GEOTECHNICAL INVESTIGATION

SLEEPY HOLLOW STEELHEAD REARING FACILITY
RAS/TREATMENT BUILDING AND WET WELL
CARMEL VALLEY, CALIFORNIA

FOR
MONTEREY PENINSULA
WATER MANAGEMENT DISTRICT
MONTEREY, CALIFORNIA

Pacific Crest
ENGINEERING INC

CONSULTING GEOTECHNICAL ENGINEERS

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April 16, 2018 Project No. 1809-M274-D63

Larry Hampson, District Engineer
Monterey Peninsula Water Management District
5 Harris Ct., Building G
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Subject: **Geotechnical Investigation – Design Phase**
Sleepy Hollow STEELHEAD Rearing Facility
Proposed RAS/TREATMENT BULDING and Wet Well
Carmel Valley, California

Dear Mr. Hampson,

In accordance with your authorization, we have performed a geotechnical investigation for the proposed improvements to the Sleepy Hollow Steelhead Rearing Facility, in Carmel Valley, California.

The accompanying report presents our findings, conclusions and recommendations for the subject project. If you have any questions concerning the information presented in this report, please call our office.

Very truly yours,

PACIFIC CREST ENGINEERING INC.

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TABLE OF CONTENTS

I. INTRODUCTION 1

PURPOSE AND SCOPE 1
PROJECT LOCATION 1
PROPOSED IMPROVEMENTS 1

II. INVESTIGATION METHODS 2

FIELD INVESTIGATION 2
LABORATORY TESTING 3

III. FINDINGS AND ANALYSIS 3

GEOLOGIC SETTING 3
SURFACE CONDITIONS 3
SUBSURFACE CONDITIONS 3
FAULTING AND SEISMICITY 4
GEOTECHNICAL HAZARDS 5

IV. DISCUSSION AND CONCLUSIONS 8

GENERAL 8
PRIMARY GEOTECHNICAL CONSIDERATIONS 9

V. RECOMMENDATIONS 10

EARTHWORK 10
FOUNDATIONS – RIGID MAT SLAB 15
PAVEMENT DESIGN 17
EROSION CONTROL 19
PLAN REVIEW 19

VI. LIMITATIONS AND UNIFORMITY OF CONDITIONS 19

VII. IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT 21

APPENDIX A

REGIONAL SITE MAP 24
SITE MAP SHOWING TEST BORINGS 25
KEY TO SOIL CLASSIFICATION 26
LOG OF TEST BORINGS 28
LIQUEFACTION ANALYSIS 31

I. INTRODUCTION

PURPOSE AND SCOPE

This report describes the geotechnical investigation and presents our conclusions and recommendations for a proposed RAS/Treatment Building to be constructed in association with the Sleepy Hollow Steelhead Rearing facility in Carmel Valley, in Santa Cruz, California.

Our scope of services for this project has consisted of:

1. Site reconnaissance to observe the existing conditions.

2. Review of the following published maps:
   - Monterey County GIS Website?
   - Geologic Map of the Monterey Peninsula and Vicinity, Monterey County, California, Dibblee Jr., 1999.
   - Geologic Map of Monterey and Seaside 7.5-Minute Quadrangles, Monterey County, California, Clark, Dupré, Rosenberg, 1997.
   - Geologic Map of Monterey County, California, Rosenberg, 2001.
   - Map Showing Liquefaction Susceptibility of Monterey County, California, Rosenberg, 2001.

3. The drilling and logging of 3 test borings.

4. Laboratory analysis of retrieved soil samples.

5. Engineering analysis of the field and laboratory test results.

6. Preparation of this report documenting our investigation and presenting geotechnical recommendations for the design and construction of the project.

PROJECT LOCATION

The steelhead rearing facility is located about one mile downstream from the former San Clemente Dam site along the south side of the Carmel River. Please refer to the Regional Site Map, Figure No. 1, in Appendix A for the general vicinity of the project site, which is located by the following coordinates:

Latitude = 36.443997 degrees
Longitude = -121.716385 degrees

PROPOSED IMPROVEMENTS

A pump house/building and associated utilities is proposed for improvement of the fish rearing facility. The RAS/Treatment building will be about 45 feet by 32 feet in plan dimension and will consist of a concrete slab on grade supporting pumps and a single story structure. A 20 foot deep wet well is proposed adjacent to the pump house, along with utility lines to convey water between the facilities and Carmel River.

The Monterey Peninsula Water Management District is seeking geotechnical advice and design recommendations for the pump house and wet well.

II. INVESTIGATION METHODS

FIELD INVESTIGATION

Three, 8-inch diameter test borings were drilled at the site on March 5, 2018. The approximate location of the test borings are shown on the Regional Site Map, Figure No. 2, in Appendix A. The drilling method used was hydraulically operated continuous flight augers on a truck mounted drill rig. An engineer from Pacific Crest Engineering Inc. was present during the drilling operations to log the soil encountered and to choose sampler type and locations.

Relatively undisturbed soil samples were obtained at various depths by driving a split spoon sampler 18 inches into the ground. This was achieved by dropping a 140 pound hammer a vertical height of 30 inches. The hammer was actuated with a wire winch. The number of blows required to drive the sampler each 6 inch increment and the total number of blows required to drive the last 12 inches was recorded by the field engineer. The outside diameter of the samplers used was 3 inch or 2 inch and is designated on the Boring Logs as “L” or “T”, respectively.

The field blow counts in 6 inch increments are reported on the Boring Logs adjacent to each sample as well as the Standard Penetration Test data (SPT). All STP data has been normalized to a 2 inch O.D. sampler and is reported on the Boring Logs as SPT "N" values. The normalization method used was derived from the second edition of the Foundation Engineering Handbook (H.Y. Fang, 1991). The method utilizes a Sampler Hammer Ratio which is dependent on the weight of the hammer, height of hammer drop, outside diameter of sampler, and inside diameter of sample.

The soils encountered in the borings were continuously logged in the field and visually described in accordance with the Unified Soil Classification System (ASTM D2488) as described in the Boring Log Explanation, Figures No. 3 and 4, in Appendix A. The soil classification was verified upon completion of laboratory testing in accordance with ASTM D2487.

Appendix A contains the site plan showing the locations of the test borings, our borings logs and an explanation of the soil classification system used. Stratification lines on the boring logs are approximate as the actual transition between soil types may be gradual.

LABORATORY TESTING

The laboratory testing program was developed to aid in evaluating the engineering properties of the materials encountered at the site. Laboratory tests performed include:

- Moisture Density relationships in accordance with ASTM D2937.
- Gradation testing in accordance with ASTM D1140 and D422.

The results of the laboratory testing is presented on the boring logs opposite the sample tested and/or presented graphically in Appendix A.

III. FINDINGS AND ANALYSIS

GEOLOGIC SETTING

The surficial geology in the area of the project site is mapped as Granodiorite to Quartz Monzonite (Dibblee, 1999). The granodiorite and quartz monzonite local to the Carmel Valley area are described as "coarse grained plutonic and igneous rocks that are part of the Salinian Block". In our borings bedrock was generally encountered about 15 feet below ground surface and is overlain by alluvial deposits from the nearby Carmel River.

SURFACE CONDITIONS

The RAS building is proposed on a roughly level alluvial terrace located about 14 feet above the elevation of the adjacent Carmel River. A mild, six foot high, 5:1 slope (horiz:vert) is located immediately north of the building as the area slopes gently down to the river. A rough surfaced driveway dead ends roughly where the building is proposed and it appears that about 2 to 3 feet of fill border portions of the north side of the driveway. The perimeter of the area is outlined by the rearing pools.

SUBSURFACE CONDITIONS

Our subsurface exploration consisted of a three test borings drilled in the area of the proposed improvements. The borings extended between 16½ and 20½ feet below existing grade. B-1 was located on the north side of the proposed building and is about 3½ feet lower in elevation than the other two borings. The soil profiles and classifications, laboratory test results and groundwater conditions encountered for each test boring are presented in the Logs of Test Borings, in Appendix A. The general subsurface conditions are described below.

Subsurface conditions encountered within B-1 consisted of a silty sand overlying bedrock. The surficial 4 feet of this material was medium dense and became loose at about 5 feet below ground surface. Granodiorite bedrock was encountered at 14½ feet below ground surface and extended to the base of the boring at 20½ feet. The bedrock was hard and drilling within the bedrock material was slow.

B-2 and B-3 encountered bedrock overlain by interbedded alluvial deposits consisting of sand, silty sand and sand with silt. These materials were generally loose in density. B-3 encountered dense, sandy gravel at 10 feet below ground surface. These alluvial soils are underlain by granodiorite bedrock at 14 to 16 feet below ground surface.

Groundwater was encountered within B-1, B-2 and B-3 at 9, 15 and 14 feet below ground surface respectively. It should be noted that the groundwater level was not allowed to stabilize for more than a few hours; therefore, the actual groundwater level may be higher or lower than initially encountered. The groundwater conditions described in this report reflect the conditions encountered during our drilling investigation in March 2018 at the specific locations drilled. It must be anticipated that the perched and regional groundwater tables may vary with location and could fluctuate with variations in river level, rainfall, runoff, irrigation and other changes to the conditions existing at the time our measurements were made.

Please refer the Logs of Test Borings in Appendix A, for a more detailed description of the subsurface conditions encountered in each of our test borings at the subject site.

FAULTING AND SEISMICITY

Faulting

Mapped faults which have the potential to generate earthquakes that could significantly affect the subject site are listed in Table No. 1. The fault distances are approximate distances based the U.S. Geological Survey and California Geological Survey, Quaternary fault and fold database, accessed November 2017 from the USGS website (http://earthquake.usgs.gov/hazards/qfaults/) and overlaid onto Google Earth.

Table No. 1 - Distance to Significant Faults

Fault Name	Distance (miles)	Direction
Monterey Bay-Tularcitos	1	Northeast
San Andreas	28	Northeast
Reliz	11	Northeast
San Gregorio	11½	West

Seismic Shaking and CBC Design Parameters

Due to the proximity of the site to active and potentially active faults, it is reasonable to assume the site will experience high intensity ground shaking during the lifetime of the project. Structures founded on thick soft soil deposits are more likely to experience more destructive shaking, with higher amplitude and lower frequency, than structures founded on bedrock. Generally, shaking will be more intense closer to earthquake epicenters. Thick soft soil deposits large distances from earthquake epicenters, however, may result in seismic accelerations significantly greater than expected in bedrock.

Selection of seismic design parameters should be determined by the project structural designer. The site coefficients and seismic ground motion values shown in the table below were developed based on CBC 2016 incorporating the ASCE 7-10 standard, and the project site location.

Table No. 2 - 2016 CBC Seismic Design Parameters [1]

Seismic Design Parameter	ASCE 7-10 Value
Site Class	D[3]
Spectral Acceleration for Short Periods	S_s = 1.34g
Spectral Acceleration for 1-second Period	S_1 = 0.49g
Short Period Site Coefficient	F_a = 1.0
1-Second Period Site Coefficient	F_v = 1.5
MCE Spectral Response Acceleration for Short Period	S_{MS} = 1.34g
MCE Spectral Response Acceleration for 1-Second Period	S_{M1} = 0.74g
Design Spectral Response Acceleration for Short Period	S_{DS} = 0.89g
Design Spectral Response Acceleration for 1-Second Period	S_{D1} = 0.49g
Seismic Design Category [2]	D

Note 1: Design values have been obtained by using the Ground Motion Parameter Calculator available on the USGS website at http://earthquake.usgs.gov/hazards/designmaps/usdesign.php.
Note 2: The Seismic Design Category assumes a structure with Risk Category I, II or III occupancy as defined by Table 1604.5 of the 2016 CBC. Pacific Crest Engineering Inc. should be contacted for revised Table 2 seismic design parameters if the proposed structure has a different occupancy rating than that assumed.
Note 3: The site would normally be Site Class F because it is underlain by potentially liquefiable soils If the fundamental period of vibration of the structures is less than 0.5 seconds, the site class can be determined by assuming there is no liquefaction (ASCE 7-05 Section 20.3.1). Therefore, Site Class D was selected for the project site

The recommendations of this report are intended to reduce the potential for structural damage to an acceptable risk level, however strong seismic shaking could result in architectural damage and the need for post-earthquake repairs. It should be assumed that exterior improvements such as pavements or sidewalks may need to be repaired or replaced following strong seismic shaking.

GEOTECHNICAL HAZARDS

The geotechnical hazards associated with the project site include seismic shaking (discussed above), ground surface fault rupture, liquefaction, lateral spreading, landsliding and expansive soils. A discussion of these hazards is presented below.

Ground Surface Fault Rupture

Pacific Crest Engineering Inc. has not performed a specific investigation for the presence of active faults at the project site. Based upon our review of the Monterey County Geologic Hazard Maps, the project site is not mapped within a fault hazard zone.

Ground surface fault rupture typically occurs along the surficial traces of active faults during significant seismic events. Since the nearest known active, or potentially active fault trace is mapped approximately one mile from the site, it is our opinion that the potential for ground surface fault rupture to occur at the site should be considered low.

Liquefaction and Lateral Spreading

Based upon our review of the regional liquefaction maps the site is located in an area classified as having a high potential for liquefaction.

Liquefaction tends to occur in loose, saturated fine grained sands and coarse silt, or clays with low plasticity. Liquefaction occurs when the soil grains are cyclically accelerated such that they begin to loose contact, allowing pressurized pore water to flow between soil particles. The soil, which derives its strength from point-to-point contact between grains, can become fluidized, resulting in significantly lower shear strengths. When the cyclic accelerations cease, the water pressure dissipates and the soil grains settle, regaining contact. Settlement can be differential due to the presence of non-homogeneous earth materials and due to differential densification and dewatering processes. Liquefaction can result in bearing failure and differential ground settlement, which can be highly damaging to structures, pavements and utilities.

