CRESCENT CITY HARBOR
SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

Stover Engineering
Crescent City, California

7 October 2011
Project 731577601

Treadwell&Rollo
A LANGAN COMPANY

Treadwell&Rollo
A LANGAN COMPANY

7 October 2011
Project 731577601

Mr. Ward Stover
Stover Engineering
711 H Street
Crescent City, California 95531

Subject: Crescent City Harbor
Supplemental Geotechnical Consultation
Crescent City, California

Dear Mr. Stover:

Treadwell & Rollo is pleased to present this report presenting the results of our supplemental geotechnical consultation for the planned improvements at of the Crescent City Harbor in Crescent City, California. Specifically this report addresses the on-shore improvements and planned slope repairs. The proposed improvements include steepening the basin slopes, installation of rock slope protection, new walkway abutments, and a new restroom building. Our report is in fulfillment of our proposal dated 1 September 2011.

The purpose of our supplemental consultation was to develop conclusions and recommendations for the design and construction of the proposed improvements. We used the results of our recently completed geotechnical investigation where appropriate.

On the basis of the results of our engineering studies, we conclude that the primary geotechnical concerns affecting the proposed improvements are strong to very strong ground shaking during an earthquake, the presence of potentially liquefiable sand, stability of the basin slopes, and relatively shallow groundwater. We conclude that the planned improvements are feasible provided that the recommendations presented in the attached report are incorporated into the design of the planned improvements. Therefore, anyone relying on this report should read it in its entirety.

The recommendations and conclusions contained in the attached report are based on limited subsurface exploration and laboratory testing programs. Consequently, variations between expected and actual soil conditions may exist and be found during construction. Therefore, we should be notified if variations are encountered during installation of foundations, so we may modify our recommendations, if deemed necessary.

We appreciate the opportunity of being of service to you on this project. Please call, if you have questions.

Sincerely,
TREADWELL & ROLLO, A LANGAN COMPANY

Haze Rodgers

REGISTERED PROFESSIONAL ENGINEER
HAZE M. RODGERS
NO. 2818
Exp. 06/30/12
GEOTECHNICAL
STATE OF CALIFORNIA

Haze Rodgers, G.E.
Project Engineer

Richard D. Rodgers

Richard D. Rodgers, G.E.
Senior Principal

REGISTERED PROFESSIONAL ENGINEER
RICHARD D. RODGERS
NO. 732
EXP. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

730577601.01_HMR_Crescent City Harbor_LTR

CRESCENT CITY HARBOR
SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

Stover Engineering
Crescent City, California

7 October 2011
Project 731577601

Treadwell&Rollo
A LANGAN COMPANY

Treadwell&Rollo
A LANGAN COMPANY

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF APPENDICES

CRESCENT CITY HARBOR
SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

1.0 INTRODUCTION

This report presents the results of our supplemental geotechnical studies for the Crescent City Harbor Rehabilitation Project, located in Crescent City, California. Specifically this report addresses the on-shore improvements and planned slope repairs. We previously performed a geotechnical investigation at the site, and engineering studies for design of the proposed inner harbor improvements. The results of our previous investigation and studies were presented in our report titled *Geotechnical Investigation, Crescent City Harbor Rehabilitation, Crescent City, California,* dated 13 June 2011 and have been incorporated into this report where appropriate. The project site is located within and around the Inner Harbor as shown on Figure 1.

The on-shore portion of the site is relatively flat with ground surface elevations in the vicinity of the planned on-shore improvements (walkway abutments and restroom building) ranging from approximately 12 to 14 feet[1]. Based on our review of the improvement plans prepared for construction of the inner harbor prepared by SWINC Engineering, Inc., the basin slopes were originally constructed at inclinations of 1.5:1 (horizontal to vertical) with a zone of compacted rock below the submerged portion of the slopes. We understand the basin side slopes have slumped due to dredging below the bottom of the rock slope reinforcement resulting in the existing side slopes of the Inner Harbor having inclinations ranging from approximately 1.2:1 to 2:1.

The proposed improvements will consist of dredging the inner basin to a bottom elevation of -12 feet, installing new walkway abutments, a new restroom building near the northeast corner of the boat basin, steepening the basin side slopes to their original inclination (1.5:1), and installing new rock slope protection (RSP) against the steepened basin slopes. To limit the horizontal extents of the RSP, we understand a six foot wide rock filled gabion wall is planned at the top of the basin slopes. Preliminary plans indicate that the RSP will be approximately 11.3 feet thick (perpendicular to the slope face), and the gabion wall will extend 14 feet below the existing ground surface.

---

1 All elevations are referenced to Mean Low Lower Water (MLLW).

2.0 SCOPE OF SERVICES

Our scope of services was outlined in our proposal dated 1 September 2011. Our scope of services included supplemental engineering evaluations, and developing conclusion and recommendations for design and construction of the proposed improvements. No additional subsurface exploration or laboratory testing was performed. Specifically, we developed conclusions and recommendations regarding the following:

- soil and bedrock conditions at the site
- site seismicity and seismic hazards including potential for liquefaction, and liquefaction-induced hazards
- evaluation of the stability of the proposed basin slope configuration (static and seismic)
- appropriate foundation type(s) for the proposed restroom building and walkway abutments
- design parameters for recommended foundation type(s) including axial and lateral load resistance parameters
- estimated settlement of recommended foundation types
- lateral earth pressures for permanent and temporary (shoring) walls
- site class and seismic coefficients per the 2010 California Building Code
- construction considerations.

3.0 RECENT FIELD INVESTIGATION

A detailed discussed of the recently completed field exploration is provided in our 13 June 2011 report. No additional field exploration was performed for this project. A total of six borings were drilled during our recent investigation (three over water within the Inner Harbor and three on land adjacent to the top of the basin slopes). The approximate locations of the borings are shown on Figure 2. In addition, the approximate location of borings drilled for the original design and construction of the Inner Harbor are also shown on Figure 2. The details of our field investigation program are presented in our 13 June 2011 report. For reference we have included the logs of the recently completed borings in Appendix A.

4.0 SUBSURFACE CONDITIONS AND LABORATORY TESTING

The land portion of the site is underlain by fill, native beach sand, and rock. The basin is underlain by a thin layer of sand and silt over bedrock or bedrock. The fill and beach sand generally consist of loose to very dense sand and silty sand. The bedrock primarily consists of 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. Sandstone was not encountered in all the borings and is anticipated to be discontinuous below the site. As shown on Figure 3, the top of bedrock varies from Elevation -12 to -21.5 feet (Figure 3). We developed two idealized subsurface profiles presenting our interpretation of the subsurface conditions beneath the site (Figures 4 and 5). It should be noted that the effects of the 11 March 2011 tsunami are not reflected on Figures 4 and 5 or the boring logs. The amount of sedimentation or scour resulting from this tsunami is not known. Groundwater is expected to correspond to the water elevation within the inner boat basin and ocean tides. Groundwater is anticipated to fluctuate daily, and seasonally.

The details of our recently completed laboratory testing are presented in our 13 June 2011 report. For reference the results have been include in Appendix B.

5.0 REGIONAL SEISMICITY

The major active faults in the area are the Big Lagoon Bald Mountain, Cascadia, and Trinidad Faults (Figure 6). For each of the active faults, the distance from the site and estimated mean characteristic Moment magnitude[2] [Working Group on California Earthquake Probabilities (WGCEP) (2003) and Cao et al. (2003)] are summarized in Table 1.

---

[2] Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
Big Lagoon – Bald Mountain	22	West	7.3
Cascadia – Revised	24	West	9.2
Trinidad	38	Southwest	7.3
Cascadia - Transition Zone	58	West	9.2
Cascadia – Midpoints	76	West	9.2
McKinleyville	76	South	7.0
Mad River	80	South	7.1
Fickle Hill	86	South	6.9
Little Salmon – Offshore	91	Southwest	7.1
Cascadia – Elastic Zone	92	West	9.2

Figure 6 also shows the earthquake epicenters for events with magnitude greater than 4.0 from January 1800 through January 2000.

6.0 DISCUSSIONS AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the issues identified and discussed in this report are addressed during design and construction. The primary geotechnical issues to be addressed during site development are:

- Strong ground shaking during an earthquake
- Liquefaction of loose to medium dense sand layers
- stability of the inner harbor side slopes
- construction considerations.

These and other geotechnical issues as they pertain to the proposed development are discussed in following sections.

6.1 Seismic Hazards

The site is not within a state-designated seismic hazard zone. However, during a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction, lateral spreading,[3] and cyclic densification.[4] We used the results of our field exploration and laboratory testing to evaluate the potential for these phenomena to occur at the project site. The results of our evaluation are discussed below.

6.1.1 Ground Shaking

The seismicity of the site is governed by the activity of the Big Lagoon and Cascadia Faults. However, ground shaking from future earthquakes on any of the regional faults could be felt at the site. The intensity of earthquake ground motions at the site will depend upon the characteristics of the generating fault, distance from the rupture, magnitude and duration of the earthquake, and specific subsurface conditions. We judge ground shaking at the site during a major earthquake on one of the nearby regional faults will be strong.

6.1.2 Soil Liquefaction and Associated Hazards

When saturated, cohesionless and low plasticity fine grained (silts and clays) soil liquefies, these types of soil experience a temporary loss of shear strength due to a transient rise in excess pore pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction. We evaluated liquefaction potential at the site in accordance with SP 117 (CGS 2008), as described below.

6.1.2.1 Liquefaction Potential and Liquefaction-Induced Settlement

We used the data from our land borings to evaluate the liquefaction potential and liquefaction-induced settlement. The liquefaction analysis was performed in accordance with the methodology presented in the publication titled *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCFEA/NSF Workshop on Evaluation of Liquefaction Resistance on Soil* (Youd et al. 2001). A peak ground acceleration (PGA) of 0.41 times gravity which corresponds to the 2010 California Building Code (CBC) Design Earthquake (DE) was used in our liquefaction analyses.

---

3 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported down slope or in the direction of a free face by earthquake and gravitational forces.

4 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground-surface settlement.

We used the method developed by Tokimatsu and Seed (1987), which relates normalized clean sand SPT N-Values [$(N_1)_{60,CS}$] values to strain potential to estimate the liquefaction-induced settlement potential of the liquefiable layers. The results of our liquefaction analyses including the thickness of the liquefiable layer and estimated settlement are presented in Table 2.

TABLE 2
Estimates of Liquefaction-Induced Settlements

Test Boring	Elevation of Top of Liquefiable Layer (feet)	Thickness of Liquefiable Layer (feet)	Estimated Settlement (inches)
B-2	5	5	1.5
B-3	-0.5	10	1.6

The results of our liquefaction analyses indicate that in general the loose to medium dense sand encountered in borings B-2 and B-3 are susceptible to liquefaction, and liquefaction-induced settlements (total and differential) up to approximately 1.6 inches. The dense to very dense sand is generally sufficiently dense to resist liquefaction. We understand the walkway abutments will be located at the top of the basin slopes on the slope face. The proposed RSP will remove a significant amount of the potentially liquefiable material from below the proposed walkway abutments reducing the estimated settlements in these areas significantly. We estimate that the liquefaction-induced settlements below the walkway abutments will be approximately half those presented in Table 2.

The potential for manifestation of liquefaction at the ground surface, such as sand boils, depends on the thickness of the liquefiable soil layer relative to the thickness of the overlying non-liquefiable material. Ishihara (1985) developed an empirical relationship that provides criteria that can be used to evaluate whether surface manifestation of liquefaction would be expected to occur under a given level of shaking for a liquefiable layer of given thickness overlain by a resistant, or protective, surficial layer. In the vicinity of borings B-2 and B-3 the thickness of the non-liquefiable surface layer is not adequate to prevent liquefaction-induced ground failure; therefore, we conclude that in the vicinity of borings B-2 and B-3 the risk for surface manifestation of liquefaction is high.

6.1.2.2 Lateral Spreading

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as an open slope cut, or in the direction of a regional slope or gradient. Based on the results of our liquefaction analyses, the data collected during our subsurface exploration, and the proximity of the Inner Harbor Basin slopes, we conclude that without the installation of the proposed RSP the potential for lateral spreading to occur near test borings B-2 and B-3 is high. If the proposed RSP is installed as planned we conclude the risk of significant permanent lateral slope displacements will be low. A more detailed discussion of the estimate magnitude of permanent lateral displacement is presented in Section 6.2.

6.1.3 Cyclic Densification

Seismically-induced compaction or cyclic densification of non-saturated sand (i.e. sand above the groundwater table) resulting from earthquake vibrations may cause differential settlement. We evaluated the cyclic densification potential for the medium dense sand above the groundwater encountered in our borings. The results of our analyses indicate that the amount of cyclic densification should be less than 1/4-inch. Therefore, the potential hazard associated with cyclic densification is low.

6.1.4 Fault Rupture

Historically, ground surface displacements closely follow the traces of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no active faults have been mapped at the site. Therefore, we conclude that the risk for fault rupture at the site is low.

6.2 Slope Stability

We performed static and seismic slope stability analyses considering the proposed slope configurations (slopes inclined at 1.5:1) including the proposed RSP and dredging. We considered the basin will be dredged to Elevation -12 feet. We previously evaluated the existing slopes configurations and presented the results in our 13 June 2011 report. Our stability analyses considered the subsurface data and laboratory test results from our previous subsurface investigation at the site. We evaluated three generalized slope configurations in the vicinity of test borings B-1, B-2, and B-3 (Figures 4 and 5). We performed our slope stability analyses using the program SLOPE W version 6.22 developed by GEOSLOPE International. The slope configurations and the results of our slope stability analyses are presented in

Appendix C. Details of the slope stability analysis performed are discussed below in the following sections.

6.2.1 Static Slope Stability

We performed static slope stability analyses considering water levels at MLLW and High Water Elevation (7 MLLW), and drained strength (frictional) parameters. These strengths were based on the results of our field and laboratory tests, and our professional judgment. The soil parameters used in our analyses are presented in Table 3.

TABLE 3
Soil Parameters for Slope Stability Analyses

Material	Total Unit Weight (pcf)	Cohesion (psf)	Friction Angle (Degrees)
Medium Dense to Dense Sand	125	0	36
Compacted Rock	130	0	38
Dense to Very Dense Sand	133	0	40
Rock Slope Protection (RSP), and Rock Gabions	145	0	45

The results of our static slope stability analyses are presented in Table 4. For comparison purposes we have presented the previously determined static factors of safety of the existing basin slopes.

