# Appendix F

Preliminary Geotechnical Investigation prepared by Pacific Crest Engineers

# PRELIMINARY GEOTECHNICAL INVESTIGATION

RIO RANCH RETAIL
3705 RIO ROAD
CARMEL, CALIFORNIA

FOR
RINCON CONSULTANTS
MONTEREY, CALIFORNIA

Pacific Crest
ENGINEERING INC

CONSULTING GEOTECHNICAL ENGINEERS

1713-M251-C562
SEPTEMBER 2017
www.4pacific-crest.com

Pacific Crest
ENGINEERING INC

GEOTECHNICAL | ENVIRONMENTAL | CHEMICAL | MATERIAL TESTING | SPECIAL INSPECTIONS

October 12, 2017 Project No. 1713-M251-C562

Ms. Christy Sabdo
Senior Environmental Project Manager
Rincon Consultants
437 Figueroa Street, Suite 203
Monterey, California 93940

Subject: **Geotechnical Investigation**
Rio Ranch Retail
3705 Rio Road
Carmel, California 93923

Dear Ms. Sabdo,

In accordance with your authorization, we have performed a preliminary geotechnical investigation for the proposed Rio Ranch Retail development located at 3705 Rio Road, in Carmel, 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.

Prepared by:

REGISTERED PROFESSIONAL ENGINEER
SOMA B. GORESKY
GE 2252
EXP. 6-30-2019
GEOTECHNICAL
STATE OF CALIFORNIA

Soma B. Goresky, G.E.
Associate Geotechnical Engineer
G.E. 2252

Reviewed by:

REGISTERED PROFESSIONAL ENGINEER
ELIZABETH MAE BAHNS MITCHELL
No. GE2718
Exp. 12/31/18
GEOTECHNICAL
STATE OF CALIFORNIA

Elizabeth M. Mitchell, G.E.
President\Principal Geotechnical Engineer
G.E. 2718

Copies: 3 to Client

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# I. INTRODUCTION

## PURPOSE AND SCOPE

This report describes our geotechnical investigation and presents our findings, conclusions and recommendations for the proposed Rio Ranch Retail located in Carmel, California.

Our scope of services for this project has consisted of:

1. Review of the conceptual plans prepared by Perkins Williams and Cotterill and dated August 10, 2016.
2. Review of the following published maps:
   - 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 Relative Earthquake-Induced Landslide Susceptibility of Monterey County, California, Rosenberg, 2001.
   - Map Showing Liquefaction Susceptibility of Monterey County, California, Rosenberg, 2001.
   - Map Showing Relative Fault Hazards of Monterey County, California, Rosenberg, 2001.
3. Geographic Information System – Monterey County, "Geologic Hazards Map for Monterey County".
4. Obtaining soil boring permits from the County of Monterey.
5. Drilling and logging of 3 test borings.
6. Observation during advancement of 3 cone penetration tests (CPT soundings) by our subcontractor, Gregg Drilling,
7. Laboratory analysis of retrieved soil samples.
8. Engineering analysis of the field and laboratory test results.
9. Preparation of this report documenting our investigation and presenting preliminary geotechnical recommendations for the project.

## PROJECT LOCATION

The subject site is located at 3705 Rio Road, in Carmel, California. 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 = 36.538837 degrees
Longitude = -121.907766 degrees

OVERVIEW

The proposed project consists of roughly 46,000 square feet of retail commercial building space. We understand the buildings will be single story structures with slab-on-grade floors and wood frame construction. No building loads are available at this time but we anticipate they will be typical of this type of structure. The project will include 186 parking spaces and associated landscaping..

The property is located within the FEMA defined Special Hazard Area of the Carmel River and the proposed grading includes import of about 13, 500 cubic yards of fill to raise the building pads above the flood plain. Depth of fill is anticipated to be about 2 to 5 feet. The project will likely also include utilities, small retaining walls and miscellaneous site improvements.

## II. INVESTIGATION METHODS

SOIL BORINGS

Three, 8-inch diameter test borings were drilled at the site on August 29, 2017. The location of the test borings are shown on the Site Map, Figure No. 2, in Appendix A. The drilling method used was hydraulically operated continuous flight, hollow-stem augers on a truck mounted drill rig. A geologist 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 height of 30 inches. The hammer was actuated with a down hole safety hammer that was “rodded up” to the ground surface. 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. All standard penetration test 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, Figure No. 3, Appendix A. The soil classification was verified by laboratory testing performed 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.

CONE PENETROMETER TESTING (CPT)

Three CPT soundings were performed on August 15, 2017. All were performed following standard ASTM procedures in accordance with test method D5778.

The cone penetrometer soundings with pore pressure measurements were advanced using a 1.4 inch diameter hydraulically operated electronic cone with a friction sleeve. A saturated piezo element is placed between the cone and the friction sleeve to obtain dynamic pore pressure parameters. Continuous measurements were made of the tip resistance, the friction sleeve resistance, and the dynamic pore pressure as the cone was pushed into the ground. Static pore pressures were obtained in 2 of our 3 soundings to estimate the static ground water level. Real time data along with correlations between these measurements and soil properties were observed as the probe was advanced so that our engineer could determine the depth of soundings required.

All sounding holes were backfilled with grout by means of a tremie pipe that was extended to the base of the hole.

The CPT data and typical interpreted soil properties are included at the end of Appendix A.

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.
- R-value testing in accordance with California 301.

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

## III. FINDINGS AND ANALYSIS

GEOLOGIC SETTING

The majority of the site is mapped as being underlain by "older flood plain deposits", with the southeast corner (near the proposed Store B) mapped as "younger flood plain deposits" ( Clark, 1997). The native soils we encountered during our field investigation were consistent with these general descriptions.

Correspondingly, the liquefaction potential is mapped as high in the southeast corner and low to moderate in the majority of the site (Rosenberg, 2001).

SURFACE CONDITIONS

The property is about 3.77 acres in total size and bounded by Rio Road to the southwest and developed commercial properties on the remaining 3 sides. It is roughly flat with a half a dozen large oak trees scattered in the northeast portion.

Large volumes of fill material have been dumped on the site over the years and the approximate areas underlain by fill is depicted on Figure 2. The fill depth is roughly 3 to 5 feet in the eastern half of the site and large granitic cobbles and boulders up to 12 inches in size are scattered across the surface in this area. Fill in the western portion of the property is generally less than 2½ feet in depth and appears to have less cobbles and boulders and contain more fine grained material.

SUBSURFACE CONDITIONS

Our subsurface exploration consisted of three test borings and three cone penetrometer soundings. The borings extended to 25 feet below existing grade and the cone penetrometer soundings extended to 50 feet. The soil profiles and classifications, laboratory test results and groundwater conditions encountered for each test boring are presented in the Logs of Test Borings and Cone Penetrometer Reports in Appendix A. The general subsurface conditions are described below.

Earth Materials:

Imported artificial fill covers the majority of the site and its approximate location is shown on Figure 2. The composition is generally a silty sand with gravel and the depth of fill is described in the previous section. Beneath the fill, native earth materials encountered in all three borings were similar. The upper 10 to 13 feet of soil predominantly consists of silty sand and silty sand with gravel (SM) with fines content (percent silt and clay) ranging from about 15 to 45 %. The density of the sand ranged from loose to medium dense. Boring 2 encountered a layer of firm sandy silt that extends from about 8½ to 13 feet below ground surface.

Below about 13 feet and extending to the bottom of our drill holes at 25 feet the soils encountered by our borings were more consistently a loose to medium dense sand which contained 1 to 4% fines. Based on the CPT soundings it appears that clean, loose to medium dense sands extend to at least 48 feet below ground surface.

Groundwater:

Groundwater was encountered in all three of our borings and was measured to be between 16½ and 18½ feet below ground surface on August 29, 2017. The groundwater conditions described in this report reflect the conditions encountered during our drilling investigation in August of 2017 at the specific locations drilled. It must be anticipated that perched and regional groundwater tables may vary with location and could fluctuate with variations in 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 below. The fault distances are approximate distances based the U.S. Geological Survey and California Geological Survey, Quaternary fault and fold database, accessed on March 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 | 1½ | Northeast |
| San Gregorio | 4½ | Southwest |
| Reliz | 12½ | Northeast |
| San Andreas | 30 | Northeast |

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[2] | D |
| Spectral Acceleration for Short Periods | $S_s = 1.57g$ |
| Spectral Acceleration for 1-second Period | $S_1 = 0.591g$ |
| Short Period Site Coefficient | $F_a = 1.0$ |
| 1-Second Period Site Coefficient | $F_v = 1.5$ |
| MCE Spectral Response Acceleration for Short Period | $S_{MS} = 1.572g$ |
| MCE Spectral Response Acceleration for 1-Second Period | $S_{M1} = 0.886g$ |
| Design Spectral Response Acceleration for Short Period | $S_{DS} = 1.048g$ |
| Design Spectral Response Acceleration for 1-Second Period | $S_{D1} = 0.591g$ |
| Seismic Design Category [3] | 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 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. If ground modification is performed the site class should be reassessed as needed.

**Note 3:** 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.

GEOTECHNICAL HAZARDS

In general the geotechnical hazards associated with projects located in the Carmel River area of Monterey County include seismic shaking (discussed above), ground surface fault rupture, liquefaction, lateral spreading, landsliding and expansive soils. A qualitative 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 local regional 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 is mapped approximately 1.5 miles 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 regional geologic hazard maps portions of the property are mapped as having a high potential for liquefaction and the remainder as having a low to moderate potential.

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.

We analyzed the potential for liquefaction to occur on the site using the methodologies presented by Seed et. al. (2003) and updated by Robertson (2009) and incorporated into the software, "CLIQ", version 2.2.0.28 by Geologismiki. The following criteria were used for our analysis:

1. Estimated mean peak ground accelerations of 0.62 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 15 feet below the ground surface.

The results of our liquefaction analysis indicate that there is a high to very high probability of liquefaction occurring in the sand strata that extends from about 15 feet to about 48 feet below ground surface. Total ground settlement due to liquefaction in these loose sand layers is estimated to be between about 5.8 and 8.5 inches. 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.

Seismically Induced Settlement of Dry Sands:

Non-saturated "dry" sands may settle and densify when subjected to earthquake shaking. Settlement tends to occur in loose clean sands with little or few cohesive fines. Settlement of dry sands, and the corresponding effects on structures, is a function of the magnitude and duration of the earthquake, the ground accelerations that occur at the site, the relative density of the sand, the amount and cohesiveness of the fines within the sand, and the thickness and depth of the sand strata. Based on our boring and CPT data we roughly estimate that the magnitude of dry sand settlement will be on the order of ½ to 1 inch.

It must be cautioned that liquefaction and lateral spreading analysis is an inexact science and the mathematical models of the liquefaction and liquefiable soils contain many simplifying assumptions, not the least of which are isotropy and homogeneity. In this environment estimating earthquake-induced settlement is an inexact science and the mathematical model contains many simplifying assumptions. Studies have shown that total liquefaction-induced settlement estimates can vary from 50 to 200% of the estimated values. Less accuracy should be expected for more complex predictions involving earthquakes and non-homogeneous subsurface conditions such as those found at the subject site. Actual settlement at the site may be greater or less than that predicted and will depend on numerous factors including the magnitude and duration of the earthquake, and the location and design of the structure.

Landsliding

There are no significant slopes on or adjacent to the project site. It is our opinion that the potential for shallow or localized slope failures to occur and cause damage to the proposed project should be considered to be low.

Expansive Soils

The subject site is directly underlain by sand with fines content on the order of 7% to 8%. This is a soil type with a low potential for expansion.

# IV. CONCLUSIONS AND DISCUSSION

GENERAL

1. The results of our investigation indicate that from a geotechnical engineering standpoint the property may be developed as proposed. It is our opinion that provided our recommendations are followed; the proposed project can be designed and constructed to an "ordinary" level of seismic risk and performance as defined below:

> **"Ordinary Risk":** Resist minor earthquakes without damage: resist moderate earthquakes without structural damage, but with some non-structural damage: resist major earthquakes of the intensity or severity of the strongest experienced in California without collapse, but with some structural damage as well as non-structural damage. In most structures it is expected that structural damage, even in a major earthquake, could be limited to reparable damage. *(Source: Meeting the Earthquake Challenge, Joint Committee on Seismic Safety of the California Legislature, January 1974).*

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 building damage and the need for post-earthquake repairs. It should be assumed that exterior improvements such as pavements, slabs, sidewalks or patios will need to be repaired or replaced following strong seismic shaking. If the property owner desires a higher level of seismic performance for this project, supplemental design and construction recommendations will be required.

2. 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.

3. Prior to final design and construction of the proposed improvements mitigations measures will be required to reduce the liquefaction hazard at the site and to reduce the liquefaction-induced settlement. More detailed recommendations are provided below.

4. Proposed ground improvement techniques should be reviewed by Pacific Crest Engineering Inc. during their preparation and mitigation measures observed when performed. Verification testing should be done after the mitigation measures are completed to confirm that the site soils have been densified sufficiently to reduce liquefaction-induced settlement to an acceptable level.

