Geotechnical Investigation

Proposed Inter-Agency Visitor Center Intersection of Highway 101 and Citizen's Dock Road, Crescent City, Del Norte County, California

Prepared for:

Crescent City Harbor District

SHN Consulting Engineers & Geologists, Inc.

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January 2013
012226

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Reference: 012226

January 25, 2013

Richard Young, Harbor Master
Crescent City Harbor District
101 Citizens Dock Road
Crescent City, CA 95531

Subject: Geotechnical Report, Proposed Inter-Agency Visitor Center; Intersection of Highway 101 and Citizen's Dock Road, Crescent City, CA

Dear Mr. Young:

This report presents the results of a geotechnical investigation conducted by SHN Consulting Engineers & Geologists, Inc. (SHN) for the Crescent City Harbor District's proposed inter-agency visitor center. The visitor center is to be constructed at the intersection of Redwood Highway (Highway 101) and Citizen's Dock Road. The primary purpose of this investigation is to assess site surface and subsurface conditions and to develop geotechnical recommendations in support of the design and construction of the proposed structure. Our investigation included a) reviewing subsurface information developed for surrounding projects during previous studies, b) conducting a field exploration and laboratory testing program, and c) developing geotechnical recommendations, including earthwork and foundation recommendations, for the planned construction. The accompanying report documents the services provided and presents the results of our investigation and our geotechnical recommendations.

We appreciate this continued opportunity to work with you on this project. If there are any questions as to the content of this report, please feel free to contact either of us.

Sincerely,

SHN Consulting Engineers & Geologists, Inc.

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JPB:JHD/GAV:lms

Enclosure: Geotechnical Report

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Geotechnical Investigation

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Crescent City, CA

Prepared for:

Crescent City Harbor District
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Crescent City, California 95531

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QA/QC: GDS gDS

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List of Illustrations

Abbreviations and Acronyms

psf	pounds per square foot
km	kilometers
ASTM	American Society for Testing and Materials-International
Caltrans	California Department of Transportation
CBC	California Building Code
CC-#, CPT-#	cone penetrometer test-number
CDMG	California Department of Mines and Geology
CGS	California Geological Survey
CPT	cone penetrometer test
CSZ	Cascadia Subduction Zone
FEMA	Federal Emergency Management Agency
FIRM	flood insurance rate map
FOS	factor of safety
H:V	horizontal to vertical
M	magnitude
MRfz	Mad River fault zone
NR	no reference
OSHA	U.S. Occupational Health and Safety Administration
SHN	SHN Consulting Engineers & Geologists, Inc.
SPT	standard penetration test
TP-#	test pit-number
USGS	U.S. Geologic Survey

1.0 Introduction

SHN Consulting Engineers & Geologists, Inc. (SHN) performed a geotechnical investigation in support of the design and construction of the proposed Inter-Agency Visitor Center to be constructed on property of the Crescent City Harbor District. The subject property and project site is at the intersection of Redwood Highway and Citizen's Dock Road in Crescent City. The general site location is shown on Figure 1.

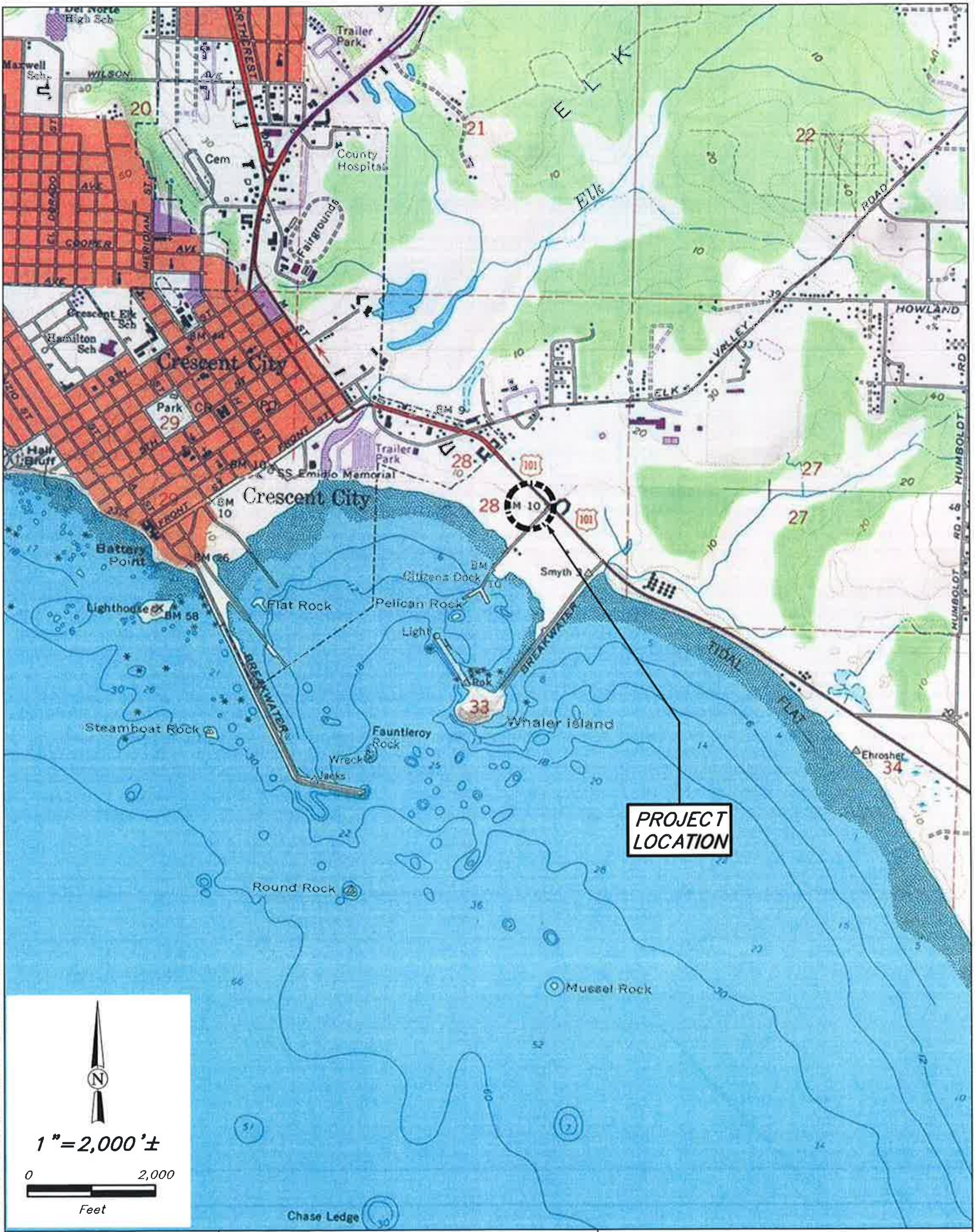
Our general understanding of the proposed project is based on the most recent conceptual design for floor plans and building location, provided by Crow/Clay & Associates, Inc., dated January 2013. The proposed conceptual building footprint location is as illustrated on Figure 2 Map.

1.1 Purpose and Scope

The primary purposes of this investigation were to explore and evaluate subsurface soil and bedrock conditions at the site and to develop geotechnical recommendations and design criteria for earthwork construction and foundation support for the proposed structure.

The scope of services included reviewing available subsurface information, conducting cone penetrometer tests (CPT), excavating backhoe test pits, performing laboratory tests on selected soil samples, and developing recommendations for site grading and foundation design. Specifically, the following information, recommendations, and design criteria are presented in this report:

- Description of site terrain and local geology
- Description of subsurface soil, bedrock, and groundwater conditions interpreted based on our field exploration, laboratory testing, and review of existing geotechnical information
- Logs of CPT soundings and test pits, and results of laboratory tests conducted for this investigation
- Assessment of potential earthquake-related geologic/geotechnical hazards (e.g., surface fault rupture, liquefaction, differential settlement, site instability, tsunami inundation) and discussion of possible mitigation measures, as necessary
- Seismic design parameters in accordance with the applicable portions of the most recent edition of the California Building Code (CBC), including site soil classification, seismic design category, and spectral response accelerations
- Recommendations for earthwork, including site and subgrade preparation, fill material, placement and compaction requirements, criteria for temporary excavation support, and possible dewatering issues
- Discussion of appropriate foundation options
- Recommendations regarding foundation elements, including
    - allowable bearing pressures or capacities (dead, live, and seismic loads)
    - estimates of settlement (total and differential)
    - allowable lateral passive and sliding resistance characteristics for footings
    - minimum foundation embedment



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Inter-Agency Visitors Center
Crescent City, California

Location Map

SHN 012226

January 2013

Location_Map

Figure 1

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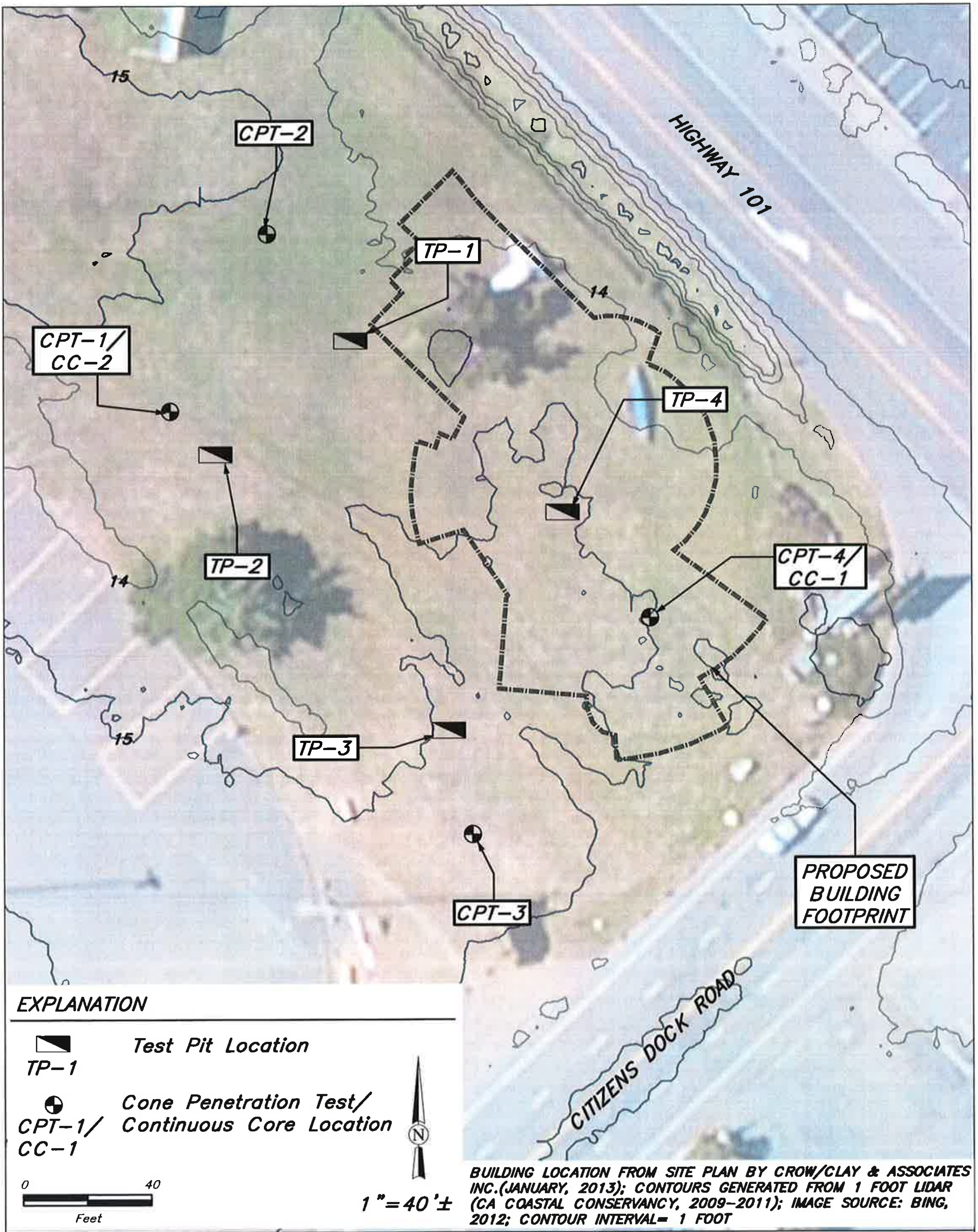

BUILDING LOCATION FROM SITE PLAN BY CROW/CLAY & ASSOCIATES INC.(JANUARY, 2013); CONTOURS GENERATED FROM 1 FOOT LIDAR (CA COASTAL CONSERVANCY, 2009–2011); IMAGE SOURCE: BING, 2012; CONTOUR INTERVAL= 1 FOOT

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SHN Consulting Engineers & Geologists, Inc.	Crescent City Harbor District Inter-Agency Visitors Center Crescent City, California	Site Plan Map SHN 012226
	January 2013	Site_Plan_Map
		Figure 2

- Recommendations for support of slabs-on-grade
- Recommendations for observation of foundation installation, materials testing and inspection, and other construction considerations

In addition to the geotechnical investigation performed for this study, we have reviewed geologic and geotechnical data from reports for projects surrounding the site. These reports include geotechnical investigation for the Crescent City Harbor Rehabilitation (Treadwell & Rollo, June 2011) and geotechnical investigation for the Harbor Promenade (LACO Associates, February 2012).

1.2 Project Authorization

The geotechnical investigation activities presented in this report were performed in accordance with the scope and services outlined in our revised proposal to Mr. Jonathon Olson, Project Engineer for Stover Engineering, dated September 14, 2012, and the Agreement for Professional Services by and between the Crescent City Harbor District and SHN, executed on October 2, 2012.

2.0 Project Description

Pre-design conceptual plans for the proposed development call for constructing a two-story structure with finished floor likely to be 2.5± feet above adjacent grade. The lower floor footprint is 10,700 square feet and will include a lobby, reception area, interpretive center, conference rooms, gift shop, offices, storage and work rooms, and restroom facilities. The upper floor contains an area of 5,005 square feet and will include a balcony, interpretive display, open office space, break room, and additional restroom facilities. We understand that the type of construction and building loads have not been determined at the time this report was prepared. However, we anticipate that the building will be metal- or wood-framed with wood siding, and foundation loads (dead plus long-term live) will be less than 3 kips per lineal foot for wall loads and less than 40 kips for column loads.

The adjoining map for lands within the City of Crescent City shows a flood elevation of 17 feet for the areas along Elk River Valley. Grades at the project site average about 14 feet to 15 feet. Therefore, it is our understanding that the building will be designed so that the finished floor elevation is a minimum required height of 17 feet above sea level to meet the standard within the City for the adjoining area.

3.0 Field Exploration and Laboratory Testing

General descriptions of the field and laboratory testing programs performed for the current site investigation are presented below. More detailed descriptions of the subsurface explorations and laboratory testing programs including the final CPT, test pit logs, and laboratory test data are presented in Appendices A, B, and C, respectively. Logs of geotechnical borings performed by Treadwell & Rollo for the Inner Harbor Basin rehabilitation project are presented in Appendix D.

3.1 Field Exploration Program

The field exploration program for this investigation consisted of installing four CPT soundings, two continuous soil cores, excavating four backhoe test pits, logging the soils encountered and obtaining samples of the subsurface materials, and performing geotechnical laboratory tests on selected representative samples. The locations of the CPT soundings, continuous soil cores, and backhoe test pits are shown on Figure 2. Locations of the geotechnical borings drilled at the Inner Harbor by Treadwell & Rollo during their 2011 investigation are shown with respect to the current project site on Figure 3.

3.2 Cone Penetrometer Tests and Continuous Soil Cores

CPT soundings and continuous soil cores were advanced on November 13, 2012, using a GeoProbe 6600 operated by Fisch Drilling of Hydesville, California. The CPT soundings were advanced to depths of between 27.5 feet and 32.5 feet below ground surface. The continuous soil cores were both advanced to a depth of about 32 feet below ground surface. CPT and continuous core locations were approximately located in the field to encompass the building footprint of the proposed structure. Digital CPT logs indicating the soil behavior type were prepared by Fisch Drilling on behalf of SHN. Electronic text files of the CPT data were also supplied to SHN for the quantitative liquefaction potential analysis. Continuous soil cores were reviewed at the SHN office to verify the soil behavior types identified by the CPT soundings.

3.3 Backhoe Test Pits

Four exploratory backhoe test pits, denoted as TP-1 through TP-4, were excavated by Bayside Construction of Crescent City simultaneously with the CPTs. The exploratory test pits were excavated to depths ranging from 8.5 to 10.5 feet below grade, to characterize the shallow subsoils (especially the distribution of uncontrolled fill soils) visually, and to collect relatively undisturbed drive-tube soil samples. Soils encountered in the test pits were logged in general accordance with American Society for Testing and Materials-International (ASTM) D2488 (Visual-Manual Procedure). Test pits were located in proximity to the CPTs to correlate the visually observed soils with the soil behavior types interpreted from the CPT logs.

3.4 Laboratory Testing

Selected soil samples were tested in SHN's certified materials testing laboratory to evaluate their physical characteristics and engineering properties. Samples were tested for their moisture content and unit weight, percent passing the #200 sieve (combined silt and clay), and shear strength. Laboratory test results are presented in Appendix C and adjacent to the corresponding sample intervals on the test pit logs in Appendix B.

4.0 Site Conditions

The following sections describe the proposed visitor center building site and current surface conditions, the geologic and seismic settings of the site, and subsurface soil and groundwater conditions encountered at the time of our field exploration.