TABLE 4
Static Slope Stability Results

Location	Water Level (Feet MLLW)	Static Factor of Safety	
		Existing Slope Configuration	Proposed Slope Configuration
B-1	0	1.61	1.52
	7	1.62	1.57
B-2	0	1.57	1.53
	7	1.39	1.54
B-3	0	1.45	1.52
	7	1.46	1.52

On the basis of the results of our static slope stability analyses, we conclude that the proposed slope configuration and RSP improvement will have factors of safety against deep rotational failures of at least 1.5.

6.2.2 Seismic Slope Stability

We evaluated the potential permanent lateral displacement of the proposed slopes during an earthquake with a moment magnitude of 7.5, generating a 2010 CBC Design Earthquake (DE) peak horizontal ground acceleration (U_{max}) of 0.41 times gravity (g's). Furthermore, the strength of the potentially liquefiable material encountered in Borings B-2 and B-3 was assigned as the lesser of the residual post liquefaction undrained shear strength or the drained shear strength in accordance with the recommendations of Seed et al. (2003). We used the Makdisi and Seed (1978) approach to estimate the permanent lateral displacement of the slopes. The results of our slope stability analysis are presented in Appendix C. These results indicate that the yield accelerations of the proposed slopes will range from 0.13 to 0.17 times gravity. The estimated magnitude of the permanent lateral displacement of the proposed slopes configurations are presented below in Table 5. For comparison purposes we have also presented the previously estimated slope displacements of the existing slopes in table 5.

TABLE 5
Estimated Permanent Lateral Slope Displacement

Location	Water Level (Feet MLLW)	Yield Acceleration (g's)	Permanent Lateral Displacement (inches)	
			Existing Slope Configuration	Proposed Slope Configuration
B-1	0	0.15	≤ 1	≤ 1
	7	0.14	9	≤ 1
B-2	0	0.17	≤ 1	≤ 1
	7	0.14	26	≤ 1
B-3	0	0.14	≤ 1	≤ 1
	7	0.13	26	≤ 1

Although the static factors of safety against deep rotational failure are similar to those for the existing slopes, the proposed slope configurations and RSP results in critical surfaces that are deeper and generally limited to the RSP for both static stability and seismic displacement. The results of our previous slope stability evaluations indicated that the critical failure surfaces for static stability and seismic displacement were relatively shallow and generally isolated to the liquefiable material and/or just behind the existing relatively thin RSP. On the basis of the results of our seismic slope stability evaluation we conclude the proposed RSP and gabion walls will improve the performance of the basin slopes during an earthquake, and the estimated permanent lateral displacement of the proposed slopes will be low.

6.3 Appropriate Foundations

We understand the walkway abutments will primarily resist axial compression loads (dead and live loads). On the basis of the results of our studies, we conclude that the walkway abutments and restroom building can be founded on shallow foundations. Estimated total and differential settlements of properly design and constructed shallow foundations should generally be less than 1- and 1/2-inches respectively.

6.4 Construction Considerations

The gabion walls will extend to Elevation 0 to -1 feet. Because of the tidal fluctuations and the granular near surface soil, we anticipate the excavation for the gabion walls will require dewatering. The contractor should be prepared to dewater the excavations appropriately to prevent softening, basal heaving, or loss of soil during installation of the gabions. In addition, we understand the gabion wall will be placed within close proximity to existing settlement sensitive improvements (existing restroom buildings, wash stations, light poles, etc.). In areas adjacent to existing settlement sensitive improvements we recommend shoring be used to reduce the potential for settlement and/or loss of soil below these facilities.

The foundations for walkway abutments are anticipated to be on the RSP basin slopes, at the tops and will likely be supported on the RSP. Since the RSP is anticipated to consist of relatively large particles with relatively large of void space between particles, a separation fabric may be required to prevent loss of concrete into the RSP.

7.0 RECOMMENDATIONS

Recommendations for the design and construction of the proposed improvements are provided in the following sections.

7.1 Earthwork and Grading

7.1.1 Site Preparation

Grading operations should commence after demolition and removal of the existing pavements, foundation slabs, and underground utilities within the development area. Following demolition, all areas to receive improvements should be stripped of vegetation and organic topsoil. The pavement material, including asphalt, may be segregated from organic topsoil and used as compacted fill, provided it meets the fill requirements presented in a subsequent paragraph of this section and is acceptable from an environmental standpoint. The stripped organic soil can be stockpiled for later use in landscaped areas, if approved by the architect; organic topsoil should not be used as compacted fill.

Following stripping, the restroom building pad and exterior concrete slab areas should be excavated to provide a layer of properly compacted select fill a minimum of 12 inches thick below the building floor and exterior concrete slabs. The excavations should extend at least five feet beyond the building footprint. The surface exposed by excavation/stripping should be scarified to a depth of at least six inches, moisture-conditioned to near the optimum moisture content, and compacted to at least 95 percent relative compaction.[5] The exposed ground surface should be kept moist during subgrade preparation.

Select fill should consist of either on-site or imported soil that is non-hazardous, non-corrosive, free of organic matter, smaller than three inches in greatest dimension, has a liquid limit less than 40 and a plasticity index less than 12, and is approved by the geotechnical engineer. In general, the existing near-surface soil is expected to meet the criteria for select fill. A sample of proposed select fill(s) should be submitted to the geotechnical engineer for testing at least three business days prior to use at the site.

Fill should be placed in horizontal layers not exceeding eight inches in loose thickness, moisture-conditioned to near the optimum moisture content, and compacted to at least 95 percent relative

---

[5] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-00 laboratory compaction procedure.

compaction. In areas where wet and/or weak subgrade soils are encountered, an alternative to mitigate this problem is scarifying and aerating the soil to reduce its moisture content so that it can be compacted to the required compaction. For this alternative, several days of dry, warm weather may be required. Other alternatives to mitigate weak subgrade areas are: 1) excavating the upper 12 to 18 inches of the weak soil, and backfilling with a lean concrete, and 2) excavating the upper 12 to 18 inches of the weak soil, placing a geotextile (Mirafi 500X or equivalent), and placing and compacting select fill over the fabric.

7.1.2 Utility Trenches

Excavations for utility trenches can be readily made with a backhoe. Despite careful site preparation, unexpected obstructions may make some of the trenching operations difficult. All trenches should conform to the current CAL-OSHA requirements. Backfill for utility trenches and other excavations is also considered fill, and it should be compacted according to the recommendations presented in Section 7.1.1, except that it need only be compacted to 90 percent relative compaction unless clean sand is used as backfill in which case it should be compacted to at least 95 percent. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Where necessary, excavations should be shored and braced to prevent cave-ins in accordance with all safety regulations. Due to the high risk of liquefaction, where sheet piling is used as shoring it should not be installed or removed using vibratory methods, and is to be removed after backfilling. Sheet piling should be placed a minimum of two feet away from the pipes or conduits to prevent disturbance to them as the sheet piles are extracted. Where trenches extend below the groundwater level, it will be necessary to temporarily dewater them to allow for placement of the pipe and/or conduits, and backfill.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped.

7.1.3 Surface Drainage

Drainage control design should include provisions for positive surface gradients so that surface runoff is not permitted to pond, particularly above slopes or adjacent to building foundations, roadways, pavements, or slabs. We recommend the ground surface be sloped at least 2% in unpaved areas and

1% in paved areas away from building foundations. Surface runoff should be directed away from slopes and foundations and collected in lined ditches or drainage swales. The water collected should be directed to a storm drain or paved roadway. Discharge from the roof gutter and downspout systems should be included in the collection system and not allowed to infiltrate the subsurface near the structures or in the vicinity of slopes.

7.1.4 Temporary and Permanent Slopes

Excavations deeper than five feet that will have to be entered by workers should be shored or sloped in accordance with the California Occupational Safety and Health Administration (CALOSHA standards (29 CFR Part 1926). The contractor should be responsible for the design, construction, and safety of temporary shoring. We judge that temporary cuts in native sand and compacted fill which are less than 10 feet high and inclined no steeper than 1.5:1 will be stable provided that they are not surcharged by equipment or building material.

If necessary, temporary shoring should be designed considering the appropriate lateral earth and surcharge pressures. For preliminary design of cantilever type temporary shoring (soldier beam and lagging, sheet pile, etc. without horizontal supports) we recommend the lateral earth pressures presented in Section 7.5 for retaining walls be used. The design, installation, and removal of any shoring system(s) should be the responsibility of the contractor.

7.2 Restroom Building Foundations

Design information for the restroom building was not available at the time this report was prepared; however, we anticipate the restroom building will be a single story structure located approximately 10 feet from the top of the northern basin slope. We recommend that restroom building be founded on conventional shallow foundations bearing on undisturbed existing soil material or properly compacted fill. For design of the restroom building foundations we recommend an allowable bearing capacity of 3,400 psf be used for dead plus live loads, with a one-third increase for total loads, including wind and/or seismic loads. Continuous footings should be at least 12-inches wide and embedded at least 12 inches below the lowest adjacent ground surface, and isolated spread footings should be at least 24 inches wide, and embedded at least 12 inches below the lowest adjacent ground surface.

Lateral loads may be resisted through passive pressure against the vertical face of the foundations and sliding resistance along the bottom of the foundations. We recommend an equivalent fluid weight of

320 pcf for passive resistance for soil above the groundwater elevation. For sliding resistance along the bottom of the foundations we recommend a coefficient of sliding of 0.3. These values include a factor of safety of 1.5 and may be considered in combination without reduction.

Weak soil or non-engineered fill encountered in the bottom of footing excavations should be excavated and replaced with engineered fill or lean concrete. We should check footing excavations prior to placement of reinforcing steel. Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete.

7.3 Walkway Abutment Foundations

According to the preliminary abutment configurations and loads provided by Stover Engineering, we understand the anticipated bearing pressures for the walkway abutments will vary from approximately 300 to 1,200 psf (dead plus live loads). For design of the walkway abutment foundations we recommend an allowable bearing capacity of 2,000 psf be used. Walkway foundations should be at least three feet wide, and embedded at least two feet below the lowest adjacent ground surface. This value includes a factor of safety of approximately 2.0.

Although significant lateral loads are not anticipated at the walkway abutments, lateral loads may be resisted through passive pressure against the vertical face of the foundations and sliding resistance along the bottom of the foundations. We recommend an equivalent fluid weight of 220 and 180 pcf for passive resistance for soil above and below the groundwater respectively. For sliding resistance along the bottom of the foundations we recommend a coefficient of sliding of 0.3. These values include a factor of safety of 1.5 and may be considered in combination without reduction.

Weak soil or non-engineered fill encountered in the bottom of footing excavations should be excavated and replaced with engineered fill or lean concrete. We should check footing excavations prior to placement of reinforcing steel. Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete.

7.4 Floor Slabs

The near-surface soil is anticipated to consist of medium dense to dense sand; therefore, we conclude the slab can be supported on grade, provided the subgrade is prepared in accordance with Section 7.1. Where soft or loose soil is present in localized areas, the weak soil should be removed and replaced with engineered fill or lean concrete (see Section 7.1). If the subgrade is disturbed during construction (excavation for footings and utilities) it should be re-compacted to provide a firm unyielding surface. Loose, disturbed materials should be excavated, removed, and replaced with engineered fill during final subgrade preparation.

Moisture is likely to condense on the underside of the slabs, even though they will be above the design groundwater table. Consequently, a moisture barrier and capillary moisture break should be installed beneath the slabs if movement of water vapor through the slabs is considered undesirable. The capillary moisture break should consist of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 6.

TABLE 6
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

The sand overlying the membrane should be dry at the time concrete is cast. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.5 Gabion Wall Design

Gabion walls should be designed to resist both static lateral earth pressure and if appropriate, lateral earth pressures caused by earthquakes. We recommend that permanent below grade walls be designed for the more critical of the following criteria:

- At rest equivalent fluid weight of 55 and 90 pcf above and below the groundwater elevation respectively.
- Active pressure of 35 and 80 pcf above and below the groundwater elevation respectively, plus a seismic increment of 7 pcf.

If vehicular traffic is anticipated within 10 feet of the wall an additional lateral traffic surcharge should be considered. The traffic surcharge should be considered as a uniform (rectangular distribution) lateral pressure of 100 psf applied to the upper 10 feet of the wall If foundations of adjacent neighboring buildings bear above an imaginary 1.5:1 (horizontal to vertical) line extending up from the base of the wall, the proposed wall should be designed to resist an additional lateral surcharge of 0.5 times the applied foundation pressure.

Lateral forces can be resisted by a combination of friction along the base and passive resistance against the vertical face of the footing. We recommend an allowable passive resistance of 220 and 180 pcf above and below the groundwater elevation respectively, and coefficient of sliding of 0.3 for footings in contact with on-site soil. The top foot of soil should be neglected for passive resistance unless confined by pavement or a floor slab.

Retaining/gabion walls bearing on the native soil below the RSP may be designed using an allowable bearing pressure of 4,800 psf for dead plus live loads. This value may be increased by one-third for resistance to loads of short durations (wind and seismic). In addition, since portions of the gabion walls will be underlain by potentially liquefiable material some sections will likely settle following an earthquake. This is not anticipated to compromise the gabion wall; however some re-leveling maintenance may be required following an earthquake.

7.6 Rock Slope Protection and Gabion Wall Considerations

The proposed RSP consists of three layers of different size rock (6.2 feet of 2.5 ton rock underlain by 3.3 feet of ¼ ton rock; underlain by 1.8 feet of Caltrans Backing Class 1 rock); however, because the existing soil consists of a fine to medium grained sands and silty sands we recommend a separation fabric be placed at the contact of the RSP and gabion wall and the existing soil. The separation fabric will prevent the migration of the fine to medium grained sands into the RSP and gabions.

7.7 Seismic Design

Although areas of the site are susceptible to liquefaction (borings B-2 and B-3), considering that the thickness of the potentially liquefiable material is relatively thin and discontinuous, and bedrock is relatively shallow we do not anticipate that the ground surface accelerations will be significantly affected by liquefaction should it occur. Therefore for seismic design in accordance with the provisions of 2010 California Building Code (CBC), we recommend the following:

- Maximum Considered Earthquake (MCE) S_s and S_1 of 1.58g and 0.75g, respectively.
- Site Class C
- Site Coefficients F_A and F_V of 1.00 and 1.3
- Maximum Considered Earthquake (MCE) spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.54g and 0.97g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.03g and 0.65g, respectively.

