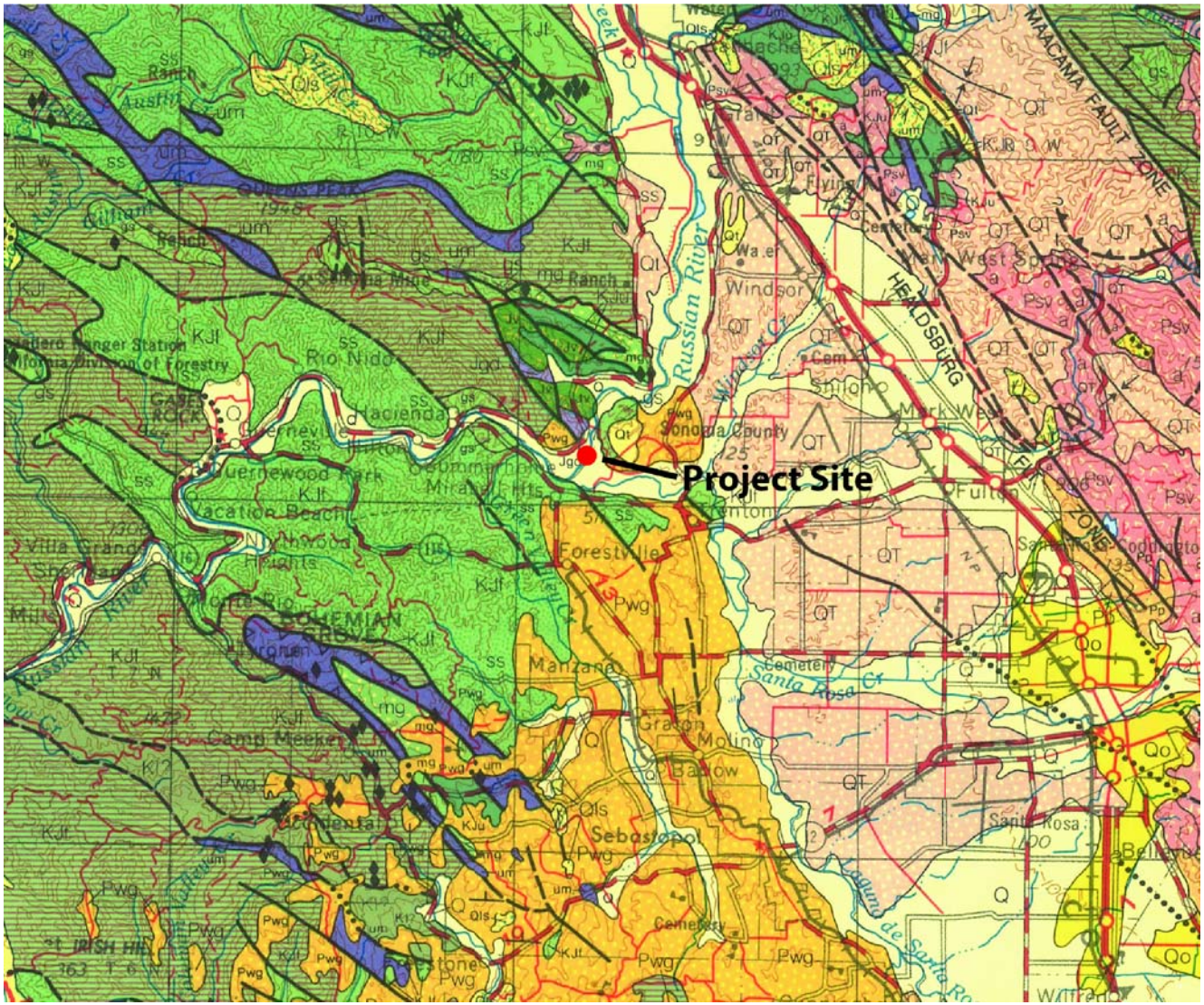


EXPLANATION
GEOLOGIC UNITS

LEGEND FOR KEY ROCK UNITS

Q	Alluvium (Holocene)
Qt	Terrace Deposits (Pleistocene)
Pwg	Wilson Grove Formation (Pliocene)
Kjf	Franciscan Complex (Cretaceous) ss = sandstone member gs = greenstone member
Jgd	Gabbro and Diabase (Jurassic)
Jv	Volcanic Rocks, mainly basalt (Jurassic)

N

NOT TO SCALE

REF: Source: Wagner, D.L., and Bortugno, E.J. (compilers), 1982, Geologic Map of the Santa Rosa Quadrangle, California: California Division of Mines and Geology, Regional Geologic Map Series.

SONOMA COUNTY WATER AGENCY

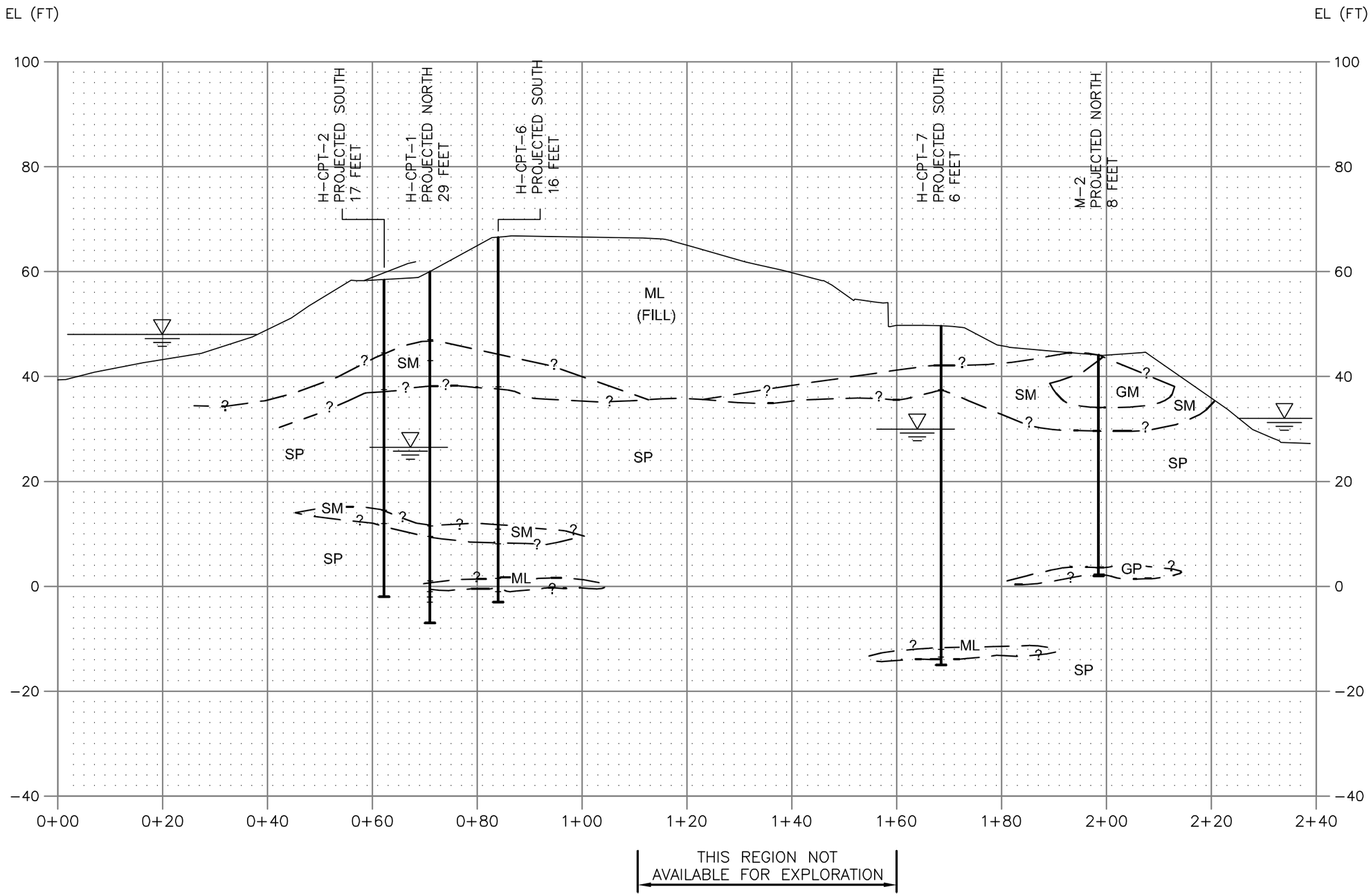
SCALE: AS NOTED	DATE: 9.2.2011
DRAWN: ---	
REVIEWED:	

MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT

GEOLOGY MAP

FILE NAME: 164479-c0_fig04.dwg
CONTRACT NUMBER: ####

DRAWING NUMBER: Figure 4



PROFILE

1" = 20'

A–A'

REF: DATUM NGVD 29

BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

SCALE: AS NOTED	DATE: 8.22.2013	MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT
DRAWN: PVM		CROSS SECTION A-A'
REVIEWED:		
FILE NAME: 164479-c0_fig05.dwg		DRAWING NUMBER: FIGURE 5
CONTRACT NUMBER: 164479		

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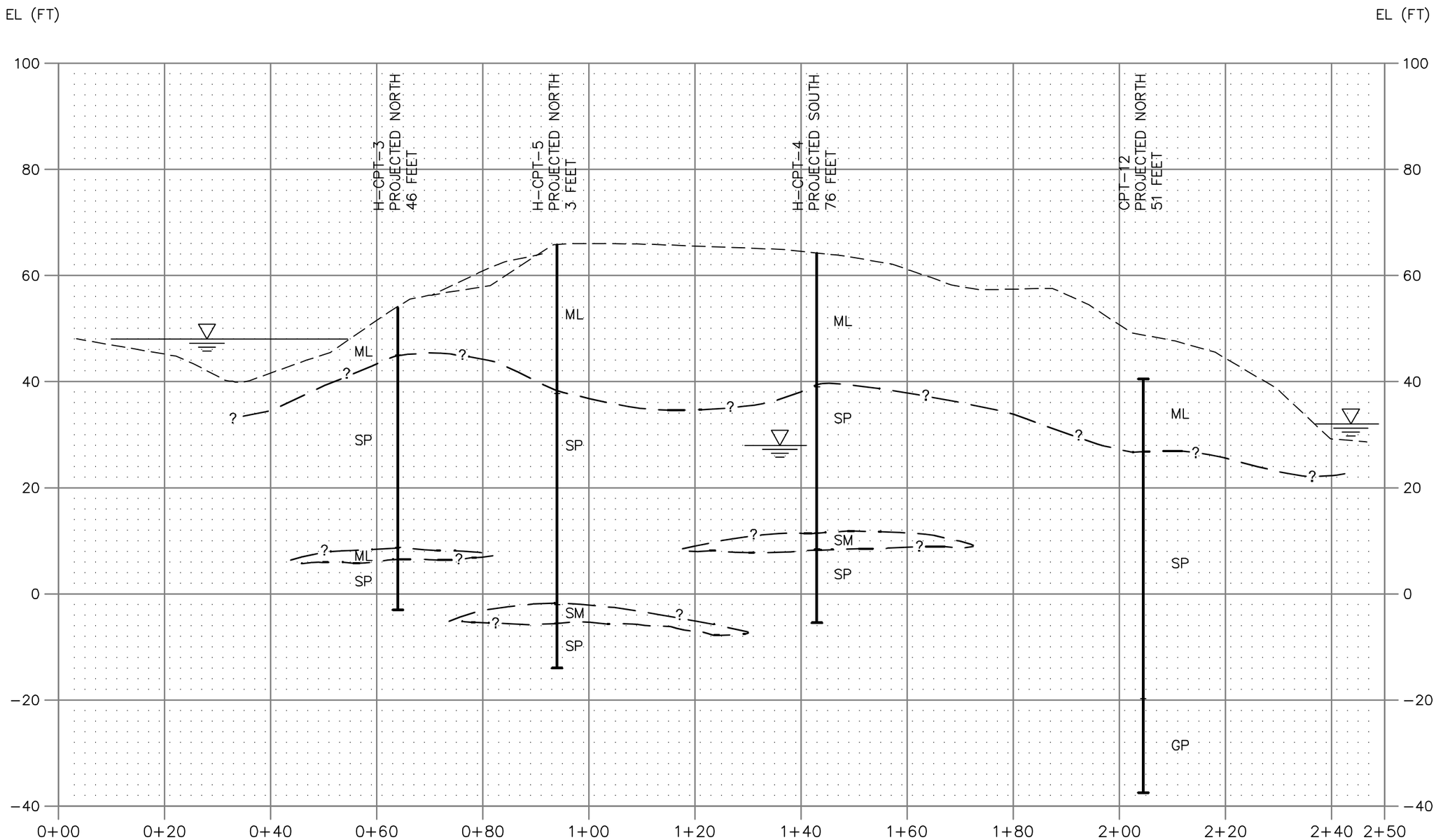

C:\pwworking\sea\d0966427\164479-C0_Fig06

BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

PROFILE
1" = 20'

B–B'

REF: DATUM NGVD 29

SONOMA COUNTY WATER AGENCY

SCALE: AS NOTED	DATE: 8.22.2013
DRAWN: PVM	
REVIEWED:	

MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT

CROSS SECTION B-B'

FILE NAME: 164479-c0_fig06.dwg
CONTRACT NUMBER: 164479

DRAWING NUMBER: FIGURE 6

C:\pwworking\sea\d0966427\164479-C0_Fig07.dwg, Faults, 8/22/2013 2:13:37 PM, pvanmeu

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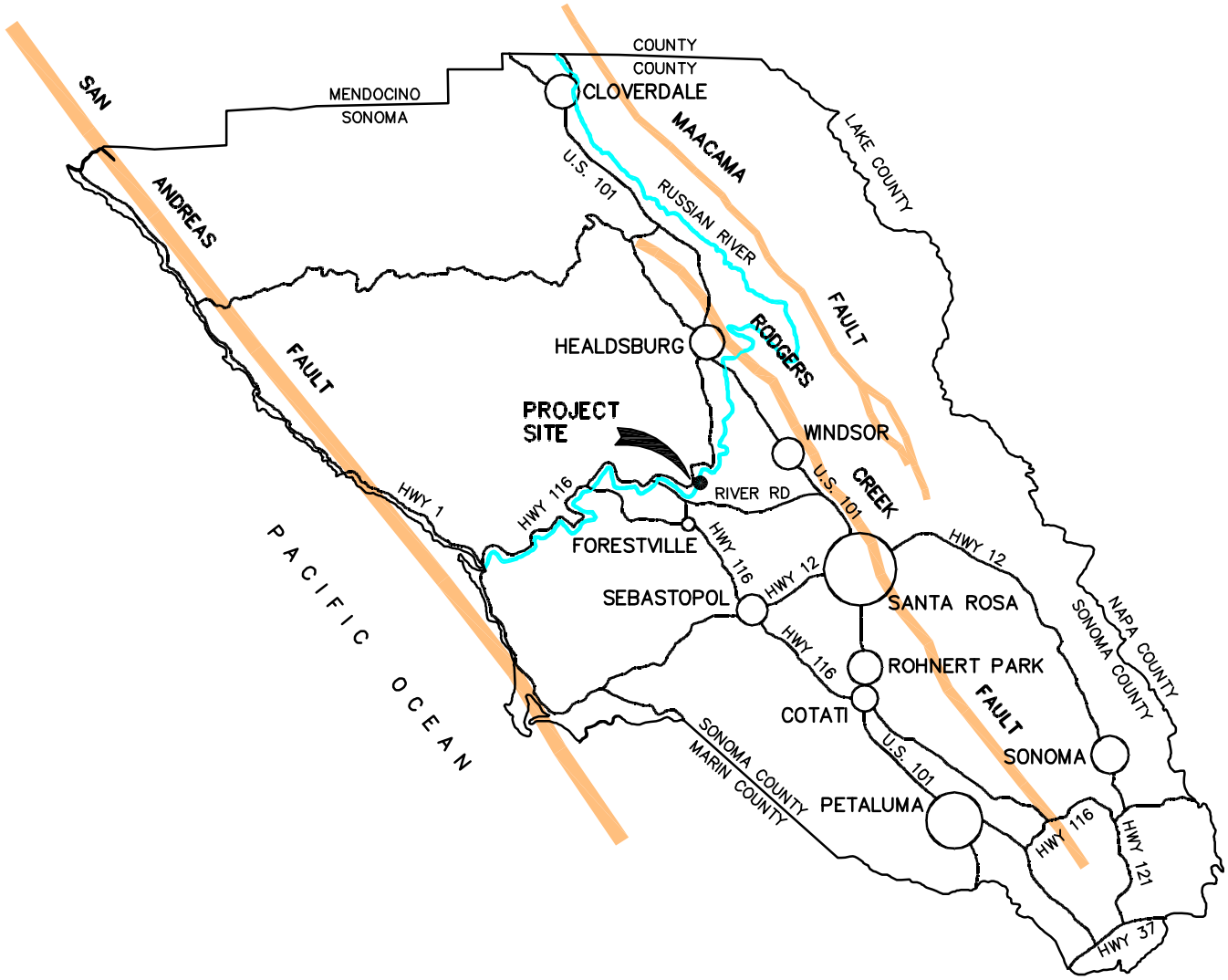


REF: Information Obtained From '2010 Fault Activity Map of California', http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html

SONOMA COUNTY WATER AGENCY

SCALE: AS NOTED	DATE: 9.2.2011
DRAWN: ---	
REVIEWED:	

MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT

FAULT MAP

FILE NAME: 164479-c0_fig04.dwg

CONTRACT NUMBER: ####

DRAWING NUMBER: Figure 7

Appendix A – Current Exploration Logs

Appendix A contains the following:

Cone Penetration Tests and Boring logs from HDRs current field investigation for this report.

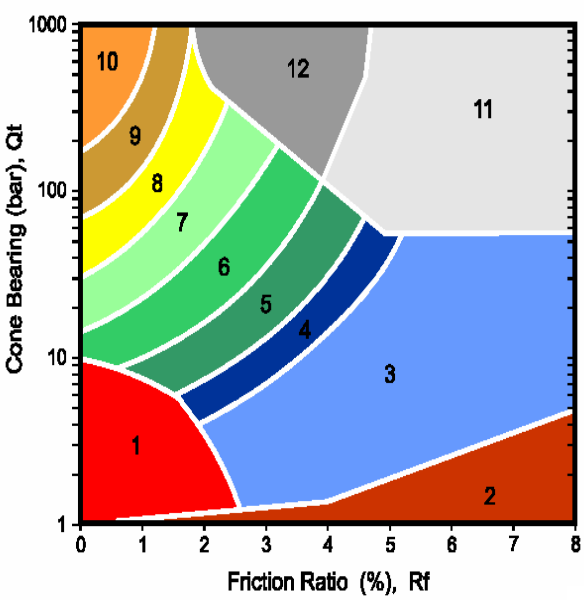
Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected from your site are presented in graphical form in the attached report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings extending greater than 50 feet, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. do not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and do not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



(After Robertson, et al., 1986)

ZONE		SBT
1		Sensitive, fine grained
2		Organic materials
3		Clay
4		Silty clay to clay
5		Clayey silt to silty clay
6		Sandy silt to clayey silt
7		Silty sand to sandy silt
8		Sand to silty sand
9		Sand
10		Gravely sand to sand
11		Very stiff fine grained*
12		Sand to clayey sand*

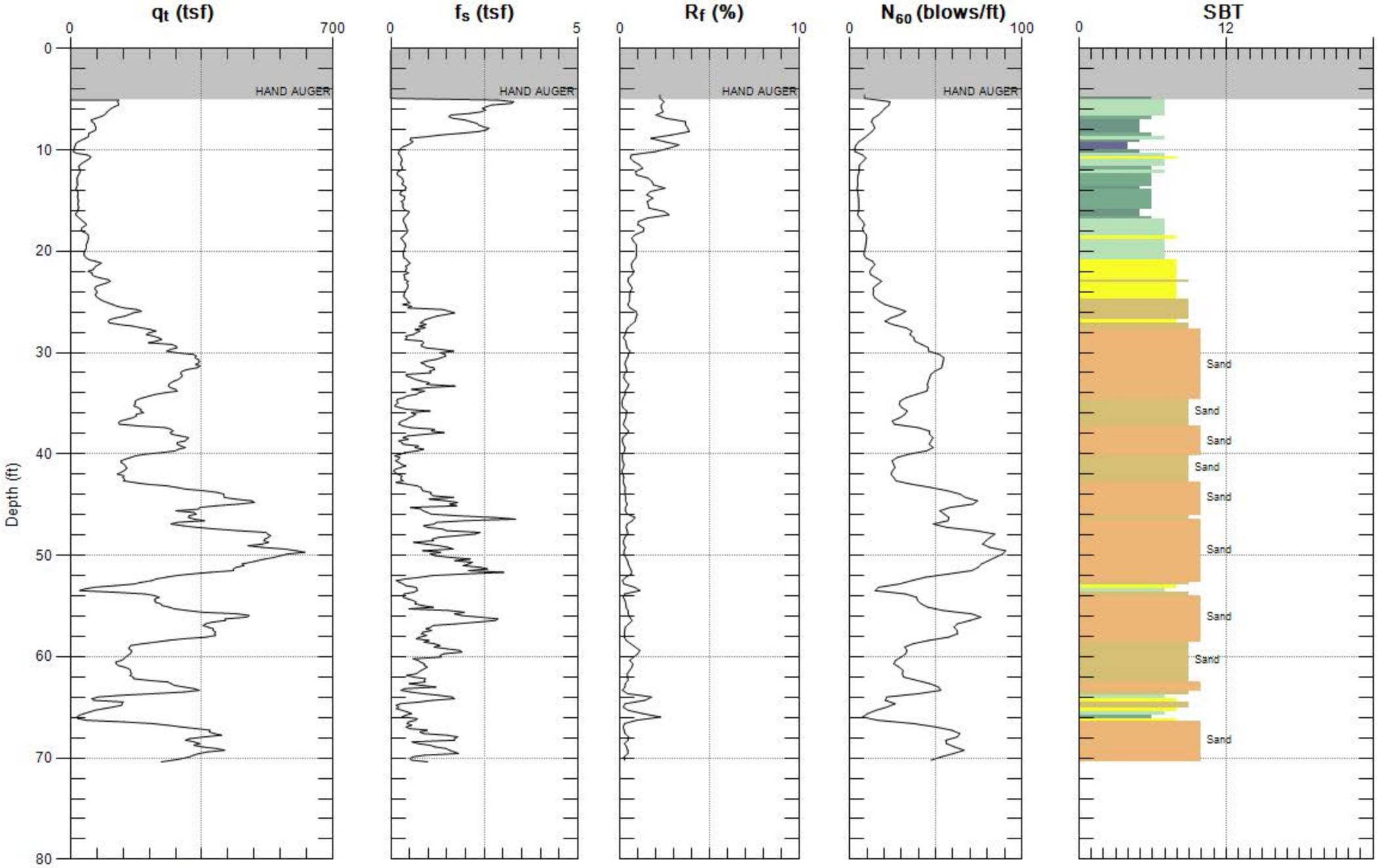
*over consolidated or cemented

Figure SBT

HDR ENGINEERING

Site: MIRABEL FISH LADDER Engineer: G.HARRIS

Sounding: CPT-1 Date: 4/22/2013 07:01

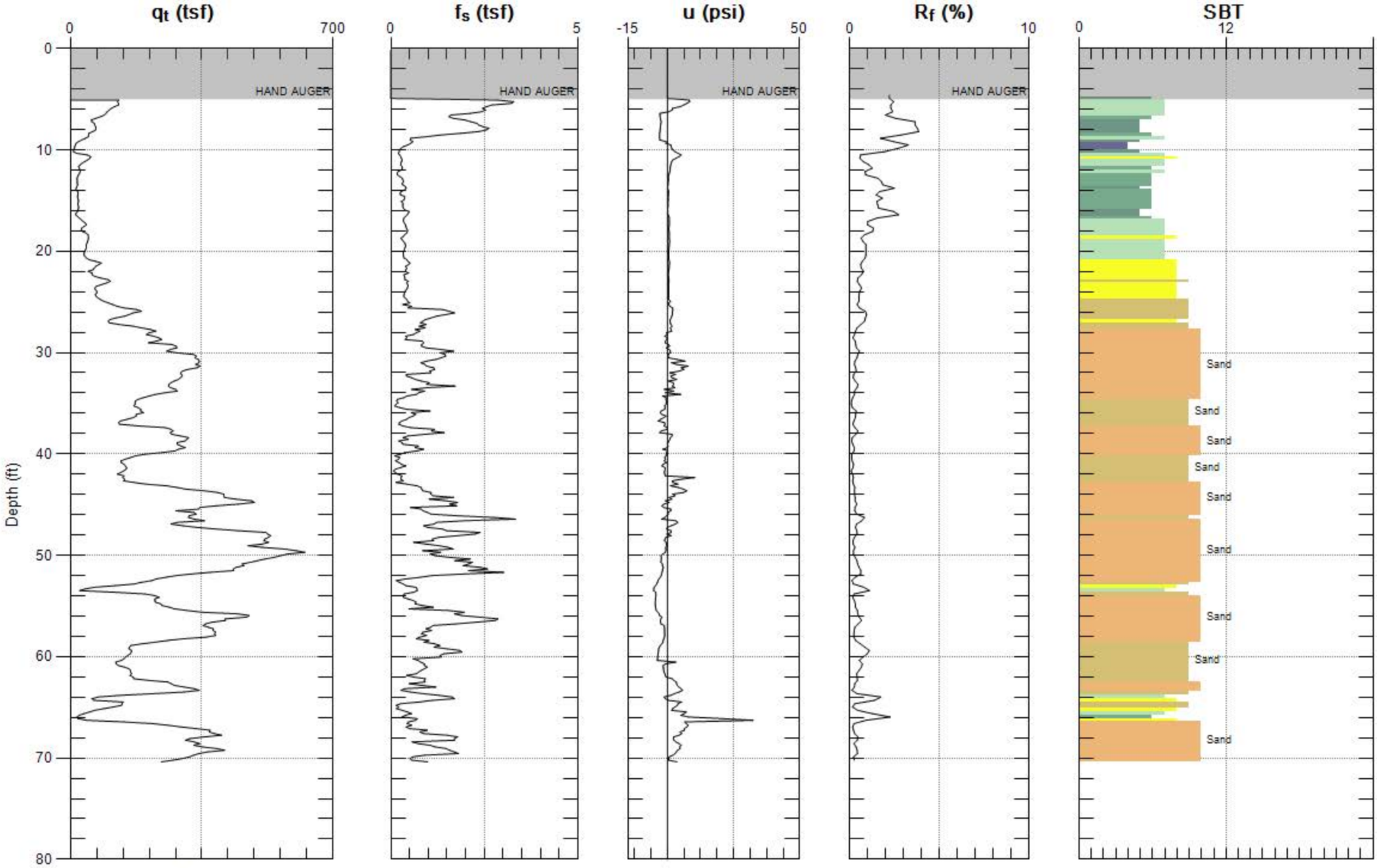


Max. Depth: 70.374 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER
Engineer: G.HARRIS
Sounding: CPT-1
Date: 4/22/2013 07:01



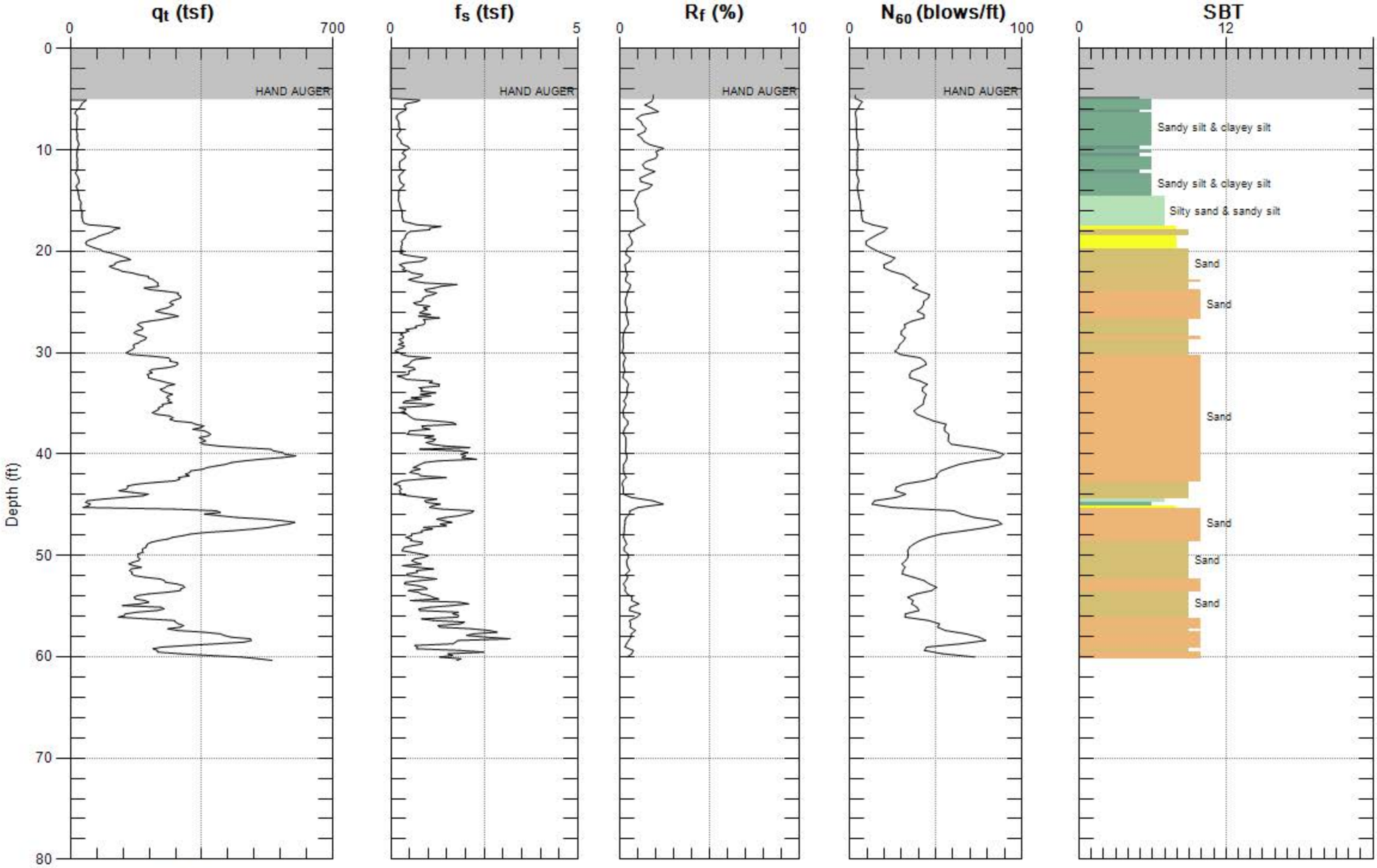
Max. Depth: 70.374 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER Engineer: G.HARRIS

Sounding: CPT-2 Date: 4/22/2013 09:57

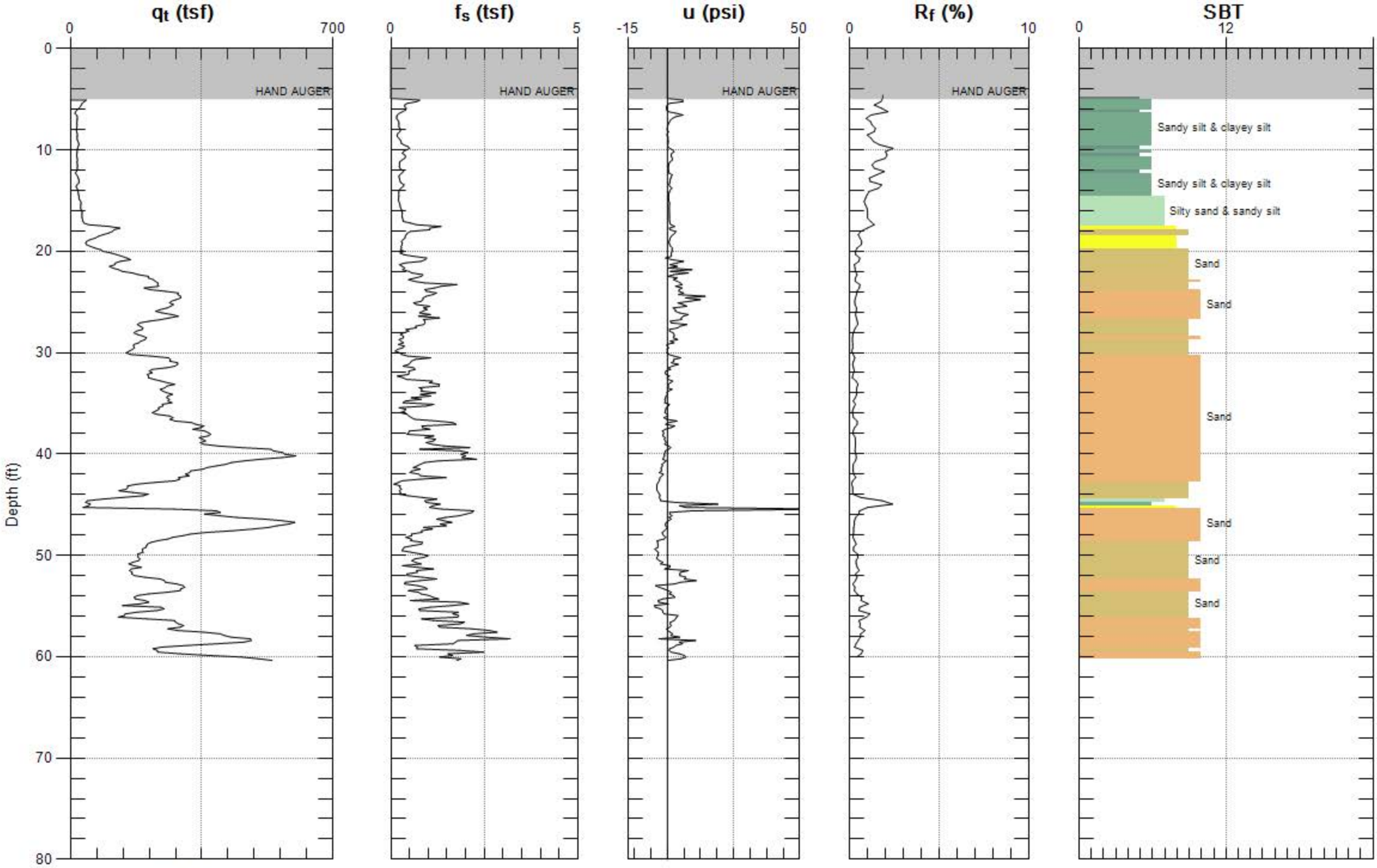


Max. Depth: 60.367 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER
Engineer: G.HARRIS
Sounding: CPT-2
Date: 4/22/2013 09:57



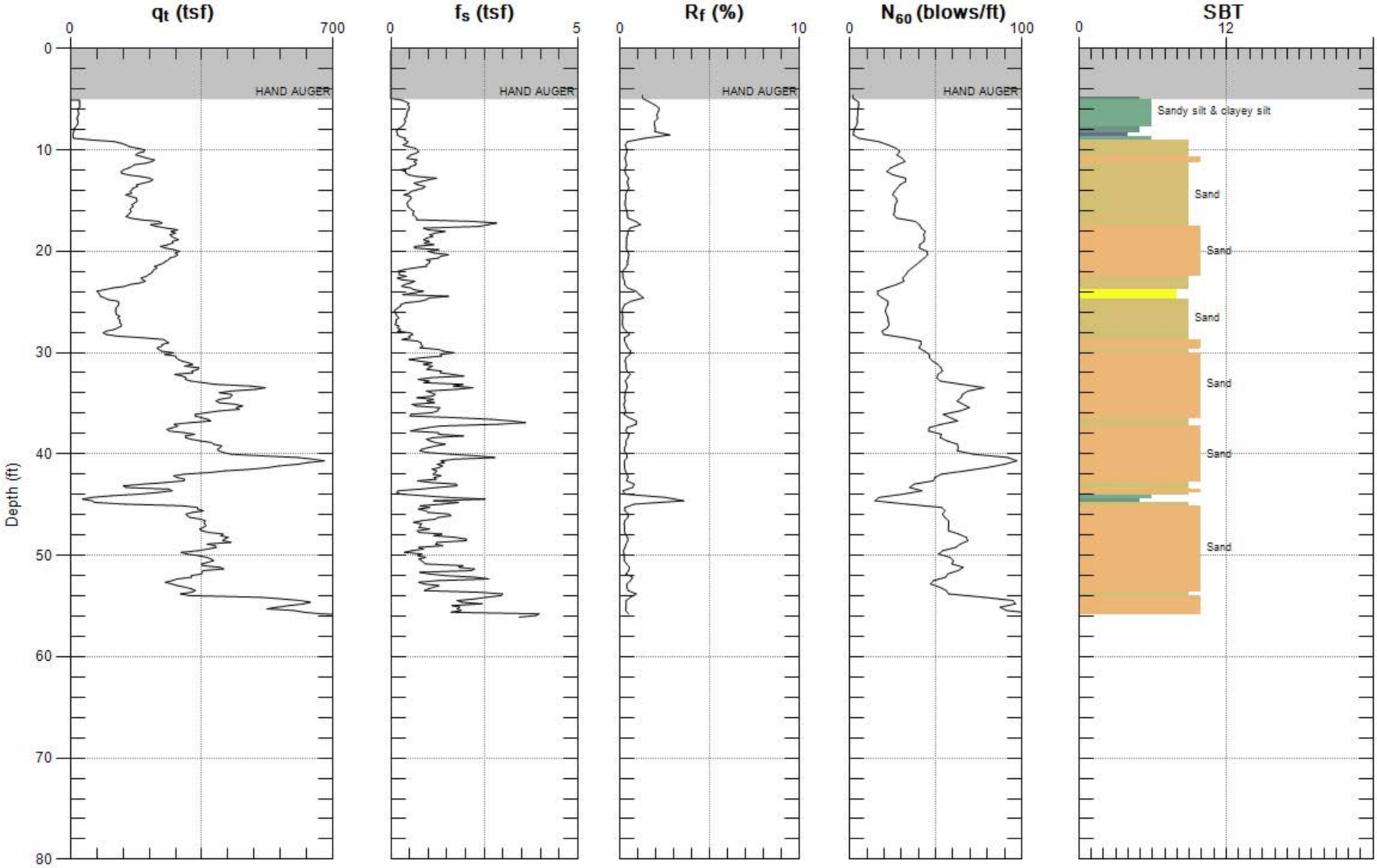
Max. Depth: 60.367 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER Engineer: G.HARRIS

Sounding: CPT-3 Date: 4/22/2013 11:18

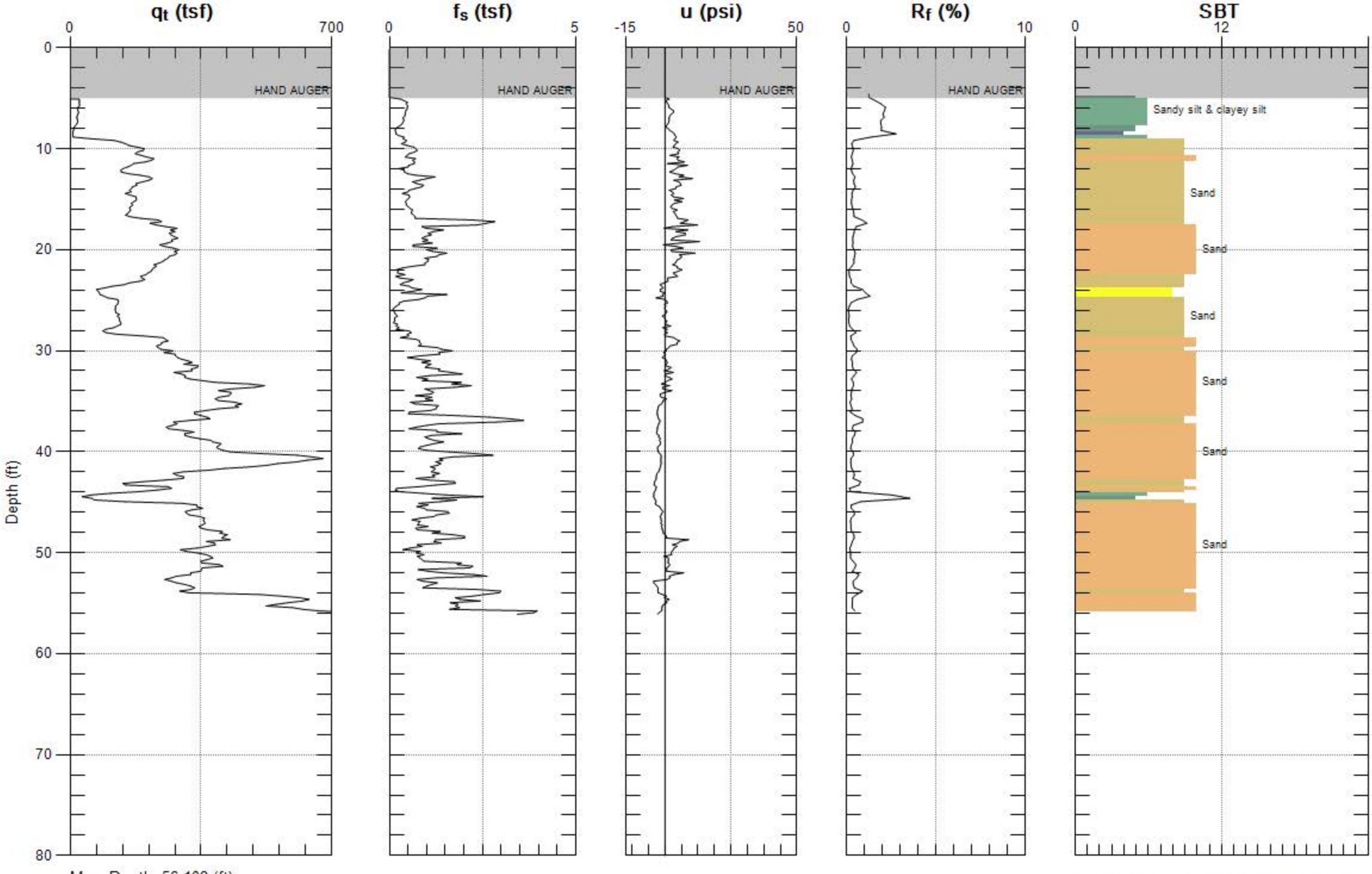


Max. Depth: 56.102 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER
Engineer: G.HARRIS
Sounding: CPT-3
Date: 4/22/2013 11:18



