APPENDIX K
GEOTECHNICAL INVESTIGATION REPORT – WESTERN PARCEL

CHRISTIAN WHEELER
ENGINEERING

REPORT OF GEOTECHNICAL INVESTIGATION

NAGATA PROPERTY

4617 NORTH RIVER ROAD

OCEANSIDE, CALIFORNIA

PREPARED FOR

NAGATA BROTHERS FARMS, INC.

PO BOX 220

OCEANSIDE, CALIFORNIA 92068

PREPARED BY

CHRISTIAN WHEELER ENGINEERING

3980 HOME AVENUE

SAN DIEGO, CALIFORNIA 92105

CHRISTIAN WHEELER
ENGINEERING

December 2, 2015

Nagata Brothers Farms, Inc.
PO Box 220
Oceanside, California 92068
Attention: Neil Nagata

CWE 2140692.01

Subject: **Report of Geotechnical Investigation**
Nagata Property, 4617 North River Road, Oceanside, California

Ladies and Gentlemen:

In accordance with our Proposal dated November 20, 2014, we have completed a preliminary geotechnical investigation for the subject project. We are presenting herein our findings and recommendations.

In general, we found the subject property suitable for the proposed construction, provided the recommendations provided herein are followed. Based on the results of our investigation, the most significant geotechnical conditions to affect the proposed construction are the presence of deep alluvial soils that are potentially liquefiable under earthquake loads and surficial soils that are potential compressible under additional static loads. The liquefaction potential will require mitigation in the form of ground improvement below the planned buildings while the surficial soils will require overexcavation and recompaction.

If you have any questions after reviewing this report, please do not hesitate to contact our office. This opportunity to be of professional service is sincerely appreciated.

Respectfully submitted,
CHRISTIAN WHEELER ENGINEERING

Shawn Caya, R.G.E. #2748

David. Russell, C.E.G. #2215

Distribution: (1) Neil Nagata via email

REGISTERED PROFESSIONAL ENGINEER
SHAWN C. CAYA
GE2748
EXP. 6/30/2016
GEOTECHNICAL
STATE OF CALIFORNIA

REGISTERED GEOLOGIST
DAVID R. RUSSELL
No. 2215
CERTIFIED
ENGINEERING
GEOLOGIST
Exp. 09-15
STATE OF CALIFORNIA

3980 Home Avenue ✦ San Diego, CA 92105 ✦ 619-550-1700 ✦ FAX 619-550-1701

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

PAGE

TABLES

FIGURES

PLATES

APPENDICES

CHRISTIAN WHEELER
ENGINEERING

REPORT OF GEOTECHNICAL INVESTIGATION

NAGATA PROPERTY
4617 NORTH RIVER ROAD
OCEANSIDE, CALIFORNIA

INTRODUCTION AND PROJECT DESCRIPTION

This report presents the results of a geotechnical investigation performed for a proposed residential development to be constructed at 4617 North River Road, in the city of Oceanside, California. Figure Number 1, on the following page, presents a vicinity map showing the location of the project.