4.1 Topography

Based on the Sister Rocks 7.5-minute topographic quadrangle (USGS, 1966), the site is in an area of very low relief and lies below the 10-foot topographic contour (National Geodetic Vertical Datum of 1929). According to the site plan provided by the project architect, elevation of the site ranges from

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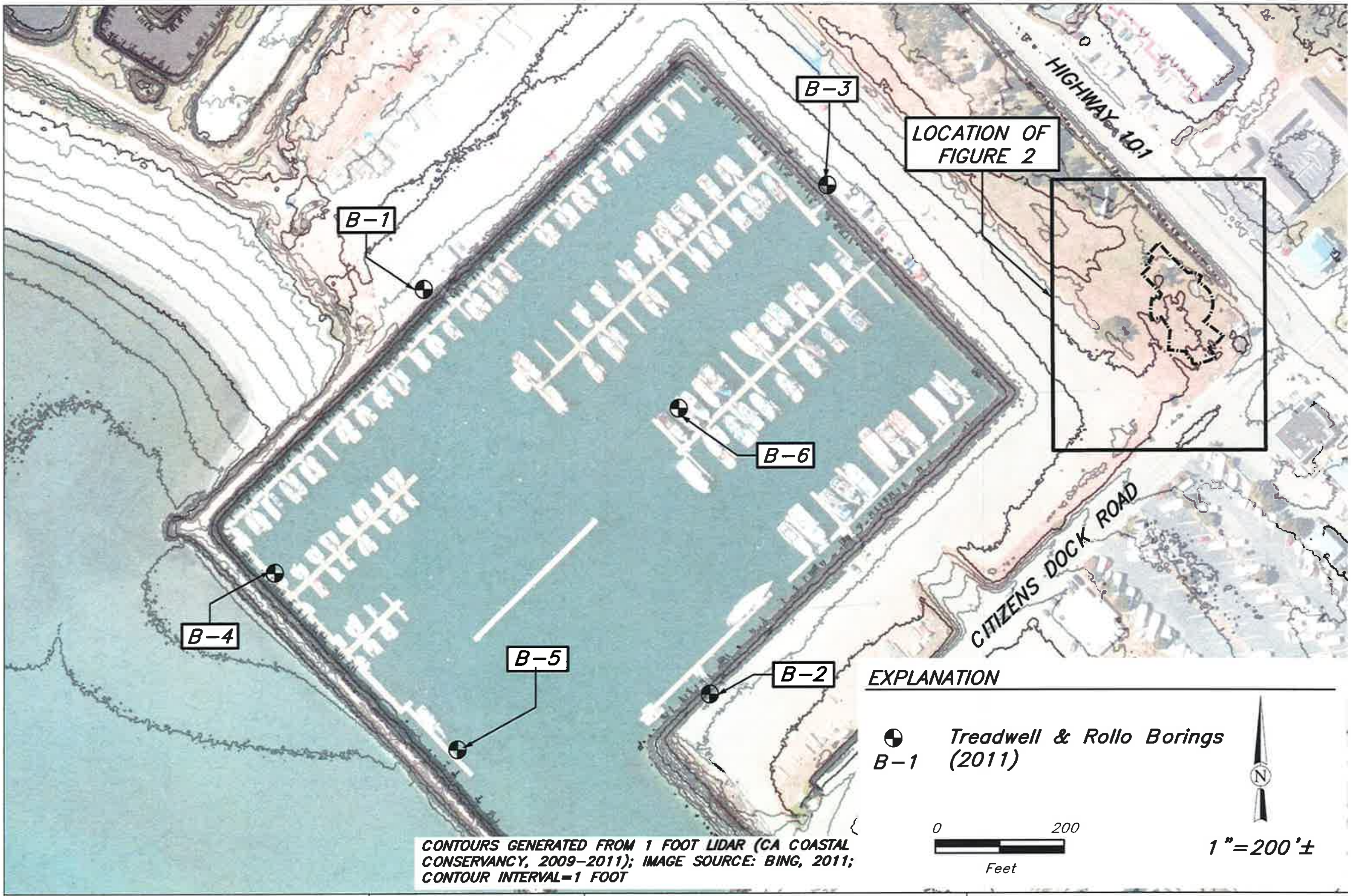

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EXPLANATION

● B-1 Treadwell & Rollo Borings (2011)

1"=200'±

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Inter-Agency Visitors Center
Crescent City, California

Borings by Others
Treadwell & Rollo (2011)
SHN 012226

January 2013

Previous_Subsurface_Investigation

Figure 3

about 13 feet to 16 feet (project datum is unknown). The lower elevations occur at the location of a roadside drainage ditch where the site borders Redwood Highway. The higher elevations are present at the north end of the site. The proposed building footprint encompasses elevations of 14 feet to 15 feet. Based on the lack of any surface slope, drainage of stormwater likely occurs by direct infiltration into the ground surface.

4.2 Geology

Coastal bluff exposures along Pebble Beach and Point St. George indicate the Crescent City area to be underlain by Jurassic to Cretaceous age Franciscan Complex bedrock, Pliocene age St. George formation mudstone, and Pleistocene age Battery formation marine terrace deposits (Davenport, 1982). Younger beach deposits composed of silty sand overlies the Battery formation at relatively shallow depths (less than 5 feet below ground).

At the site, Franciscan Complex bedrock is present at an unknown depth, but is anticipated to be less than about 100 feet below ground. Outcrops of Franciscan Complex bedrock occur at or near sea level at Battery Point, located about 6,000 feet west of the site, and at Whaler Island located about 3,000 feet south of the site (Figure 1). Franciscan Complex bedrock consists of consolidated arkosic sandstone with some shale and minor amounts of chert, conglomerate, and greenstone. Franciscan rocks are the relatively more resistant rock type in the local area and form the numerous small offshore islands and sea stacks visible from the Crescent City waterfront.

Overlying the Franciscan Complex bedrock is the St. George formation consisting of consolidated massive marine siltstone and shale with thin beds of sand and scattered pebbles. Exposures along Pebble Beach indicate bedding attitudes within the St. George Formation strike north-northwest and dip shallowly to the east at 8- to 15-degrees. At the site, the St. George formation was encountered at the locations of our CPT soundings beginning at about 30 feet below ground surface. Similar depths below ground surface to the top of the St. George formation were reported by Treadwell & Rollo at the locations of their land-based geotechnical borings around the Inner Harbor Basin located about 600 feet to the west. Borings by LACO Associates near the Harbor Promenade, located about 2,000 feet to the south-southeast indicate the top of the St. George formation to be 26 feet (±2 feet) below ground surface. The combined subsurface data indicates the top of the St. George formation to be relatively planar and level in the vicinity of the site.

Overlying the St. George formation are younger marine terrace deposits of the Battery formation. The Battery formation consists of unconsolidated medium-grained quartz sands alternating with silty clay and imbricated gravels. At the site, the Battery formation was encountered at the locations of our CPT soundings and backhoe test pits beginning at 4± feet below ground surface. A thin veneer of beach deposits overlies the Battery formation and forms the modern-day low relief surface in and around the site. Beach deposits are composed of unconsolidated, loose, silt and sand that are of relatively low density.

4.3 Seismicity

The Crescent City area is located in a complex tectonic region dominated by northeast-directed compression associated with collision of the Gorda and North American tectonic plates. The Gorda plate is being actively subducted beneath North America north of Cape Mendocino, along the southern portion of the Cascadia Subduction Zone (CSZ). This plate convergence has resulted in a broad fold-and-thrust belt along the western edge of the accretionary margin of the North American plate. In the offshore Crescent City area, this fold-and-thrust belt is manifested as a series of northwest-trending, southwest-verging thrust faults (i.e., dipping to the northeast beneath Crescent City). The activity status of these faults are unknown but are on trend with Holocene to Late Pleistocene age faults to the north offshore of the Oregon coastline (USGS, 2010), and with active faults to the south that comprise the Mad River and Little Salmon fault zones (Carver, 1987). On the basis of this association, faults within the offshore Crescent City area should be considered potentially active and capable of generating moderate- to large-magnitude earthquakes. The nearest onshore fault in proximity to the site consists of the Del Norte fault, located along the base of the range front east of the Smith River plain (Davenport, 1982). The activity status of this fault, however, is unknown.

Northwestern California in general, is one of the most seismically active regions in the continental United States. More than 60 earthquakes have produced discernable damage in the region since the mid-1800s (Dengler et al., 1992).

In addition to the faults offshore of Crescent City and the southern Oregon coastline, there are several other potential sources for strong seismic shaking including:

1. **The Gorda Plate.** Gorda Plate earthquakes account for the majority of historic seismicity. These earthquakes occur primarily offshore along left-lateral faults, and are generated by the internal deformation within the plate as it moves toward the subduction zone. Significant historic Gorda Plate earthquakes have ranged in magnitude from M5 to M7.5. The November 8, 1980, earthquake (M7.2) and the more recent January 9, 2010 (M6.5) were both generated on left-lateral faults within the Gorda Plate.
2. **The Mendocino Fault.** The Mendocino fault is the second most frequent source of earthquakes in the region. The fault represents the plate boundary between the Gorda and Pacific plates, and typically generates right lateral strike-slip displacement. Historic Mendocino fault events have ranged in magnitude from M5 to M7.5. The September 1, 1994, M7.2 event west of Petrolia was generated along the Mendocino Fault.
3. **The Mendocino Triple Junction.** The Mendocino triple junction was identified as a separate seismic source only after the August 17, 1991, M6.0 earthquake. Events associated with the triple junction are shallow onshore earthquakes that appear to range in magnitude from about M5 to M6. Raised Holocene terraces near Cape Mendocino suggest larger events are possible in this region.
4. **The Northern End of the San Andreas Fault.** Northern San Andreas fault events are rare, but can be very large. The northern San Andreas fault is a right lateral strike-slip fault that represents the plate boundary between the Pacific and North American plates. The fault extends through the Point Delgada region and terminates in the Mendocino triple junction region. The 1906 San Francisco earthquake (M8.3) caused the most significant damage in the north coast region, with the possible exception of the 1992 Petrolia earthquake.
5. **The North American Plate.** Earthquakes originating within the North American plate can be anticipated from a number of intraplate sources, including the Mad River fault zone (MRfz) and Little Salmon fault. The MRfz is located at the northern end of Humboldt Bay, and is entirely south of the site. There has been no large magnitude earthquakes associated with faults within the North American plate, although the December 21, 1954, magnitude 6.5 event may have occurred in the MRfz. Damaging North American plate earthquakes are expected to range from magnitude 6.5 to 8.

6. **The Cascadia Subduction Zone.** The Cascadia Subduction Zone (CSZ) represents the most significant potential seismic source in the north coast region. A great subduction event has the potential to rupture up to 200 kilometers (km) or more, beginning off the coast from Cape Mendocino and extending north to British Columbia. CSZ events may be up to M9.5, and are associated with extensive tsunami inundation in low-lying coastal areas, such as, those within the Crescent City waterfront area. The April 25, 1992, Petrolia earthquake (M7.1) appears to be the only documented historic earthquake involving slip along the subduction zone, but this event was confined to the southernmost portion of the fault. Paleoseismic studies along the subduction zone suggest that great earthquakes are generated along the zone every 300 to 500 years. The last large subduction earthquake occurred in 1700. A great subduction earthquake would generate long duration, very strong ground shaking throughout the Pacific Northwest and northern California, resulting in areas of localized coseismic uplift and subsidence.

4.4 Subsurface Conditions

The results of our subsurface exploration indicate that the site is underlain by a relatively uniform stratigraphic sequence consisting of 2 feet of uncontrolled fill material overlying about 3 feet of relatively young, unconsolidated beach deposits. The beach deposits, in turn, overlie medium dense to dense, Pleistocene age marine terrace sediments of the Battery formation, which exist to the maximum depths explored. CPT refusal occurred at depths ranging from 27.5 feet to 32.5 feet below ground surface on what is interpreted to be the upper contact of the Pliocene age St. George Formation bedrock. The transition from unconsolidated beach deposits to the much older and denser Battery formation occurs at depths ranging from about 3.5 feet to 5 feet below ground surface (generally deeper toward the north end of the site).

4.4.1 Undocumented Fill and Topsoil

Fill material and topsoil consisting of loose to medium dense sand with silt and lesser amounts of gravel, were encountered in each of our test pits. Thickness of the fill material was generally uniform, ranging from 1.5 feet to 2 feet thick. The fill material appears to have been emplaced directly on the original ground surface overlying the native topsoil. The time at which the fill was placed, and the amount of relative compaction achieved during fill emplacement is unknown.

4.4.2 Beach Deposits

Beach deposits at the site consist of relatively low-density, unconsolidated silts and sands. Borings excavated by Treadwell & Rollo around the Inner Harbor Basin indicates that the beach deposits are loose to medium dense, and composed of fine- to medium-grained sand with shell fragments. The beach deposits encountered in their borings occur to a depth of about 10± feet. Beach deposits at the current project site occur to a depth of about 4± feet.

4.4.3 Battery Formation

The top of the Battery formation was encountered within our test pits beginning at about 4 feet below ground surface. The upper exposures consist of medium stiff to stiff clay and silt grading downward to loose to medium dense poorly graded sand with silt and poorly graded gravel with sand. Below the depths of the test pits, the Battery formation sediments inferred from the CPT logs consist of medium dense to dense, alternating layers of sand and silty sand to the maximum depths explored. The CPT data appears consistent with the standard penetration test (SPT) N-Values from the Treadwell & Rollo borings which recorded the presence of medium dense to very dense, poorly graded sand and poorly graded sand with gravel.

4.4.4 St. George Formation

The Treadwell & Rollo report describes the St. George formation bedrock as "crushed to intensely fractured, weak, friable, plastic, moderate to deeply weathered mudstone/claystone. The mudstone/claystone is interbedded with moderate to deeply weathered, weakly cemented sandstone with a low hardness." SPT N-Values indicate the bedrock material to be very dense with typical blow counts of 50 or more recorded in less than 6-inches of sampler penetration.

At the project site, St. George formation is present beginning at depths ranging from about 28 feet to 32 feet below ground surface. The depth to the top of the St. George formation is interpreted from the CPT data on the basis of an abrupt and significant increase in cone tip pressure and cone sleeve friction. The depth to the St. George formation indicated by the CPT soundings was confirmed from a visual assessment of the continuous soil cores.

4.4.5 Groundwater

Free groundwater was encountered in test pits TP-3 and TP-4 at depths of 9.5 feet and 8 feet below ground surface, respectively. Perched groundwater was observed in TP-1 at 3.5 feet below ground surface. Groundwater was not encountered in TP-2, excavated to a depth of 8.5 feet below ground surface. Because our field investigation was completed in mid-November prior to the onset of the wet season, it is likely that shallower groundwater conditions will persist during the winter and spring months. Groundwater levels should, therefore, be expected to fluctuate seasonally on the order of several feet in elevation, and may likely be tidally-influenced at this site as well.

4.5 Geologic Hazards

Potential geologic/geotechnical hazards that the project site may be subject to include seismic ground shaking, surface fault rupture, seismically induced ground deformation (liquefaction, lateral spreading, and slope failure), and tsunami inundation. Our assessment of these potential hazards is presented below.

4.5.1 Seismic Ground Shaking

The site is located within a seismically active area proximal to multiple seismic sources capable of generating moderate to strong ground motions. Given the proximity of the site to these active seismic sources, the probability that strong ground shaking associated with large magnitude earthquakes will occur during the design life of the proposed structure is considered high.

4.5.2 Surface Fault Rupture

The site is not located within an Alquist-Priolo Earthquake Fault Hazard Zone (Bryant and Hart, 2007). Faults in proximity to the site include the queried trace of the north-striking Del Norte fault (Davenport, 1982) located along the base of the range front east of the Smith River plain, and the numerous faults in the offshore areas of Del Norte County (CGS, 2010).

The closest fault to the site recognized by the State of California as "active" under the provisions of the Alquist-Priolo Earthquake Fault Zoning Act is the on-land segment of the Trinidad fault (within the MRfz described in Section 4.3), located approximately 47 miles to the south. No geomorphic evidence of fault scarps or other fault-related features was observed at the project site during our reconnaissance. Because no active faults are known to be present within the Crescent City area, it is our opinion that the potential for surface fault rupture to occur at this site is negligible.

4.5.3 Liquefaction

Liquefaction is a soil behavior phenomenon in which a soil loses a substantial amount of strength due to high excess pore-water pressure generated by strong earthquake ground shaking. Relatively young (i.e., deposited within last few thousand years) and unconsolidated soils and artificial fills located below the groundwater surface are considered susceptible to liquefaction (Youd and Perkins, 1978). Typically, the soils that are most susceptible to liquefaction include relatively clean, loose, uniformly graded sand, silty sand, and non-plastic deposits.

As previously discussed in this report, the CPT data collected during this investigation indicates that the soils underlying the site are predominantly medium stiff to stiff, and medium dense to dense marine terrace deposits of the Pleistocene age Battery formation. Underlying the Battery formation is very dense bedrock of the St. George formation.

The potential for liquefaction and liquefaction-induced settlement was evaluated for the project site using the data collected from the CPT soundings. The evaluation was performed in accordance with the methodology presented in the publication *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCFEA/NSF Workshop on Evaluation of Liquefaction Resistance on Soil* (Youd and Idriss, 2001) using the software program LiqIT, version 4.7.7.1, by GeoLogismiki, Inc. A peak ground acceleration of 0.41 times gravity, which corresponds to the 2010 CBC Design Earthquake, was used in our analyses. Graphical results of the analyses are presented in Appendix E.

The results of the liquefaction analyses indicate that multiple intervals identified in the CPT soundings are susceptible to liquefaction. In CPT-1 and CPT-2, the intervals susceptible to liquefaction are relatively thin (less than 1-foot in thickness) and discrete, and are bound by non-liquefiable layers with a factor of safety (FOS) greater than 3.5. Intervals susceptible to liquefaction at the locations of CPT-3 and CPT-4 are generally on the order of 2 feet to 13 feet in thickness, and are also bound by non-liquefiable layers with a FOS greater than 3.5. At the location of CPT-3 and CPT-4, the potential for liquefaction is primarily indicated for sand to silty sand soil behavior types within the Battery formation below the water table and at a depth of about 15 feet to 30 feet below ground surface. The cumulative amount of potential liquefaction-induced settlement calculated at each CPT location is 0.3 inches at CPT-1, 0.8 inches at CPT-2, 3 inches at CPT-3, and 5 inches at CPT-4.

Treadwell & Rollo performed a liquefaction analysis using the method which relates normalized clean sand SPT N-Values to strain potential. Their analysis identified potentially liquefiable layers composed of loose to medium dense sand encountered below the water table and at a depth of about 10 feet to 20 feet below ground. The potential liquefaction-induced settlement calculated from their analysis was up to 1.6 inches.

The analysis methods discussed above do not account for the geologic age of the material, the results can therefore be viewed as conservative estimates. In general, liquefaction potential is considered low in late Pleistocene marine terrace deposits (such as, the Battery formation sediments underlying the site). Based on the age of geologic materials, the analysis performed by Treadwell & Rollo, and our analysis, which indicated the intervals susceptible to liquefaction generally to be thin and bound by non-liquefiable layers, we judge the liquefaction hazard to be relatively low.

We provide a discussion of the liquefaction potential and risks associated with it in Section 5.2, below.

4.5.4 Lateral Spreading

Lateral spreading is the displacement of soil that occurs when a continuous soil layer liquefies and the overlying soil layers move toward an unsupported slope face. The distance of the nearest slope face to the planned Visitor Center is about 200± feet, and is supported by boulder rip-rap. Based on the results of SHN's and Treadwell & Rollo's liquefaction analyses indicating a relatively low liquefaction hazard and the distance of the Inner Harbor Basin slopes, we judge the potential for lateral spreading to occur to be low.

4.5.5 Tsunami Inundation

The site is located within the mapped Tsunami Inundation Area on the Tsunami Inundation Map for Emergency Planning, Crescent City/Sister Rocks Quadrangle (CGS, 2009). Since the tidal gauge was installed in the harbor in 1934, 34 tsunamis have been recorded in Crescent City. At least four of those tsunamis have caused damage. The 1964 tsunami remains the largest and most destructive recorded tsunami to ever strike the United States Pacific Coast. The Crescent City area is particularly vulnerable to tsunami inundation due to the offshore bathymetry, which amplifies tsunami wave energy and directs it toward the inland-curving coastline.

Tsunami waves from the Great Alaskan Earthquake of March 28, 1964, affected the entire California coastline, but were most severe in Crescent City. The travel time of the first tsunami wave to Crescent City was 4.1 hours after the occurrence of the earthquake in Alaska. It caused no significant damage other than flooding. The second and third waves were reportedly smaller than the first. The fourth was the largest of the waves with a height of approximately 20 to 21 feet. It was preceded by a withdrawal of the water which left the inner harbor almost dry. The tsunami waves covered the entire length of Front Street, destroyed half of the waterfront business district, inundated 60 city blocks, 30 of which were devastated. Lumber, automobiles, and other debris carried by the waves were responsible for a majority of the damage to the buildings. A tsunami run-up map of the 1964 event indicates the current project site to have been within the inundation zone at that time, with the greatest run-up occurring within the Elk River Valley. The dolo that currently lies near the northern corner of the proposed building footprint was knocked off its concrete pad by the force of a log entrained in a tsunami wave.