7.8 Geotechncial Services During Construction

We should be allowed to review the final plans and specifications to check that they are in general conformance with the intent of our recommendations. During site grading, we should observe site preparation, abandonment of existing underground utilities, and grading of the site. We should also observe placement of fill and perform field density tests to check that adequate compaction and moisture conditioning has been achieved. These observations and test results will allow us to compare the actual with anticipated soil conditions, and to check that the contractor's work conforms with the geotechnical aspects of the plans and specifications.

8.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited engineering studies based on our interpretation of the existing foundation and geotechnical conditions. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Treadwell & Rollo should be notified so that supplemental recommendations can be made.

REFERENCES

California Building Code (2010).

California Division of Mines and Geology (CDMG) (*1996). Probabilistic Seismic Hazard Assessment for the State of California*, CDMG Open-File Report 96-08.

California Division of Mines and Geology (CDMG) (*1998). Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada*, California Department of Conservation, February.

California Geological Survey (CGS) (1997). *Guidelines for Evaluating and Mitigating Seismic Hazards in California*. Special Publication 117.

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Willis, C.J. (2003). *The Revised 2002 California Probabilistic Seismic Hazard Maps*.

Ishihara, K. (1985). *Stability of Natural Deposits During Earthquakes.* 11th International Conference on Soil Mechanics and Foundations Engineering, Proceedings, San Francisco, Vol. 1, pp. 321-376.

Ishihara, K. and Yoshimine, M. (1992). *Evaluation of settlements in sand deposits following liquefaction during earthquakes, Soils and Foundations*, Vol. 32, No. 1, pp. 173-188.

Makdisi, F. and Seed H.B. (1978). *Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations,* Journal of Geotechnical Engineering; 104(7), 849-867.

Multidisciplinary Center for Earthquake Engineering Research Technical Report MCEER-99-0019, p. 99-114.

National Center for Earthquake Engineering Research (NCEER) (1997). *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils,* Technical Reports NCEER-97-002, Youd, T.L. and Idriss, I.M, eds.

Pradel, Daniel (1998). *Procedure to Evaluate Earthquake-Induced Settlements in Dry Sand*, "Journal of Geotechnical and Geoenvironmental Engineering, April 1998.

Seed, H. Bolton (1979). *The Rankine Lecture, 1979, Considerations in Earthquake-Resistant Design of Earth and Rockfill Dams*, Geotechnique, 29(3), 215-263.

Seed, H. Bolton and Idriss, I.M. (1971). *Simplified Procedure for Evaluating Soil Liquefaction Potential.* Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 97, No. SM9, September.

Seed, H. Bolton and Idriss, I.M. (1982). *Ground Motions and Soil Liquefaction During Earthquakes*. Earthquake Engineering Research Institute, Monograph.

Seed et al. (2003). *Recent Advances in Soil Liquefaction Engineering: A unified and Consistent Framework.* 26th Annual ASCE Los Angeles Geotechnical Spring Seminar

REFERENCES (Cont.)

Southern California Earthquake Center, University of Southern California (2008). *Recommended Procedures for Implementation of DMG. Special Publication 117A. Guidelines for Analyzing and Mitigating Liquefaction in California.*

SWINC Engineering, Inc. (1972). *Crescent City Harbor District Boat Basin,* Improvement Plans

Tokimatsu, K. and Seed, H.B. (1984). *Simplified Procedures for the Evaluation of Settlements in Clean Sands*, Report No. UCB/GT-84/16, Earthquake Engineering Research Center, University of California, Berkeley.

Tokimatsu, K., and Seed, H.B. (1987). *Evaluation of Settlements in Sands Due to Earthquake Shaking*. Journal of the Geotechnical Engineering Division, ASCE, Vol. 113, No. 8.

Townley, S. D. and Allen, M. W. (1939). "Descriptive Catalog of Earthquakes of the Pacific Coast of the United States 1769 to 1928." Bulletin of the Seismological Society of America, 29(1).

United States Geological Survey (2000). Earthquake Hazards Programs, website URL, December 29 2000.

United States Geological Survey (2008). *Documentation for the 2008 Update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open-File Report 2008–1128.*

United States Geological Survey (2010). *Earthquake Hazards Program*, website URL http://earthquake.usgs.gov/hazards/designmaps/javacalc.php, November 2010.

Wells, D. L. and Coppersmith, K. J. (1994). "New Empirical Relationships among Magnitude, Rupture Length, Rupture Width, Rupture Area, and Surface Displacement." Bulletin of the Seismological Society of America, 84(4), 974-1002.

Wesnousky, S. G. (1986). "Earthquakes, Quaternary Faults, and Seismic Hazards in California." Journal of Geophysical Research, 91(1312).

Working Group on California Earthquake Probabilities (WGCEP) (2007). "The Uniform California Earthquake Rupture Forecast, Version 2." Open File Report 2007-1437.

Youd, T.L., et al. (1997). *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, Technical Report NCEER-97-0022, December 31.

Youd, T.L., et al. (2001). *Liquefaction Resistance Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils*, Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Youngs, R. R., and Coppersmith, K. J. (1985). "Implications of Fault Slip Rates and Earthquake Recurrence Models to Probabilistic Seismic Hazard Estimates." *Bulletin of the Seismological Society of America*, 75, 939-964.

FIGURES

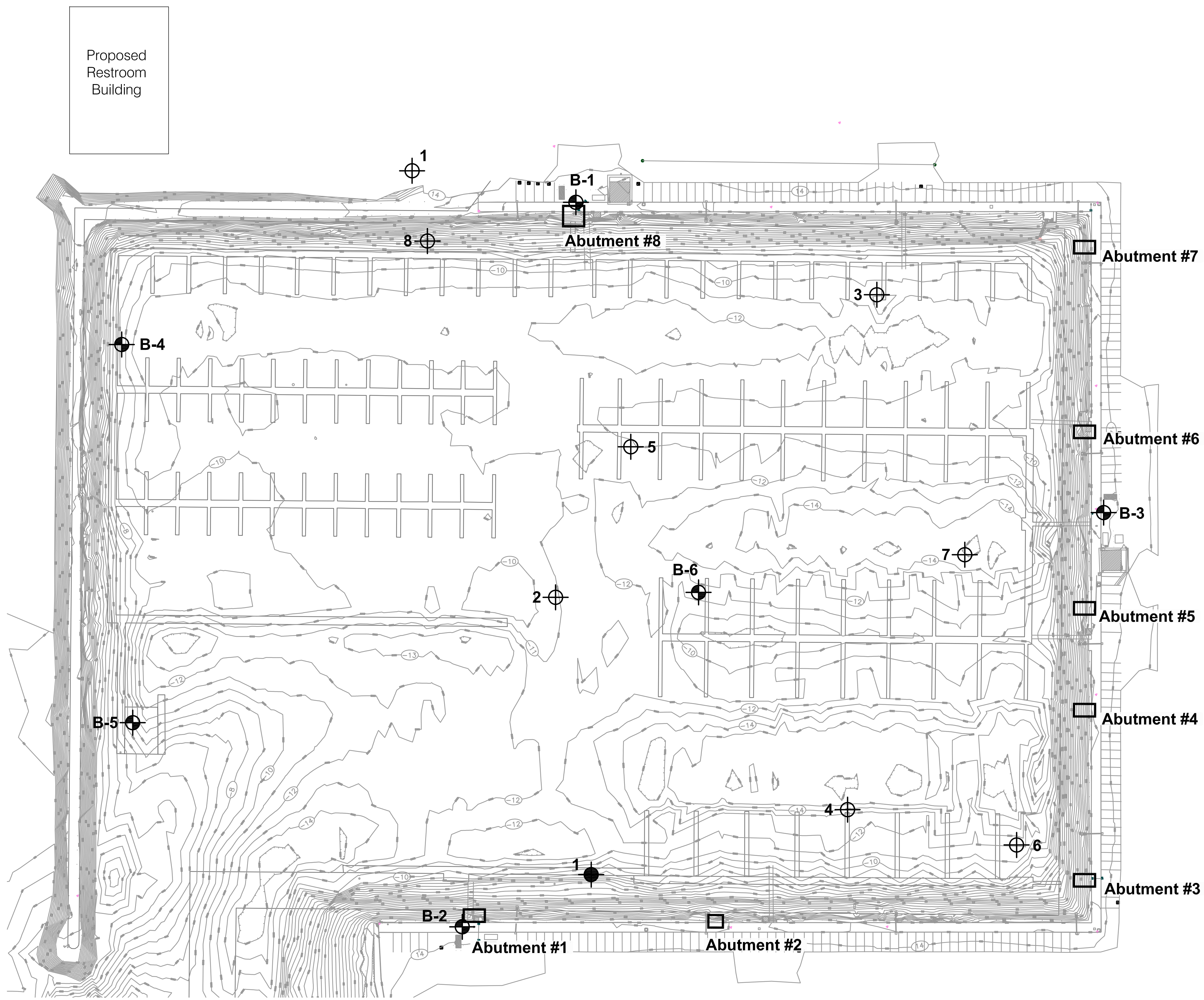
Base map: Google Earth Pro, 2009.

0 1/4 1/2 Mile

Approximate scale

N

CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION Crescent City, California	SITE LOCATION MAP		
Treadwell&Rollo A LANGAN COMPANY	Date 09/30/11	Project No.731577601	Figure 1



EXPLANATION

B-1 — Approximate location of boring by Treadwell & Rollo, Inc., November 2009

1 — Approximate location of boring by others, May 1971

1 — Approximate location of boring by others, September 1972

12 — Existing ground surface elevation, at 1 foot contour interval (feet, MLLW)

0 120 Feet

Approximate scale

Reference: Base map from a drawing titled "Cresent City Harbor District Boat Basin, Crescent City, California, Alternate No. 1 ", by Stover Engineering, dated 05/21/08 and a drawing titled "Inner Harbor Reconstruction 20090918", by Stover Engineering, recieved via email on 12 November 2009.

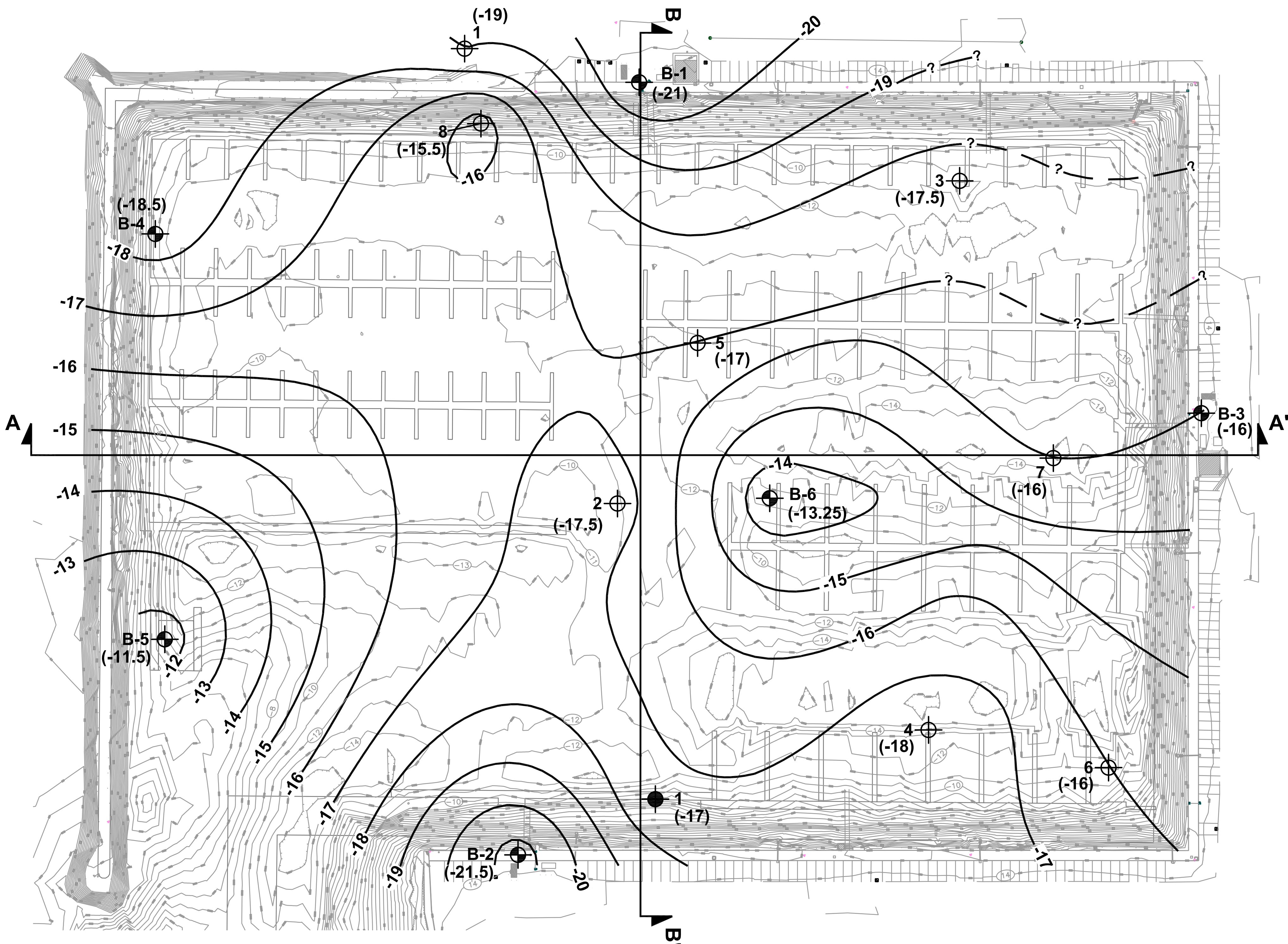
CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

SITE PLAN

Date 10/04/11	Project No. 731577601	Figure 2

Treadwell&Rollo
A LANGAN COMPANY

\\langan.com\data\SF\data6\731577601\Cadd Data - 731577601\731577601 Site Plan.dwg 10/07/11



EXPLANATION

B-1 Approximate location of boring by Treadwell & Rollo, Inc., November 2009

1 Approximate location of boring by others, May 1971

1 Approximate location of boring by others, September 1972

(-17) Elevation of top of bedrock

-15 Estimated Top of bedrock contour elevation (feet, MLLW)

-12 Existing ground surface elevation, at 1 foot contour interval (feet, MLLW)

A A' Idealized subsurface profile location

0 120 Feet

Approximate scale

Reference: Base map from a drawing titled "Cresent City Harbor District Boat Basin, Crescent City, California, Alternate No. 1 ", by Stover Engineering, dated 05/21/08 and a drawing titled "Inner Harbor Reconstruction 20090918", by Stover Engineering, recieved via email on 12 November 2009.