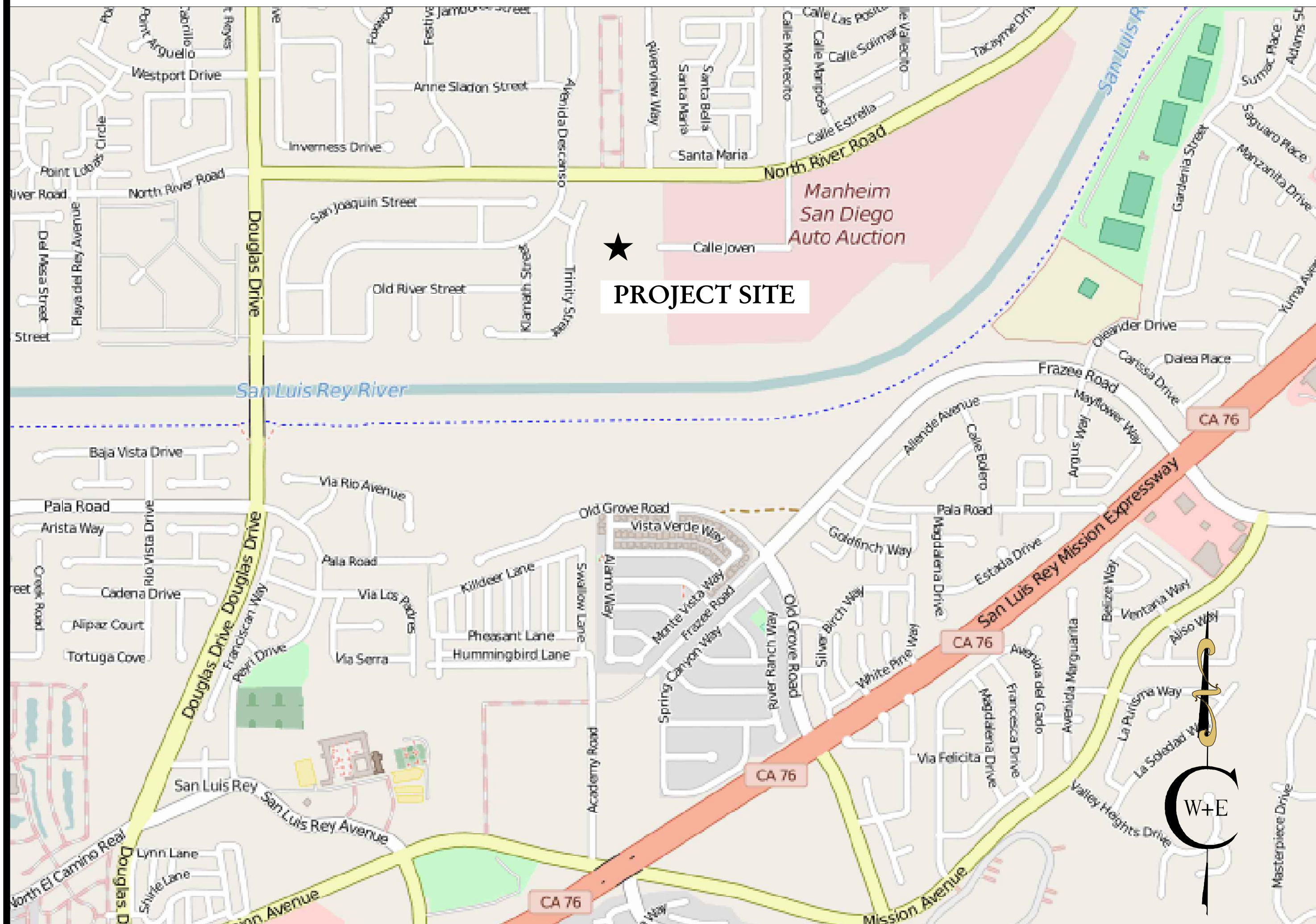
Although no plans are available, we understand that it is proposed to raze the existing structures and other improvements on the land and redevelop the property with multiple residential units. The residential units may consist of single-family homes, townhomes, and or multi-family dwellings. The buildings are expected to be two- and/or three-story, wood frame structures with conventional foundations and slab-on-grade floors. Additional improvements will include typical pavements, utilities, and other light miscellaneous exterior improvements.

To assist with the preparation of this report, we have reviewed the information for a previous geotechnical investigation performed by our firm at the project site. The subsurface exploration information and laboratory test results from that previous investigation are included in this report as Appendix B.

This report has been prepared for the exclusive use of Nagata Brothers Farms, Inc. and its consultants for specific application to the project described herein. Should the project be modified, the conclusions and recommendations presented in this report should be reviewed by Christian Wheeler Engineering for conformance with our recommendations and to determine whether any additional subsurface investigation, laboratory testing and/or recommendations are necessary. Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, expressed or implied.