5. 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 contractor and one of our engineers present. At this meeting, the project specifications and the testing and inspection responsibilities will be outlined and discussed.

6. 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

7. 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 at the subject site are the following:

a. *Liquefaction:* The site soils are liquefiable and our analysis indicates that significant settlement of the ground surface may occur during a major earthquake. To mitigate earthquake induced settlement ground improvement techniques should be employed to densify the soils at depth. Such techniques could include stone columns, vibrocompaction or other methods. Providing recommendations for ground improvement is beyond our current scope of work. At your request, we can provide supplemental recommendations for the design and construction of ground improvement techniques. Alternatively to aid in assessing which method may be most effective and practical, we recommend the findings and data in this report be presented to contractors specializing in deep ground improvement.

b. *Existing Loose Fill and Native Soils:* Approximately 3½ to 5 feet of loose non-engineered fill has been imported to the site. The loose fill is prone to settlement with poor load bearing characteristics. To mitigate any adverse effects due to the presence of the loose fill we recommend that all existing fills within areas to support building foundations, concrete slabs-on-grade, engineered fill or pavements be removed and replaced as engineered fill. Detailed recommendations are provided in the EARTHWORK Section of this report.

c. *Earthquake induced settlement of dry sands.* Our analysis indicates that there is a potential for earthquake-induced settlement to occur due to the generally loose condition of the sand.

   To mitigate the adverse effects of seismically induced settlement, should it occur, we recommend that the proposed structures be supported by mat foundation systems that are designed to behave as a unit, resist differential ground settlement, and span seismically induced voids. The buildings should be designed to tolerate re-leveling, should this become necessary. Preliminary design recommendations are provided in the FOUNDATIONS section of this report.

d. *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 demolition of the existing structures and their foundations and removal of designated trees and debris. All foundation elements from existing structures must be completely removed from the building areas. Tree removal should include the entire stump and root ball. Septic tanks and leaching lines, 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 old structures and their foundations, 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. Two wells exist at the site. Wells within proposed improvement areas should 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 of 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. Significant portions of the site are covered by imported fill. All imported fill should 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. Following stripping and subexcavation of all existing fill, exposed soils that are to underlie building foundations and concrete slabs-on-grade should be removed to a minimum depth of 24 inches below finished pad grade or as designated by a representative of Pacific Crest Engineering during construction. Within pavement areas the exposed soils should be removed to a depth of 18 inches below finished soil subgrade. Following approval of the excavation by the Geotechnical Engineer in the field, the base of the excavation should be scarified, moisture conditioned and compacted as an engineered fill. The scarification and compaction of the base of the excavation may be waived at the sole discretion of the Geotechnical Engineer, based on the conditions exposed. There should be a minimum of 18 inches of

engineered fill under all foundation elements. Recompacted sections should extend 5 feet horizontally beyond the building perimeter and at least 2 feet beyond pavements and exterior slabs or hardscape.

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

Material for Engineered Fill

9. 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.

***It should be noted that oversized cobbles and boulders were observed within the existing imported fill and screening of this material will likely be required.***

10. 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.

11. 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.

Engineered Fill Placement and Compaction

12. 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.

13. All soil on the project should be compacted to the minimum compaction requirements outlined in the following table:

**Table No. 3 - Minimum Compaction Requirements**

| Percent of Maximum Dry Density | Location |
|---|---|
| 95% | • All aggregate base and subbase in pavement areas<br>• The upper 8 inches of subgrade in pavement areas<br>• All utility trench backfill in pavement areas |
| 90% | All remaining native soil and fill material |

14. 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).

15. We recommend field density testing be performed in maximum 18 inch elevation differences. In general, 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.

Cut and Fill Slopes

16. No permanent cut or fill slopes are currently proposed. If proposed we recommend cut and fill slopes be constructed at a 2:1 (horiz:vert) inclination or flatter. Foundations, pavements and concrete slabs-on-grade should be set back at least 8 feet from the tops of cut and fill slopes.

Soil Moisture and Weather Conditions

17. 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.

Excavations and Shoring

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

## UTILITY TRENCHES

19. Utility trenches that are parallel to the sides of buildings 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.

20. 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.

21. 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.

22. 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.

23. 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

24. 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.

25. 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.

26. 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.

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

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

## FOUNDATIONS – STRUCTURAL MAT

29. Preliminary foundation recommendations are presented for preliminary design only. Recommendations presented below assume that ground improvement techniques are employed and liquefaction-induced settlement has been reduced to tolerable levels as presented below. We request the opportunity to review proposed ground modification measures, observe the operation and verify that the site soils have been densified to acceptable levels to perform as assumed below.

30. 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.

31. Considering the potential for significant ground settlement during a major earthquake, it is our opinion that the structures should be supported by a reinforced-concrete, structural mat with tie-beams. The mat should be designed to move as a unit and to tolerate the differential settlement and lateral spreading values outlined in this report.

32. A structural grid or mat foundation system designed in accordance with the recommendations of this report should result in a structure that survives episodes of seismically-induced ground settlement with repairable damage.

33. The structural mat should be designed using the following criteria:

a. The mat foundation should include tie-beams with a maximum spacing of 25 feet.
b. Tie-beams and a thickened edge beam that should extend a minimum of 18 inches below the lowest adjacent compacted pad grade.
c. The structural mat should be designed to span a void of 10 feet appearing anywhere under the foundation and accommodate a differential settlement of 2 inches in 15 feet.
d. The structural mat should be designed for an allowable bearing capacity of 1500 psf, (dead plus live load). The allowable bearing capacity may be increased by ⅓$^{rd}$ for wind or seismic loads.
e. The recommended coefficient of vertical subgrade reaction ($K_v$) to be used in design is 60 tons per cubic foot.
f. The embedded portion of the mat may be assumed to have a lateral bearing pressure resistance value of 300 psf/ft for the section of mat embedded below the ground surface.
g. The mat may be assumed to have a resistance to lateral sliding of 0.35.

34. The footings should contain steel reinforcement as determined by the project structural engineer in accordance with applicable CBC, ACI or other applicable standards.

35. Excavations for tie beams and the deepened slab edges should be adequately moisture conditioned prior to placing concrete. Requirements for moisture conditioning the footing subgrade will depend on the soil type and seasonal moisture conditions, and will be determined by the Geotechnical Engineer at the time of construction.

36. All footing excavations must be observed by a representative of Pacific Crest Engineering before steel is placed and concrete is poured to insure bedding into proper material.

37. 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.

38. Where floor coverings are anticipated or 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.

39. 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.

40. 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. 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 of the National Ready Mixed Concrete Association (NRMCA), 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. Bleed water trapped below a finished surface can cause delaminations (CIP 20) or blisters (CIP 13) if finishing operations are not performed at the correct time after bleed water has disappeared from the surface. Concrete may stiffen slower, which means that trowel finishing operations must be delayed; thus increasing the susceptibility of plastic shrinkage cracking. Curling (CIP 19) can occur due to differential drying and related shrinkage at different levels in the slab. 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.

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

## SLAB-ON-GRADE CONSTRUCTION

41. In addition to the recommendations presented below, design and construction of concrete slab-on-grade floors should also follow Section 4.505.2 of the 2016 California Green Building Standards Code, which includes installing a vapor retarder in direct contact with concrete and a mix design that addresses bleeding, shrinkage and curling.

42. Interior concrete slabs should bear upon compacted engineered fill in accordance with the recommendations presented in the EARTHWORK section of this report.

43. All exterior slabs, patios, walkways, etc., should be structurally independent of structural foundation system(s).

44. All interior concrete slabs-on-grade 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.

45. Where floor coverings are anticipated or 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.

46. 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.

47. 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**.

48. 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.

49. 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.

50. 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.

51. Final slab thickness, reinforcement, doweling and crack control devices should be determined by the project civil or structural engineer. The use of welded wire mesh is not recommended for slab reinforcement.

RETAINING WALLS

52. We anticipate that retaining walls on the order of 3 to 4 feet may be proposed as site walls. The following recommendations are presented for preliminary design purposes. We request the opportunity to review the location of any proposed retaining walls to confirm that our recommendations apply.

53. We recommend retaining walls be founded on spread footings embedded a minimum of 18 inches below the lowest adjacent grade. Landscape area retaining walls independent of the buildings and constructed in accordance with the preceding requirements should be designed to the following criteria:

**Table No. 4 - Retaining Wall Footings**

| Footing Width (minimum) | Embedment Depth | Bearing Capacity |
|---|---|---|
| 2 feet | 18 inches | 1,500 psf |
| 3 feet | 18 inches | 1,700 psf |
| 4 feet | 18 inches | 1,900 psf |

54. Retaining walls with full drainage should be designed using the following lateral earth pressure values:

**Table No. 5 - Active and At-Rest Earth Pressure Values**

| Maximum Backfill Slope (H:V) | Active Earth Pressure (psf/ft of depth) | At-Rest Earth Pressure (psf/ft of depth) |
|---|---|---|
| Level | 40 | 60 |
| 2:1 | 50 | 70 |

a. Should the slope behind the retaining walls be other than shown in Table 5, supplemental design criteria will be provided for the active earth or at rest pressures for the particular slope angle.

b. Active earth pressure values may be used when walls are free to yield an amount sufficient to develop the active earth pressure condition (about ½% of height). The effect of wall rotation should be considered for areas behind the planned retaining wall (pavements, foundations, slabs, etc.). When walls are restrained at the top or to design for minimal wall rotation, at-rest earth pressure values should be used.

55. For resisting passive earth pressure use 300 psf/ft of depth.

56. A "coefficient of friction" between base of foundation and soil of 0.35.

57. If the structural designer wishes to include seismic forces in their design, the wall may be designed using the above active soil pressures plus a horizontal seismic force of $10H^2$ pounds per lineal foot (where H is the height of retained material). The resultant seismic force should be applied at a point 1/3rd above the base of the wall. This force has been estimated using the Mononobe-Okabe method of analysis as modified by Whitman (1990) and Lew and Sitar (2010). A reduced factor of safety for overturning and sliding may be used in seismic design as determined by the structural designer.

58. The above seismic forces should not be used in combination with at rest lateral soil pressures.

Retaining Wall Drainage

59. The above design criteria are based on fully drained conditions. Therefore, we recommend that permeable material meeting the State of California Standard Specification Section 68-1.025, Class 1, Type A, be placed behind the wall, with a minimum width of 12 inches and extending for the full height of the wall to within 1 foot of the ground surface. The top of the permeable material should be covered with Mirafi 140N filter fabric or equivalent and then compacted native soil placed to the ground surface. A 4-inch diameter perforated rigid plastic drain pipe should be installed within 3 inches of the bottom of the permeable material and be discharged to a suitable, approved location. The perforations should be placed downward; oriented along the lower half of the pipe. Neither the pipe nor the permeable material should be wrapped in filter fabric. Please refer to the Typical Retaining Wall Drain Detail, Figure 10, in Appendix A for details.

60. The area behind the wall and beyond the permeable material should be compacted with approved material to a minimum relative compaction of 90%.

SURFACE DRAINAGE

61. 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.

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

63. 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.

64. 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.

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

66. 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.

67. 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

68. The surface soils are classified as having a high 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.

## PAVEMENT DESIGN

69. Two bulk samples of soil were recovered from the upper 3 feet of soil near B-3. "R" values of 35 and 52 were measured in the laboratory on these samples. Based on the Caltrans Highway Design Manual Chapter 630 latest edition, the following table provides alternative flexible pavement sections for traffic indices of 5.5 and 6.0 and two options of pavement rehabilitation as outlined in the following section. This procedure assumes a 20-year design life and an R value of 35. Final pavement section and design traffic index should be determined by the project civil engineer.

**Table No. 6. Recommended Alternative Pavement Sections**

| Traffic Index | Asphaltic Concrete (inches) | Class 2 Baserock R= 78 min (inches) | Total Section (inches) |
|---|---|---|---|
| 4.5 | 2.5 | 4.5 | 7.0 |
| 5.0 | 2.5 | 5.5 | 8.0 |
| 5.5 | 3.0 | 6.0 | 9.0 |
| 6.0 | 3.5 | 6.5 | 10.0 |
| 6.5 | 3.5 | 8.0 | 11.5 |
| 7.0 | 4.0 | 8.0 | 12.0 |
| 7.5 | 4.5 | 8.5 | 13.0 |

70. 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.