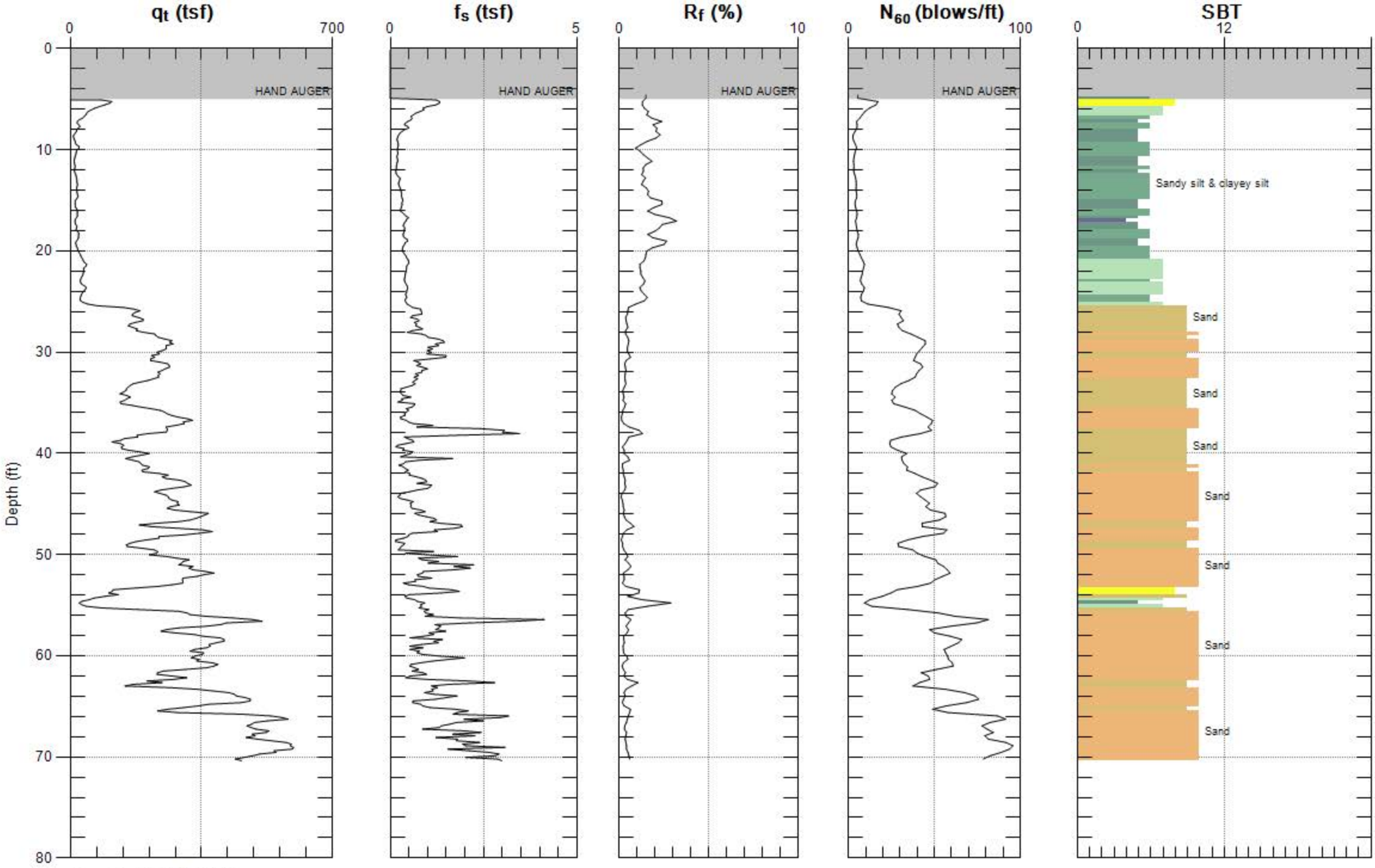
Max. Depth: 56.102 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER Engineer: G.HARRIS

Sounding: CPT-4 Date: 4/22/2013 01:25



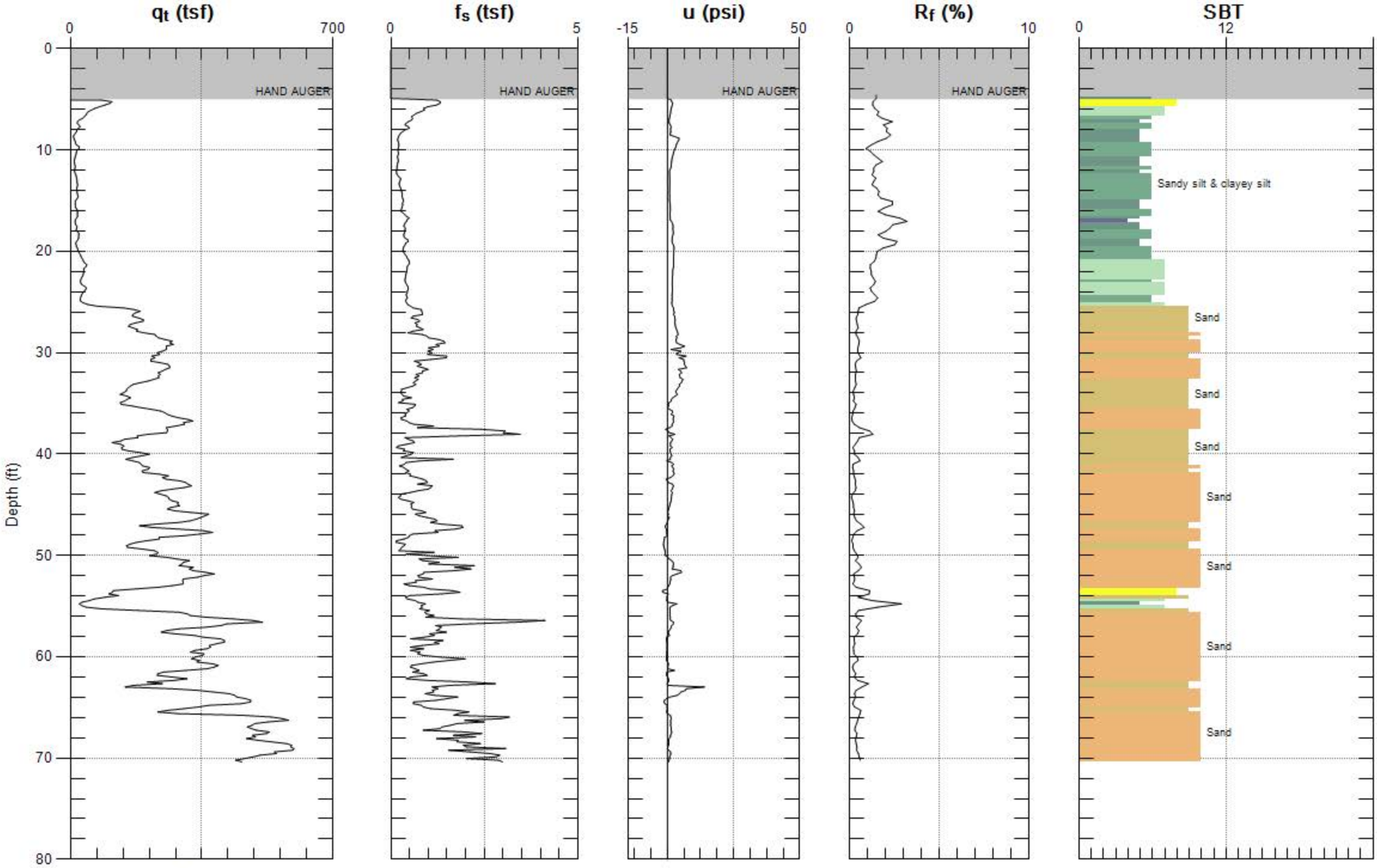
Max. Depth: 70.374 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER Engineer: G.HARRIS

Sounding: CPT-4 Date: 4/22/2013 01:25



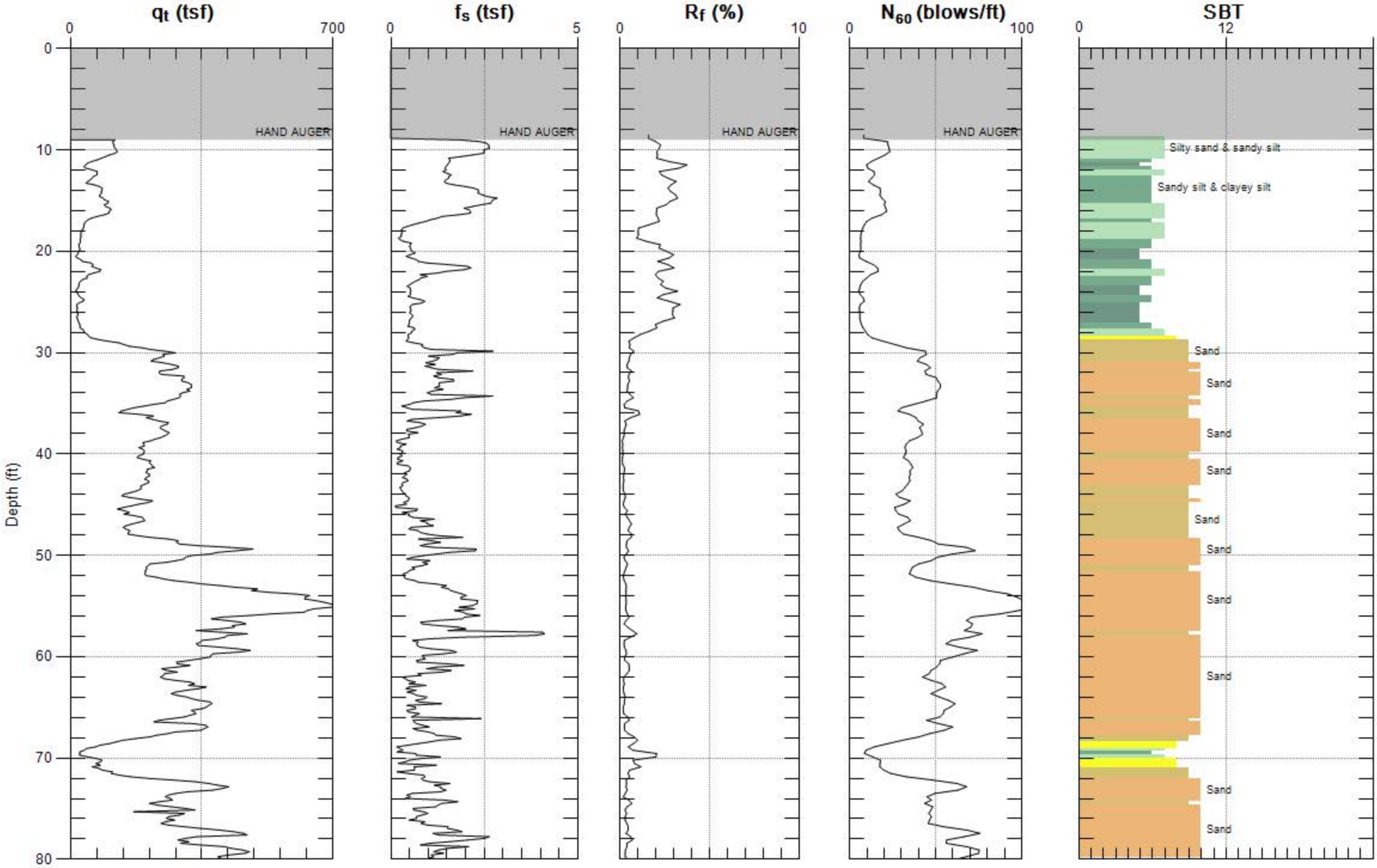
Max. Depth: 70.374 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER Engineer: G.HARRIS

Sounding: CPT-5 Date: 4/22/2013 03:20



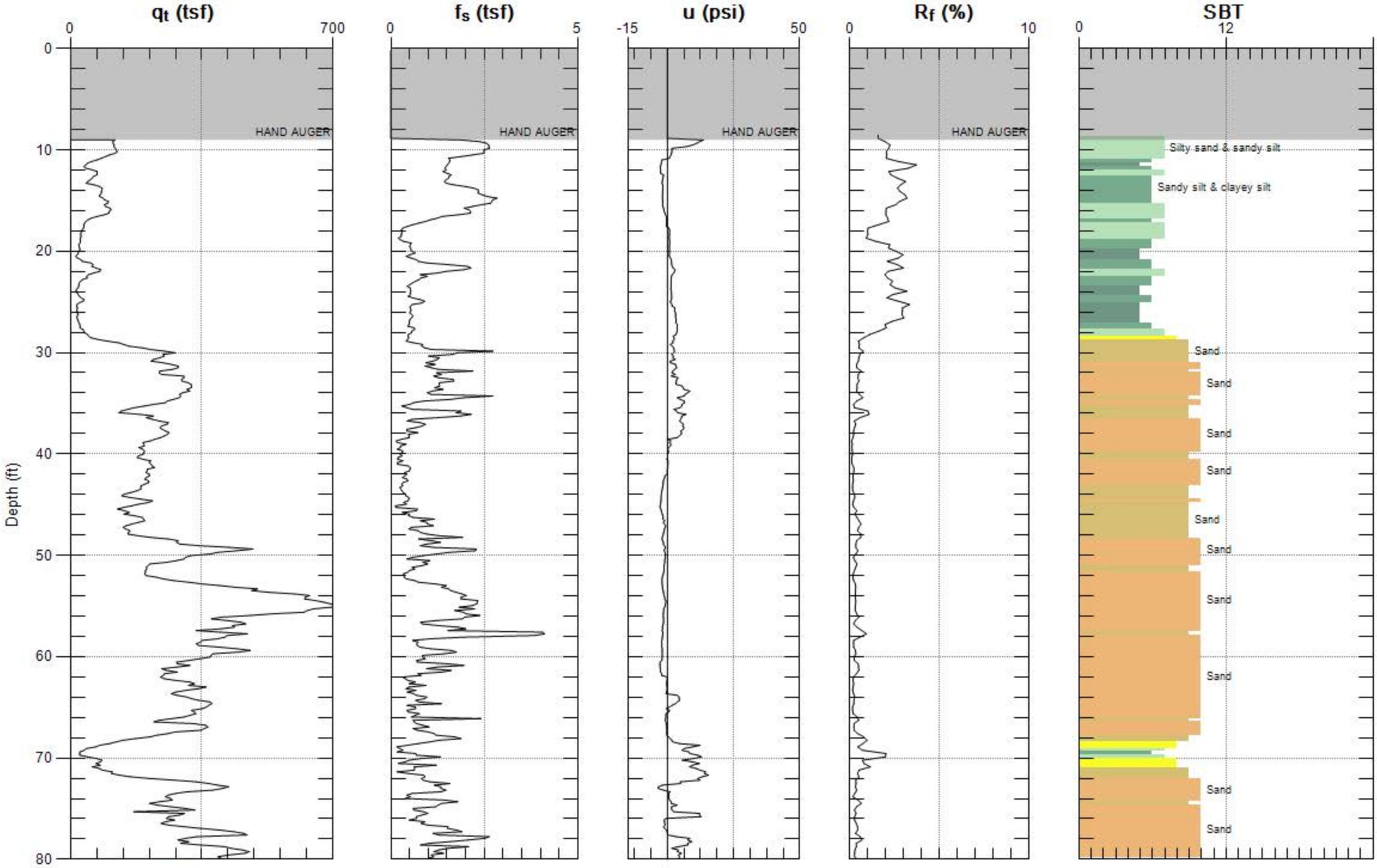
Max. Depth: 80.217 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER Engineer: G.HARRIS

Sounding: CPT-5 Date: 4/22/2013 03:20



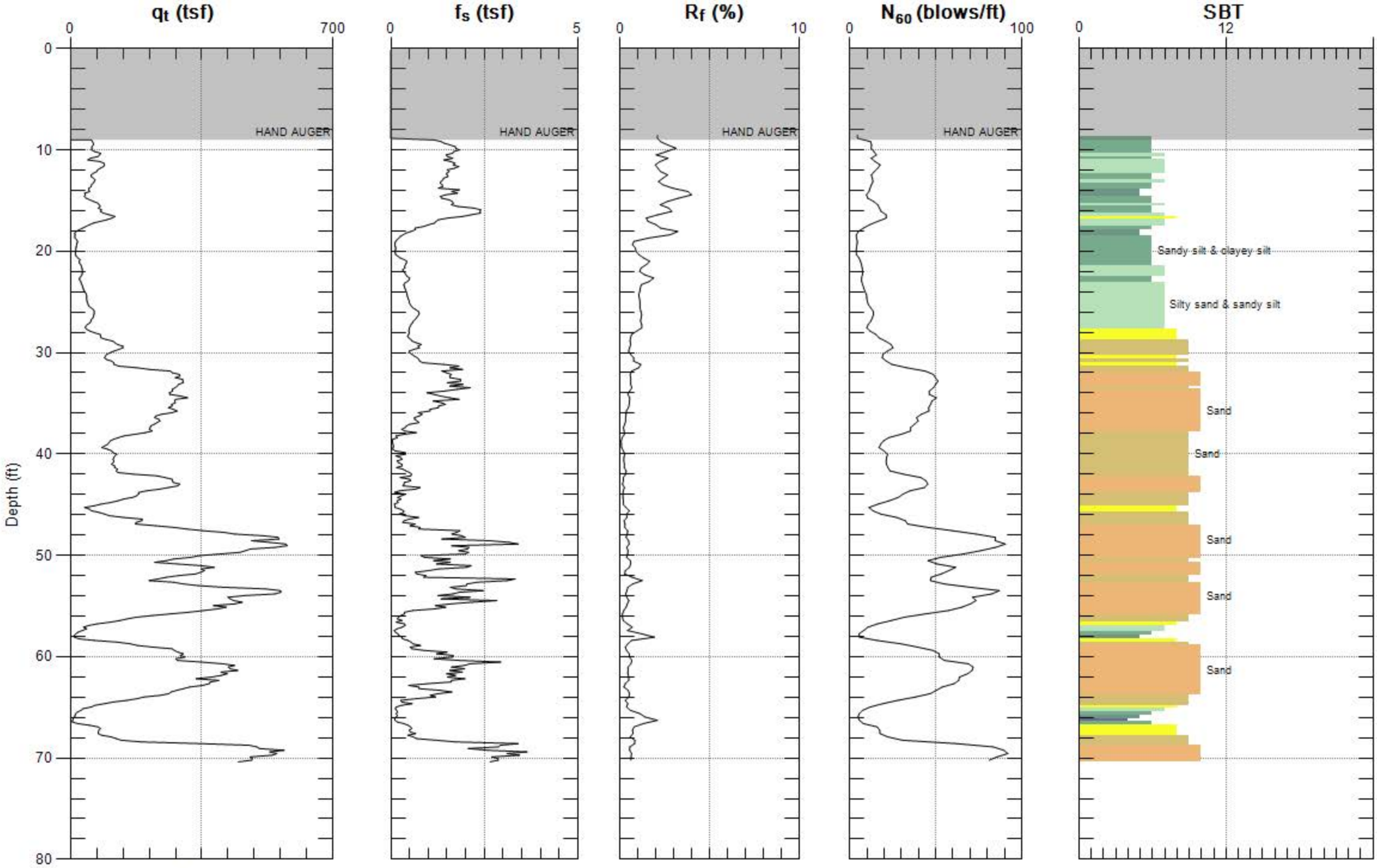
Max. Depth: 80.217 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER
Sounding: CPT-6

Engineer: G.HARRIS
Date: 4/23/2013 07:45

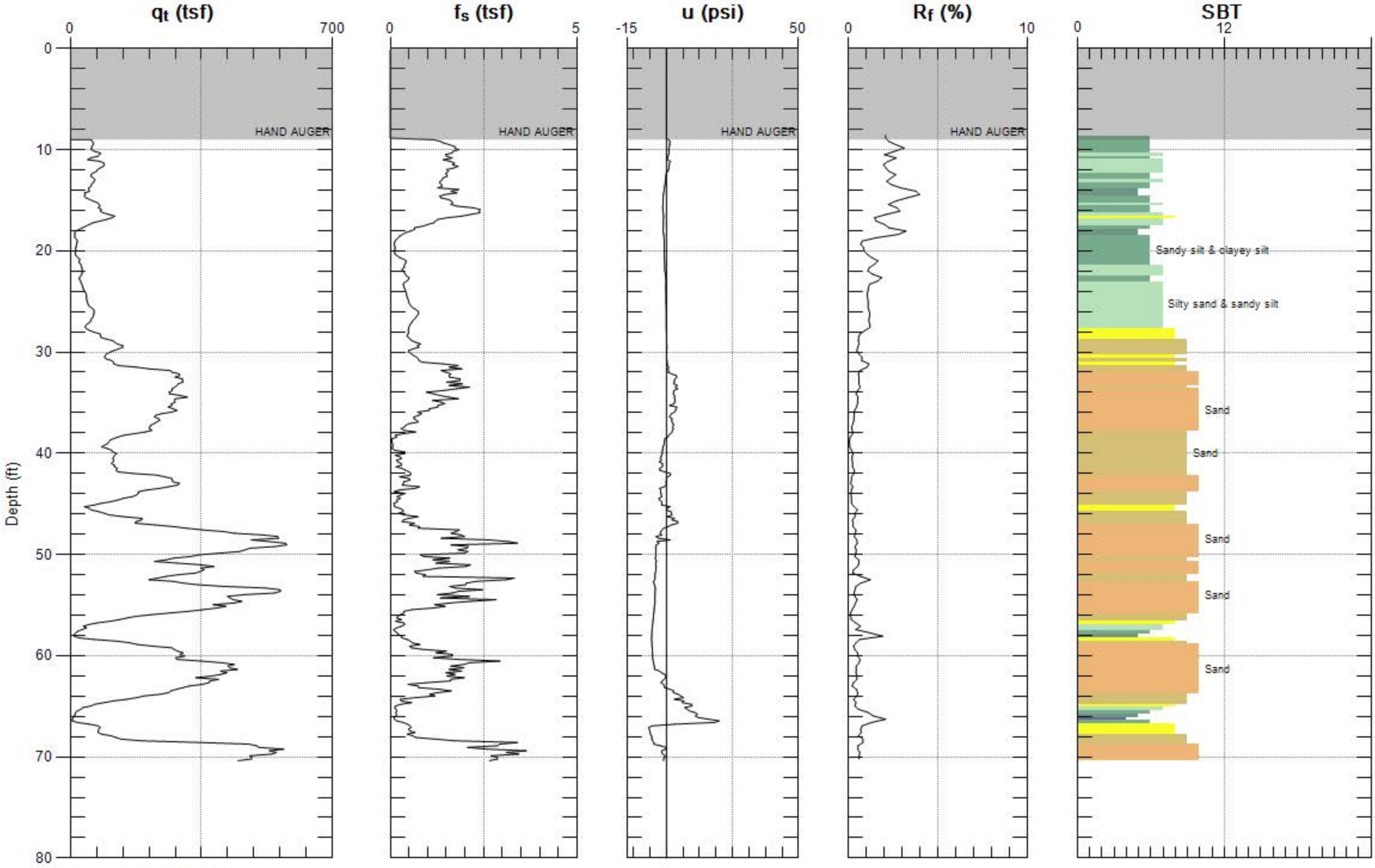


Max. Depth: 70.374 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER
Engineer: G.HARRIS
Sounding: CPT-6
Date: 4/23/2013 07:45



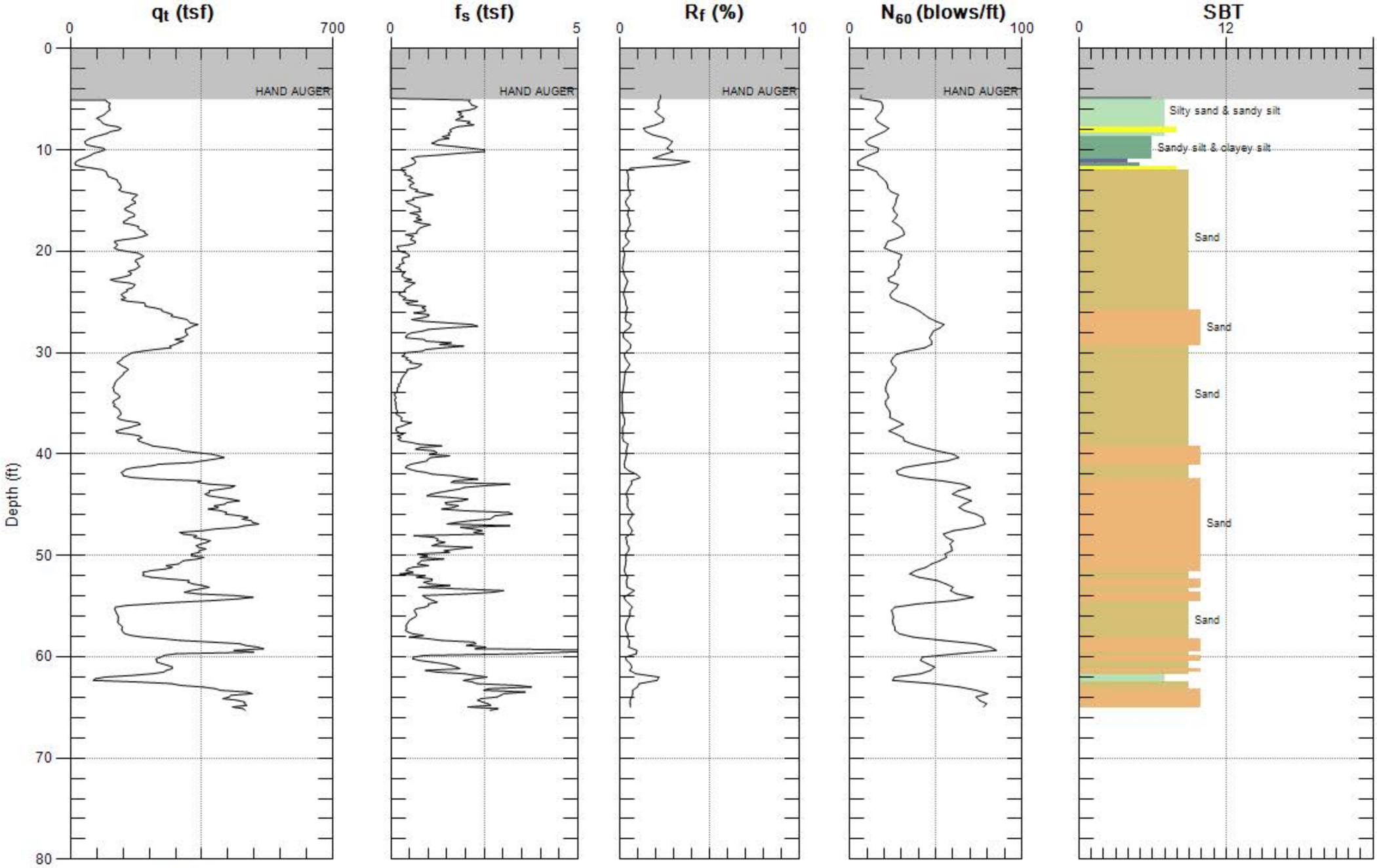
Max. Depth: 70.374 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER Engineer: G.HARRIS

Sounding: CPT-7 Date: 4/23/2013 09:39



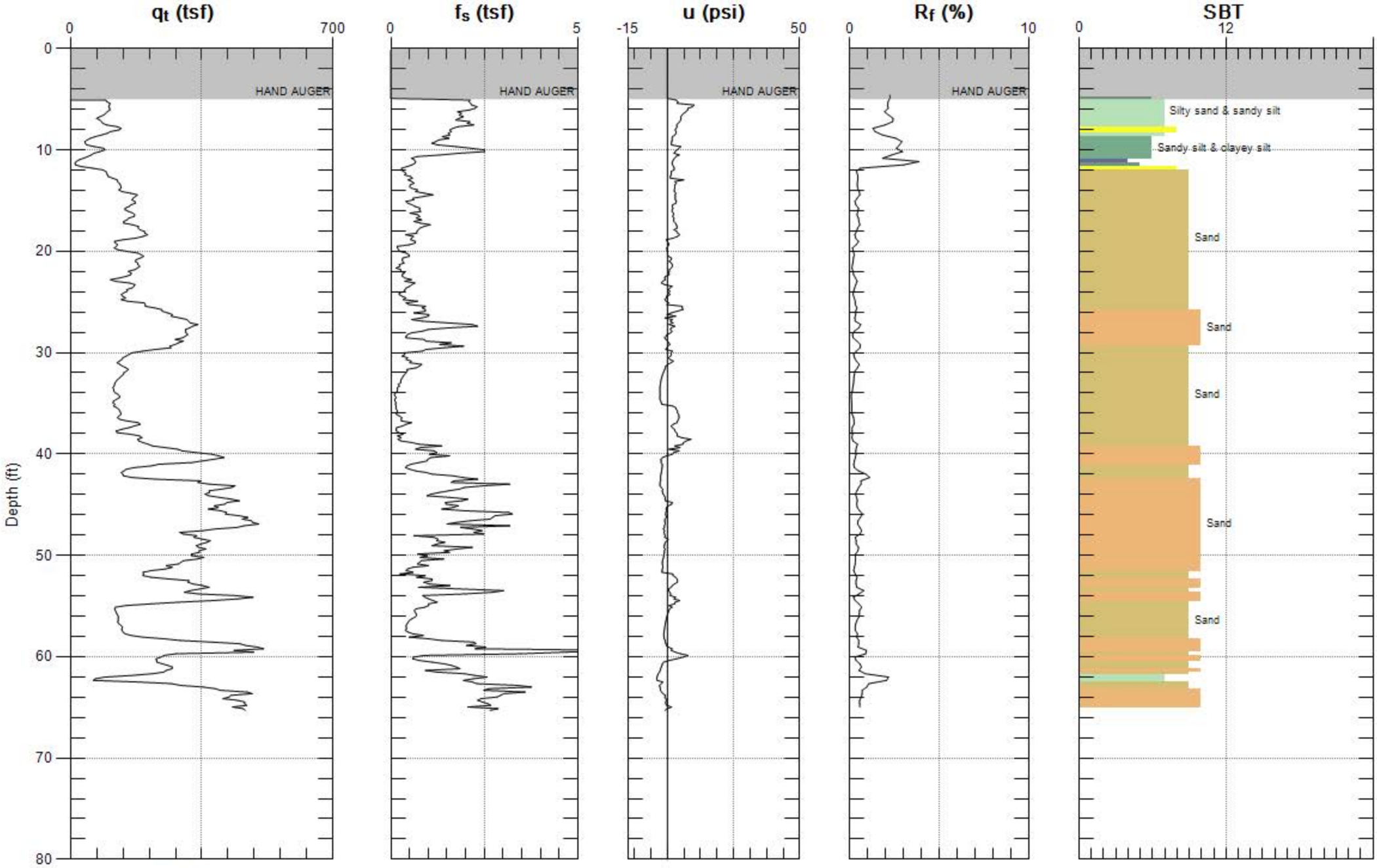
Max. Depth: 65.289 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

HDR ENGINEERING

Site: MIRABEL FISH LADDER
Sounding: CPT-7

Engineer: G.HARRIS
Date: 4/23/2013 09:39



Max. Depth: 65.289 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Boring Designation: B-1

DRILLING LOG	DIVISION -	INSTALLATION -	SHEET 1 OF 2 SHEETS

1. PROJECT: Mirabel Fish Passage and Seismic Improevement Project (Forestville, CA)	9. COORDINATE SYSTEM: - | HORIZONTAL: - | VERTICAL: NGVD29
	10. SIZE AND TYPE OF BIT: 6"
2. HOLE NUMBER: B-1 | LOCATION COORDINATES:	11. MANUFACTURER'S DESIGNATION OF DRILL (Type, Efficiency, Weight, Drop): CME 75 (Automatic, 85%) 140-lb, 30 in | EFFICIENCY MEASURED? ☐
3. DRILLING AGENCY: Clear Heart Drilling	12. TOTAL SAMPLES: 2 | DISTURBED: 2 | UNDISTURBED: 0
4. NAME OF DRILLER: Eric	13. TOTAL NUMBER CORE BOXES:
5. DIRECTION OF BORING: ☒ VERTICAL ☐ INCLINED | DEG FROM VERTICAL: --- | BEARING:	14. ELEVATION GROUND WATER: INITIAL 26.0 | STATIC
	15. DATE STARTED: 4/30/13 | DATE COMPLETED: 4/30/13
6. THICKNESS OF OVERBURDEN: N/A	16. ELEVATION TOP OF BORING: 66.0 ft
7. DEPTH DRILLED INTO ROCK: N/A	17. TOTAL CORE RECOVERY FOR BORING: N/A
8. TOTAL DEPTH OF BORING: 60.0ft	18. LOGGED BY: Garrett Harris | CHECKED BY:

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory: Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
65.8	0.2						Aggregate Base.											
							FILL Moist, brown silty SAND (SM).											Soil identified via cuttings unless samples were obtained Hand Auger to 5 feet
61.0	5.0						Moist, brown Sandy SILT (ML), very fine-grained sand.											Start Hollow Stem Auger; Observed from cuttings
							Grades clayey.											Cuttings observed to be more plastic
																		Material is coming up in rolled balls
																		RPM's down, soft/loose soils
46.0	20.0						Moist, brown Silty SAND (SM), with clay.											

DRILLING LOG (Cont Sheet)

INSTALLATION: -		SHEET 2 OF 2 SHEETS
PROJECT: Mirabel Fish Passage and Seismic Improevement Project (Forestville, CA)	COORDINATE SYSTEM: -	HORIZONTAL: - / VERTICAL: NGVD29
LOCATION COORDINATES	ELEVATION TOP OF BORING: 66.0 ft	

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory: Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
30.0							Moist, brown Silty SAND (SM), with clay. *(continued)*											
32.5																		
35.0							Gravels in cuttings (subangular, 1/4 to 1/2").											
37.5																		
40.0																		Saturated conditions observed at 43 feet bgs. After 0.5 HRS water measured at 40 feet bgs. Driller added water to augers to maintain head at 35 feet bgs
42.5																		
45.0	21.0 / 19.5	45.0 / 46.5		6 6 7	13	26		ALLUVIUM Medium dense, wet, brown, fine Sandy GRAVEL (GP) (rounded, 1/8 to 1/4").	100	1								
47.5								Medium dense, wet, brown Silty SAND (SM), fine-grained.										
50.0																		
52.5																		
55.0																		
57.5	8.5 / 8.0	57.5 / 58.0		12 40 30	70	138		Hard, wet, brown SAND with Silt (SP-SM), fine-grained.	100	2								
60.0	6.0	60.0						Hard, wet, light gray Clayey GRAVEL (GC), fine (rounded to subrounded, 1/4 to 1/2").										

Boring terminated at 60 feet bgs and backfilled with cuttings per SCWA requirements

Boring Designation: B-2

DRILLING LOG	DIVISION -	INSTALLATION -	SHEET 1 OF 2 SHEETS

1. PROJECT: Mirabel Fish Passage and Seismic Improevement Project (Forestville, CA)	9. COORDINATE SYSTEM: - / HORIZONTAL: - / VERTICAL: NGVD29
	10. SIZE AND TYPE OF BIT: 6"
2. HOLE NUMBER: B-2 / LOCATION COORDINATES:	11. MANUFACTURER'S DESIGNATION OF DRILL (Type, Efficiency, Weight, Drop): CME 75 (Automatic, 85%) 140-lb, 30 in / EFFICIENCY MEASURED? ☐
3. DRILLING AGENCY: Clear Heart Drilling	12. TOTAL SAMPLES: 2 / DISTURBED: 2 / UNDISTURBED: 0
4. NAME OF DRILLER: Eric	13. TOTAL NUMBER CORE BOXES
5. DIRECTION OF BORING: ☒ VERTICAL ☐ INCLINED / DEG FROM VERTICAL: --- / BEARING:	14. ELEVATION GROUND WATER: INITIAL 29.0 / STATIC
	15. DATE STARTED: 4/30/13 / DATE COMPLETED: 4/30/13
6. THICKNESS OF OVERBURDEN: N/A	16. ELEVATION TOP OF BORING: 65.0 ft
7. DEPTH DRILLED INTO ROCK: N/A	17. TOTAL CORE RECOVERY FOR BORING: N/A
8. TOTAL DEPTH OF BORING: 56.0ft	18. LOGGED BY: Garrett Harris / CHECKED BY:

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory: Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
64.5	0.5						Grass, topsoil.											Soil identified via cuttings unless samples were obtained Hand Auger to 5 feet
							FILL Moist, brown Sandy SILT (ML), fine-grained sand.											
							Grades clayey.											
							Grades sandy.											
							Clayey SILT with Sand (ML).											

0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0, 22.5, 25.0, 27.5, 30.0

DRILLING LOG (Cont Sheet)

INSTALLATION: -		SHEET 2 OF 2 SHEETS
PROJECT: Mirabel Fish Passage and Seismic Improevement Project (Forestville, CA)	COORDINATE SYSTEM: -	HORIZONTAL: - / VERTICAL: NGVD29
LOCATION COORDINATES	ELEVATION TOP OF BORING: 65.0 ft	

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	N_{60}	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory: Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
30.0							FILL Moist, brown Sandy SILT (ML), fine-grained sand. *(continued)*											
32.5							ALLUVIUM Gravels in cuttings (subangular, 1/4 to 1/2").											RPM's up slightly
35.0																		Saturation observed at 38.5 feet bgs. After 0.5 HRS water measured at 36 feet bgs. Driller added water to augers to maintain heade at 35 feet bgs.
37.5																		
40.0	25.0 / 40.0																	
	24.5 / 40.5		5				Loose, wet, brown SAND with Silt (SP-SM), very fine-grained.	100	1									
	24.0 / 41.0		9 12	21	41		Dense, wet, gray GRAVEL (GP), fine (subangular, 1/4").											
42.5							Dense, wet, gray SAND with Silt (SP-SM), fine- to coarse-grained.											
	20.5 / 44.5																	
45.0							Wet, brown Silty SAND (SM), fine-grained.											Cuttings
47.5																		
50.0																		
52.5																		
55.0	10.0 / 55.0																	
	9.5 / 55.5		18 50/6"	50/6"			Wet, brown, SAND with Silt (SP-SM), very fine-grained.	100	2									
	9.0 / 56.0						Brown SAND (SP), fine- to coarse-grained, with fine gravel.											
							Boring terminated at 56 feet bgs and backfilled with cuttings per SCWA requirements											

Appendix B – Previous Investigations

Logs and Laboratory Test Results

Brunsing Associates, Inc.

Log of Boring M-1

Equipment: MARL M11; 8-inch hollow stem auger

Date: 6/23/11

Logged By: KAC **Elevation:** 43 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Sample	
					1		GRAY BROWN SILTY SANDY GRAVEL (GM) dense, dry
					2		
					3		
	SPT			57	4		OLIVE-BROWN SILTY SAND (SM) medium dense to dense, damp
					5		
					6		BROWN SILTY FINE SAND (SM) with gravel medium dense, damp
Tx 1191 (720) 29% Passing #200 84% Passing #4	CA	11.7	109	8**	7		
					8		
					9		BROWN SAND (SP) with gravel medium dense, moist
	SPT			13	10		
					11		BROWN SAND (SP) with gravel loose, moist
					12		
	CA			5**	13		
					14		
					15		GRAY COARSE SAND (SP) very loose, saturated
	SPT			2	16		
					17		BROWN FINE SAND (SP) very loose, saturated
					18		BROWN POORLY GRADED SAND (SP) medium dense, saturated
Tx 2450 (1440) 3% Passing #200 86% Passing #4	CA	19.9	110	20**	19		
					20		
					21		GRAY COARSE SAND (SP) medium dense, saturated
	SPT			18	22		GRAY GRAVEL (GP) with sand medium dense, saturated
					23		
					24		BROWN POORLY GRADED GRAVEL (GP) with sand medium dense, saturated
3% Passing #200 51% Passing #4	CA	8.8	131	25**	25		
					26		
					27		BROWN POORLY GRADED SAND (SP) medium dense, saturated
	SPT			14	28		
					29		

Fill

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Mirabel Fish Screen and Fish Ladder Replacment Overall Site Plan, dated 11.4.2011, prepare by HDR Fisheries.

Scale: 1" = 4'

Brunsing Associates, Inc.
5468 Skylane Blvd., Suite 201
Santa Rosa, California 95403
Tel: (707) 838-3027

Job No.: 12225.01
Appr.: KAC
Date: 02/05/13

LOG OF BORING M-1
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE A1

Log of Boring M-1

Equipment: MARL M11; 8-inch hollow stem auger

Date: 6/23/11

Logged By: KAC **Elevation:** 43 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Sample	
5% Passing #200 91% Passing #4	CA	19.6	109	24**	30 31		BROWN SILTY SANDY GRAVEL (GM) medium dense, saturated
					32 33		GRAY-BROWN SAND (SP) dense, saturated
	SPT			40	34		
					35 36		BROWN SAND (SP) dense, saturated
	SPT			38	37		BROWN SILTY SANDY GRAVEL (GM) dense, saturated
					38 39		BROWN SAND (SP-SM) with silt medium dense, saturated
	SPT			41	40		
					41 42 43		BROWN SAND (SP) medium dense, saturated
					44 45		
	SPT			16	46		
					47 48		BROWN SILTY SAND (SM) dense, saturated
					49 50		
	SPT			61	51		

Notes:
1. Free water encountered at about 14 feet below ground surface during drilling
2. Boring caved while removing hollow stem auger

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Mirabel Fish Screen and Fish Ladder Replacment Overall Site Plan, dated 11.4.2011, prepare by HDR Fisheries.

Scale: 1" = 4'

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Appr.: KAC
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LOG OF BORING M-1
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE A1

Log of Boring M-2

Equipment: MARL M11; 8-inch hollow stem auger

Date: 6/23/11

Logged By: KAC **Elevation:** 44 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Sample	
					1		BROWN SILTY SANDY GRAVEL (GM) medium dense, damp
					2		
					3		
					4		
					5		Fill
	SPT			40	6		
					7		
					8		
					9		
					10		
Tx 1070 (1008) 38% Passing #200 98% Passing #4	CA	17.9	105	12**	11		BROWN SILTY SAND (SM) medium dense, damp to saturated
					12		
					13		
					14		
					15		BROWN SAND (SP) with gravel medium dense, saturated
	SPT			21	16		
					17		
					18		
					19		GRAY-BROWN POORLY GRADED SAND (SP) with gravel medium dense, saturated
					20		
4% Passing #200 78% Passing #4	CA	10.2	86	20**	21		
					22		
					23		
					24		BROWN SAND (SP) medium dense, saturated
					25		
	SPT			26	26		
					27		
					28		
					29		BROWN POORLY GRADED SAND (SP)

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
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LOG OF BORING M-2
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE A2

Log of Boring M-2

Equipment: MARL M11; 8-inch hollow stem auger

Date: 6/23/11

Logged By: KAC **Elevation:** 44 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Sample	
					30		loose, saturated
2% Passing #200 100% Passing #4	CA	23.9	76	10**	31		
					32		
					33		
					34		BROWN COARSE SAND (SP) with gravel dense, saturated
					35		
	SPT			38	36		
					37		
					38		
					39		BROWN FINE SAND (SP) dense, saturated
					40		
3% Passing #200 49% Passing #4	CA	4.6	107	61/11"**	41		BROWN GRAVEL (GP) with sand dense, saturated
					42		

Notes:
1. Free water encountered at about 14 feet below ground surface during drilling

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Mirabel Fish Screen and Fish Ladder Replacment Overall Site Plan, dated 11.4.2011, prepare by HDR Fisheries.