SITE VICINITY

© OpenStreetMap contributors



NAGATA PROPERTY 4617 NORTH RIVER ROAD OCEANSIDE, CALIFORNIA		CHRISTIAN WHEELER ENGINEERING
DATE: DECEMBER 2015	JOB NO.: 2140692	
BY: SRD	FIGURE NO.: 1	

PROJECT SCOPE

Our preliminary geotechnical investigation consisted of surface reconnaissance, subsurface exploration, review of previous subsurface explorations and laboratory testing by our firm, analysis of the field data, and review of relevant geologic literature. Our scope of service did not include assessment of hazardous substance contamination, recommendations to prevent floor slab moisture intrusion or the formation of mold within the structure, or any other services not specifically described in the scope of services presented below. More specifically, our intent was to provide the services listed below.

- Explore the subsurface conditions of the site to the depths influenced by the proposed construction.
- Evaluate, by laboratory tests and our past experience with similar soil types, the engineering properties of the various soil strata that may influence the proposed construction, including bearing capacities, expansive characteristics and settlement potential.
- Describe the general geology at the site, including possible geologic hazards that could have an effect on the proposed construction, and provide the seismic design parameters as required by the 2013 edition of the California Building Code.
- Address potential construction difficulties that may be encountered due to soil conditions, groundwater or geologic hazards, and provide recommendations concerning these problems.
- Address the potential for soil liquefaction at the site.
- Provide site preparation and grading recommendations for the anticipated work.
- Provide foundation recommendations for the type of construction anticipated and develop soil engineering design criteria for the recommended foundation designs.
- Provide design parameters for restrained and unrestrained retaining walls.
- Provide preliminary pavement sections.
- Prepare this report, which includes, in addition to our conclusions and recommendations, a plot plan showing the areal extent of the geological units and the locations of our exploratory borings, exploration logs, and a summary of the laboratory test results.

Although tests were previously performed to categorize the potential corrosivity of the on-site the soils that may be in contact with below grade structures, it should be understood Christian Wheeler Engineering does not practice corrosion engineering. If such an analysis is considered necessary, we recommend that the client retain an engineering firm that specializes in this field to consult with them on this matter. The results of these tests should only be used as a guideline to determine if additional testing and analysis is necessary.

FINDINGS

SITE DESCRIPTION

The subject site is a rectangular, 16.6-acre parcel of land located about 3,000 feet east of Douglas Drive, between North River Road, which bounds the site on the north, and the San Luis Rey River, which bounds the site on the south. The property is identified by the address of 4617 North River Road and by Assessor's Parcel Number 157-060-17. The site is predominantly undeveloped, but does support a single-family residence, an old warehouse, a packing shed, small out buildings, and an abundant amount of old farm equipment and miscellaneous materials and machinery. The property is relatively level and is about 20 feet above the improved channel for the San Luis Rey River. A grouted rip-rap embankment separates the property from the channel bottom.

GENERAL GEOLOGY AND SUBSURFACE CONDITIONS

GEOLOGIC SETTING AND SOIL DESCRIPTION: The subject site is located within the Coastal Physiographic Province of San Diego County. Based on our subsurface explorations and analysis of readily available, pertinent geologic literature, the areas of the site investigated were found to be underlain by artificial fill, alluvium, and old paralic deposits. Each of these units is discussed below in order of increasing age:

> **ARTIFICIAL FILL (Qaf):** Seven of our nine borings encountered artificial fill material that was most likely placed during a leveling operation for the farmland (B-1, B-2, B-3, B-6, B-7, B-8 and B-9). The fill material was noted to range from 1 foot to 3½ feet in thickness and consisted of dry to damp, relatively loose, silty sand (SM). Some of the fill material contained debris and trash that will need to be removed by hand-picking during the grading operation. The fill material is expected to have a "low" to "very low" Expansion Index, low strength parameters, and a moderate settlement potential. Based on the relatively loose condition of the existing fill, it will need to be removed and replaced as properly compacted fill as part of the remedial grading operation.
>
> **TOPSOIL:** An approximately 1- to 2-foot-thick layer of natural topsoil was noted within all but one of our exploratory borings. The topsoil generally consisted of fine-grained, silty sand (SM) that was dry to moist and loose to medium dense in consistency. The topsoil is expected to possess a "low" Expansion Index, low strength parameters, and a moderate settlement potential. Based on the relatively loose condition of the existing topsoil, it will need to be removed and replaced as properly compacted fill as part of the remedial grading operation.