Recent history has shown that tsunami waves with a potential to impact the Crescent City harbor severely may be generated by a variety of distant sources throughout the Pacific Ocean. In 2006, a tsunami generated by a magnitude 8.3 earthquake off the Kuril Islands in the western Pacific caused damage to three docks and several boats. And most recently in 2011, a tsunami generated by a great subduction zone earthquake off the coast of Japan again caused significant damage to docks and boats. As these recent cases have demonstrated, the Inner Harbor Basin is subject to the destructive forces of harbor resonance where swift currents from long-period tsunami waves enter the narrow harbor entrance, but with no inundation of the project site.

As evidenced by the 1964 event, the project site and surrounding low-lying areas are subject to inundation from teletsunamis (source of tsunami more than 1,000 km away) resulting from great subduction zone earthquakes occurring at great distances from northwest California. Under the scenario of a great subduction earthquake along the Cascadia margin located less than 50 miles off the California and Oregon coast, the project site would be subject to tsunami inundation heights and horizontal inundation distances that are predicted to exceed the destructive forces observed in 1964 by far. The travel time of the first tsunami wave will be very short, arriving soon after strong ground shaking has ceased and will leave little time to evacuate to higher ground.

Recent investigations of the back-ridge marshes along Highway 101 south of the site identified six paleotsunami sand sheets deposited in the past 300 to 3,000 years, yielding a 450± year mean recurrence interval for a near field Cascadia tsunami (Peterson et al., 2011). Two paleotsunami sand deposit records, likely correlated to Cascadia subduction zone earthquakes between 1,000 and 1,500 years ago, are traced for a distance of nearly 4,000 feet inland to an elevation of about 30 feet. The paleotsunami sand sheets were compared to sand sheets deposited during the 1964 far field tsunami, which closely correspond to the landward extent of large debris transport and structural damage in the Crescent City waterfront. The paleotsunami sand deposits associated with CSZ events record nearly twice the run-up height, and four times the inundation distance of the 1964 tsunami sand sheet in the same marsh system.

Based on historical and geologic evidence, we conclude that the site is subject to a high exposure potential to tsunami inundation, especially in the event of a Cascadia subduction zone generated tsunami.

4.5.6 Flooding

The subject property is not situated within the Federal Emergency Management Agency (FEMA) 100-year flood zone. The tsunami inundation hazard aside, flooding from stream flow is not anticipated to pose a significant hazard at the site.

The adjoining map for lands within the City of Crescent City show a flood elevation of 17 feet for the areas along Elk River valley (Zone VE 17, Panel 331, FIRM [flood insurance rate map] Del Norte County California, map number 06015C0331E effective Sept 26, 2008). Grades at the project site currently average about 14 feet to 15 feet. Therefore, it is our understanding that the building will be designed so that the finished floor elevation is a minimum required height above 17 feet to meet the standard within the City for the adjoining area.

5.0 Geotechnical Site Conditions

5.1 General

Soils underlying the site are composed of uncontrolled fill, beach, and marine terrace deposits. The marine terrace deposits rest on very dense bedrock beginning at about 30± feet below ground surface. The relatively soft to loose, unconsolidated nature of the fill and beach deposits within the upper 4 feet of the ground surface poses a potential risk of settlement under the application of new building and structural fill loads. Below 4 feet, the marine terrace deposits are medium stiff to stiff, to medium dense, and well-consolidated.

Groundwater was observed in our test pits at a depth of 8 to 9 feet below the ground surface at the time our field investigation was conducted in November 2012. These observations are assumed to represent groundwater levels at or near their seasonal low. Shallower groundwater conditions are likely to be present during the wet season.

The principal geologic/geotechnical engineering considerations affecting design and construction of the project include the following:

1) Strong earthquake ground shaking
2) Tsunami inundation from both far- and near-field sources
3) The presence of underlying stratigraphic layers, which are potentially susceptible to liquefaction during relatively infrequent, upper-bound seismic events (Although we interpret this potential to be relatively low, our quantitative liquefaction analysis indicates that up to 5 inches of seismically-induced differential settlement may occur during these rare events.)
4) The presence of uncontrolled fill and unconsolidated, low density silt and sand within the upper 4 feet of the ground surface that appear prone to consolidation settlement (both total and differential) under new building loads and fill material loads

Recommendations presented in Section 6 below include design parameters for ground improvements and for the foundation system, which will reduce the hazard associated with seismically-induced settlement and static settlement.

5.2 Liquefaction

Because the liquefaction potential of the site appears relatively low (based on our interpretation of the geologic age of the deposits), yet our quantitative models suggest up to 5 inches of differential settlement, it is difficult to identify the appropriate mitigation strategy for reduction of liquefaction hazards at the project site. Due to the inherent uncertainties, it is prudent to evaluate the potential mitigation strategies relative to the acceptable level of acceptable risk. For example, if the owners or stakeholders have a low tolerance for risk, the structure should be designed to withstand approximately 5 inches of differential settlement indicated by the quantitative analysis. Alternatively, if a "low" level of risk is acceptable, the structure could be designed to withstand a smaller settlement (for example, 2 inches of differential settlement). Risk in this context not only involves the likelihood of the occurrence, which we have concluded to be relatively low, but also includes the type of structure and its vulnerability to damage and the economic feasibility of mitigating the hazard.

Within the recommendations section we have provided criteria for foundation design that is appropriate for mitigating potential differential settlement of approximately 5 inches. In our professional judgment, this is likely to result in a relatively conservative foundation design.

5.3 Coseismic Compaction

Another potentially adverse secondary seismic effect is coseismic compaction of moderately consolidated, sandy, relatively cohesionless soils above or below groundwater, such as, those encountered below the project site. Coseismic compaction is soil densification resulting from dynamic loading of relatively loose, non-cohesive soil materials. That is, shaking or vibration can densify loose to moderately consolidated granular soils, resulting in settlement of the ground surface.

In our opinion, the geologic age of the deposits at the site minimizes most coseismic compaction risks, and we estimate coseismic compaction would typically be negligible in all but major earthquakes. Relatively rare, very strong seismic events may result in a minor lowering of finished grades associated with coseismic compaction.

5.4 Settlement under Static Conditions

The upper 4 feet of the soil profile is composed of uncontrolled fill and soft to loose, low density beach deposits. These deposits are potentially compressible under new structural and fill material loads.

In our opinion, under normal static conditions, the risk of significant post-construction foundation settlement will be mitigated to a low level if the recommended ground improvements are completed and foundation design criteria are adhered to. Due to the variability of site soils and the inherent limitations of current engineering and construction practices, some post-construction vertical settlement may occur. We estimate that with the project constructed in accordance with the following recommendations, total post-construction settlement is not likely to exceed ½ inch, and post-construction differential settlement is not likely to exceed ¼ inch.

5.5 Expansive Soils

Silt and clay-rich soils were encountered in our test pits at depths of between 4 feet and 8 feet. Below 8 feet, soils become granular and non-cohesive. Based on field texturing, the silt and clay-rich soils in the upper 8 feet were determined to be of low plasticity and are not considered potentially expansive. For these reasons, risk of adverse consequences to the proposed structure from expansive soils is considered low.

6.0 Recommendations

We recommend the structure be designed to withstand strong seismic shaking in accordance with the seismic design requirements of the most recent edition of the CBC. The liquefaction-induced settlement risk may be mitigated by supporting the proposed structure on a spread footing foundation system interconnected with grade beams. The consolidation settlement risk may be mitigated by either lowering the elevation of the building foundation to bear on the relatively dense Battery formation terrace sediments (at depths of approximately 4 feet), or excavating the uncontrolled fill and low-density soils and replacing with compacted engineered fill to a similar depth.

6.1 Earthwork

It is our understanding that the finished floor elevation will be about 2.5± feet above the existing grade. Filling beneath landscaped areas and walkways adjacent to building foundations will therefore be required to provide for positive drainage. Otherwise, site grades are not expected to change appreciably during site preparation for this essentially flat site. Recommendations for site and subgrade preparation, fill and backfill quality and compaction, and surface drainage are presented in the following sections.

6.1.1 Site Preparation

1. As appropriate, notify Underground Service Alert (1-800-642-2444) prior to commencing site work to provide utility clearance.
2. The proposed building footprint is underlain by up to 4 feet, and possibly more, of undocumented fill material and compressible silty topsoil. We recommend removing the undocumented fill material and buried dark-colored topsoil, and also stripping and removing any vegetation or root systems. The bottom of the excavation should extend to the level of suitably firm, undisturbed terrace sediments at about 4 feet below existing ground surface, and for a horizontal distance of 5 feet beyond the outside edge of the foundation. The final subgrade should be reviewed by the project geotechnical engineer or their designated representative prior to the placement of engineered structural fill.
3. All active or inactive utility lines within the construction area should be relocated, abandoned in-place, or fully protected during and following construction. Pipelines to be abandoned in place should be filled with a two-sack cement slurry. If utilities are removed, the remaining excavation should be backfilled with compacted fill or two-sack cement slurry.

6.1.2 Fill Placement and Compaction

1. Following stripping and removal, the surface of the newly created excavation should be scarified to a depth of at least 6 inches and compacted to 90 percent of the same soils maximum dry density per ASTM D 1557, with moisture conditioning as necessary. Pumping, yielding, and/or unstable subgrade soils should be over-excavated and replaced with stabilization material.
2. Following overexcavation and recompaction of the exposed soils, place and compact imported fill to achieve the new planned subgrade elevation. Engineered fill should be placed in loose lifts no greater than 8-inches in thickness and compacted to a minimum of 90 percent of the same soils maximum dry density per ASTM D 1557.
3. To construct the working surface for placement of the grade beam foundation system, the grade may be additionally raised. Additional fill placement should be to minimum relative compaction of 90 percent per ASTM D 1557.
4. Fill material should consist of relatively non-plastic (Liquid Limit less than 40, Plasticity Index less than 14) material containing no organic material or debris, and no individual particles over 4 inches in greatest dimension, and no more than 15 percent larger than 2½ inches. The geotechnical engineer should approve all fill prior to placement.

5. As required by the 2010 CBC, a qualified field technician should be present to observe fill placement and perform field density tests in accordance with ASTM D 6938 at random locations throughout each lift to verify that the specified compaction is being achieved.
6. Fill slopes to remain up to 4 feet in height should be placed no steeper than 2:1 horizontal to vertical (H:V). If higher or steeper slopes are planned, they should be reviewed by the geotechnical engineer.

6.2 Utility Trench Backfill

1. Utility trenches excavated parallel to spread footing foundations should be set back from the footings such that the trench bottoms lie outside a projected hypothetical 1.5:1 H:V line extending downward from the footing bottom.
2. Unless concrete bedding is required around utilities, bedding should consist of sand having a Sand Equivalent of at least 30. The bedding should extend from 6 inches below to 1 foot above the conduit or pipe. Sand bedding should not be jetted or ponded into place and should be mechanically compacted to a minimum of 90 percent relative compaction based on ASTM Test Method D1557.
3. In areas to support improvements (such as, slabs and pavements) and adjacent to structure foundations, backfill placed above the bedding in utility trenches (including culvert and sprinkler lines) should be properly placed and adequately compacted to minimize settlement and provide a stable subgrade. If possible, the trench backfill should be compacted following rough grading but prior to final grading and compaction. On-site inorganic soils meeting the requirements for engineered fill may be used as trench backfill. Backfill consisting of on-site soils should be placed in layers not exceeding 8 inches in loose thickness, moisture-conditioned, and compacted to at least 90 percent relative compaction as described for engineered fill. Trench backfill need only be compacted to 85 percent relative compaction in landscape areas or in areas more than 5 feet beyond the limits of buildings, pavements, concrete slabs-on-grade, sidewalks, or other flatwork. The upper 6 inches of trench backfill under pavements should be compacted to at least 95 percent relative compaction.
4. Special care should be given to ensuring that adequate compaction is made beneath the haunches of utility pipes (that area from the pipe springline to the pipe invert) and that no voids remain in this space.
5. All temporary excavations must comply with applicable local, state, and federal safety regulations, including the current Occupational Health and Safety Administration (OSHA) Excavation and Trench Safety Standards. Construction site safety generally is the responsibility of the contractor, who should be solely responsible for the means, methods, and sequencing of construction operations so that a safe working environment is maintained.
6. Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a 1:1 H:V projection from the toe of open excavations to the ground surface. Support systems (such as, shoring or bracing) should be used to provide structural stability and to protect personnel working within the excavation in accordance with good construction practices and all applicable safety regulations. Soils that are subject to caving should be anticipated within trenches at the project site.

7. Shallow or perched groundwater may be encountered within the depths of typical trench excavations, depending upon the depth of excavation and the season of construction. The contractor should install measures to divert groundwater, or channel groundwater to flow toward collection points to be removed from the trench and disposed of at an approved area.

6.3 Seismic Design

We recommend that the structure be designed and constructed to withstand seismic shaking as required by the CBC. Based on the subsurface conditions encountered at our exploration locations and our general knowledge of the bedrock conditions within 100 feet of the ground surface, we classify the site as a Site Class C consisting of "Very dense soil and soft rock" (Table 1613.5.2, 2010 CBC; CBSC, 2010). On this basis, the design spectral response accelerations were determined using the seismic calculator software provided by the United States Geological Survey (USGS, 2011) in accordance with the American Society of Civil Engineers Standard 7-05, Minimum Design Loads for Buildings and Other structures. Calculated values are presented in Table 1.

Table 1
Code-Based Seismic Design Criteria
Crescent City Inter-Agency Visitor Center

Latitude	41.7490° N
Longitude	-124.1812° W
Site Class	C
S_S	1.534
S_1	0.746
F_a	1.0
F_v	1.3
S_{MS}	1.534
S_{M1}	0.970
S_{DS}	1.023
S_{D1}	0.647
Occupancy Category	III
Seismic Design Category	E

6.4 Foundations

We have not been provided a foundation plan at the time of this report being prepared. The following foundation recommendations assume a two-story structure supported on continuous perimeter wall footings and isolated interior spread footings to support column loads, with a ground floor concrete slab. Building loads (dead plus long-term live) are assumed to be in the range of 2 to 3 kips per lineal foot for walls and up to 40 kips for columns.

1. To minimize potentially adverse affects from liquefaction-induced settlement across the site that may occur in response to infrequent upper-bound seismic events, we recommend that the foundations be structurally interconnected by a series of reinforced concrete grade beams. Columns should have relatively large, rigid spread foundations that are structurally integrated with the grade beam system to limit differential settlement potential. The floor slab should be reinforced and structurally integrated with the grade beam system. Grade beams should be tied (in a grid) to perimeter and column footings in both directions and be spaced appropriately for the loads (on the order of 15 to 20 feet on center). Spread footings (including grade beams) should be designed to span an unsupported length of 10 feet.
2. We recommend creating a working surface underlain by engineered fill material into which the grade beam foundation system will be embedded. Following site preparation as recommended, foundations may be constructed. Foundations should be sized, embedded, and reinforced to at least the minimums presented in the current edition of the CBC.

3. Foundations may be designed so they do not exceed an allowable bearing capacity of 2,500 pounds per square foot for dead plus long-term live loads. This value may be increased by one-third to account for the short-term effects of wind and/or seismic loading. The provided bearing values are applicable to engineered structural fill placed as recommended.
4. Lateral forces will be primarily resisted by the spread footings and grade beams embedded into the engineered fill material. A horizontal friction coefficient of 0.35 may be used for the footing/fill contact. An allowable lateral passive pressure represented by an equivalent fluid weighing 300 pounds per cubic foot can be used against the sides of the grade beams, beginning at 1 foot below the engineered fill surface, unless the ground is covered and confined by a concrete slab-on-grade or pavement.
5. Embedment depth should be determined starting at the surface of competent, undisturbed, native soils, or the surface of engineered structural fill placed as recommended above. Ignore surficial landscaping fill or flatwork area fills in determining minimum embedment depth.
6. Footing lines located adjacent and generally parallel to utility trenches should extend below a 1.5:1 plane projected upward from the bottom of the trench. Two sacks per cubic yard concrete slurry can be used beneath the regular reinforced concrete foundations to extend the foundations effectively deeper in this regard.
7. To provide adequate lateral support for foundations embedded into engineered structural fill, the engineered structural fill should extend horizontally beyond the exterior footing perimeters a minimum distance of 5 feet.
8. The ground surface around the structure perimeter should be sloped away, or other design measures implemented to provide positive surface water drainage away from perimeter foundation areas.

6.5 Floor Slabs

1. To limit water vapor transmission upward through floor slabs, the concrete floor slab where not supported on grade beams, should be constructed on a minimum 4-inch thick layer of compacted capillary break material. The capillary break material should be free-draining, clean gravel or rock (such as, No. 4 by ¾-inch pea gravel or permeable aggregate complying with the California Department of Transportation [Caltrans] Standard Specification, Section 68, Class 1, Type B Permeable Material). If a vapor retarder is used and placed over crushed rock or rough granular fill, a thin protective layer of approximately ½-inch layer of fine-graded, compactable material should be placed over the base prior to installation of the vapor retarder to reduce the possibility of puncture. In addition, we recommend that the vapor retarder be protected using a 3-inch-thick cover of compactable, granular fill, which will remain stable and support construction traffic. Sand is difficult, if not impossible, to compact and maintain until concrete placement is complete, and is not recommended. The vapor retarder should be installed in conformance with ASTM Test Method E1643, *Standard of Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs*. Where dampness or water vapor transmission through the slab is not objectionable, such as, for exterior slabs-on-grade, the capillary break material may be omitted and the slab may be constructed directly on the prepared subgrade or on a layer of compacted base rock.

2. It is important that the subgrade be moist and free of desiccation cracks at the time the slab is cast. Recommendations for slab reinforcement, strength, thickness, control and construction joints, etc., should be provided by others.

6.6 Drainage and Erosion Control

To facilitate site drainage and mitigate erosion potential, we recommend the following measures:

1. Final site grading should provide for surface drainage away from foundations. Grades should be sloped away from foundations a minimum gradient of five percent in landscaped areas and two percent in concrete areas for a horizontal distance of at least 10 feet.
2. Rainwater collected at the building roof levels should generally be transported through gutters, downspouts, and tightlines that discharge onto concrete or paved areas, or directly into the site stormwater system.
3. Wherever possible, design finished grade to allow sheet runoff rather than concentrated runoff. Where concentrated runoff will occur, minimize its velocity by controlling slopes, and protect the channel and discharge area by dissipating flow energy, using rock or other erosion resistant surfacing as appropriate.
4. Compact exposed fill slopes, and protect both cut and fill slopes from concentrated runoff or heavy sheet runoff by using brow ditches or other drainage control facilities.
5. Erodible cut or fill slopes, or other soil surfaces, should be protected by using vegetative cover, jute mesh and straw, rock slope protection, or other measures to provide erosion resistance.
6. Perform site work and vegetation establishment during seasons not subject to repeated or prolonged rainfall.
7. Provide periodic maintenance of erosion control measures.

6.7 Sidewalks and Other Flatwork Areas

The proposed project includes the construction of new sidewalks between the parking lot and new building. We expect that subgrade for flatwork areas will consist of structural fill in some areas and native soils in others.

1. Concrete slab and steel reinforcement should be designed for the anticipated loads.
2. In general, we recommend that flatwork be supported on a minimum of 4 inches of Class 2 base compacted to a minimum of 90 percent relative compaction per ASTM D 1557.

7.0 Additional Services

We suggest communications be maintained during the design phase between the design team and SHN to optimize compatibility between the design and soil and groundwater conditions. We also recommend that SHN be retained during the construction phase to verify the implementation of our recommendations related to earthwork.