Scale: 1" = 4'

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LOG OF BORING M-2
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE
A2
SHEET 2 of 2

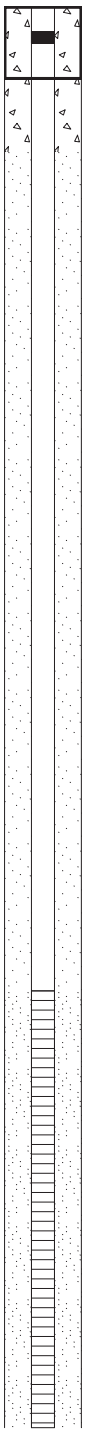
Log of Boring and Well Detail M-3

Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 11/6/12

Logged By: KAC **Elevation:** 62 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Sample	Description
					1		OLIVE-BROWN SILTY SAND (SM) soft to medium stiff fine sand
					2		
					3		
	CA			26/9"**	4		
					5		LIGHT OLIVE-BROWN SAND (SP-SM) with silt medium dense, damp fine sand
					6		
	CA			23**	7		Fill
					8		
		17.7	97		9		
	CA			7**	10		
					11		
					12		
					13		BROWN SAND (SP) loose, moist
					14		
	CA			5**	15		OLIVE-BROWN SILTY SAND (SM) loose, moist fine sand
					16		OLIVE-BROWN FINE SAND (SP-SM) with silt loose, moist
					17		
					18		
		14.9	88		19		
	CA			6**	20		
					21		
					22		
					23		BROWN FINE SAND (SP) loose, moist
					24		
	SPT			10	25		
					26		
					27		
					28		
					29		GRAY BROWN MEDIUM SAND (SP) medium dense, moist



* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Mirabel Fish Screen and Fish Ladder Replacment Overall Site Plan, dated 11.4.2011, prepare by HDR Fisheries.

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LOG OF BORING and WELL DETAIL M-3
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE
A3
SHEET 1 of 2

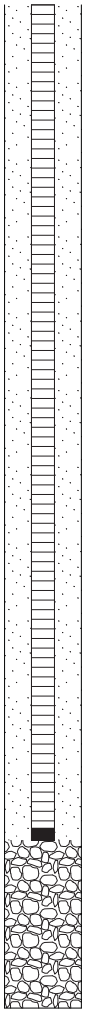
Log of Boring and Well Detail M-3

Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 11/6/12

Logged By: KAC **Elevation:** 62 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Sample	Description
	SPT			25	30		GRAY MEDIUM TO COARSE SAND (SP) medium dense, saturated coarse gravel in sampler shoe
					31		
					32		
					33		
DS 555 (500) DS 1268 (1000) DS 2304 (1500)	CA			21**	34		
					35		
					36		
					37		
					38		GRAY SILTY GRAVELLY SAND (SM) medium dense, saturated coarse gravel in sampler shoe
					39		
	SPT			31	40		
					41		
					42		
					43		GRAY-BROWN GRAVELLY SAND (SP) dense, saturated
					44		
	SPT			40	45		
					46		
					47		
					48		GRAY-BROWN FINE SOIL (SP)
					49		GRAY-BROWN COARSE SAND (SP)
	SPT			46	50		BROWN SAND (SP-SM) with silt



Notes
1. Free water encountered at about 33 feet below ground surface during drilling
2. Free water measured at 30 feet below ground surface 5 hours after drilling completed
3. Hollow stem became clogged around 35 feet below ground surface, advanced to 50 feet to install piezometer

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Mirabel Fish Screen and Fish Ladder Replacment Overall Site Plan, dated 11.4.2011, prepare by HDR Fisheries.

Scale: 1" = 4'

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LOG OF BORING and WELL DETAIL M-3
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE A3
SHEET 2 of 2

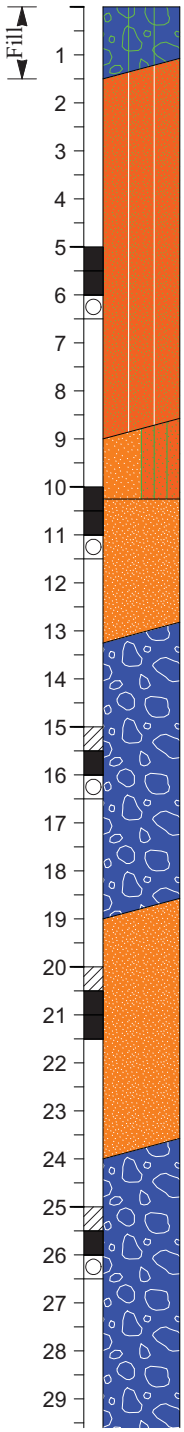
Log of Boring and Well Detail M-4

Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 11/6/12

Logged By: KAC **Elevation:** 49 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot
	CA			25**
35% Passing #200		14.5	108	
	CA			13**
2% Passing #200		4.0	111	
	CA			15**
2% Passing #200	CA	8.4	122	13**
	SPT			24



GRAY SILTY SANDY GRAVEL (GM)
dense, dry

BROWN SILTY GRAVELLY SAND (SM)
dense, damp

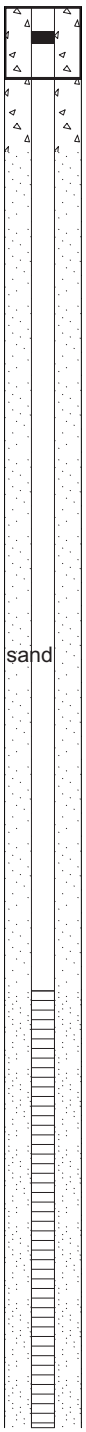
BLUE GRAY FINE SAND (SP-SM) with silt
medium dense, damp

BROWN SAND (SP)
medium dense, damp
fine grained sand

BROWN POORLY GRADED GRAVEL (GP) with sand
medium dense, wet

GRAY MEDIUM SAND (SP)
medium dense, saturated

GRAY BROWN SANDY GRAVEL (GP)
medium dense, saturated



* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Mirabel Fish Screen and Fish Ladder Replacment Overall Site Plan, dated 11.4.2011, prepare by HDR Fisheries.

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LOG OF BORING and WELL DETAIL M-4
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE
A4
SHEET 1 of 2

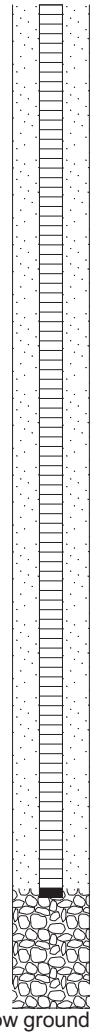
Log of Boring and Well Detail M-4

Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 11/6/12

Logged By: KAC **Elevation:** 49 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Sample	
					30		
	SPT			34	31		BROWN SANDY GRAVEL (GP) medium dense, saturated
					32		
					33		
					34		
					35		
					36		
					37		
					38		
					39		
					40		
					41		
					42		
					43		
					44		
					45		
					46		
					47		
					48		
					49		
					50		



Notes
1. Free water encountered at about 19 feet below ground surface during drilling
2. Free water measured at 17.7 feet below ground surface on November 13, 2012

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Mirabel Fish Screen and Fish Ladder Replacment Overall Site Plan, dated 11.4.2011, prepare by HDR Fisheries.

Scale: 1" = 4'

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LOG OF BORING and WELL DETAIL M-4
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE
A4
SHEET 2 of 2

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% fines)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (Greater than 12% fines)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS 50% OR MORE OF COARSE FRACTION PASSING THROUGH NO. 4 SIEVE	CLEAN SANDS (Less than 5% fines)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES (Greater than 12% fines)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILT, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMOUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

KEY TO TEST DATA

Consol - Consolidation
LL - Liquid Limit
PI - Plasticity Index
EI - Expansion Index
SA - Sieve Analysis

■ Sample Retained
▨ Sample Recovered, Not Retained
⊠ Bulk Sample
◯ Sample Not Recovered

CA - California Modified Split Barrel Sampler 3.0-inch O.D.
CM - California Modified Split Barrel Sampler 2.5-inch O.D.
SPT - California Split Barrel Sampler 2.0-inch O.D.
SH - Shelby Tube
RC - Rock Coring

Shear Strength, psf	Confining Pressure, psf	
Tx	320 (2600)	- Unconsolidated Undrained Triaxial
TxCU	320 (2600)	- Consolidated Undrained Triaxial
DS	2750 (2600)	- Consolidated Drained Direct Shear
FVS	470	- Field Vane Shear
UC	2000	- Unconfined Compression
PP	2000	- Field Pocket Penetrometer
Sat		- Sample saturated prior to test

▽ Groundwater Level Reading
▼ Second Groundwater Level Reading

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SOIL CLASSIFICATION CHART & KEY TO TEST DATA

SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER REPLACEMENT PROJECT
Sonoma County, California

PLATE A5

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	Standard Penetration Test Blow Count (blows per foot)
Very loose	4 or less
Loose	5 to 10
Medium dense	11 to 30
Dense	31 to 50
Very dense	More than 50

CONSISTENCY OF FINE-GRAINED SOILS

Consistency	Identification Procedure	Approximate Shear Strength (psf)
Very soft	Easily penetrated several inches with fist	Less than 250
Soft	Easily penetrated several inches with thumb	250 to 500
Medium stiff	Penetrated several inches by thumb with moderate effort	500 to 1000
Stiff	Readily indented by thumb, but penetrated only with great effort	1000 to 2000
Very stiff	Readily indented by thumb nail	2000 to 4000
Hard	indented with difficulty by thumb nail	More than 4000

NATURAL MOISTURE CONTENT

Dry	No noticeable moisture content. Requires considerable moisture to obtain optimum moisture content* for compaction.
Damp	Contains some moisture, but is on the dry side of optimum.
Moist	Near optimum moisture content for compaction.
Wet	Requires drying to obtain optimum moisture content for compaction.
Saturated	Near or below the water table, from capillarity, or from perched or ponded water. All void spaces filled with water.

* Optimum moisture content as determined in accordance with ASTM Test Method D1557, latest edition.

Where laboratory test data are not available, the above field classifications provide a general indication of material properties; the classifications may require modification based upon laboratory tests.

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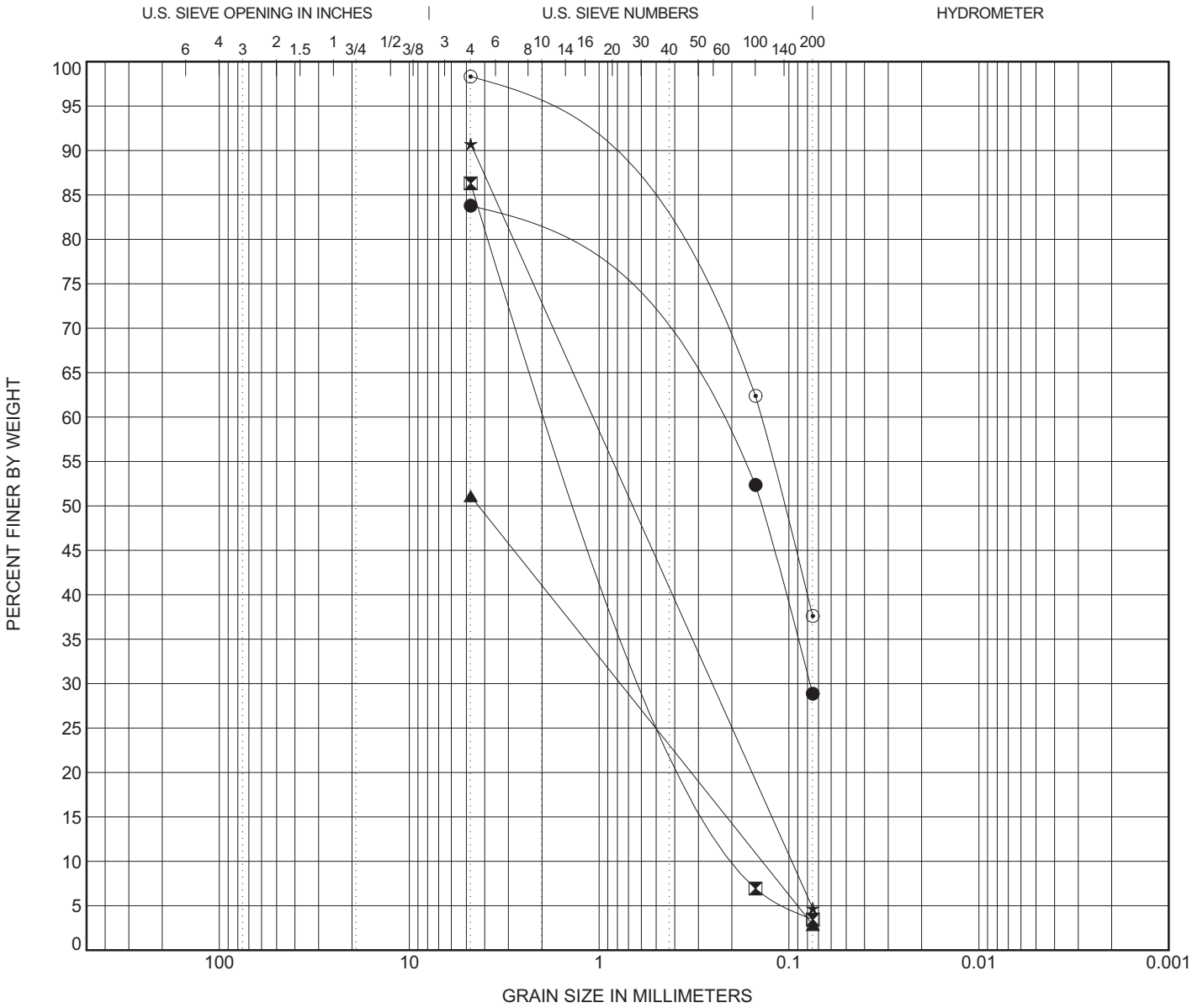
SOIL DESCRIPTIVE PROPERTIES
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE
A6



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	M-1	6.5 ft	**BROWN SILTY FINE SAND (SM) with gravel**					
⧗	M-1	19.0 ft	**BROWN POORLY GRADED SAND (SP)**				**0.65**	**8.81**
▲	M-1	25.0 ft	**BROWN POORLY GRADED GRAVEL (GP) with sand**					
★	M-1	30.5 ft	**BROWN POORLY GRADED SAND (SP)**				**0.62**	**11.14**
⊙	M-2	10.5 ft	**BROWN SILTY SAND (SM)**					

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	M-1	6.5 ft	**4.75**	**0.347**	**0.078**			**54.9**	**28.9**	
⧗	M-1	19.0 ft	**4.75**	**1.511**	**0.409**	**0.171**		**82.9**	**3.5**	
▲	M-1	25.0 ft	**4.75**		**0.774**	**0.139**		**48.3**	**2.8**	
★	M-1	30.5 ft	**4.75**	**1.079**	**0.254**	**0.097**		**86.0**	**4.7**	
⊙	M-2	10.5 ft	**4.75**	**0.14**				**60.7**	**37.6**	

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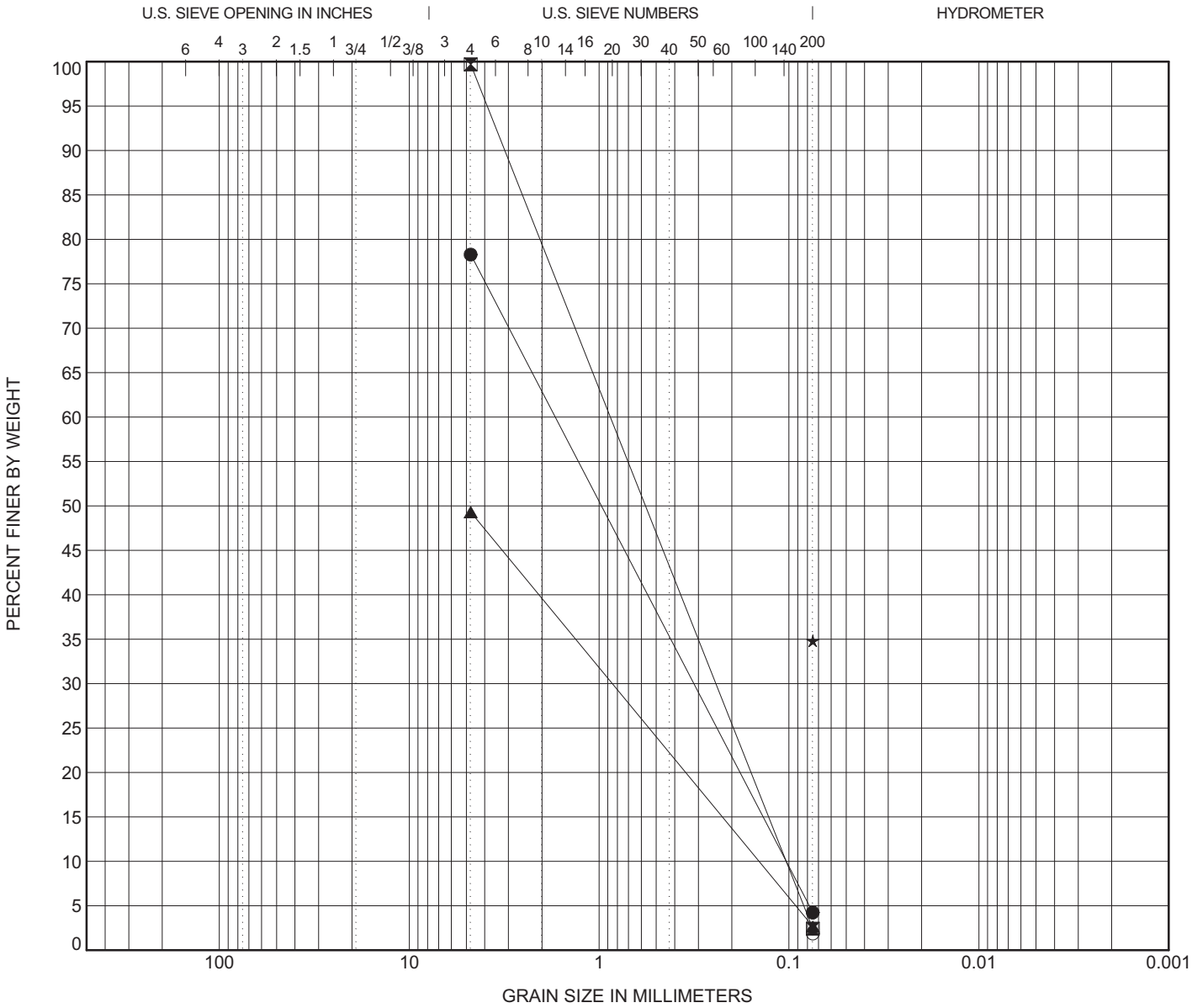
GRAIN SIZE DISTRIBUTION
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE
B1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	M-2	20.5 ft	GRAY-BROWN POORLY GRADED SAND (SP) with gravel				0.57	16.46
⧗	M-2	30.5 ft	BROWN POORLY GRADED SAND (SP)				0.65	8.44
▲	M-2	41.0 ft	BROWN GRAVEL (GP) with sand					
★	M-4	10.0 ft	BLUE GRAY FINE SAND (SP-SM) with silt					
⊙	M-4	15.5 ft	BROWN POORLY GRADED GRAVEL (GP) with sand					

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	M-2	20.5 ft	4.75	1.705	0.317	0.104		74.0	4.2	
⧗	M-2	30.5 ft	4.75	0.874	0.243	0.104		97.3	2.4	
▲	M-2	41.0 ft	4.75		0.854	0.144		46.6	2.7	
★	M-4	10.0 ft	0.075						34.8	
⊙	M-4	15.5 ft	0.075						1.9	

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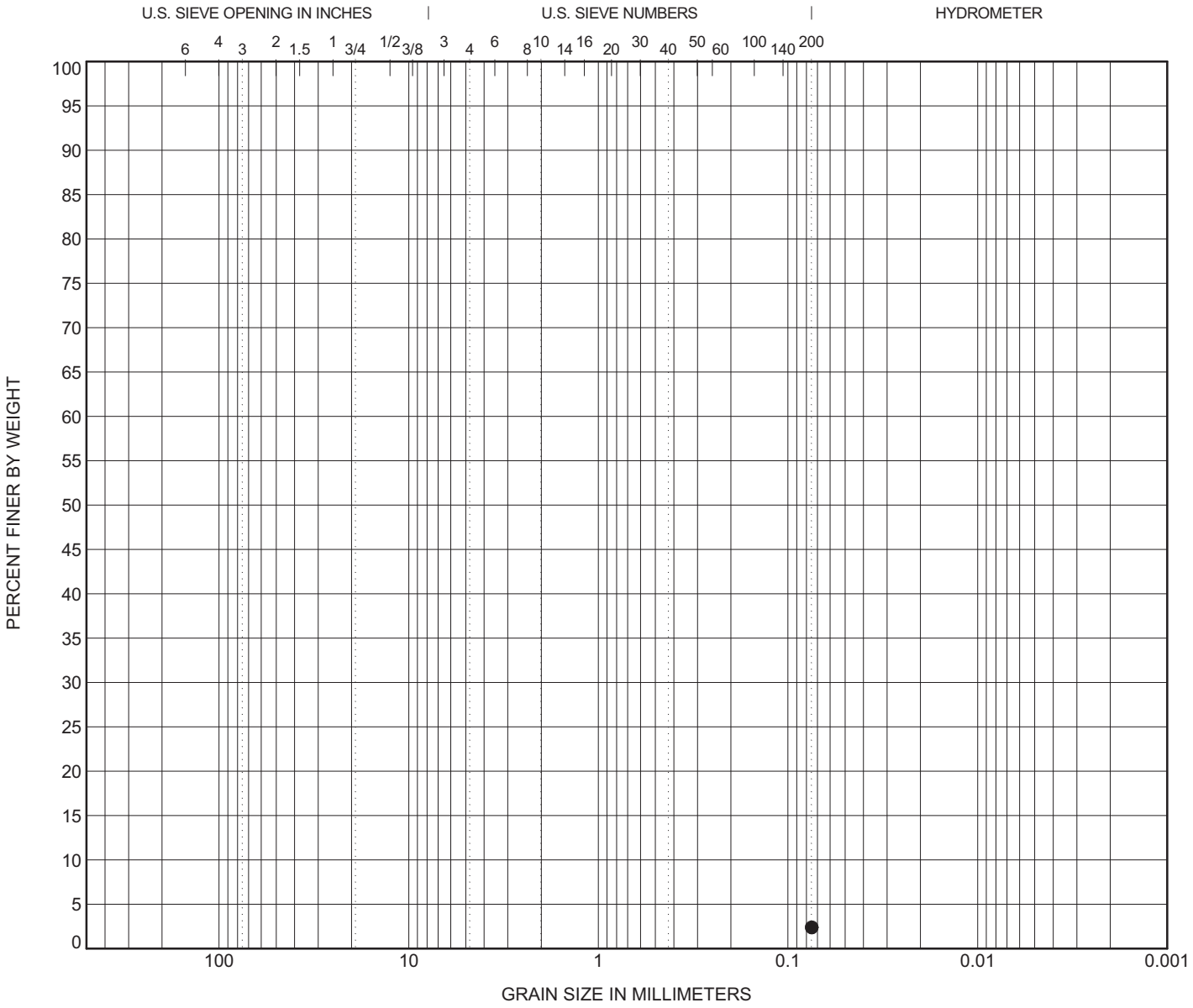
GRAIN SIZE DISTRIBUTION
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE
B2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**M-4**	**21.0 ft**	**GRAY MEDIUM SAND (SP)**					

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**M-4**	**21.0 ft**	**0.075**						**2.4**	

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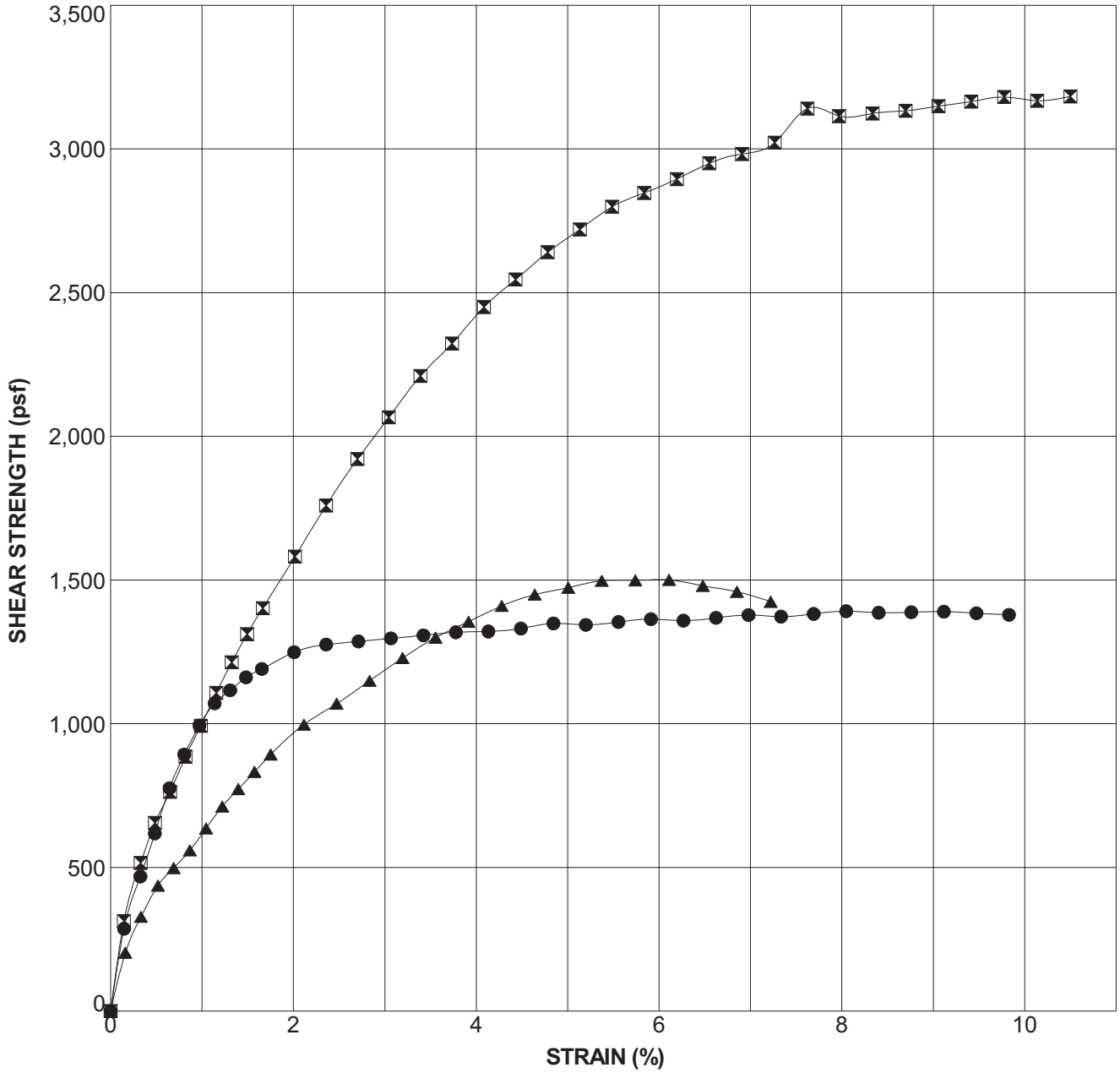
GRAIN SIZE DISTRIBUTION
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE
B3



Sample Source	Classification	Confining Pressure (psf)	Yield Strength (psf)	Strain (%)	Dry Density (pcf)	Moisture Content (%)
● M-1 at 6.5 ft	BROWN SILTY FINE SAND (SM) with gravel	720	1191	1.7	109	11.7
⊠ M-1 at 19 ft	BROWN POORLY GRADED SAND (SP)	1440	2450	4.1	110	19.9
▲ M-2 at 10.5 ft	BROWN SILTY SAND (SM)	1008	1070	2.5	105	17.9

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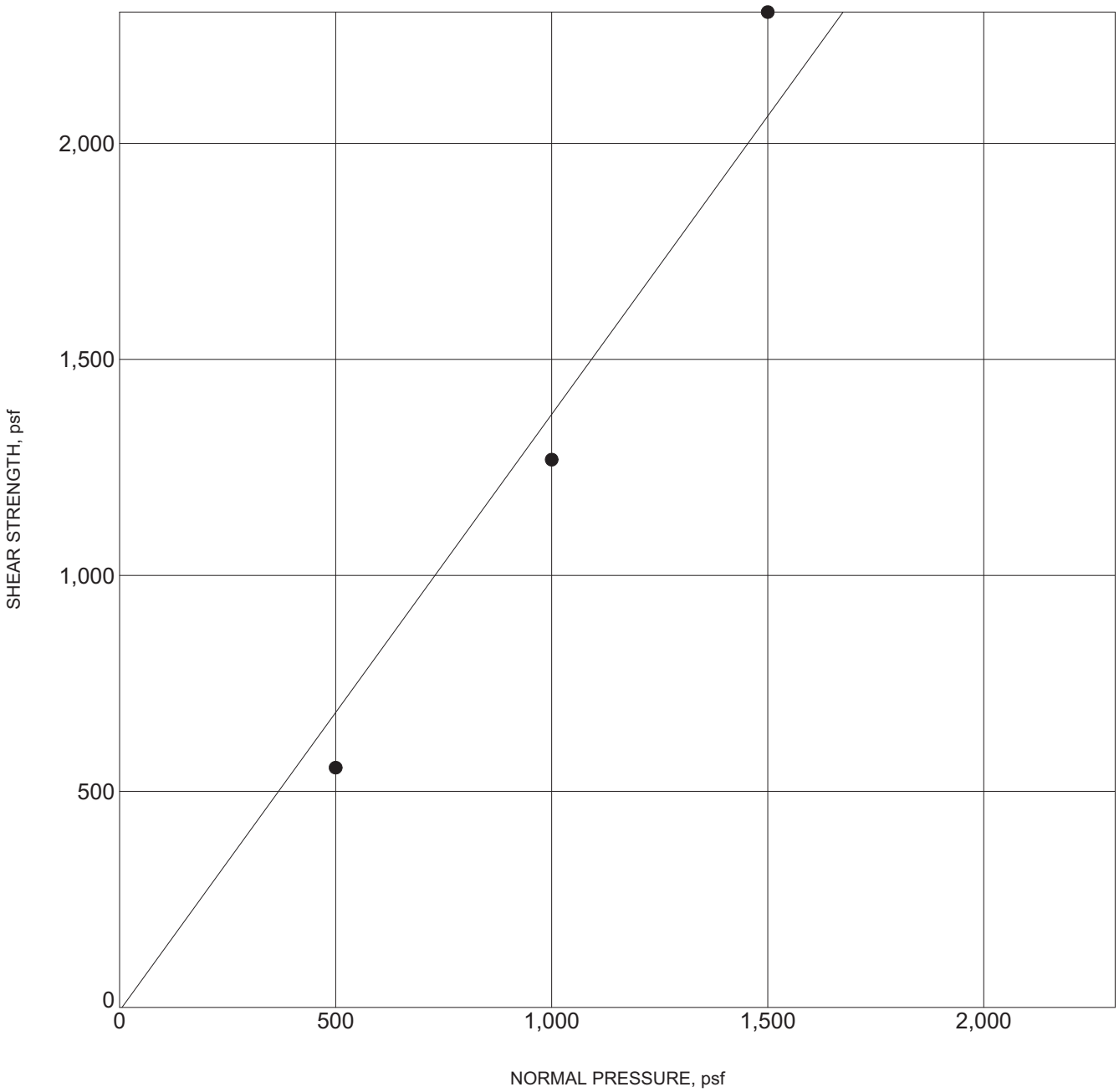
Tel: (707) 838-3027

Job No.: 12225.01
Appr.: KAC
Date: 02/05/13

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER REPLACEMENT PROJECT
Sonoma County, California

PLATE
B4



Specimen Identification				Classification	γ_d	MC%	c	ϕ (degree)
●	M-3	at	34.0 ft	GRAY MEDIUM TO COARSE SAND (SP)	133	10.9	0	52

Brunsing Associates, Inc.
5468 Skylane Blvd., Suite 201
Santa Rosa, California 95403
Tel: (707) 838-3027

Job No.: 12225.01
Appr.: KKC
Date: 02/05/13

DIRECT SHEAR TEST RESULTS
SONOMA COUNTY WATER AGENCY
MIRABEL FISH SCREEN AND LADDER
REPLACEMENT PROJECT
Sonoma County, California

PLATE
B5

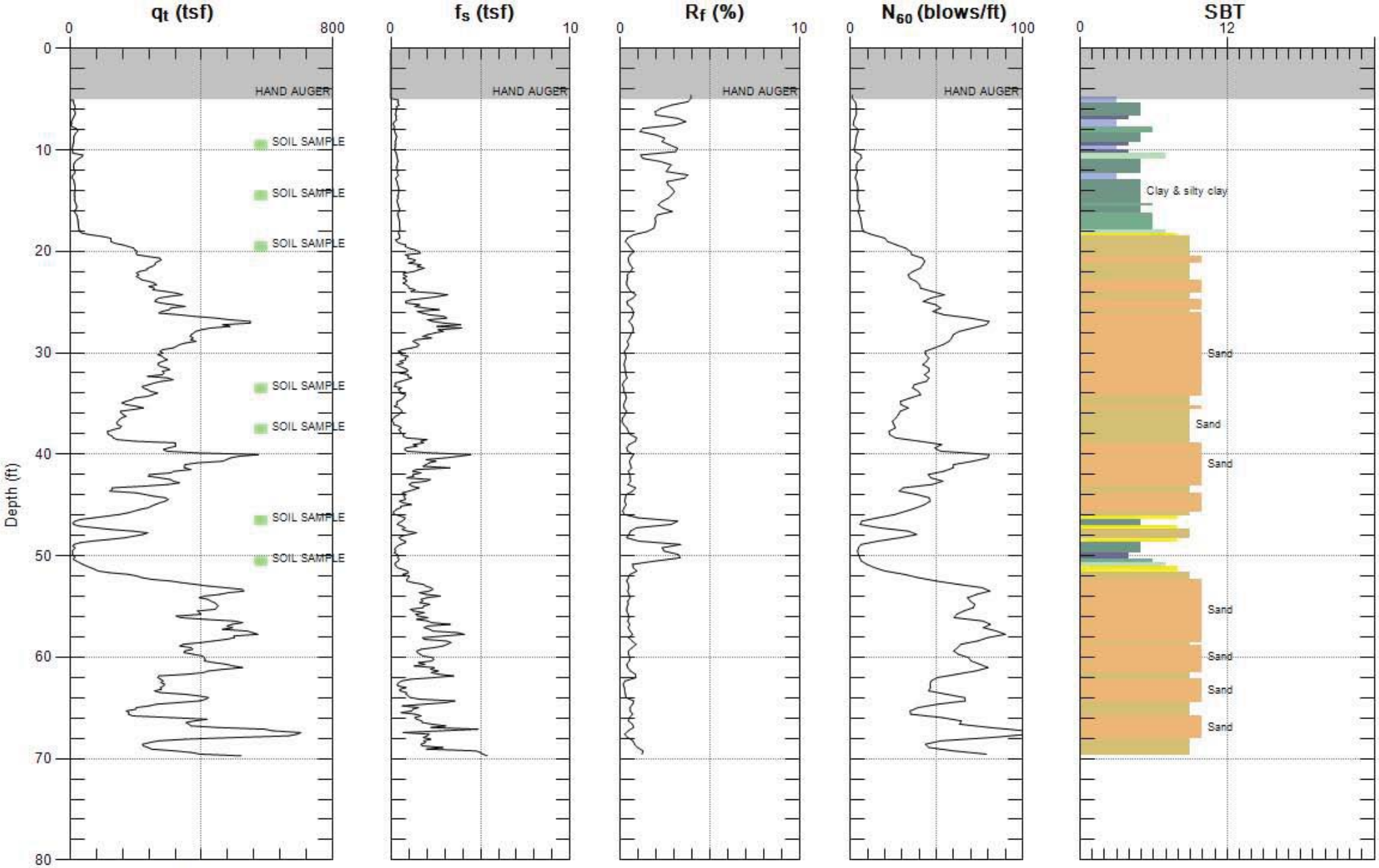
MMI/Geosyntec

GREGG

GEOSYNTEC

Site: SONOMA RUSSIAN RIV. Engineer: JESSICA B.

Sounding: CPT-GSC07 Date: 6/21/2011 01:58



Max. Depth: 69.718 (ft)
Avg. Interval: 0.328 (ft)

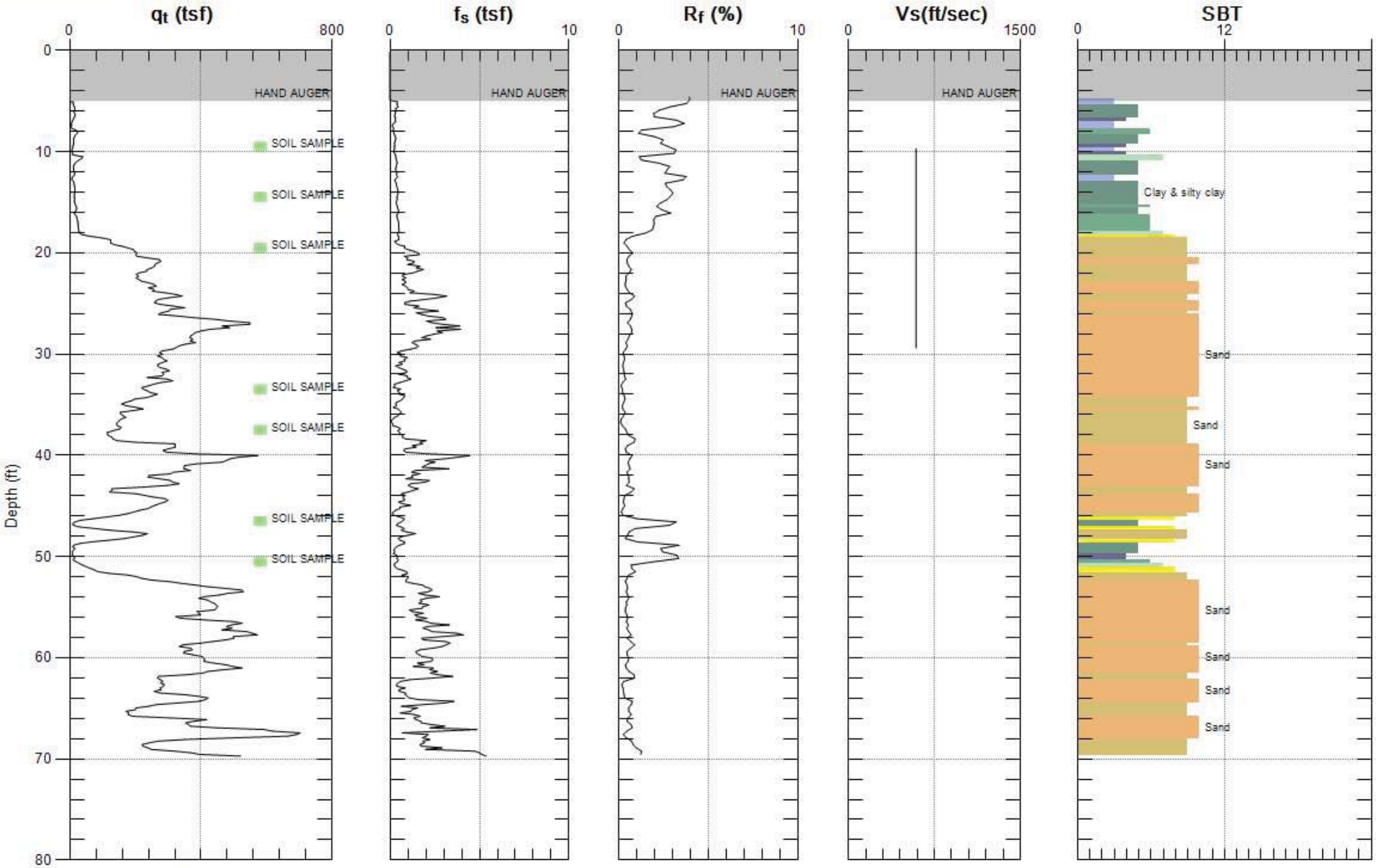
SBT: Soil Behavior Type (Robertson 1990)

GREGG

GEOSYNTEC

Site: SONOMA RUSSIAN RIV. Engineer: JESSICA B.

Sounding: CPT-GSC07 Date: 6/21/2011 01:58



Max. Depth: 69.718 (ft)
Avg. Interval: 0.328 (ft)

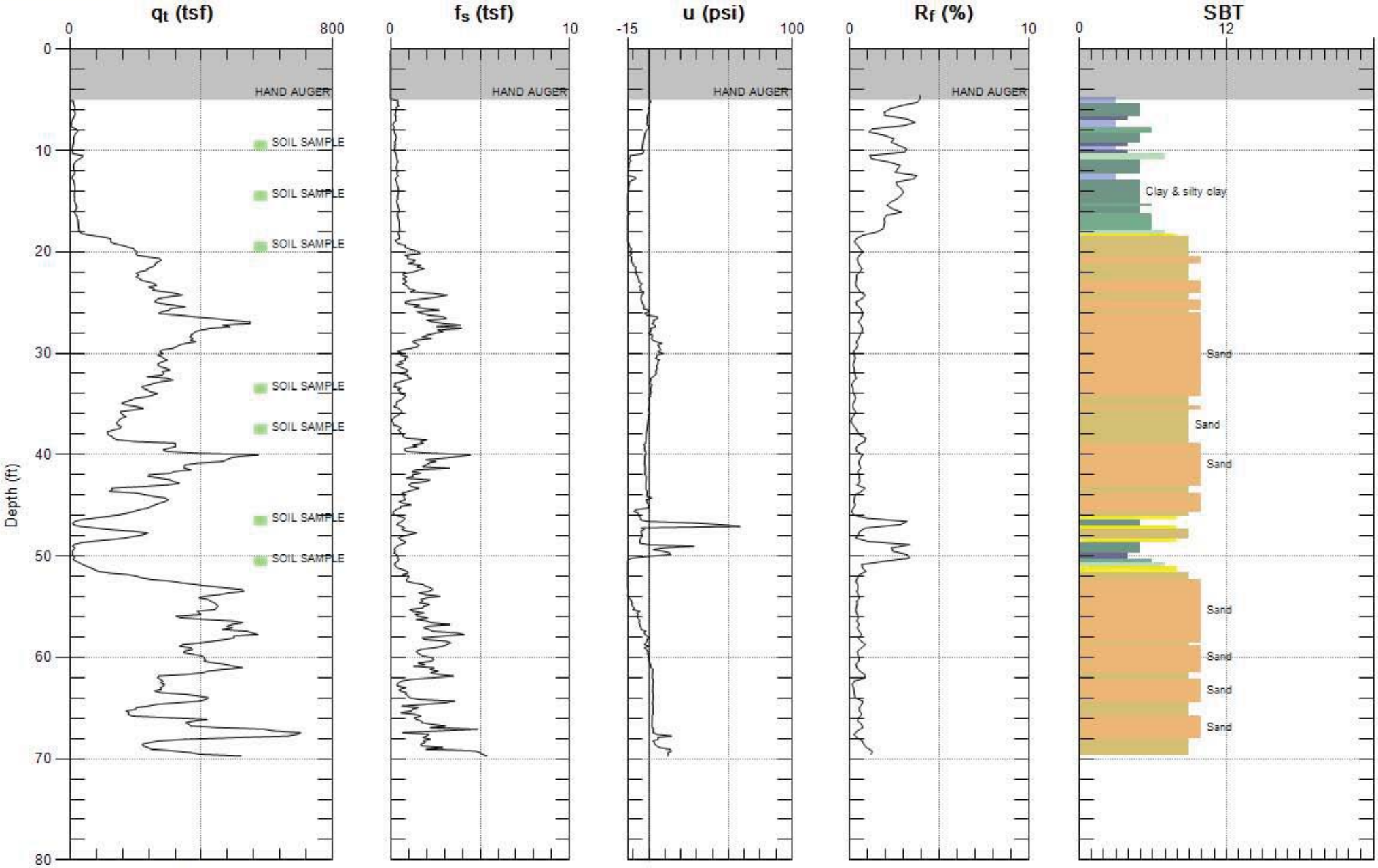
SBT: Soil Behavior Type (Robertson 1990)

GREGG

GEOSYNTEC

Site: SONOMA RUSSIAN RIV. Engineer: JESSICA B.