PLAN REVIEW

71. 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 Rincon Consultants 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.*

*While you cannot eliminate all such risks, you can manage them. The following information is provided to help.*

## 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

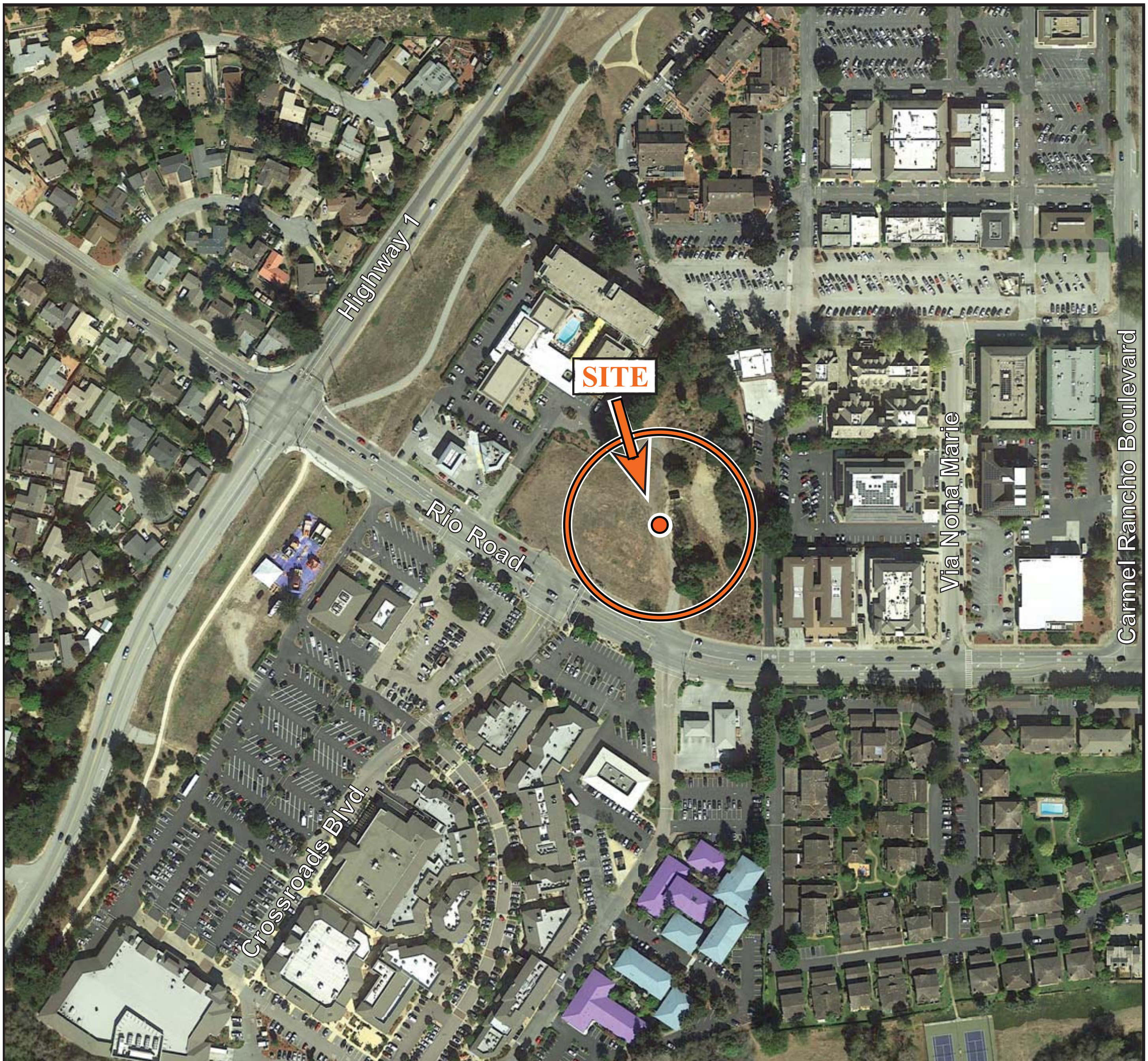
Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

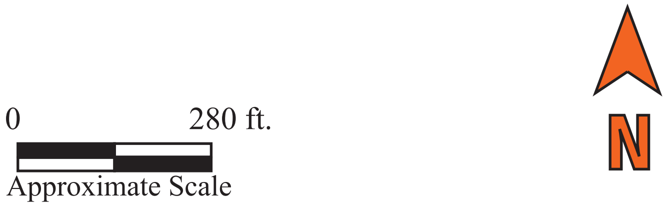
ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

# APPENDIX A

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




Base Map from Google Maps

| Pacific Crest Engineering Inc.<br>444 Airport Blvd., Suite 106<br>Watsonville, CA 95076 | **Regional Site Map**<br>Rio Ranch Marketplace<br>Carmel, California | Figure No. 1<br>Project No. 1713<br>Date: 10/12/17 |
|---|---|---|


## EXPLANATION

B-3 Location of Test Boring

CPT-3 Location of Cone Penetrometer Test (CPT)

Approximate Area underlain by Existing Fill

NORTH

0 80 160

Scale: 1 inch = 80 feet

Base Map: Titled "Rio Ranch Marketplaceby Perkins, Williams & Cotterill Architects 1" = 40', dated 8/10/16

Pacific Crest ENGINEERING INC

**Site Map Showing Proposed Test Boring and CPT Locations**
Rio Ranch Marketplace
Carmel, California

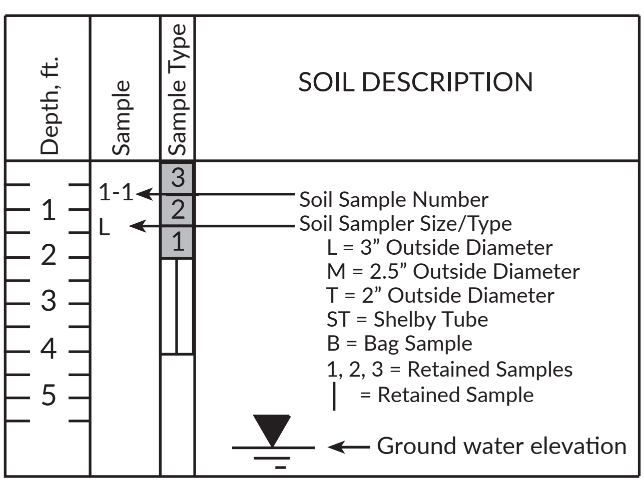
Figure No. 2
Project No. 1713
Date: 10/12/17

# 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**
Rio Ranch Marketplace
Carmel, California

Figure No. 3
Project No. 1713
Date: 10/12/17

## 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**
Rio Ranch Marketplace
Carmel, California

Figure No. 4
Project No. 1713
Date: 10/12/17

LOGGED BY CLA DATE DRILLED 8/29/17 BORING DIAMETER 8" HS BORING NO. 1

DRILL RIG EGI Truck Mounted Mobile B61 HAMMER TYPE 140 lb Down-Hole Safety Hammer

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–2 | 1-1 L | 2, 1 | **FILL: SILTY SAND WITH GRAVEL:** Dark grayish brown (10YR 4/2), very fine to fine grained with trace medium grains, poorly graded, poorly indurated, sub-angular to sub-rounded shaped quartz and sandstone gravels up to ¼ inch in diameter, dry, medium dense | SM | 22 23 34 | 29 | | | 111 | 4 | |
| 3 | 1-2 L | | BASEROCK | | 50/5" | | | | | | |
| 4.5 | | | ? — ? — ? — ? — ? — ? — ? — ? — ? — ? | | | | | | | | |
| 5–6 | 1-3 L | 2, 1 | **NATIVE: SILTY SAND:** Light olive brown (2.5Y 5/3), fine grained with trace medium grains, quartz rich, poorly graded, clean, poorly indurated, scattered mica flakes, 75% lithics, slightly moist to dry, medium grains | SM | 11 11 11 | 12 | | 17 | 101 | 9 | |
| 8 | | | ? — ? — ? — ? — ? — ? — ? — ? — ? — ? | | | | | | | | |
| 8.5–10 | 1-4 T | | **SILTY SAND:** Mottled dark grayish brown (10YR 4/2) and yellowish red (5YR 5/8), very fine grained, poorly graded, quartz rich, micaceous, poorly indurated, slightly moist, loose, (trace rootlets) | SM | 3 4 5 | 9 | | | | | |
| 13 | | | — — — — — — — — — — — — — | | | | | | | | |
| 13–15 | 1-5 L | 3, 2 | **SILTY SAND WITH GRAVEL:** Light yellowish brown (2.5Y 6/3) and white (2.5Y 8/1), medium to coarse grained with trace fine grains, sub-angular to rounded shaped, clean, quartz rich, scattered mica flakes, rounded quartz and sandstone gravels up to ¼ inch in diameter, dry, medium dense | SM | 6 10 12 | 12 | | 15 | 95 | 8 | |
| 16 | | | ▽ | | | | | | | | |
| 18.5–20 | 1-6 T | | Light olive brown (2.5Y 5/3), micaceous, slight decrease in gravel content, wet | | 7 9 14 | 23 | | | | | |
| 23 | | | ? — ? — ? — ? — ? — ? — ? — ? — ? — ? | | | | | | | | |

Pacific Crest ENGINEERING INC

**Log of Test Borings**
Rio Ranch Marketplace
Carmel, California

Figure No. 5
Project No. 1713
Date: 10/12/17

LOGGED BY CLA DATE DRILLED 8/29/17 BORING DIAMETER 8" HS BORING NO. 1

DRILL RIG EGI Truck Mounted Mobile B61 HAMMER TYPE 140 lb Down-Hole Safety Hammer

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 24<br>25 | 1-7<br>L | 2<br>1 | **SAND:** Light olive gray (5Y 6/2) and white (5Y 8/1) changing to gray (5Y 6/1) and white (5Y 8/1), medium to coarse grained with trace fine to trace coarse grading to medium to coarse grianed with depth, sub-angular to sub-rounded shaped, moderately graded, clean, quartz rich, poorly indurated, > 5% lithics, wet, medium dense | SP | 20<br>25<br>30 | 28 | | 4 | 109 | 16 | |
| 26<br>27 | | | Boring terminated at 25 feet. Groundwater initially encountered at 16½ feet. Measured at 17.80 feet at the end of drilling activities. | | | | | | | | |
| 28 | | | | | | | | | | | |
| 29 | | | | | | | | | | | |
| 30 | | | | | | | | | | | |
| 31 | | | | | | | | | | | |
| 32 | | | | | | | | | | | |
| 33 | | | | | | | | | | | |
| 34 | | | | | | | | | | | |
| 35 | | | | | | | | | | | |
| 36 | | | | | | | | | | | |
| 37 | | | | | | | | | | | |
| 38 | | | | | | | | | | | |
| 39 | | | | | | | | | | | |
| 40 | | | | | | | | | | | |
| 41 | | | | | | | | | | | |
| 42 | | | | | | | | | | | |
| 43 | | | | | | | | | | | |
| 44 | | | | | | | | | | | |
| 45 | | | | | | | | | | | |
| 46 | | | | | | | | | | | |

LOGGED BY CLA DATE DRILLED 8/29/17 BORING DIAMETER 8" HS BORING NO. 2

DRILL RIG EGI Truck Mounted Mobile B61 HAMMER TYPE 140 lb Down-Hole Safety Hammer

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–2 | 2-1 L | 2, 1 | **FILL: SILTY SAND WITH GRAVEL:** Brown (10YR 4/3), very fine to fine grained, poorly graded, quartz rich, poorly to slightly indurated, sub-angular shaped granitic gravels up to 2 inches in diameter, slightly moist to dry, medium dense, (fines with depth, trace medium and coarse grains from 1 to 1½ feet) | SM | 22 19 15 | 18 | | | | | |
| 2–4 | 2-2 L | 2, 1 | Increase in coarseness of sand, very fine to medium grained with trace coarse grains, decrease in gravel content | | 12 10 13 | 12 | | 45 | 92 | 13 | |
| 4–6 | 2-3 L | 2, 1 | **NATIVE: SILTY SAND:** Mottled very dark grayish brown (10YR 3/2) and yellowish brown (10YR 5/8), very fine grained, poorly graded, quartz rich, poorly to slightly indurated, micaceous, trace sub-rounded to well rounded sandstone gravels up to ¼ inch in diameter, slightly moist, medium dense | SM | 13 14 15 | 15 | | | 106 | 9 | |
| 6–9 | | | Yellowish brown (10YR 5/4) and pale brown (10YR 6/3), slight increase in coarseness of sand, very fine to fine grained, increase in gravel content from 5½ to 6 feet, lack of gravels from 6 to 6½ feet, dry to slightly moist, (sandstone, granitic, and chert gravels) | | | | | | | | |
| 8.5–10 | 2-4 T | | **SANDY SILT:** Very dark grayish brown (10YR 3/2) and yellowish red (5YR 4/6), slightly moist, loose/firm | ML | 4 3 5 | 8 | | 77 | | 27 | |
| 13–15 | 2-5 L | 2, 1 | **SAND:** Light yellowish brown (2.5Y 6/3) and white (2.5Y 8/1), medium to coarse grained with trace fine and trace very coarse grains, sub-angular to sub-rounded shaped, moderately graded, quartz rich, clean, poorly indurated, trace sub-angular to sub-rounded shaped quartz and granitic gravels up to 1 inch in diameter, dry, loose, (scattered mica flakes) | SP | 6 6 6 | 6 | | 1 | 99 | 2 | |
| 18.5–20 | 2-6 T | | Grayish brown (2.5Y 5/2), sample fines to medium grained from 19 to 19½ feet, lack of gravels, wet, medium dense | | 6 9 13 | 22 | | | | | |

| LOGGED BY CLA | DATE DRILLED 8/29/17 | BORING DIAMETER 8" HS | BORING NO. 2 |
|---|---|---|---|
| DRILL RIG EGI Truck Mounted Mobile B61 | | HAMMER TYPE 140 lb Down-Hole Safety Hammer | |