Substantial advances in liquefaction engineering have occurred over the past 15 years. Liquefaction science has expanded to examine strength loss of low plasticity silts and clays during cyclic earthquake shaking. We have the following understanding of the current state of the liquefaction science:

Classic cyclic liquefaction, as described above, can occur in undrained soil with low cohesion (Plasticity Index less than about 7 to 12). Liquefaction of “sand-like” soils occurs at the “onset of high excess water pressures and large shear strains during undrained cyclic loading” (Boulanger, 2004). Undrained soils with relatively high cohesion (Plasticity Index greater than about 12 to 20) may be subject to “cyclic failure”, which may result in similar surface manifestations as liquefaction. The transition between “cyclic liquefaction” of sand-like soils and “cyclic failure” of clay-like soil is thought to be gradual depending on the fines content, the water content, and the plasticity of the soil.

The potential for liquefaction was evaluated quantitatively for this project, based upon the data obtained from our exploratory borings. Our analysis utilized the software program LiqSVs, version 1.2.1.1 by Geologismiki, which is based upon the recommendations of the NCEER (1997) and SP117 Implementation. The program calculates a factor of safety against liquefaction and also estimates seismically induced settlement due to both liquefaction of saturated soils and of dynamic compaction of loose, unsaturated soils located above the design water table. Please refer to Appendix A for the model parameters and results we obtained.

The following criteria were used for our analysis:

1. Estimated mean peak ground accelerations of 0.51 g and a 7.5 magnitude (M) earthquake occurring on the San Gregorio Fault, as derived from a deaggregation tool available from the USGS website.
2. Design groundwater at 9 to 10 feet below the ground surface.

The results of our analysis indicate that liquefaction occurs for the materials encountered in B-2 between 9½ and 14 feet below ground surface. Our other two borings either encountered gravelly material that is too dense to liquefy or where the water table was absent at the elevation of the loose, potentially liquefiable sands. Dry sand settlements were incorporated in the analysis for the materials above the water table.

Estimated settlements due to liquefaction-induced settlement and dynamic compaction of loose, dry sands were calculated using LiqSVs, based upon the work by Pradel 1998. On the basis of our analysis, we estimate the magnitude of possible seismically-induced ground surface settlement could be in the range of 3 inches or more. Differential settlement is typically estimated to be about ⅔ to ¾ of the total settlement values.

Lateral spreading can occur when a liquefied soil moves toward a free slope face during the cyclic earthquake loading. Liquefaction-induced lateral spreading can also occur on mild slopes (flatter than 5%) underlain by loose sands and a shallow groundwater table. If liquefaction occurs, the unsaturated overburden soil can slide as intact blocks over the lower, liquefied deposit, creating fissures and scarps. Based on the site topography and the lack of a topographical "free face" in the near vicinity, in our opinion the potential of lateral spreading is low to moderate.

It must be cautioned that estimating earthquake-induced settlement is an inexact science and the mathematical model contains many simplifying assumptions. Less accuracy should be expected for more complex predictions involving earthquakes and non-homogeneous subsurface conditions such as those found in the alluvial environment at the subject site. Actual settlement at the site may be greater or less than that predicted and studies have shown that total liquefaction-induced settlement estimates can vary from 50 to 200% of the estimated values.

The recommendations of this report are intended to reduce the potential for structural damage to an acceptable risk level, however strong seismic shaking could result in architectural damage and the need for post-earthquake repairs. It should be assumed that exterior improvements around the building such as pavements, slabs, sidewalks or patios will need to be repaired or replaced following strong seismic shaking. An increased depth of subgrade compaction below exterior improvements will assist in minimizing the damage to these elements.

Landsliding

Landsliding is a hazard which has, and may continue to affect the hillsides that surround the project. A rigorous numerical analysis of the stability of the slopes on and surrounding the project site was beyond our scope of services on this project. Given the relatively gentle sloping topography surrounding the immediate area of the proposed building site, we infer that there is a low hazard of landsliding for the building site. Landslides issuing from the steep slopes to the west of the project area could impact the proposed improvements.

Slope failures can also occur where surface drainage is allowed to concentrate onto unprotected slopes. Appropriate landscaping and good control of surface drainage around the project area becomes very important to reduce potential for shallow slumping of slopes. Erosion control measures should be implemented and maintained. Under no circumstances should surface runoff be directed toward, or discharged upon, any topographic slopes.

Expansive Soils

Surficial soils at the site consist of coarse grained sands and silty sand that have a low plasticity and a low expansion potential.

IV. DISCUSSION AND CONCLUSIONS

GENERAL

1. The results of our investigation indicate that the proposed development is feasible from a geotechnical engineering standpoint, provided our recommendations are included in the design and construction of the project.

2. Grading and foundation plans should be reviewed by Pacific Crest Engineering Inc. during their preparation and prior to contract bidding.

3. Pacific Crest Engineering Inc. should be notified at least four (4) working days prior to any site clearing and grading operations on the property in order to observe the stripping and disposal of unsuitable materials, and to coordinate this work with the grading contractor. During this period, a pre-construction conference should be held on the site, with at least the client or their representative, the grading contractor, a County representative and one of our engineers present. At this meeting, the project specifications and the testing and inspection responsibilities will be outlined and discussed.

4. Field observation and testing must be provided by a representative of Pacific Crest Engineering Inc., to enable them to form an opinion as to the degree of conformance of the exposed site conditions to those foreseen in this report, the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the earthwork construction and the degree of compaction comply with the specification requirements. Any work related to grading or foundation excavation that is performed without the full knowledge and direct observation of Pacific Crest Engineering Inc., the Geotechnical Engineer of Record, will render the recommendations of this report invalid, unless the Client hires a

new Geotechnical Engineer who agrees to take over complete responsibility for this report's findings, conclusions and recommendations. The new Geotechnical Engineer must agree to prepare a Transfer of Responsibility letter. This may require additional test borings and laboratory analysis if the new Geotechnical Engineer does not completely agree with our prior findings, conclusions and recommendations.

PRIMARY GEOTECHNICAL CONSIDERATIONS

5. Based upon the results of our investigation, it is our opinion that the primary geotechnical issues associated with the design and construction of the proposed project are the following:

a. *Seismically Induced Settlement:* Total seismically induced settlement is estimated to be on the order of about 3 inches at the site, with differential settlements of roughly 2 inches. To reduce potential damage to proposed improvements we recommend the RAS building be founded on a mat foundation that is designed to behave as a structural unit, and to resist differential ground settlement and span seismically induced voids. Preliminary design recommendations are provided in the Foundations section of this report.

b. *Loose Surficial Soils*: Surficial soils at the site consist of loose to medium dense silty/clayey sands. Improvements that bear on these materials may be subject to settlement and distress. To reduce the magnitude of potential settlement we recommend the concrete slab that supports the treatment building be underlain by at least three feet of compacted engineered fill. Detailed recommendations are provided in the earthwork section of this report.

c. *Difficult Excavation Conditions*: The wet well will extend about 20 feet below ground surface and construction excavations will likely encounter difficult drilling conditions with caving cohesionless soils, cobbles and boulders and hard bedrock conditions where excavations extend greater than about 14 feet below ground surface. Underground contractors should be made aware of this difficult drilling conditions at the site and employ the appropriate equipment.

d. *Backfilling of Wet Wells/Vaults*: Depending on the dimensions of overexcavations in the area of the wet well or other underground structures different means of backfilling may be preferable. If the excavation walls are relatively tight against the improvements then backfilling any voids with sand-cement slurry may be the most practical method. If excavations are laid back exposing larger areas then backfilling with compacted engineered fill may be preferable. All excavations should be backfilled by whichever of these means provides the most stable subgrade and reduces the potential for future settlement around the structure. All backfilling processes should be tested and approved by our representative in the field.

e. *Strong Seismic Shaking:* The project site is located within a seismically active area and strong seismic shaking is expected to occur within the design lifetime of the project. Improvements should be designed and constructed in accordance with the most current CBC and the recommendations of this report to minimize reaction to seismic shaking. Structures built in accordance with the latest edition of the California Building Code have an increased potential for experiencing relatively minor damage which should be repairable, however strong seismic shaking could result in architectural damage and the need for post-earthquake repairs.

V. RECOMMENDATIONS

EARTHWORK

Clearing and Stripping

1. The initial preparation of the site may consist of removal of any designated trees and debris. Tree removal, if needed, should include the entire stump and root ball. Septic tanks and leaching lines and any buried structures if found, must be completely removed. The extent of this soil removal will be designated by a representative of Pacific Crest Engineering Inc. in the field. This material must be removed from the site.

2. Any voids created by the removal of tree and root balls, septic tanks, and leach lines must be backfilled with properly compacted engineered fill which meets the requirements of this report.

3. Any wells encountered shall be capped in accordance with the requirements and approval of the County Health Department. The strength of the cap shall be equal to the adjacent soil and shall not be located within 5 feet of a structural footing.

4. Surface vegetation, tree roots and organically contaminated topsoil should then be removed ("stripped") from the area to be graded. In addition, any remaining debris or large rocks must also be removed (this includes asphalt or rocks greater than 2 inches in greatest dimension). This material may be stockpiled for future landscaping.

5. It is anticipated that the depth of stripping may be 2 to 4 inches. Final required depth of stripping must be based upon visual observations by a representative of Pacific Crest Engineering Inc., in the field. The required depth of stripping will vary based upon the type and density of vegetation across the project site and with the time of year.

Subgrade Preparation

6. It is possible that there are areas of man-made fill at the site that our field investigation did not detect. Areas of man-made fill, if encountered, will need to be completely excavated to undisturbed native material. The excavation process should be observed and the extent designated by a representative of Pacific Crest Engineering Inc., in the field. Any voids created by fill removal must be backfilled with properly compacted engineered fill.

7. After clearing and stripping the exposed soils in areas to receive exterior/interior concrete slabs-on-grade should be subexcavated to a minimum depth of 36 inches below bottom of all foundations.

Areas proposed for pavement should be subexcavated at least 18? inches below the base of the pavement section.

8. Subexcavations should extend at least 5 feet horizontally beyond foundations and at least 2 feet horizontally beyond pavements and flatwork.

9. Final depth of subexcavation should be determined by a representative of Pacific Crest Engineering Inc., in the field.

10. Following clearing, stripping and any necessary subexcavations, the exposed subgrade soil that is to support concrete slabs-on-grade, foundations, and pavements should then be scarified 8 inches, and the soil moisture conditioned and compacted as outlined below. The moisture conditioning procedure will depend upon the time of year that the work is done, but it should result in the soils being 1 to 3 percent over optimum moisture content at the time of compaction.

<u>Material for Engineered Fill</u>

11. Native or imported soil proposed for use as engineered fill should meet the following requirements:

a. free of organics, debris, and other deleterious materials,
b. free of "recycled" materials such as asphaltic concrete, concrete, brick, etc.,
c. granular in nature, well graded, and contain sufficient binder to allow utility trenches to stand open,
d. free of rocks in excess of 2 inches in size.

12. In addition to the above requirements, import fill should have a Plasticity Index between 4 and 12, and a minimum Resistance "R" Value of 30, and be non-expansive.

13. Samples of any proposed imported fill planned for use on this project should be submitted to Pacific Crest Engineering Inc. for appropriate testing and approval not less than ten (10) working days before the anticipated jobsite delivery. This includes proposed import trench sand, drain rock and for aggregate base materials. Imported fill material delivered to the project site without prior submittal of samples for appropriate testing and approval must be removed from the project site.

<u>Engineered Fill Placement and Compaction</u>

14. Following the subexcavation and subgrade preparation, areas should be brought up to design grades with engineered fill that is moisture conditioned and compacted according to the recommendations of this report. **This should result in a minimum of 36 inches of engineered fill beneath the proposed concrete slab-on-grade for the RAS building and 18 inches beneath new pavement**. Recompacted sections should extend at least 5 feet horizontally beyond all footings, slabs and pavement areas, where possible.

15. Engineered fill should be placed in maximum 8 inch lifts, before compaction, at a water content which is within 1 to 3 percent of the laboratory optimum value.

16. The soil on the project site should be compacted as follows:

   a. In pavement areas the upper 8 inches of subgrade, and all aggregate subbase and aggregate base, should be compacted to a minimum of 95% of its maximum dry density,
   b. In pavement areas all utility trench backfill should be compacted to 95% of its maximum dry density,
   c. All remaining soil on the project site should be compacted to a minimum of 90% of its maximum dry density.

17. The maximum dry density will be obtained from a laboratory compaction curve run in accordance with ASTM Procedure #D1557. This test will also establish the optimum moisture content of the material. Field density testing will be performed in accordance with ASTM Test #D6938 (nuclear method).

18. We recommend field density testing be performed in maximum 1 foot elevation differences. In general terms, we recommend at least one compaction test per 200 linear feet of utility trench or retaining wall backfill, and at least one compaction test per 2,000 square feet of building or structure area. This is a subjective value and may be changed by the geotechnical engineer based on a review of the final project layout and exposed field conditions.

19. Although not anticipated, engineered fill placed on existing slopes that are steeper than 5:1 (horizontal:vertical) should be keyed and benched into competent native material. Toe keys should be constructed at the base of the fill slope with a minimum 10 foot wide width and sloped negatively at least 2% into the bank. The depth of the keyways will vary, depending on the materials encountered. It is anticipated that the depth of the keyways may be 3 to 6 feet, but at all locations shall be at least 2 feet into firm material.

20. Subsequent benches may be required as the fill section progresses upslope. Benches and keys will be designated in the field by a representative of Pacific Crest Engineering Inc.

Cut and Fill Slopes

21. No permanent cut or fill slopes are presently proposed. The following recommendations are general in nature. We request the opportunity to review grading plans should any cut or fill slopes be proposed to confirm that our recommendations apply and to provide supplemental recommendations as necessary.

22. Fill slopes should be constructed with engineered fill meeting the minimum density requirements of this report and have a gradient no steeper than 2:1 (horizontal to vertical).