7.1 Plan and Specification Review

We have assumed, in preparing our recommendations, that SHN will be retained to review those portions of the plans and specifications that pertain to earthwork and foundations. The purpose of this review is to confirm that our earthwork and foundation recommendations have been properly interpreted and implemented during design. If we are not provided this opportunity for review of the plans and specifications, our recommendations could be misinterpreted.

7.2 Construction Phase Monitoring

In order to assess construction conformance with the intent of our recommendations, it is important that a representative of SHN perform the following tasks:

1. Monitor site stripping, including removal of the undocumented fill material and buried topsoil, and any other unsuitable material if it is determined that this is required.
2. Monitor subgrade preparation.
3. Observe and test placement of structural fill and backfill.
4. Observe foundation excavations.

This construction phase monitoring is important as it provides the stakeholders and SHN the opportunity to verify anticipated site conditions, and recommend appropriate changes in design or construction procedures if site conditions encountered during construction vary from those described in this report. It also allows SHN to recommend appropriate changes in design or construction procedures if construction methods adversely affect the competence of on-site soils to support the structural improvements.

8.0 Closure and Limitations

The analyses, conclusions, and recommendations contained in this report are based on site conditions that we observed at the time of our investigation, data from our subsurface explorations and laboratory tests, our current understanding of proposed project elements, and on our experience with similar projects in similar geotechnical environments. We have assumed that the information obtained from our limited subsurface explorations is representative of subsurface conditions throughout the areas of proposed development addressed in this report.

We recommend that a representative of our firm confirm site conditions during the construction phase. If subsurface conditions differ significantly from those disclosed by our investigation, we should be given the opportunity to re-evaluate the applicability of our conclusions and recommendations. Some alteration of recommendations may be appropriate.

If the scope of the proposed construction, including the proposed loads, grades, or structural locations, changes from that described in this report, our recommendations should also be reviewed.

If there is a substantial lapse of time between the submission of our report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, we should review our report to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse. This report is applicable only to the project and site studied.

The conclusions and recommendations presented in this report are professional opinions derived in accordance with current standards of professional practice. Our recommendations are tendered on the assumption that design of the improvements will conform to their intent. No representation, express or implied, of warranty or guarantee is included or intended.

The field and laboratory work was conducted to investigate the site characteristics specifically addressed by this report. Assumptions about other site characteristics, such as, hazardous materials contamination, or environmentally sensitive or culturally significant areas, should not be made from this report.

9.0 References

American Society for Testing and Materials-International. (NR). *ASTM Test Method E1643, Standard of Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs*. NR:ASTM.

Bryant, W.A. and E.W. Hart. (2007). *Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act (with Index to Earthquake Fault Zones Maps): California Division of Mines and Geology Special Publication 42*. NR: California Department of Conservation, California Geological Survey.

California Building Standards Commission. (2010). *2010 California Building Code–Title 24 Part 2, Two-Volumes*. Based on International Building Code (2009) by the International Code Council. Sacramento:California Building Standards Commission.

California Geological Survey. (March 30, 2009). "Tsunami Inundation Map for Emergency Planning, State of California ~ County of Del Norte, Crescent City and Sister Rocks Quadrangle," NR:California Emergency Management Agency, California Geological Survey, and University of Southern California,.

---. (2010), "Fault Activity Map of California," California Geological Survey Geologic Data Map No. 6, compilation and interpretation by Charles W. Jennings and William A. Bryant; accessed at: http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html

Carver, G.A. (1987). "Late Cenozoic Tectonics of the Eel River Basin Region, Coastal Northern California," in Schymiczek, H. and Suchsland, R. (Eds): *Tectonics, Sedimentation and Evolution of the Eel River and Associated Coastal Basins of Northern California*. p. 61-72. Bakersfield: San Joaquin Geological Society.

Crow/Clay & Associates. (January 2013). "Site Plan." Coos Bay:Crow/Clay.

Dengler, L., R. McPherson, and G.A. Carver. (1992). "Historic Seismicity and Potential Source Areas of Large Earthquakes In North Coast California," in Burke, R.M. and G.A. Carver, (Eds), *Guidebook for the 1992 Friends of the Pleistocene Field Trip, Pacific Cell*, p. 112-118. NR:Friends of the Pleistocene.

Davenport, C. W. (1982), "Geology and Geomorphic Features Related to Landsliding, Crescent City 7.5-Minute Quadrangle, Del Norte County, California," Open-File Report 82-21, Department of Conservation, Division of Mines and Geology.

LACO Associates Consulting Engineers. (February 10, 2012). "Geotechnical Exploration, Proposed Promenade and Restroom Building, Crescent City Harbor District, Crescent City, California" prepared for Stover Engineering, unpublished Client report. Eureka:LACO.

Peterson, C. D., G. A. Carver, K. M. Cruikshank, H. F. Abramson, C. E. Garrison-Laney, L. A. Dengler. (June 15, 2011), "Evaluation of the use of Paleotsunami Deposits to Reconstruct Inundation Distance and Runup Heights Associated with Prehistoric Inundation Events, Crescent City, Southern Cascadia Margin," *Earth Surface Processes and Landforms, Vol. 36, Issue 7*, pages 967-980. Malden MA:John Wiley & Sons, Ltd.

Treadwell & Rollo. (June 13, 2011). "Geotechnical Investigation, Crescent City Harbor Rehabilitation, Crescent City, California," prepared for Stover Engineering, unpublished Client report. San Francisco:Treadwell & Rollo.

United States Geologic Survey. (February 10, 2011). "Seismic Hazard Curves, Response Parameters, Design Parameters: Seismic Hazard Curves, and Uniform Hazard Response Spectra," v. 5.1.0. NR:USGS.

---. (1966). "Sister Rocks 7.5-Minute Topographic Quadrangle." NR:USGS.

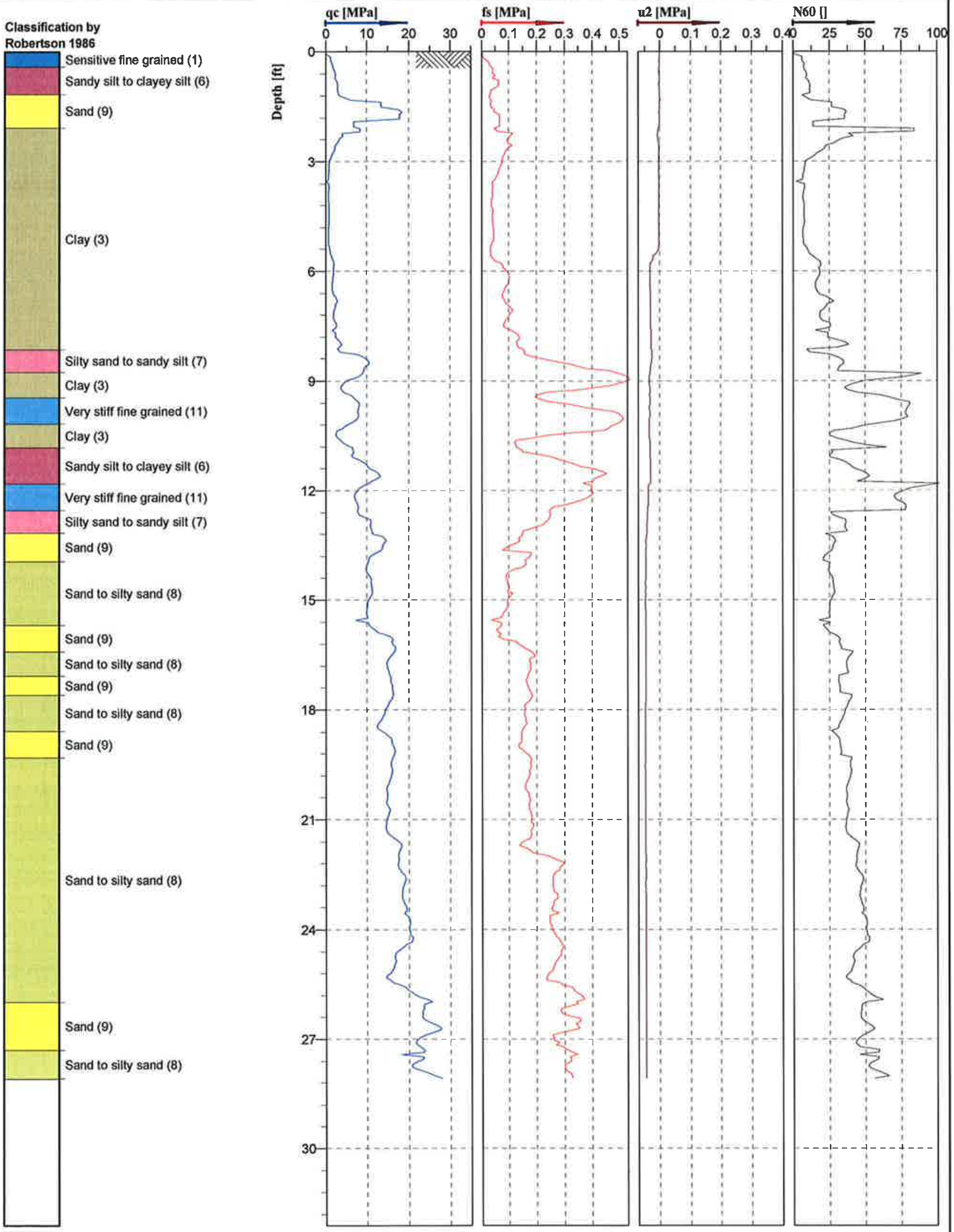
United States Geological Survey and California Geological Survey. (2010), "Quaternary Fault and Fold Database for the United States," accessed January 2013, from USGS web site: http://earthquakes.usgs.gov/regional/qfaults/

Youd, T.L., and D.M. Perkins. (1978). "Mapping of Liquefaction-Induced Ground Failure Potential," in *Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT4*, pp. 433-446. NR:ASCE.

Youd T.L. and I.M. Idriss. (October 2001). "Liquefaction Resistance of Soils; Summary Report from the 1995 NCEER and 1998 NCEER/NSF Workshops on the Evaluation of Liquefaction Resistance for Soils,: in *ASCE Geotechnical and Geoenvironmental Journal, vol. 127, No. 10*, p 817-833. Reston:ASCE.

Appendix A

Cone Penetration Test Logs



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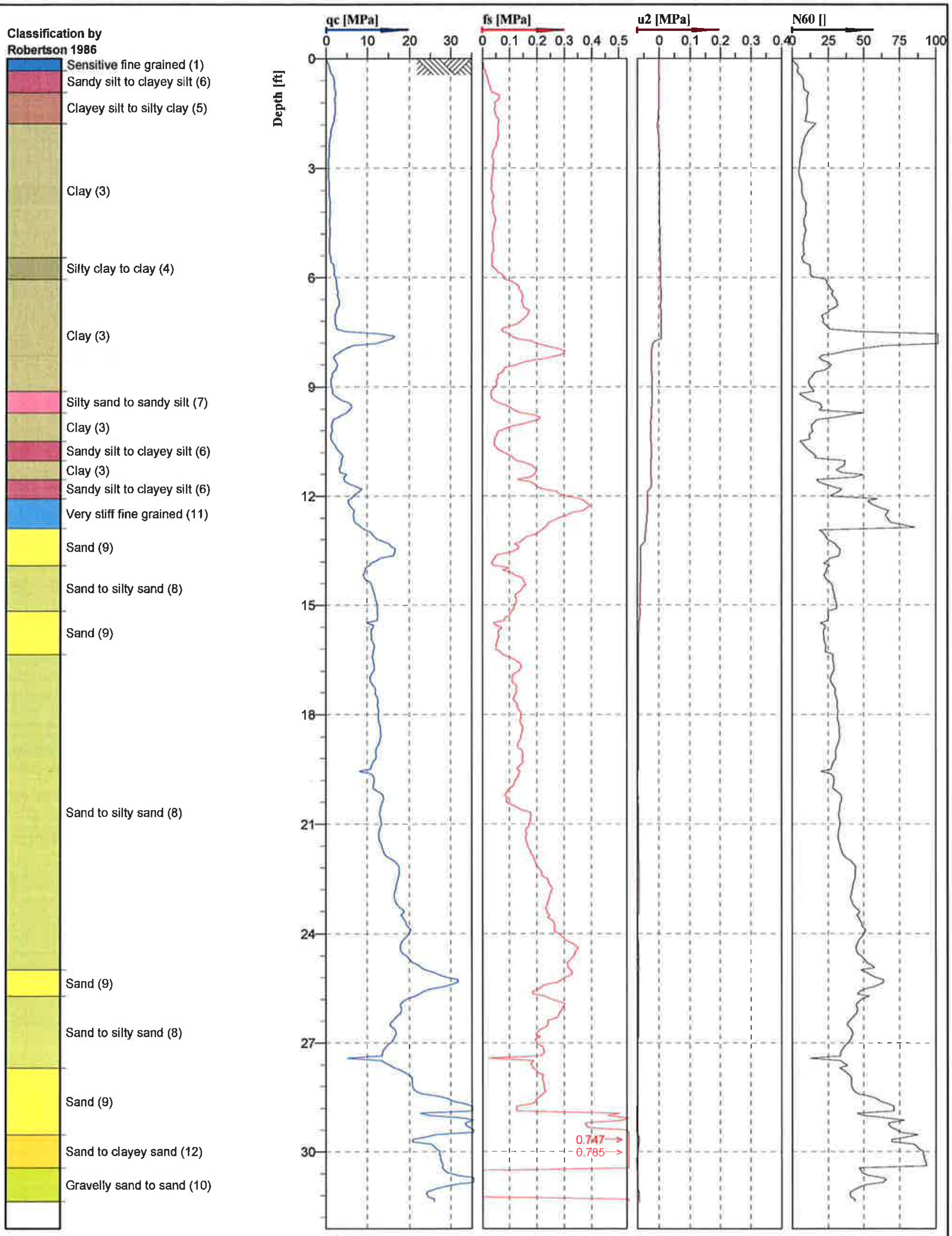
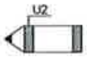
Position: X: 0.00 ft, Y: 0.00 ft
Ground level: 0.00
Test no: CPT1
Project ID: G3889
Client: SHN
Date: 11/13/2012
Scale: 1 : 45
Project: Creasent City IAVC
Page: 1/1
Fig:
File: IAVC CPT1.cpt

Fisch Drilling

U2

Cone No: 3335
Tip area [cm2]: 10
Sleeve area [cm2]: 150





Fisch Drilling

Cone No: 3335
Tip area [cm2]: 10
Sleeve area [cm2]: 150

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Project ID: G3889	Client: SHN	Date: 11/13/2012	Scale: 1 : 45
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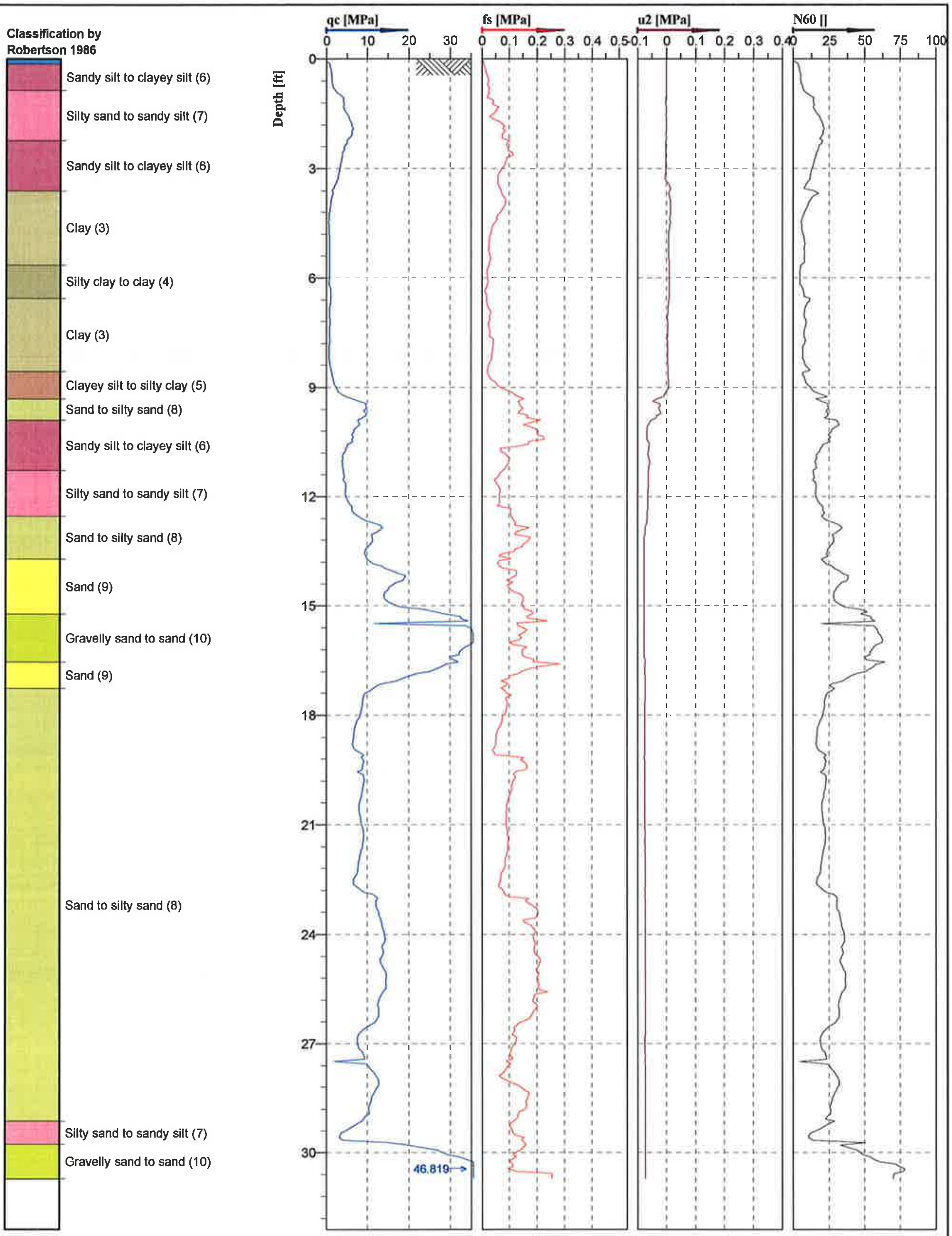
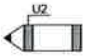
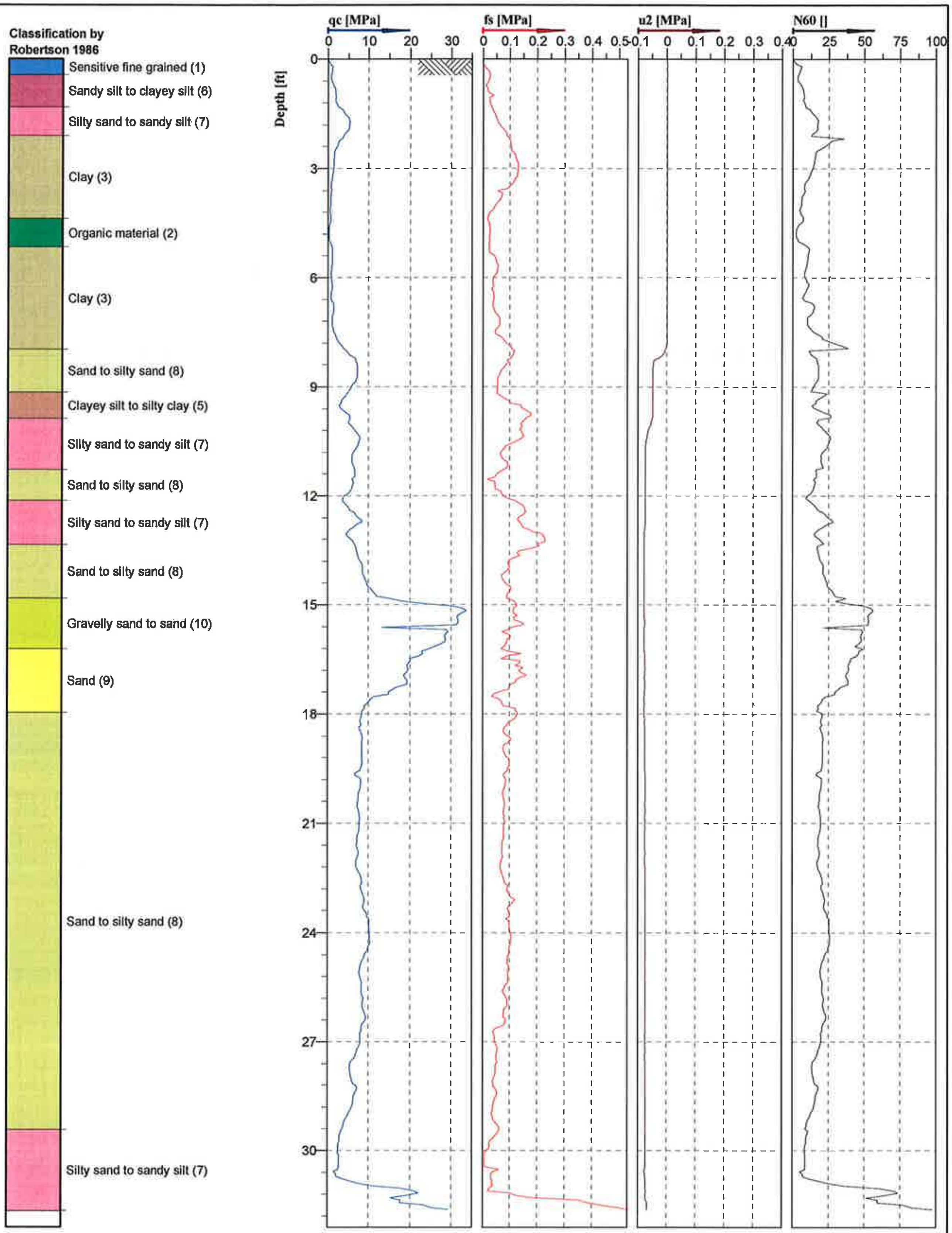