Sounding: CPT-GSC07 Date: 6/21/2011 01:58



Max. Depth: 69.718 (ft)
Avg. Interval: 0.328 (ft)

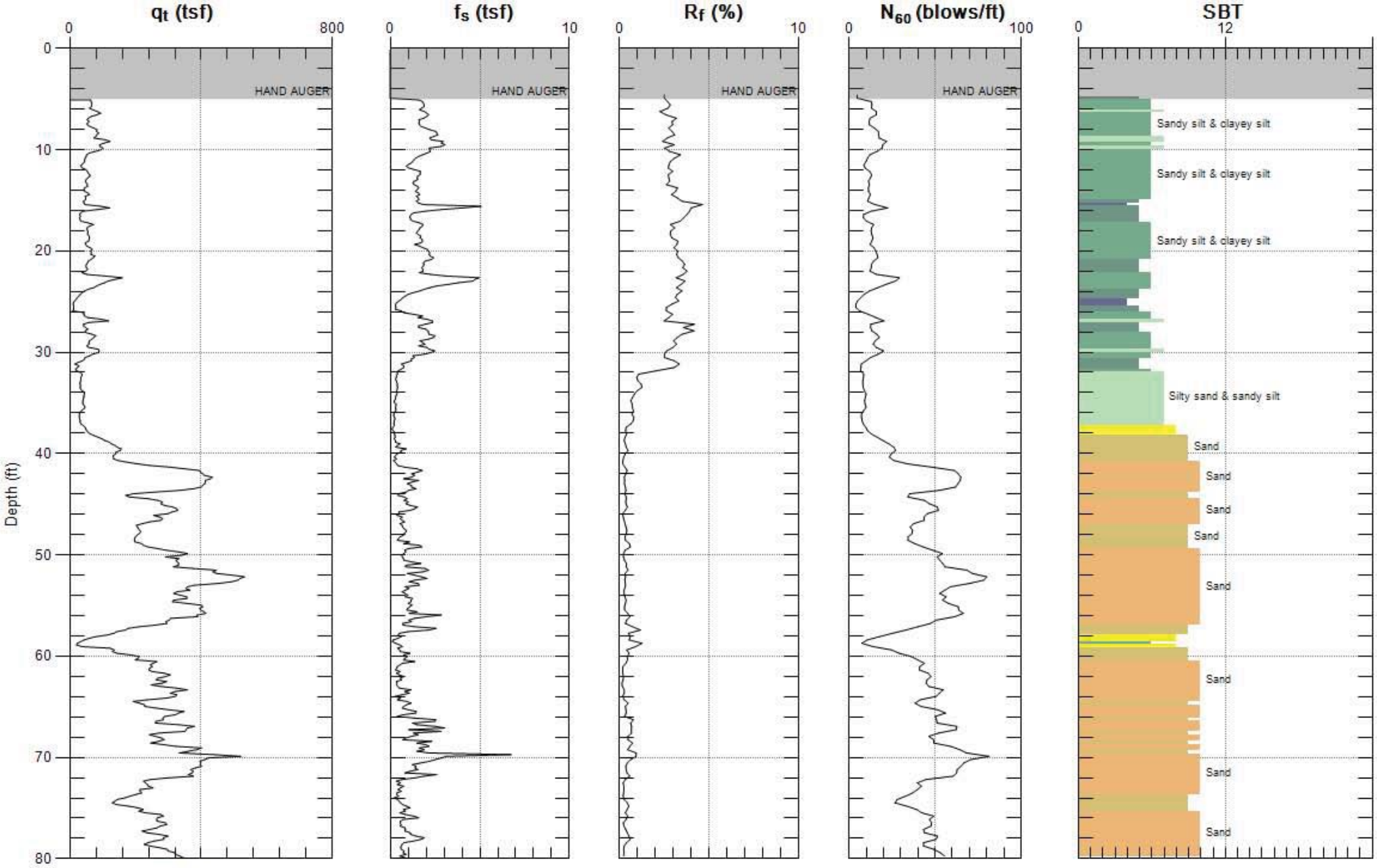
SBT: Soil Behavior Type (Robertson 1990)

GREGG

GEOSYNTEC

Site: SONOMA RUSSIAN RIV. Engineer: JESSICA B.

Sounding: CPT-GSC08 Date: 6/22/2011 10:35



Max. Depth: 80.052 (ft)
Avg. Interval: 0.328 (ft)

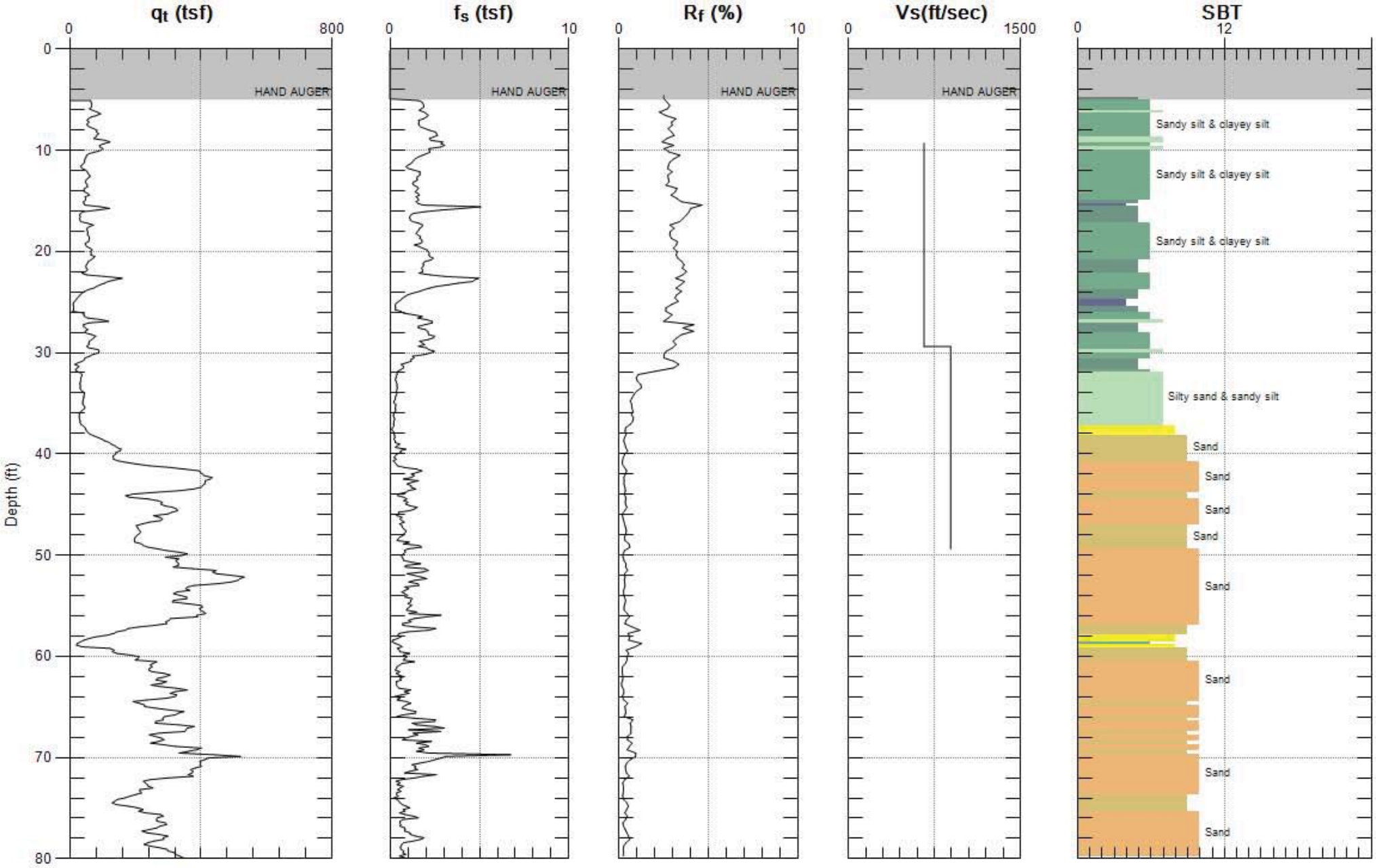
SBT: Soil Behavior Type (Robertson 1990)

GREGG

GEOSYNTEC

Site: SONOMA RUSSIAN RIV. Engineer: JESSICA B.

Sounding: CPT-GSC08 Date: 6/22/2011 10:35



Max. Depth: 80.052 (ft)
Avg. Interval: 0.328 (ft)

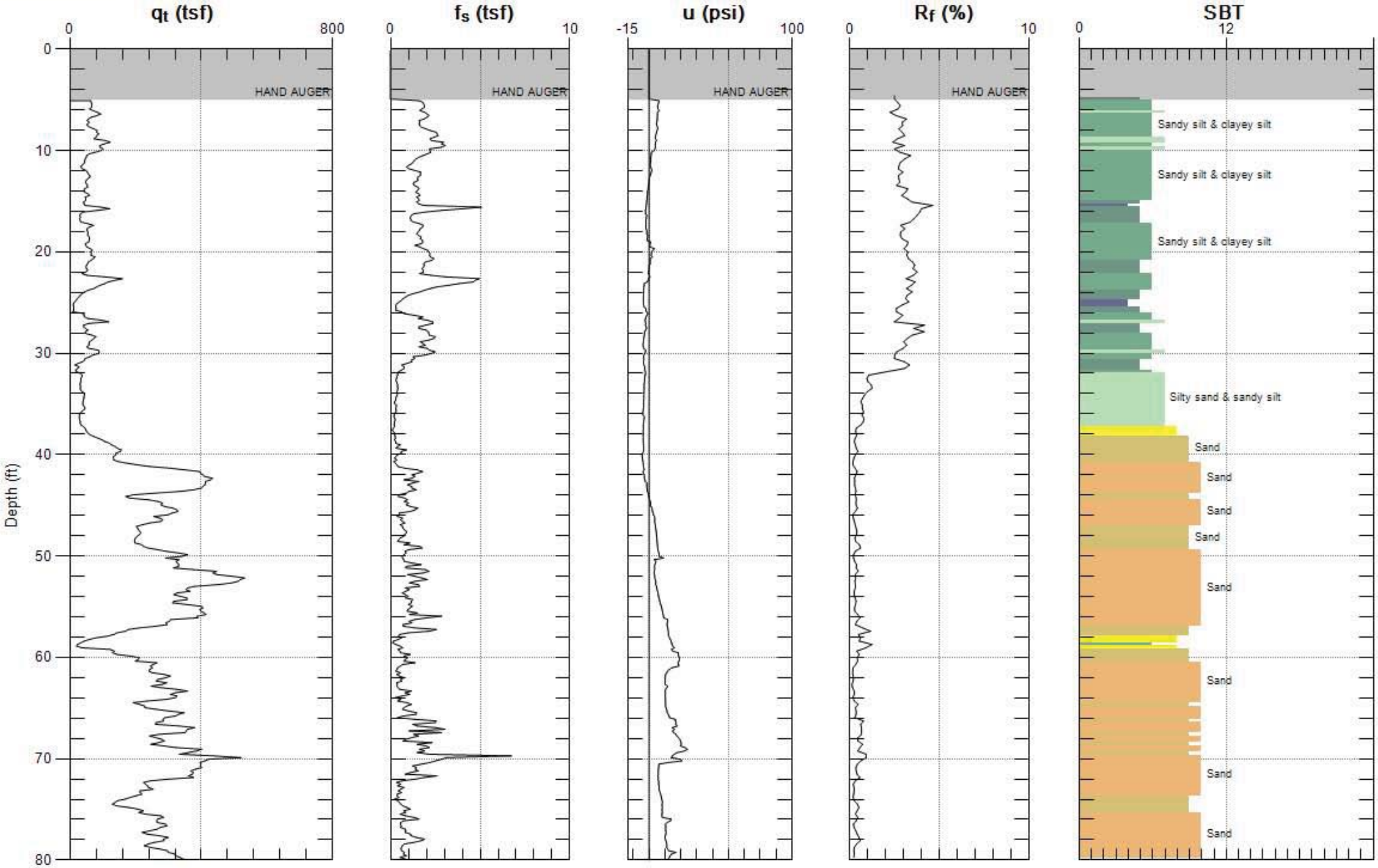
SBT: Soil Behavior Type (Robertson 1990)

GREGG

GEOSYNTEC

Site: SONOMA RUSSIAN RIV. Engineer: JESSICA B.

Sounding: CPT-GSC08 Date: 6/22/2011 10:35



Max. Depth: 80.052 (ft)
Avg. Interval: 0.328 (ft)

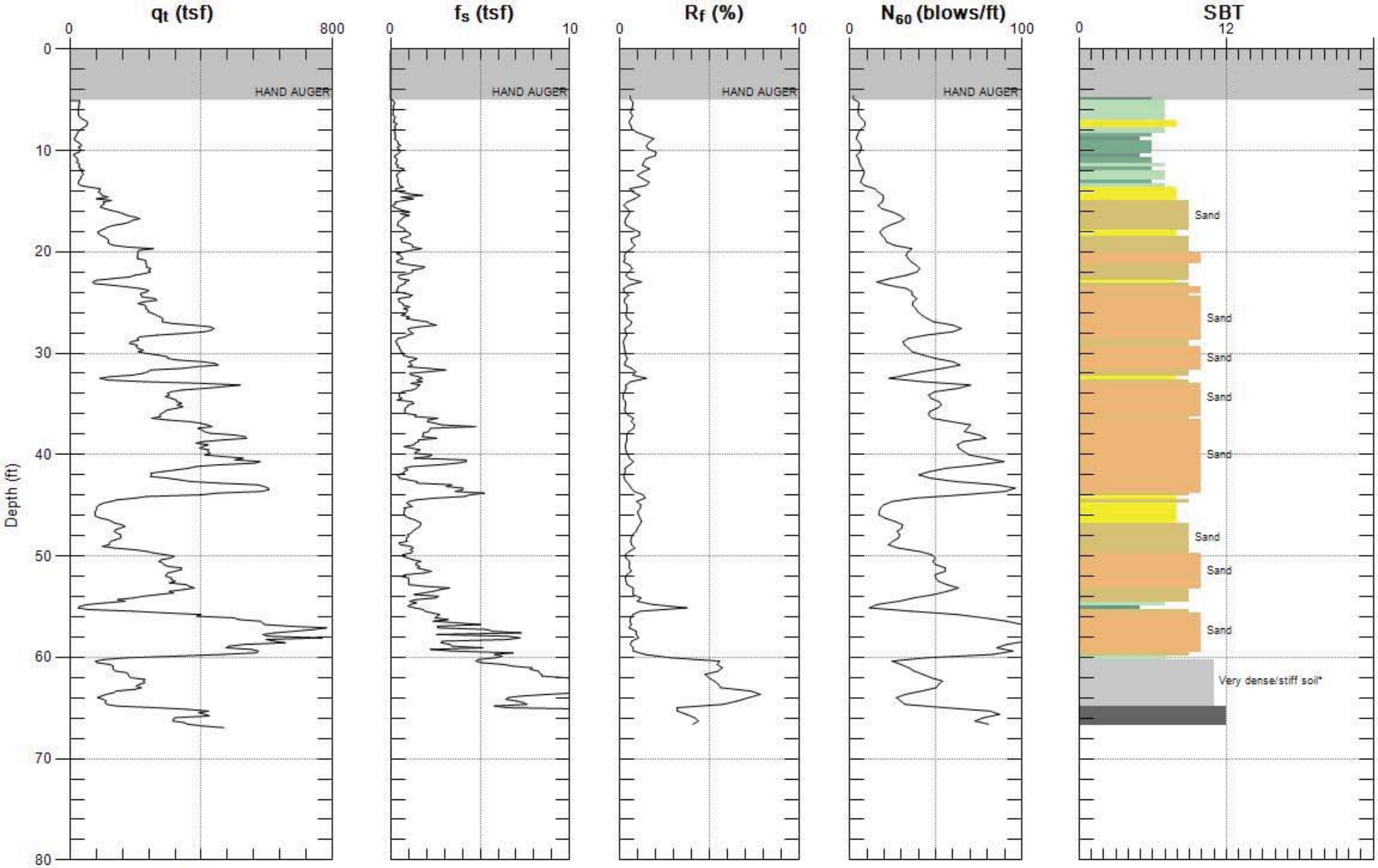
SBT: Soil Behavior Type (Robertson 1990)

GREGG

GEOSYNTEC

Site: SONOMA RUSSIAN RIV. Engineer: JESSICA B.

Sounding: CPT-GSC12 Date: 6/23/2011 02:09



Max. Depth: 66.929 (ft)
Avg. Interval: 0.328 (ft)

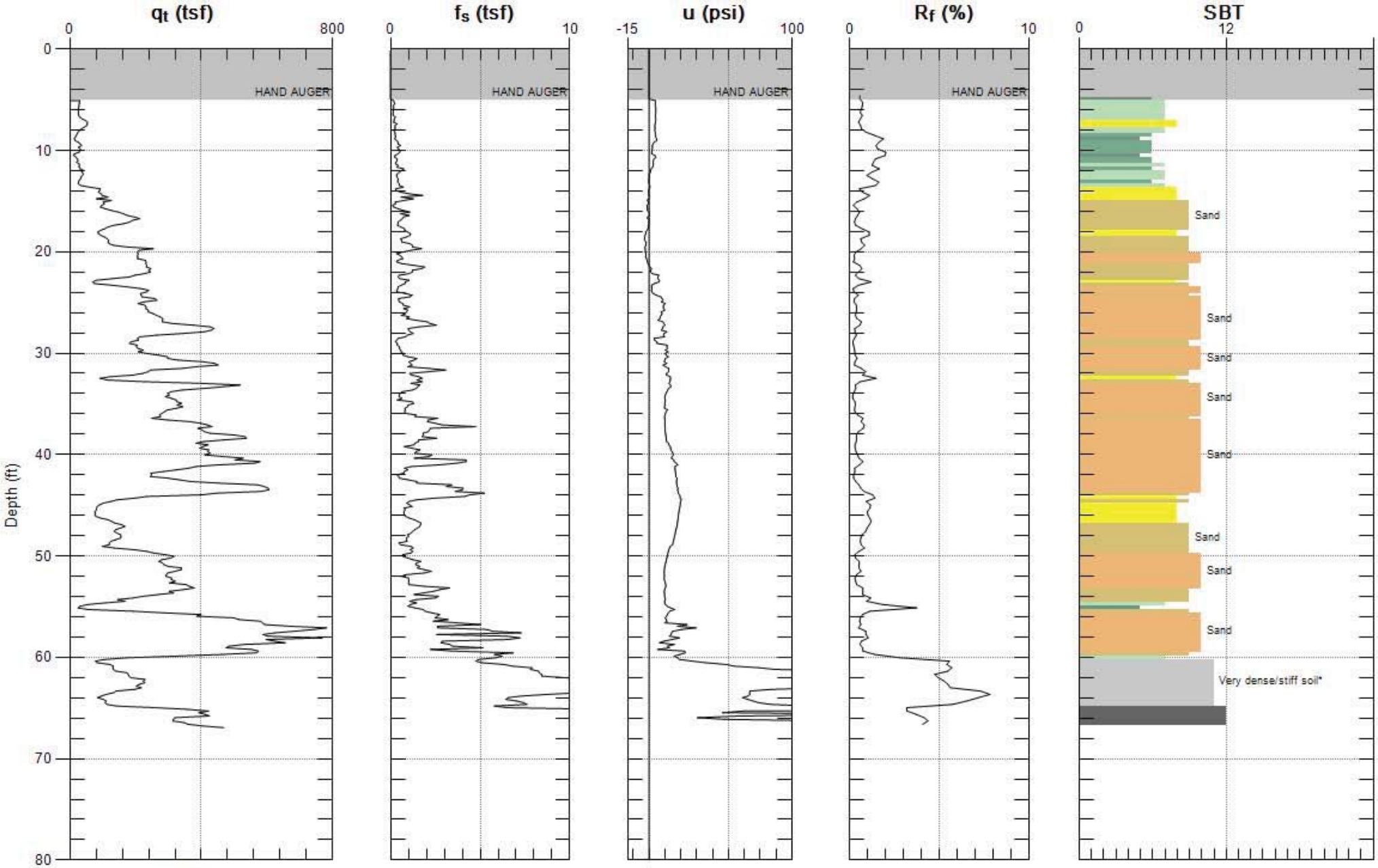
SBT: Soil Behavior Type (Robertson 1990)

GREGG

GEOSYNTEC

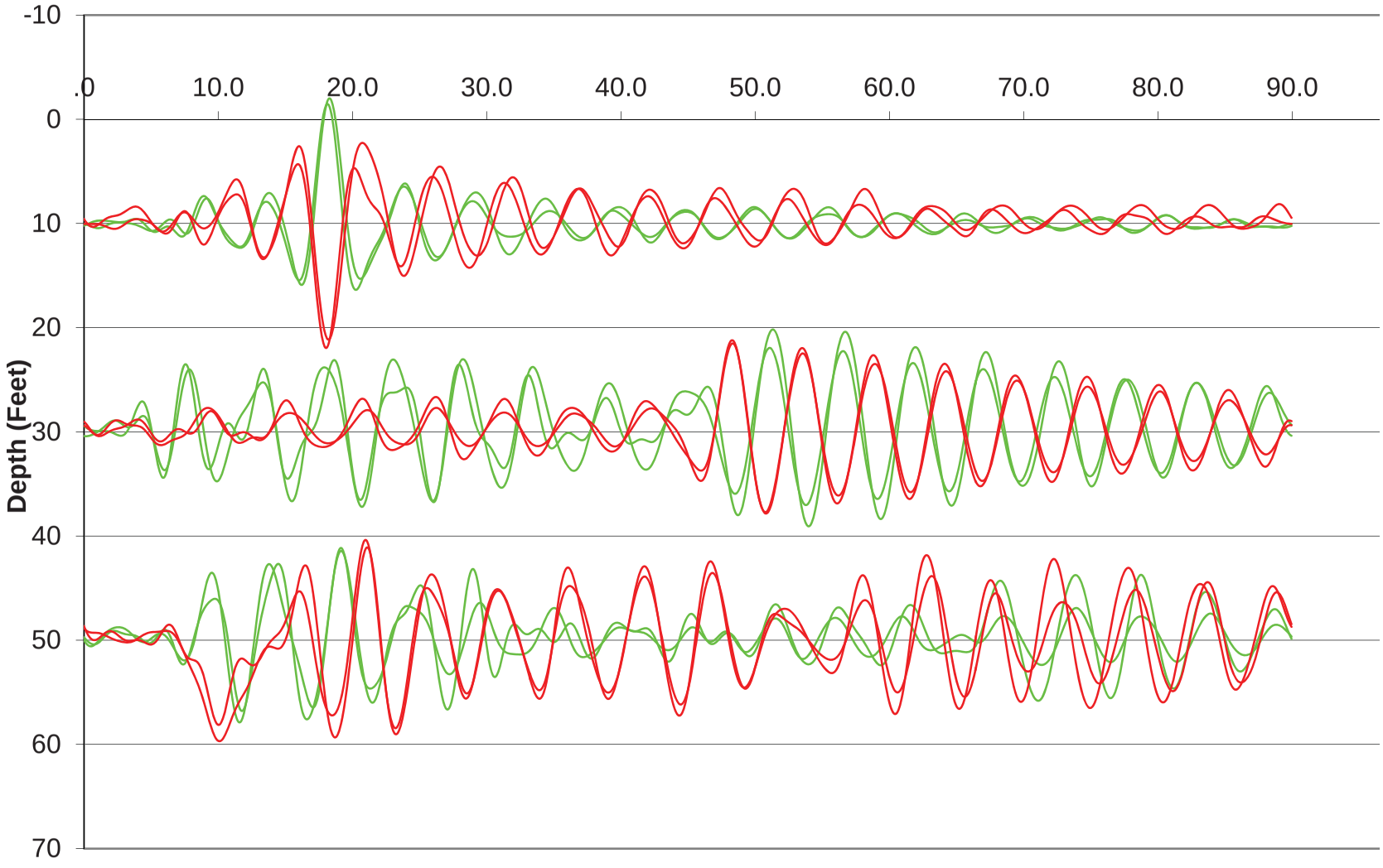
Site: SONOMA RUSSIAN RIV. Engineer: JESSICA B.

Sounding: CPT-GSC12 Date: 6/23/2011 02:09



Max. Depth: 66.929 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)


Waveforms for Sounding CPT-GSC07
Time (ms)
Depth (Feet)
-10
0
10
20
30
40
50
60
70
.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0

Shear Wave Velocity Calculations

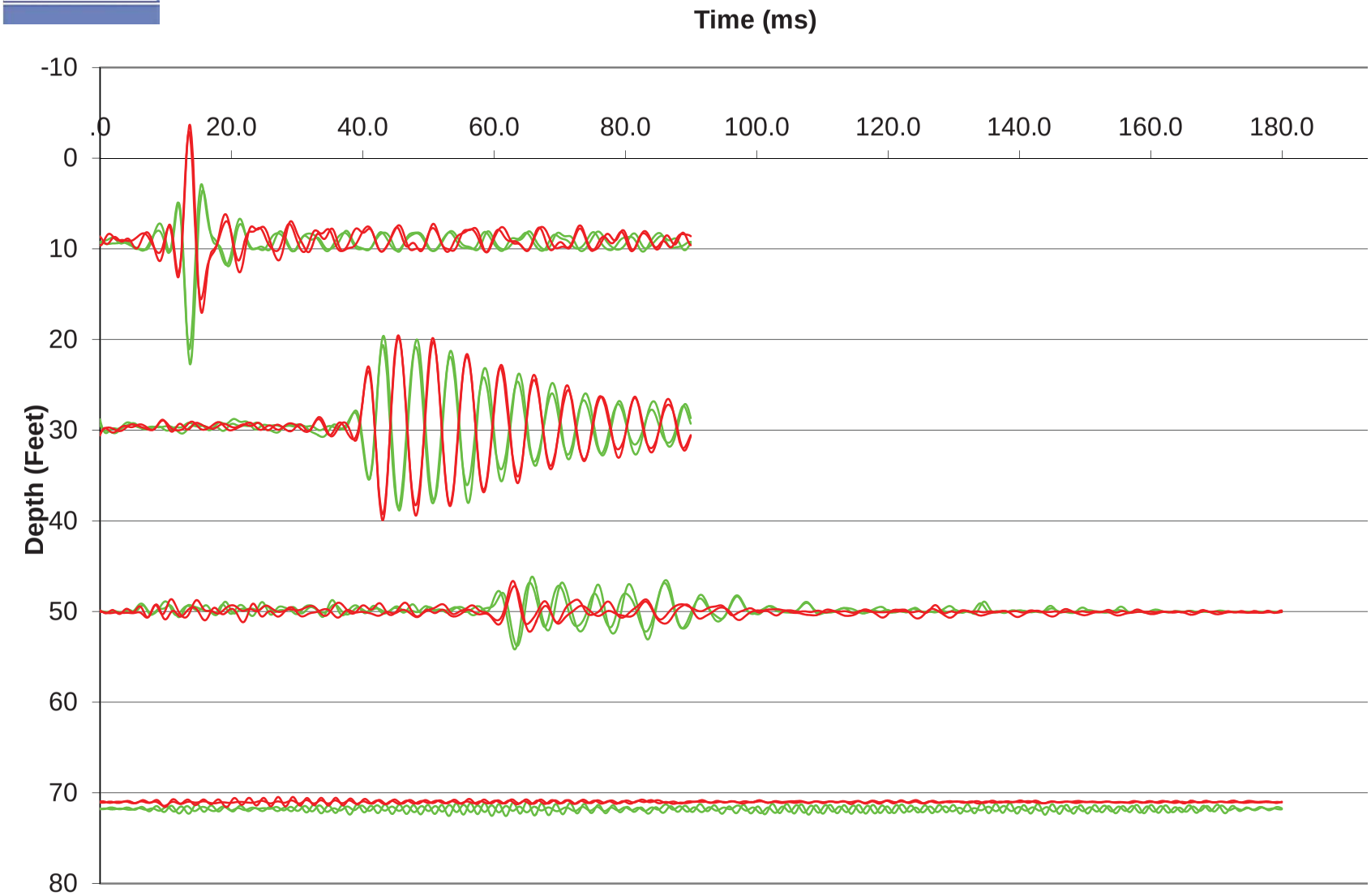
S.RUSSIAN RIVER
CPT-GSC07

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

06/21/11

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.33	9.67	9.82	9.82	18.1000			
30.02	29.36	29.41	19.59	51.3500	33.2500	589.2	19.52

Waveforms for Sounding CPT-GSC08



Shear Wave Velocity Calculations

S.RUSSIAN RIVER
CPT-GSC08

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

06/22/11

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.01	9.35	9.49	9.49	10.6000			
30.02	29.36	29.41	19.91	40.7500	30.1500	660.4	19.35
50.03	49.37	49.40	19.99	63.1500	22.4000	892.6	39.37

Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D 2937)

CTL Job No:	461-181b	**Project No.**	MMW597	**By:**	RU
Client:	Geosyntec Consultants	**Date:**	08/05/11		
Project Name:	Sonoma-Healdsburg	**Remarks:**			

Boring:	Collector RDS	Collector RDS	Collector RDS	Collector RDS	Collector #6	Collector #6	Collector #6	
Sample:	GSC-07	GSC-07	GSC-07	GSC-07	GSC-09	GSC-09	GSC-09	
Depth, ft:	33-34	37-38	46-47	50-51	49-50	55-56	65-66	
Visual Description:	Grayish Brown Well-Graded SAND w/ Silt & Gravel	Grayish Brown Clayey SAND	Grayish Brown Well-Graded SAND w/ Silt & Gravel	Grayish Brown Silty SAND w/ Gravel	Brown Silty SAND	Grayish Brown Well-Graded SAND w/ Silt & Gravel	Grayish Brown Well-Graded SAND w/ Silt & Gravel	
Actual G_s								
Assumed G_s	2.70	2.70	2.70	2.70	2.70	2.70	2.70	
Moisture, %	8.9	13.9	10.3	7.7	15.4	6.5	9.4	
Wet Unit wt, pcf	135.8	118.0	137.8	134.8	121.9	120.0	139.0	
Dry Unit wt, pcf	124.6	103.6	124.9	125.2	105.6	112.6	127.0	
Dry Bulk Dens.pb, (g/cc)	2.00	1.66	2.00	2.00	1.69	1.80	2.03	
Saturation, %	68.2	59.7	79.2	60.1	69.6	35.5	77.4	
Total Porosity, %	26.1	38.6	25.9	25.8	37.4	33.2	24.7	
Volumetric Water Cont, θw	17.8	23.0	20.6	15.5	26.0	11.8	19.1	
Volumetric Air Cont., θa	8.3	15.5	5.4	10.3	11.4	21.4	5.6	
Void Ratio	0.35	0.63	0.35	0.35	0.60	0.50	0.33	
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity

NOTE: Extremely small samples, such as these, tend to be highly disturbed and less representative of the in-situ conditions. A diameter of 2.5 inches is the recommended minimum for this type of testing. This should be taken into account when interpreting these results.

Moisture-Density-Porosity Report

Cooper Testing Labs, Inc. (ASTM D 2937)

CTL Job No: 461-181a
Client: Geosyntec Consultants
Project Name: Sonoma-Healdsburg
Project No. MMW597
Date: 08/02/11
By: RU
Remarks: Collector #3;GSC-02 @ 18-19' - sample disturbed; m/c only.

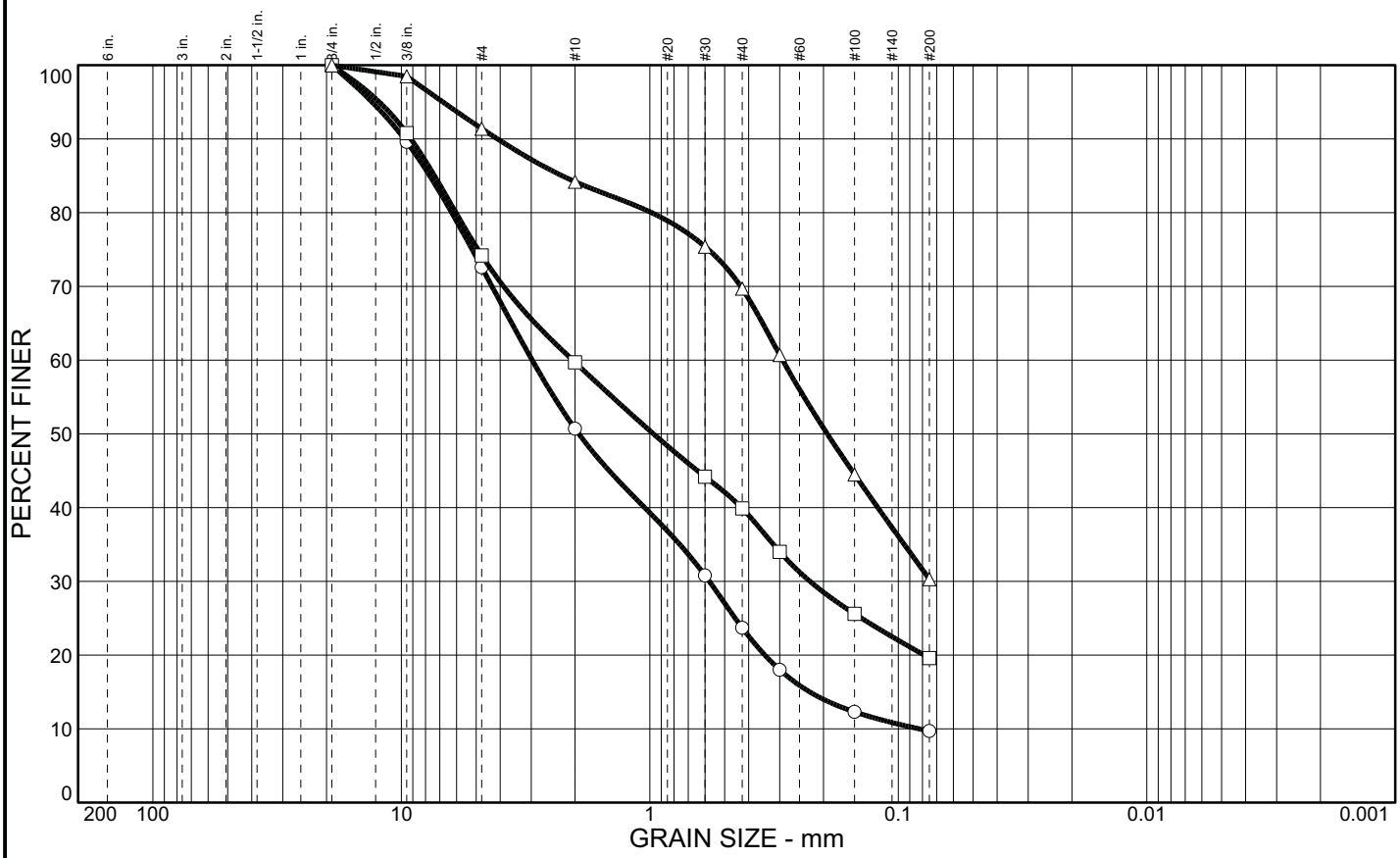
Boring:	Collector #3	Collector #3	Collector #3	Collector #3	Collector #3	Collector RDS	Collector RDS	Collector RDS
Sample:	GSC-02	GSC-02	GSC-02	GSC-02	GSC-02	GSC-07	GSC-07	GSC-07
Depth, ft:	18-19	22-23	30-31	54-55	48.5-49	9-10	14-15	19-20
Visual Description:	Brown Well-Graded SAND w/ Silt & Gravel	Grayish Brown Silty SAND	Brown Poorly Graded SAND w/ Silt & Gravel	Grayish Brown Well-Graded SAND w/ Silt & Gravel	Brown Silty SAND	Brown SILT w/ Sand	Brown Sandy SILT	Grayish Brown Silty SAND w/ Gravel
Actual G_s								
Assumed G_s		2.70	2.70	2.70	2.70	2.70	2.70	2.70
Moisture, %	4.9	10.7	11.9	10.6	18.3	28.2	19.4	3.7
Wet Unit wt, pcf		142.2	140.4	139.7	131.5	112.0	98.2	125.1
Dry Unit wt, pcf		128.4	125.5	126.3	111.2	87.3	82.3	120.6
Dry Bulk Dens.ρb, (g/cc)		2.06	2.01	2.02	1.78	1.40	1.32	1.93
Saturation, %		92.4	93.4	85.5	95.4	81.8	49.9	25.2
Total Porosity, %		23.9	25.6	25.1	34.1	48.2	51.2	28.5
Volumetric Water Cont,θw		22.1	23.9	21.5	32.5	39.4	25.5	7.2
Volumetric Air Cont., θa		1.8	1.7	3.6	1.6	8.8	25.7	21.3
Void Ratio		0.31	0.34	0.34	0.52	0.93	1.05	0.40
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity

NOTE: Extremely small samples, such as these, tend to be highly disturbed and less representative of the in-situ conditions. A diameter of 2.5 inches is the recommended minimum for this type of testing. This should be taken into account when interpreting these results.