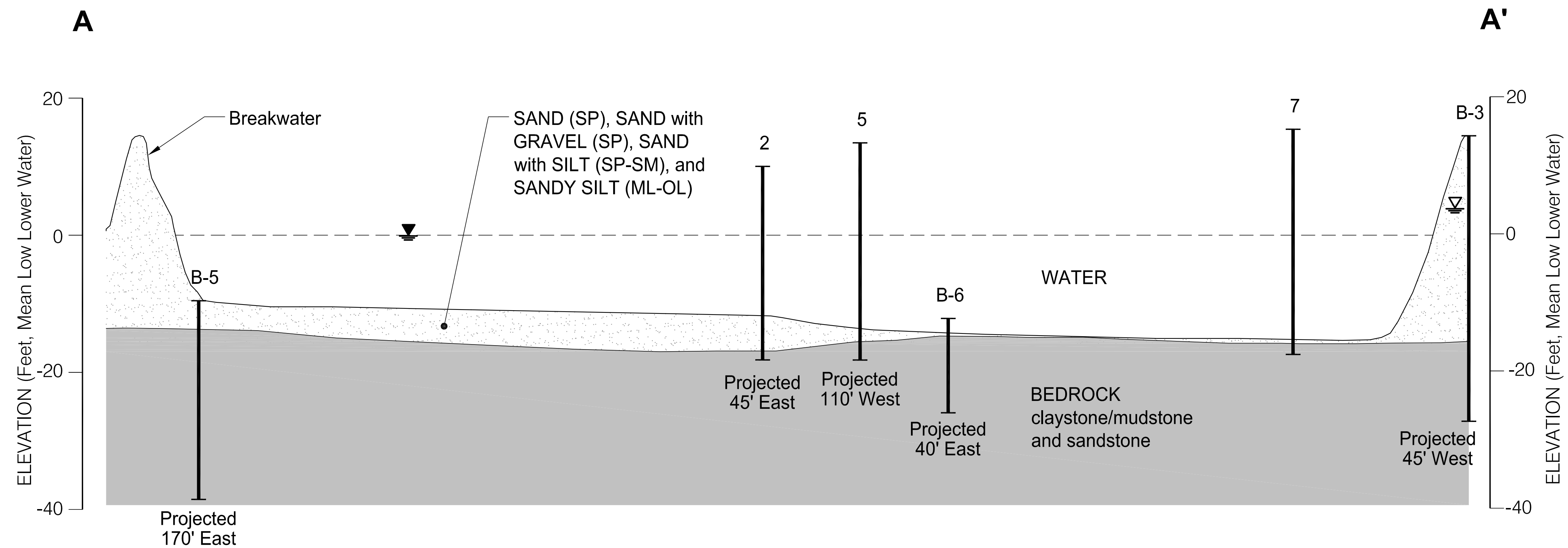
CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

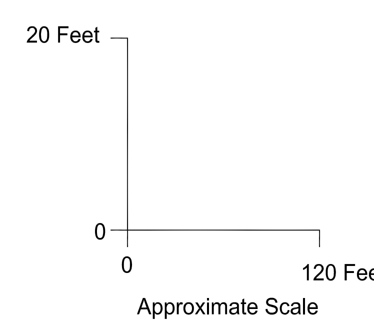
TOP OF BEDROCK CONTOURS

Date 09/30/11	Project No. 731577601	Figure 3

Treadwell&Rollo
A LANGAN COMPANY

\\langan.com\data\SF\data6\731577601\Cadd Data - 731577601\731577601-B-Bed Contours Site Plan.dwg 10/07/11





Notes:
1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.
2. The borings on which the above profile is based were drilled prior to the 11 March 2011 tsunami. The tsunami may have changed the soil profile significantly.

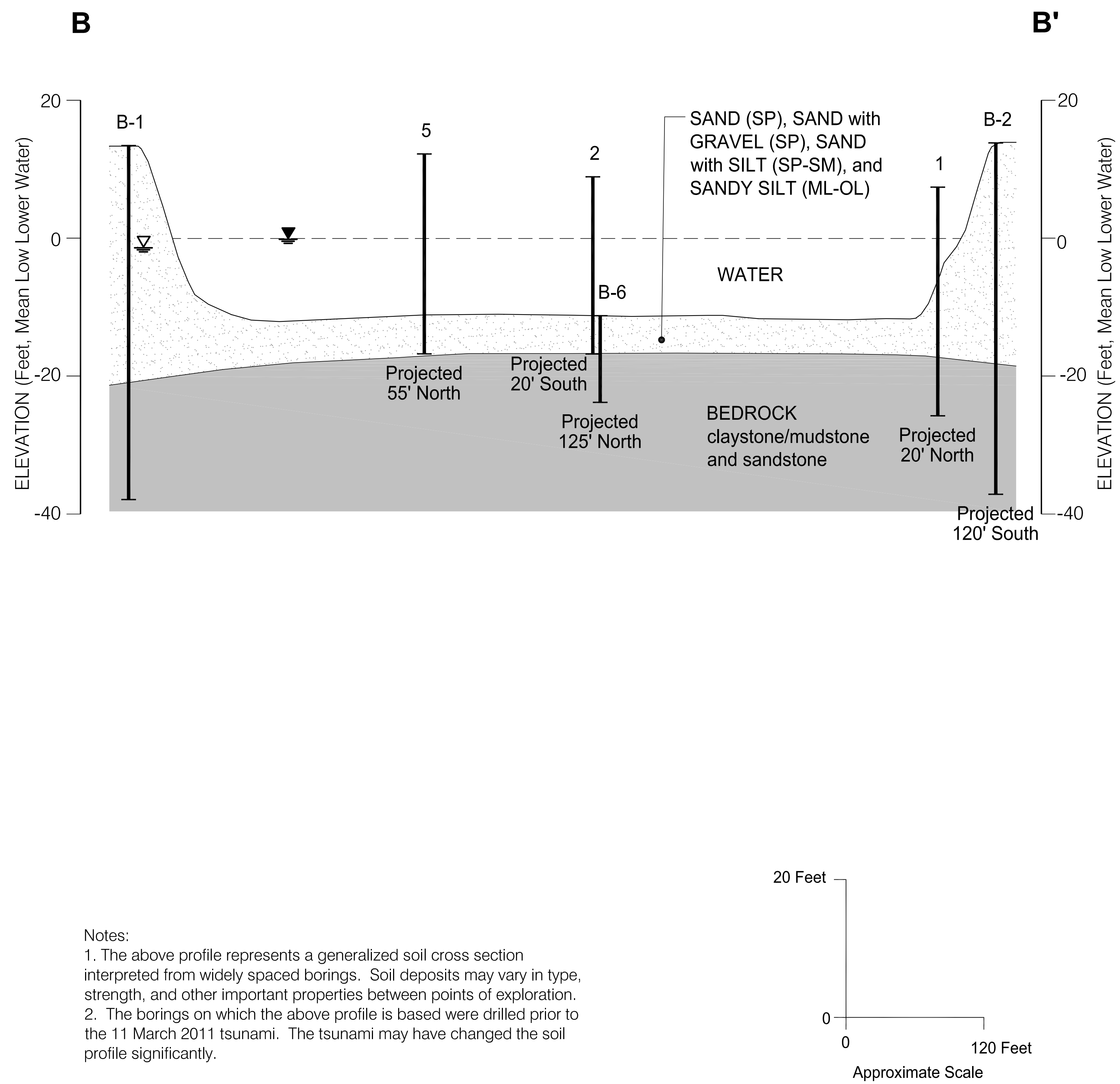
CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

IDEALIZED SUBSURFACE PROFILE A-A'

Date 09/30/11	Project No. 731577601	Figure 4

Treadwell&Rollo
A LANGAN COMPANY

\\langan.com\data\SF\data6\731577601\Cadd Data - 731577601\731577601 Idealized Subsurface Profile A-A' and B-B'.dwg 10/07/11



Notes:
1. The above profile represents a generalized soil cross section interpreted from widely spaced borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.
2. The borings on which the above profile is based were drilled prior to the 11 March 2011 tsunami. The tsunami may have changed the soil profile significantly.

CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

IDEALIZED SUBSURFACE PROFILE B-B'

Date 09/30/11	Project No. 731577601	Figure 5

Treadwell&Rollo
A LANGAN COMPANY

\\langan.com\data\SF\data6\731577601\731577601\Cadd Data - 731577601 Idealized Subsurface Profile A-A' and B-B'.dwg 10/07/11

EXPLANATION

5.6 1962 — Earthquake epicenter with magnitude 4 or greater and date

Fault

0 — 8 Miles

Approximate scale

Reference: Google Earth Pro, 2011;USDA Farm Services Agency; Digital Globe and USGS.

CRESENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION

Crescent City, California

Treadwell&Rollo

A LANGAN COMPANY

MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS

Date 10/07/11	Project No. 731577601	Figure 6

\\langan.com\data\SF\data6\731577601\Cadd Data – 731577601\731577601-B-IU0101.dwg 10/07/11

I	**Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.** Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.	
II	**Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.** As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.	
III	**Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.** Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.	
IV	**Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.** Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.	
V	**Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.** Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.	
VI	**Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.** Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.	
VII	**Frightens everyone. General alarm, and everyone runs outdoors.** People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.	
VIII	**General fright, and alarm approaches panic.** Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.	
IX	**Panic is general.** Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.	
X	**Panic is general.** Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.	
XI	**Panic is general.** Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.	
XII	**Panic is general.** Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.	

CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION Crescent City, California	**MODIFIED MERCALLI INTENSITY SCALE**	
Treadwell&Rollo A LANGAN COMPANY	Date 09/30/11	Project No. 731577601 — Figure 7

APPENDIX A

Logs of Borings

PROJECT:	CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION Crescent City, California	**Log of Boring B-1**

Boring location: See Site Plan, Figure 2

Logged by: R Severn

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)

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 13.5 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						
1					3-inches Aggregate Base						
2				SP	SAND (SP) brown, moist						
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
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					∇ (11/04/09, 4:29 pm) dense						
16	SPT	10 19 27	46								
20	SPT	50 50/ 1/2"	50/ 1/2"		olive-brown and red-brown, very dense						
25	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		32									

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 731577601 | Figure: A-1a

PROJECT: **CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION**
Crescent City, California

Log of Boring B-1

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
30–31	SPT		50/ 1"	50/ 1"	SP	SAND (SP) (continued) olive-gray						
35–36	S&H		24 34 36	42	BEDROCK	MUDSTONE olive-gray, crushed to intensely fractured, soft to low hardness, plastic to weak						
37–38	S&H		50/ 3"	30/ 3"								
40–45	C					RQD = 0 Drill rate = 0.54 feet/min						
45	SPT		50/ 5"	60/ 5"		light olive-gray, soft to low hardness, plastic to friable RQD = 0 Drill rate = 0.53 feet/min						
45.5–50	C											
50–51.5	SPT		23 50/ 3"	60/ 3"		gray, plastic to weak Triaxial Test, see Figure B-3	TxUU	2,250	910		24.1	104

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.: 731577601

Figure: A-1b

PROJECT:	CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION Crescent City, California	Log of Boring B-2

Boring location: See Site Plan, Figure 2		Logged by: R Severn
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)		

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 13 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
						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						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												

PROJECT:	CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION Crescent City, California	Log of Boring B-2 PAGE 2 OF 2

DEPTH (feet)	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
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 Drill rate = 0.20 feet/min						
43	C					BEDROCK						
44												
45						soft to low hardness, plastic to friable						
46						RQD = 0 Drill rate = 0.13 feet/min						
47												
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
A LANGAN COMPANY

Project No.: 731577601
Figure: A-2b

TEST GEOTECH LOG 731577601.GPJ TR.GDT 9/30/11

PROJECT:	CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION Crescent City, California	Log of Boring B-3

Boring location: See Site Plan, Figure 2

Logged by: R Severn

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)

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION 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
0–1						3-inches Asphaltic Concrete 4-inches Aggregate Base						
1–5					SP	SAND (SP) olive-brown, medium dense, moist, fine-grained sand, with trace silt						
5–6.5	SPT		4 7 8	18	SP							
10–11.5	SPT		8 10 12	26	SP	(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					SP-SM	SAND with SILT (SP-SM) olive with orange staining, medium dense, wet						
15–16.5	SPT		5 5 7	14	SP-SM	Particle Size Analysis, see Figure B-1				12.0		
18					SP	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)						
20–21.5	SPT		4 5 9	17	SP							
25–26.5	SPT		11 13 12	30	SP	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						

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 731577601

Figure: A-3a

TEST GEOTECH LOG 731577601.GPJ TR.GDT 9/30/11

DEPTH (feet)	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
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 RQD = 0 Drill rate = 0.31 feet/min						
32	C											
33–34	SPT		10 24 50/ 4"	60/ 4"		low to moderate hardness, friable to weak, with occasional fragments of chert						
35–38	C					RQD = 0 Drill rate = 0.92 feet/min						
39–40	SPT					low hardness, friable						

BEDROCK

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.

PROJECT:	CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION Crescent City, California	Log of Boring B-4 PAGE 1 OF 2

Boring location: See Site Plan, Figure 2	Logged by: R Severn
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)	

Ground Surface Elevation: 10 feet[2]

DEPTH (feet)	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
0–19						WATER						
19–20					ML-OL	SANDY SILT (ML-OL) dark gray, wet, with organics, strong odor						
20–21.5	SPT		25 21 9	36	SP	SAND (SP) dark gray grading to olive-brown, dense, wet, fine-grained sand with trace silt						
25–26.5	SPT		8 15 11	31	SP	olive-gray to dark gray, dense, fine-to medium-grained sand, with shell fragments						
28.5–30	C					SANDSTONE olive-brown to olive-gray, low hardness, weak	UC	N/A	12,260		17.3	107

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 731577601 | Figure: A-4a

DEPTH (feet)	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
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.

PROJECT:	CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION 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)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 10 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
0–19					WATER						
19–20	SPT	0 1 9	12	ML-OL	SANDY SILT (ML-OL) black, very soft, wet, with organics, strong odor						
20–21.5				SP	SAND (SP) dark gray, medium dense, wet, fine-grained sand						
21.5–26	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						
	C				BEDROCK						
26–30	C				MUDSTONE gray, low hardness, friable to weak, with fine-grained sand in rock matrix						

LABORATORY TEST DATA

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 731577601 | Figure: A-5a

TEST GEOTECH LOG 731577601.GPJ TR.GDT 9/30/11

DEPTH (feet)	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
						MUDSTONE (continued)	BEDROCK						
31													
32	C	•											
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.