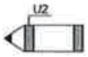
Fisch Drilling



Cone No: 3335
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Cresent City, Ca	Position: X: 0.00 ft, Y: 0.00 ft	Ground level: 0.00	Test no: CPT3
Project ID: G3889	Client: SHN	Date: 11/13/2012	Scale: 1 : 45
Project: Cresent City IAVC		Page: 1/1	Fig:
		File: IAVC CPT3.cpt	





Fisch Drilling

Cone No: 3335
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Creasent City, Ca	Position: X: 0.00 ft, Y: 0.00 ft	Ground level: 0.00	Test no: CPT4
Project ID: G3889	Client: SHN	Date: 11/13/2012	Scale: 1 : 45
Project: Creasent City IAVC		Page: 1/1	Fig:
		File: IAVC CPT4.cpt	

Appendix B

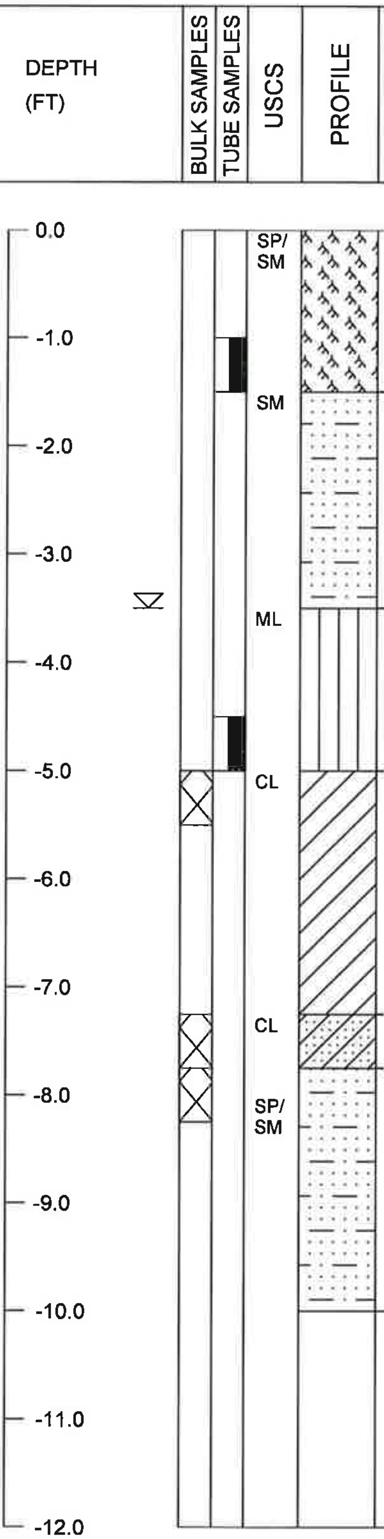
Test Pit Logs

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Inter-Agency Visitors Center
LOCATION: Crescent City, CA
GROUND SURFACE ELEVATION: 14 Feet (Project Datum)
EXCAVATION METHOD: Backhoe
LOGGED BY: PRS

JOB NUMBER: 012226
DATE EXCAVATED: 11/13/12
TOTAL DEPTH OF TEST PIT: 10 Feet
SAMPLER TYPE: 2.5" O.D. brass tube

TEST PIT NUMBER
TP-1



USCS	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	U.C. (psf) by P.P.	% Passing 200	REMARKS
SP/ SM	FILL; Sand w/Silt, Very dark brown, moist, loose, approximately 10% gravel, roots and grass. Becomes medium dense.	20	95				
SM	SILTY SAND; Very dark brown, moist, loose.						
ML	SILT w/SAND; Yellowish-brown, wet, medium stiff.	28	93			86	Battery Fm.
CL	CLAY; Light yellowish-brown to strong brown (mottled), moist, medium stiff to stiff.						
CL	SANDY CLAY; Bluish-gray, moist, stiff.						
SP/ SM	POORLY GRADED SAND w/SILT; Dark yellowish-brown, wet, medium dense.						
	Excavation terminated at a depth of 10 feet. Groundwater encountered at a depth of 3.5 feet. Test pit backfilled with soil and compacted with bucket.						

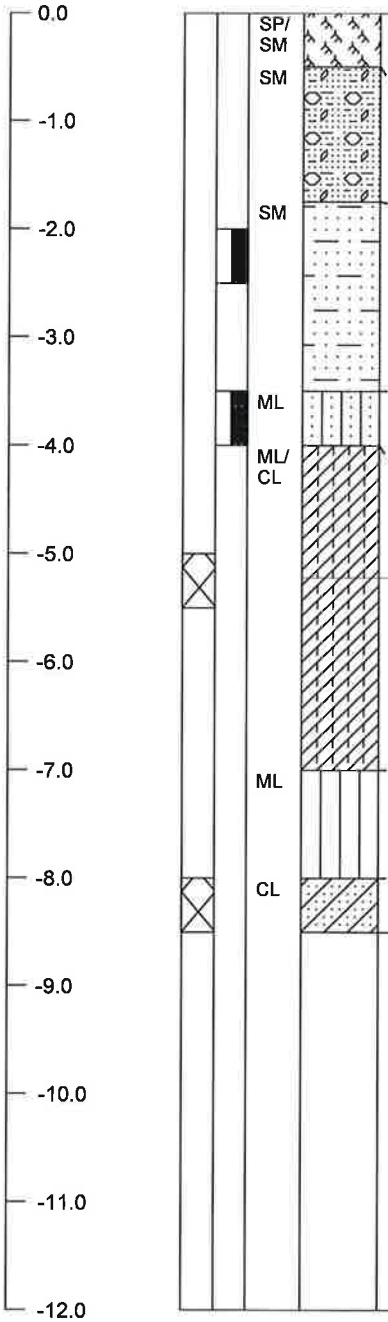
LOG OF TEST PIT

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Inter-Agency Visitors Center
LOCATION: Crescent City, CA
GROUND SURFACE ELEVATION: 14 Feet (Project Datum)
EXCAVATION METHOD: Backhoe
LOGGED BY: PRS

JOB NUMBER: 012226
DATE EXCAVATED: 11/13/12
TOTAL DEPTH OF TEST PIT: 8.5 Feet
SAMPLER TYPE: 2.5" O.D. brass tube

TEST PIT NUMBER
TP-2



USCS	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	U.C. (psf) by P.P.	% Passing 200	REMARKS
SP/ SM	FILL; Sand w/Silt, Very dark brown, moist, medium dense.						
SM	FILL; Gravelly Silty Sand, Very dark brown, moist, loose to medium dense, approximately 30% gravel, contains roots.						
SM	SILTY SAND; Dark yellowish-brown, moist, loose.	29	96				
ML	SANDY SILT; Dark yellowish-brown, moist, medium stiff.	38	81			80	
ML/ CL	CLAY/SILT; Yellowish-brown, moist, medium stiff.						Battery Fm.
ML	SILT w/SAND; Yellowish-brown (mottled), moist, medium stiff.						
CL	SANDY CLAY; Bluish-gray, moist, stiff.						
	Excavation terminated at a depth of 8.5 feet. Groundwater not encountered. Test pit backfilled with soil and compacted with bucket.						

LOG OF TEST PIT

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Inter-Agency Visitors Center
LOCATION: Crescent City, CA
GROUND SURFACE ELEVATION: 15 Feet (Project Datum)
EXCAVATION METHOD: Backhoe
LOGGED BY: PRS

JOB NUMBER: 012226
DATE EXCAVATED: 11/13/12
TOTAL DEPTH OF TEST PIT: 10 Feet
SAMPLER TYPE: 2.5" O.D. brass tube

TEST PIT NUMBER
TP-3

DEPTH (FT)	BULK SAMPLES	TUBE SAMPLES	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	U.C. (psf) by P.P.	% Passing 200	REMARKS
0.0			SP/ SM		FILL; Sand w/Silt, Very dark brown, moist, loose to medium dense, roots and grass.						
-1.0			GW		FILL; Well-Graded Gravel w/Sand, Dark gray, moist, medium dense.						
-2.0		■	SM		SILTY SAND; Dark brown, moist, loose.	26	89				
-3.0			ML		SANDY SILT; Very dark brown, moist, soft.						
			ML		SILT; Strong brown, moist, medium stiff.						Battery Fm.
-4.0											
-5.0		■			Becomes wet to saturated. Grades yellowish-brown.	35	86				
-6.0											
-7.0			SM/ ML		SILTY SAND/SANDY SILT; Yellowish-brown, saturated, soft/loose.						
-8.0											
▼											
-9.0											
			GP		POORLY GRADED GRAVEL w/SAND; Gray, saturated, loose to medium dense.						
-10.0											
					Excavation terminated at a depth of 10.5 feet. Groundwater encountered at a depth of 9.5 feet. Test pit backfilled with soil and compacted with bucket.						
-11.0											
-12.0											

LOG OF TEST PIT

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Inter-Agency Visitors Center
LOCATION: Crescent City, CA
GROUND SURFACE ELEVATION: 15 Feet (Project Datum)
EXCAVATION METHOD: Backhoe
LOGGED BY: PRS

JOB NUMBER: 012226
DATE EXCAVATED: 11/13/12
TOTAL DEPTH OF TEST PIT: 9.75 Feet
SAMPLER TYPE: 2.5" O.D. brass tube

TEST PIT NUMBER
TP-4

DEPTH (FT)	BULK SAMPLES	TUBE SAMPLES	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	U.C. (psf) by P.P.	% Passing 200	REMARKS
0.0 -1.0			SP/SM		FILL; Sand w/Silt, Very dark brown, moist, loose to medium dense, grass and roots.						
-2.0		■	SM		SILTY SAND; Very dark brown, moist, loose.	39	83				
			ML		SILT w/SAND; Dark brown, moist, soft.						
-3.0 -4.0 -5.0			ML		SILT; Strong brown, moist, soft to medium stiff.						Battery Fm.
-6.0 -7.0					Grades yellowish-brown						
-8.0 ▼ -9.0			GP		POORLY GRADED GRAVEL w/SAND; saturated, medium dense.						
-10.0 -11.0 -12.0					Excavation terminated at a depth of 9.75 feet. Groundwater encountered at a depth of 8.0 feet. Test pit backfilled with soil and compacted with bucket.						

LOG OF TEST PIT

SHN CONSULTING ENGINEERS
& GEOLOGISTS

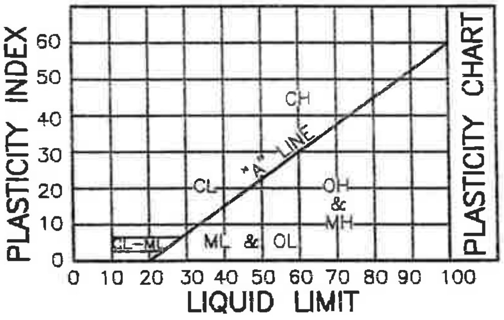
METHOD OF SOIL CLASSIFICATION

MAJOR DIVISIONS		SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (MORE THAN 1/2 OF SOIL >NO. 200 SIEVE SIZE)	GRAVELS (MORE THAN 1/2 OF COARSE FRACTION > NO.4 SIEVE SIZE)	GW	WELL GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
		GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (MORE THAN 1/2 OF COARSE FRACTION < NO.4 SIEVE SIZE)	SW	WELL GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SM	SILTY SANDS, SAND-SILT MIXTURES
		SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS (MORE THAN 1/2 OF SOIL <NO. 200 SIEVE SIZE)	SILTS & 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 & CLAYS LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTY CLAYS, ORGANIC SILTS
HIGHLY ORGANIC SOILS		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

CLASSIFICATION CHART

CLASSIFICATION	U.S. STANDARD SIEVE SIZE
BOULDERS	ABOVE 12"
COBBLES	12" TO 3"
GRAVEL	3" TO NO. 4
COARSE	3" TO 3/4"
FINE	3/4" TO NO. 4
SAND	NO. 4 TO NO. 200
COARSE	NO. 4 TO NO. 10
MEDIUM	NO. 10 TO NO. 40
FINE	NO. 40 TO NO. 200
SILT & CLAY	BELOW NO. 200

GRAIN SIZE CHART



PLASTICITY CHART

CONSISTENCY OF FINE GRAINED SOILS		DENSITY OF COARSE GRAINED SOILS	
CLASSIFICATION	COHESION (PSF)	CLASSIFICATION	STANDARD PENETRATION (BLOW COUNT)
VERY SOFT	0-250	VERY LOOSE	0-4
SOFT	250-500	LOOSE	4-10
MEDIUM STIFF	500-1000	MEDIUM	10-30
STIFF	1000-2000	DENSE	30-50
VERY STIFF	2000-4000	VERY DENSE	50+
HARD	4000+		

MOISTURE CLASSIFICATIONS
DRY
DAMP
MOIST
WET

BASED ON UNIFIED SOILS CLASSIFICATION SYSTEM

SHN CONSULTING ENGINEERS
& GEOLOGISTS

BORING LOG KEY

SAMPLE TYPES

☒	DISTURBED SAMPLE (BULK)
▮	HAND DRIVEN TUBE SAMPLE
⌶	1.4" I.D. STANDARD PENETRATION TEST SAMPLE (SPT)
◩	2.5" I.D. MODIFIED CALIFORNIA SAMPLE (SOLID WHERE RETAINED)
▯	CORE BARREL SAMPLE (NOT RETAINED)
◪	CORE BARREL SAMPLE (RETAINED)

SYMBOLS

▽	INITIAL WATER LEVEL
▼	STABILIZED WATER LEVEL
- - - - -	GRADATIONAL CONTACT
———	WELL DEFINED CONTACT
SS	SPLIT SPOON

Appendix C

Laboratory Test Data

CONSULTING ENGINEERS & GEOLOGISTS, INC.

812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

PERCENT PASSING # 200 SIEVE (ASTM - D1140)

Project Name: IVAC **Project Number:** 012226
Performed By: JMA **Date:** 12/5/2012
Checked By: Dh **Date:** 12/10/12
Project Manager: JPB

Lab Sample Number	12-851	12-853			
Boring Label	TP1	TP2			
Sample Depth	4.5-5	3.5-4			
Pan Number	ss8	ss10			
Dry Weight of Soil & Pan	313.7	302.2			
Pan Weight	193.0	196.2			
Weight of Dry Soil	120.7	106.0			
Soil Weight Retained on #200&Pan	209.5	217.5			
Soil Weight Passing #200	104.2	84.7			
Percent Passing #200	**86.3**	**79.9**			

Lab Sample Number					
Boring Label					
Sample Depth					
Pan Number					
Dry Weight of Soil & Pan					
Pan Weight					
Weight of Dry Soil					
Soil Weight Retained on #200&Pan					
Soil Weight Passing #200					
Percent Passing #200					

SHN CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

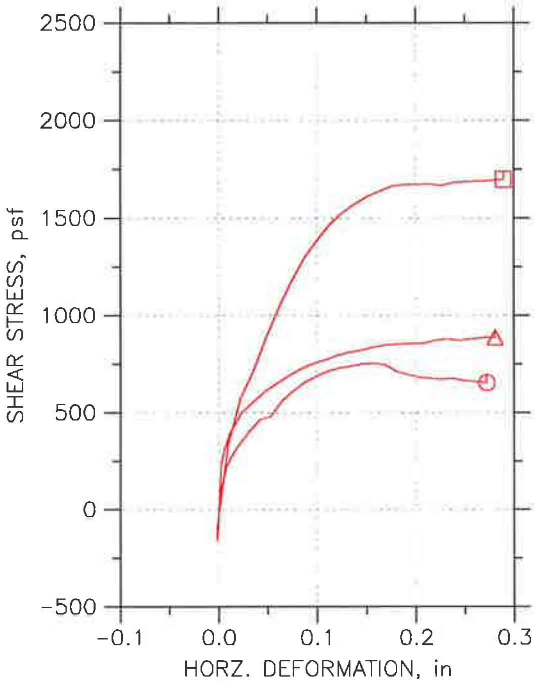
DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

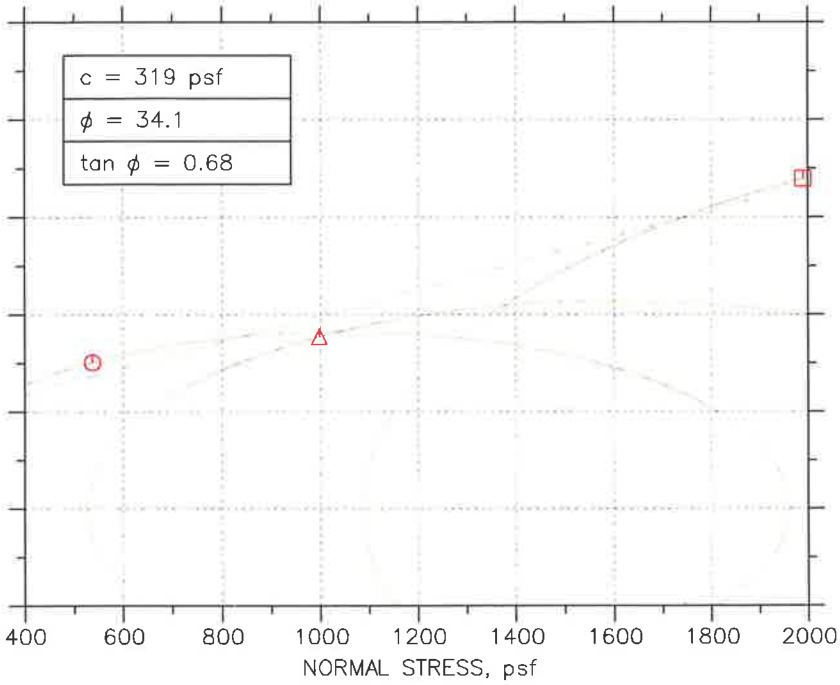
Project Name:	**IVAC**	**Project Number:**	**012226**
Performed By:	**JMA**	**Date:**	**11/30/2012**
Checked By:	DL	**Date:**	12/10/12
Project Manager:	**JPB**		