ALLUVIUM (Qal): Quaternary-age alluvial deposits were encountered below the topsoil and/or fill layers within each of exploratory borings and cone penetration tests. The alluvial materials were encountered at depths of 2 to 4 feet below the existing site grades and were noted to extend to depths greater than the maximum explored depth of 60 feet below existing site grades. The alluvial deposits generally consisted of poorly-graded sand (SP) and silty sand – poorly-graded sand (SM-SP) with lesser amounts of silty sand (SM), silty sand – sandy silt (SM-ML), and poorly-graded sand (SP). The exposed alluvium was typically damp to moist to depths of about 15 feet below the existing grades, very moist to wet within a few feet of the water table, and saturated below the water table (described in the following section).

GROUNDWATER: Groundwater was encountered in our previous borings (drilled June 14, 2005) at depths ranging from about 18 to 24 feet below the existing site grades. Pore Pressure Dissipation tests performed during the cone penetration testing measured the water table at depths ranging from 23 to 26½ feet below the existing grades. It should be noted that variations in subsurface water (including perched water zones and seepage) may result from fluctuations in the ground surface topography, subsurface stratification, precipitation, irrigation, and other factors that may not have been evident at the time of the investigation. It should also be recognized that minor groundwater seepage problems might occur after development of a site even where none were present before development. These are usually minor phenomena and are often the result of an alteration in drainage patterns and/or an increase in irrigation water. It is further our opinion that these problems can be most effectively corrected on an individual basis if and when they occur.

TECTONIC SETTING: No active or potentially active faults are known to traverse the subject site. However, it should be noted that much of Southern California, including the San Diego County area, is characterized by a series of Quaternary-age fault zones that consist of several individual, en echelon faults that generally strike in a northerly to northwesterly direction. Some of these fault zones (and the individual faults within the zone) are classified as "active" according to the criteria of the California Division of Mines and Geology. Active fault zones are those that have shown conclusive evidence of faulting during the Holocene Epoch (the most recent 11,000 years). The Division of Mines and Geology used the term "potentially active" on Earthquake Fault Zone maps until 1988 to refer to all Quaternary-age (last 1.6 million years) faults for the purpose of evaluation for possible zonation in accordance with the Alquist-Priolo Earthquake Fault Zoning Act and identified all Quaternary-age faults as "potentially active" except for certain faults that were presumed to be inactive based on direct geologic evidence of inactivity during all of Holocene time or longer. Some faults considered to be "potentially active" would be considered to be "active" but lack specific criteria used by the State Geologist, such as *sufficiently active* and *well-defined*. Faults older than Quaternary-age are not specifically defined in Special Publication 42, Fault Rupture Hazard Zones in California, published by the

California Division of Mines and Geology. However, it is generally accepted that faults showing no movement during the Quaternary period may be considered to be "inactive".

A review of available geologic maps indicates that a portion of the Newport-Inglewood Fault Zone is located approximately 14 kilometers west of the site. In addition to the Newport-Inglewood Fault Zone, other active fault zones in the region that could possibly affect the site include the Palos Verdes Fault Zone to the northwest, the Rose Canyon and Coronado Bank Fault Zones to the southwest, and the Elsinore, Earthquake Valley, and San Jacinto Fault Zones to the east. The following Table I presents the proximal faults that are anticipated to most significantly contribute to the ground-motion hazard at the site.

TABLE I: PROXIMAL FAULT ZONES

Fault Zone	Distance
Newport-Inglewood	14 km
Rose Canyon	16 km
Elsinore-Julian	31 km
Coronado Bank	42 km
Palos Verdes	59 km
San Jacinto (Anza)	66 km
Earthquake Valley	68 km