PROJECT:	CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION 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)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 3.5 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
0–15						WATER						
15–16	SPT		11 15 15	36	ML-OL	SANDY SILT (ML-OL) black, very soft, wet, with organics, strong odor						
16–17.5					SP	SAND (SP) dark gray to olive-brown, dense, wet, fine- to coarse-grained sand, with gravel and shell fragments, trace fines						
17.5–21.5	C					SILTSTONE/MUDSTONE olive-gray, low hardness, friable, moderate weathering, with SANDSTONE cobbles and gravel RQD = 0 Drill rated = 0.32 feet/min						
21.5–27.5	C					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						
26–27.5	C					RQD = 0 Drill rate = 0.09 feet/min						

BEDROCK (17–27.5 feet)

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.

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 731577601 | Figure: A-6

TEST GEOTECH LOG 731577601.GPJ TR.GDT 9/30/11

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	Range of Grain Sizes: 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 SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

Treadwell&Rollo
A LANGAN COMPANY

CLASSIFICATION CHART

Date 09/30/11 | Project No. 731577601 | 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

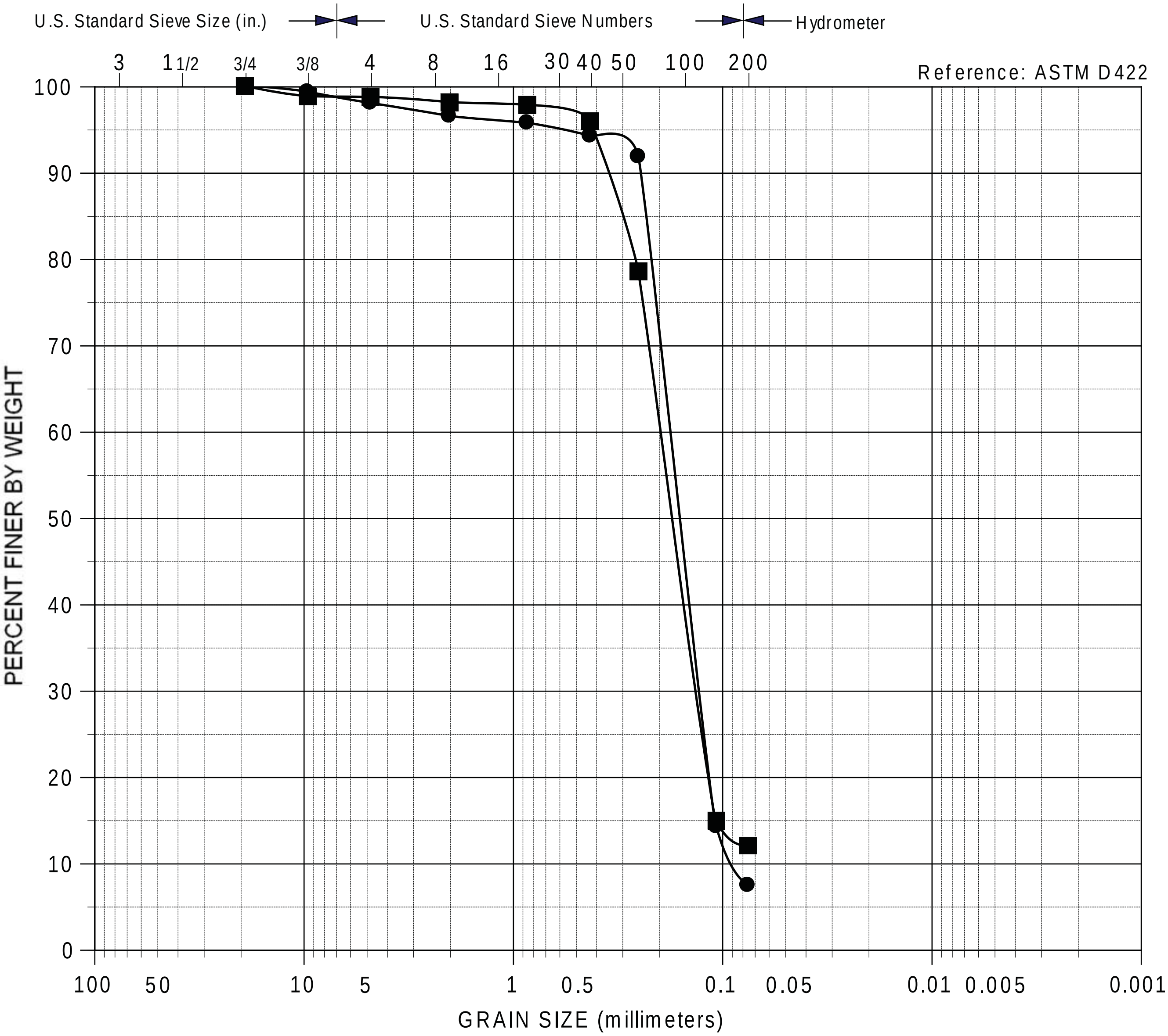
VI BEDDING OF SEDIMENTARY ROCKS

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

Treadwell&Rollo
A LANGAN COMPANY

APPENDIX B

Laboratory Test Results



% Gravel		%Sand			% Fines	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

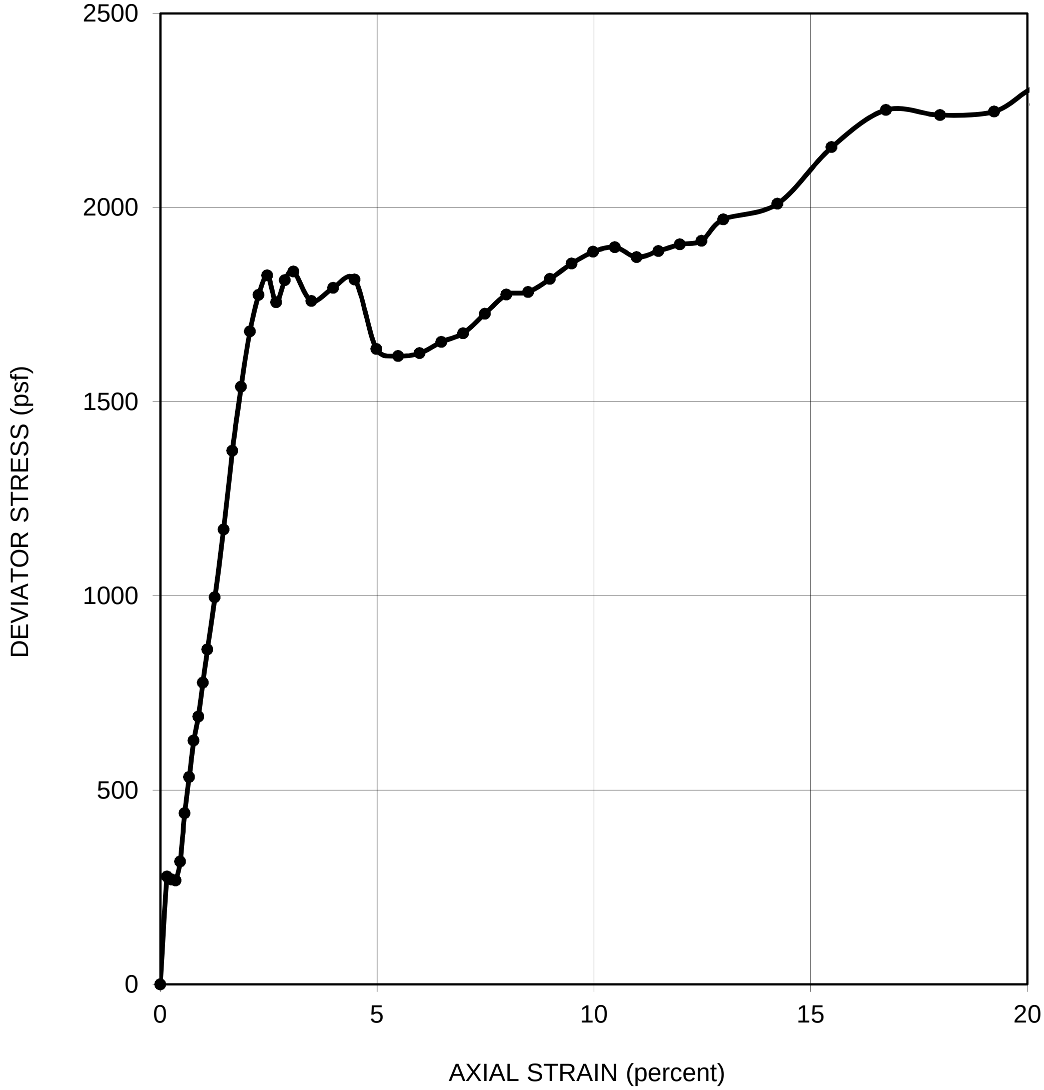
Symbol	Sample Source	Classification
●	B-2 at 10 feet	SAND with SILT (SP-SM), dark gray
■	B-3 at 15 feet	SAND with SILT (SP-SM), olive with orange staining

CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

Treadwell&Rollo
A LANGAN COMPANY

PARTICLE SIZE ANALYSIS

Date 09/30/11 | Project No. 731577601 | Figure B-1



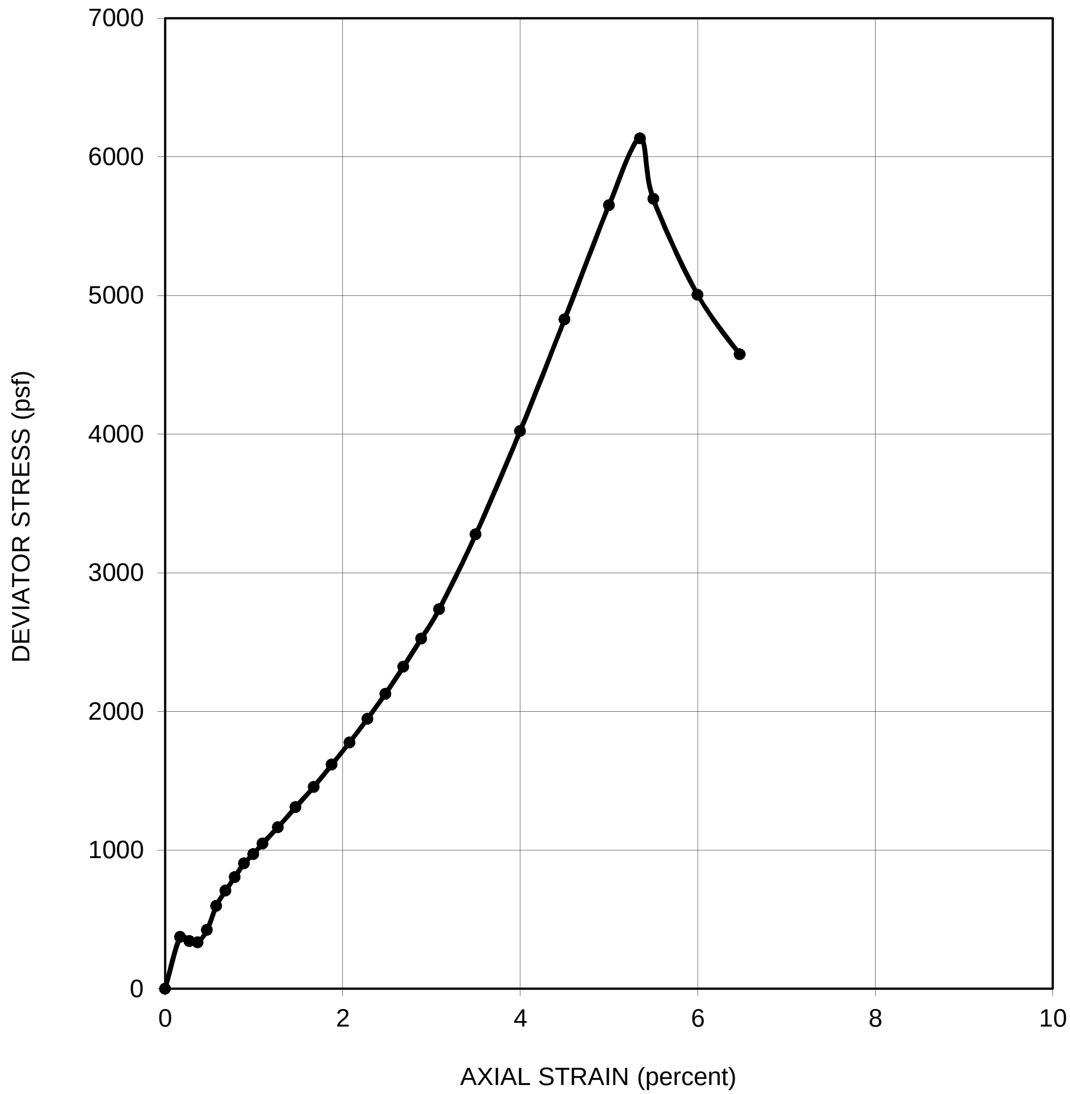
SAMPLER TYPE	Standard Penetration Test (SPT)			SHEAR STRENGTH	910	psf
DIAMETER (in.)	1.49	HEIGHT (in.)	3.62	STRAIN AT FAILURE	2.5	%
MOISTURE CONTENT		24.1	%	CONFINING PRESSURE	2,250	psf
DRY DENSITY		104	pcf	STRAIN RATE	0.25	% / min
DESCRIPTION	MUDSTONE, gray			SOURCE	B-1 @ 50.5 feet	

CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

Treadwell&Rollo
A LANGAN COMPANY

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date	09/30/11	Project No.	731577601	Figure	B-2