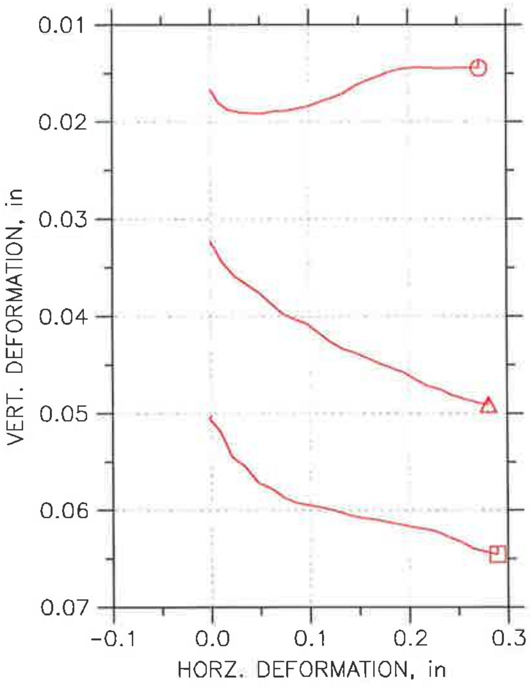
Lab Sample Number	12-850	12-851	12-854	12-855	
Boring Label	TP1	TP1	TP3	TP3	
Sample Depth (ft)	1-1.5	4-4.5	2-2.5	5-5.5	
Diameter of Cylinder, in	2.38	2.38	2.38	2.38	
Total Length of Cylinder, in.	8.00	8.00	7.20	7.90	
Length of Empty Cylinder A, in.	2.70	0.25	0.00	0.00	
Length of Empty Cylinder B, in.	0.00	3.15	1.95	2.90	
Length of Cylinder Filled, in	5.30	4.60	5.25	5.00	
Volume of Sample, in^3	23.58	20.46	23.36	22.24	
Volume of Sample, cc.	386.38	335.35	382.74	364.51	

Pan #	ss9	ss6	ss8	ss10	
Weight of Wet Soil and Pan	904.6	838.5	881.4	874.5	
Weight of Dry Soil and Pan	785.2	698.0	740.1	696.8	
Weight of Water	119.4	140.5	141.3	177.7	
Weight of Pan	196.6	196.3	193.1	195.5	
Weight of Dry Soil	588.6	501.7	547.0	501.3	
Percent Moisture	20.3	28.0	25.8	35.4	
Dry Density, g/cc	1.52	1.50	1.43	1.38	
Dry Density, lb/ft^3	95.1	93.4	89.2	85.9	

DIRECT SHEAR TEST REPORT







	Symbol	○	△	□	
	Test No.	12-852	12-853	12-856	
	Sample No.	TP2@2'	TP2@3.5'	TP4@2'	
	Shape	Circular	Circular	Circular	
Initial	Dimension, in	2.415	2.415	2.415	
	Area, in^2	4.5806	4.5806	4.5806	
	Height, in	1	1	1	
	Water Content, %	28.57	38.34	38.85	
	Dry Density, pcf	95.88	81.27	83.83	
	Saturation, %	104.34	98.12	105.77	
	Void Ratio	0.72551	1.0356	0.97338	
	Consol. Height, in	0.98333	0.96776	0.94971	
	Consol. Void Ratio	0.69674	0.96996	0.87414	
Final	Water Content, %	27.04	35.16	33.80	
	Dry Density, pcf	97.28	85.47	89.62	
	Saturation, %	102.27	99.59	105.88	
	Void Ratio	0.70059	0.93564	0.84594	
	Normal Stress, psf	537.88	998.13	1990.7	
	Max. Shear Stress, psf	755.35	886.88	1698.6	
	Ult. Shear Stress, psf	653.89	886.88	1698.6	
	Time to Failure, min	7.0038	11.004	12.004	
	Disp. Rate, in/min	0.025	0.025	0.025	
	Estimated Specific Gravity	2.65	2.65	2.65	
	Liquid Limit	0	0	0	
	Plastic Limit	0	0	0	
	Plasticity Index	0	0	0	

Project: IAVC

Location: Crescent City

Project No.: 012226

Boring No.: TP2 & TP4

Sample Type: 2.5" Shelby

Description: Dark Brown Silty SAND

Remarks: Chck DH 12/4/12

Tue, 04-DEC-2012 13:39:22

Appendix D

Boring Logs (Treadwell & Rollo, 2011)

PROJECT: **CRESCENT CITY HARBOR REHABILITATION**
Crescent City, California

Log of Boring B-1

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 11/4/09

Date finished: 11/5/09

Drilling method: Hollow Stem Auger and Rotary Wash

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Safety Wireline & Auto

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), HQ Core Barrel (C)

Logged by: R Severn

LABORATORY TEST DATA

Ground Surface Elevation: 13.5 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						3-inches Asphaltic Concrete 3-inches Aggregate Base						
1						SAND (SP) brown, moist						
5–6	S&H		6 6 11	10		olive-gray, loose to medium dense, fine- to medium-grained sand, trace fines, trace shell fragments					10.8	83
10–11	SPT		17 23 31	54		olive-gray, very dense, wet, fine- to medium-grained sand, trace shell fragments, gravel, and fines Corrosion Test, see Figure B-5						
15–16	SPT		10 19 27	46	SP ▽	(11/04/09, 4:29 pm) dense						
20	SPT		50 50/ 1/2"	50/ 1/2"		olive-brown and red-brown, very dense						
25–26	SPT		35 50 50/ 2"	100/ 8"		olive-gray with some orange staining, trace fine- to coarse-grained, rounded gravel and trace coarse-grained, rounded sand						
30												

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 730500101

Figure: A-1a

PROJECT: **CRESCENT CITY HARBOR REHABILITATION**
Crescent City, California

Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		32 50/ 1"	50/ 1"	SP	SAND (SP) (continued) olive-gray						
35–36	S&H		24 34 36	42		MUDSTONE olive-gray, crushed to intensely fractured, soft to low hardness, plastic to weak						
37–38	S&H		50/ 3"	30/ 3"								
40–41						RQD = 0 Drill rate = 0.54 feet/min						
42–43	C											
45	SPT		50/ 5"	60/ 5"		light olive-gray, soft to low hardness, plastic to friable RQD = 0 Drill rate = 0.53 feet/min						
48	C											
51	SPT		23 50/ 3"	60/ 3"		gray, plastic to weak Triaxial Test, see Figure B-3	TxUU	2,250	910		24.1	104

BEDROCK (35 to 51.5 feet)

Boring terminated at a depth of 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 15.1 feet below ground surface during drilling.
RQD = rock quality designation

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively for the safety hammer and 0.8 and 1.2, respectively for the automatic hammer to account for sampler type and hammer energy.

[2] Elevations based on Mean Low Low Water Datum.

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 730500101

Figure: A-1b

PROJECT: **CRESCENT CITY HARBOR REHABILITATION**
Crescent City, California

Log of Boring B-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 11/5/09

Date finished: 11/5/09

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Automatic

Sampler: Standard Penetration Test (SPT), HQ Core Barrel (C)

Logged by: R Severn

LABORATORY TEST DATA

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES Blows/ 6"	SAMPLES SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						Ground Surface Elevation: 13 feet[2]						
						6-inches Asphaltic Concrete						
1						4-inches Aggregate Base						
2						SAND with SILT (SP-SM) olive-brown, medium dense, moist to wet, fine- to medium-grained sand, trace shell and timber fragments						
3												
4												
5												
6	SPT		7 10 7	20		lig.?				9.8		
7					SP-SM							
8												
9												
10												
11	SPT		3 3 3	7		dark gray, loose, trace gravel, shell fragments, and organics (plant fibers) Particle Size Analysis, see Figure B-1				7.4		
12												
13						SAND (SP) gray, dense, wet, with abundant shell fragments and fine- to coarse-grained sand and subrounded to angular gravel > 2-inches diameter at 15 feet (possible slough)						
14												
15												
16	SPT		10 13 15	34								
17												
18												
19												
20					SP							
21	SPT		11 14 17	37		olive-gray, with coarse-grained sand and shell fragments at 20 feet (possible slough)						
22												
23												
24												
25												
26	SPT		9 9 13	26		olive, medium dense, trace fines, with coarse-grained sand and shell fragments at 25 feet (possible slough)						
27						SAND with GRAVEL (SP) olive-gray, dense, wet, with fine- to coarse-grained angular to subangular gravel, abundant shell fragments, and trace fines						
28					SP							
29												
30												

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 730500101

Figure: A-2a

PROJECT: **CRESCENT CITY HARBOR REHABILITATION**
Crescent City, California

Log of Boring B-2

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	SPT		15 16 13	35		SAND with GRAVEL (SP) (continued)						
32					SP							
33												
34												
35	SPT		22 50/ 6"	60/ 6"		MUDSTONE olive-gray, crushed to closely fractured, soft to low hardness, friable to weak, moderate to deep weathering						
36												
37												
38												
39												
40	SPT		50/ 6"	60/ 6"		gray, intensely to closely fractured, low hardness, weak, with fine-grained sand Triaxial Test, see Figure B-4	TxUU	1,850	3,070		22.5	107
41												
42						RQD = 0 BEDROCK						
43	C					Drill rate = 0.20 feet/min						
44												
45						soft to low hardness, plastic to friable						
46						RQD = 0						
47						Drill rate = 0.13 feet/min						
48	C											
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 50.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.
RQD = rock quality designation

---

[1] SPT blow counts for the last two increments were converted to SPT N-Values using a factor of 1.2, to account for sampler type and hammer energy.

[2] Elevations based on Mean Low Low Water Datum.

Treadwell&Rollo
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Project No.: 730500101

Figure: A-2b

PROJECT: **CRESCENT CITY HARBOR REHABILITATION**
Crescent City, California

Log of Boring B-3

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 11/6/09
Date finished: 11/6/09
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Standard Penetration Test (SPT), HQ Core Barrel (C)
Logged by: R Severn

LABORATORY TEST DATA

	SAMPLES					MATERIAL DESCRIPTION						
DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	Ground Surface Elevation: 12 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						3-inches Asphaltic Concrete 4-inches Aggregate Base						
1						SAND (SP) olive-brown, medium dense, moist, fine-grained sand, with trace silt						
5–6	SPT		4 7 8	18	SP							
10–11	SPT		8 10 12	26		▽ (11/06/09, 12:48 pm) dark gray, wet, with coarse-grained sand to fine-grained gravel and shells at 10 feet, grades to olive-brown at 11 feet						
13						SAND with SILT (SP-SM) olive with orange staining, medium dense, wet						
15–16	SPT		5 5 7	14	SP-SM	Particle Size Analysis, see Figure B-1				12.0		
18						SAND (SP) olive-brown with orange staining, medium dense, wet, with coarse-grained sand to fine-grained rounded to subangular gravel from 20.5 feet to 21 feet and interbedded layers of SAND with SILT (SP-SM)						
20–21	SPT		4 5 9	17								
23					SP							
25–26	SPT		11 13 12	30		olive-brown to olive-gray, medium dense to dense, fine-grained rounded gravel from 25.5 feet to 26 feet, with intermittent thin bands of staining						
28–30						MUDSTONE (see next page for description) BEDROCK						

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Project No.: 730500101
Figure: A-3a

PROJECT: **CRESCENT CITY HARBOR REHABILITATION**
Crescent City, California

Log of Boring B-3

PAGE 2 OF 2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value¹	LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		50/ 6"	60/ 6"		MUDSTONE (continued) olive-gray, crushed, low hardness, friable, moderately to deeply weathered, with fine-grained sand in rock matrix	BEDROCK						
32	C					RQD = 0							
33			10			Drill rate = 0.31 feet/min							
34	SPT		24 50/ 4"	60/ 4"		low to moderate hardness, friable to weak, with occasional fragments of chert							
35						RQD = 0							
36						Drill rate = 0.92 feet/min							
37	C												
38													
39						low hardness, friable							
40	SPT												
41													
42													
43													
44													
45													
46													
47													
48													
49													
50													
51													
52													
53													
54													
55													
56													
57													
58													
59													
60													

Boring terminated at a depth of 40 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 10.4 feet below ground surface during drilling.
RQD = rock quality designation

[1] SPT blow counts for the last two increments were converted to SPT N-Values using a factor of 1.2, to account for sampler type and hammer energy.

[2] Elevations based on Mean Low Low Water Datum.

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Project No.: 730500101
Figure: A-3b

PROJECT: **CRESCENT CITY HARBOR REHABILITATION**
Crescent City, California

Log of Boring B-4

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 11/6/09
Date finished: 11/7/09
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Standard Penetration Test (SPT), HQ Core Barrel (C)
Logged by: R Severn

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 10 feet[2]	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1						WATER						
2												
3												
4												
5												
6												
7												
8												
9												
10												
11												
12												
13												
14												
15												
16												
17												
18												
19					ML-OL	SANDY SILT (ML-OL) dark gray, wet, with organics, strong odor						
20						SAND (SP) dark gray grading to olive-brown, dense, wet, fine-grained sand with trace silt						
21	SPT	◪	25 21 9	36								
22												
23												
24					SP							
25												
26	SPT	◪	8 15 11	31		olive-gray to dark gray, dense, fine-to medium-grained sand, with shell fragments						
27												
28												
29						SANDSTONE olive-brown to olive-gray, low hardness, weak	UC	N/A	12,260		17.3	107
30	C											

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 730500101
Figure: A-4a

PROJECT: **CRESCENT CITY HARBOR REHABILITATION**
Crescent City, California

Log of Boring B-4

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	C					SANDSTONE (continued) Unconfined Compression Test, see Figure B-2 RQD = 0 Drill rate = 0.56 feet/min	BEDROCK						
32													
33						MUDSTONE olive-gray, soft to low hardness, plastic to friable, with fine-grained sand in matrix RQD = 0 Drill rate = 1.23 feet/min							
34	C												
35													
36													
37	SPT		11 50/ 5"	60/ 5"									
38													
39													
40													
41													
42													
43													
44													
45													
46													
47													
48													
49													
50													
51													
52													
53													
54													
55													
56													
57													
58													
59													
60													

Boring terminated at a depth of 37.4 feet below water surface.
Boring backfilled with bentonite chips.
RQD = rock quality designation

---

[1] SPT blow counts for the last two increments were converted to SPT N-Values using a factor of 1.2, to account for sampler type and hammer energy.

[2] Elevations based on Mean Low Low Water Datum.

Treadwell&Rollo
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Project No.: 730500101
Figure: A-4b

TEST GEOTECH LOG 5001.01.GPJ TR.GDT 6/10/11

PROJECT: **CRESCENT CITY HARBOR REHABILITATION**
Crescent City, California

Log of Boring B-5

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Logged by: R Severn

Date started: 11/7/09

Date finished: 11/7/09

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Automatic

Sampler: Standard Penetration Test (SPT), HQ Core Barrel (C)

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Water Surface Elevation: 10 feet[2]	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1												
2												
3												
4												
5												
6												
7												
8												
9												
10						WATER						
11												
12												
13												
14												
15												
16												
17												
18												
19												
20	SPT		0 1 9	12	ML-OL	SANDY SILT (ML-OL) black, very soft, wet, with organics, strong odor						
21					SP	SAND (SP) dark gray, medium dense, wet, fine-grained sand						
22	SPT		20 50/4"	60/4"		SANDSTONE olive-gray and yellow-brown with red-brown mottling, low hardness, friable, moderate to deep weathering, weakly cemented RQD = 0 Drill rate = 0.17 feet/min						
23												
24												
25	C					BEDROCK						
26						MUDSTONE gray, low hardness, friable to weak, with fine-grained sand in rock matrix						
27												
28												
29	C											
30												

Treadwell&Rollo
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Project No.: 730500101

Figure: A-5a

PROJECT: **CRESCENT CITY HARBOR REHABILITATION**
Crescent City, California

Log of Boring B-5

PAGE 2 OF 2

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						MUDSTONE (continued) BEDROCK						
31	C	•										
32												
33												
34												
35												
36												
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 33 feet below water surface.
Boring backfilled with bentonite chips.
RQD = rock quality designation

[1] SPT blow counts for the last two increments were converted to SPT N-Values using a factor of 1.2, to account for sampler type and hammer energy.

[2] Elevations based on Mean Low Low Water Datum.

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Project No.: 730500101

Figure: A-5b

PROJECT: **CRESCENT CITY HARBOR REHABILITATION**
Crescent City, California

Log of Boring B-6

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: R Severn

Date started: 11/8/09

Date finished: 11/8/09

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Automatic

Sampler: Standard Penetration Test (SPT), HQ Core Barrel (C)

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Water Surface Elevation: 3.5 feet[2]		LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1													
2													
3													
4													
5													
6													
7							WATER						
8													
9													
10													
11													
12													
13													
14													
15			11		ML-OL	SANDY SILT (ML-OL) black, very soft, wet, with organics, strong odor							
16	SPT		15 15	36	SP	SAND (SP) dark gray to olive-brown, dense, wet, fine- to coarse-grained sand, with gravel and shell fragments, trace fines							
17													
18						SILTSTONE/MUDSTONE olive-gray, low hardness, friable, moderate weathering, with SANDSTONE cobbles and gravel RQD = 0 Drill rated = 0.32 feet/min							
19	C												
20													
21						MUDSTONE olive-gray, low hardness, friable to weak, with sand in rock matrix RQD = 0 Drill rate = 0.21 feet/min increased sand content in rock matrix							
22							BEDROCK						
23	C												
24													
25													
26	C					RQD = 0 Drill rate = 0.09 feet/min							
27													
28													
29													
30													

Boring terminated at a depth of 27.5 feet below water surface.
Boring backfilled with bentonite chips.
RQD = rock quality designation

[1] SPT blow counts for the last two increments were converted to SPT N-Values using a factor of 1.2, to account for sampler type and hammer energy.

[2] Elevations based on Mean Low Low Water Datum.

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Project No.: 730500101

Figure: A-6

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	**Grain Size in Millimeters**
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⊟	Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
⧄	Classification sample taken with Standard Penetration Test sampler
◫	Undisturbed sample taken with thin-walled tube
⊠	Disturbed sample, hand auger
⊙	Sampling attempted with no recovery
◫	Core sample
●	Analytical laboratory sample
▥	Sample taken with Direct Push sampler

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

CRESCENT CITY HARBOR REHABILITATION
Crescent City, California

Treadwell&Rollo
A LANGAN COMPANY

CLASSIFICATION CHART

Date 11/13/09 | Project No. 730500101 | Figure A-7

I FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

II HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

III STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

IV WEATHERING - The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.

M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.