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		27.4	62.9	9.7					
□		25.8	54.6	19.6					
△		8.6	61.1	30.3		SM		20.2	21.7

SIEVE inches size	PERCENT FINER ○	□	△
3/4"	100.0	100.0	100.0
3/8"	89.6	90.8	98.5
GRAIN SIZE			
D_{60}	2.98	2.04	0.292
D_{30}	0.576	0.227	
D_{10}	0.0824		
COEFFICIENTS			
C_c	1.36		
C_u	36.11		

SIEVE number size	PERCENT FINER ○	□	△
#4	72.6	74.2	91.4
#10	50.7	59.7	84.2
#30	30.8	44.2	75.4
#40	23.7	39.9	69.7
#50	18.0	34.0	60.7
#100	12.3	25.6	44.5
#200	9.7	19.6	30.3

SOIL DESCRIPTION

○ Grayish Brown Well-Graded SAND w/ Silt & Gravel

□ Grayish Brown Silty SAND w/ Gravel

△ Gray Silty SAND

REMARKS:

○

□

△

○ Source: Collector RDS Sample No.: GSC-07 Elev./Depth: 46-47'

□ Source: Collector RDS Sample No.: GSC-07 Elev./Depth: 50-51'

△ Source: Collector RDS Sample No.: DP6B Elev./Depth: 8-12'

COOPER TESTING LABORATORY

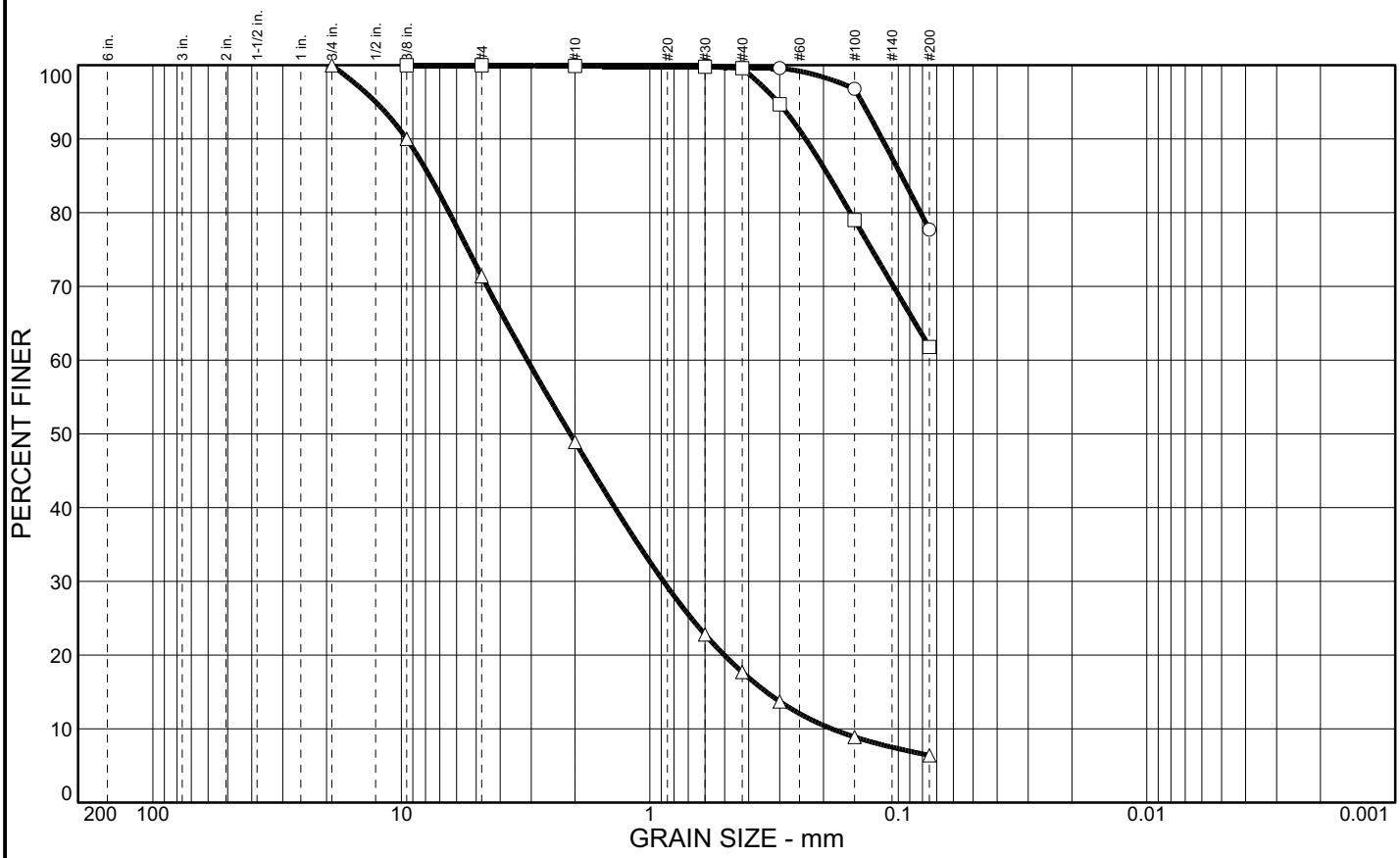
Client: Geosyntec Consultants

Project: Sonoma-Healdsburg - MMW597

Project No.: 461-181

Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○			22.3	77.7		ML		NP	
□			38.2	61.8		ML		NP	
△		28.6	65.0	6.4					

SIEVE inches size	PERCENT FINER ○	□	△
3/4"			100.0
3/8"		100.0	90.0
GRAIN SIZE			
D_{60}			3.11
D_{30}			0.881
D_{10}			0.185
COEFFICIENTS			
C_c			1.35
C_u			16.82

SIEVE number size	PERCENT FINER ○	□	△
#4		100.0	71.4
#10	100.0	99.9	48.9
#30	99.9	99.8	22.8
#40	99.7	99.6	17.7
#50	99.6	94.7	13.7
#100	96.8	79.0	8.9
#200	77.7	61.8	6.4

SOIL DESCRIPTION

○ Brown SILT w/ Sand

□ Brown Sandy SILT

△ Grayish Brown Well-Graded SAND w/ Silt & Gravel

REMARKS:

○

□

△

○ Source: Collector RDS Sample No.: GSC-07 Elev./Depth: 9-10'

□ Source: Collector RDS Sample No.: GSC-07 Elev./Depth: 14-15'

△ Source: Collector RDS Sample No.: GSC-07 Elev./Depth: 33-34'

COOPER TESTING LABORATORY

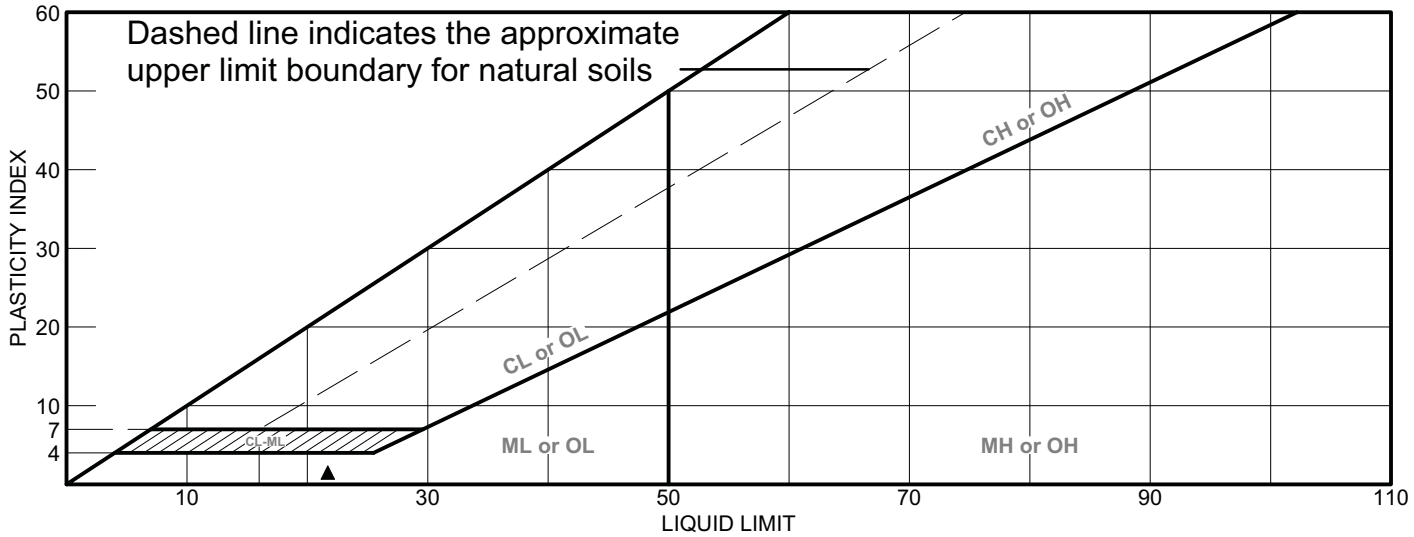
Client: Geosyntec Consultants

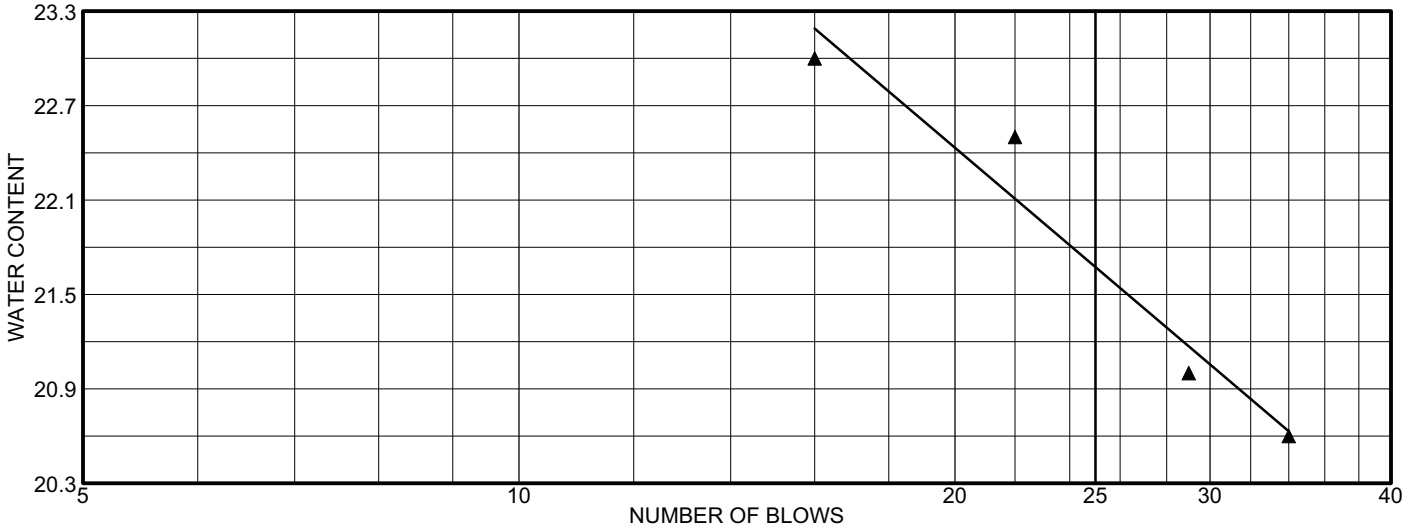
Project: Sonoma-Healdsburg - MMW597

Project No.: 461-181

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown SILT w/ Sand		NP	NP	99.7	77.7	ML
■	Brown Sandy SILT		NP	NP	99.6	61.8	ML
▲	Gray Silty SAND	21.7	20.2	1.5	69.7	30.3	SM

Project No. 461-181 **Client:** Geosyntec Consultants

Project: Sonoma-Healdsburg - MMW597

● **Source:** Collector RDS **Sample No.:** GSC-07 **Elev./Depth:** 9-10'

■ **Source:** Collector RDS **Sample No.:** GSC-07 **Elev./Depth:** 14-15'

▲ **Source:** Collector RDS **Sample No.:** DP6B **Elev./Depth:** 8-12'

Remarks:

● Sample was prepared using the wet prep method. Could not roll out. Sample slides in bowl. Non-plastic.

■ Sample was prepared using the wet prep method. Could not roll out. Sample slides in bowl. Non-plastic.

▲ Sample was prepared using the wet prep method.

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

Appendix C – Laboratory Results

Treadwell&Rollo
9608 Kiefer Boulevard, Suite 7
Sacramento, California 95827
(916) 476-6790

Client: HDR - Garrett Harris

Project Number: 164479

Project Name: 164479 HDR

Notes / Deviation from ASTM:

Moisture Content, Dry Density, and Percent Passing No. 200 Sieve Determination

Boring :	B-1	B-1	B-2	B-2				
Sample No.								
Depth (feet):	45.00	58.00	40.00	55.00				
Date Tested:	05/21/13	05/22/13	05/22/13	05/22/13				
Tested BY:	KEJ	KEJ	KEJ	KEJ				
Sample Description:	SAND (SP), Gray	SAND (SP), Gray-brown	SAND (SP), Gray	SAND (SP), Gray				

Wet Unit Weight Determination (ASTM D2937)

	Notes/Deviation from ASTM								
A.	Height (inches):								
B.	Diameter (inches):								
C.	Wet Weight + Tare (g):								
D.	Tare (g)								
E.	Wet Weight Soil (g) =[C-D]	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Moisture Content Determination (ASTM 2216)

	Notes/Deviation from ASTM								
	Pan No.:	M	O	N	L				
F.	Pan Weight (g)	174.7	193.5	194.5	110.5				
G.	Wet Weight Soil + Pan (g)	251.9	264.3	239.9	150.4				
H.	Dry Soil Weight + Pan (g)	244.5	251.9	231.6	145.7				
I.	Moisture Content (%)	10.6%	21.3%	22.6%	13.6%	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
J.	Wet Density Soi (pcf) =[E/(58049.89*A*PI*B^2/4)]	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K.	Dry Density Soil (pcf) =[J/(1+I)]	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

Fines Content Determination (ASTM 1140)

	Notes/Deviation from ASTM								
	Method A or B	A	A	A	A				
L.	Sample Soaked? Length of Time	-	-	-	-				
M.	Pan No.:	E	I	KEJ	F				
N.	Pan Weight (g)	240.23	225.62	226.56	241.16				
	Dry Weight Soil + Pan (Before Wash) (g)	373	334	327	355				
	Dry Weight Soil + Pan (After Wash) (g)	369.3	330.5	321.6	350.0				
	% passing #200 =100%*[(M-N)/(M-L)]	2.6%	3.2%	5.6%	4.5%	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

Appendix D – Seismic

Accelerations were developed for this project using available seismic information and site information as summarized in the text of this report. The faults that were considered are listed in Section 7.1. Characteristics for each fault included rupture distance; depth to top of rupture; dip angle; depth to materials with shear wave velocity of 1,000 meters per second; and fault rupture width were used to develop appropriate M_w for design consideration.

Discussions with the client and their 3rd part review consultant requested that HDR review several methods for developing anticipated site ground accelerations. The parameters listed above were used as input parameters into Next Generation Attenuation (NGA) relations models, Probabilistic Seismic Hazard Analysis (PSHA), and CBC/ASCE 7 where appropriate. Resulting pga's are presented in Table D-1.

- NGA models included models by Abraham and Silva (2008); Boore & Atkinson (2008); Campbell & Bozorgnia (2008); and Chiou & Youngs (2008). Fault characteristics were used in as input parameters into a published spreadsheet to develop acceleration response spectra. These models rely in part on shear wave velocities from site and distances from the faults. PGAs were selected from the median and 84 percentile curves of the resultant attenuation plots which are presented herein Appendix D. The attenuation plots were weighted equally to develop a median and 84 percentile PGA for the Rodgers Creek, San Andreas and Maccaama-Garberville faults. The PGAs for each of these faults are presented the table below.

- PGAs were also separately developed using the National Seismic Hazard Mapping Project (NSHMP) (2008). Using the Hazards Tool applications on the USGS website http://geohazards.usgs.gov/hazardtool/ and https://geohazards.usgs.gov/deaggint/2008/ with the project site latitude and longitude (38.500, -122.886) along with respective input parameters to develop Uniform Hazard Response Spectra (UHRS), PGAs and seismic-hazard deaggregation. The resultant UHRS' were used with the 2010 CBC and ASCE 7. The resultant UHRS' are presented in Appendix D. The sites shear wave velocities obtained by MMI/GeoSyntec along with the sites latitude and longitude was used as the input parameters into USGS deaggregation tool. The deaggregation tool selected and combined faults to provide the outputs presented herein Appendix D. The largest PGA resultant from these tools is presented in the table below.

Table 7. PGA Results

Methodology	Fault	PGA (g)	M_w
NGA Median	Rodgers Creek	0.26	6.9
	Maacama-Garberville	0.19	6.9
	San Andreas	0.23	7.9
NGA 84%	Rodgers Creek	0.42	6.9
	Maacama-Garberville	0.32	6.9
	San Andreas	0.38	7.9
Deaggregated (USGS PSHA Tool)	Combined	0.44	7.1
CBC/ASCE 7	Rodgers Creek	0.44	6.9

The above table presents the results of both deterministic and probabilistic results for comparison purposes.

CALIFORNIA DEPARTMENT OF CONSERVATION
DIVISION OF MINES AND GEOLOGY
OPEN-FILE REPORT 96-08

U.S. DEPARTMENT OF THE INTERIOR
U.S. GEOLOGICAL SURVEY OPEN-FILE REPORT 96-706

APPENDIX A - CALIFORNIA FAULT PARAMETERS

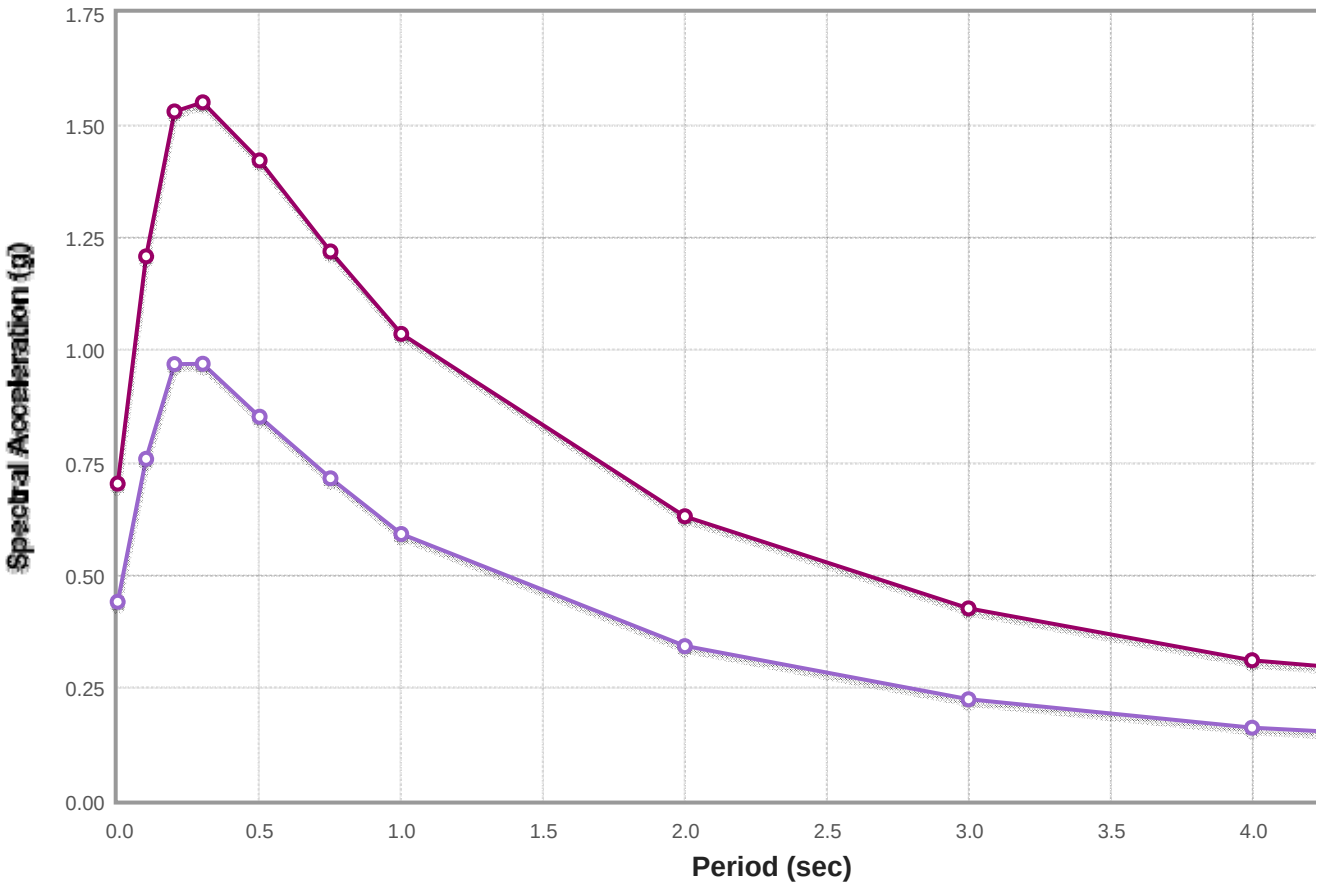
FAULT NAME AND GEOMETRY (ss) strike slip, (r) reverse, (n) normal (rl) rt. lateral, (ll) left lateral, (o) oblique	LENGTH (km)		SLIP RATE (mm/yr)		RANK (1)	Mmax (2)	CHAR. RATE (events/yr)	R.I. (3)	Down dip Width (km)(4)		ruptop (5)	rupbot (6)	rake	dip	daz (7)	Endpt N	Endpt S	COMMENTS
Rodgers Creek (rl-ss)	63	6	9.00	2.00	M	7.0	0.00450	222	10	2	0	10	180	90	0	-122.77; 38.54	-122.34; 38.09	Slip rate is composite of slip rate reported by Schwartz, et al. (1992) and slip rate from Hayward fault (Lienkaemper and Borchardt, 1996). Recurrence (222yrs) and slip per event (2.0 m) are based on WGCEP (1990
San Andreas (1906) (rl-ss)	470	47	24.00	3.00	M	7.9	0.00476	210	12	2	0	12	180	90	0	-124.41; 40.25	-121.51; 36.82	Slip rate based on Niemi and Hall (1992) and Prentice, et al (1991). Assumption that 1906 events rupture North Coast, Pennisula, and Santa Cruz Mtns. segments to San Juan Bautista. Max. magnitude based on 1906 average 5 m displacement (WGCEP, 1990; Lienkaemper, 1996).
Maacama (south) (rl-ss)	41	4	9.00	2.00	P	6.9	0.00454	220	12	2	0	12	180	90	0	-123.00; 38.86	-122.69; 38.58	Slip rate of 9 mm/yr based on assumption that dextral slip from Hayward - Rodgers Crk. flt carried NW along Maacama zone (WGNCEP, 1996). Max. magnitude based on assumed 2 m slip.

Geologic Hazards Science Center

Hazard Curve Application

Set Location | Hazard Curves | UHRS | AFE vs. Site Class | Data Access | Help & Info

Latitude: **38.50000** Longitude: **-122.88600**



Curve Selection

B | BC | C | CD | D | DE

2.0 %
5.0 %
10.0 %
20.0 %

Add Custom PE

Add value as:

PE (% in 50 yrs.)

RP (yrs.)

Add

--

Cursor Values

SA: 1.519e+0

Period: 5.37

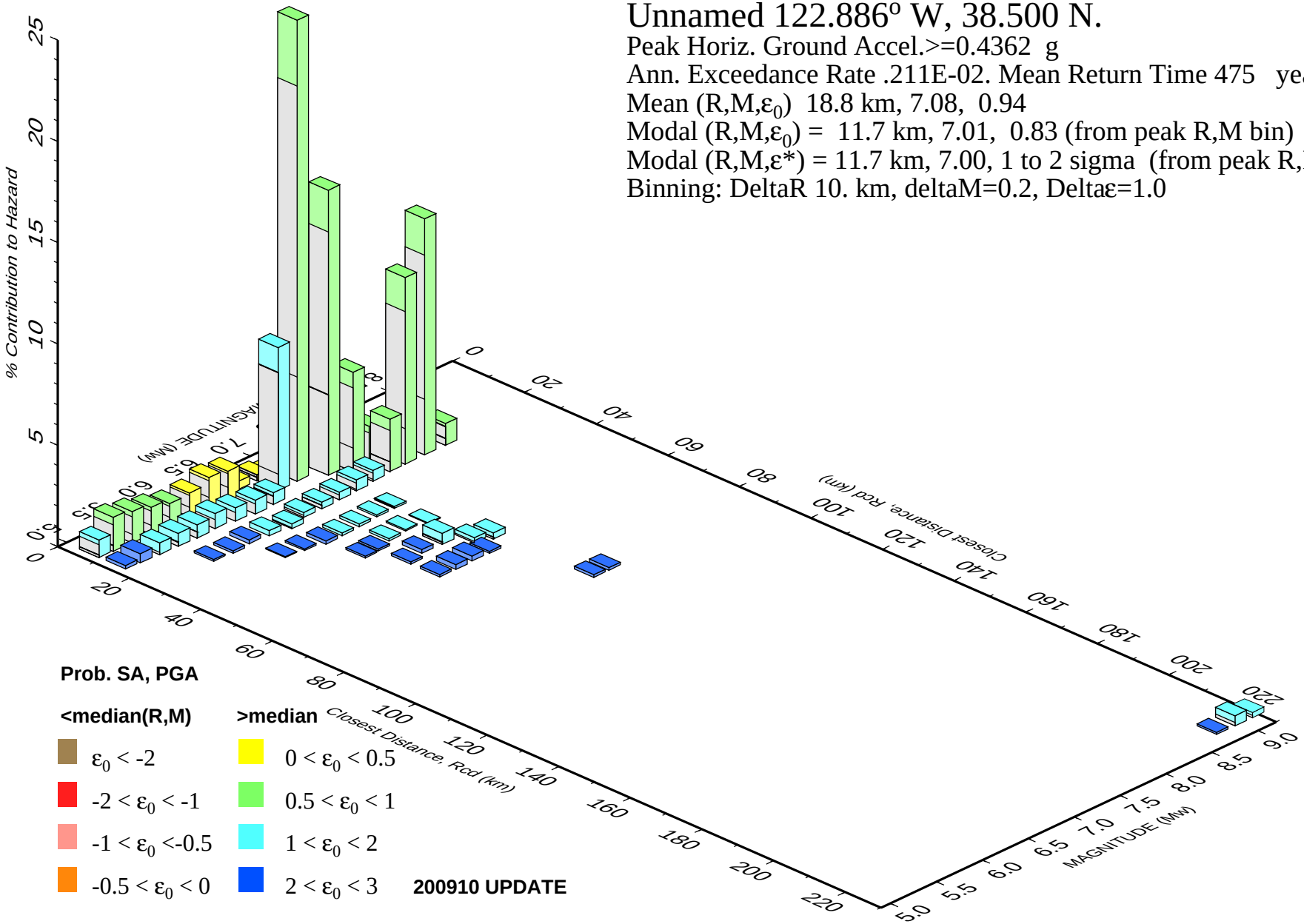
Plot Options

Crosshair

Value tooltip

PSH Deaggregation on NEHRP D soil
Unnamed 122.886° W, 38.500 N.

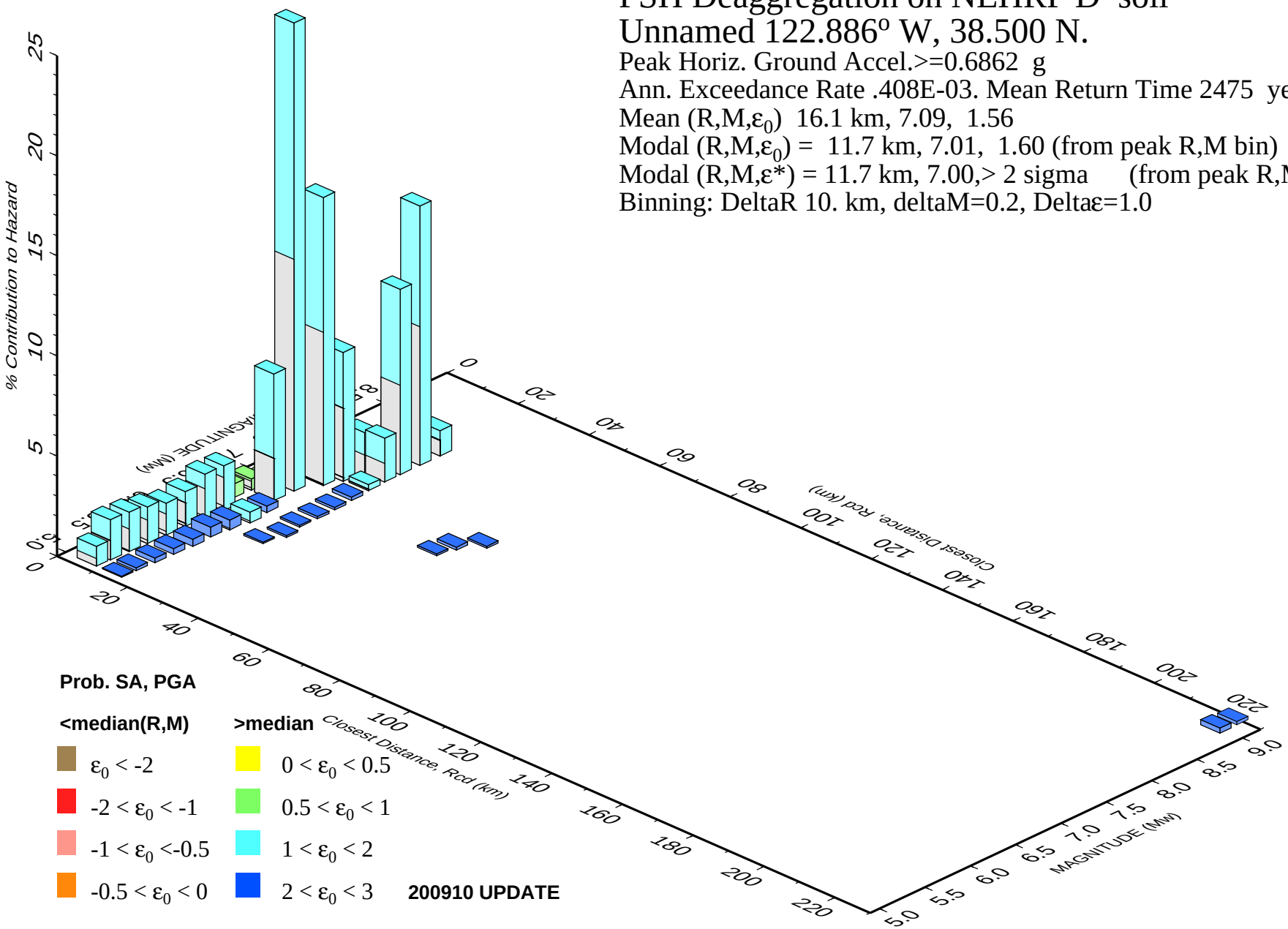
Peak Horiz. Ground Accel.>=0.4362 g
Ann. Exceedance Rate .211E-02. Mean Return Time 475 years
Mean (R,M,ε_0) 18.8 km, 7.08, 0.94
Modal (R,M,ε_0) = 11.7 km, 7.01, 0.83 (from peak R,M bin)
Modal (R,M,ε*) = 11.7 km, 7.00, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0



GMT 2013 May 7 23:29:24 **Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 252. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted**

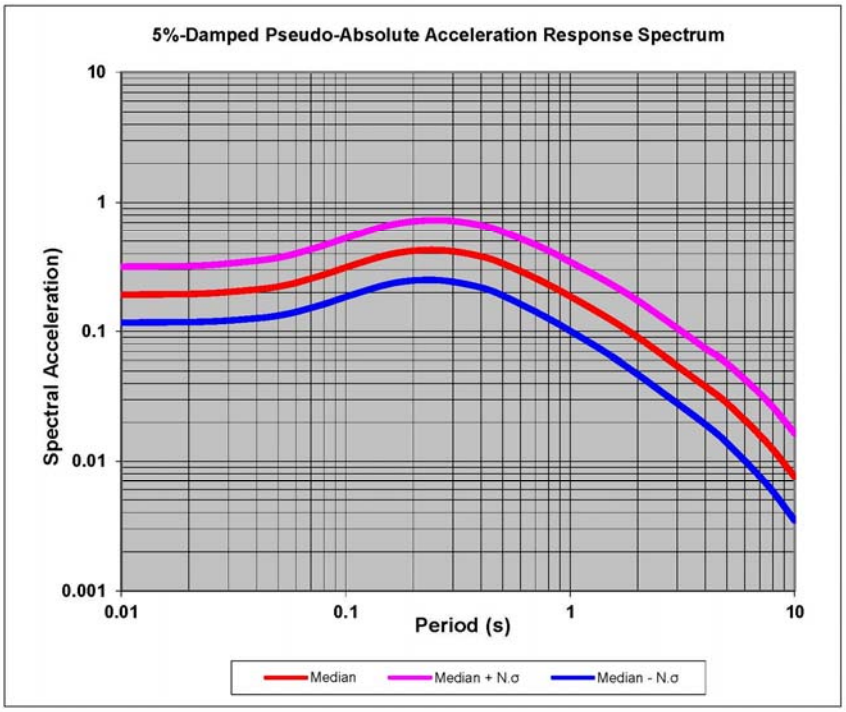
PSH Deaggregation on NEHRP D soil
Unnamed 122.886° W, 38.500 N.

Peak Horiz. Ground Accel.>=0.6862 g
Ann. Exceedance Rate .408E-03. Mean Return Time 2475 years
Mean (R,M,ε_0) 16.1 km, 7.09, 1.56
Modal (R,M,ε_0) = 11.7 km, 7.01, 1.60 (from peak R,M bin)
Modal (R,M,ε*) = 11.7 km, 7.00,> 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0

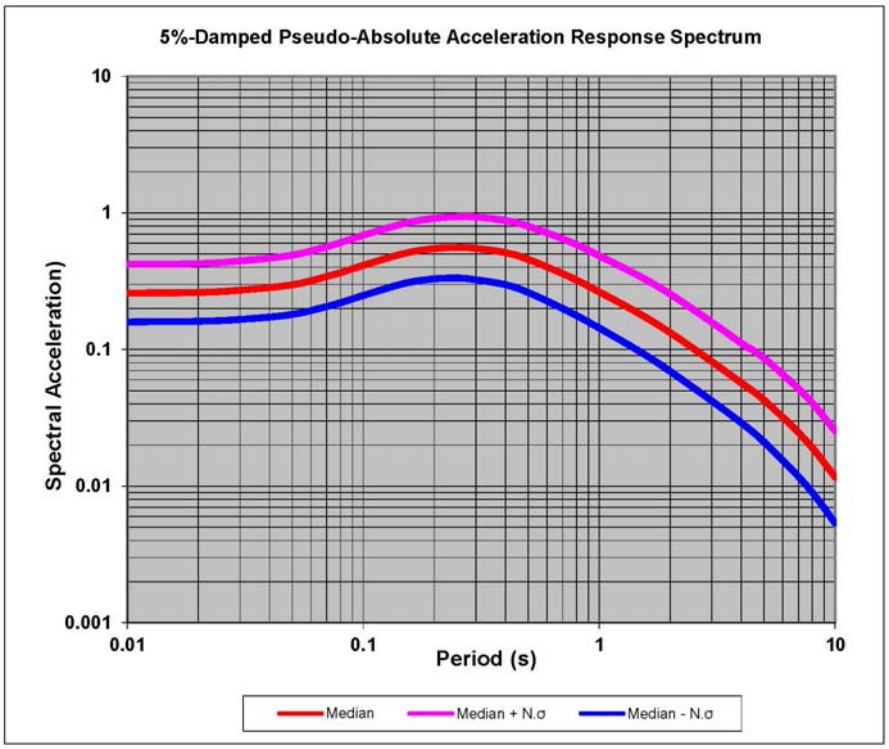


GMT 2013 May 17 19:33:52 **Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 252. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted**

PEER NGA Model
Results

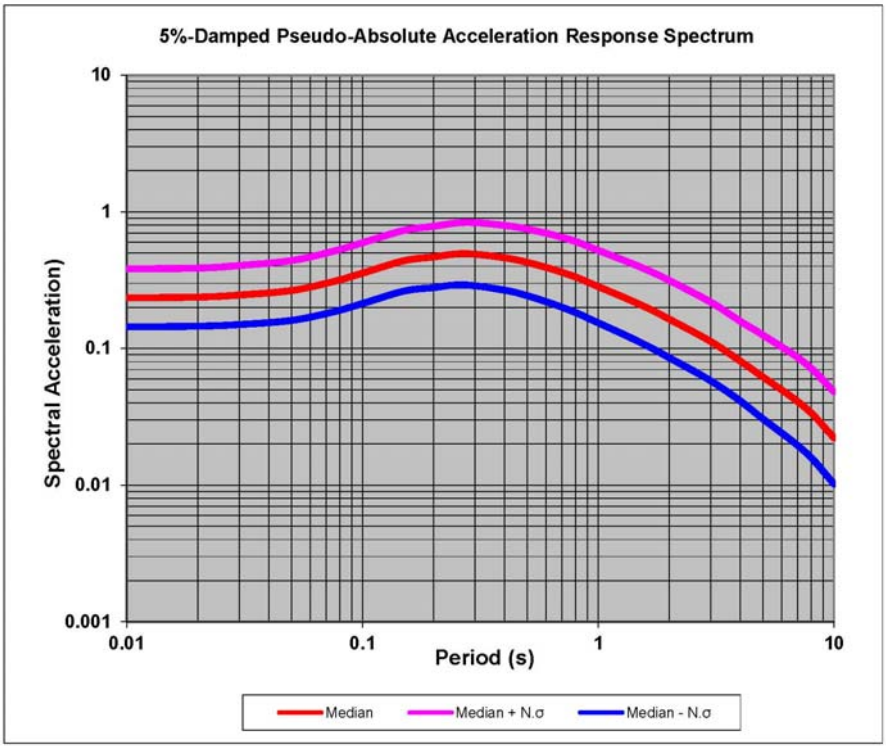


Macaama-Garberville Fault



Rodgers Creek Fault

PEER NGA Model
Results



San Andreas Fault

Appendix E – Liquefaction Analysis

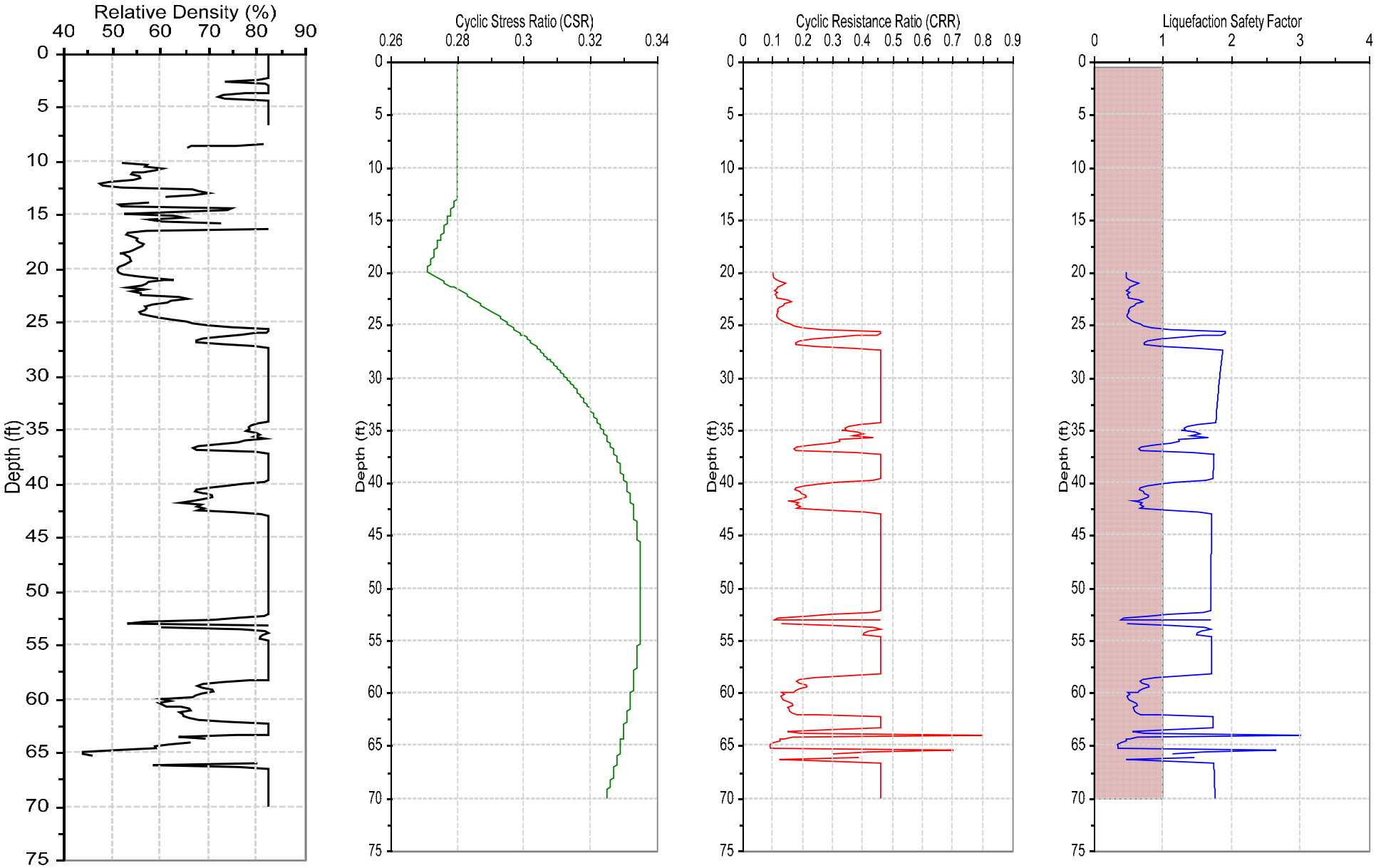
Liquefaction analysis was performed using methods presented by Idriss and Boulanger (2008) and Ishihara (1985) for triggering and vertical deformations, respectively. CPTs were the primary source of data used in the analysis for the reasons described in the main text. NovaCPT by NovaTech was used to calculate and develop the plots presented herein Appendix E. Vertical deformations in layers that were determined to have factors of safety of 1.0 or less were calculated using methods by Ishihara (1985) using the computer program

CPT Liquefaction Analysis Results

HDR Inc

Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-1.cor
Groundwater : 20 ft
Coordinates : n.a.
Calculated By :
Checked By :

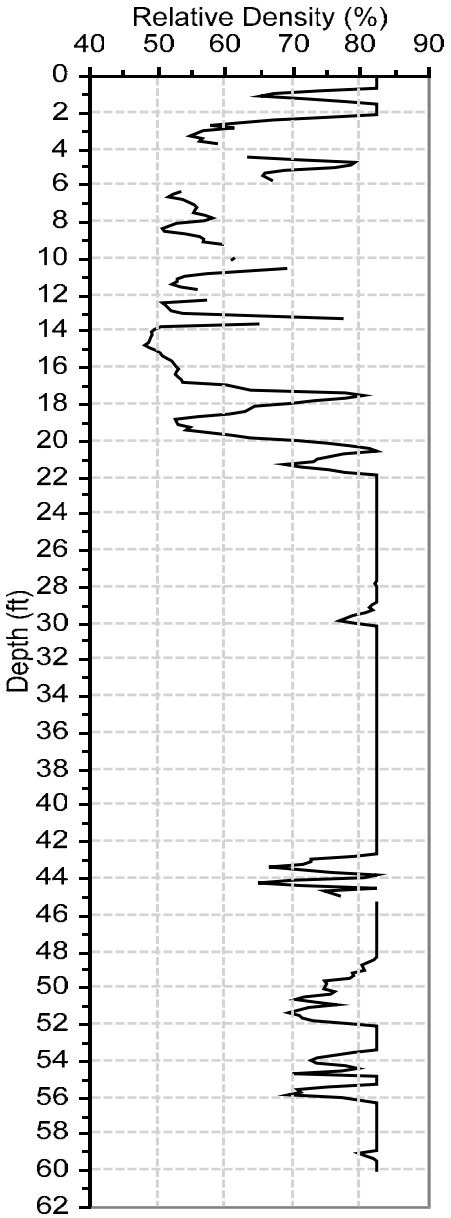


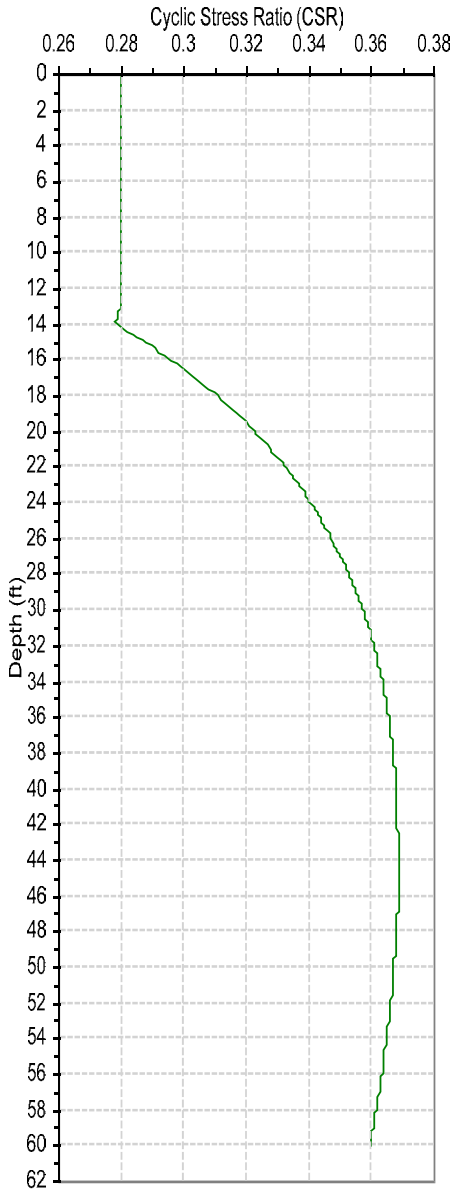
CPT Liquefaction Analysis Results

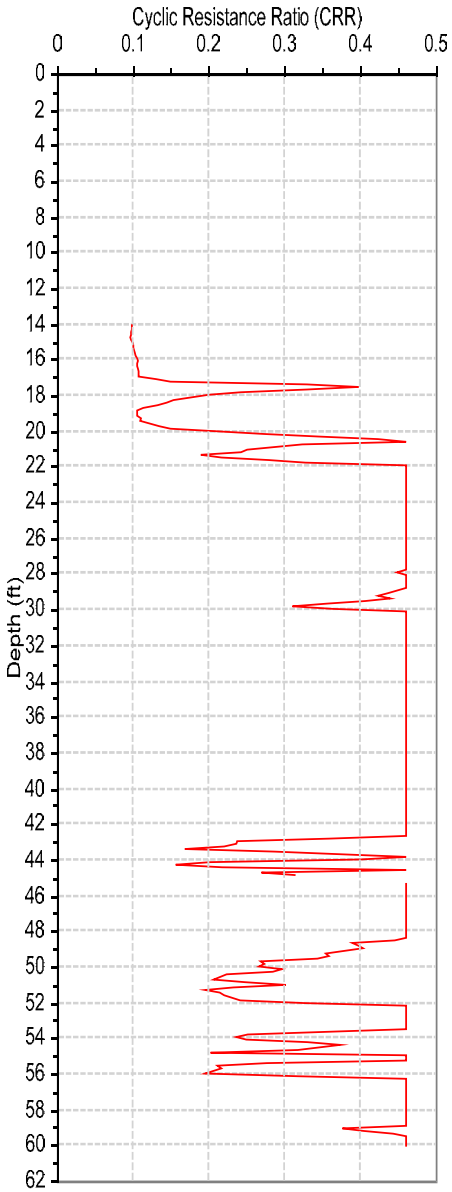
HDR Inc

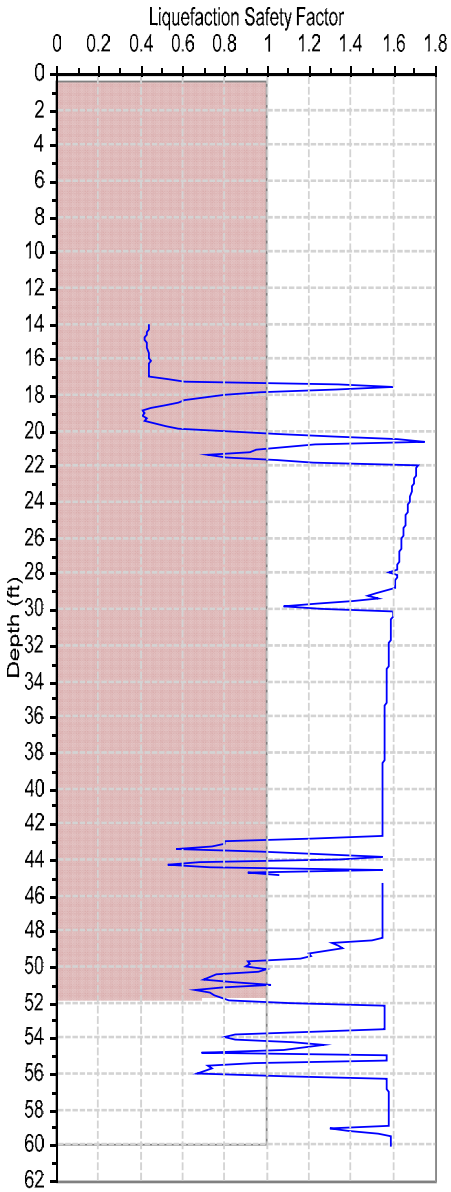
Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-2.cor
Groundwater : 14 ft
Coordinates : n.a.
Calculated By :
Checked By :