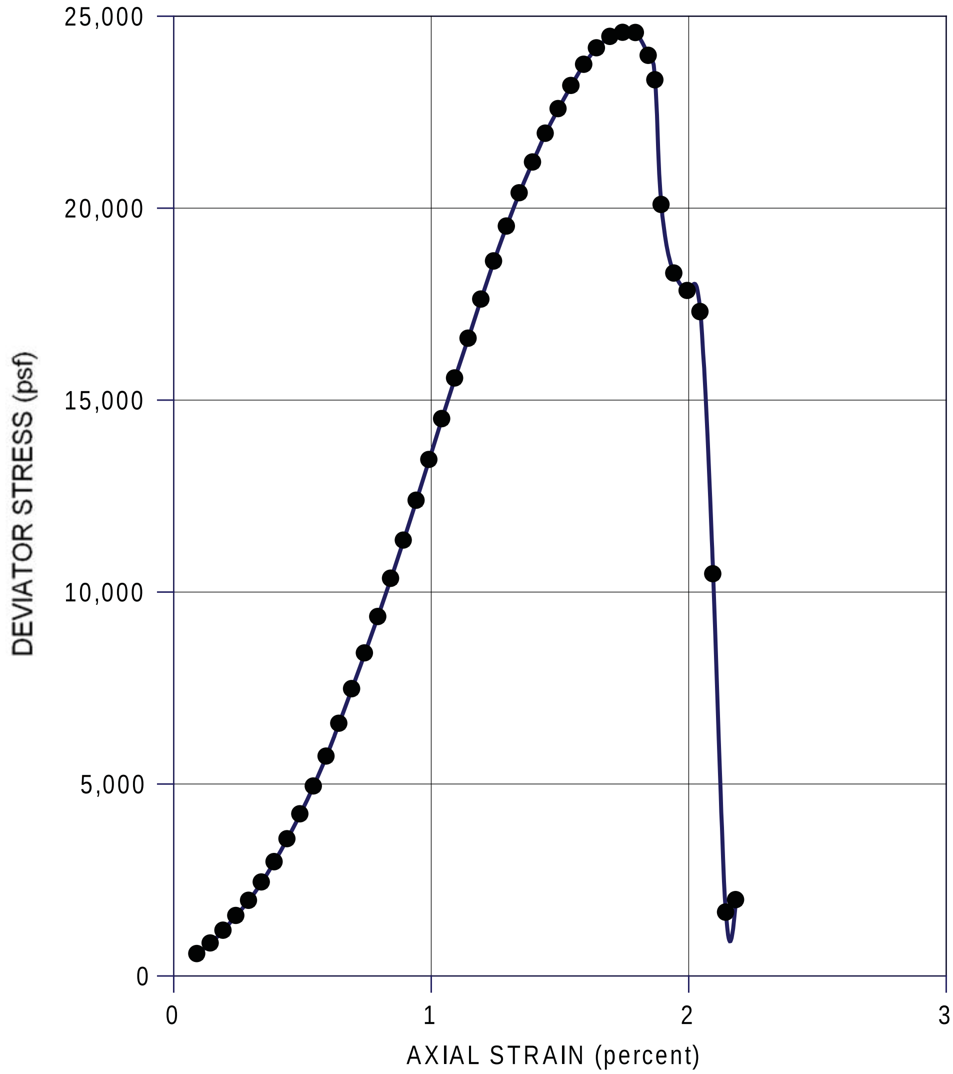
SAMPLER TYPE	Standard Penetration Test (SPT)		SHEAR STRENGTH	3,070	psf
DIAMETER (in.) 1.41	HEIGHT (in.) 3.08		STRAIN AT FAILURE	5.3	%
MOISTURE CONTENT	22.5	%	CONFINING PRESSURE	1,850	psf
DRY DENSITY	107	pcf	STRAIN RATE	0.25	% / min
DESCRIPTION	MUDSTONE, gray		SOURCE	B-2 @ 40 feet	

CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

Treadwell&Rollo
A LANGAN COMPANY

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 09/30/11 | Project No. 731577601 | Figure B-3



SAMPLER TYPE	HQ Core Barrel (C)	SHEAR STRENGTH	12,260 psf
DIAMETER (in) 1.49	HEIGHT (in) 3.18	STRAIN AT FAILURE	1.75 %
MOISTURE CONTENT	17.3 %	CONFINING PRESSURE	0 psf
DRY DENSITY	107 pcf	STRAIN RATE	0.25 %/min
DESCRIPTION	SANDSTONE, olive-brown to olive-gray	SOURCE	B-4 at 29 feet

CRESCENT CITY HARBOR SUPPLEMENTAL GEOTECHNICAL CONSULTATION
Crescent City, California

Treadwell&Rollo
A LANGAN COMPANY

UNCONFINED COMPRESSION TEST

Date 09/30/11	Project No. 731577601	Figure B-4

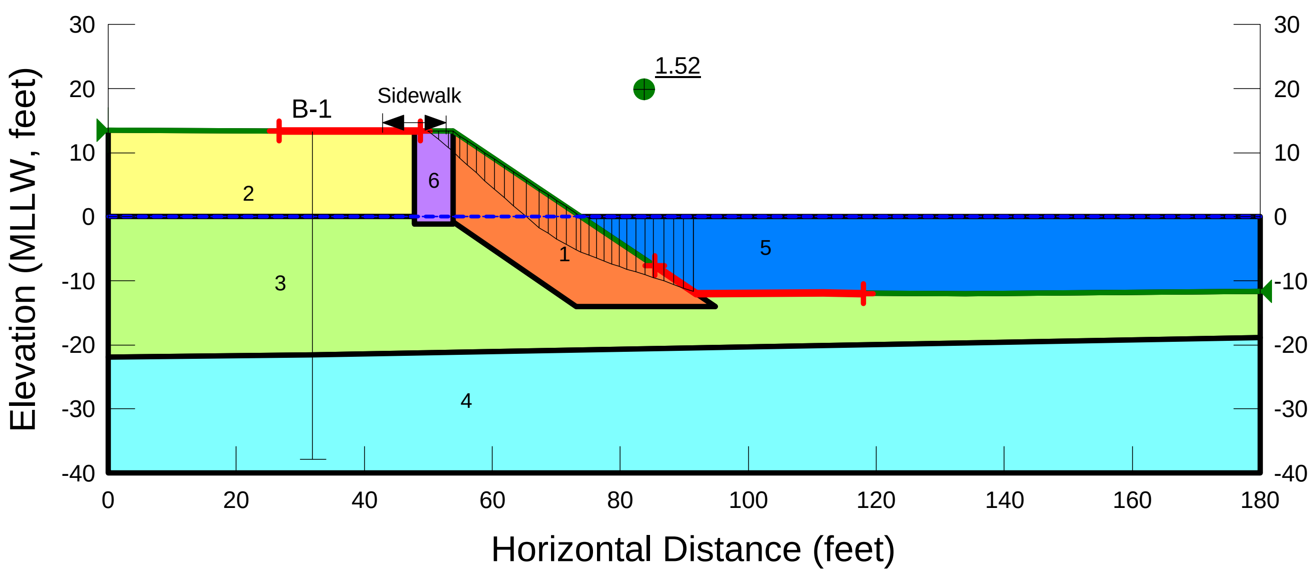
Treadwell&Rollo
A LANGAN COMPANY

APPENDIX C

Slope Stability Results

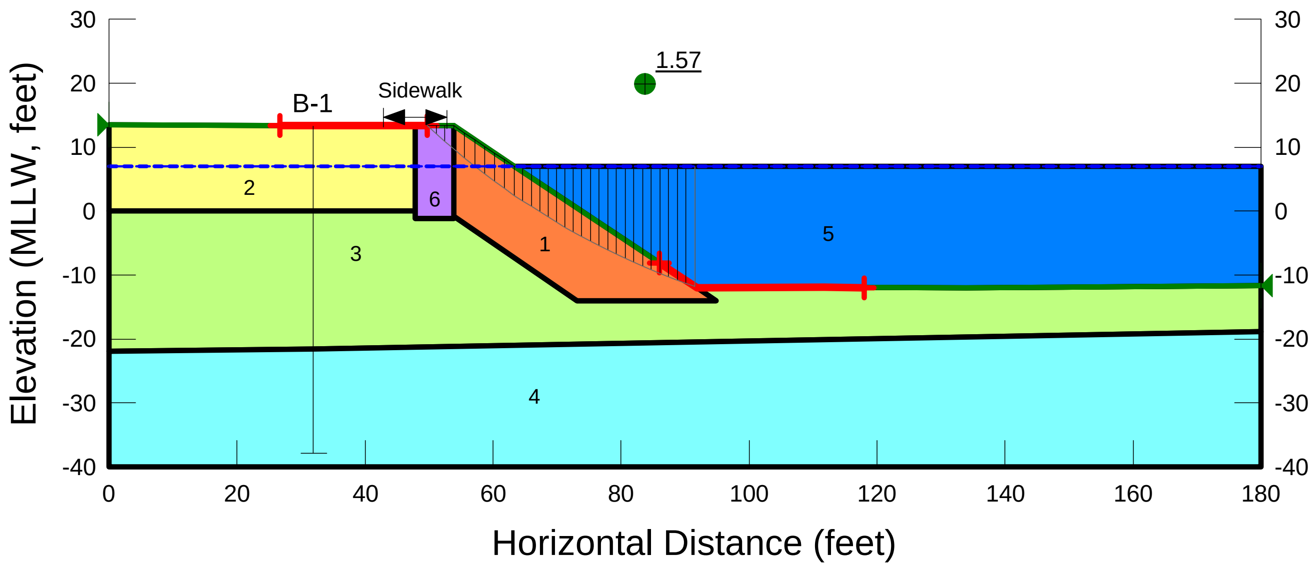
Crescent City Harbor Rehabilitation - Walkway Bent (B-1)
Static Slope Stability 1.5:1 (H:V); MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0 g
Factor of Safety: 1.52

Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium Dense to Dense Sand	125	0	36	1
3	Dense to very Dense Sand	133	0	40	1
4	Bedrock	--	--	--	1
5	Water	62.4	--	--	1
6	Gabion	145	0	45	1



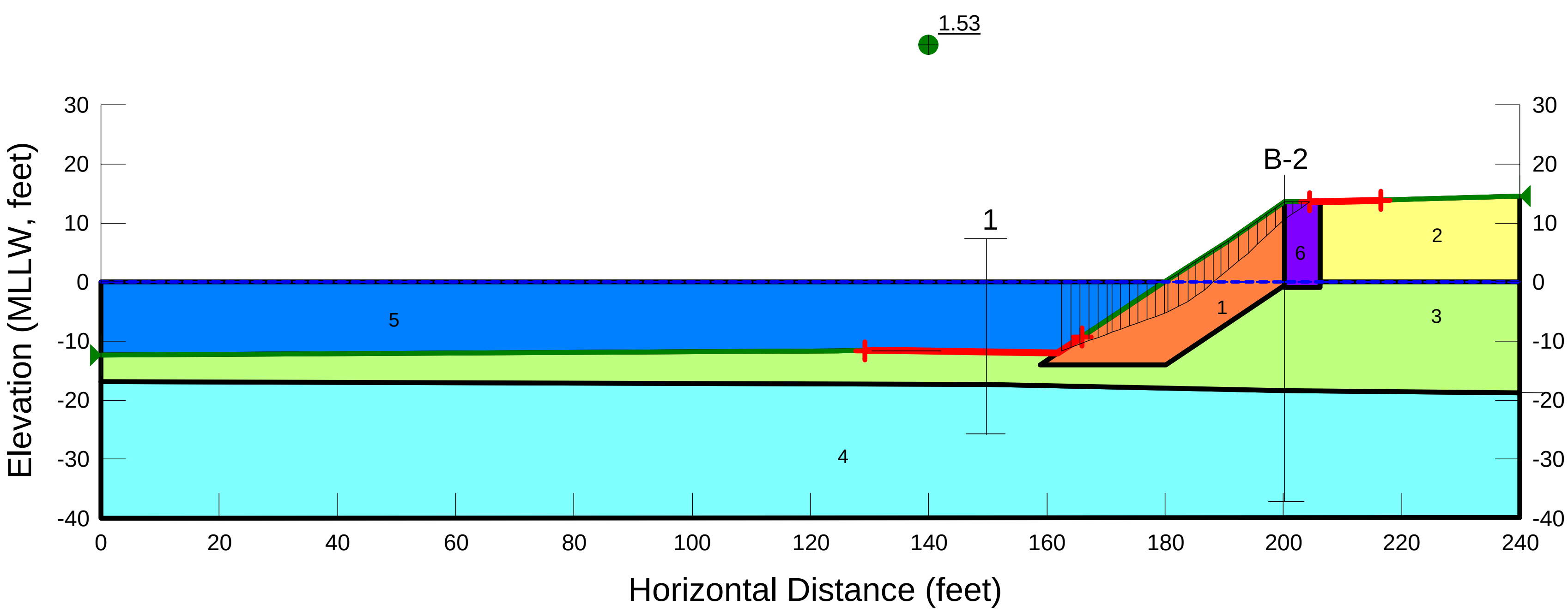
Crescent City Harbor Rehabilitation - Walkway Bent (B-1)
Static Slope Stability 1.5:1 (H:V), 7 feet MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0 g
Factor of Safety: 1.57

Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium Dense to Dense Sand	125	0	36	1
3	Dense to very Dense Sand	133	0	40	1
4	Bedrock	--	--	--	1
5	Water	62.4	--	--	1
6	Gabion	145	0	45	1