F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

V CONSOLIDATION OF SEDIMENTARY ROCKS: usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

VI BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated

CRESCENT CITY HARBOR REHABILITATION
Crescent City, California

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PHYSICAL PROPERTIES CRITERIA FOR ROCK DESCRIPTIONS

Date 11/13/09 | Project No. 730500101 | Figure A-8

Appendix E

Liquefaction Analysis Reports

GeoLogismiki
Geotechnical Engineering Software
Merarhias 56, 621 25 - Serrai, Greece
url: http://www.geologismiki.gr - email: info@geologismiki.gr

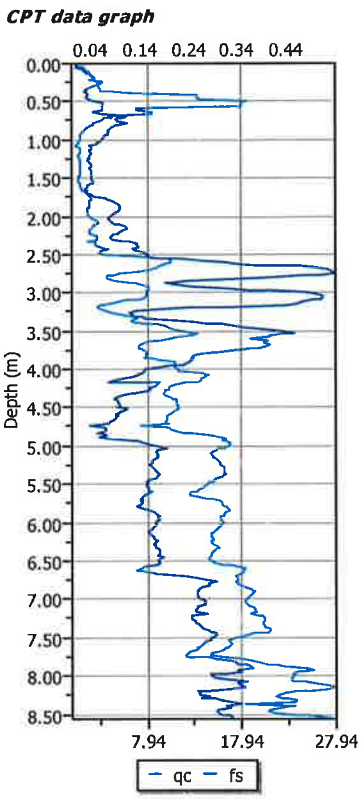
LIQUEFACTION ANALYSIS REPORT

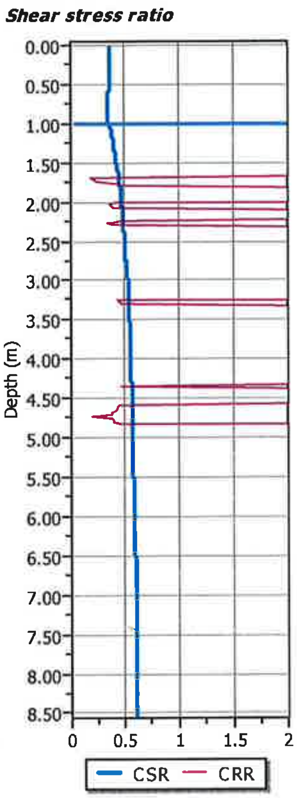
Project title : Crescent City IAVC

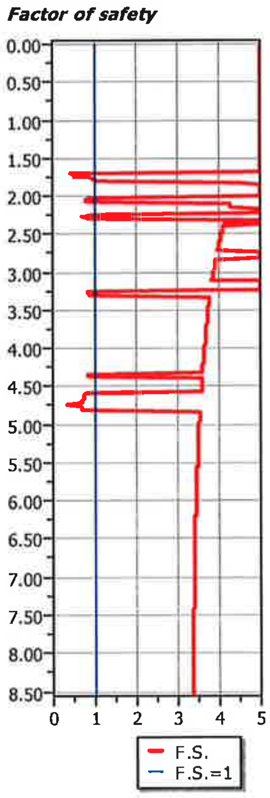
Project subtitle : CPT-1

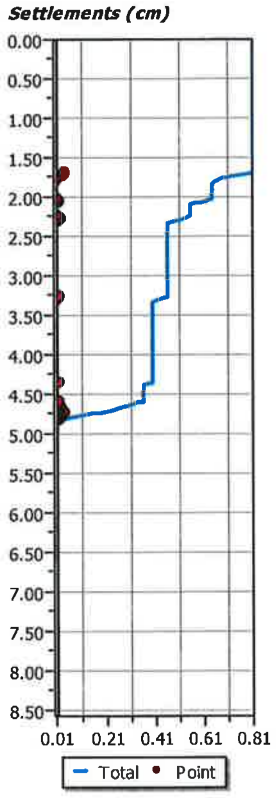
Input parameters and analysis data

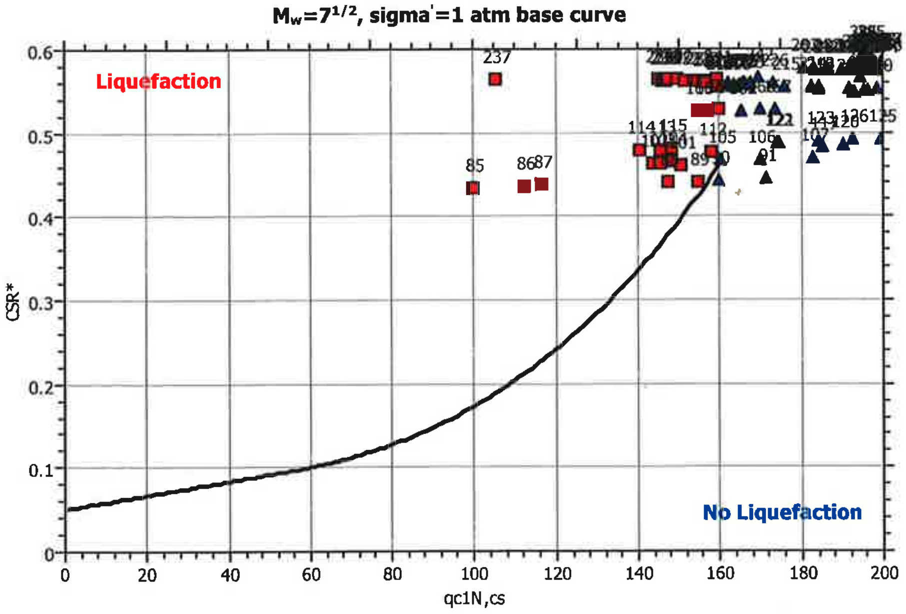
In-situ data type:	Cone Penetration Test	Depth to water table:	1.00 m
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.50
Analysis method:	Robertson (1998)	Peak ground accelaration:	0.41 g
Fines correction method:	Robertson (1998)	User defined F.S.:	1.30

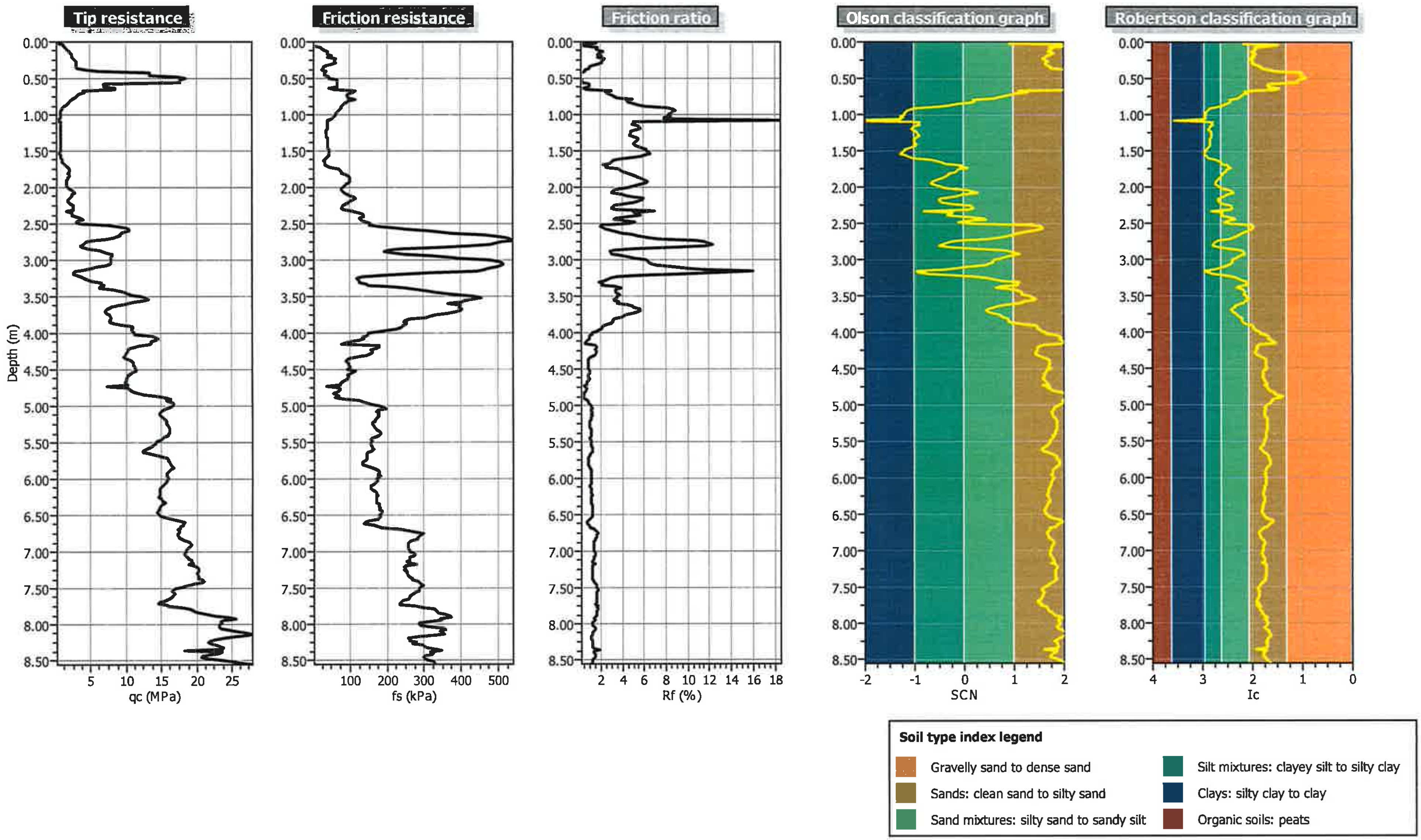

CPT data graph


Shear stress ratio


Factor of safety


Settlements (cm)


$M_w=7^{1/2}$, sigma'=1 atm base curve


Tip resistance
Depth (m)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
5.50
6.00
6.50
7.00
7.50
8.00
8.50
5
10
15
20
25
qc (MPa)
Friction resistance
100
200
300
400
500
fs (kPa)
Friction ratio
2
4
6
8
10
12
14
16
18
Rf (%)
Olson classification graph
-2
-1
0
1
2
SCN
Robertson classification graph
4
3
2
1
0
Ic
Soil type index legend
Gravelly sand to dense sand
Sands: clean sand to silty sand
Sand mixtures: silty sand to sandy silt
Silt mixtures: clayey silt to silty clay
Clays: silty clay to clay
Organic soils: peats

GeoLogismiki
Geotechnical Engineering Software
Merarhias 56, 621 25 - Serrai, Greece
url: http://www.geologismiki.gr - email: info@geologismiki.gr

LIQUEFACTION ANALYSIS REPORT

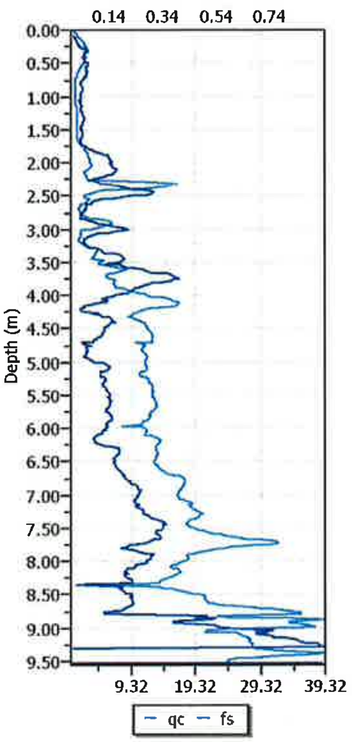
Project title : Crescent City IAVC

Project subtitle : CPT-2

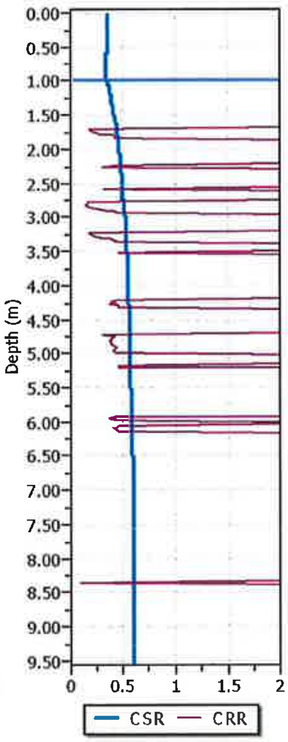
Input parameters and analysis data

In-situ data type:	Cone Penetration Test	Depth to water table:	1.00 m
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.50
Analysis method:	Robertson (1998)	Peak ground accelaration:	0.41 g
Fines correction method:	Robertson (1998)	User defined F.S.:	1.30

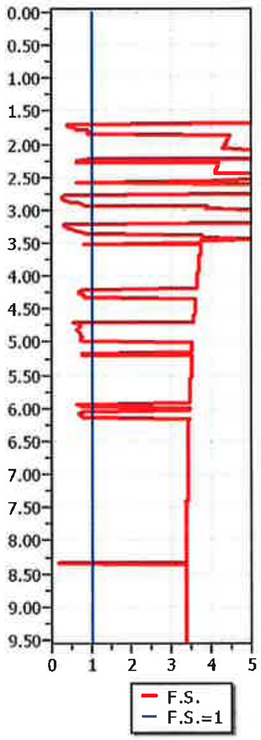
CPT data graph



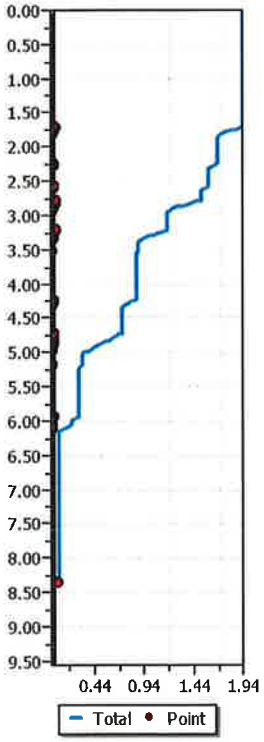
Shear stress ratio



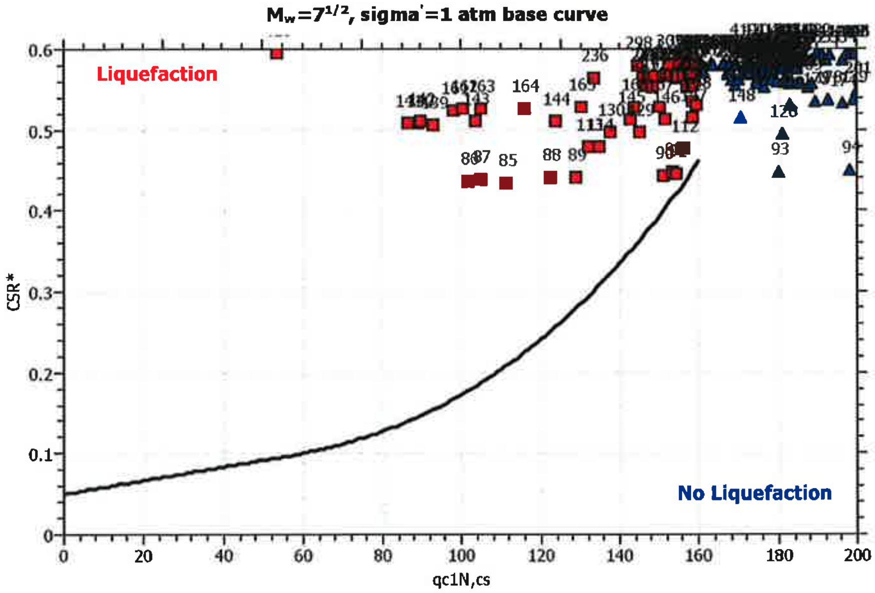
Factor of safety

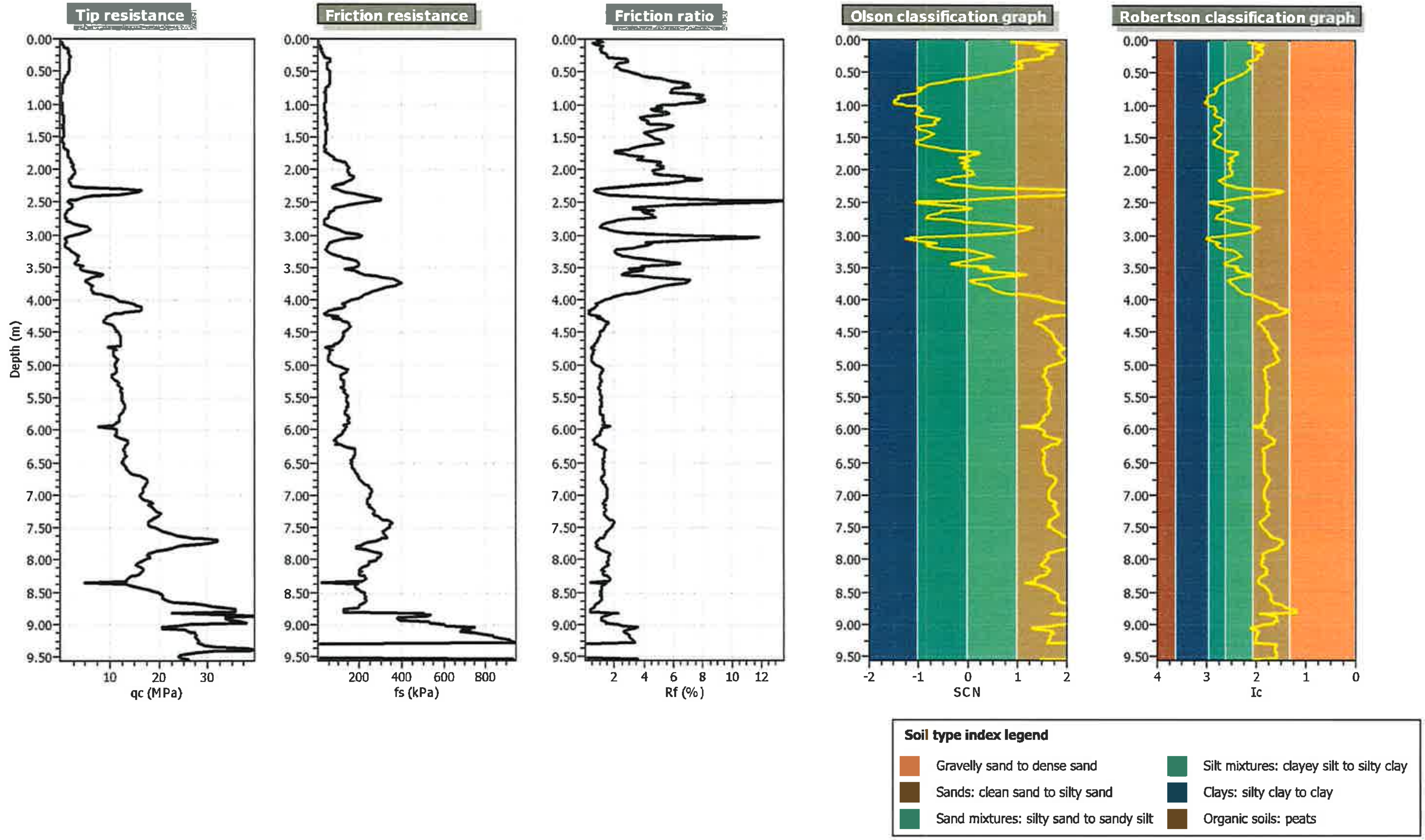


Settlements (cm)