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 24<br>25 | 2-7<br>L | 2<br>1 | **SAND:** Grayish brown (2.5Y 5/2), medium to coarse grained with trace fine and trace very coarse grained, sub-angular to sub-rounded shaped, well graded, scattered mica flakes, quartz rich, > 5% lithics clean, poorly indurated, trace silt and sandstone gravels up to ½ inch in diameter, wet, medium dense | SW | 10<br>15<br>30 | 23 | | | 111 | 16 | |
| 26<br>27 | | | Boring terminated at 25 feet. Groundwater initially encountered at 18½ feet. Measured at 17.80 feet at the end of drilling activities. | | | | | | | | |
| 28<br>29<br>30<br>31<br>32<br>33<br>34<br>35<br>36<br>37<br>38<br>39<br>40<br>41<br>42<br>43<br>44<br>45<br>46 | | | | | | | | | | | |

Pacific Crest ENGINEERING INC

**Log of Test Borings**
Rio Ranch Marketplace
Carme, California

Figure No. 8
Project No. 1713
Date: 10/12/17

LOGGED BY CLA DATE DRILLED 8/29/17 BORING DIAMETER 8" HS BORING NO. 3

DRILL RIG EGI Truck Mounted Mobile B61 HAMMER TYPE 140 lb Down-Hole Safety Hammer

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–2 | 3-1 L | 2, 1 | **FILL: CLAYEY SAND WITH GRAVEL:** Very dark grayish brown (10YR 3/2), very fine grained, clay appears to exhibit low to medium plasticity, rootlets, angular shaped siltstone and granitic gravels up to ¼ inch, moist, medium dense | SC | 18 19 22 | 22 | | | | | |
| 2–5.5 | 3-2 L | 2, 1 | **FILL: SILTY SAND WITH GRAVEL:** Light olive brown (2.5Y 5/3), very fine grained with trace fine to very coarse grains, quartz rich, poorly graded, sub-angular to rounded siltstone and sandstone clasts and gravels up to ¼ inch, trace clay lenses, slightly moist, medium dense | SM | 12 15 30 | 23 | | | 103 | 7 | |
| | | | Dark grayish brown (2.5Y 4/2), gravels to 1 inch, dry | | | | | | | | |
| 5.5–8 | 3-3 L | 2, 1 | **NATIVE: SAND:** Light olive brown (2.5Y 5/3), very fine to fine grained, poorly graded, quartz rich, poorly indurated, micaceous, trace coarse to very coarse grained sand, trace rootlets, slightly moist to dry, medium dense | SP | 11 12 16 | 15 | | 22 | 96 | 10 | |
| | | | Lack of very coarse grains, slightly moist, medium dense | | | | | | | | |
| 8–10.5 | 3-4 T | | **SAND:** Light olive brown (2.5Y 5/3), very fine to coarse grained with trace very coarse grains, sub-angular to sub-rounded shaped, well graded, quartz rich, clean, poorly indurated, trace rounded siltstone and sandstone gravels to ¼ inch, slightly moist, medium dense | SW | 3 4 7 | 11 | | | | | |
| 10.5–13 | | | **SILTY SAND:** Mottled dark grayish brown (10YR 4/2) and yellowish red (5YR 5/8), very fine grained, poorly graded, quartz rich, poorly indurated, micaceous, charcoal noted near 9 feet, moist, medium dense | SM | | | | | | | |
| | | | ? — ? — ? — ? — ? — ? — ? — ? — ? — ? | | | | | | | | |
| 13– | 3-5 L | 2, 1 | **SAND:** Light yellowish brown (2.5Y 6/3) and pale brown (2.5Y 8/2), medium to very coarse grained, sub-angular to sub-rounded shaped, well graded, clean, quartz rich, poorly indurated, scattered mica flakes, slightly moist, loose | SW | 8 7 6 | 7 | | 1 | 95 | 5 | |
| 19 | 3-6 T | | Light olive brown (2.5Y 5/3), medium to coarse grained, medium grained lens from 19 to 19½ feet, micaceous, trace rounded sandstone and quartz gravels up to ¼ inch in diameter, wet, medium dense | | 9 10 11 | 22 | | 4 | | 19 | |

**Log of Test Borings**
Rio Ranch Marketplace
Carmel, California

Pacific Crest Engineering Inc

Figure No. 9
Project No. 1713
Date: 10/12/17

LOGGED BY CLA DATE DRILLED 8/29/17 BORING DIAMETER 8" HS BORING NO. 3

DRILL RIG EGI Truck Mounted Mobile B61 HAMMER TYPE 140 lb Down-Hole Safety Hammer

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 24<br>25 | 3-7<br>L | 2<br>1 | **SAND:** Light brownish gray (2.5Y 6/2) and white (2.5Y 8/1), medium to very coarse grained grading to medium to coarse grianed with depth, trace fine grains, sub-angular to sub-rounded shaped, clean, quartz rich, > 5% lithics, poorly indurated, moist, medium dense | SW | 7<br>10<br>20 | 16 | | 2 | 104 | 19 | |
| 26<br>27<br>28<br>29<br>30<br>31<br>32<br>33<br>34<br>35<br>36<br>37<br>38<br>39<br>40<br>41<br>42<br>43<br>44<br>45<br>46 | | | Boring terminated at 25 feet. Groundwater initially encountered at 18 feet. Measured at 16.86 feet at the end of drilling activities. | | | | | | | | |

Pacific Crest ENGINEERING INC

**Log of Test Borings**
Rio Ranch Marketplace
Carme, California

Figure No. 8
Project No. 1713
Date: 10/12/17

# APPENDIX B

Summary of CPT Soundings
Summary of Liquefaction Induced Settlement

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

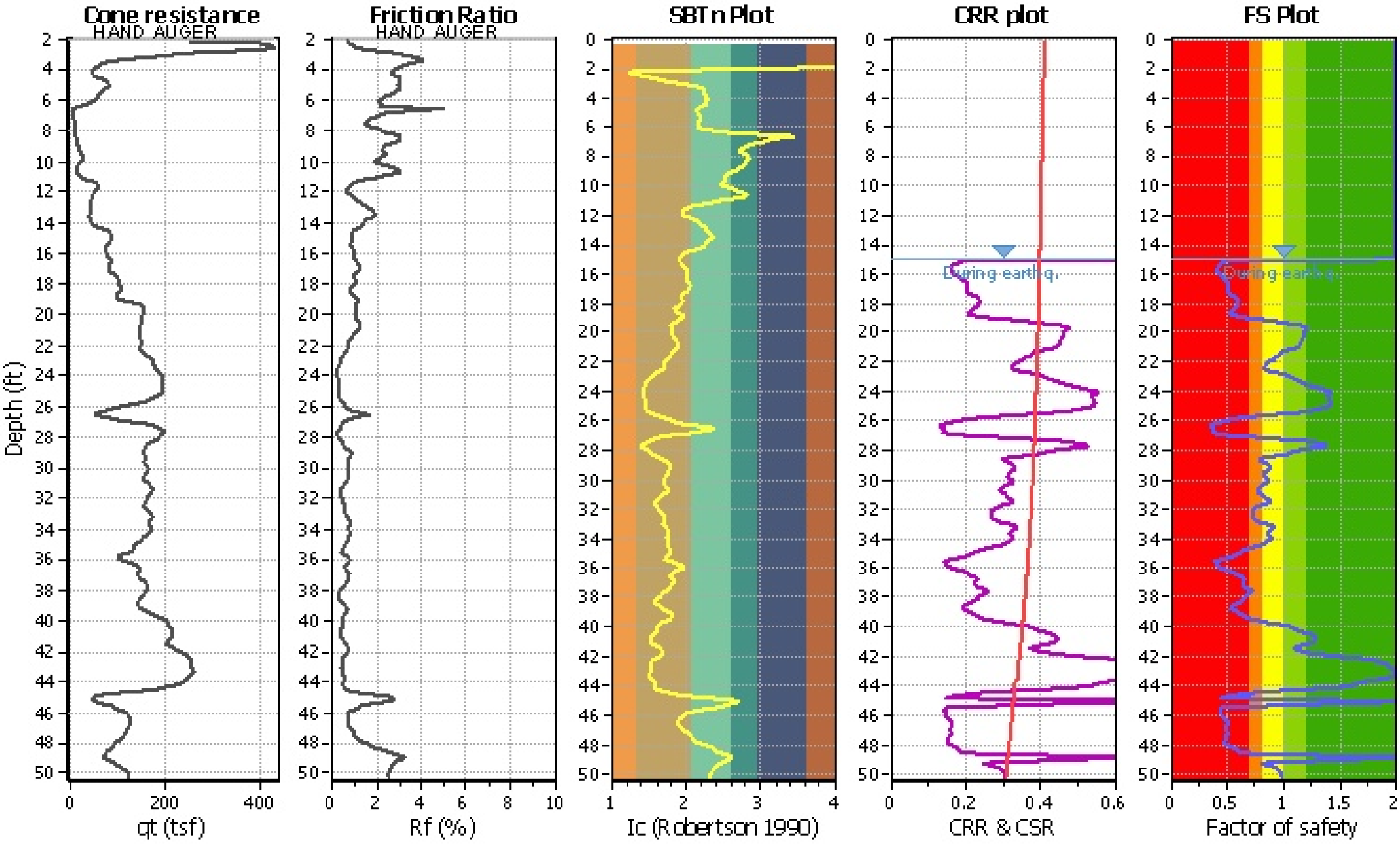
# LIQUEFACTION ANALYSIS REPORT

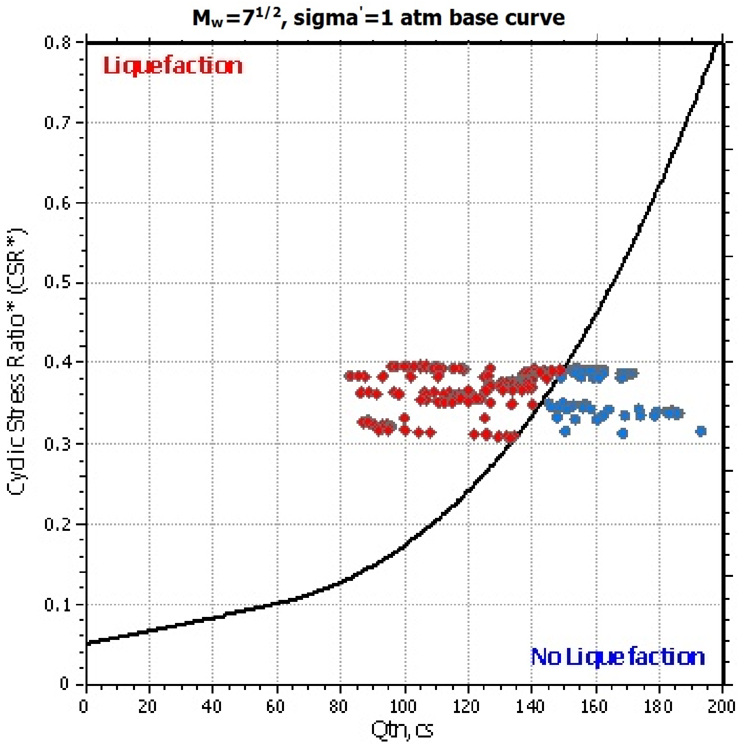
**Project title : Rio Ranch Marketplace** **Location :**

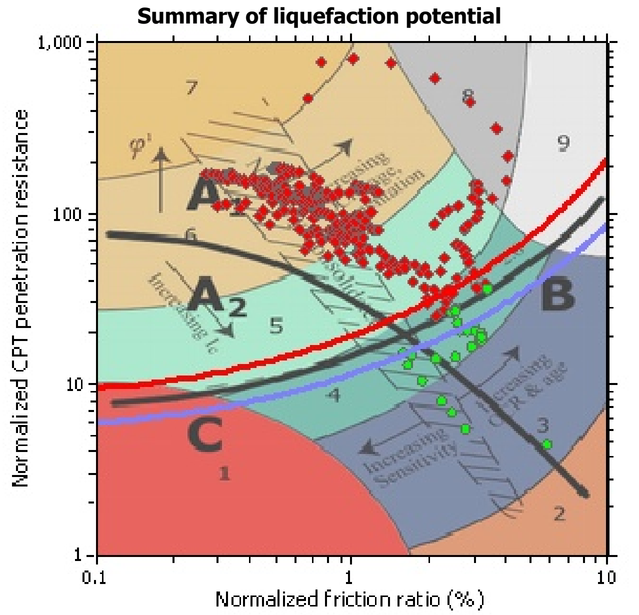
**CPT file : CPT-1**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | G.W.T. (in-situ): | 16.00 ft | Use fill: | No | Clay like behavior applied: | All soils |
| Fines correction method: | Robertson (2009) | G.W.T. (earthq.): | 15.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.50 | Ic cut-off value: | 2.60 | Trans. detect. applied: | No | MSF method: | Method based |
| Peak ground acceleration: | 0.63 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | No | | |