GEOLOGIC HAZARDS

SEISMIC HAZARD: A likely geologic hazard to affect the site is ground shaking as a result of movement along one of the major active fault zones mentioned in the "Tectonic Setting" section of this report. Per Chapter 16 of the 2013 California Building Code (CBC), the Risk-Targeted Maximum Considered Earthquake (MCE_R) ground acceleration is that which results in the largest maximum response to horizontal ground motions with adjustments for a targeted risk of structural collapse equal to one percent in 50 years. Figures 1613..3.1(1) and 1613.3.1(2) of the CBC present MCE_R accelerations for short (0.2 sec.) and long (1.0 sec.) periods, respectively, based on a soil Site Class B (CBC 1613.3.2) and a structural damping of five percent. For the subject site, correlation with estimated blow counts indicates that the upper 100 feet of geologic subgrade can be characterized as Site Class D. In this case, the mapped MCE_R accelerations are modified using the Site Coefficients presented in Tables 1613.3.3(1) and (2). The modified MCE spectral accelerations are then multiplied by two-thirds in order to obtain the design spectral accelerations. These seismic design parameters for the subject site (33.2377°, -117.3095°), based on Chapter 16 of the CBC, are presented in Table II below.

TABLE II: CBC 2013 EDITION – SEISMIC DESIGN PARAMETERS

CBC – Chapter 16 Section	Seismic Design Parameter	Recommended Value
Section 1613.3.2	Soil Site Class	D
Figure 1613.3.1 (1)	MCE_R Acceleration for Short Periods (0.2 sec), S_s	1.058 g
Figure 1613.3.1 (2)	MCE_R Acceleration for 1.0 Sec Periods (1.0 sec), S_1	0.413 g
Table 1613.3.3 (1)	Site Coefficient, F_a	1.077
Table 1613.3.3 (2)	Site Coefficient, F_v	1.587
Section 1613.3.3	S_{MS} = MCE_R Spectral Response at 0.2 sec. = $(S_s)(F_a)$	1.140 g
Section 1613.3.3	S_{M1} = MCE_R Spectral Response at 1.0 sec. = $(S_1)(F_v)$	0.656 g
Section 1613.3.4	S_{DS} = Design Spectral Response at 0.2 sec. = $2/3(S_{MS})$	0.760 g
Section 1613.3.4	S_{D1} = Design Spectral Response at 1.0 sec. = $2/3(S_{M1})$	0.437 g
Section 1803.2.12	PGA_M per Section 11.8.3 of ASCE 7	0.44 g

It can be noted that sites underlain by liquefaction-susceptible soils should be designated as site class F, requiring a dynamic site response analysis. However, as discussed in Section 20.3.1 of ASCE Standard 7 "Minimum Design Loads for Buildings and Other Structures", for structures having fundamental periods of vibration equal to or less than 0.5 second, it is not required to perform a dynamic site response analysis. We expect that the proposed structure will have a fundamental period less than 0.5 second and can therefore be designed using soil Site Class D as described previously.

LANDSLIDE POTENTIAL AND SLOPE STABILITY: As part of this investigation we reviewed the publication, "Landslide Hazards in the Southern Part of the San Diego Metropolitan Area" by Tan, 1995. This reference is a comprehensive study that classifies San Diego County into areas of relative landslide susceptibility. According to this publication, the site is mapped within Relative Landslide Susceptibility Area 2, which is considered to be "marginally susceptible" to landsliding. Based on our findings, it is our professional opinion that the potential for slope failures within the site is very low.

FLOODING: As delineated on Flood Insurance Rate Map (FIRM) 06073C1611G prepared by the Federal Emergency Management Agency, the site is located within Zone X, which has a 0.2% annual chance to be affected by a flood hazard.

TSUNAMIS: Tsunamis are great sea waves produced by submarine earthquakes or volcanic eruptions. The risk potential for damage to the subject site caused by tsunamis is very low.

SEICHES: Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays or reservoirs. The risk potential for damage to the subject site caused by seiches is very low.

LIQUEFACTION

GENERAL: The subject site is in an area considered susceptible to liquefaction. In order to be subject to liquefaction, three conditions must be present: loose sandy or cohesionless silty deposits, shallow groundwater, and earthquake shaking of sufficient magnitude and duration. Based on our site-specific study, it appears that shallow groundwater is present at the site and strong earthquake shaking may affect the site. Additionally, as described in the Geologic Setting and Soil Description section of this report above, the materials below the shallow water table in the project area consist of Quaternary-age alluvium that contains layers of sand, silty sand, and low to medium plasticity silts that are expected to have soil properties conducive to liquefaction.