M_w=$7^{1/2}$, sigma'=1 atm base curve




Tip resistance
Friction resistance
Friction ratio
Olson classification graph
Robertson classification graph
Depth (m)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
5.50
6.00
6.50
7.00
7.50
8.00
8.50
9.00
9.50
10
20
30
qc (MPa)
200
400
600
800
fs (kPa)
2
4
6
8
10
12
Rf (%)
-2
-1
0
1
2
SCN
4
3
2
1
0
Ic
Soil type index legend
Gravelly sand to dense sand
Sands: clean sand to silty sand
Sand mixtures: silty sand to sandy silt
Silt mixtures: clayey silt to silty clay
Clays: silty clay to clay
Organic soils: peats

LIQUEFACTION ANALYSIS REPORT

Project title : Crescent City IAVC

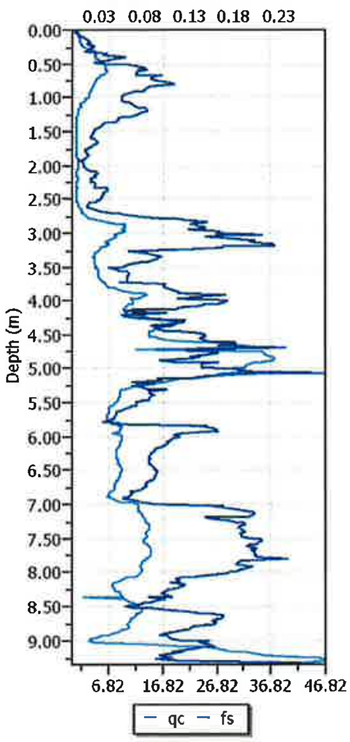
Project subtitle : CPT-3

Input parameters and analysis data

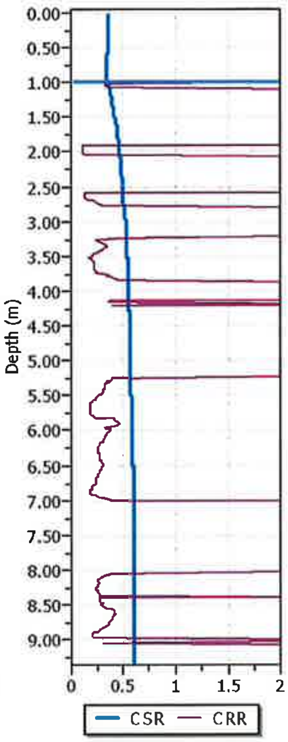
In-situ data type: Cone Penetration Test
Analysis type: Deterministic
Analysis method: Robertson (1998)
Fines correction method: Robertson (1998)

Depth to water table: 1.00 m
Earthquake magnitude M_w: 7.50
Peak ground accelaration: 0.41 g
User defined F.S.: 1.30

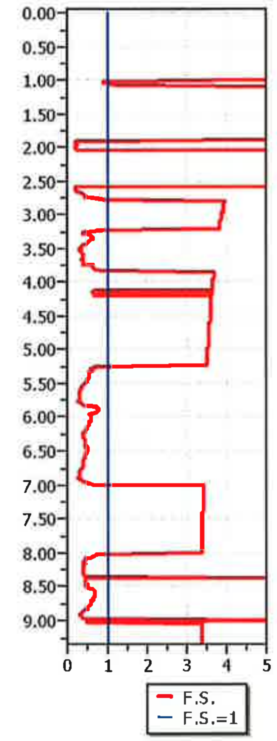
CPT data graph



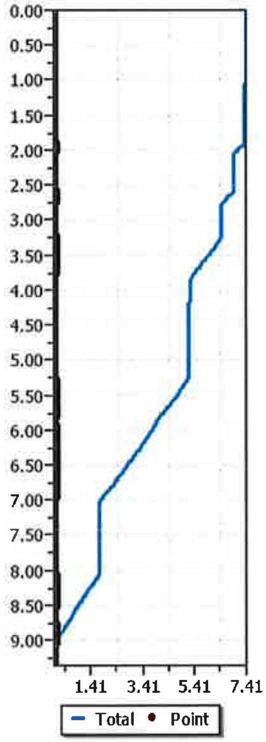
Shear stress ratio



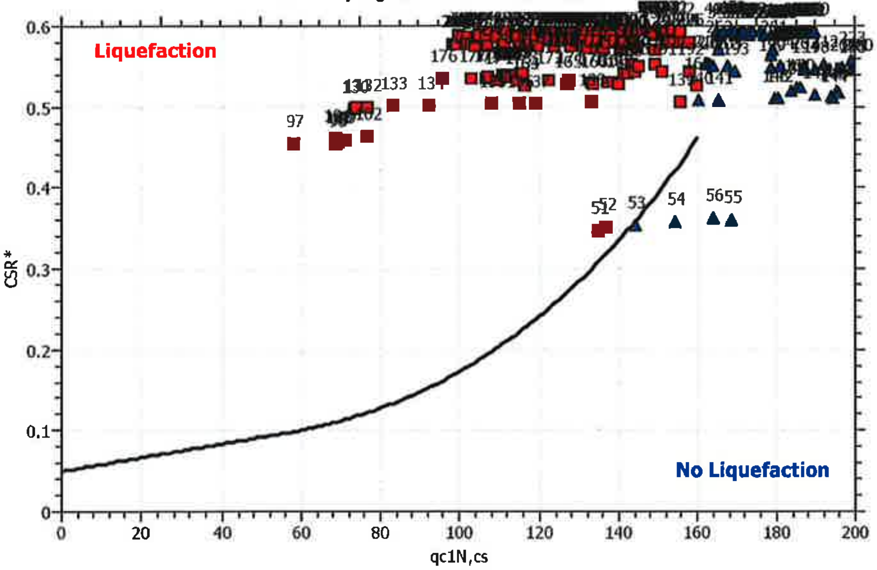
Factor of safety

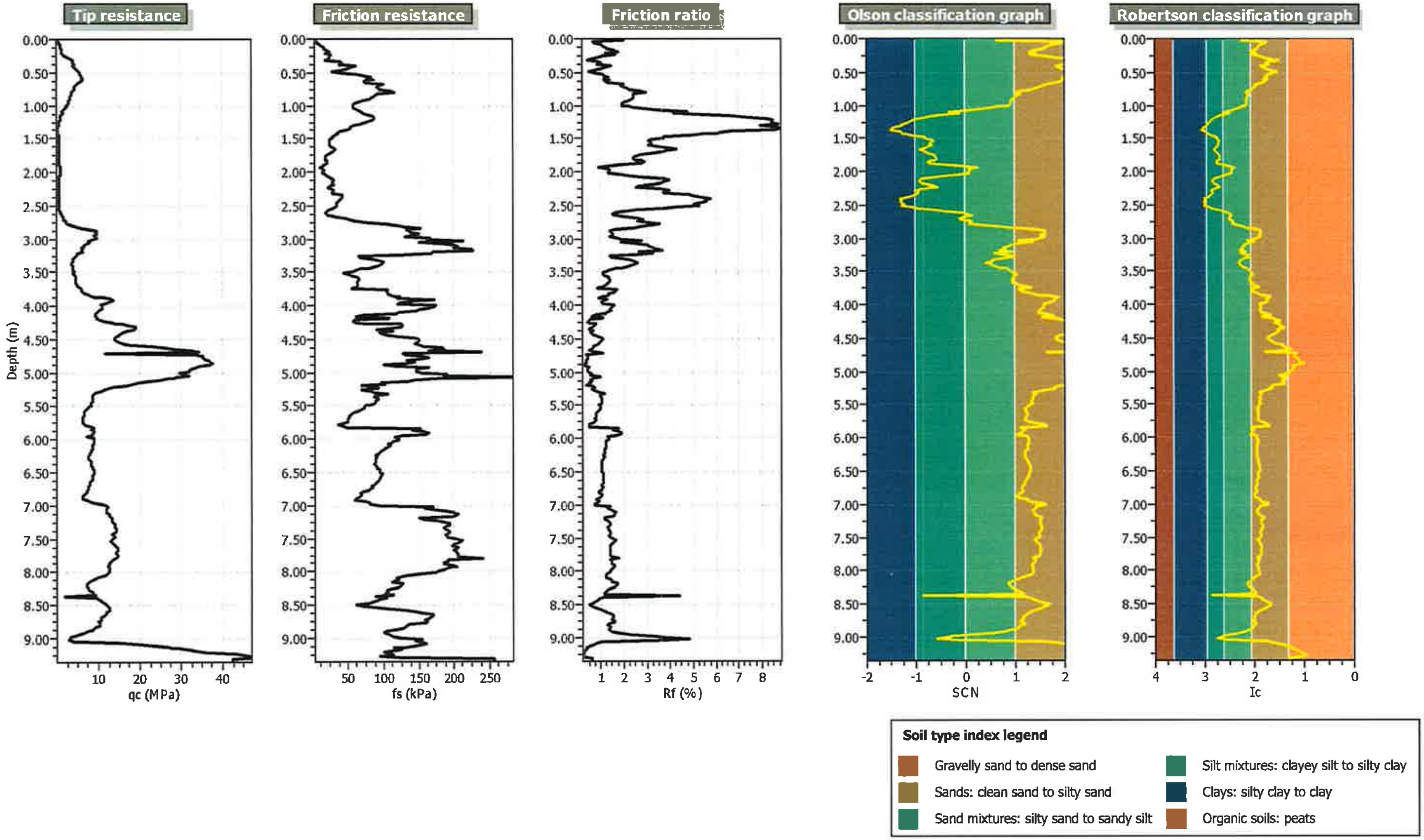


Settlements (cm)



M_w=$7^{1/2}$, sigma'=1 atm base curve




Tip resistance
Friction resistance
Friction ratio
Olson classification graph
Robertson classification graph
Depth (m)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
5.50
6.00
6.50
7.00
7.50
8.00
8.50
9.00
10
20
30
40
qc (MPa)
50
100
150
200
250
fs (kPa)
1
2
3
4
5
6
7
8
Rf (%)
-2
-1
0
1
2
SCN
4
3
2
1
0
Ic
Soil type index legend
Gravelly sand to dense sand
Sands: clean sand to silty sand
Sand mixtures: silty sand to sandy silt
Silt mixtures: clayey silt to silty clay
Clays: silty clay to clay
Organic soils: peats

GeoLogismiki
Geotechnical Engineering Software
Merarhias 56, 621 25 - Serrai, Greece
url: http://www.geologismiki.gr - email: info@geologismiki.gr

LIQUEFACTION ANALYSIS REPORT

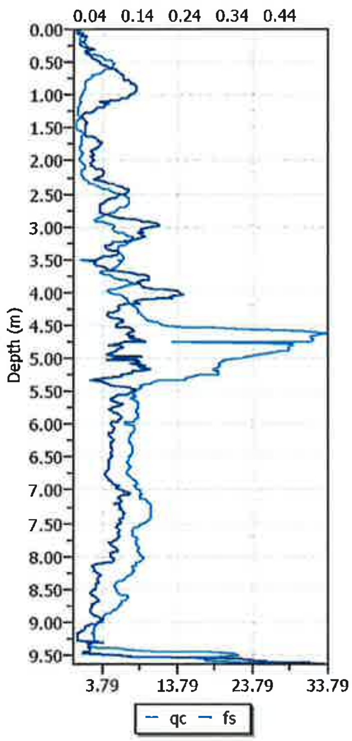
Project title : Crescent City IAVC

Project subtitle : CPT-4

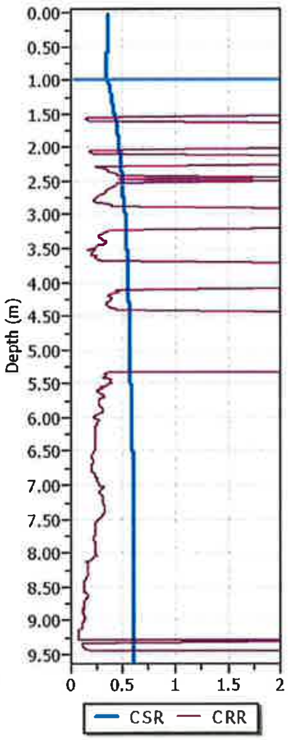
Input parameters and analysis data

In-situ data type:	Cone Penetration Test	Depth to water table:	1.00 m
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.50
Analysis method:	Robertson (1998)	Peak ground accelaration:	0.41 g
Fines correction method:	Robertson (1998)	User defined F.S.:	1.30

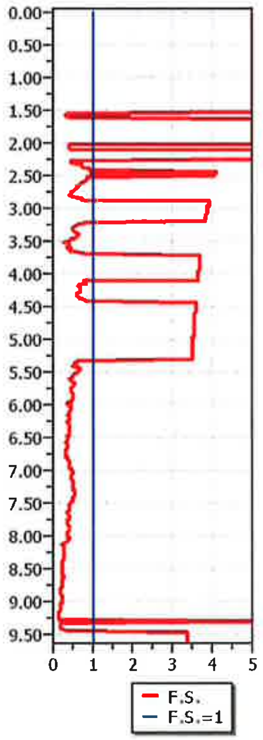
CPT data graph



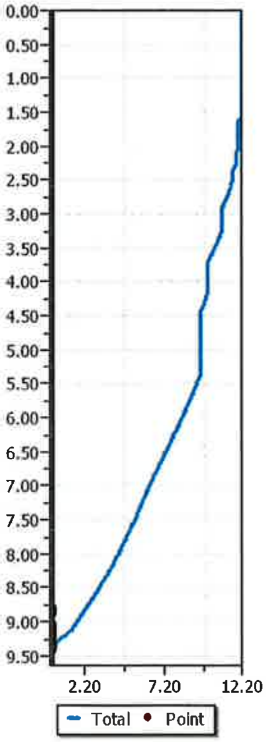
Shear stress ratio



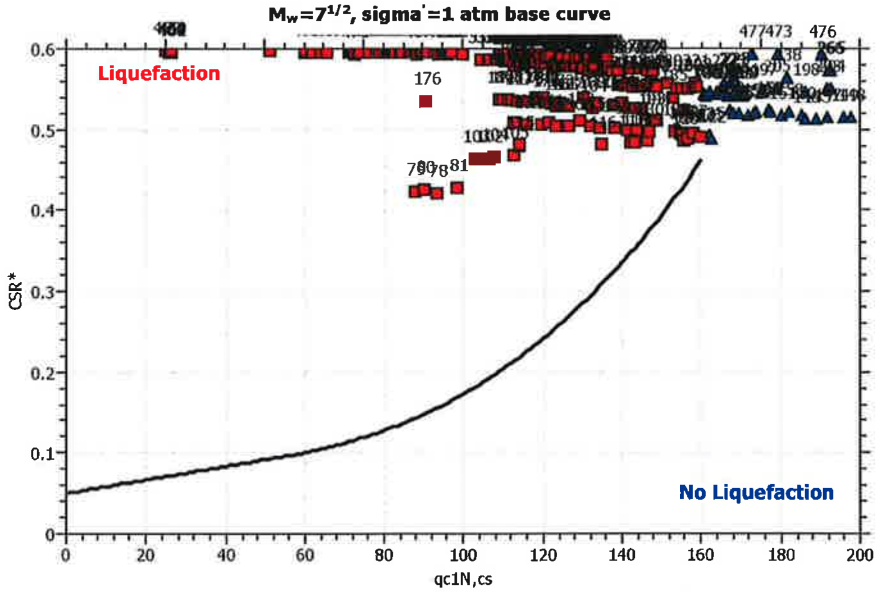
Factor of safety

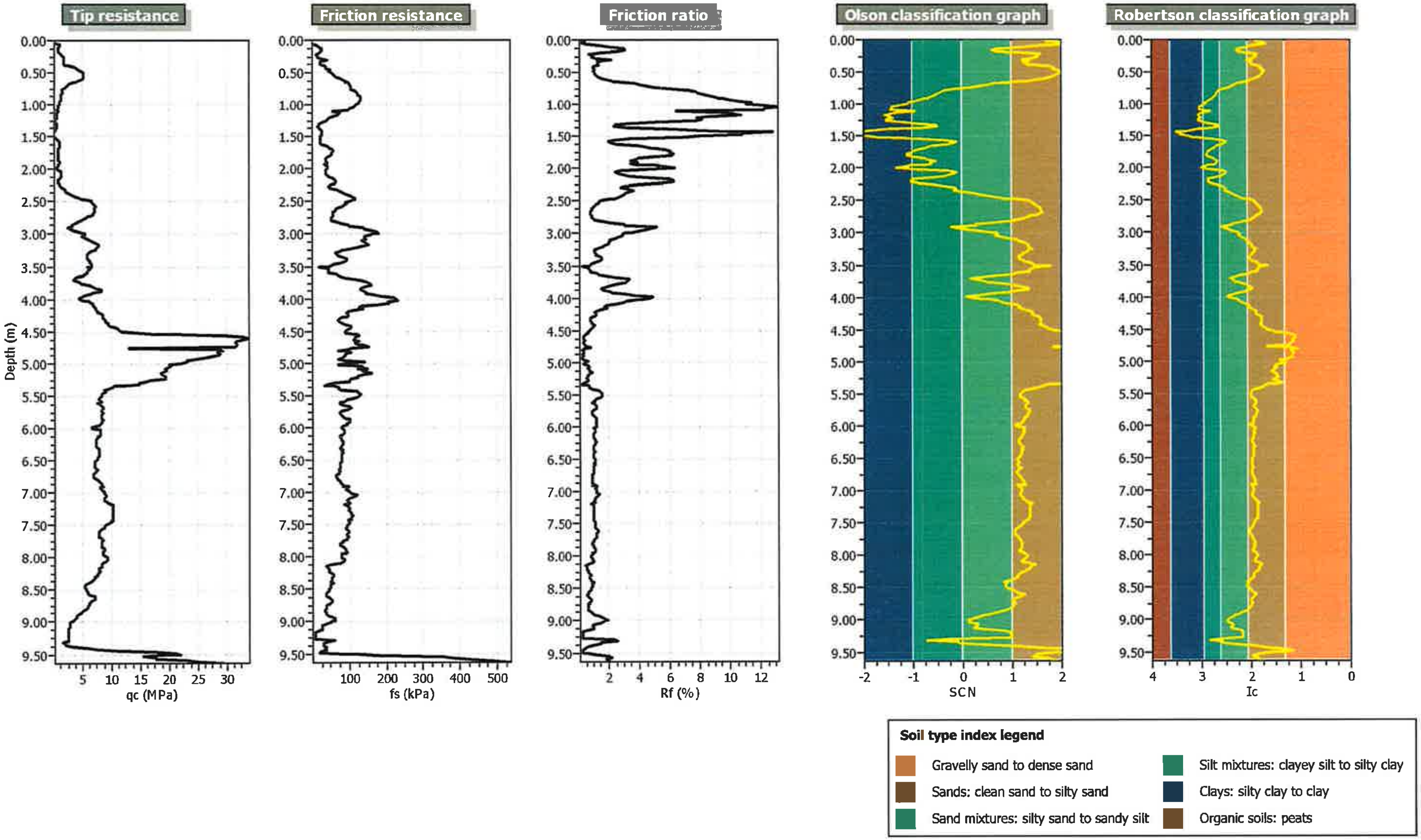


Settlements (cm)



$M_w=7^{1/2}$, sigma'=1 atm base curve




Tip resistance
Friction resistance
Friction ratio
Olson classification graph
Robertson classification graph
Depth (m)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
5.50
6.00
6.50
7.00
7.50
8.00
8.50
9.00
9.50
5 10 15 20 25 30
qc (MPa)
100 200 300 400 500
fs (kPa)
2 4 6 8 10 12
Rf (%)
-2 -1 0 1 2
SCN
4 3 2 1 0
Ic
Soil type index legend
Gravelly sand to dense sand
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