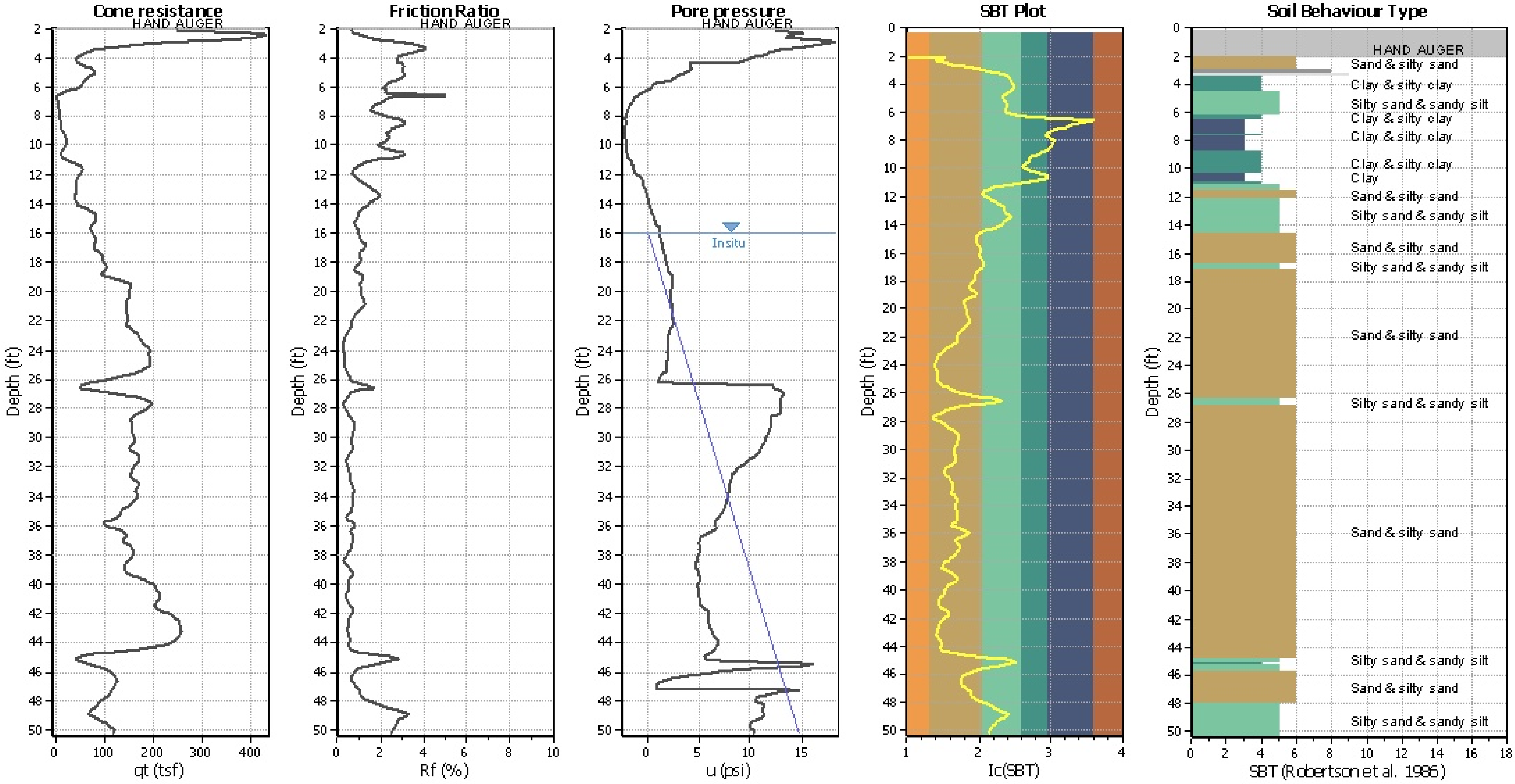



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

## CPT basic interpretation plo



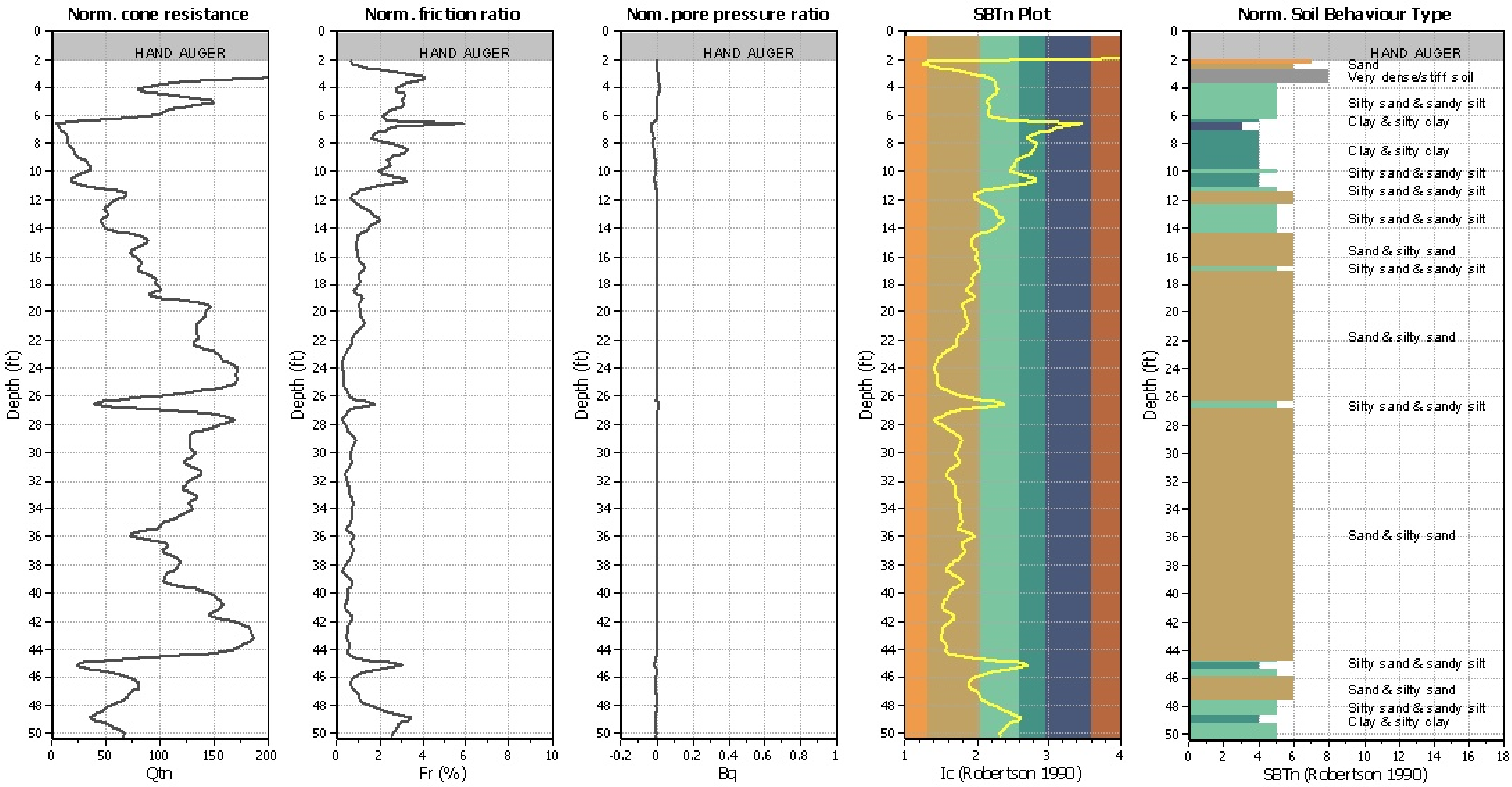

**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 15.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | No |
| Earthquake magnitude $M_w$: | 7.50 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.63 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 16.00 ft | Fill height: | N/A | Limit depth: | N/A |

**SBT legend**

| | | |
|---|---|---|
| 1. Sensitive fine grained | 4. Clayey silt to silty | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

## CPT basic interpretation plots (normaliz



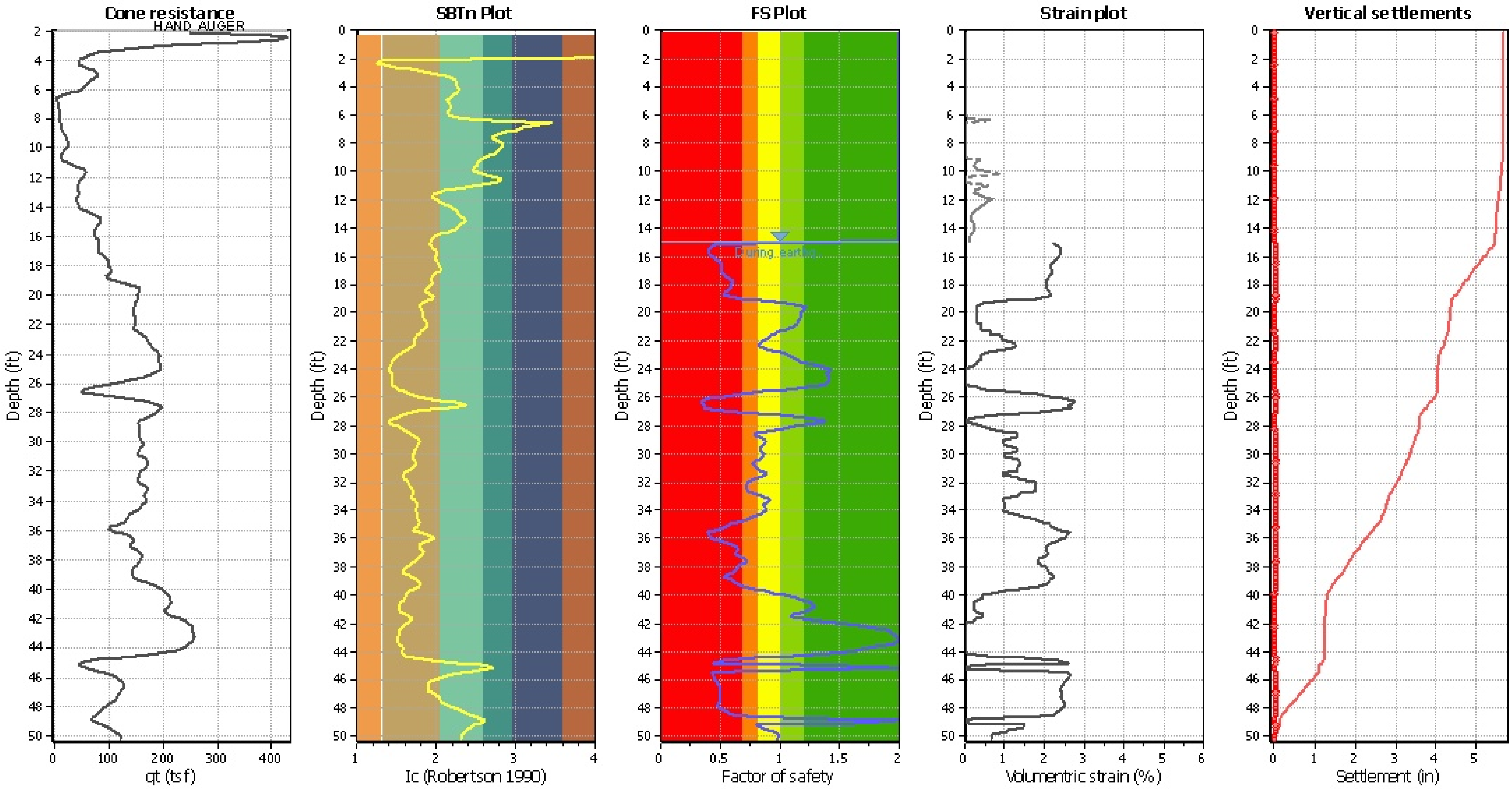

### Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 15.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | No |
| Earthquake magnitude $M_w$: | 7.50 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.63 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 16.00 ft | Fill height: | N/A | Limit depth: | N/A |

### SBTn legend

| | | |
|---|---|---|
| 1. Sensitive fine grained | 4. Clayey silt to silty | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

## Estimation of post-earthquake settlements




**Abbreviations**

| | |
|---|---|
| $q_t$: | Total cone resistance (cone resistance $q_c$ corrected for pore water effects) |
| $I_c$: | Soil Behaviour Type Index |
| FS: | Calculated Factor of Safety against liquefaction |
| Volumentric strain: | Post-liquefaction volumentric strain |