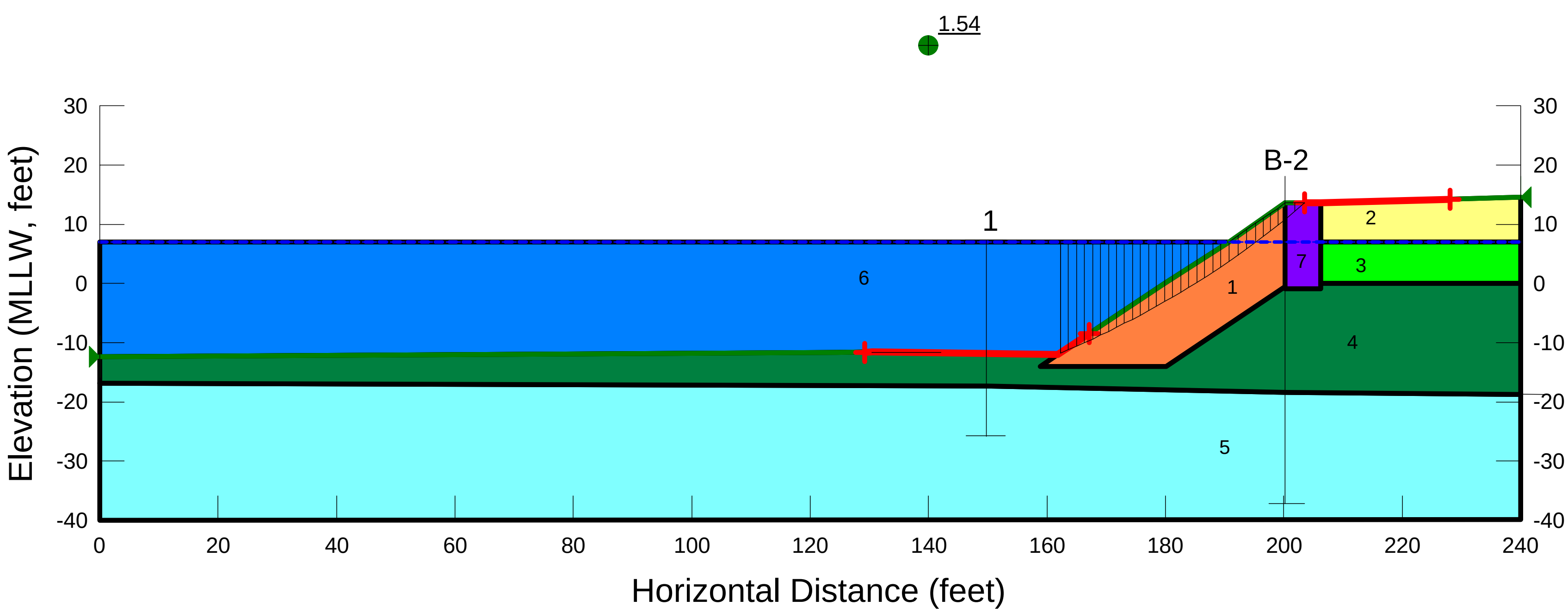
Crescent City Harbor Rehabilitation - Walkway Bent (B-2)
Static Slope Stability 1.5:1 (H:V); MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0 g
Factor of Safety: 1.53

Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium dense to Dense Sand	120	0	36	1
3	Dense to very Dense Sand	133	0	40	1
4	Bedrock	--	--	--	1
5	Water	62.4	--	--	1
6	Gabion	145	0	45	1



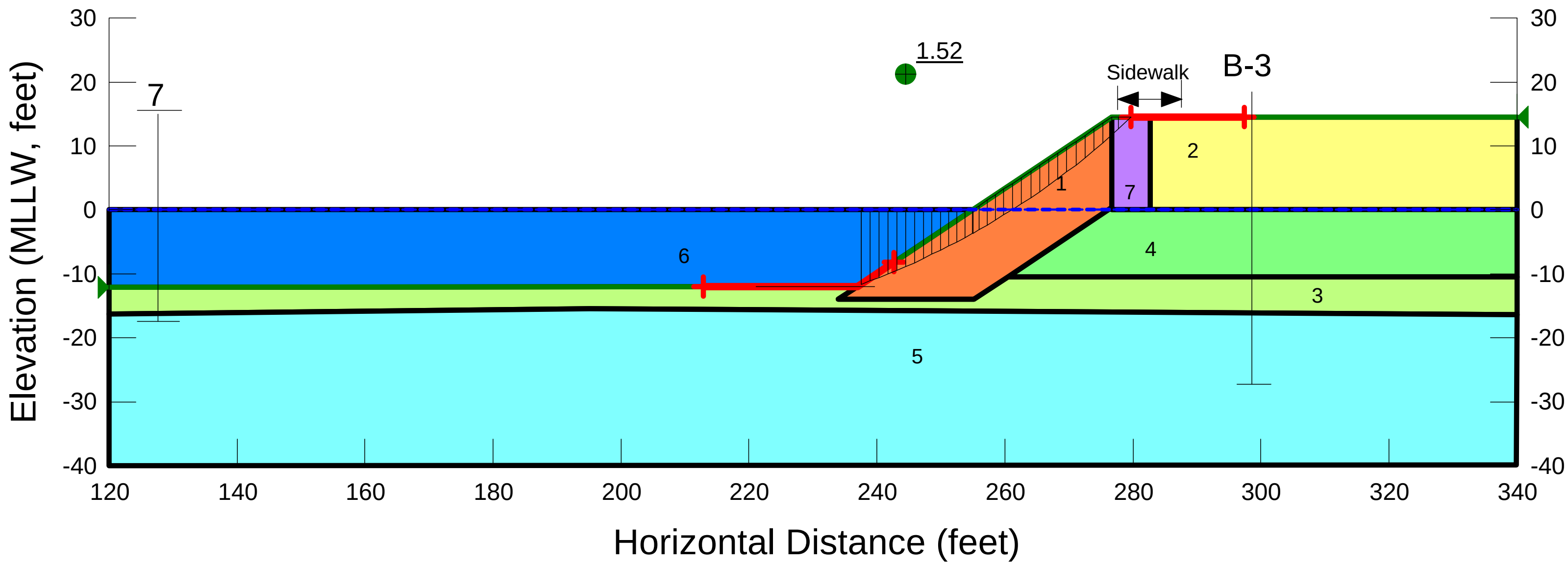
Crescent City Harbor Rehabilitation - Walkway Bent (B-2)
Static Slope Stability 1.5:1 (H:V); 7 feet MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0 g
Factor of Safety: 1.54

Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium dense to Dense Sand	120	0	36	1
3	Potentially Liquefiable Sand	130	0	35	1
4	Dense to very Dense Sand	133	0	40	1
5	Bedrock	--	--	--	1
6	Water	62.4	--	--	1
7	Gabion	145	0	45	1



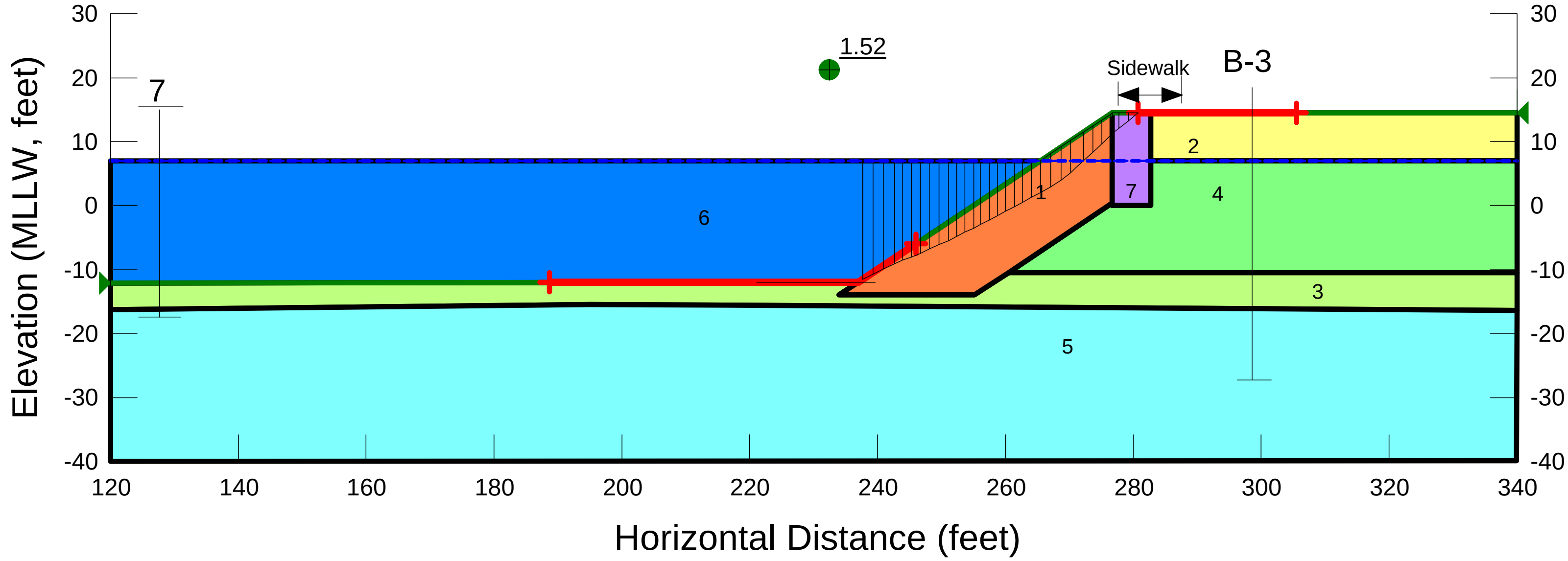
Crescent City Harbor Rehabilitation - Walkway Bent (B-3)
Seismic Slope Stability 1.5:1 (H:V); MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0 g
Factor of Safety: 1.52

Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium Dense to Dense Sand	120	0	36	1
3	Dense to very Dense Sand	133	0	40	1
4	Potentially Liquefiable Sand	130	0	35	1
5	Bedrock	--	--	--	1
6	Water	--	--	--	1
7	Gabion	145	0	45	1



Crescent City Harbor Rehabilitation - Walkway Bent (B-3)
Seismic Slope Stability 1.5:1 (H:V); 7 feet MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0 g
Factor of Safety: 1.52

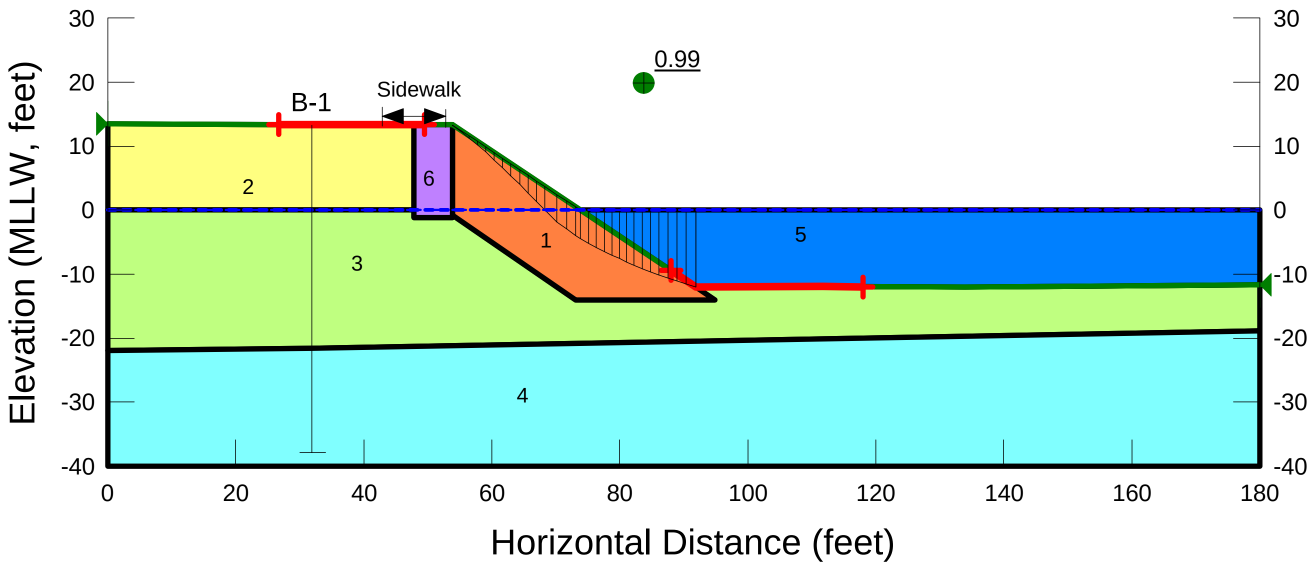
Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium Dense to Dense Sand	120	0	36	1
3	Dense to very Dense Sand	133	0	40	1
4	Potentially Liquefiable Sand	130	0	35	1
5	Bedrock	--	--	--	1
6	Water	--	--	--	1
7	Gabion	145	0	45	1



Yield Coefficient Evaluation

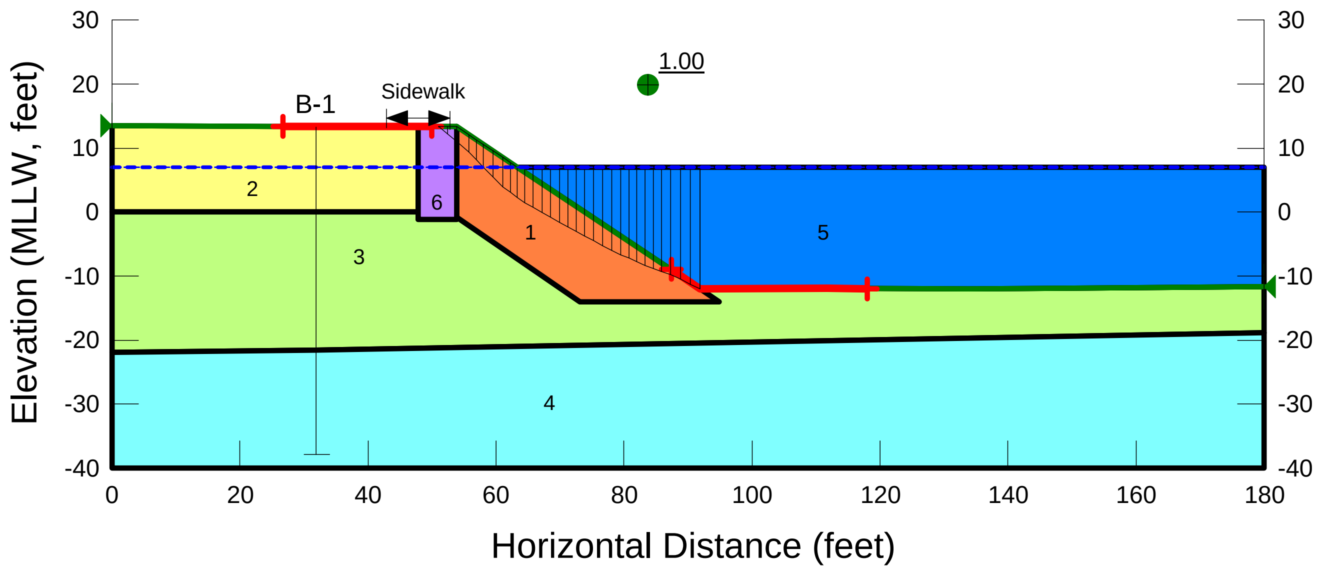
Crescent City Harbor Rehabilitation - Walkway Bent (B-1)
Static Slope Stability 1.5:1 (H:V); MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0.15 g
Factor of Safety: 0.99

Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium Dense to Dense Sand	125	0	36	1
3	Dense to very Dense Sand	133	0	40	1
4	Bedrock	--	--	--	1
5	Water	62.4	--	--	1
6	Gabion	145	0	45	1