23. Permanent cut slopes in soil shall not exceed a 3:1 (horizontal to vertical) gradient.

24. The above slope gradients are based on the strength characteristics of the materials under conditions of normal moisture content that would result from rainfall falling directly on the slope, and do not take into account the additional activating forces applied by seepage from spring areas or subsurface groundwater. Therefore, in order to maintain stable slopes at the recommended gradients, it is important that any seepage forces and accompanying hydrostatic pressure (if encountered) be relieved by adequate drainage. Drainage facilities may include subdrains, gravel blankets, rock fill surface trenches or horizontally drilled drains. Configurations and type of drainage will be determined by a representative of Pacific Crest Engineering Inc. during the grading operations.

25. The surfaces of all cut and fill slopes should be prepared and maintained to reduce erosion. This work, at a minimum, should include track rolling of the slope and effective planting. The protection of the slopes should be installed as soon as practicable so that a sufficient growth will be established prior to inclement weather conditions. It is vital that no slope be left standing through a winter season without the erosion control measures having been provided.

26. The above recommended gradients do not preclude periodic maintenance of the slopes, as minor sloughing and erosion may take place.

27. All flatwork should be set back at least 5 feet horizontally from the top of cut and fill slopes. All foundations should be set back at least 8 feet horizontally from the top of cut and fill slopes.

Soil Moisture and Weather Conditions

28. If earthwork activities are done during or soon after the rainy season, the on-site soils and other materials may be too wet in their existing condition to be used as engineered fill. These materials may require a diligent and active drying and/or mixing operation to reduce the moisture content to the levels required to obtain adequate compaction as an engineered fill. If the on-site soils or other materials are too dry, water may need to be added. In some cases the time and effort to dry the on-site soil may be considered excessive, and the import of aggregate base may be required.

Utility Trench Backfill

29. Utility trenches that are parallel to the sides of the building should be placed so that they do not extend below a line sloping down and away at a 2:1 (horizontal to vertical) slope from the bottom outside edge of all footings.

30. Utility pipes should be designed and constructed so that the top of pipe is a minimum of 24 inches below the finish subgrade elevation of any road or pavement areas. Any pipes within the top 24 inches of finish subgrade should be concrete encased, per design by the project civil engineer.

31. For the purpose of this section of the report, backfill is defined as material placed in a trench starting one foot above the pipe, and bedding is all material placed in a trench below the backfill.

32. Unless concrete bedding is required around utility pipes, free-draining clean sand should be used as bedding. Sand bedding should be compacted to at least 95 percent relative compaction. Clean sand is defined as 100 percent passing the #4 sieve, and less than 5 percent passing the #200 sieve.

33. Approved imported clean sand or native soil should be used as utility trench backfill. Backfill in trenches located under and adjacent to structural fill, foundations, concrete slabs and pavements should be placed in horizontal layers no more than 8 inches thick. This includes areas such as sidewalks, patios, and other hardscape areas. Each layer of trench backfill should be water conditioned and compacted to at least 95 percent relative compaction

34. All utility trenches beneath perimeter footing or grade beams should be backfilled with controlled density fill (such as 2-sack sand\cement slurry) to help minimize potential moisture intrusion below interior floors. The length of the plug should be at least three times the width of the footing or grade beam at the building perimeter, but not less than 36 inches. A representative from Pacific Crest Engineering Inc. should be contacted to observe the placement of slurry plugs. In addition, all utility pipes which penetrate through the footings, stemwalls or grade beams (below the exterior soil grade) should also be sealed water-tight, as determined by the project civil engineer or architect.

35. Utility trenches which carry "nested" conduits (stacked vertically) should be backfilled with a control density fill (such as 2-sack sand\cement slurry) to an elevation one foot above the nested conduit stack. The use of pea gravel or clean sand as backfill within a zone of nested conduits is not recommended.

36. A representative from our firm should be present to observe the bottom of all trench excavations, prior to placement of utility pipes and conduits. In addition, we should observe the condition of the trench prior to placement of sand bedding, and to observe compaction of the sand bedding, in addition to any backfill planned above the bedding zone.

37. Jetting of the trench backfill is not recommended as it may result in an unsatisfactory degree of compaction.

38. Trenches must be shored as required by the local agency and the State of California Division of Industrial Safety construction safety orders.

<u>Excavations and Shoring</u>

39. Excavation of the wet well will encounter cobbles, boulders and very hard bedrock conditions at depths greater than about 14 feet below ground surface. Additionally cohesionless soils are predominant at the site and excavations will be subject to caving. Casing of drilled excavations and employment of equipment capable of drilling through hard rock and cobbles should be expected. Contractors should be made aware of these difficult drilling conditions.

40. It should be understood that on-site safety is the *sole responsibility* of the Contractor, and that the Contractor shall designate a *competent person* (as defined by CAL-OSHA) to monitor the slope

excavation prior to the start of each work day, and throughout the work day as conditions change. The competent person designated by the Contractor shall determine if flatter slope gradients are more appropriate, or if shoring should be installed to protect workers in the vicinity of the slope excavation. Refer to Title 8, California Code of Regulations, Sections 1539-1543.

41. All excavations must meet the requirements of 29 CFR 1926.651 and 1926.652 or comparable OSHA approved state plan requirements.

42. Shallow ground water was encountered and therefore excavation de-watering may be necessary. Groundwater should be expected at shallower depths during or soon after the rainy season. Temporary dewatering may be achieved by sloping the excavation to a system of sump pumps placed within the excavation, trenching from the base of excavations to discharge water by gravity flow, or other means. It is the Contractor's responsibility to design an adequate de-watering system for the project site, and to submit a detailed de-watering plan to the geotechnical engineer for review at least two weeks prior to the start of construction.

43. The "top" of any temporary cut slope and excavations should be set-back at least ten feet (measured horizontally) from any nearby structure or property line. Any excavations which cannot meet this requirement will need to have a shoring system designed to support steeper sidewall gradients.

44. Temporary shoring is not currently anticipated for this project. Should these requirements change, please contact our office for additional recommendations.

<u>FOUNDATIONS – RIGID MAT SLAB</u>

45. At the time we prepared this report, foundation and grading plans had not been completed and the structure location and foundation details had not been finalized. We request an opportunity to review these items during the design stages to determine if supplemental recommendations will be required.

46. An appropriate foundation system to support the proposed treatment building will consist of a reinforced concrete, rigid mat slab system designed to withstand differential settlement and allow the structure to move as a single unit. The loading should be kept as even as possible in all areas of the structure.

47. The mat foundation should be underlain by at least three feet of engineered fill that is placed and compacted to the specifications in the earthwork section of this report.

48. The rigid mat slab should be designed for an allowable bearing capacity of 1,000 psf, (dead plus live load) and may this value may be increased by one-third for wind or seismic loads.

49. Based on the results of our liquefaction analysis, the foundation system should be designed to accommodate a total seismically-induced settlement of 3 inches and a differential settlement of 2 inches across the least dimension of the structure.

50. A coefficient of vertical subgrade reaction of 100 kcf (kips per cubic foot) may be used for design.

51. Lateral loads may be resisted using an ultimate coefficient of friction of 0.35 and an allowable passive soil resistance of 300 psf/ft depth, provided the foundation is constructed directly against compacted engineered fill. The upper 12 inches of soil should be ignored when using passive soil resistance.

52. The structural mat should have thickened edge beams which extend at least 12 inches below lowest adjacent grade, not including sand or gravel sections.

53. Slab thickness, reinforcement, and doweling should be determined by the project structural engineer in accordance with applicable CBC or ACI Standards.

54. Where moisture sensitive equipment or interior floor coverings are anticipated, or anywhere else that vapor transmission may be a problem, the structural mat should be underlain by a minimum 6 inch thick capillary break of ¾ inch clean crushed rock (no fines). It is recommended that neither Class II baserock nor sand be employed as the capillary break material.

55. Where moisture sensitive equipment or interior floor coverings are anticipated, or anywhere else that vapor transmission may be a problem, a vapor retarder/membrane should be placed between the capillary break layer and the floor slab in order to reduce the potential for moisture condensation under floor coverings. We recommend a high quality vapor retarder at least 10 mil thick and puncture resistant (Stego Wrap or equivalent). The vapor retarder must meet the minimum specifications for ASTM E-1745, Standard Specification For Water Vapor Retarder. Please note that low density polyethylene film (such as Visqueen) may meet minimum current standards for permeability but not puncture resistance. Laps and seams should be overlapped at least six inches and properly sealed to provide a continuous layer beneath the entire slab that is free of holes, tears or gaps. Joints and penetrations should also be properly sealed.

56. If required, floor coverings should be installed on concrete slabs that have been constructed according to the guidelines outlined in ACI 302.2R and the recommendations of the flooring material manufacturer.

57. Currently, ACI 302-1R recommends that concrete slabs to receive moisture sensitive floor coverings be placed directly upon the vapor retarder, with no sand cushion. ACI states that vapor retarders are not effective in preventing residual moisture within the concrete slab from migrating to the surface. Including a low water-to-cement ratio (less than 0.50) and/or admixtures into the mix design are generally necessary to minimize water content, reduce soluble alkali content, and provide workability to the concrete. As noted in CIP 29 (Concrete in Practice by the National Ready Mixed

Concrete Association), placing concrete directly on the vapor retarder can also create potential problems. If environmental conditions do not permit rapid drying of bleed water from the slab surface then the excess bleeding can delay finishing operations (refer to CIP 13, 19 and 20). Most of these problems can be alleviated by using a concrete with a low water content, moderate cement factor, and well-graded aggregate with the largest possible size. With the increased occurrence of moisture related floor covering failures, minor cracking of floors placed on a vapor retarder and other problems discussed here are considered a more acceptable risk than failure of floor coverings, and these potential risks should be clearly understood by the Client and Project Owner.

58. If a sand layer is chosen as a cushion for slabs without floor coverings, it should consist of a clean sand. Clean sand is defined as 100 percent passing the #4 sieve, and less than 5 percent passing the #200 sieve.

59. Requirements for pre-wetting of the subgrade soils prior to the pouring of the slabs will depend on the specific soils and seasonal moisture conditions and will be determined by a representative of Pacific Crest Engineering Inc. at the time of construction. It is important that the subgrade soils be properly moisture conditioned at the time the concrete is poured. Subgrade moisture contents should not be allowed to exceed our moisture recommendations for effective compaction, and should be maintained until the slab is poured.

60. Please Note: Recommendations given above for the reduction of moisture transmission through the slab are general in nature and present good construction practice. Moisture protection measures for concrete slabs-on-grade should meet applicable ACI and ASTM standards. Pacific Crest Engineering Inc. are not waterproofing experts. For a more complete and specific discussion of moisture protection within the structure, a qualified waterproofing expert should be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The waterproofing consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure as deemed appropriate.

61. Utility connections entering the building should be designed with flexible connections to accommodate anticipated total settlements.

PAVEMENT DESIGN

62. The design of the pavement section was beyond our scope of services for this project. To have the selected pavement sections perform to their greatest efficiency, it is very important that the following items be considered:

a. Properly scarify and moisture condition the upper 8 inches of the subgrade soil and compact it to a minimum of 95% of its maximum dry density, at a moisture content of 1 to 3% over the optimum moisture content for the soil.

b. Provide sufficient gradient to prevent ponding of water.

c. Use only quality materials of the type and thickness (minimum) specified. All aggregate base and subbase must meet Caltrans Standard Specifications for Class 2 materials, and be angular in shape. All Class 2 aggregate base should be ¾ inch maximum in aggregate size.

d. Compact the base and subbase uniformly to a minimum of 95% of its maximum dry density.

e. Use ½ inch maximum, Type "A" medium graded asphaltic concrete. Place the asphaltic concrete only during periods of fair weather when the free air temperature is within prescribed limits by Cal Trans Specifications.

f. **Porous pavement systems which consist of porous paving blocks, asphaltic concrete or concrete are generally not recommended due to the potential for saturation of the subgrade soils and resulting increased potential for a shorter pavement life. At a minimum, porous pavement systems should include a layer of Mirafi HP370 geotextile fabric placed on the subgrade soil beneath the porous paving section. These pavement systems should only be used with the understanding by the Owner of the increased potential for pavement cracking, rutting, potholes, etc.**

g. Maintenance should be undertaken on a routine basis.

SURFACE DRAINAGE

63. Surface water drainage is the responsibility of the project civil engineer. The following should be considered by the civil engineer in design of the project.

64. Surface water must not be allowed to pond or be trapped adjacent to foundations, or on building pads and parking areas.

65. All roof eaves should be guttered, with the outlets from the downspouts provided with adequate capacity to carry the storm water away from structures to reduce the possibility of soil saturation and erosion. The connection should be in a closed conduit which discharges at an approved location away from structures and graded areas.

66. Slope failures can occur where surface drainage is allowed to concentrate on unprotected slopes. Appropriate landscaping and surface drainage control around the project area is imperative in order to minimize the potential for shallow slope failures and erosion. Stormwater discharge locations should not be located at the top or on the face of any slope.

67. Final grades should be provided with positive gradient away from all foundation elements. Soil grades should slope away from foundations at least 5 percent for the first 10 feet. Impervious surfaces should slope away from foundations at least 2 percent for the first 10 feet. Concentrations of surface runoff should be handled by providing structures, such as paved or lined ditches, catch basins, etc.

68. Irrigation activities at the site should be done in a controlled and reasonable manner.

69. Following completion of the project we recommend that storm drainage provisions and performance of permanent erosion control measures be closely observed through the first season of significant rainfall, to determine if these systems are performing adequately and, if necessary, resolve any unforeseen issues.

70. The building and surface drainage facilities must not be altered nor any filling or excavation work performed in the area without first consulting Pacific Crest Engineering Inc. Surface drainage improvements developed by the project civil engineer must be maintained by the property owner at all times, as improper drainage provisions can produce undesirable affects.

EROSION CONTROL

71. The surface soils are classified as having a moderate potential for erosion. Therefore, the finished ground surface should be planted with ground cover and continually maintained to minimize surface erosion. For specific and detailed recommendations regarding erosion control on and surrounding the project site, the project civil engineer or an erosion control specialist should be consulted.