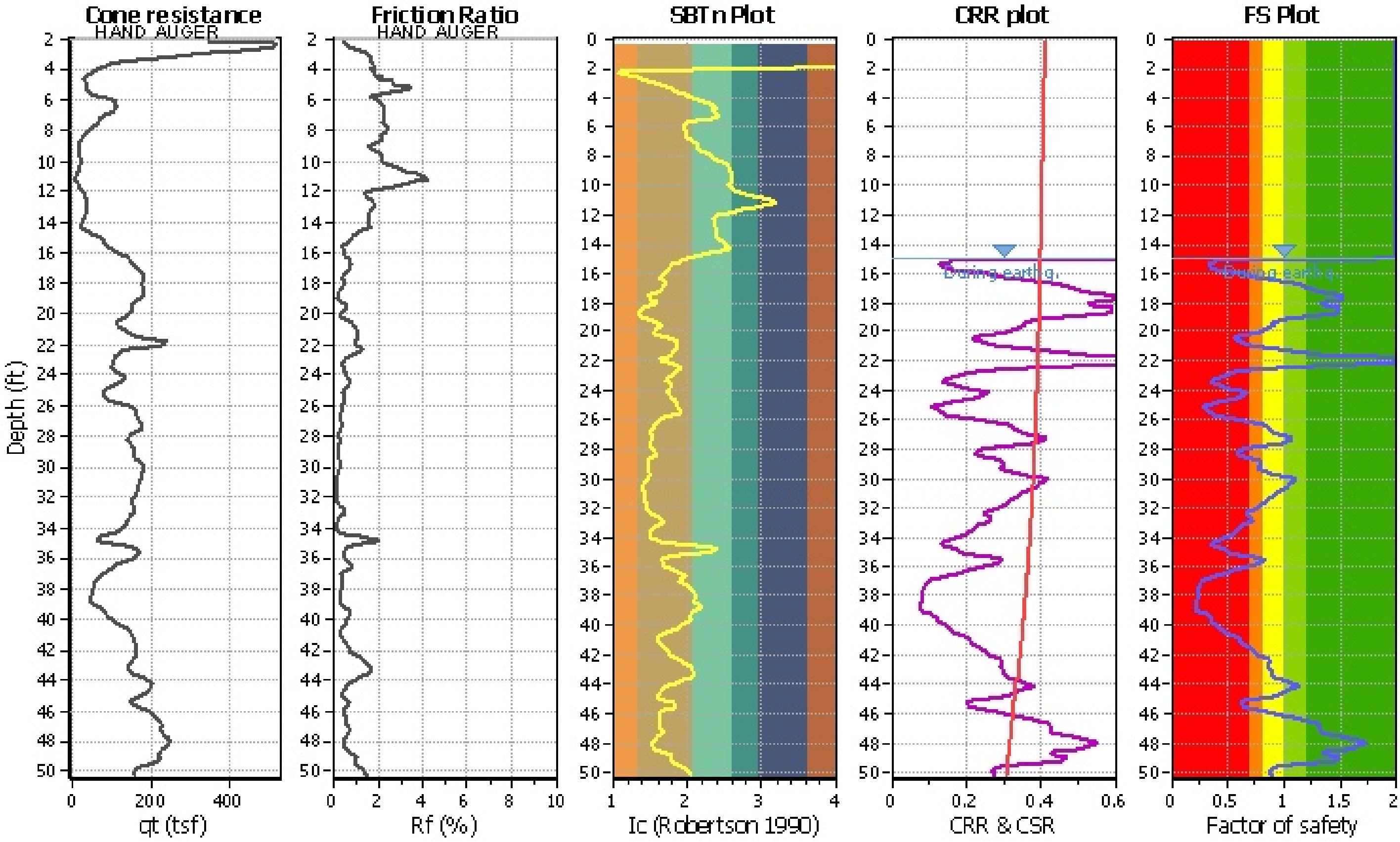
# LIQUEFACTION ANALYSIS REPORT

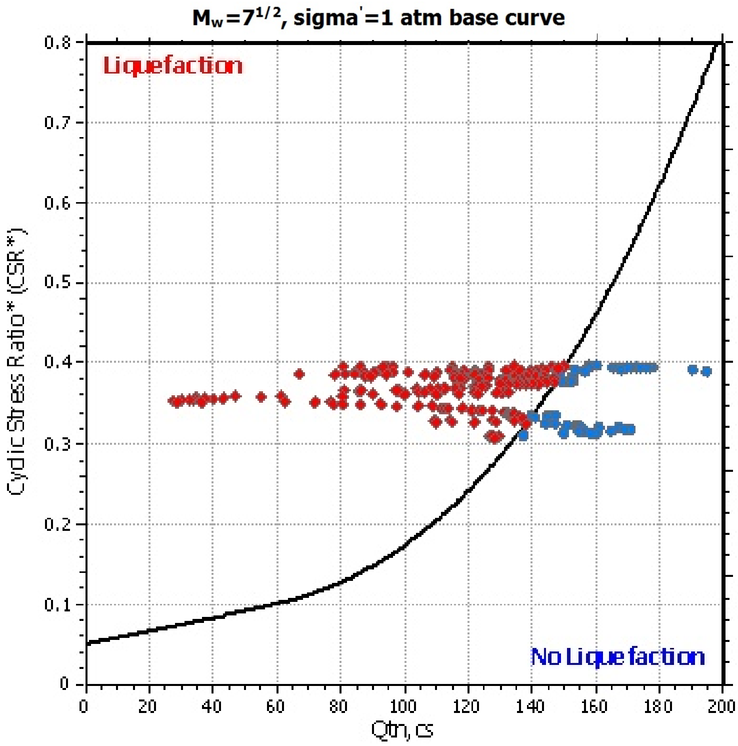
**Project title : Rio Ranch Marketplace** **Location :**

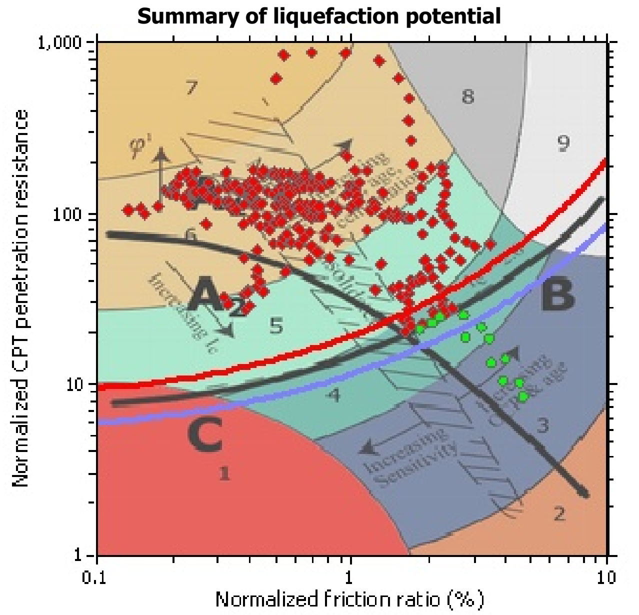
**CPT file : CPT-2**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | G.W.T. (in-situ): | 16.00 ft | Use fill: | No | Clay like behavior applied: | All soils |
| Fines correction method: | Robertson (2009) | G.W.T. (earthq.): | 15.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.50 | Ic cut-off value: | 2.60 | Trans. detect. applied: | No | MSF method: | Method based |
| Peak ground acceleration: | 0.63 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | No | | |

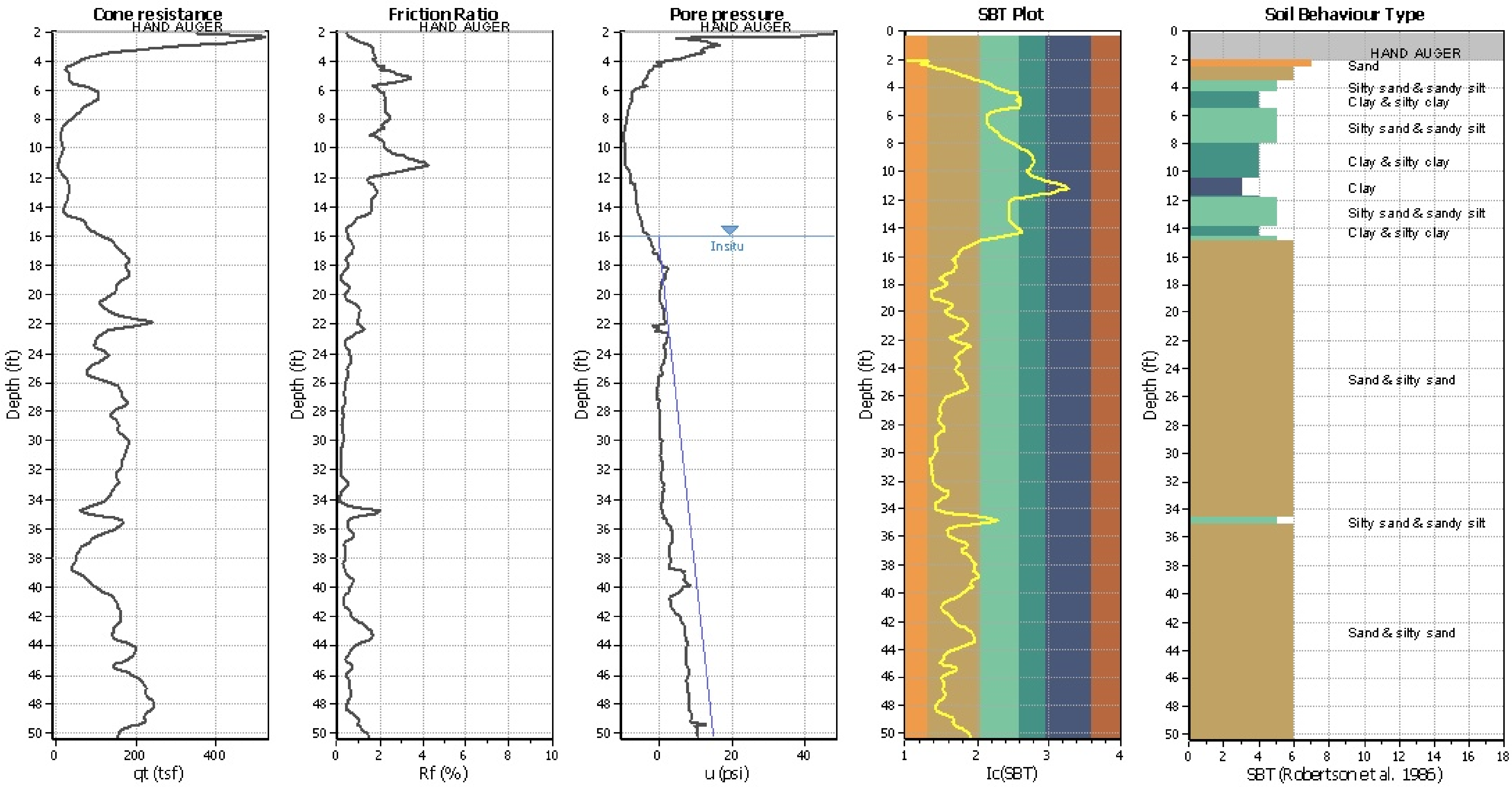



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

## CPT basic interpretation plo



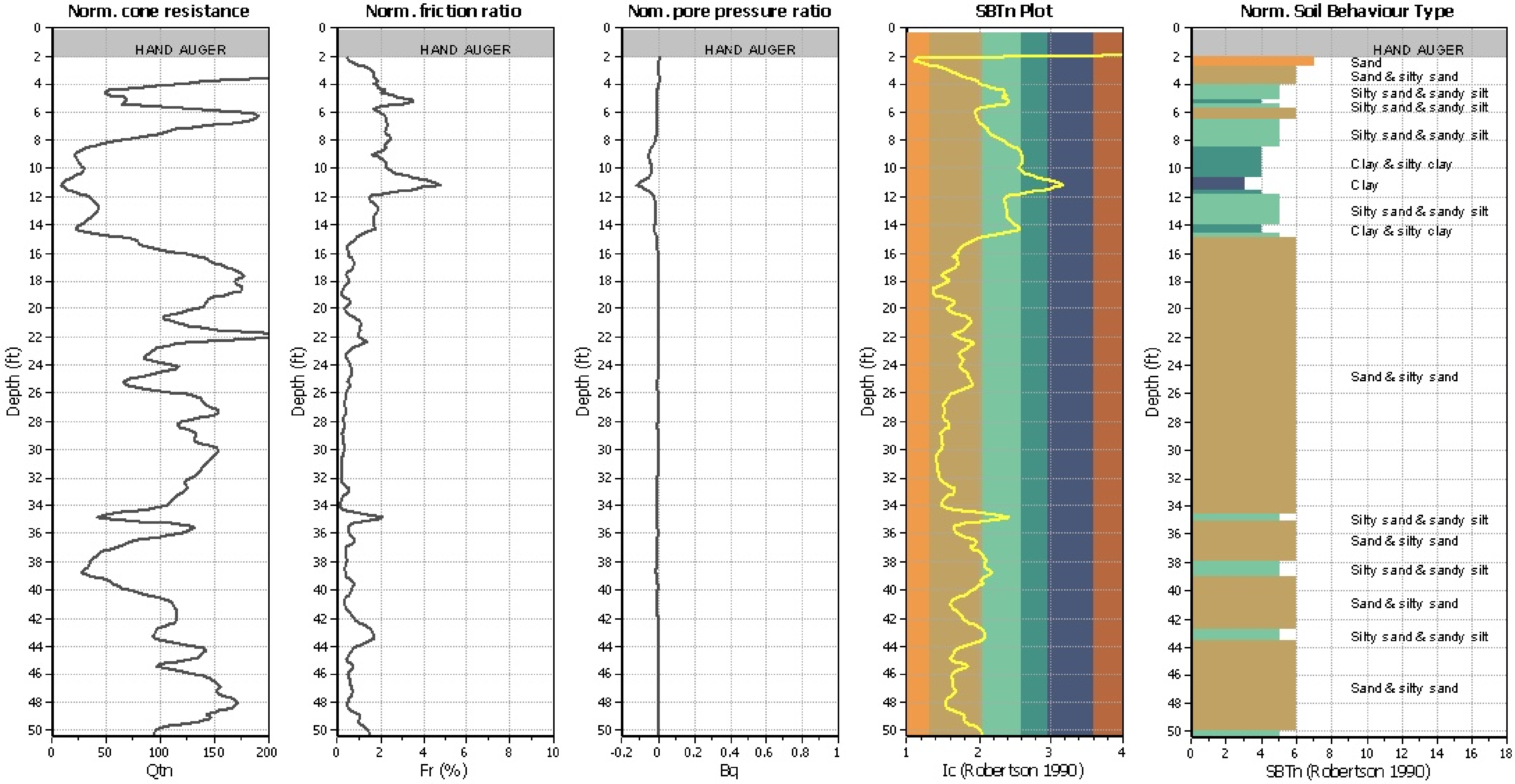

**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 15.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | No |
| Earthquake magnitude $M_w$: | 7.50 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.63 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 16.00 ft | Fill height: | N/A | Limit depth: | N/A |