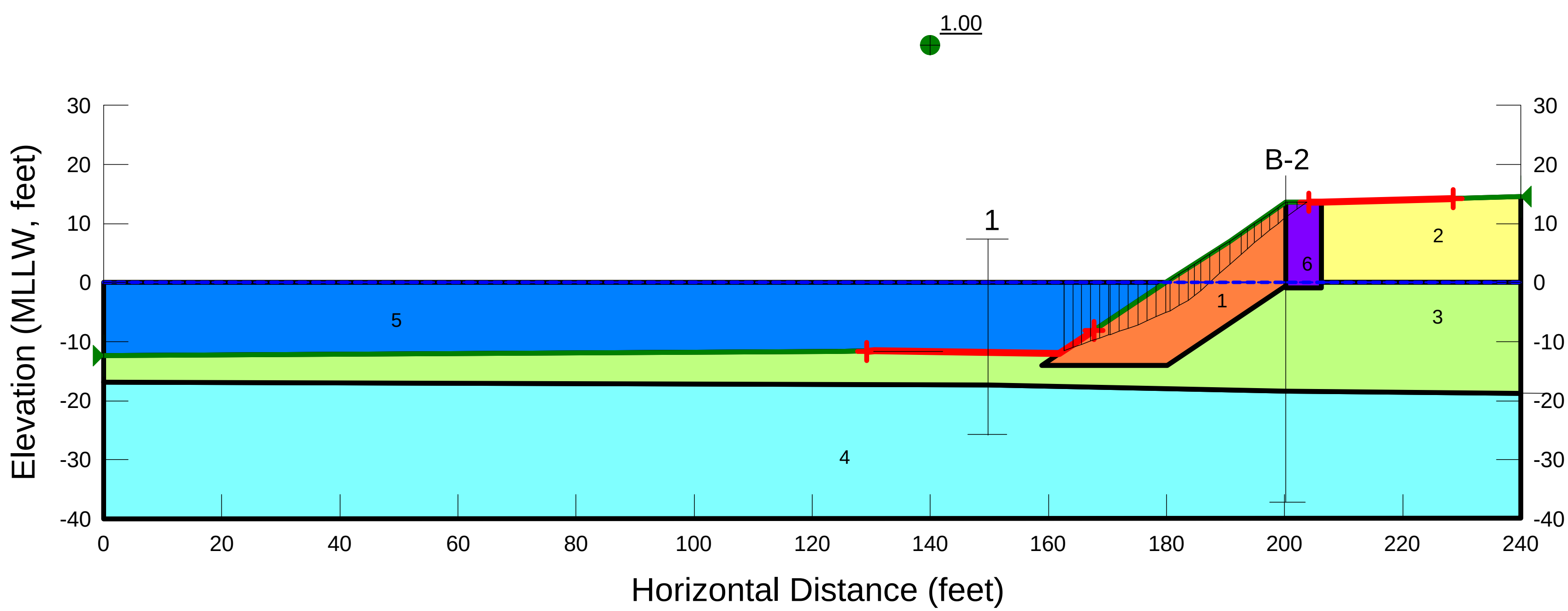
Crescent City Harbor Rehabilitation - Walkway Bent (B-1)
Static Slope Stability 1.5:1 (H:V); 7 feet MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0.135 g
Factor of Safety: 1.00

Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium Dense to Dense Sand	125	0	36	1
3	Dense to very Dense Sand	133	0	40	1
4	Bedrock	--	--	--	1
5	Water	62.4	--	--	1
6	Gabion	145	0	45	1



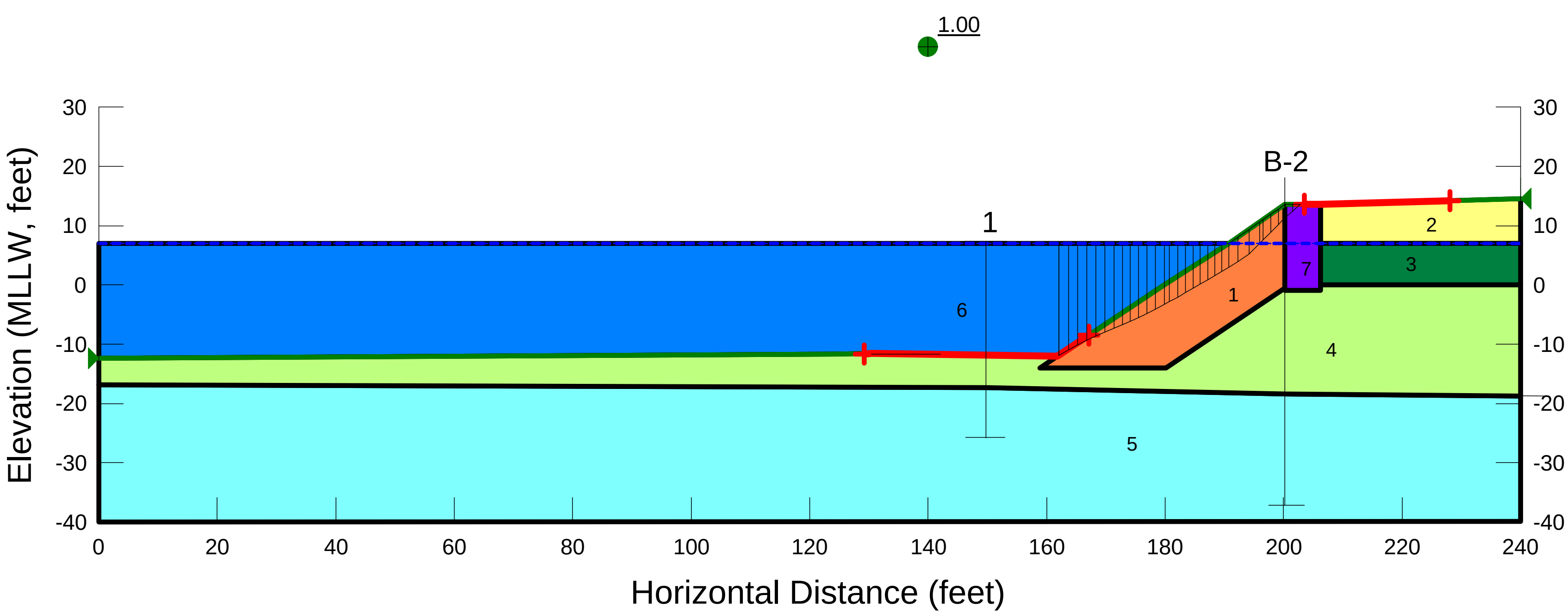
Crescent City Harbor Rehabilitation - Walkway Bent (B-2)
Slope Stability 1.5:1 (H:V); MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0.17 g
Factor of Safety: 1.00

Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium dense to Dense Sand	120	0	36	1
3	Dense to very Dense Sand	133	0	40	1
4	Bedrock	--	--	--	1
5	Water	62.4	--	--	1
6	Gabion	145	0	45	1



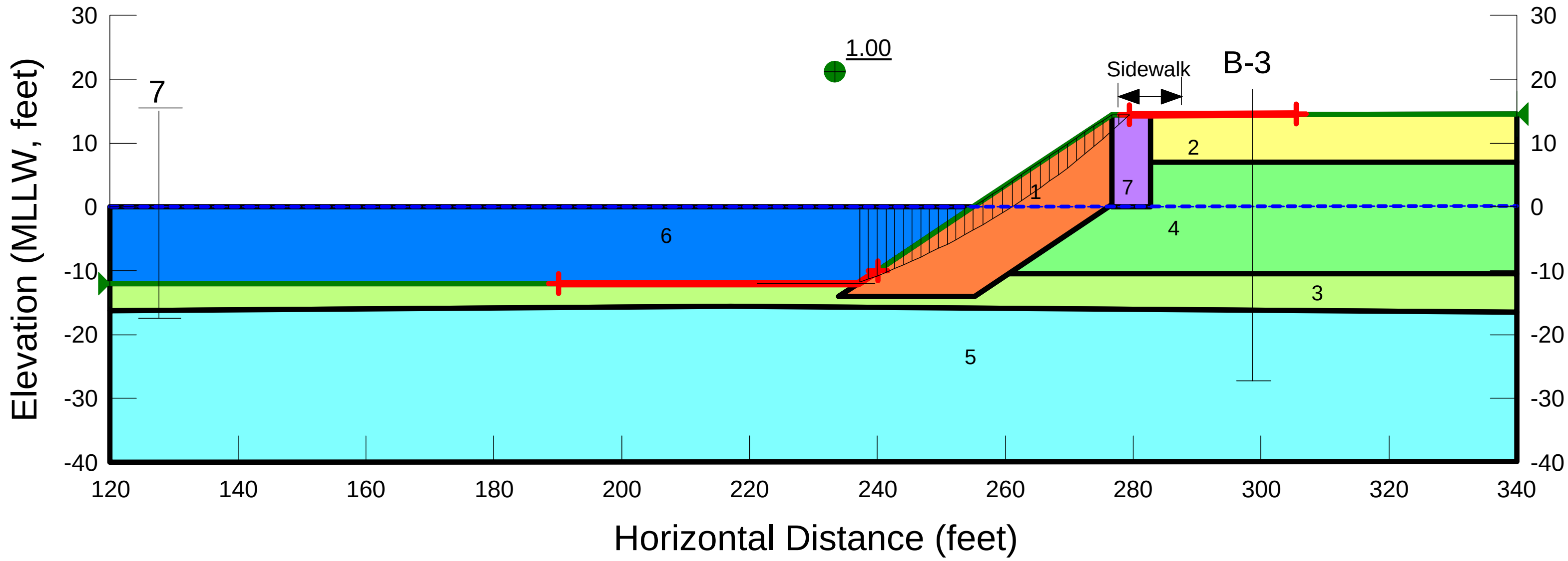
Crescent City Harbor Rehabilitation - Walkway Bent (B-2)
Static Slope Stability 1.5:1 (H:V); 7 feet MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0.135 g
Factor of Safety: 1.00

Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium dense to Dense Sand	120	0	36	1
3	Potentially Liquefiable Sand	130	0	35	1
4	Dense to very Dense Sand	133	0	40	1
5	Bedrock		--	--	1
6	Water	62.4			1
7	Gabion	145	0	45	1



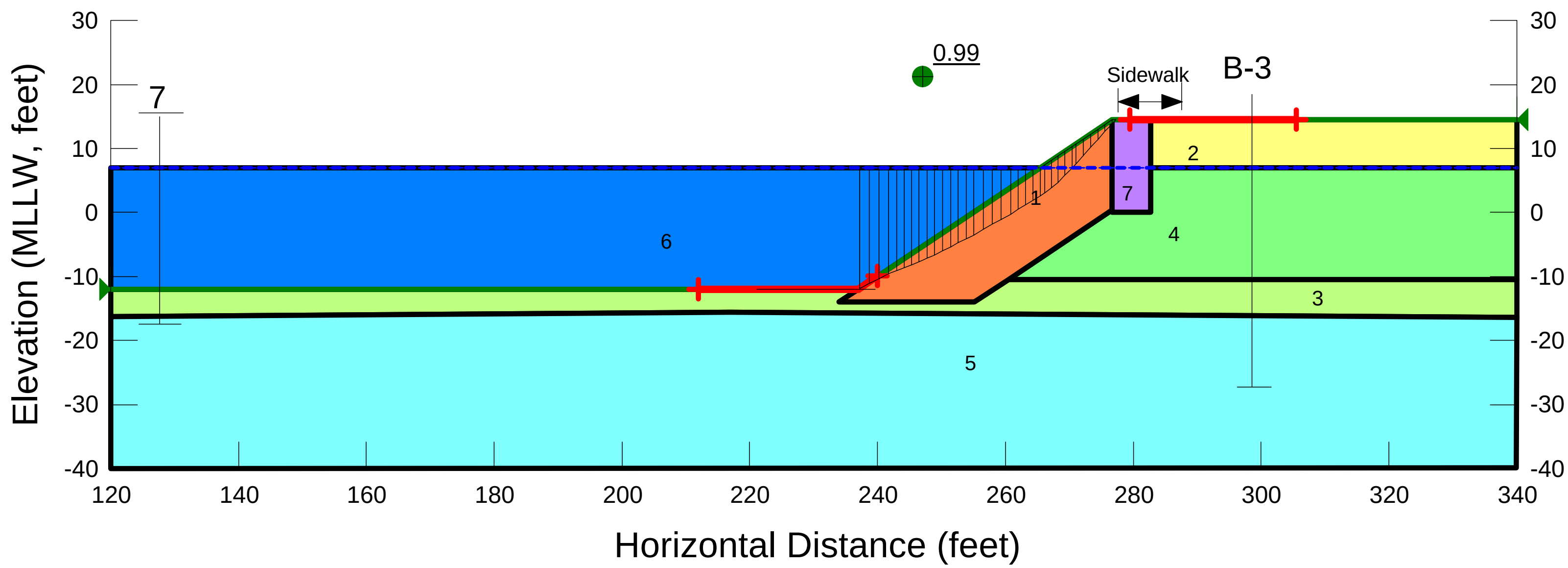
Crescent City Harbor Rehabilitation - Walkway Bent (B-3)
Seismic Slope Stability 1.5:1 (H:V); MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0.175 g
Factor of Safety: 1.00

Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium Dense to Dense Sand	120	0	36	1
3	Dense to very Dense Sand	133	0	40	1
4	Potentially Liquefiable Sand	130	0	35	1
5	Bedrock	--	--	--	1
6	Water	62.4	--	--	1
7	Gabion	145	0	45	1



Crescent City Harbor Rehabilitation - Walkway Bent (B-3)
Seismic Slope Stability 1.5:1 (H:V); 7 feet MLLW
Date: 9/27/2011
Method: Spencer
Horz Seismic Coefficient: 0.13 g
Factor of Safety: 0.99

Material No.	Description	Total Unit Weight (pcf)	Cohesion (pcf)	Friction Angle (Degrees)	Peiziometric Line No.
1	Rip rap	145	0	45	1
2	Medium Dense to Dense Sand	120	0	36	1
3	Dense to very Dense Sand	133	0	40	1
4	Potentially Liquefiable Sand	130	0	35	1
5	Bedrock	--	--	--	1
6	Water	62.4	--	--	1
7	Gabion	145	0	45	1



DISTRIBUTION

2 copies: Mr. Ward Stover
Stover Engineering
711 H Street
Crescent City, California 95531

QUALITY CONTROL REVIEWER:

Ramin Golesorkhi
Associate