PLAN REVIEW

72. We respectfully request an opportunity to review the project plans and specifications during preparation and before bidding to ensure that the recommendations of this report have been included and to provide additional recommendations, if needed. These plan review services are also typically required by the reviewing agency. Misinterpretation of our recommendations or omission of our requirements from the project plans and specifications may result in changes to the project design during the construction phase, with the potential for additional costs and delays in order to bring the project into conformance with the requirements outlined within this report. Services performed for review of the project plans and specifications are considered "post-report" services and billed on a "time and materials" fee basis in accordance with our latest Standard Fee Schedule.

VI. LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. This Geotechnical Investigation was prepared specifically for Monterey Peninsula Water Management District and for the specific project and location described in the body of this report. This report and the recommendations included herein should be utilized for this specific project and location exclusively. This Geotechnical Investigation should not be applied to nor utilized on any other project or project site. Please refer to the ASFE "Important Information about Your Geotechnical Engineering Report" attached with this report.

2. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be provided.

3. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractors and Subcontractors carry out such recommendations in the field.

4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural process or the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside of our control. This report should therefore be reviewed in light of future planned construction and then current applicable codes. This report should not be considered valid after a period of two (2) years without our review.

5. This report was prepared upon your request for our services in accordance with currently accepted standards of professional geotechnical engineering practice. No warranty as to the contents of this report is intended, and none shall be inferred from the statements or opinions expressed.

6. The scope of our services mutually agreed upon for this project did not include any environmental assessment or study for the presence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site.

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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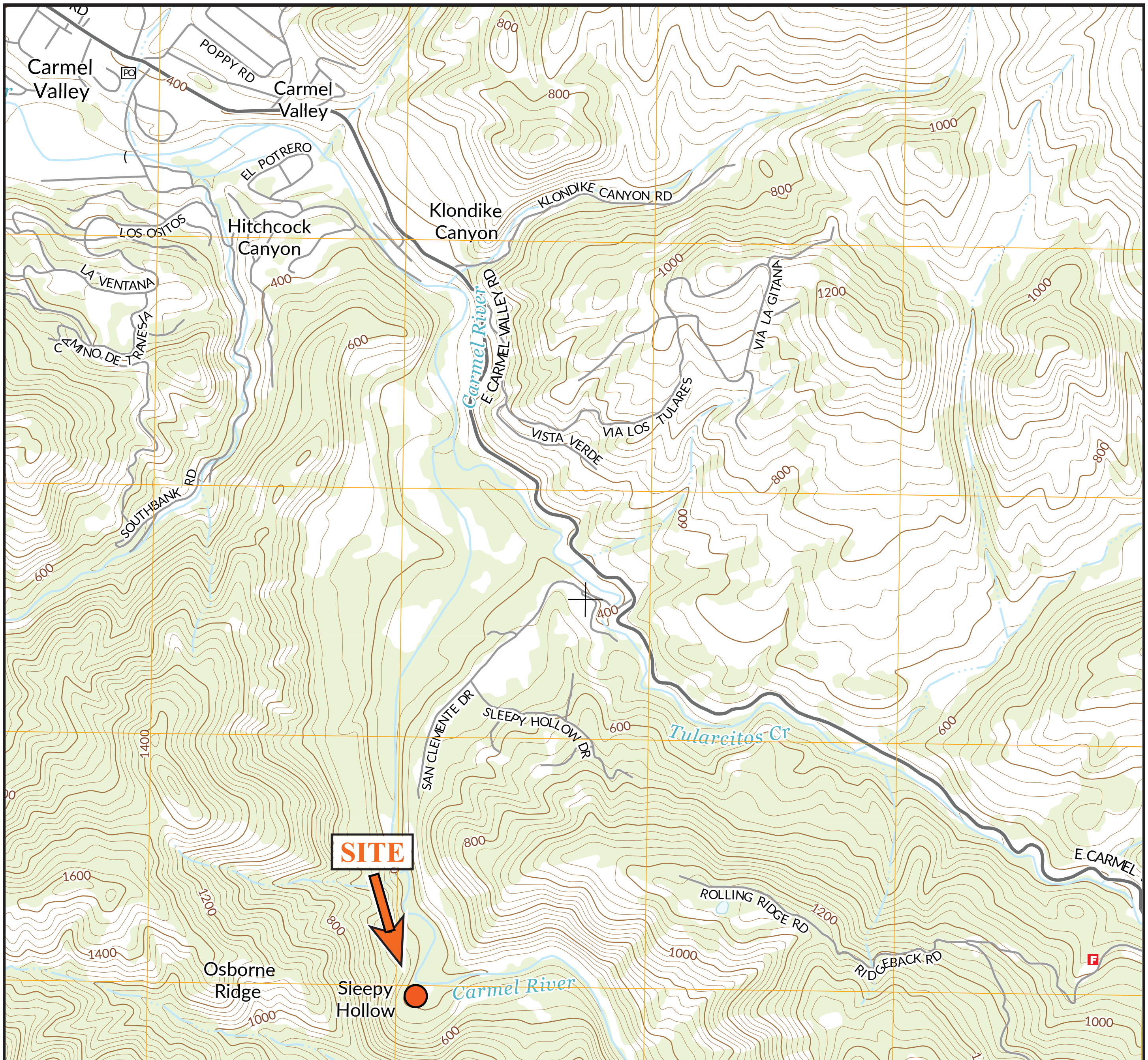
APPENDIX A

Regional Site Map
Site Map Showing Test Borings
Key to Soil Classification
Log of Test Borings
Liquefaction Analysis



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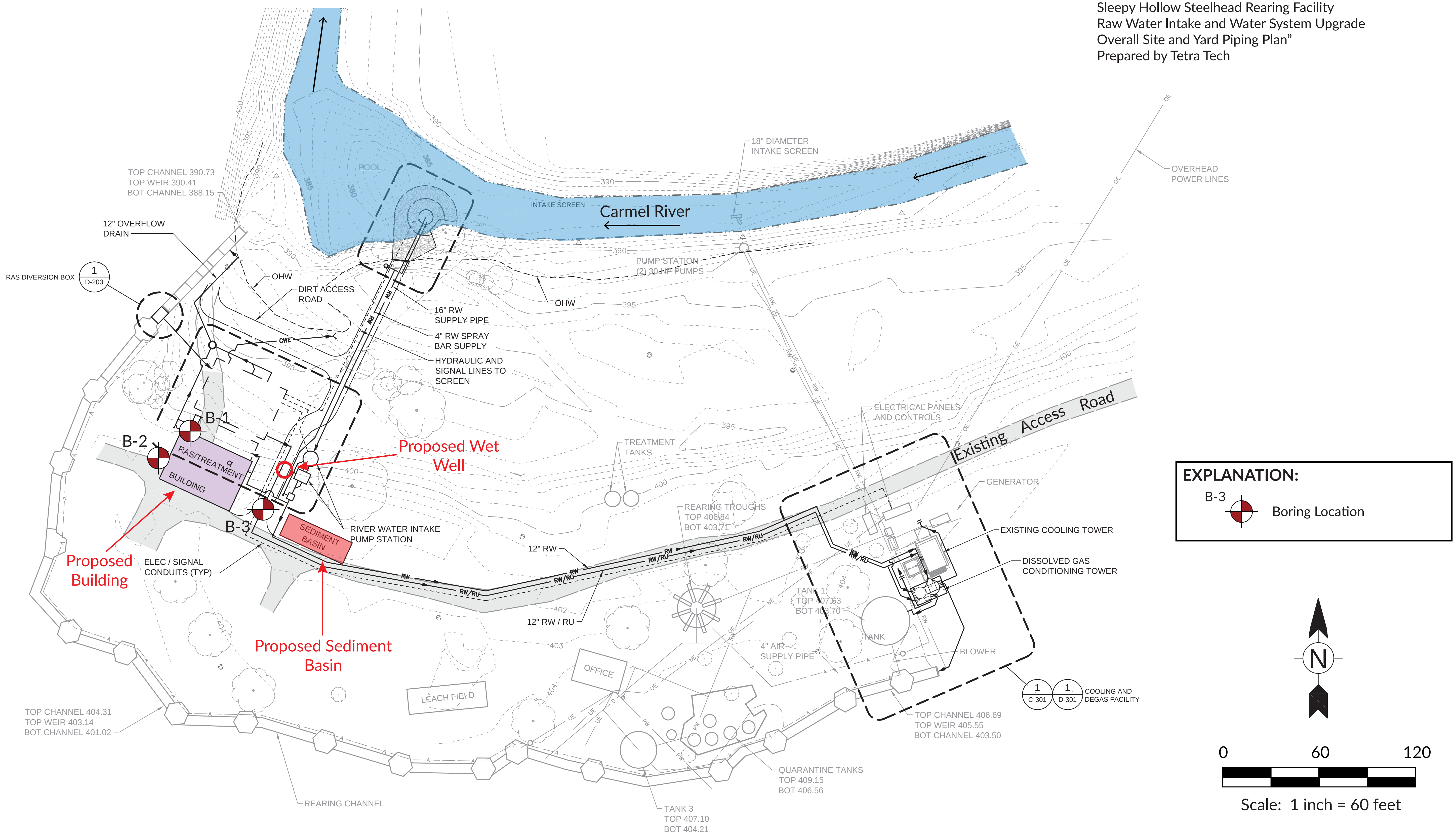
Base Map: United States Geological Survey
Carmel Valley Quadrangle, California
Monterey County, 7.5 Minute Series, 2015

Pacific Crest ENGINEERING INC	**Regional Site Map** Sleepy Hollow Rearing Facility Carmel Valley, California	Figure No. 1 Project No. 1809 Date: 4/16/18

Base Map: "Monterey Peninsula Water Management District
Sleepy Hollow Steelhead Rearing Facility
Raw Water Intake and Water System Upgrade
Overall Site and Yard Piping Plan"
Prepared by Tetra Tech



Pacific Crest ENGINEERING INC

Site Map Showing Test Boring Locations
Sleepy Hollow Rearing Facility
Carmel Valley, California

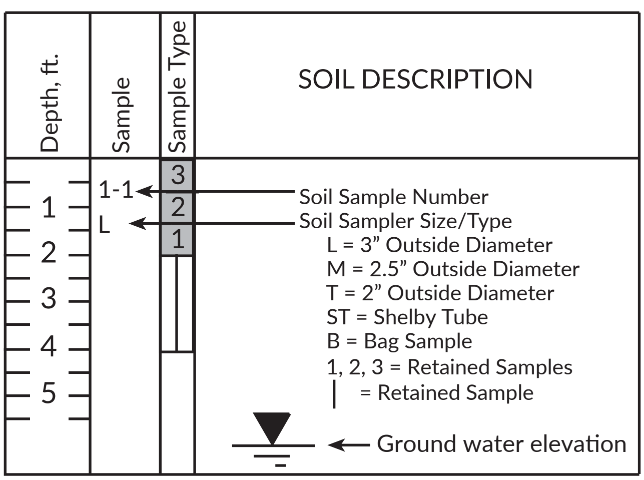
Figure No. 2
Project No. 1809
Date: 4/16/18

KEY TO SOIL CLASSIFICATION - FINE GRAINED SOILS (FGS)
UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2487 (Modified)

MAJOR DIVISIONS		SYMBOL	FINES	COARSENESS	SAND/GRAVEL	GROUP NAME
SILT AND CLAY	*LL < 35% Low Plasticity	**CL** Lean Clay PI > 7 Plots Above A Line -OR- **ML** Silt PI > 4 Plots Below A Line	<30% plus No. 200	<15% plus No. 200		Lean Clay / Silt
				15-30% plus No. 200	% sand ≥ % gravel	Lean Clay with Sand / Silt with Sand
					% sand < % gravel	Lean Clay with Gravel / Silt with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Lean Clay / Sandy Silt
					≥ 15% gravel	Sandy Lean Clay with Gravel / Sandy Silt with Gravel
				% sand < % gravel	< 15% sand	Gravelly Lean Clay / Gravelly Silt
					≥ 15% sand	Gravelly Lean Clay with Sand / Gravelly Silt with Sand
		CL - ML 4 < PI < 7	<30% plus No. 200	<15% plus No. 200		Silty Clay
				15-30% plus No. 200	% sand ≥ % gravel	Silty Clay with Sand
					% sand < % gravel	Silty Clay with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Silty Clay
					≥15% gravel	Sandy Silty Clay with Gravel
				% sand < % gravel	< 15% sand	Gravelly Silty Clay
					≥ 15% sand	Gravelly Silty Clay with Sand
	35% ≤ *LL < 50% Intermediate Plasticity	**CI**	<30% plus No. 200	<15% plus No. 200		Clay
				15-30% plus No. 200	% sand ≥ % gravel	Clay with Sand
					% sand < % gravel	Clay with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Clay
					≥ 15% gravel	Sandy Clay with Gravel
				% sand < % gravel	< 15% sand	Gravelly Clay
					≥ 15% sand	Gravelly Clay with Sand
	*LL > 50% High Plasticity	**CH** Fat Clay Plots Above A Line -OR- **MH** Elastic Silt Plots Below A Line	<30% plus No. 200	<15% plus No. 200		Fat Clay or Elastic Silt
				15-30% plus No. 200	% sand ≥ % gravel	Fat Clay with Sand Elastic Silt with Sand
					% sand < % gravel	Fat Clay with Gravel / Elastic Silt with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Fat Clay / Sandy Elastic Silt
					≥ 15% gravel	Sandy Fat Clay with Gravel / Sandy Elastic Silt with Gravel
				% sand < % gravel	< 15% sand	Gravelly Fat Clay / Gravelly Elastic Silt
					≥ 15% sand	Gravelly Fat Clay with Sand / Gravelly Elastic Silt with Sand

* LL = Liquid Limit
* PI = Plasticity Index

BORING LOG EXPLANATION



MOISTURE

DESCRIPTION	CRITERIA
DRY	Absence of moisture, dusty, dry to the touch
MOIST	Damp, but no visible water
WET	Visible free water, usually soil is below the water table