**SBT legend**

| | | |
|---|---|---|
| 1. Sensitive fine grained | 4. Clayey silt to silty | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

## CPT basic interpretation plots (normaliz



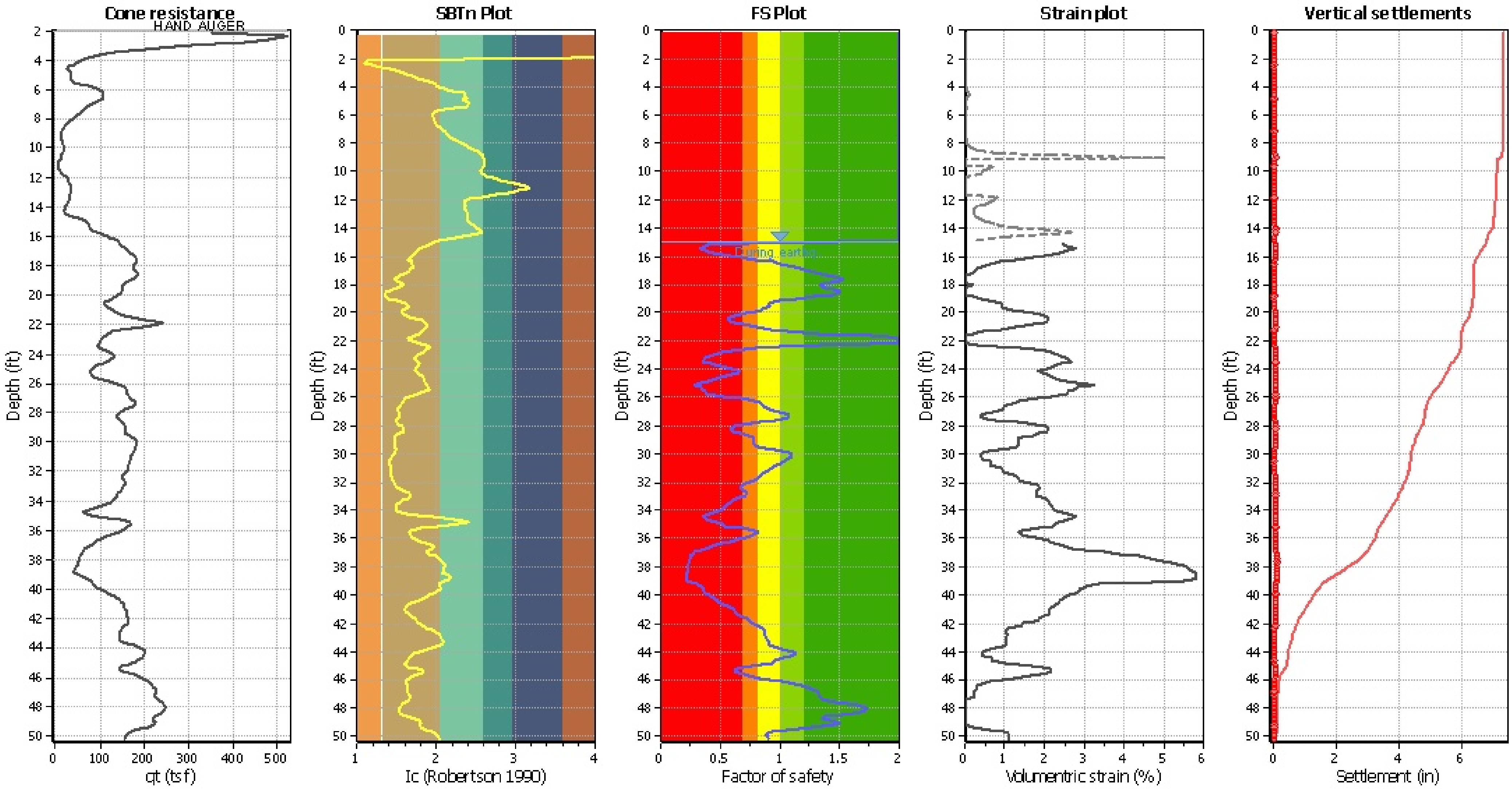

**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 15.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | No |
| Earthquake magnitude $M_w$: | 7.50 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.63 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 16.00 ft | Fill height: | N/A | Limit depth: | N/A |

**SBTn legend**

| | | |
|---|---|---|
| 1. Sensitive fine grained | 4. Clayey silt to silty | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

## Estimation of post-earthquake settlements




**Abbreviations**

| | |
|---|---|
| $q_t$: | Total cone resistance (cone resistance $q_c$ corrected for pore water effects) |
| $I_c$: | Soil Behaviour Type Index |
| FS: | Calculated Factor of Safety against liquefaction |
| Volumentric strain: | Post-liquefaction volumentric strain |

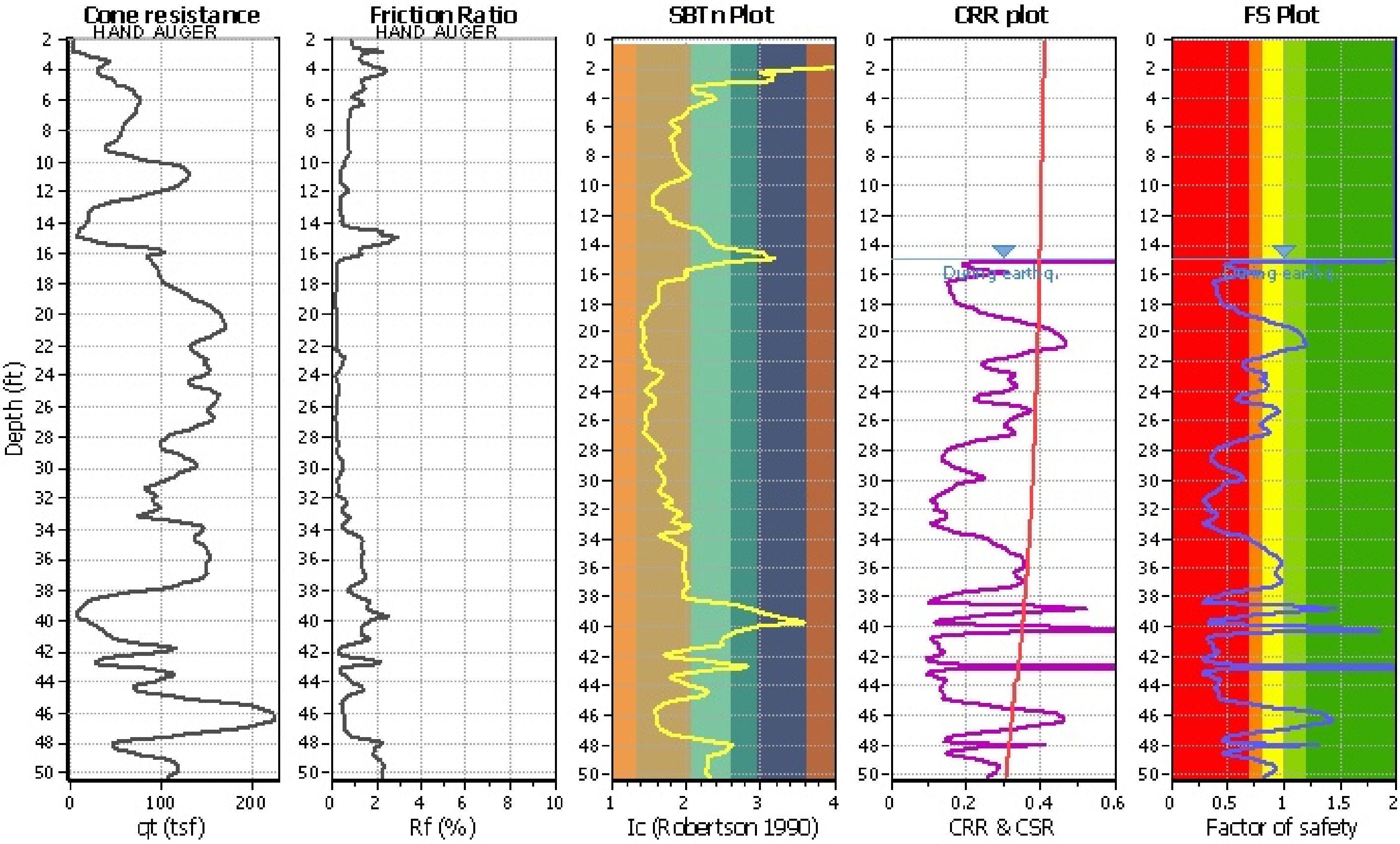
## LIQUEFACTION ANALYSIS REPORT

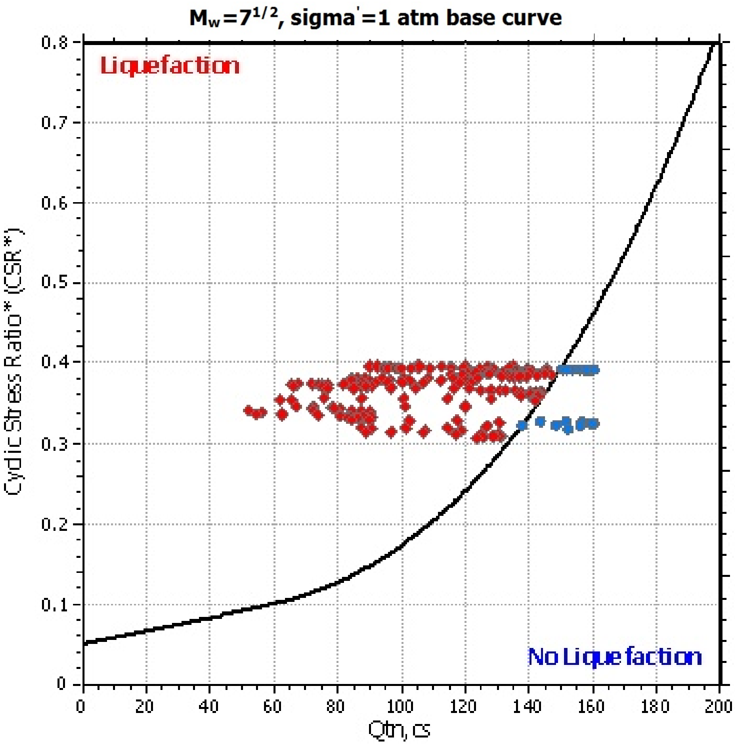
**Project title : Rio Ranch Marketplace** **Location :**

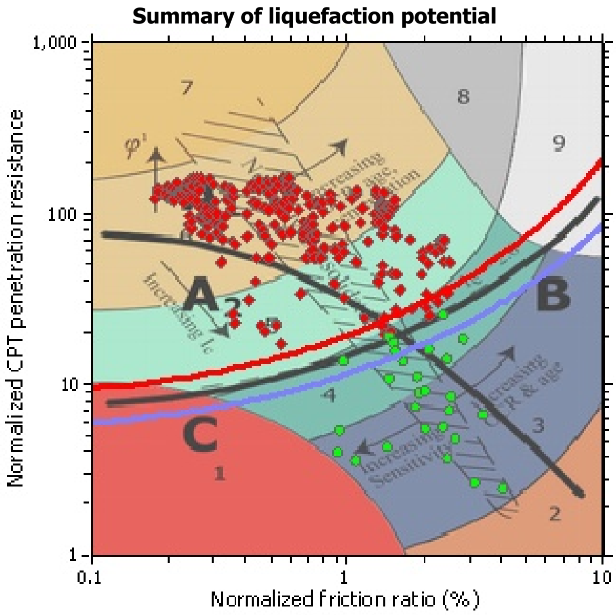
**CPT file : CPT-3**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | G.W.T. (in-situ): | 16.00 ft | Use fill: | No | Clay like behavior applied: | All soils |
| Fines correction method: | Robertson (2009) | G.W.T. (earthq.): | 15.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 7.50 | Ic cut-off value: | 2.60 | Trans. detect. applied: | No | MSF method: | Method based |
| Peak ground acceleration: | 0.63 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | No | | |