It should be noted that the following discussion is in no way a guarantee that the analysis will accurately predict the liquefaction potential at the site. The analysis provides general information only on the site liquefaction potential. It should be noted that many of the parameters used in liquefaction evaluations are subjective and open to interpretation, and that much is yet unknown about both the seismicity of the San Diego area and the phenomenon of liquefaction.

DESCRIPTION OF ANALYSIS: Our analysis was performed using the Cliq (version 1.7) software developed by Geologismiki, in which the cone penetration test results were input and evaluated in accordance with the procedure recommended by the National Center For Earthquake Engineering Research (NCEER, 1998). Our analyses were limited to the upper 50 feet of soils as liquefaction below that depth is not considered to have a significant effect on surface improvements.

EARTHQUAKE PARAMETERS: As permitted in Section 1803.5.12 of the California Building Code, our calculations were performed using a peak ground acceleration (PGA_M = 0.44g) as determined using the procedures set forth in Section 11.8.3 of ASCE 7-10. We have also performed a seismic hazard deaggregation using the interactive program available on the U. S. Geological Survey website. Within the USGS program, the site coordinates were entered and a deaggregation was performed based on the peak ground acceleration with one percent probability of exceedance in 50 years (0.45g) for soil with Vs^{30} = 200 m/s (Soil Site Class D). For the subject site, this yielded a mean earthquake magnitude of 6.4, which was used in our analysis.

POTENTIAL FOR LIQUEFACTION: Using the parameters described above, the results of our liquefaction analyses indicate that much of the saturated sandy and silty portions of the alluvium within the upper approximately 50 feet possess factors-of-safety against soil liquefaction of less than 1.3 and are therefore considered liquefiable.

POST LIQUEFACTION RECONSOLIDATION SETTLEMENT: The potential amount of total vertical settlement due to reconsolidation of the liquefied soils was estimated within the Cliq software using the methods presented by Zhang et al, 2002. The estimated settlements for our six cone penetration tests ranged from approximately 6½ to 10½ inches and averaged 8½ inches. It can be noted that, for sites with relatively small lateral displacement (i.e. less than one foot), predicted settlements are typically within a factor of two relative to those observed (Seed et al, 2003).

In terms of differential settlement, CGS Special Publication 117 notes that considerable difficulty exists in trying to "reliably estimate" the amount of differential settlement at a site caused by soil liquefaction. As such, a conservative estimate of differential settlement at any given site can be assumed to be two-thirds of the total liquefaction-induced settlement (CGS, 2008). Using this criterion, without any deep ground modification procedures, the subject project area may be assumed to be subject to approximately 5½ inches of liquefaction-induced, differential settlement.

LATERAL SPREADING: Lateral ground spreading can occur when viscous liquefied soils flow downslope, usually towards a river channel or shoreline. As presented in the referenced Conetec literature (2002), which was based on the work of Robertson and Wride (1998) and Zhang, Robertson and Brachman (2002) and which describes the use of the CPT method to estimate cyclic resistance ratios and liquefaction-induced soil deformation:

> "The equivalent clean sand normalized tip resistance $(Q_{tn})_{cs}$ can also be used as an estimate for possible flow liquefaction (Yoshimine et al., 1999). Based on the soil behavior index, the normalized tip resistance can be adjusted to account for the influence of fines (Robertson and Wride, 1998). The resulting value is the clean sand equivalent normalized tip resistance $(Q_{tn})_{cs}$. Yoshimine et al., (1999) showed that soils with a minimum undrained shear strength less than 0.1 had a tendency to be very brittle. They also showed that soils with an equivalent clean sand normalized CPT tip resistance of 50 had an undrained shear strength ratio in simple shear loading of around 0.1. Hence, Yoshimine et al. (1999) suggested that soils with an equivalent clean sand normalized CPT tip resistance less than 50 could be strain softening in simple shear loading and could also be very brittle. For flow liquefaction failure (i.e. flow slide) to occur requires a trigger event and a sufficient volume of strain softening soils where the resulting minimum undrained shear strength is less than the insitu static shear stress. The profiles of $(Q_{tn})_{cs}$ should be reviewed carefully to identify either large volumes or continuous layers of soils with values less than 50."