CONSISTENCY

DESCRIPTION	UNCONFINED SHEAR STRENGTH (KSF)	STANDARD PENETRATION (BLOWS/FOOT)
VERY SOFT	< 0.25	< 2
SOFT	0.25 - 0.5	2 - 4
FIRM	0.5 - 1.0	5 - 8
STIFF	1.0 - 2.0	9 - 15
VERY STIFF	2.0 - 4.0	16 - 30
HARD	> 4.0	> 30

Pacific Crest ENGINEERING INC	**Boring Log Explanation - FGS** Sleepy Hollow Rearing Facility Carmel Valley, California	Figure No. 3 Project No. 1809 Date: 4/16/18

KEY TO SOIL CLASSIFICATION - COARSE GRAINED SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2487 (Modified)

MAJOR DIVISIONS		FINES	GRADE/TYPE OF FINES	SYMBOL	GROUP NAME *
GRAVEL	More than 50% of coarse fraction is larger than No. 4 sieve size	<5%	Cu ≥ 4 and 1 ≤ Cc ≤ 3	**GW**	Well-Graded Gravel / Well-Graded Gravel with Sand
			Cu < 4 and/or 1 > Cc > 3	**GP**	Poorly Graded Gravel/Poorly Graded Gravel with Sand
		5-12%	ML or MH	**GW - GM**	Well-Graded Gravel with Silt / Well- Graded Gravel with Silt and Sand
				GP - GM	Poorly Graded Gravel with Silt / Poorly Graded Gravel with Silt and Sand
			CL, CI or CH	**GW - GC**	Well-Graded Gravel with Clay / Well-Graded Gravel with Clay and Sand
				GP - GC	Poorly Graded Gravel with Clay / Poorly Graded Gravel with Clay and Sand
		>12%	ML or MH	**GM**	Silty Gravel / Silty Gravel with Sand
			CL, CI or CH	**GC**	Clayey Gravel/ Clayey Gravel with Sand
			CL - ML	**GC - GM**	Silty, Clayey Gravel/ Silty, Clayey Gravel with Sand
SAND	50% or more of coarse fraction is smaller than No. 4 sieve size	<5%	Cu ≥ 6 and 1 ≤ Cc ≤ 3	**SW**	Well-Graded Sand / Well-Graded Sand with Gravel
			Cu < 6 and/or 1 > Cc > 3	**SP**	Poorly Graded Sand / Poorly Graded Sand with Gravel
		5-12%	ML or MH	**SW - SM**	Well-Graded Sand with Silt / Well- Graded Sand with Silt and Gravel
				SP - SM	Poorly Graded Sand with Silt / Poorly Graded Sand with Silt and Gravel
			CL, CI or CH	**SW - SC**	Well-Graded Sand with Clay / Well-Graded Sand with Clay and Gravel
				SP - SC	Poorly Graded Sand with Clay / Poorly Graded Sand with Clay and Gravel
		>12%	ML or MH	**SM**	Silty Sand / Silty Sand with Gravel
			CL, CI or CH	**SC**	Clayey Sand / Clayey Sand with Gravel
			CL - ML	**SC - SM**	Silty, Clayey Sand / Silty, Clayey Sand with Gravel

* The term “with sand” refers to materials containing 15% or greater sand particles within a gravel soil, while the term “with gravel” refers to materials containing 15% or greater gravel particles within a sand soil.

US STANDARD SIEVE SIZE: 3 inch | ¾ inch | No. 4 | No. 10 | No. 40 | No. 200 | 0.002 μm

	COARSE	FINE	COARSE	MEDIUM	FINE		
COBBLES AND BOULDERS	GRAVEL		SAND			SILT	CLAY

RELATIVE DENSITY

DESCRIPTION	STANDARD PENETRATION (BLOWS/FOOT)
VERY LOOSE	0 - 4
LOOSE	5 - 10
MEDIUM DENSE	11 - 30
DENSE	31 - 50
VERY DENSE	> 50

MOISTURE

DESCRIPTION	CRITERIA
DRY	Absence of moisture, dusty, dry to the touch
MOIST	Damp, but no visible water
WET	Visible free water, usually soil is below the water table

Pacific Crest ENGINEERING INC

Boring Log Explanation - CGS
Sleepy Hollow Rearing Facility
Carmel Valley, California

Figure No. 4
Project No. 1809
Date: 4/16/18

LOGGED BY SBG | DATE DRILLED 3/5/18 | BORING DIAMETER 8" HS | BORING NO. 1

DRILL RIG Mobile B56 | HAMMER TYPE Wire Winch | BORING ELEV. 400.5'

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	% Passing #200 Sieve	Dry Density (pcf)	Moisture Content (%)	Additional Lab Results
1–2	1-1	L	**SILTY SAND:** Dark brown (10YR 3/3), fine grained, fine mica flakes, damp, medium dense	SM	11 22 17	21		22	98	6	
3–4	1-2	L	**SILTY SAND:** Grayish brown (10YR 5/2), fine grained, generally less fines, dry to damp, medium dense	SM	7 7 8	12		32	89	6	
5–6	1-3	L	Loose		5 5 7	6			89	6	
8–9	1-4	T	Medium grained sand in tip		5 4 3	7					
10–11	1-5	T	Fine grained sand, moist		4 3 4	7		37		13	1% Gravel 62% Sand 37% Fines
13½			Hard drilling at 13½ feet								
15–16	1-6	T	**BEDROCK: GRANODIORITE:** Light gray (10YR 6/1), dark gray (10YR 8/1) and black, angular 1 inch gravels, angular coarse grained sand		28 22 50	72		4		9	56% Gravel 40% Sand 4% Fines
17			Crunchy, hard drilling to 20 feet, 2 inch rounded gravels in cuttings								
20	1-7	T	Angular coarse grained sand, 1 inch sub-rounded quartz, bending sampler tip		32 50/4"	50/4"					
20½			Boring terminated at 20½ feet due to drilling refusal. Groundwater encountered at 9 feet.								

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Log of Test Borings
Sleepy Hollow Rearing Facility
Carmel Valley, California

Figure No. 5
Project No. 1809
Date: 4/16/18

LOGGED BY SBG DATE DRILLED 3/5/18 BORING DIAMETER 8" HS BORING NO. 2

DRILL RIG Mobile B56 HAMMER TYPE Wire Winch BORING ELEV. 403.5'

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	% Passing #200 Sieve	Dry Density (pcf)	Moisture Content (%)	Additional Lab Results
1	2-1 T		**SILTY SAND:** Dark yellowish brown (10YR 3/4), fine grained sand, abundance of mica flakes, damp, loose	SM	3 1 1	2		23		15	
2											
3	2-2 T		**SAND WITH SILT:** Brown (10YR 5/3), fine grained sand, mica flakes, salt and pepper coloring pattern, damp, loose	SP-SM	1 3 5	8		6		7	
4											
5	2-3 T		Medium dense		3 4 10	14					
6											
7			**SILTY SAND:** Brown (10YR 5/3), damp, loose	SM							
8	2-4 T		Firmer drilling at 8½ feet		2 3 3	6		20		8	80% Sand 20% Fines
9											
10	2-5 T		**SAND:** Gray (10YR 6/1), medium to coarse grained, quartz rich, wet, loose	SP	3 2			2		2	98% Sand 2% Fines
11			**SANDY SILT:** Very dark gray and dark brown (10YR 3/2), fine grained, moist, soft	ML	3	5		52		19	48% Sand 52% Fines
12			**SAND:** Gray (10YR 6/1), medium to coarse grained, quartz rich, wet, loose	SP							
13											
14			Rocky drilling at 14 feet								
15	2-6 T		**BEDROCK: GRANODIORITE:** Gray (10YR 6/1) and white (10YR 8/1), angular to sub-angular coarse grained sand, angular 1 inch gravels, quartz rich, moist		25 33 39	62					
16											
17			Boring terminated at 16½ feet. Groundwater encountered at 15 feet.								
18											
19											
20											
21											
22											
23											

Pacific Crest ENGINEERING INC

Log of Test Borings
Sleepy Hollow Rearing Facility
Carmel Valley, California

Figure No. 6
Project No. 1809
Date: 4/16/18

LOGGED BY SBG DATE DRILLED 3/5/18 BORING DIAMETER 8" HS BORING NO. 3

DRILL RIG Mobile B56 HAMMER TYPE Wire Winch BORING ELEV. 403.5'

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	% Passing #200 Sieve	Dry Density (pcf)	Moisture Content (%)	Additional Lab Results
0			**SAND:** Dark brown (10YR 3/3), fine grained, mica rich, moist, loose	SP							
3	3-1	T			1 2 3	5					
5	3-2	T	Salt and pepper coloring, slightly coarser grained, loose		2 3 3	6		3		4	
8	3-3	T	Grayish brown (10YR 5/2), medium to coarse grained, quartz rich, sub-rounded to sub-angular, damp, loose		4 4			4 25		2 6	
			SILTY SAND: Dark grayish brown (10YR 4/2), fine grained, moist, loose	SM	3	7					
10	3-4	T	**SANDY GRAVEL:** Grayish brown and light gray, 1 to 3 inch sub-rounded cobbles in cuttings, moist, dense No sample retrieved at 10 feet	GP	50/5"	50/5"					
12			Hard drilling from 10 to 15 feet								
15	3-5	T			40 26						
16			**BEDROCK: GRANODIORITE:** Difficult drilling to 19 feet, jumping on 2 to 4 inch cobbles, auger was moving sideways, no sample recovery		14	40		3		8	
19						50/0"					
20			Boring terminated at 19½ feet due to drilling refusal. Groundwater encountered at 15½ feet.								

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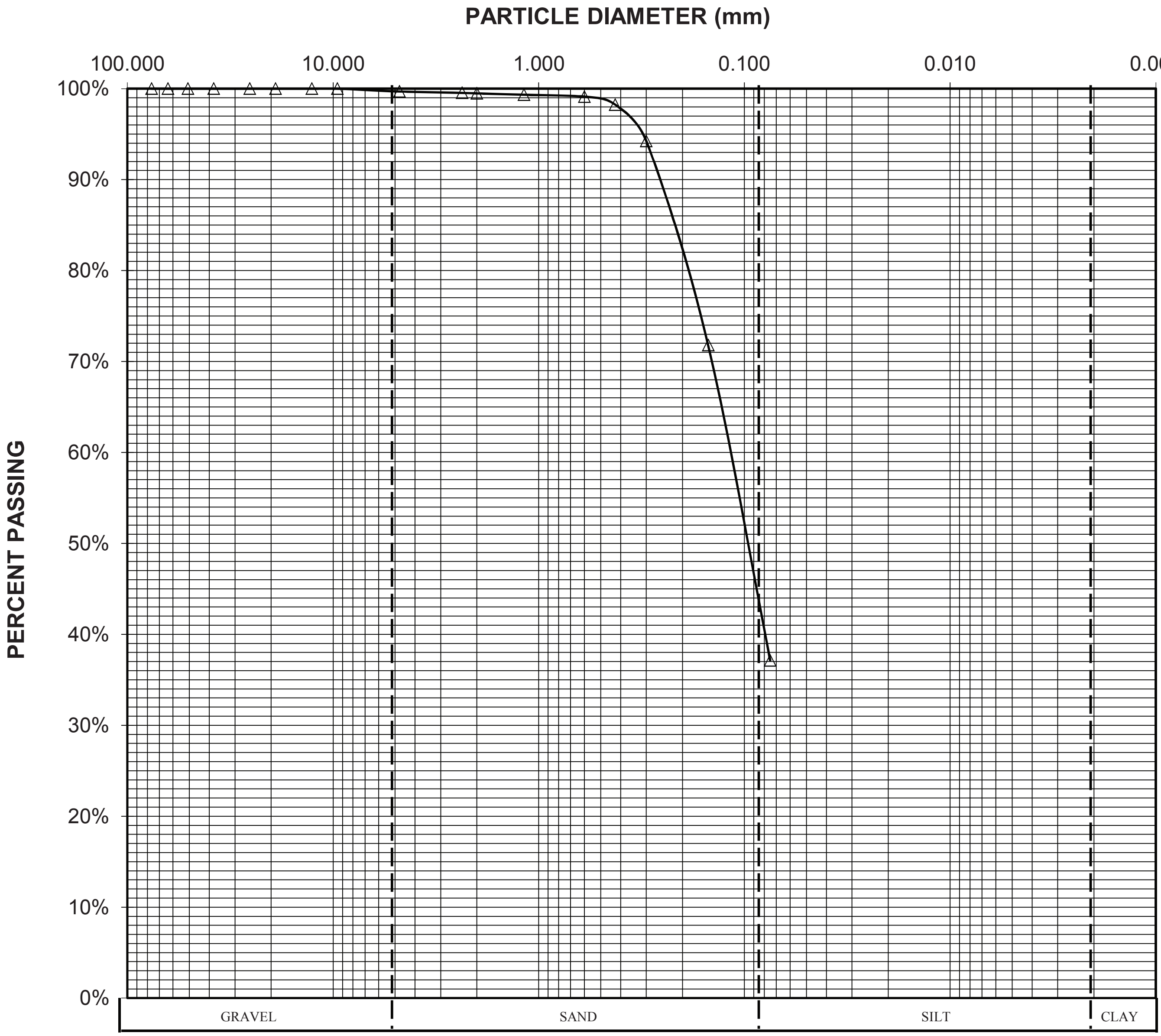
Log of Test Borings
Sleepy Hollow Rearing Facility
Carmel Valley, California