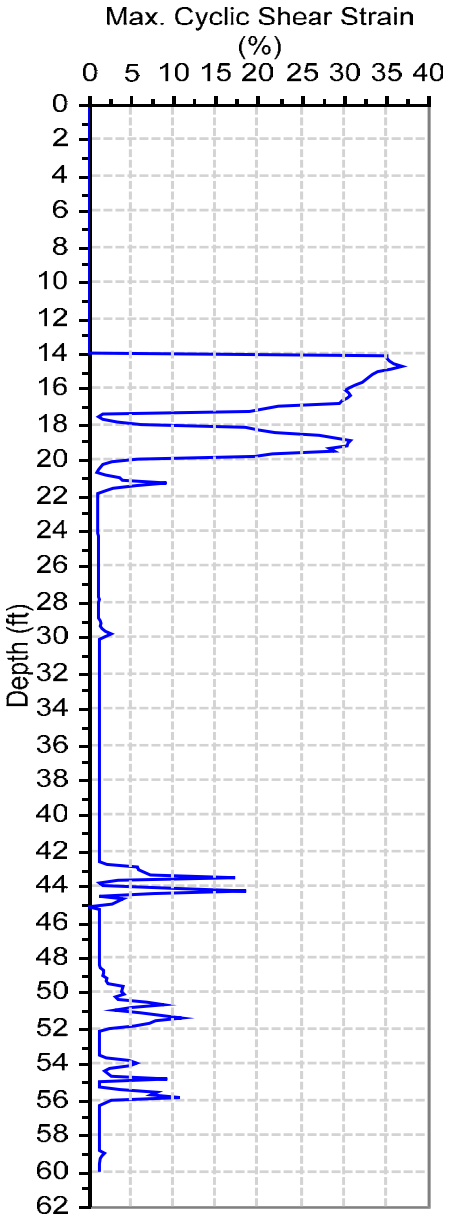


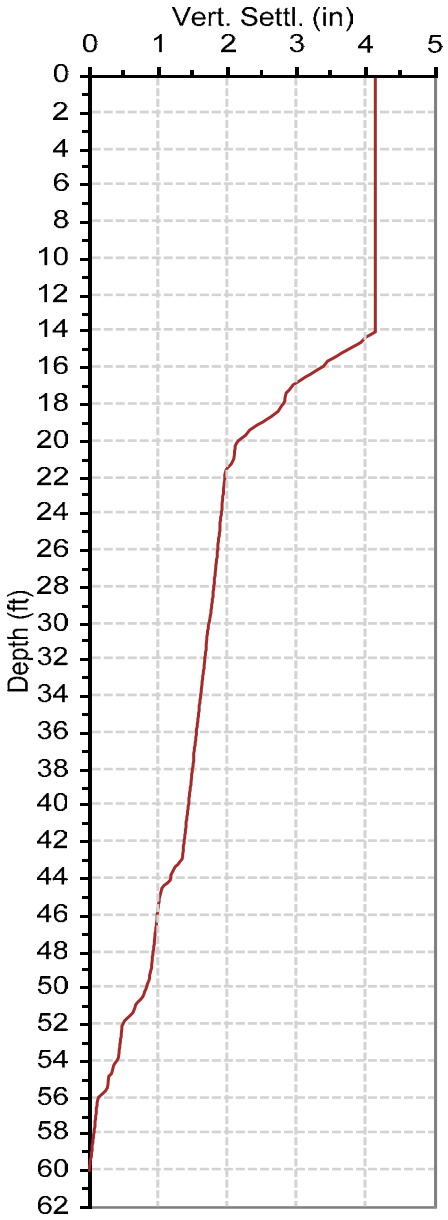
CPT Liquefaction Analysis Results

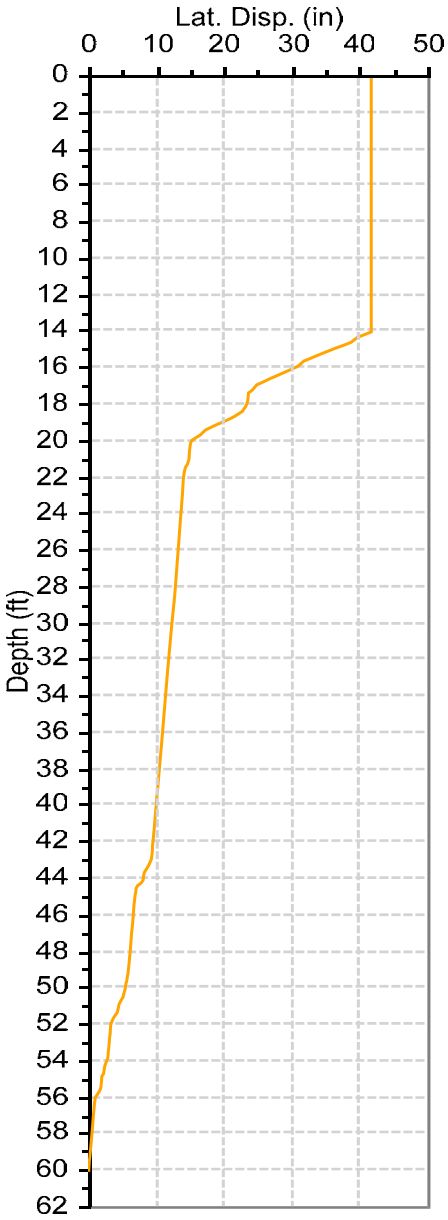
HDR Inc

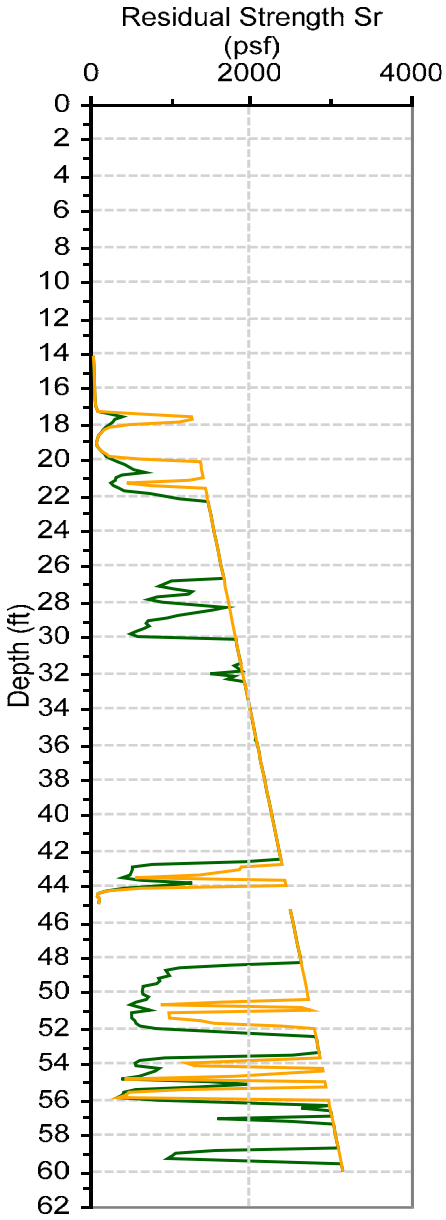
Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-2.cor
Groundwater : 14 ft
Coordinates : n.a.
Calculated By :
Checked By :







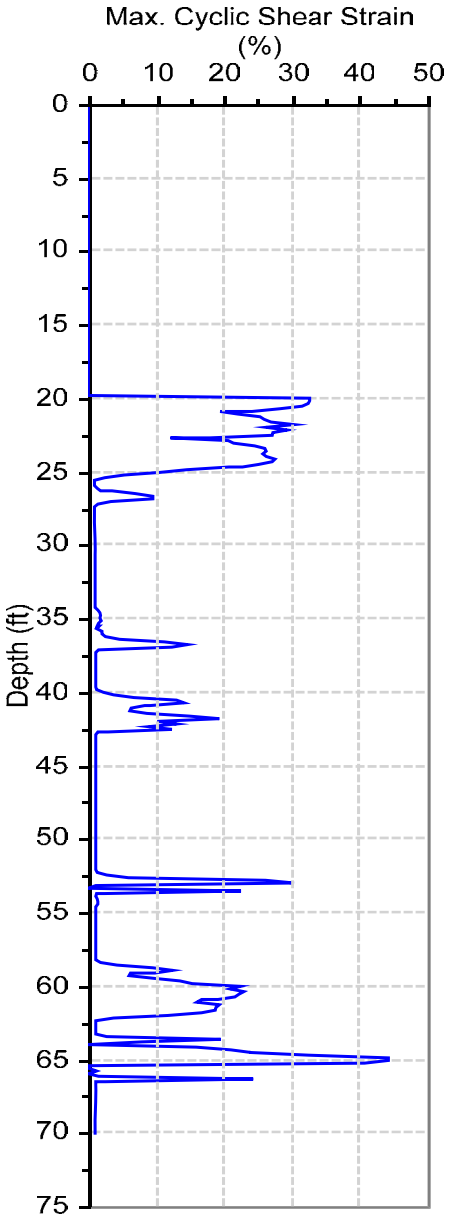


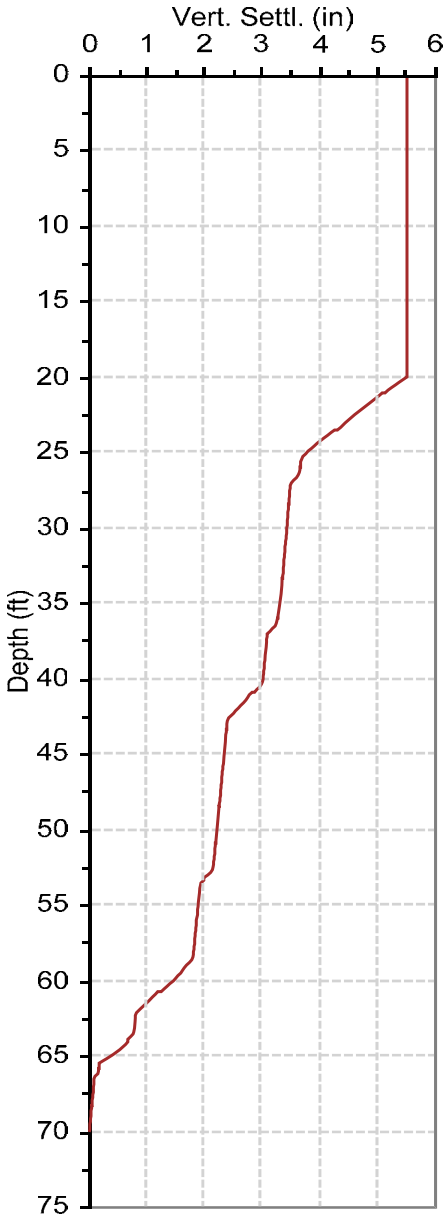
CPT Liquefaction Analysis Results

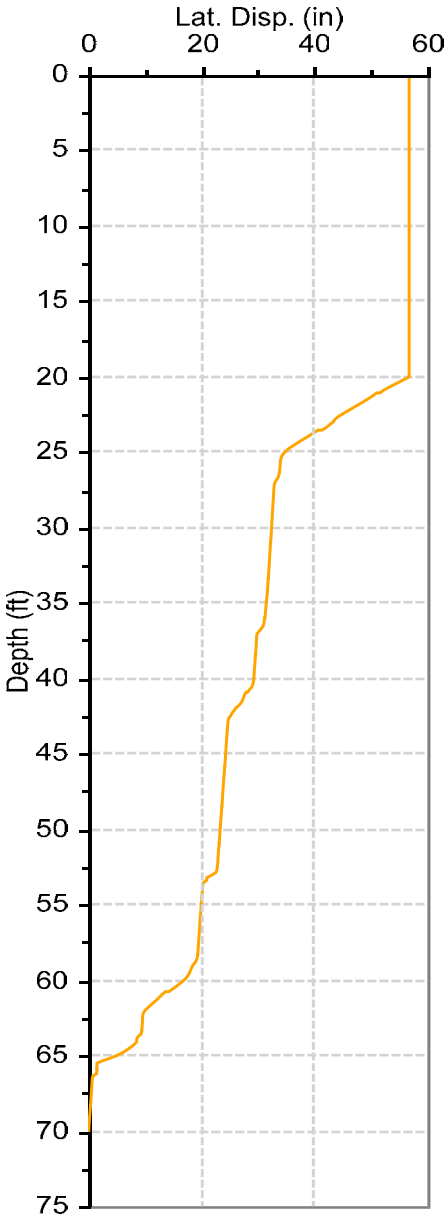
HDR Inc

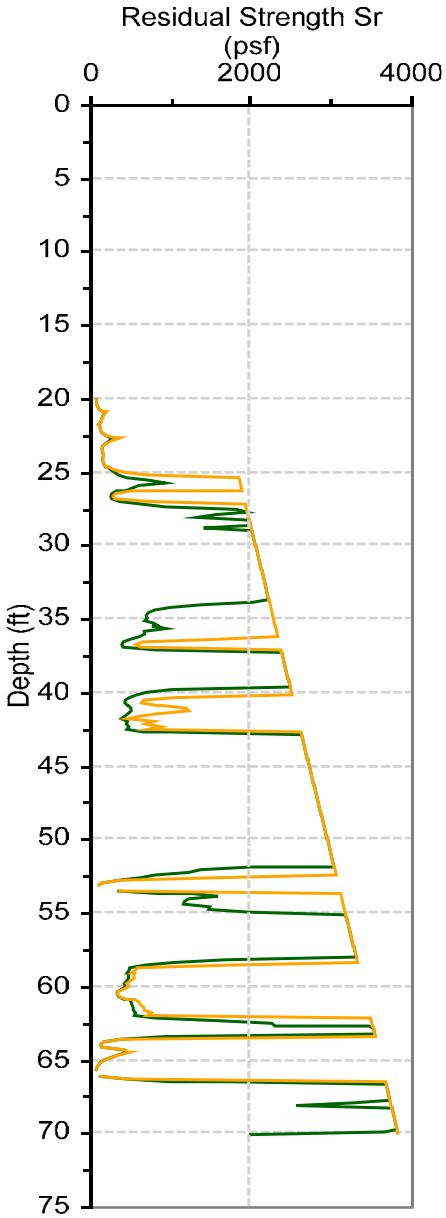
Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-1.cor
Groundwater : 20 ft
Coordinates : n.a.
Calculated By :
Checked By :







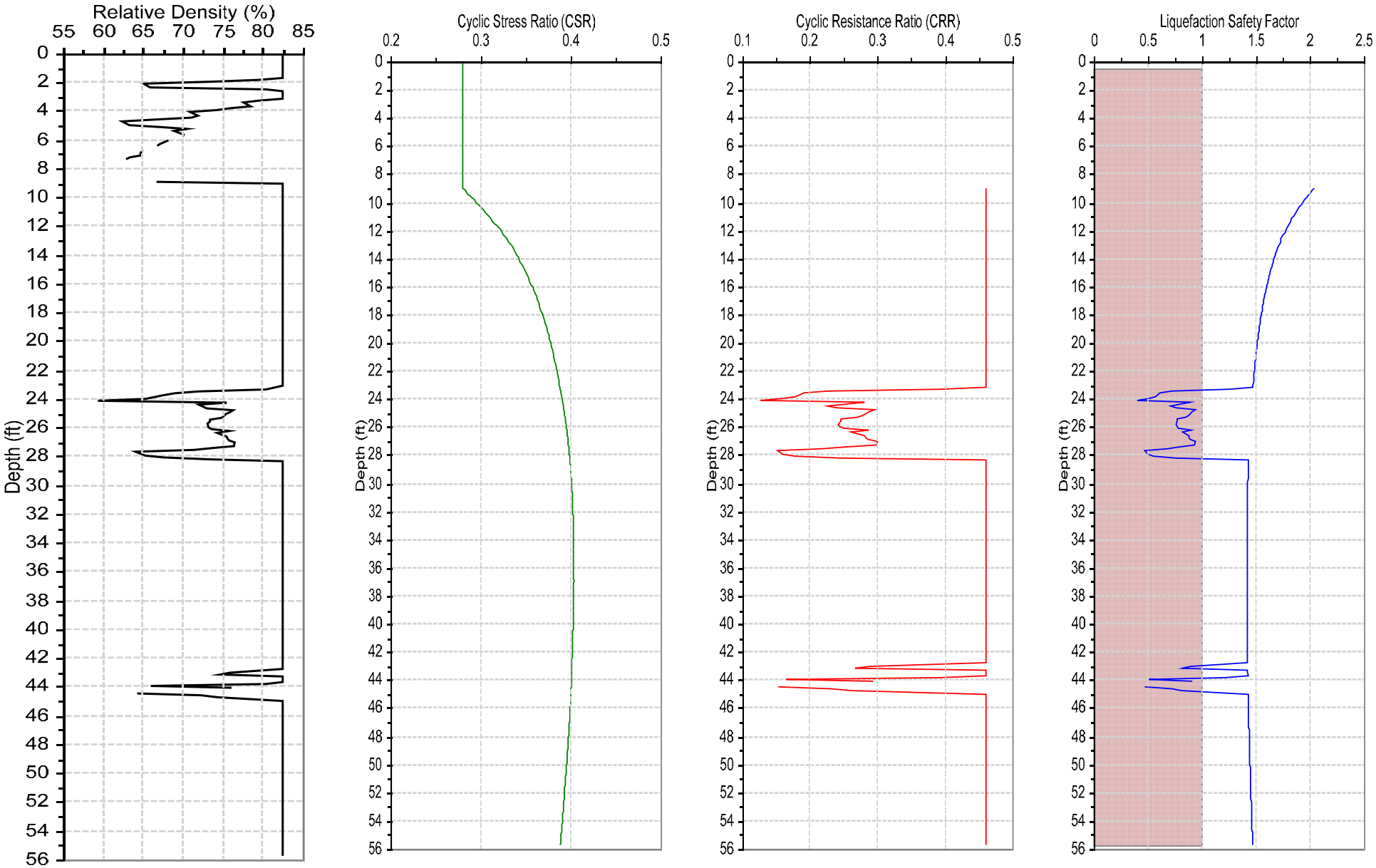


CPT Liquefaction Analysis Results

HDR Inc

Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-3.cor
Groundwater : 9 ft
Coordinates : n.a.
Calculated By :
Checked By :

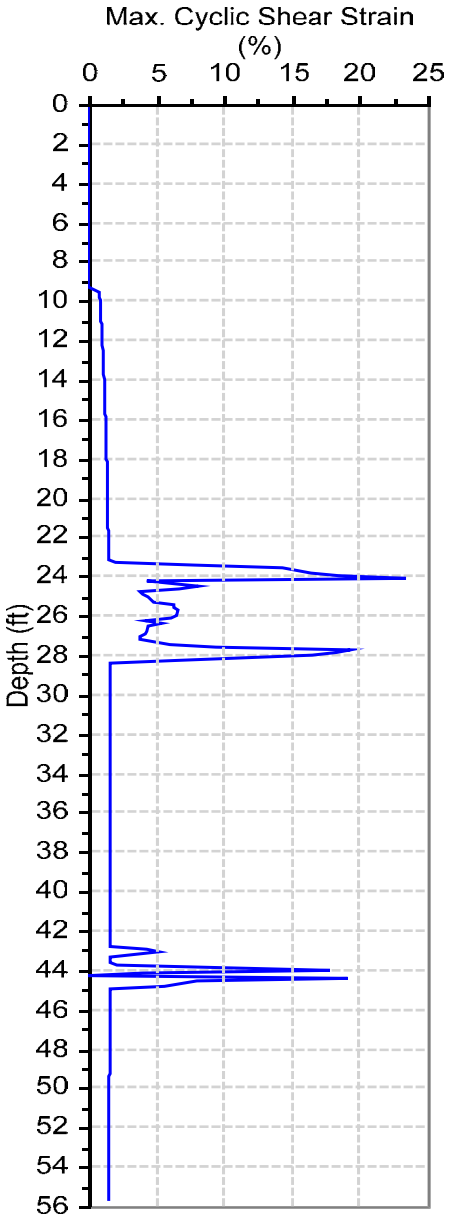


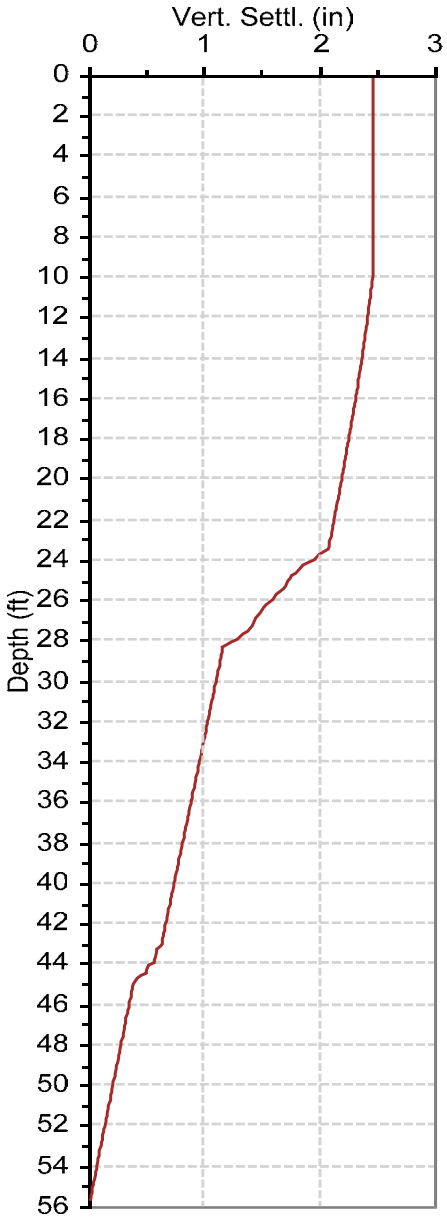
CPT Liquefaction Analysis Results

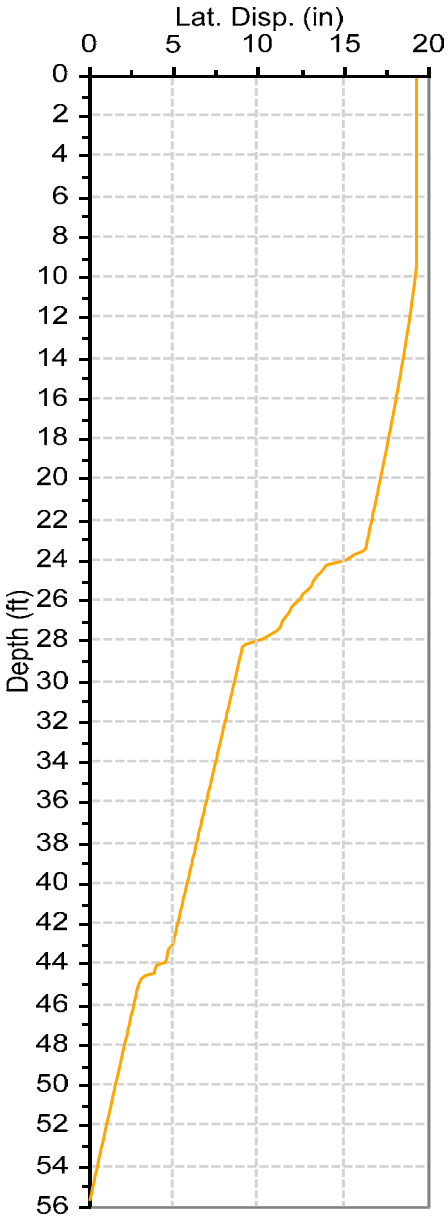
HDR Inc

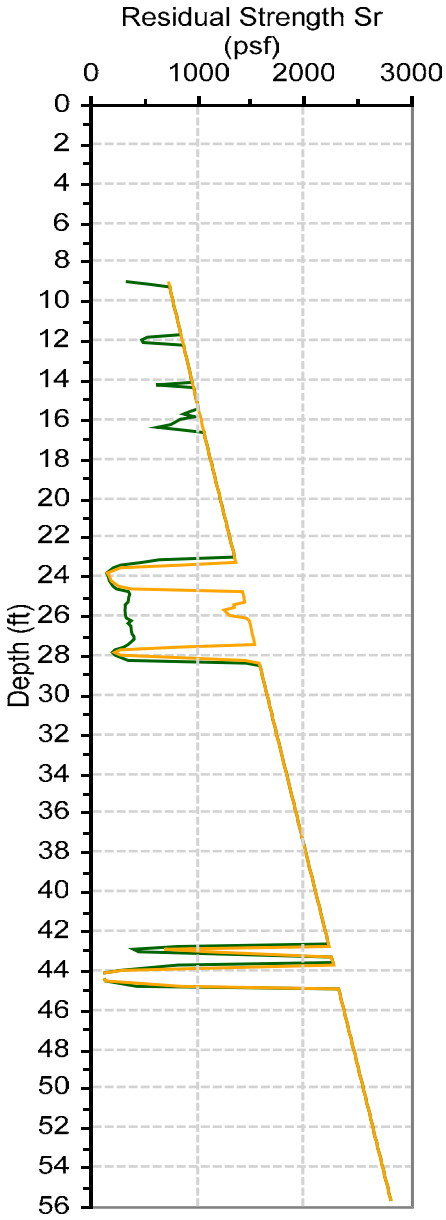
Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-3.cor
Groundwater : 9 ft
Coordinates : n.a.
Calculated By :
Checked By :







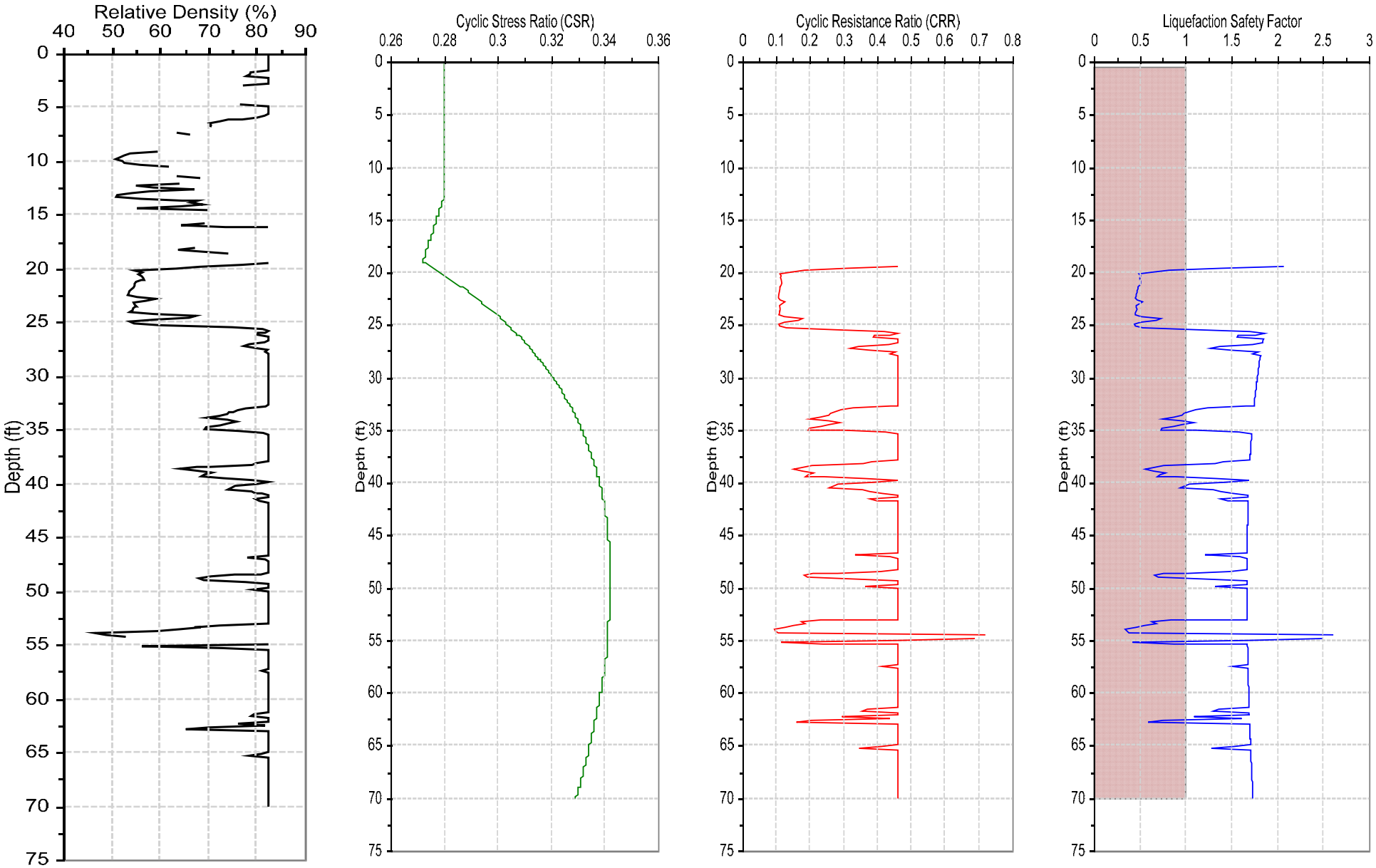


CPT Liquefaction Analysis Results

HDR Inc

Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-4.cor
Groundwater : 19 ft
Coordinates : n.a.
Calculated By :
Checked By :

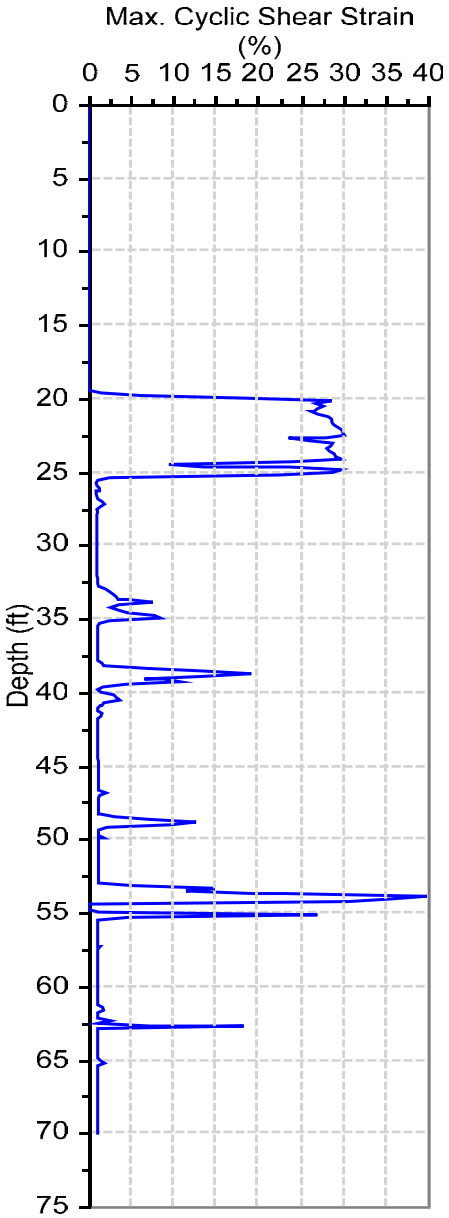


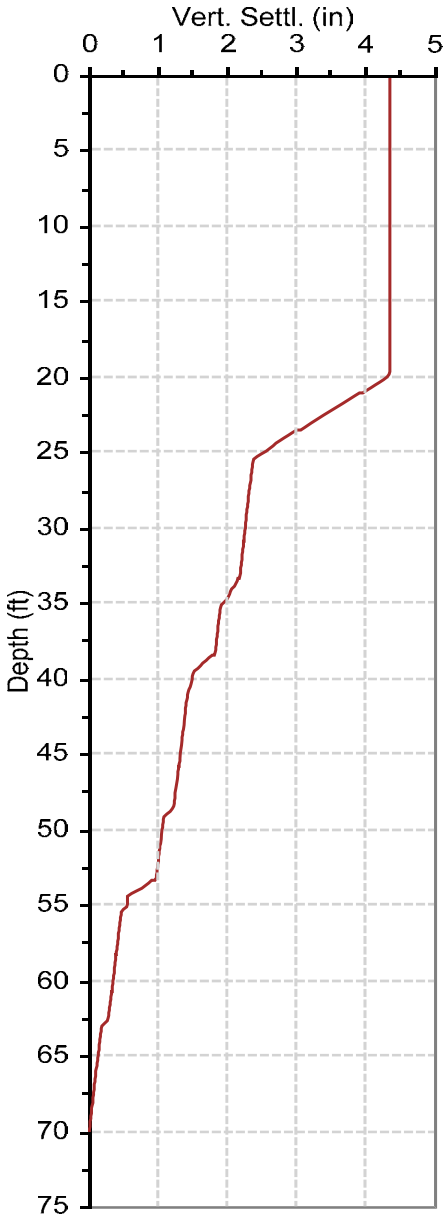
CPT Liquefaction Analysis Results

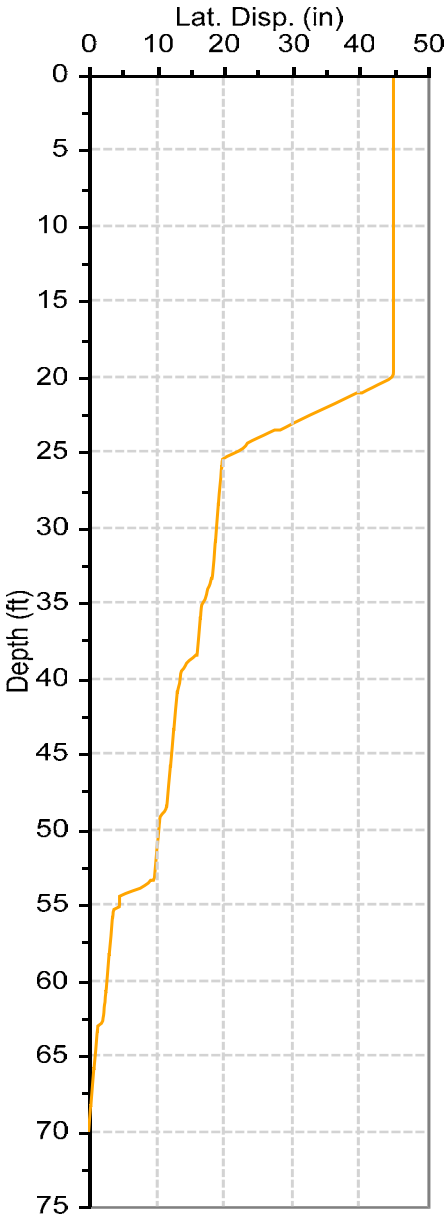
HDR Inc

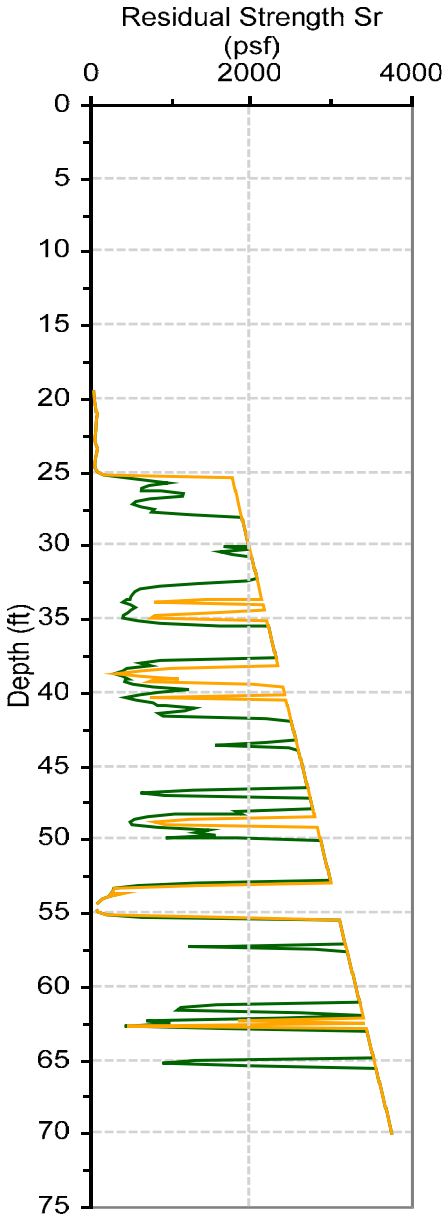
Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-4.cor
Groundwater : 19 ft
Coordinates : n.a.
Calculated By :
Checked By :







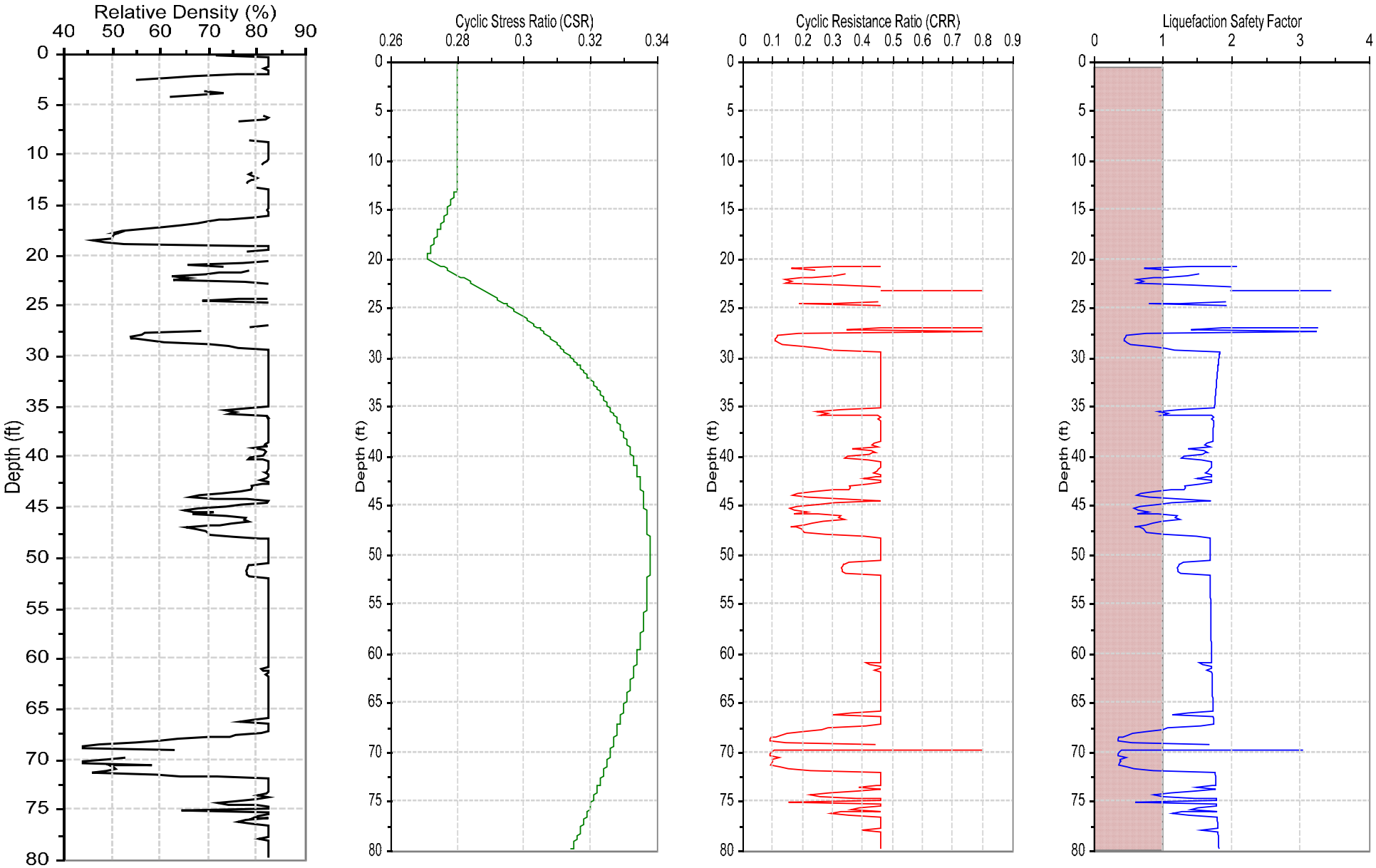


CPT Liquefaction Analysis Results

HDR Inc

Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-5.cor
Groundwater : 20 ft
Coordinates : n.a.
Calculated By :
Checked By :