Based on this criteria for identifying potentially strain softening soil layers, we have reviewed the clean sand equivalent normalized tip resistance values $(Q_{tn})_{cs}$ of the soil layers encountered in the recently conducted CPT soundings. Only minor volumes and discontinuous layers of soils with $(Q_{tn})_{cs}$ values of less than 50 were noted in our review.

Factors such as the absence of significant volumes of potentially stain softening ($(Q_{tn})_{cs} <50$) liquefiable soils beneath the site and the relatively gentle hydraulic gradient of the water table across the area are considered favorable with regards to limiting potential lateral spreading. However, based on the areal extent of materials around the area of the project site that are anticipated to be liquefiable and the location of the site in proximity to the San Luis Rey River valley, lateral earth displacements on the order a few inches across the general area of the proposed development could be expected in the event of major, proximal seismic event that triggers soil liquefaction. Measures to mitigate the potential for lateral spreading across the area of the subject site would likely require deep ground modification and improvement techniques not just at the subject site but rather across the majority of the San Luis Rey River area. Given the unlikelihood of such a large-scale ground improvement project across the region, our foundation recommendations contained herein have been given to provide a life-safety performance level for the proposed structures. Our recommendations do not, however, preclude the possibility of structural damage and horizontal displacement of the proposed structures and improvements occurring, even to the extent that they become unusable or uninhabitable, as a result of a major seismic event.

CONCLUSIONS

In general, our findings indicate that, from a geologic and geotechnical perspective, the subject property is suitable for the proposed residential development provided the recommendations presented herein are implemented. The main geotechnical and geologic conditions that will impact the proposed construction are the presence of deep alluvial soils that are subject to liquefaction during a major seismic event and surficial topsoil and fill soils that are potentially compressible under additional loads.

We have estimated that the site may be subject to post-liquefaction reconsolidation settlement on the order of 10 inches in the event of a major, proximal seismic event. Though alternative remediation options exist, it is our opinion that a successful ground improvement/ reinforcement program can be the most efficient way to simultaneously reduce the liquefaction potential and improve the bearing characteristics of the existing soils. Stone columns are the considered the most appropriate ground improvement method at this time. Stone columns, also known as Vibro Replacement, consist of columns of crushed aggregate that are installed in a grid pattern using a vibrating downhole probe to densify the surrounding cohesionless soils while filling the void with the crushed aggregate to provide stiff reinforcing elements within the looser matrix soils. Given their unique application, stone columns are typically designed and constructed by a specialty contractor that will use the subsurface data presented in this report and the structural building requirements to prepare the design. Good engineering practice requires that where the evaluation indicates that liquefaction is likely (or reasonably possible), the hazards that might reasonably be caused by liquefaction, that could result in the collapse of a structure and/or loss of life be mitigated. In our opinion, this level of life safety can be achieved by reducing the estimated post-liquefaction reconsolidation settlement to 4 inches or less. The client should realize that the site preparation and foundation recommendations presented herein are intended to provide this level of life safety. These recommendations, however, will not necessarily prevent the structures from sustaining damage, even to the extent that they may become uninhabitable. They will also not prevent damage to streets or other surface improvements as well as underground utilities.

In addition to the ground improvement for liquefaction mitigation, it will also be necessary to perform remedial grading for areas to support new fill and/or settlement-sensitive improvements. In general, this will include overexcavating the existing soils to depths ranging from 3 to 5 feet below the existing grade and replacing the material as properly compacted, structural fill.

RECOMMENDATIONS

GRADING AND EARTHWORK

GENERAL: All grading should conform to the guidelines presented in Appendix J of the California Building Code, the minimum requirements of the City of Oceanside, and the recommended Grading Specifications and Special Provisions attached hereto, except where specifically superseded in the text of this report. Prior to grading, a representative of Christian Wheeler Engineering should be present at the pre-construction meeting to provide additional grading guidelines, if necessary, and to review the earthwork schedule.