Figure No. 7
Project No. 1809
Date: 4/16/18

PARTICLE SIZE ANALYSIS - T11/C136

SAMPLE NO: 1-5	% PASSING No. 4	% PASSING No. 200
	99.7%	37.1%



GRAVEL	SAND	SILT + CLAY
0.3%	62.5%	37.1%

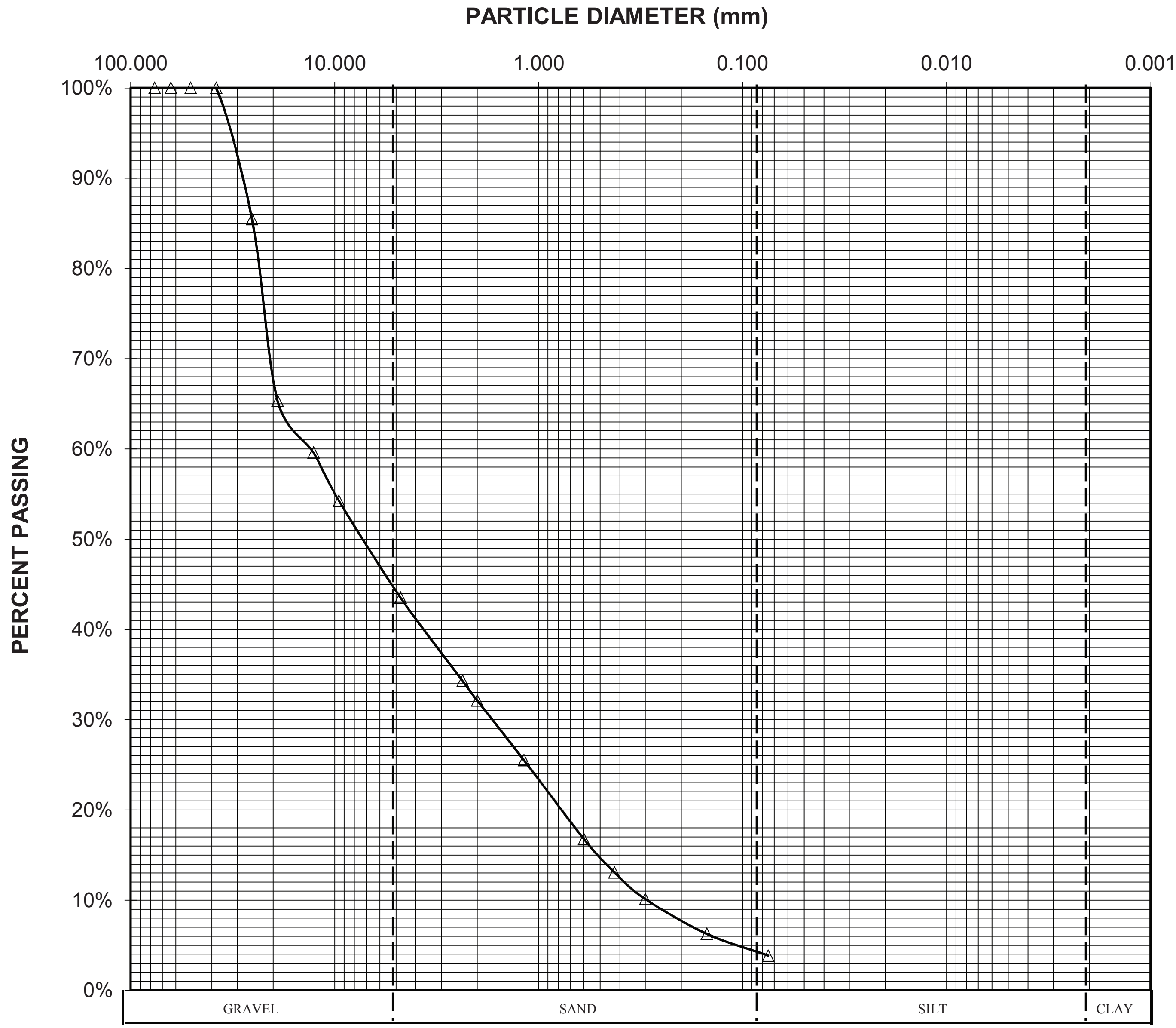
Pacific Crest ENGINEERING INC

Gradation Test Results
Sleepy Hollow Rearing Facility
Carmel Valley, California

Figure No. 8
Project No. 1809
Date: 4/16/18

PARTICLE SIZE ANALYSIS - T11/C136

SAMPLE NO: 1-6	% PASSING No. 4	% PASSING No. 200
	43.5%	3.8%



GRAVEL	SAND	SILT + CLAY
56.5%	39.7%	3.8%

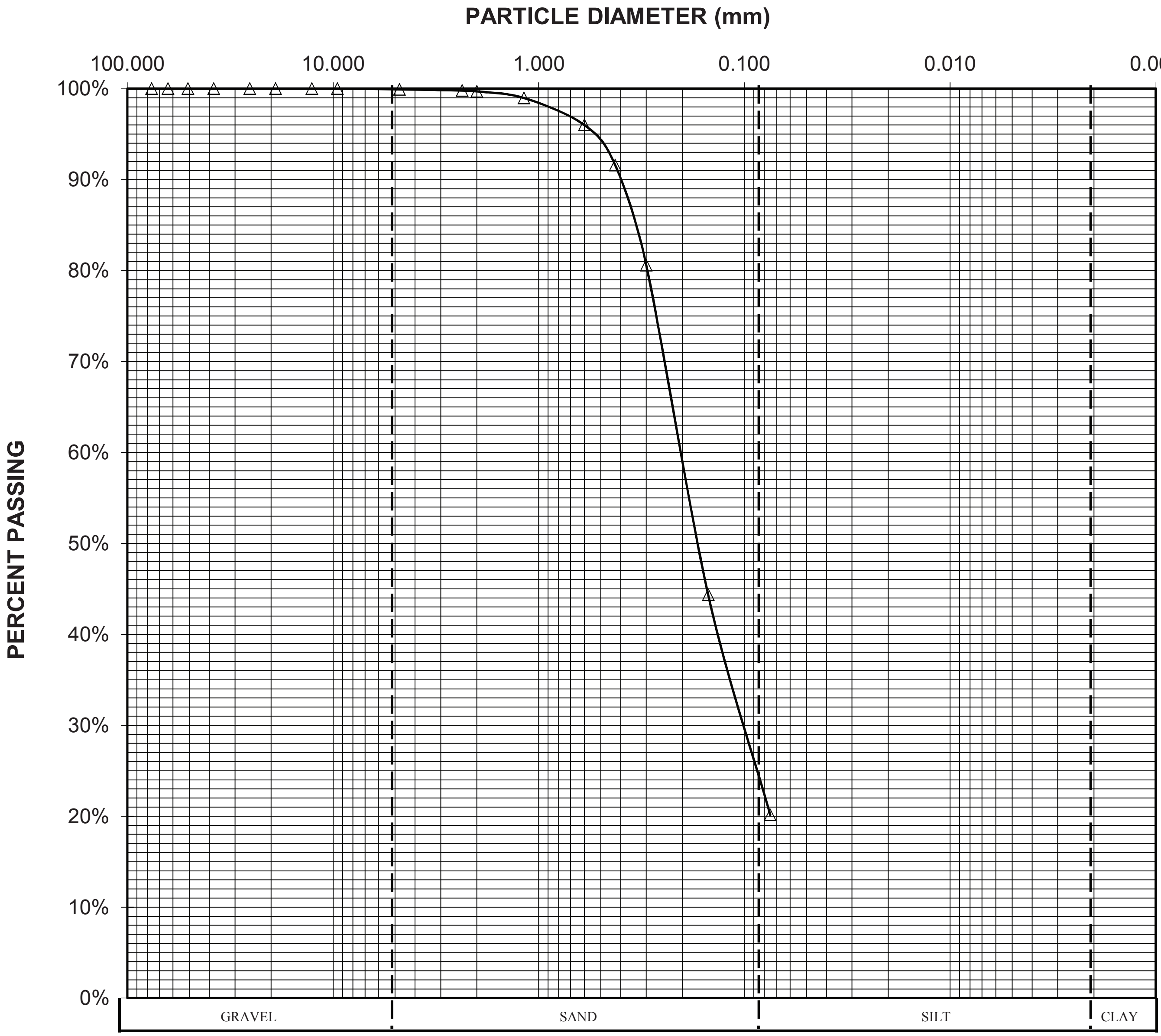
Pacific Crest ENGINEERING INC

Gradation Test Results
Sleepy Hollow Rearing Facility
Carmel Valley, California

Figure No. 9
Project No. 1809
Date: 4/16/18

PARTICLE SIZE ANALYSIS - T11/C136

SAMPLE NO: 2-4	% PASSING No. 4	% PASSING No. 200
	99.9%	20.1%



GRAVEL	SAND	SILT + CLAY
0.1%	79.8%	20.1%

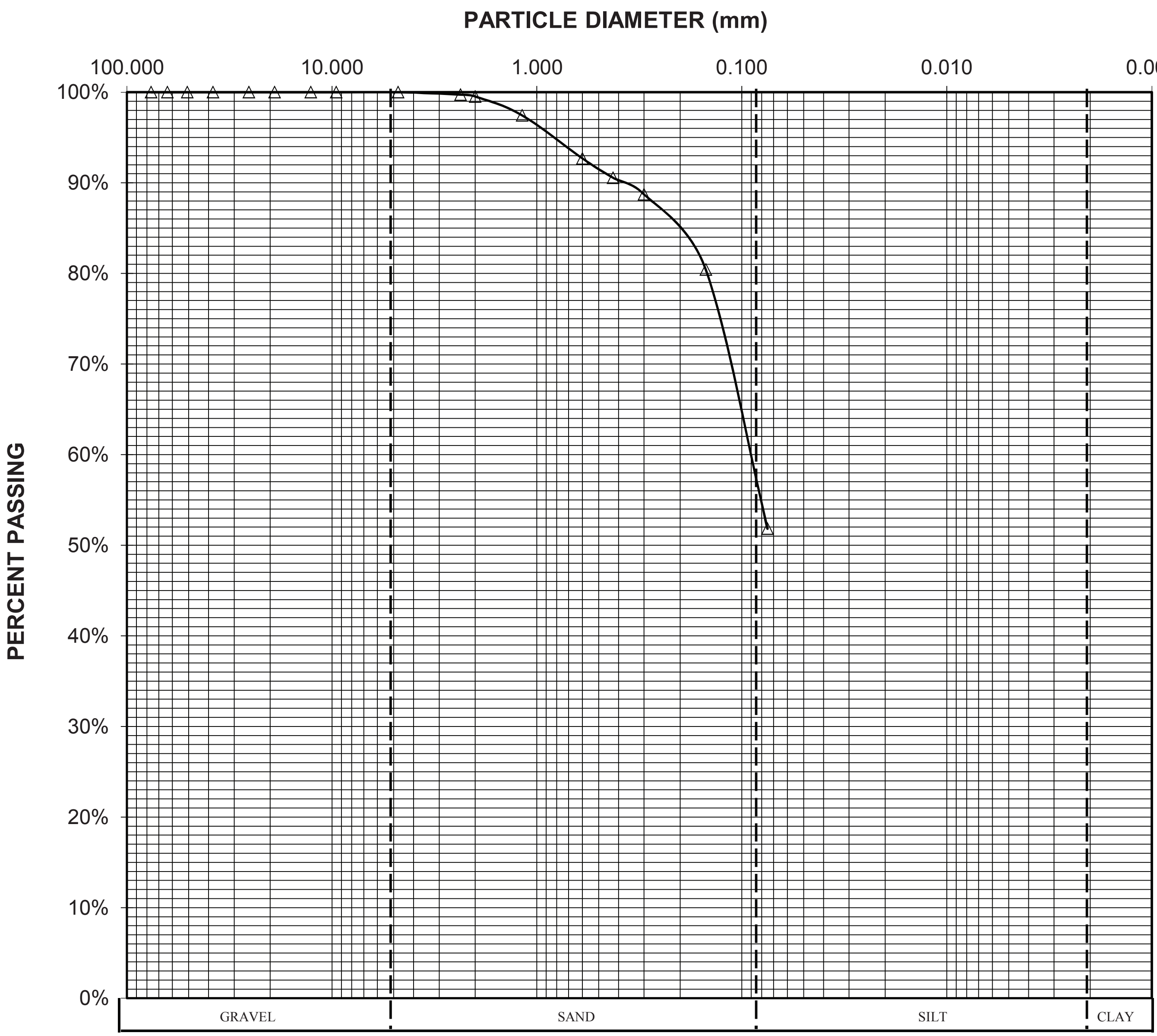
Pacific Crest ENGINEERING INC

Gradation Test Results
Sleepy Hollow Rearing Facility
Carmel Valley, California

Figure No. 10
Project No. 1809
Date: 4/16/18

PARTICLE SIZE ANALYSIS - T11/C136

SAMPLE NO: 2-5	% PASSING No. 4	% PASSING No. 200
	100.0%	51.8%



GRAVEL	SAND	SILT + CLAY