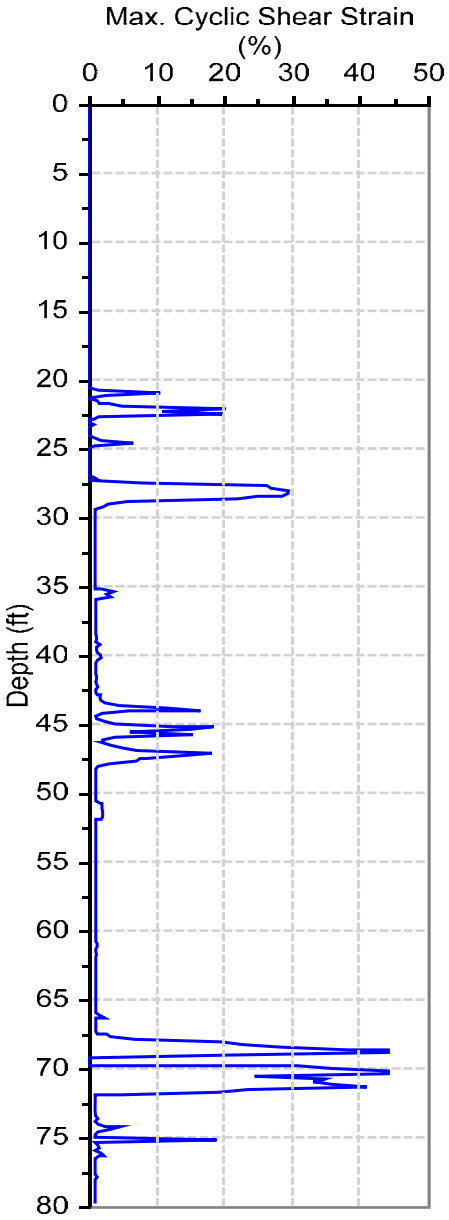


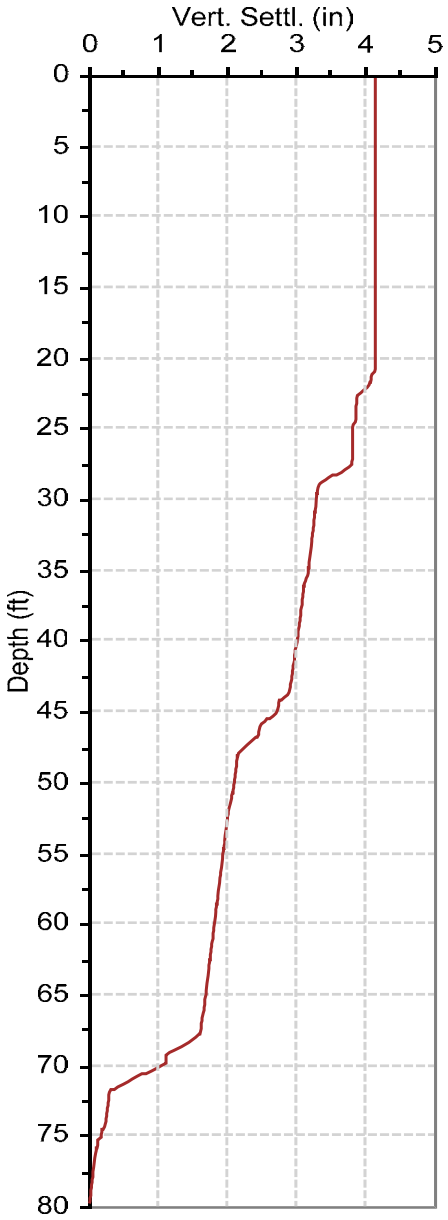
CPT Liquefaction Analysis Results

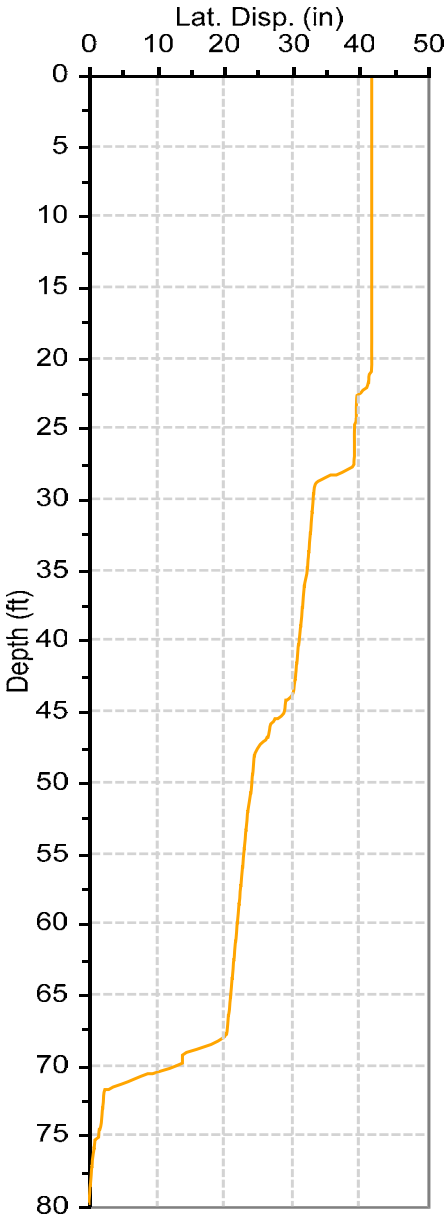
HDR Inc

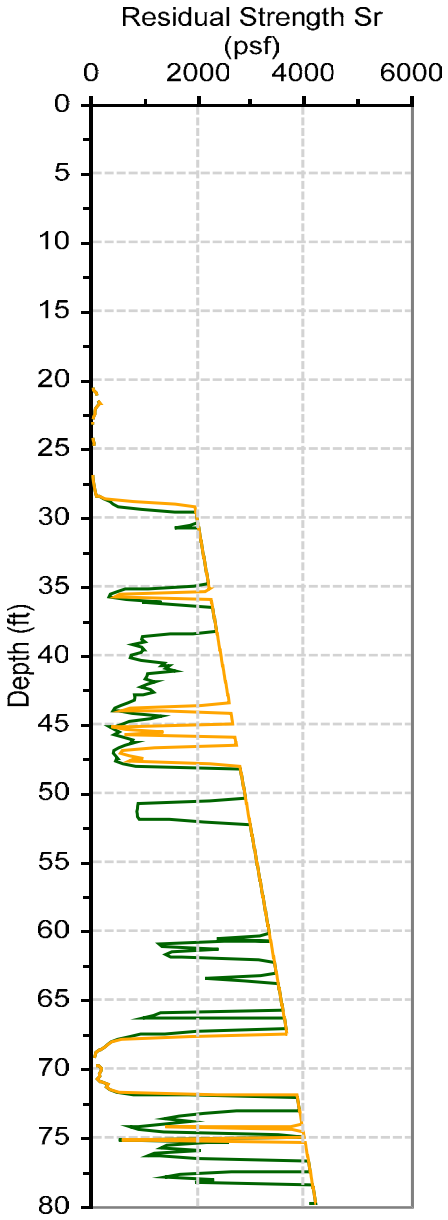
Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-5.cor
Groundwater : 20 ft
Coordinates : n.a.
Calculated By :
Checked By :







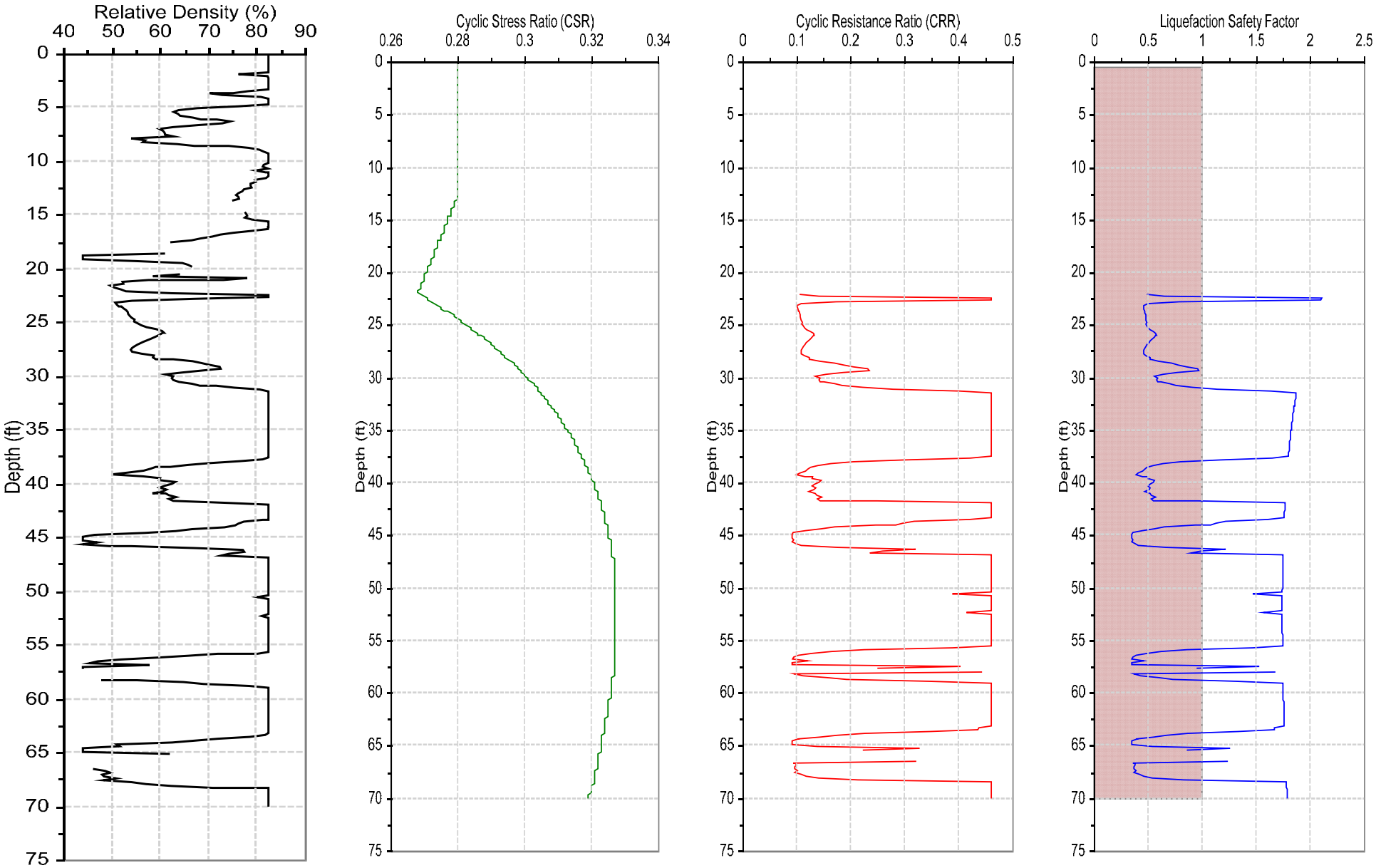


CPT Liquefaction Analysis Results

HDR Inc

Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-6.cor
Groundwater : 22 ft
Coordinates : n.a.
Calculated By :
Checked By :

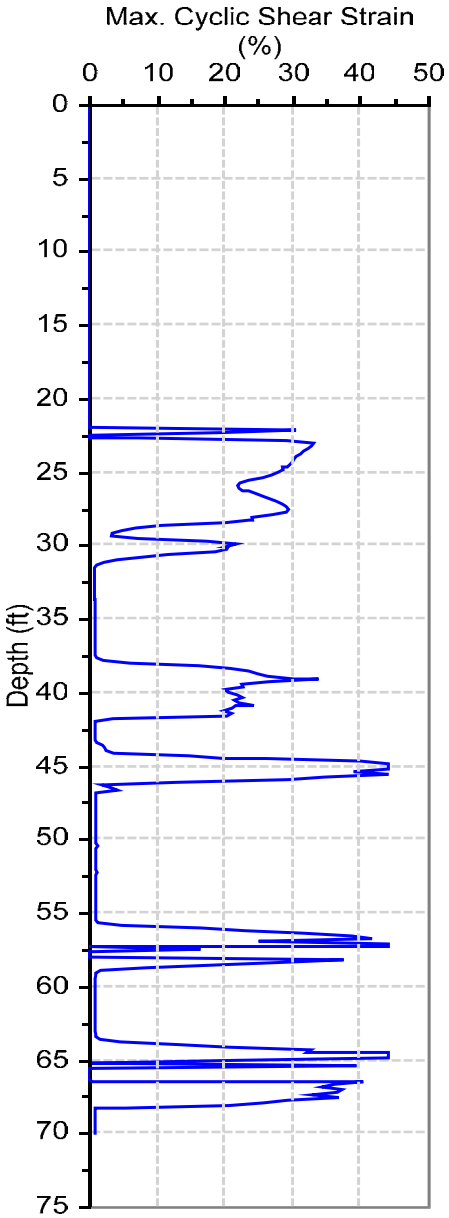


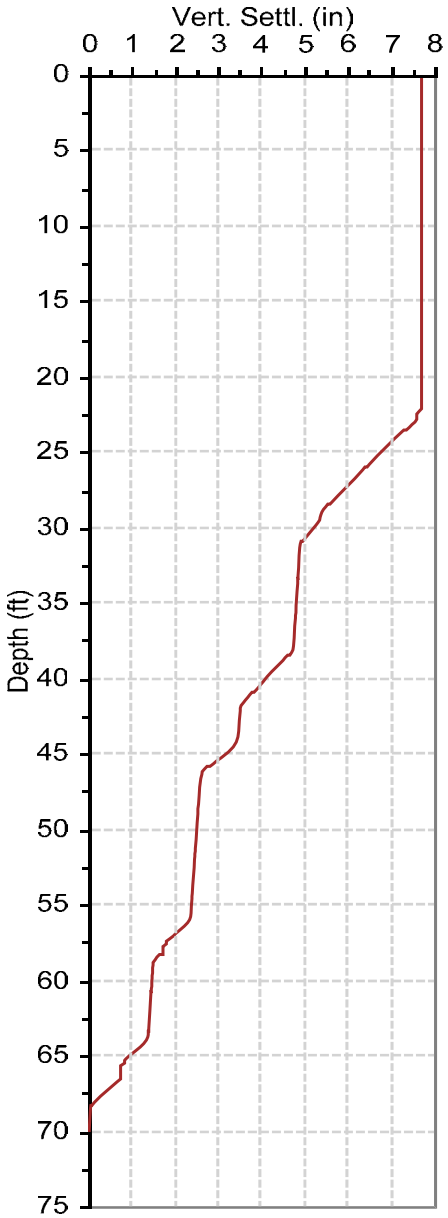
CPT Liquefaction Analysis Results

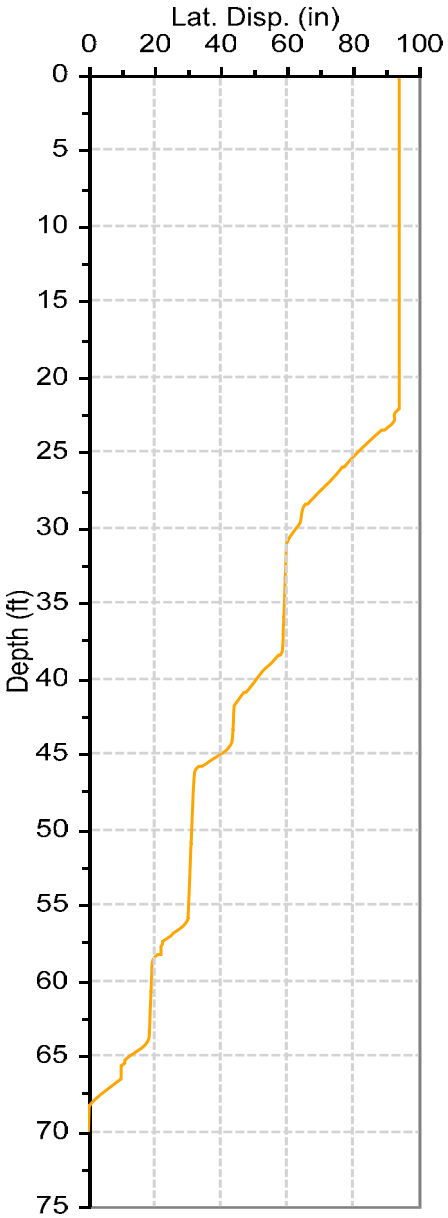
HDR Inc

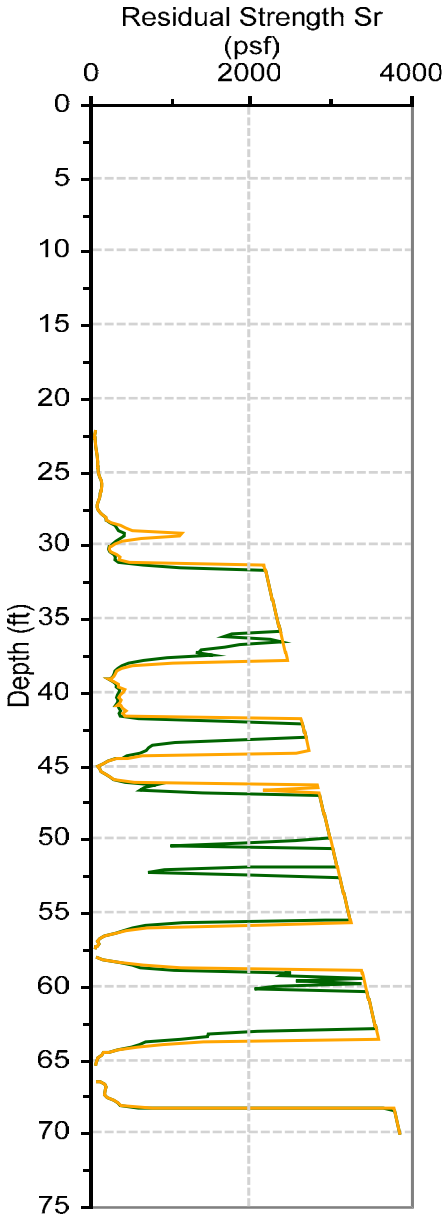
Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-6.cor
Groundwater : 22 ft
Coordinates : n.a.
Calculated By :
Checked By :







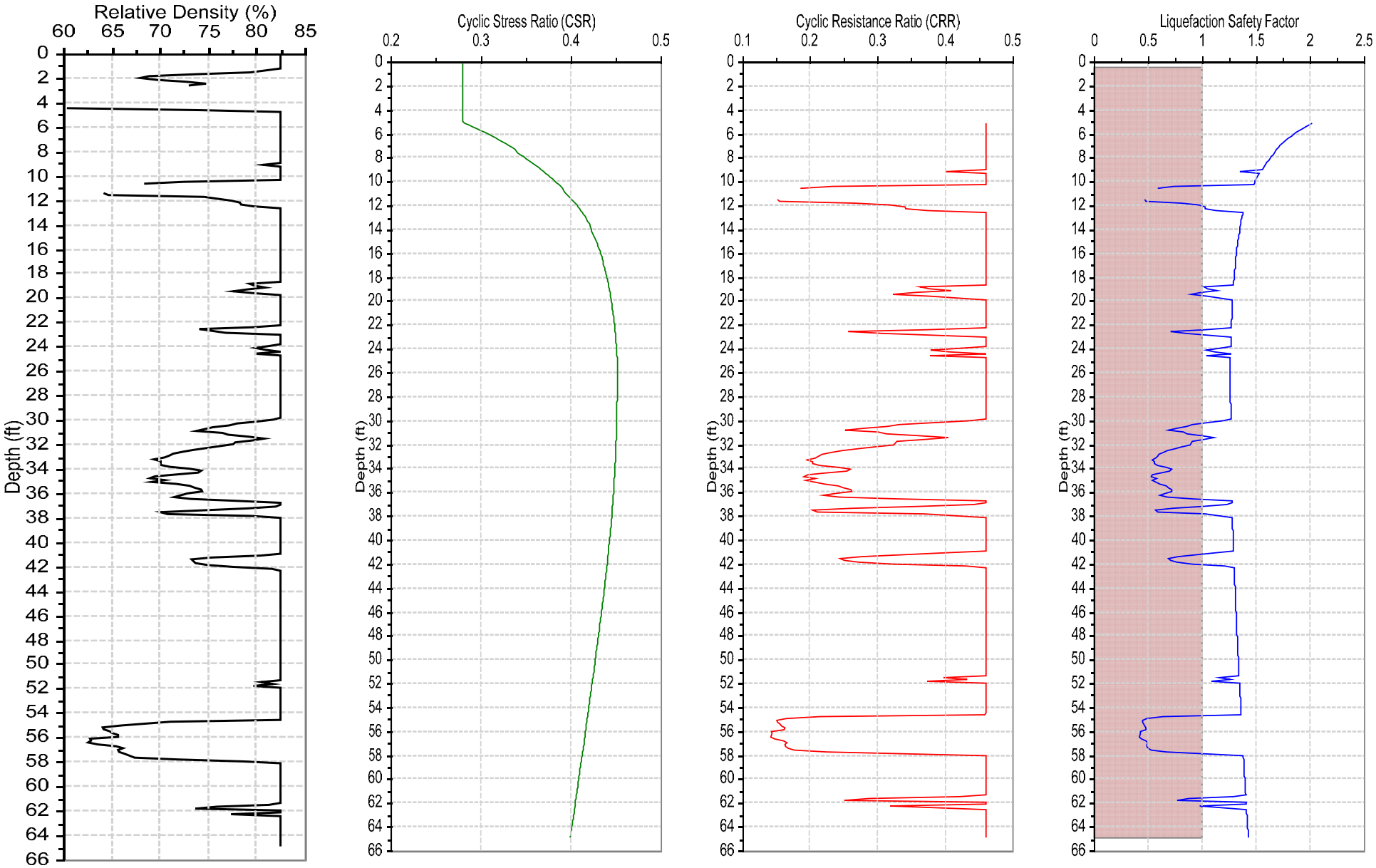


CPT Liquefaction Analysis Results

HDR Inc

Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-7.cor
Groundwater : 5 ft
Coordinates : n.a.
Calculated By :
Checked By :

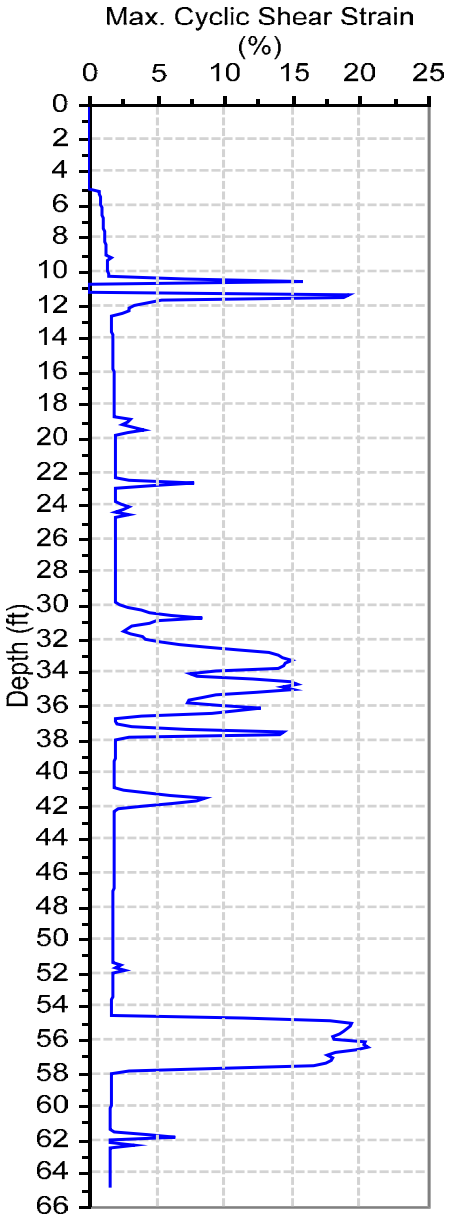


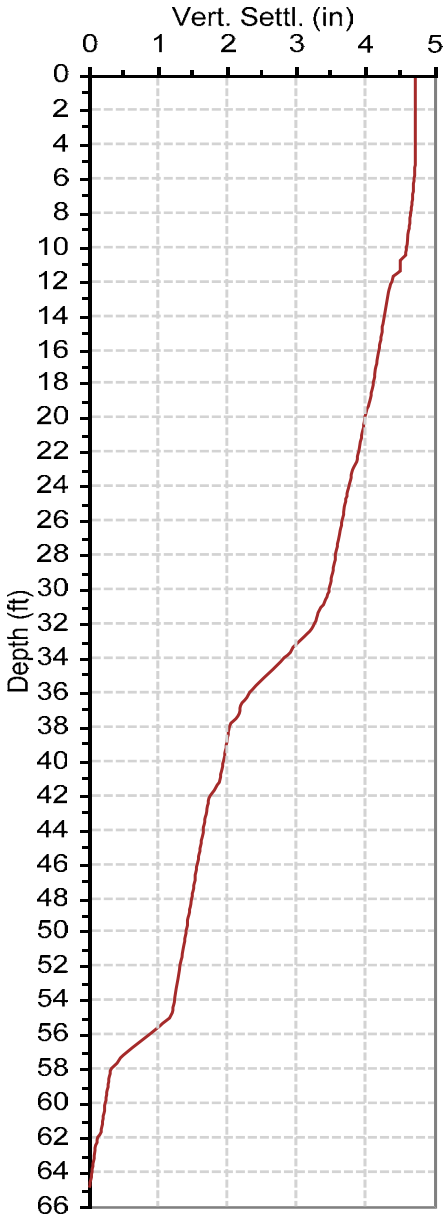
CPT Liquefaction Analysis Results

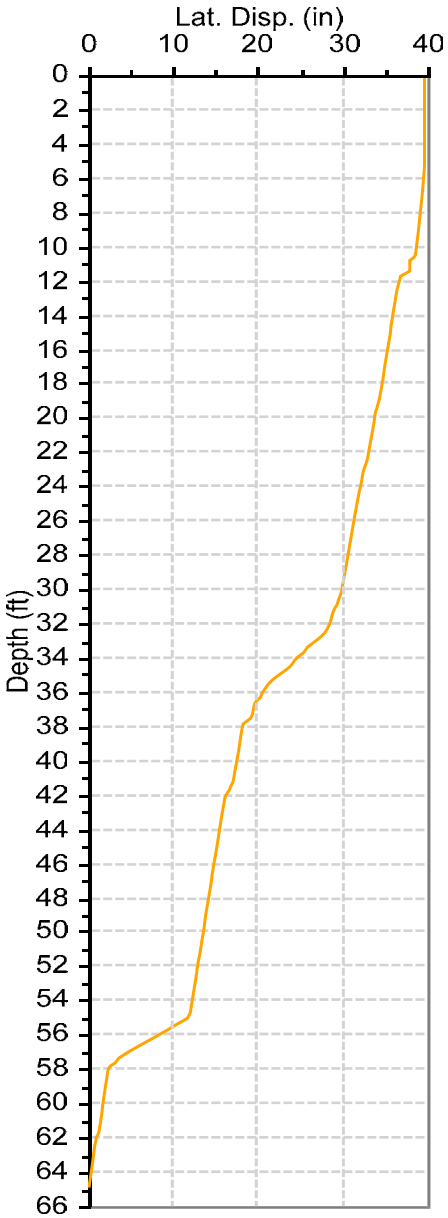
HDR Inc

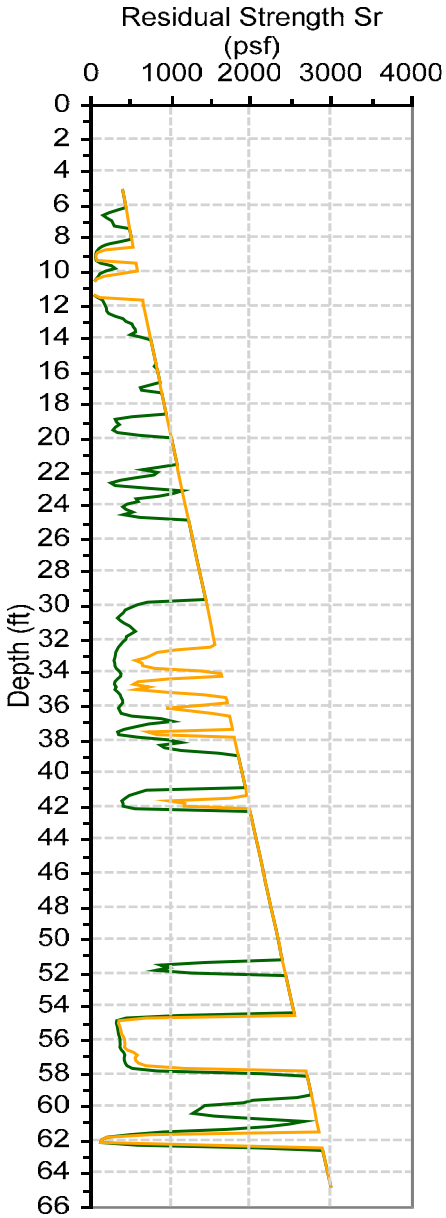
Job Title : Mirabel Fish Passage & Seismic Improvements
Job Code : 164479
Client : SCWA
Address : Westside Rd, Forestville

Borehole : 072CPT-7.cor
Groundwater : 5 ft
Coordinates : n.a.
Calculated By :
Checked By :









Appendix F – Slope Stability Analysis

Slope stability analysis was performed using SlopeW software program to analyze idealized conditions based on borings and CPTs. Two cross sections were developed and summarized into SlopeW models. These models were used to analyze current conditions and develop ground improvement strength parameters associated with an expectable factor of safeties. The following models are presented in Appendix F:

Figure F-1: Models Cross Section A-A' - existing conditions along the Russian River slope using river elevation of +45 feet and block failure mechanism.

Figure F-2: Models Cross Section A-A' - existing conditions along the Russian River slope using river elevation of +32 feet and block failure mechanism.

Figure F-3: Models Cross Section A-A' - existing conditions along the Russian River slope using river elevation of +45 feet and circular failure mechanism using Spencer's Method.

Figure F-4: Models Cross Section A-A' - existing conditions along the Russian River slope using river elevation of +32 feet and circular failure mechanism using Spencer's Method.

Figure F-5: Models Cross Section A-A' - existing conditions along the desilting pond slope using river elevation of +45 feet and circular failure mechanism using Spencer's Method.

Figure F-6: Models Cross Section A-A' - remediated conditions along the Russian River slope using river elevation of +45 feet and block failure mechanism.

Figure F-7: Models Cross Section A-A' - remediated conditions along the Russian River slope using river elevation of +32 feet and block failure mechanism.

Figure F-8: Models Cross Section A-A' - remediated conditions along the desilting pond slope using river elevation of +45 feet and circular failure mechanism using Spencer's Method.

Figure F-9: Models Cross Section A-A' - Horizontal Seismic Loading conditions along the Russian River slope using river elevation of +32 feet and block failure mechanism.

Figure F-10: Models Cross Section A-A' - Horizontal Seismic Loading conditions along the desilting pond slope using river elevation of +45 feet and circular failure mechanism using Spencer's Method.

Figure F-11: Models Cross Section B-B' - existing conditions along the Russian River slope using river elevation of +32 feet and block failure mechanism.

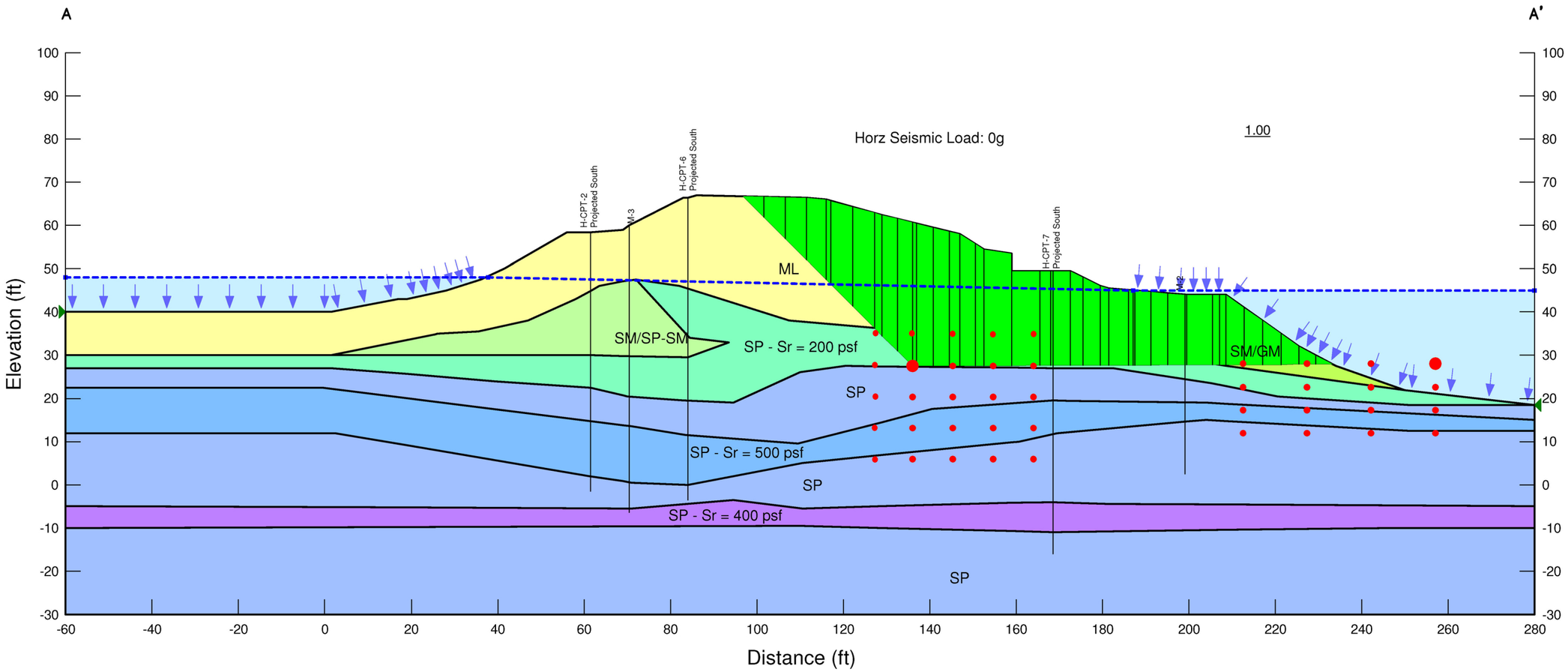
Figure F-12: Models Cross Section B-B' - existing conditions along the desilting pond slope using river elevation of +45 feet and circular failure mechanism using Spencer's Method.

Figure F-13: Models Cross Section B-B' - remediated conditions along the Russian River slope using river elevation of +32 feet and block failure mechanism.

Figure F-14: Models Cross Section B-B' - remediated conditions along the desilting pond slope using river elevation of +45 feet and circular failure mechanism using Spencer's Method.

Figure F-15: Models Cross Section B-B' - Horizontal Seismic Loading conditions along the Russian River slope using river elevation of +32 feet and block failure mechanism.

Figure F-16: Models Cross Section B-B' - Horizontal Seismic Loading conditions along the desilting pond slope using river elevation of +45 feet and circular failure mechanism using Spencer's Method.



SECTION A–A'

BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

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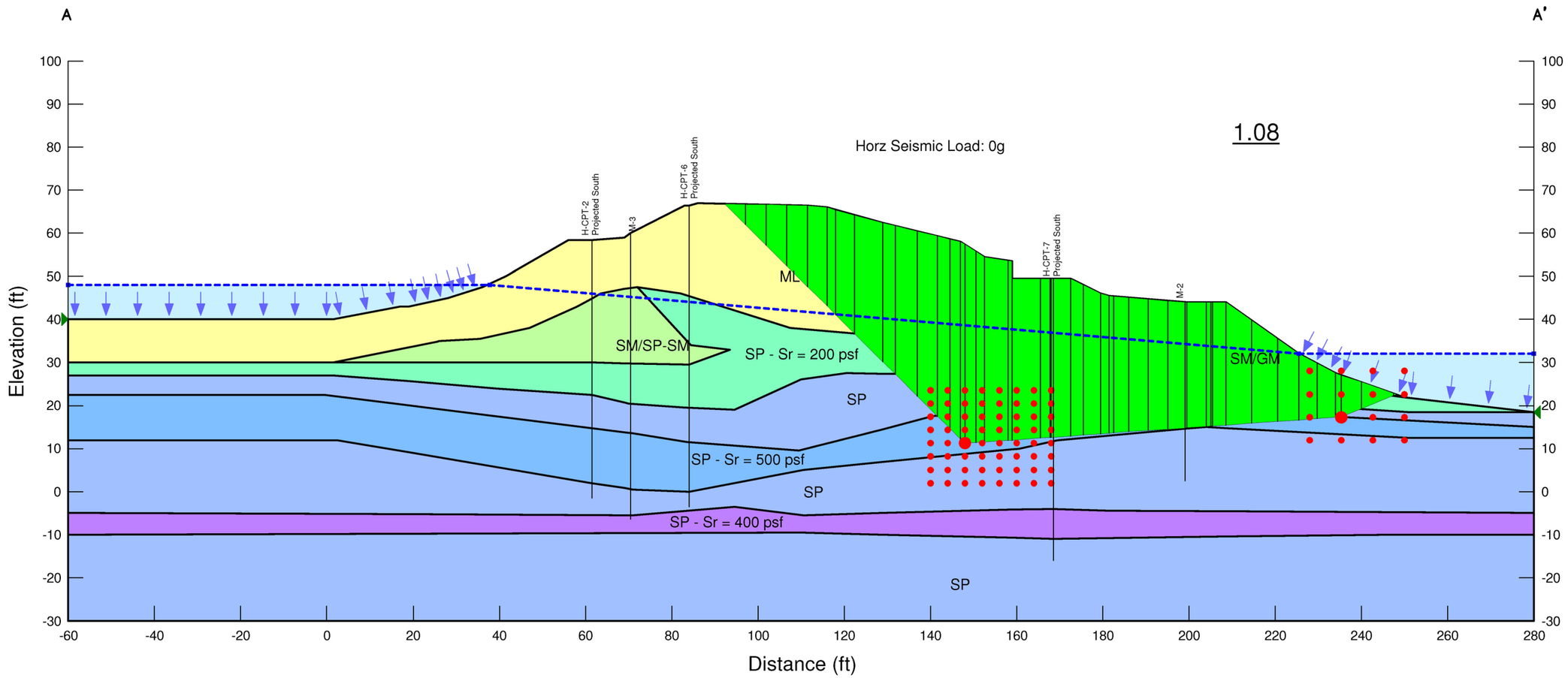

SECTION A–A'

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HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

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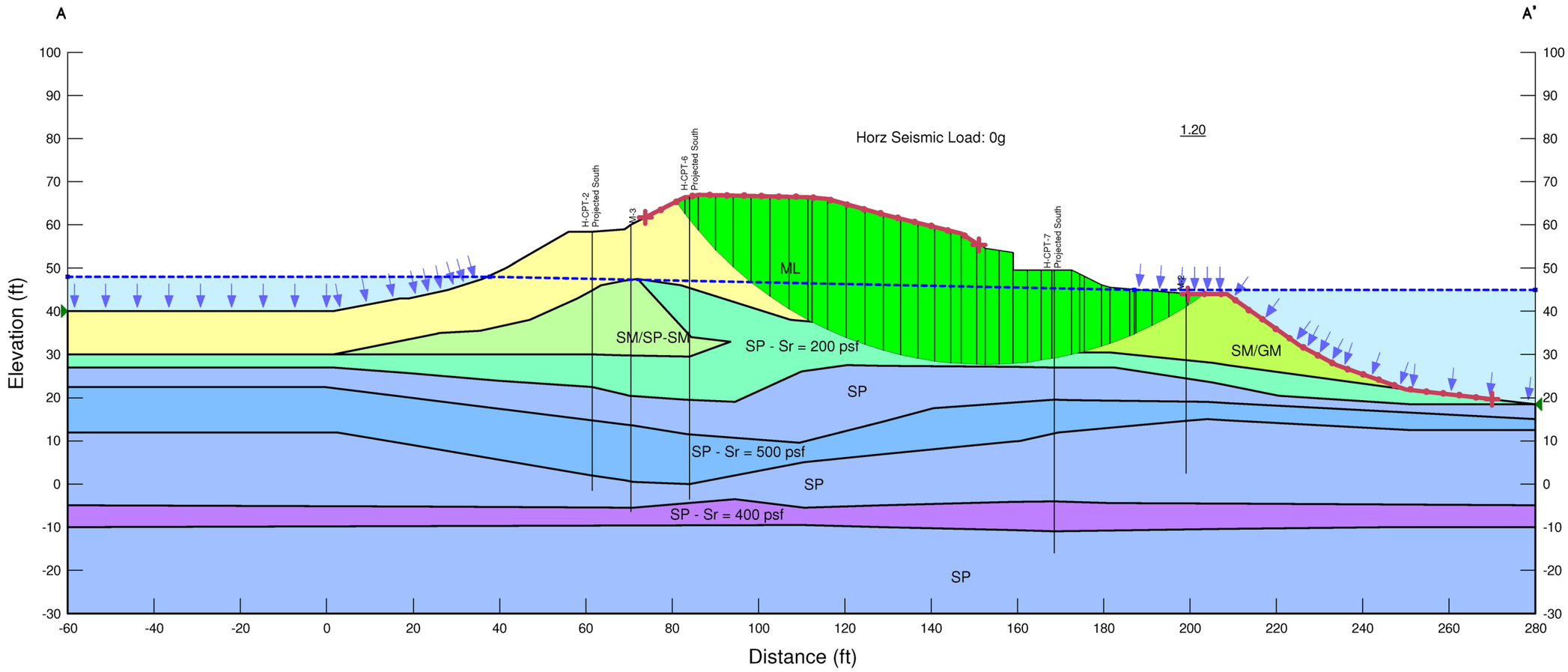

SECTION A–A'

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HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