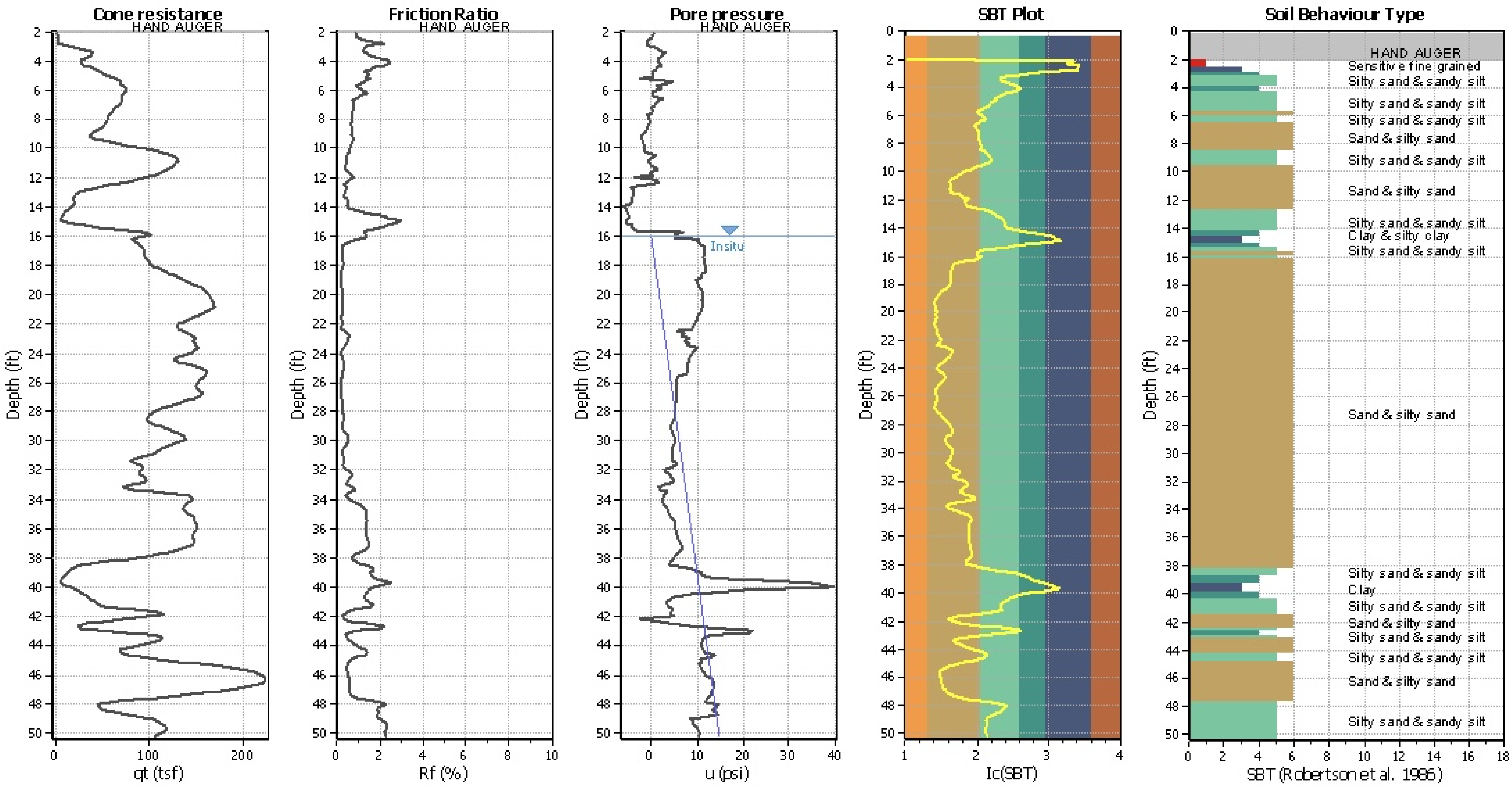



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

## CPT basic interpretation plo



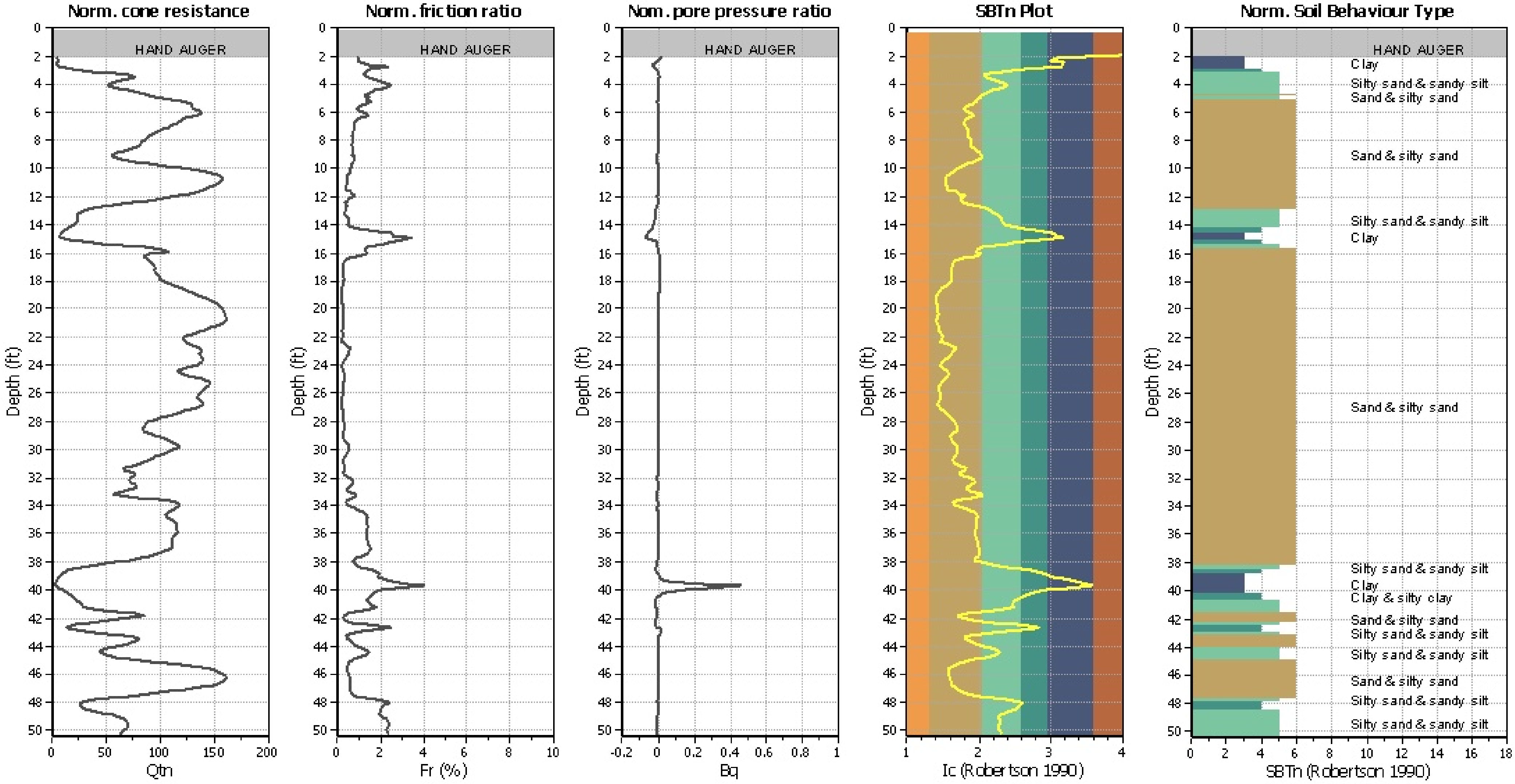

**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 15.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | No |
| Earthquake magnitude $M_w$: | 7.50 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.63 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 16.00 ft | Fill height: | N/A | Limit depth: | N/A |

**SBT legend**

| | | |
|---|---|---|
| 1. Sensitive fine grained | 4. Clayey silt to silty | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

## CPT basic interpretation plots (normaliz



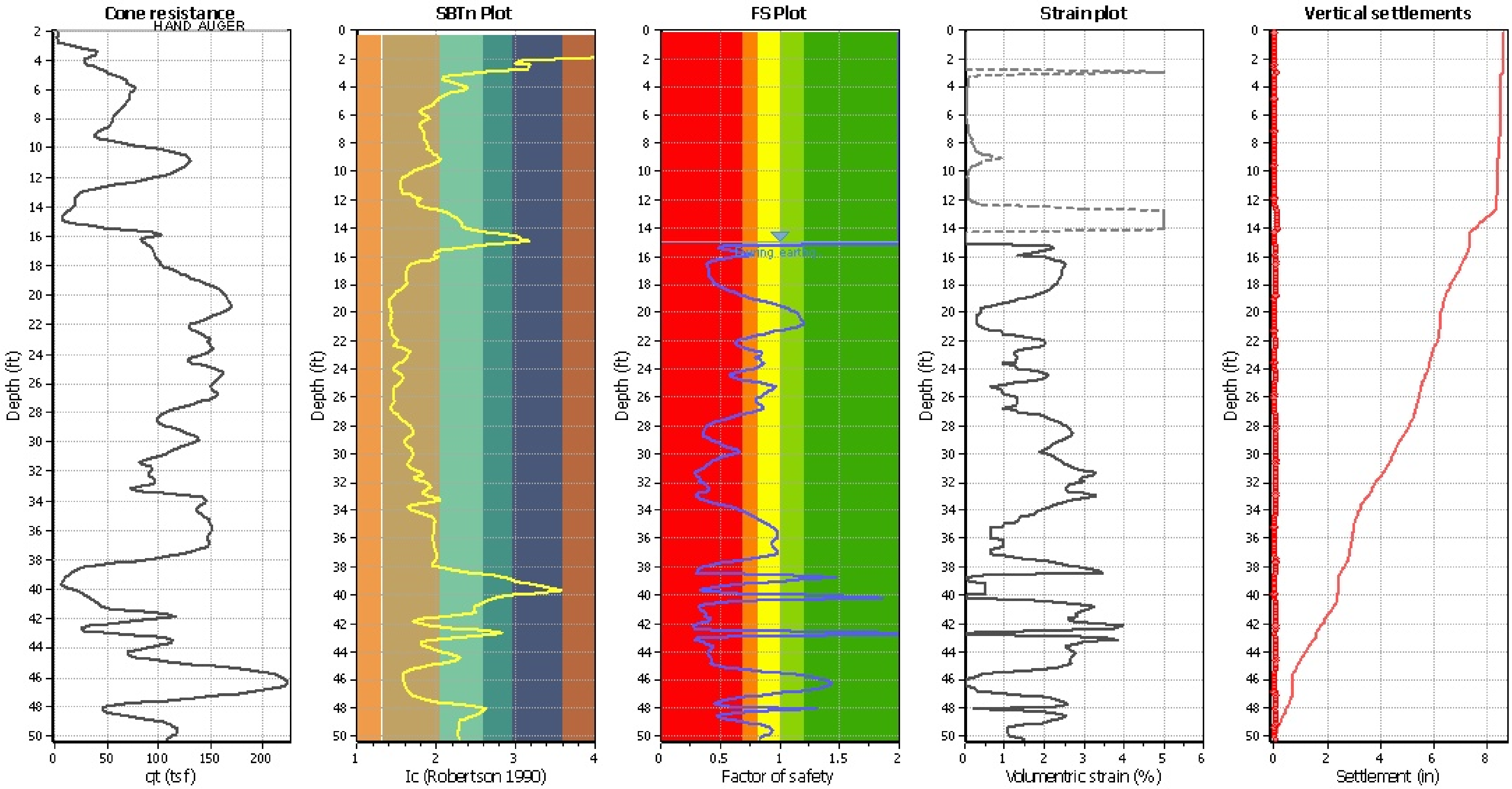

### Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | Robertson (2009) | Depth to water table (erthq.): | 15.00 ft | Fill weight: | N/A |
| Fines correction method: | Robertson (2009) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | No |
| Earthquake magnitude $M_w$: | 7.50 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | All soils |
| Peak ground acceleration: | 0.63 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 16.00 ft | Fill height: | N/A | Limit depth: | N/A |

### SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

## Estimation of post-earthquake settlements




**Abbreviations**

| | |
|---|---|
| $q_t$: | Total cone resistance (cone resistance $q_c$ corrected for pore water effects) |
| $I_c$: | Soil Behaviour Type Index |
| FS: | Calculated Factor of Safety against liquefaction |
| Volumentric strain: | Post-liquefaction volumentric strain |

**Project title : Rio Ranch Marketplace**

**Location :**

## Overall vertical settlements report