0.0%	48.2%	51.8%

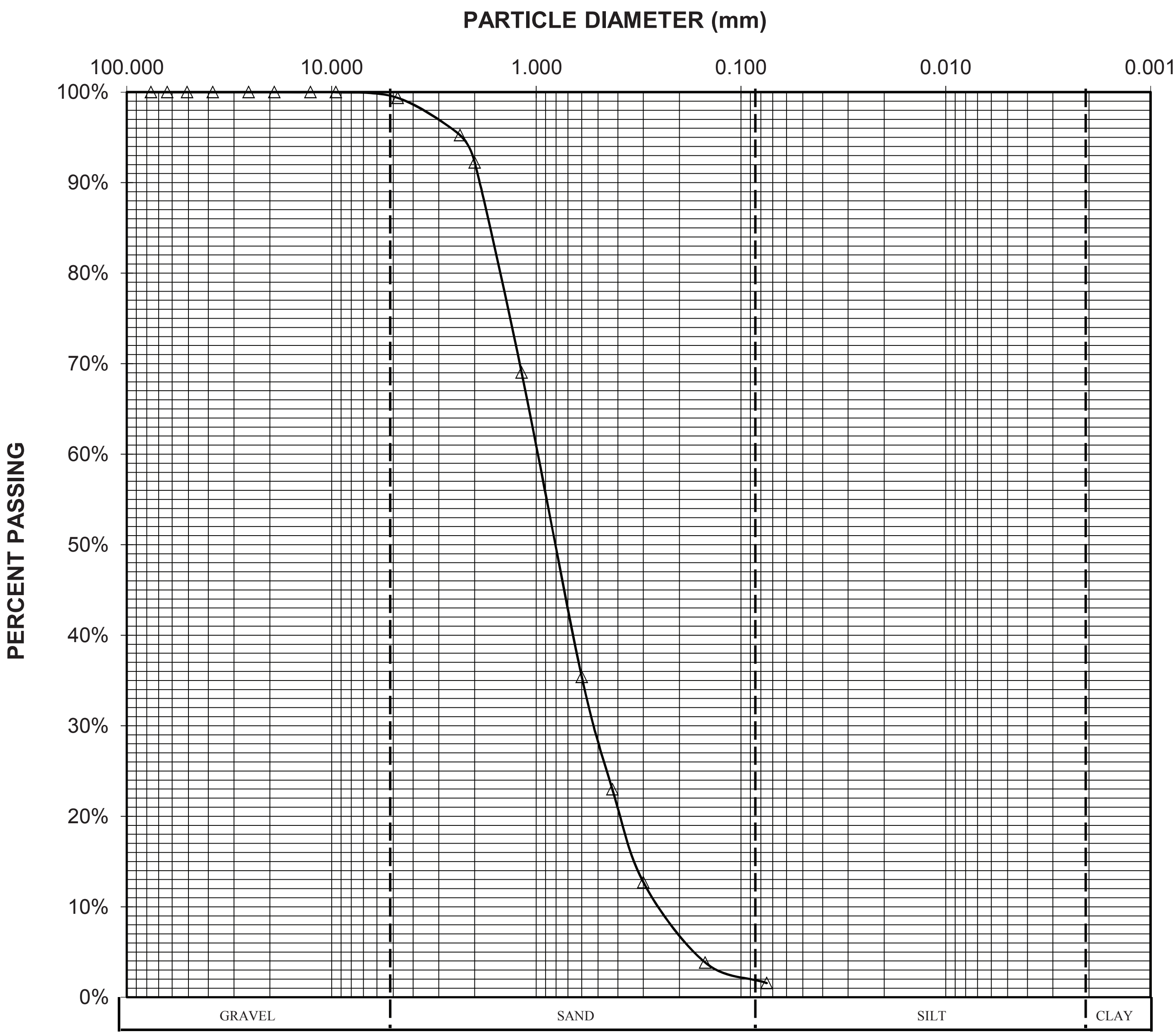
Pacific Crest ENGINEERING INC

Gradation Test Results
Sleepy Hollow Rearing Facility
Carmel Valley, California

Figure No. 11
Project No. 1809
Date: 4/16/18

PARTICLE SIZE ANALYSIS - T11/C136

SAMPLE NO: B2 @ 10.5'	% PASSING No. 4	% PASSING No. 200
	99.4%	1.6%



GRAVEL	SAND	SILT + CLAY
0.6%	97.8%	1.6%

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Gradation Test Results
Sleepy Hollow Rearing Facility
Carmel Valley, California

Figure No. 12
Project No. 1809
Date: 4/16/18

Pacific Crest ENGINEERING INC

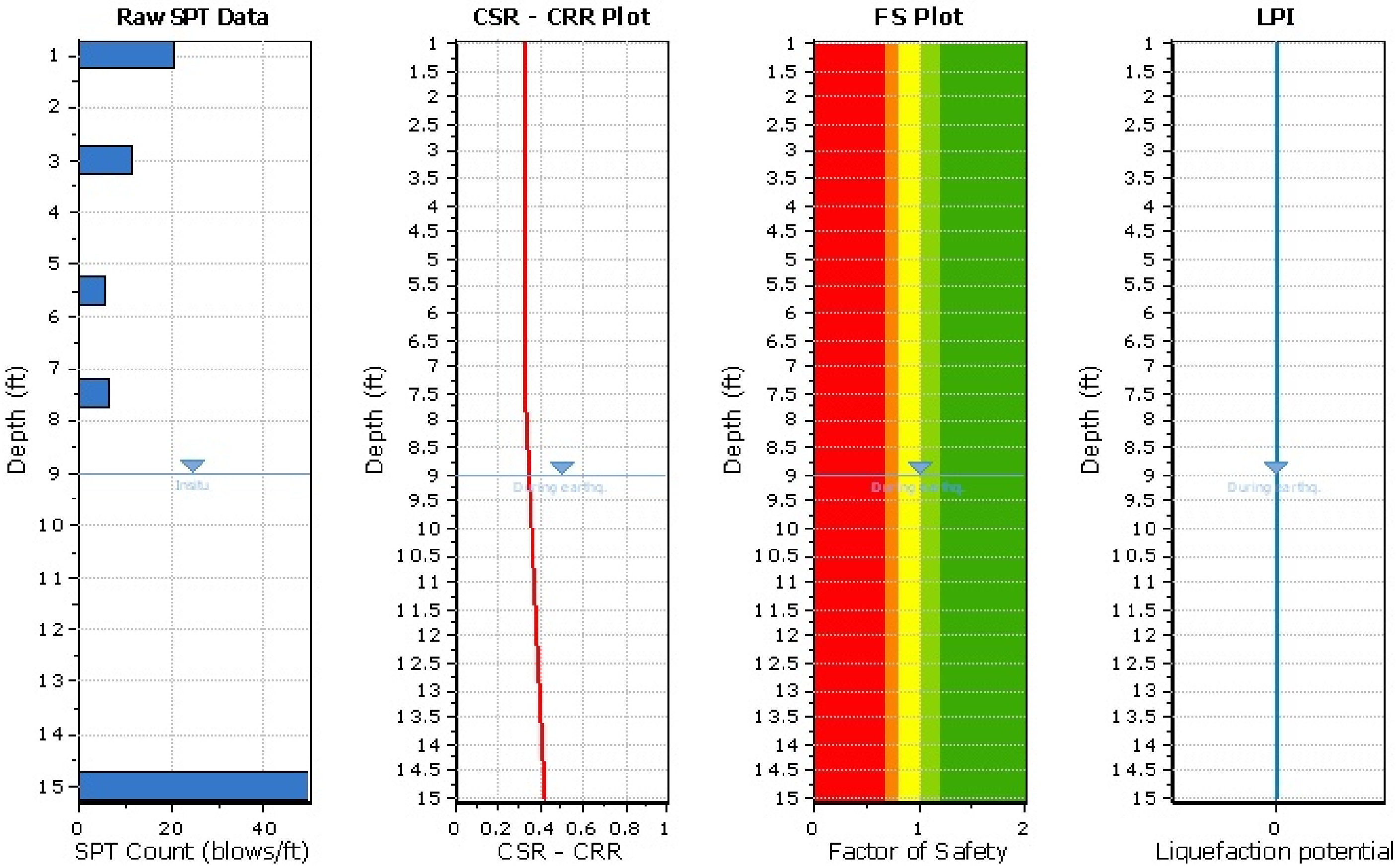
SPT BASED LIQUEFACTION ANALYSIS REPORT

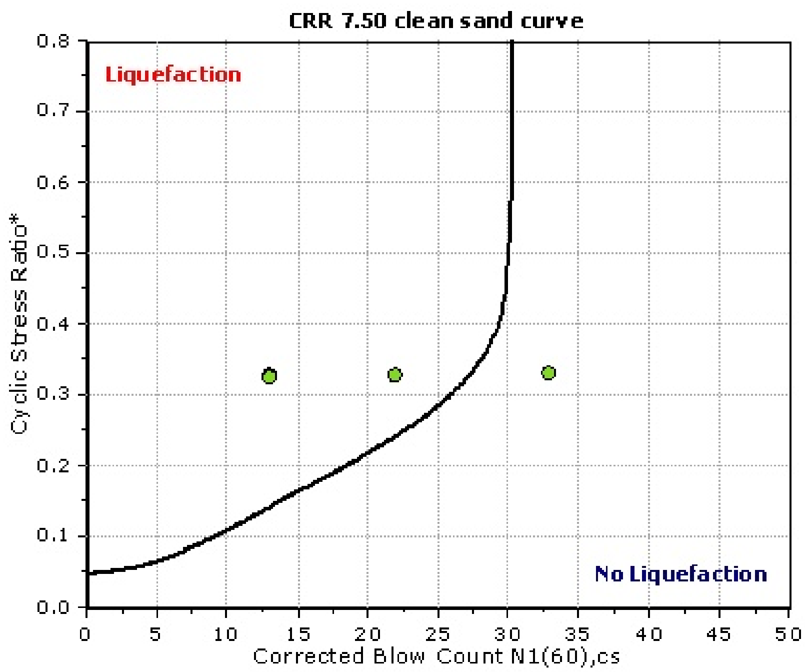
Project title : Sleepy Hollow Rearing Facility **SPT Name: B-1**

Location :

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	9.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	9.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.48 ft
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.51 g
Rod length:	2.00 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		





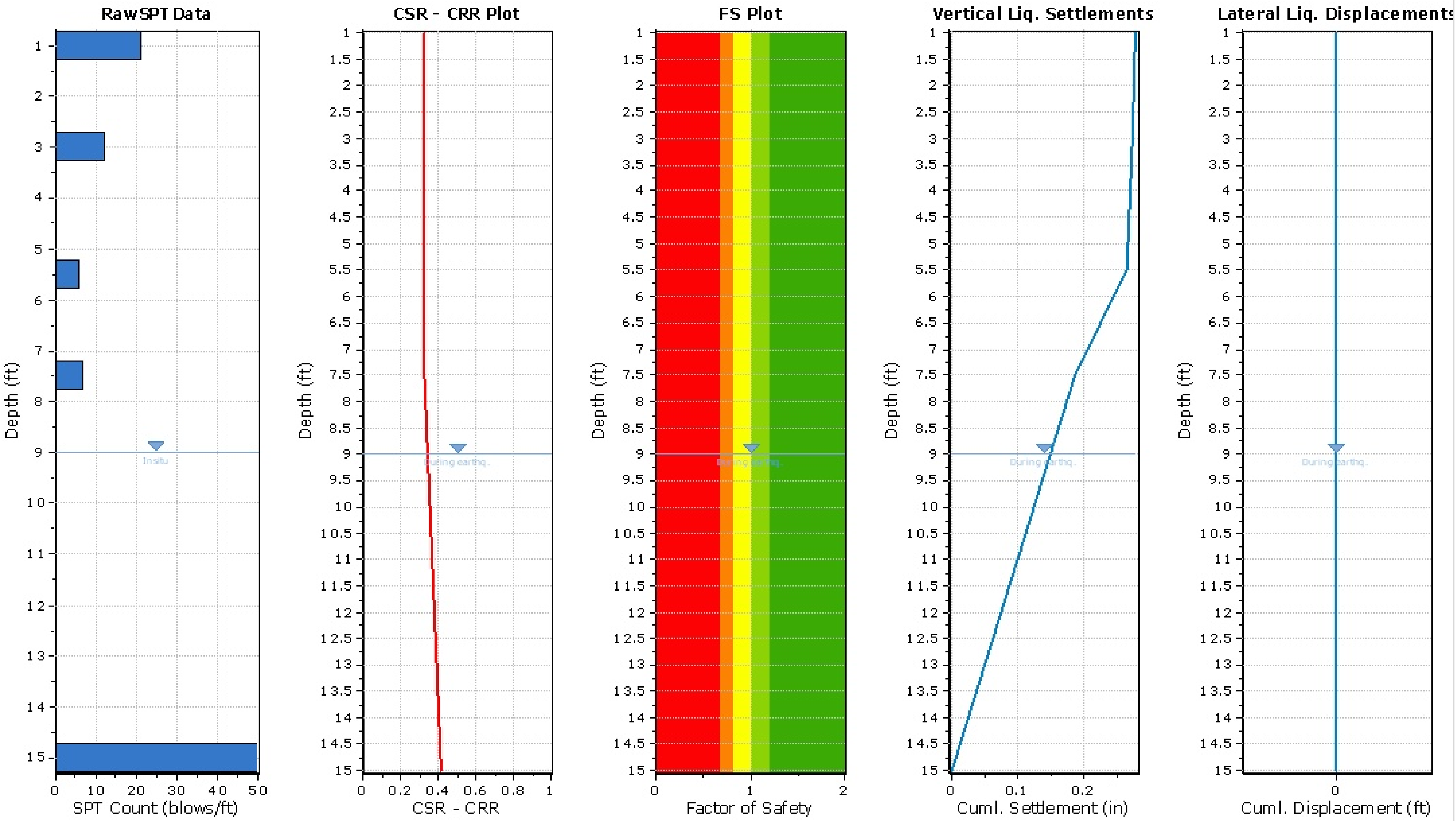
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