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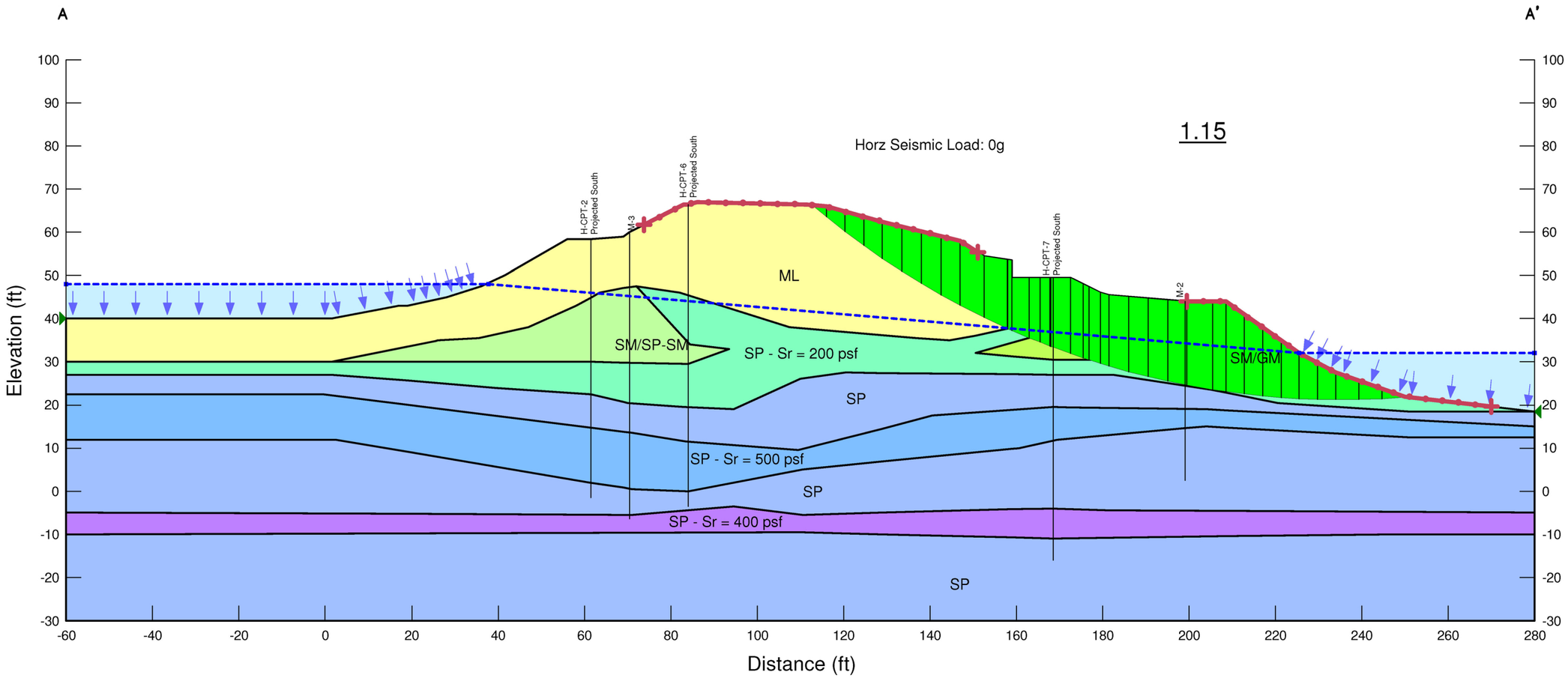

SECTION A–A'

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HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

SCALE: AS NOTED	DATE: 7.8.2013
DRAWN: PVM	
REVIEWED:	

MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT

SECTION A-A' RESIDUAL SLOPE STABILITY

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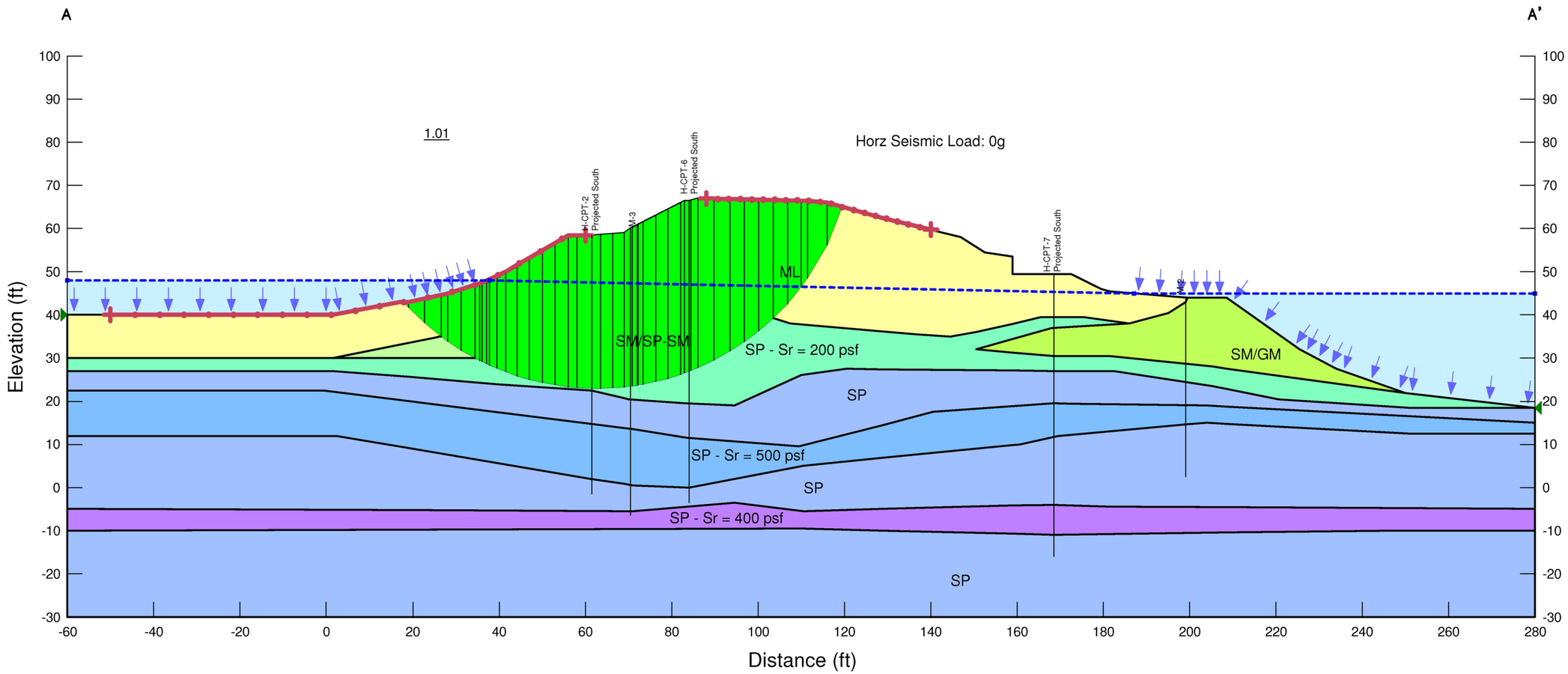

SECTION A–A'

BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

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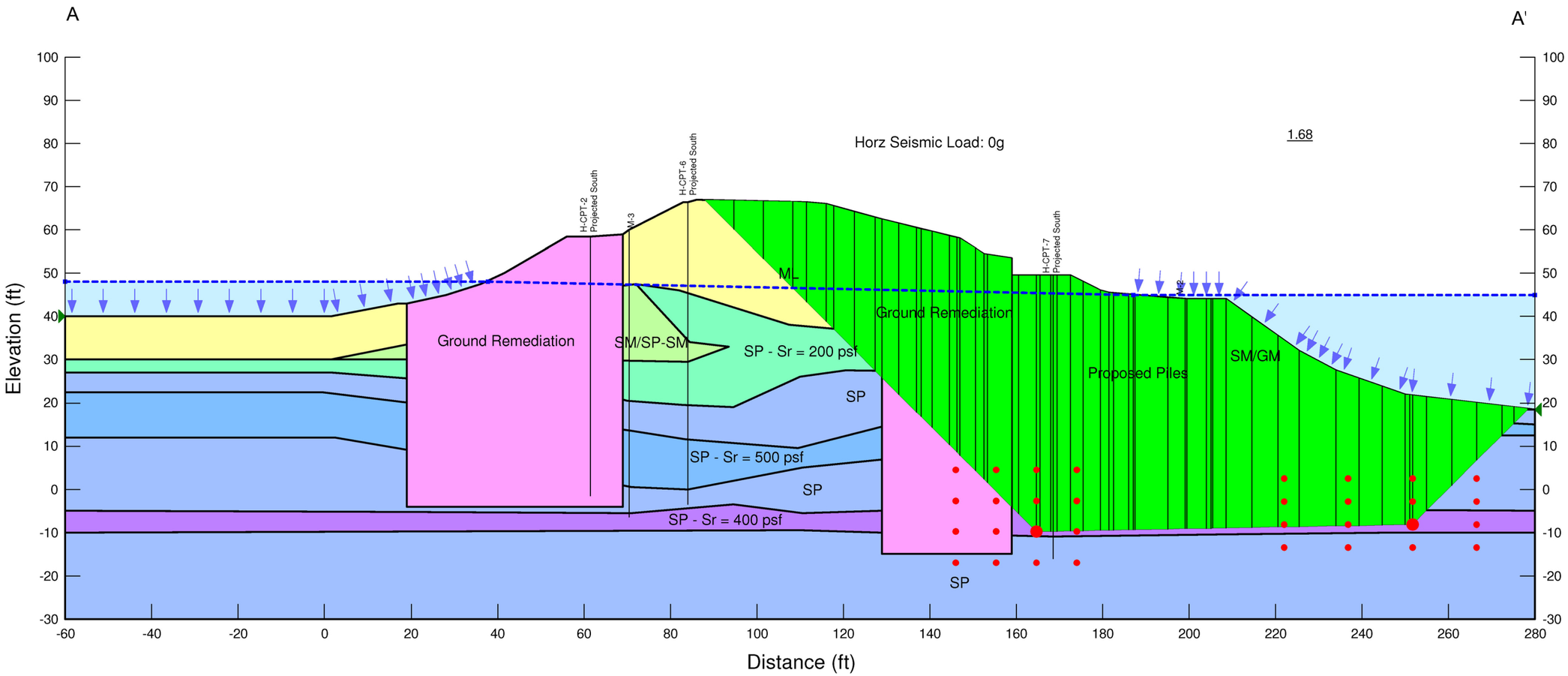

SECTION A-A'

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HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

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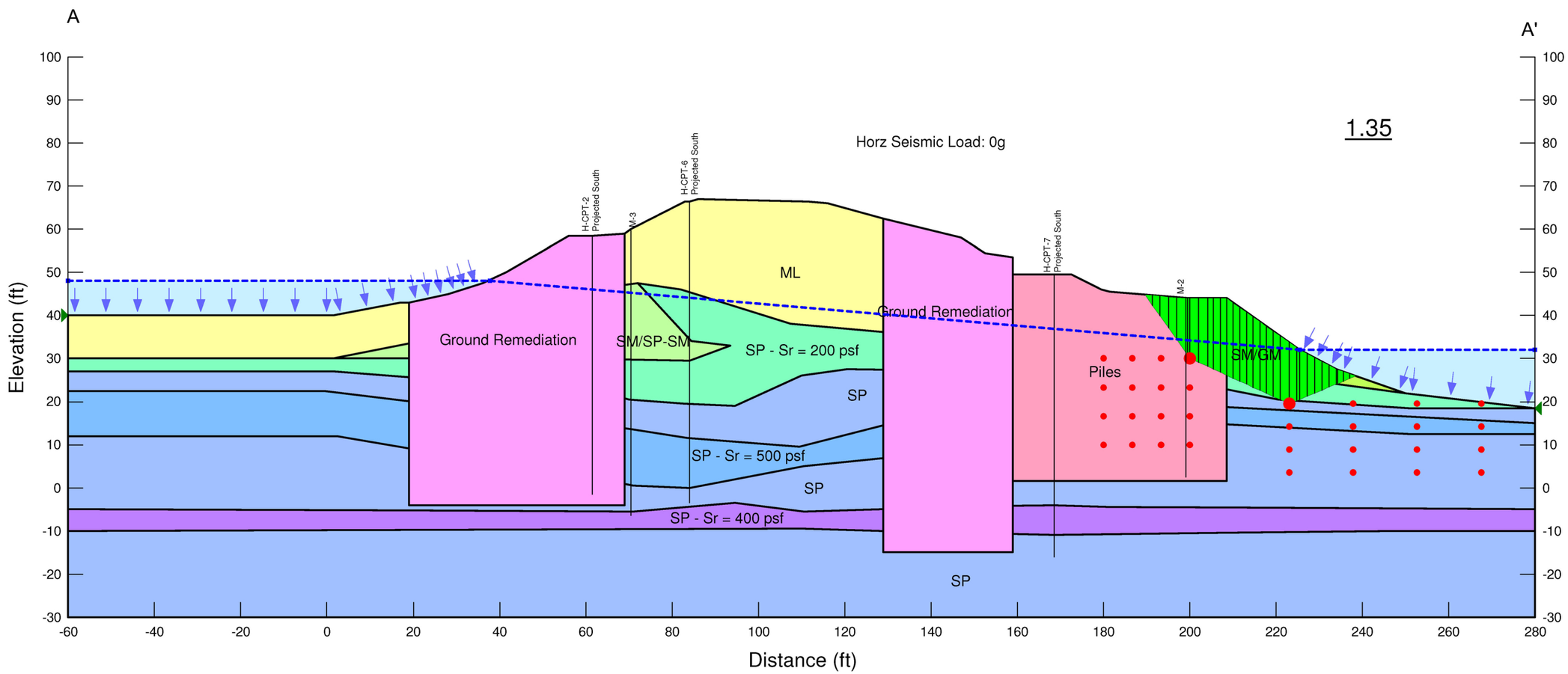

SECTION A-A'

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HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

SCALE: AS NOTED	DATE: 7.8.2013	MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT
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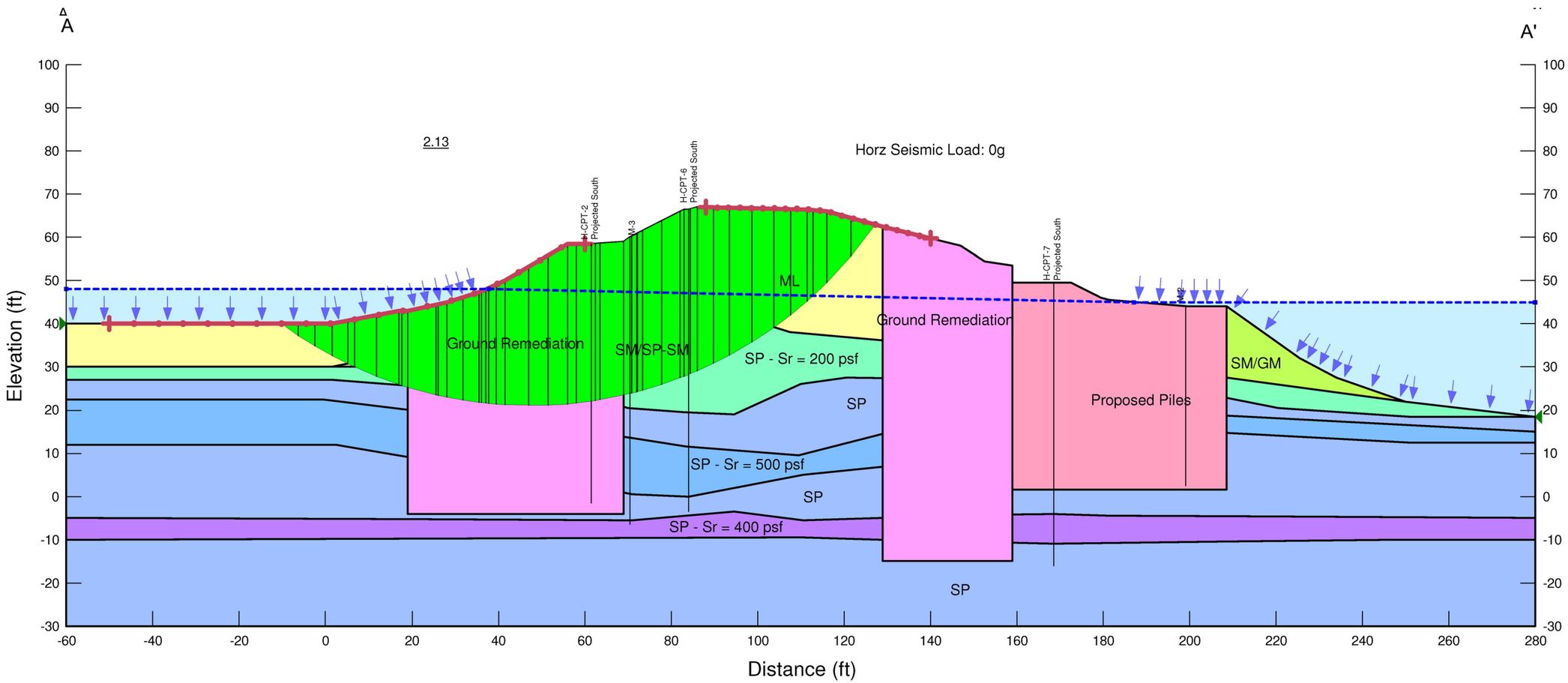

SECTION A-A'

BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

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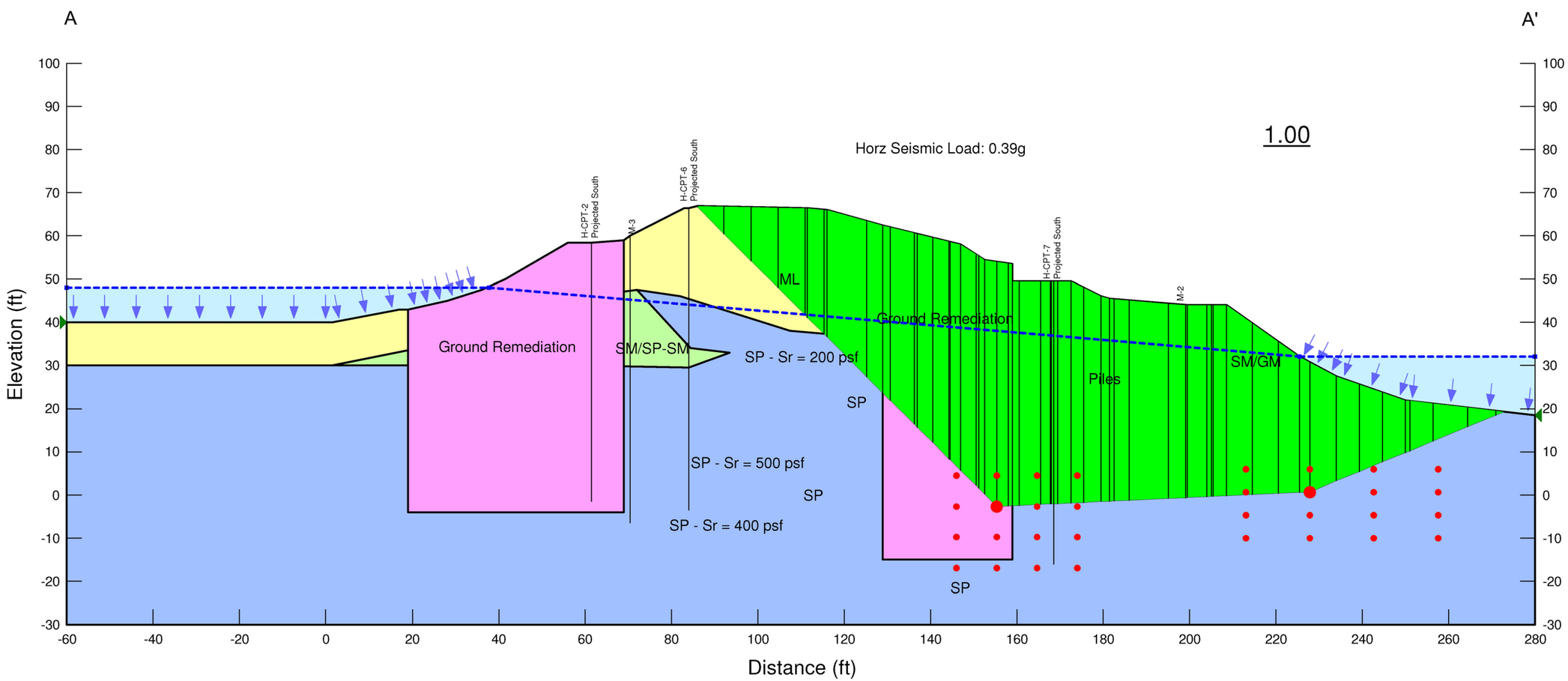

SECTION A-A'

BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

SCALE: AS NOTED
DATE: 7.8.2013
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REVIEWED:

MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT

SECTION A-A' RESIDUAL SLOPE STABILITY

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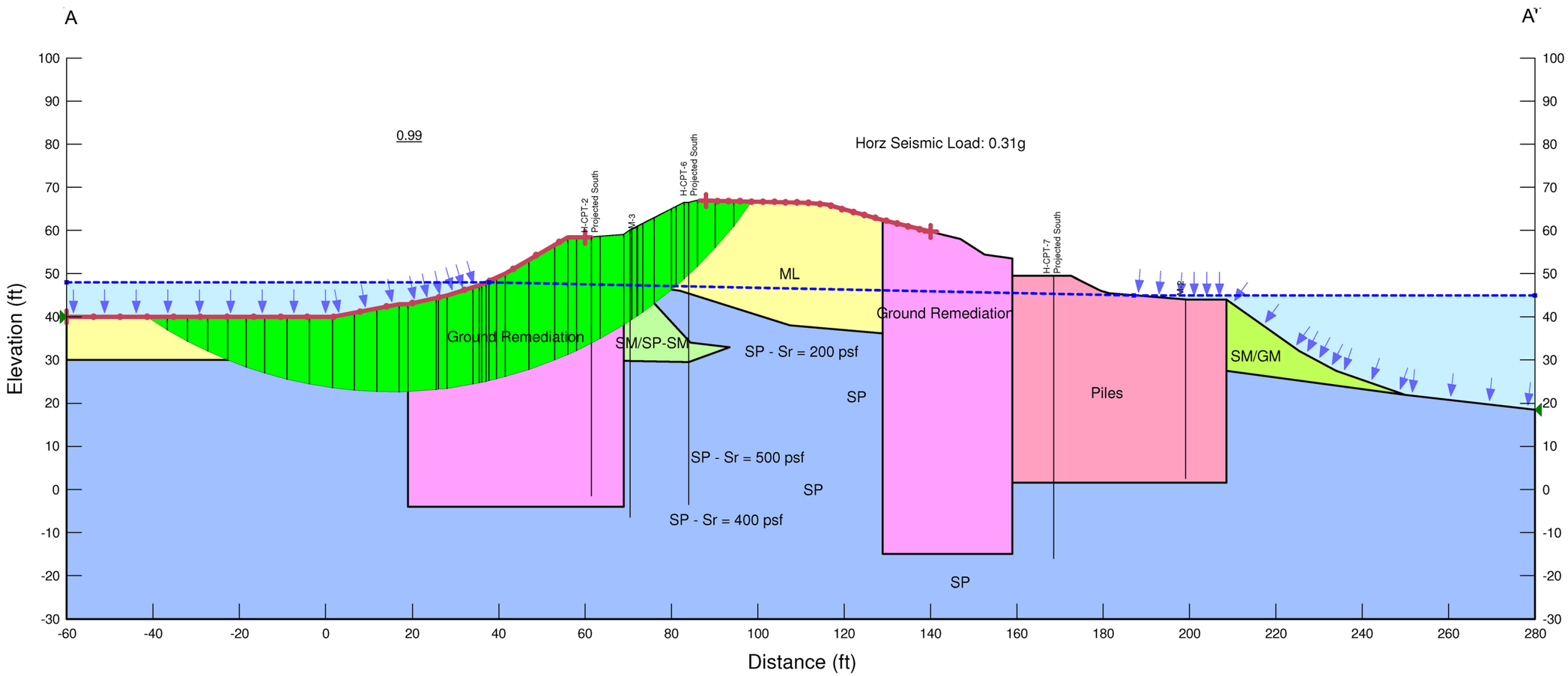

SECTION A-A'

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HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

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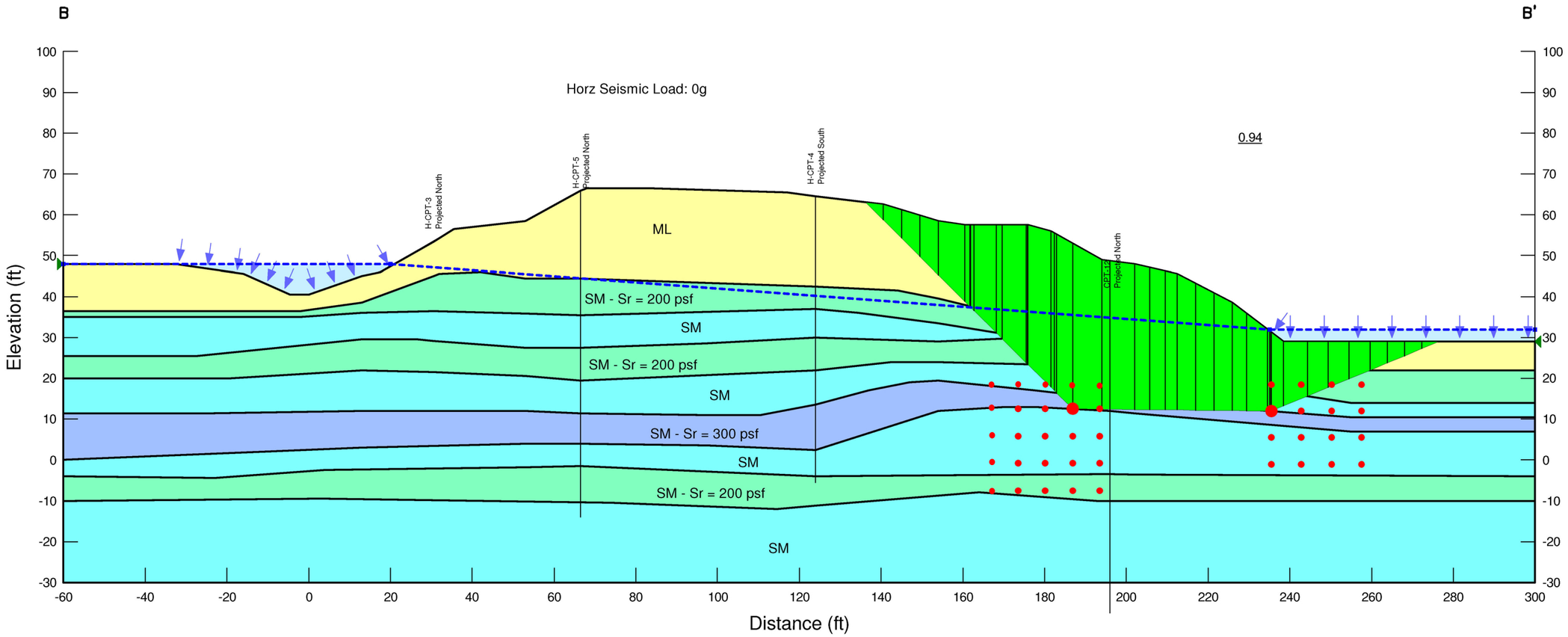

SECTION B–B'

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HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

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MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT

SECTION B-B' RESIDUAL SLOPE STABILITY

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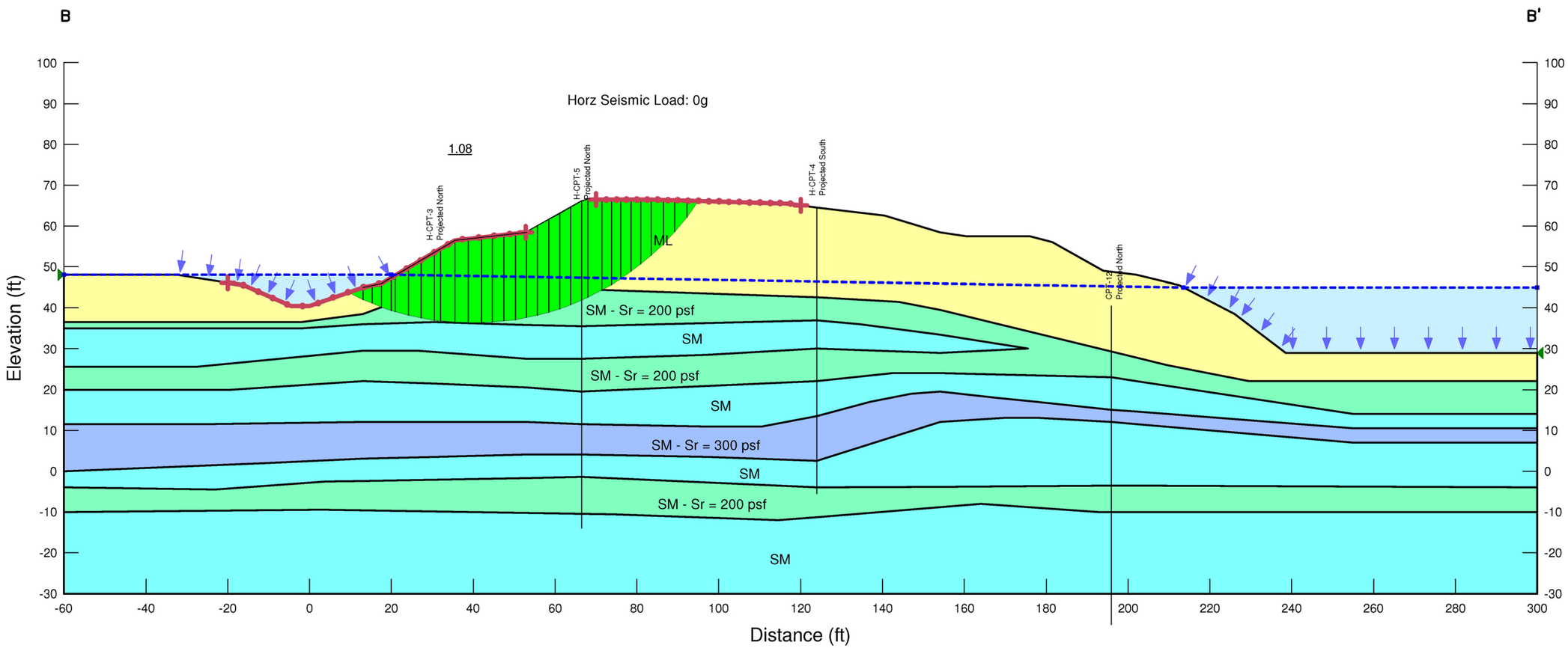

SECTION B–B'

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BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

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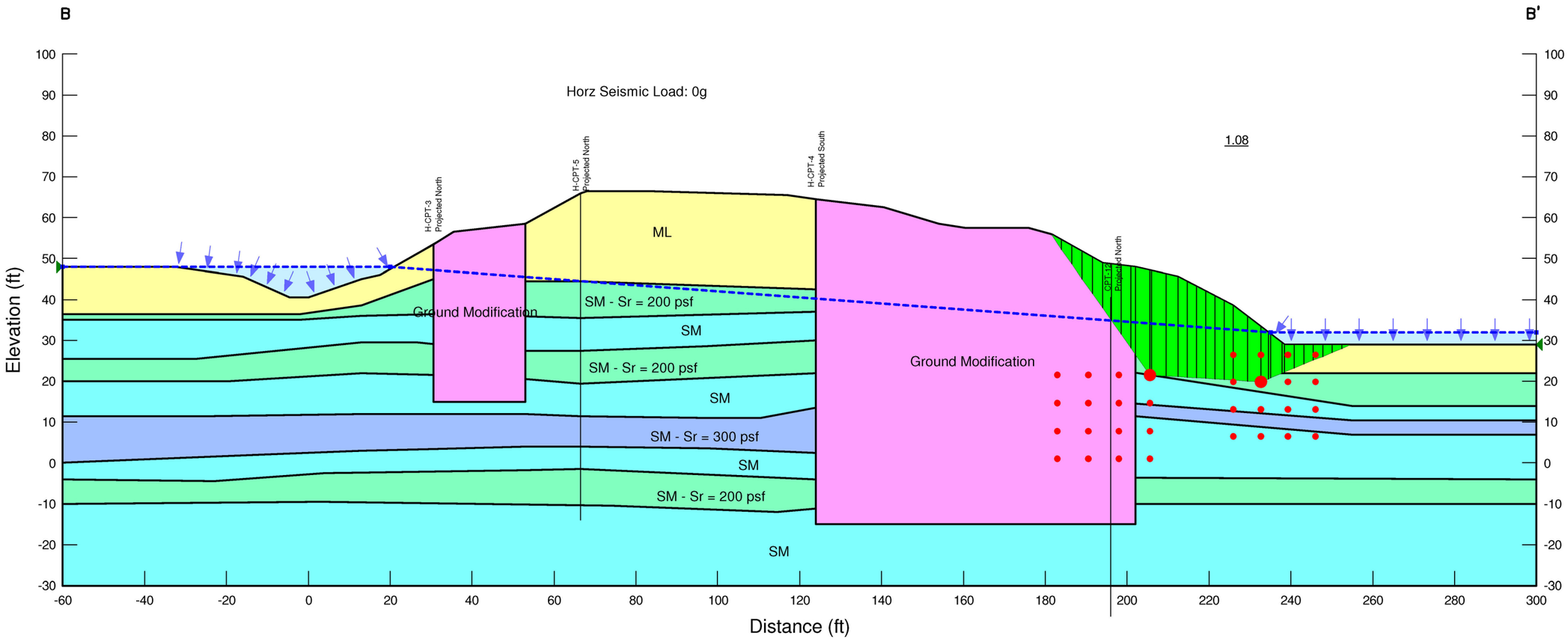

SECTION B–B'

BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

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MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT

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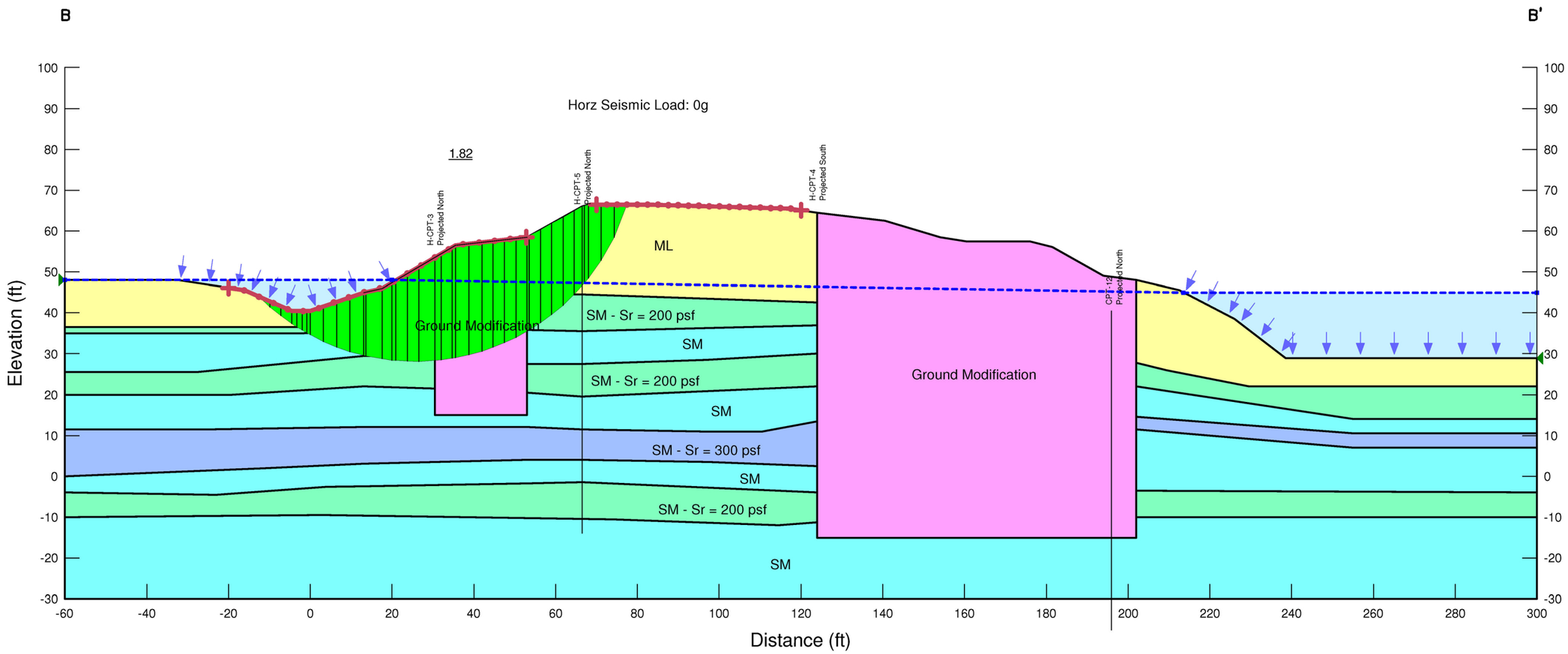

SECTION B–B'

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BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

SCALE: AS NOTED

DATE: 7.8.2013

DRAWN: PVM

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MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT

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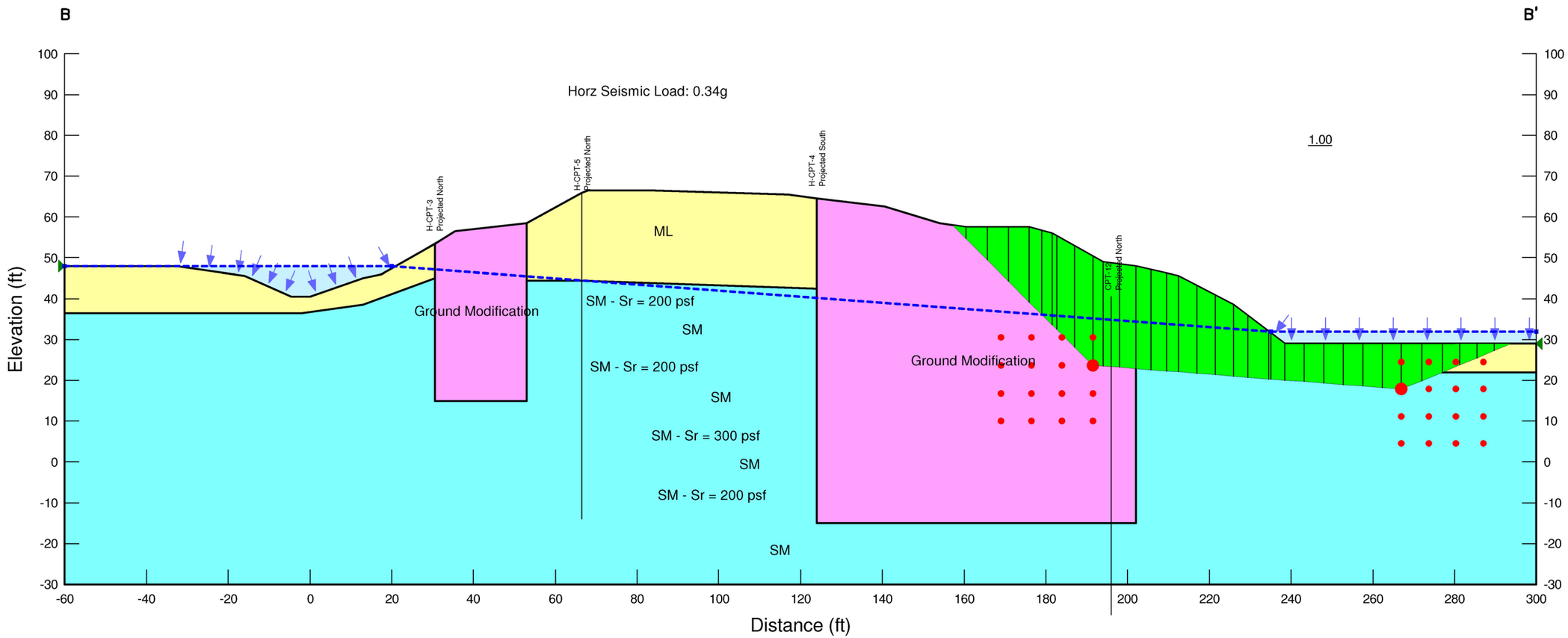

SECTION B–B'

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BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

SCALE: AS NOTED	DATE: 7.8.2013
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MIRABEL FISH PASSAGE AND SEISMIC IMPROVEMENT PROJECT

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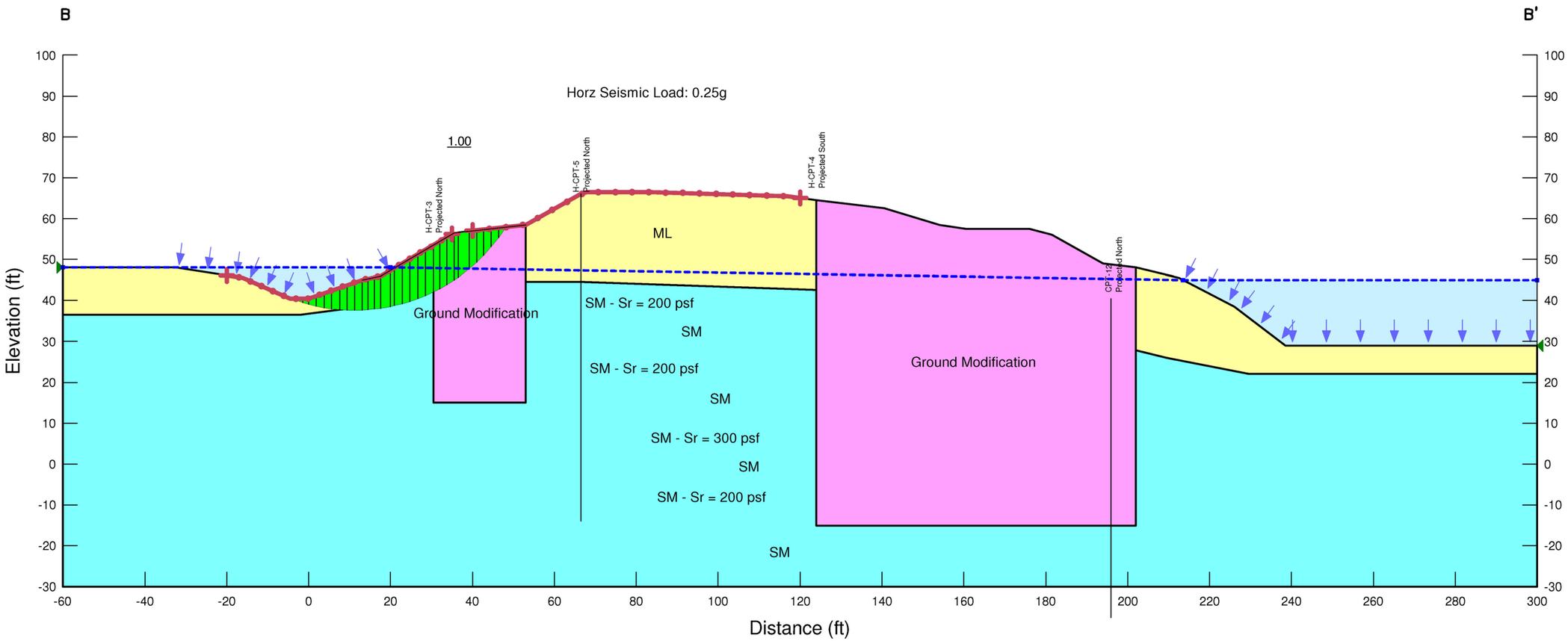

SECTION B–B'

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HDR Fisheries DESIGN CENTER

SONOMA COUNTY WATER AGENCY

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