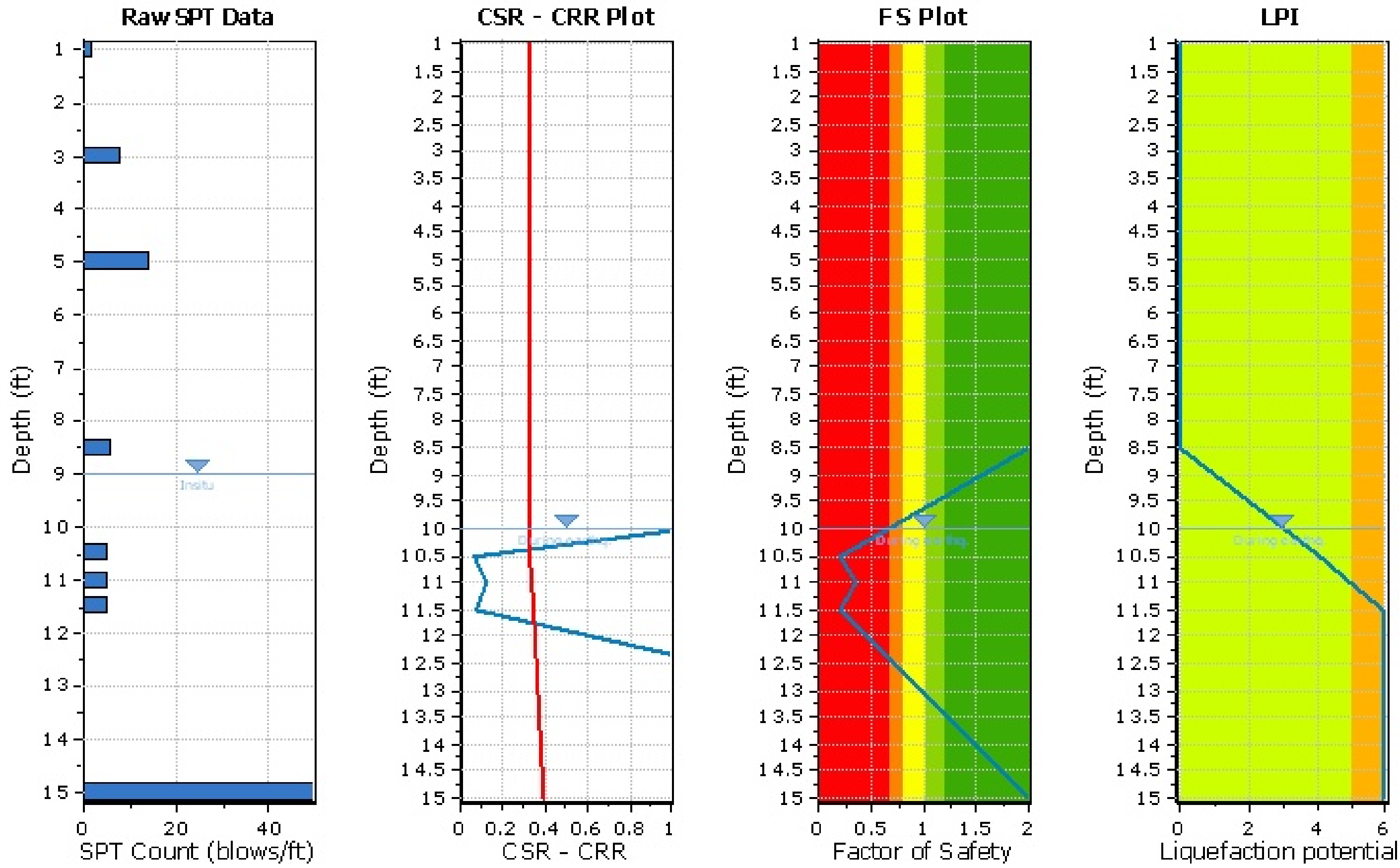
SPT BASED LIQUEFACTION ANALYSIS REPORT

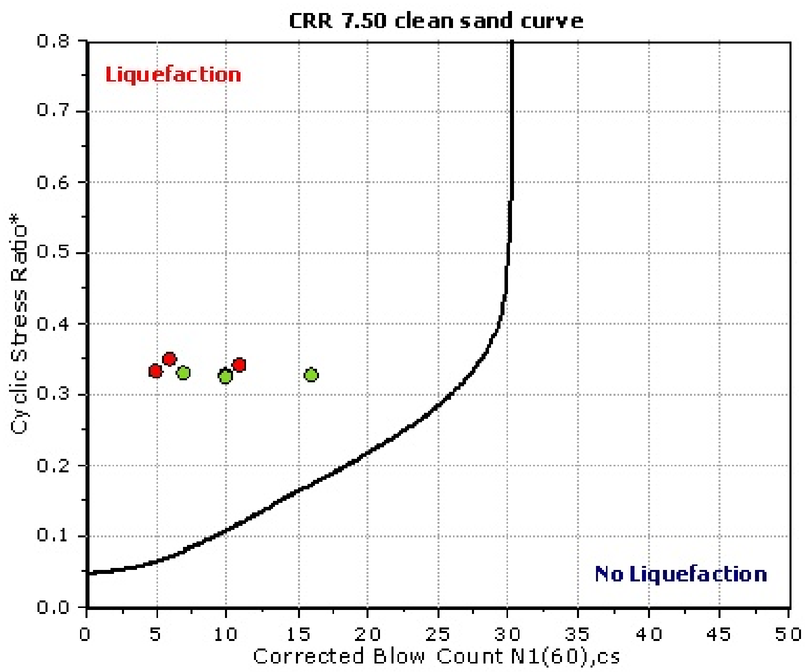
Project title : Sleepy Hollow Rearing Facility **SPT Name: B-2**

Location :

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	9.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	10.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.48 ft
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.51 g
Rod length:	2.00 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		





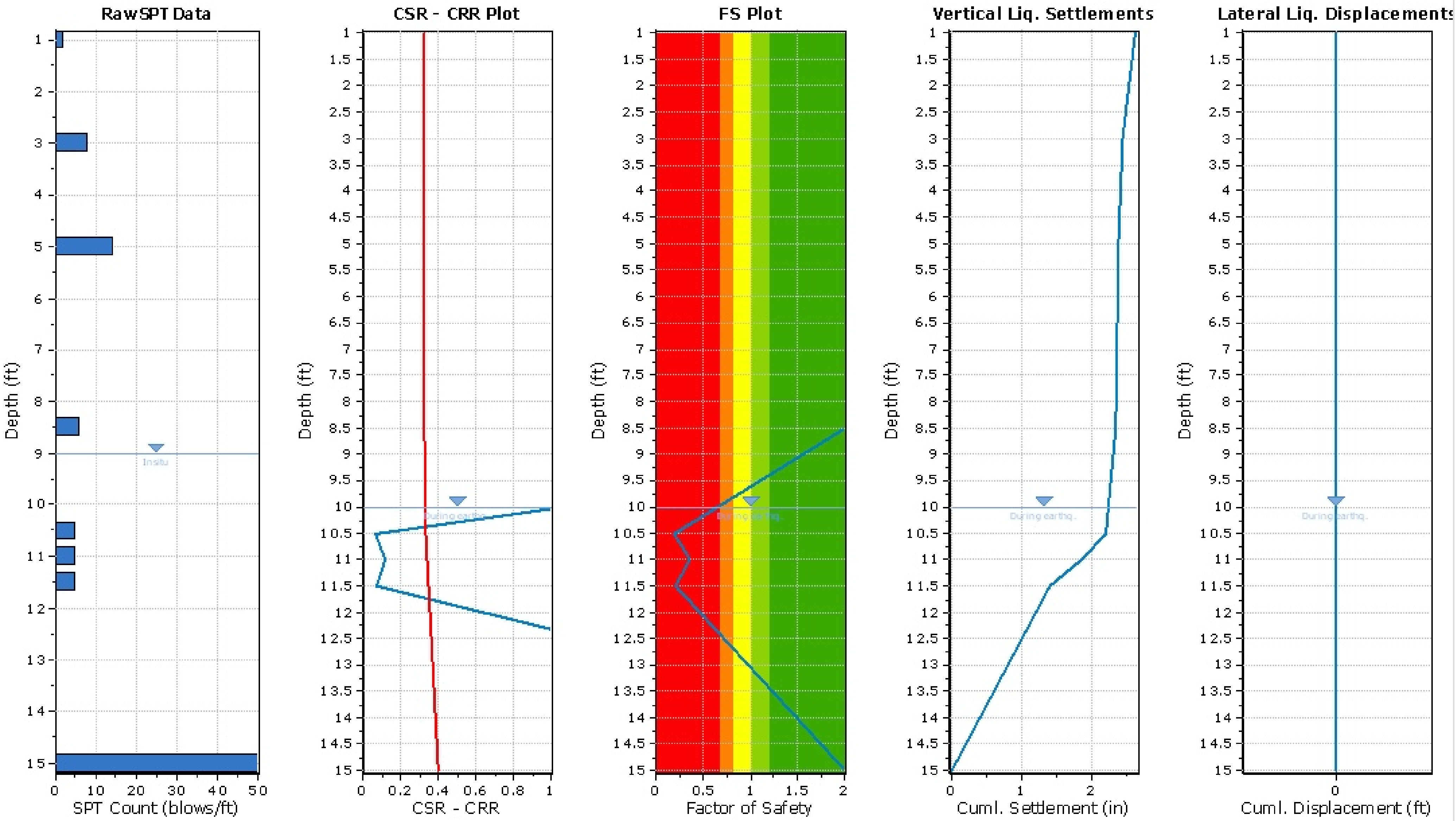
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



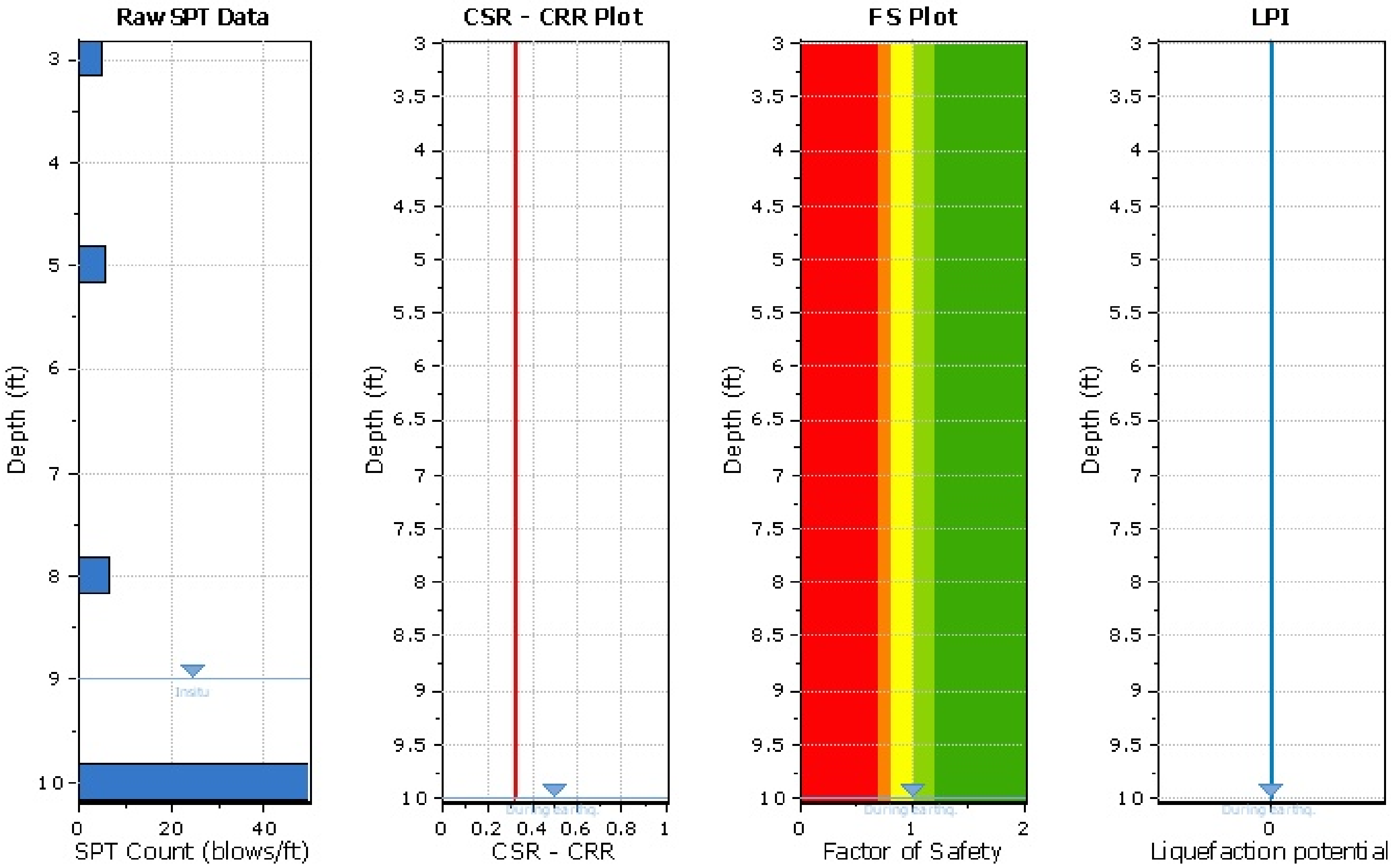
SPT BASED LIQUEFACTION ANALYSIS REPORT

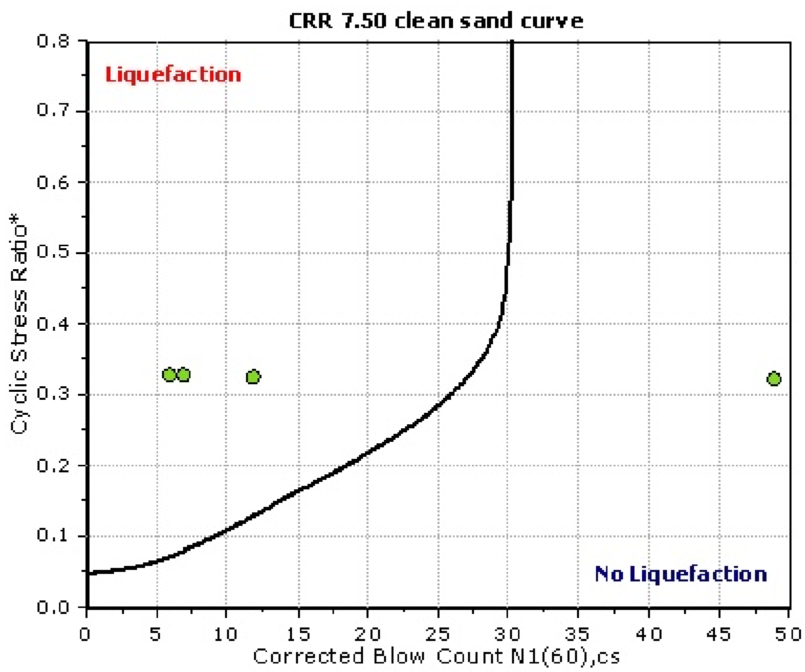
Project title : Sleepy Hollow Rearing Facility **SPT Name: B-3**

Location :

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	9.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	10.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.48 ft
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.51 g
Rod length:	2.00 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		





F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::