OBSERVATION OF GRADING: Continuous observation by the Geotechnical Consultant is essential during the grading operation to confirm conditions anticipated by our investigation, to allow adjustments in design criteria to reflect actual field conditions exposed, and to determine that the grading proceeds in general accordance with the recommendations contained herein.

CLEARING AND GRUBBING: Site grading should begin with the removal of the existing improvements, including foundations, utilities, concrete and all vegetation and miscellaneous debris from the portions of site that will be graded and/or will receive improvements. Existing on-site wells should be abandoned in accordance with

County of San Diego, Department of Environmental Health guidelines. It is unknown if there are underground utility lines within the subject proper. Any abandoned underground pipes found during the grading operation should be removed and the resulting depressions backfilled with uniformly compacted fill material. The materials resulting from clearing and grubbing should be disposed of off-site. **It should be noted that discing of the vegetation into the surficial soils is not an acceptable form of removal, and could result in the requirement that all soil contaminated with vegetation be exported from the site.**

SITE PREPARATION: The following recommendations are based on the assumption that all existing site materials are suitable for reuse on the site and are not considered contaminated or otherwise are unsuitable. As discussed in the "Liquefaction Mitigation – Stone Columns" section of this report, we expect that stone columns will be installed below the planned structures. After stone column installation, site preparation in the building pad areas should consist of overexcavating the existing soils within the upper three feet of the stone columns and replacing them as properly compacted structural fill. This overexcavation should extend horizontally a distance of at least three feet outside the perimeter of the stone column grid.

In the remaining areas of the site, we recommend that the site preparation consist of overexcavating the existing surficial fill soils and replacing them as properly compacted, structural fill. Based on the results of our subsurface explorations, we expect that the required overexcavation depth will typically be about 5 feet below the existing ground surface. Horizontally, we recommend that the overexcavation extend at least five feet outside areas to receive fill and/or settlement-sensitive improvements or to the property line, whichever distance is less.

The Geotechnical Consultant should observe the overexcavation operations and the base of removal areas prior to either filling or the construction of improvements. If soft or otherwise unsuitable soils are exposed at the removal bottom, it might be necessary to perform additional excavation or to stabilize the bottom. Specific recommendations will need to be made on a case-by-case basis.

PROCESSING OF FILL AREAS: Prior to placing any new fill soils or constructing any new improvements in areas that have been cleaned out and approved to receive fill, the exposed soils should be scarified to a depth of 12 inches, moisture-conditioned, and compacted to at least 90 percent relative compaction. If soft, pumping, or otherwise unsuitable soils are exposed at the removal bottom that cannot be properly compacted, it will be necessary to stabilize the bottom soils prior to placing structural fill.

EXCAVATION CHRACTERISTICS: Based on our exploratory excavations, the subsurface materials at the site appear to be excavatable to the anticipated excavation depths with conventional heavy-duty earthmoving equipment in good operating condition. Significant caving of the exploratory excavations was not encountered at the time of our subsurface explorations. However, due to the locally loose condition of the existing shallow materials encountered in our exploratory excavations, it should be expected that excavations in the fill and alluvial materials could experience localized caving and sloughing. Additionally, soft or spongy soils may be encountered that will necessitate lightweight equipment and/or top-loading with an excavator.

IMPORTED FILL MATERIAL: Soils to be imported to the site should be evaluated and approved by the Geotechnical Consultant prior to being imported. At least five working days-notice of a potential import source should be given to the Geotechnical Consultant so that appropriate testing can be accomplished. The type of material considered most desirable for import is granular material containing some silt or clay binder, which has an Expansion Index of less than 50. Less than 25 percent of the material should be larger than the Standard #4 sieve, and less than 25 percent finer than the Standard # 200 sieve. Soils not meeting there criteria should not be used for structural fill or backfill.

COMPACTION AND METHOD OF FILLING: All structural fill and backfill material placed at the site should be compacted to a relative compaction of at least 90 percent of maximum dry density as determined by ASTM Laboratory Test D1557. Fills should be placed at a moisture content that is two to four percent above optimum moisture content, in lifts six to eight inches thick, with each lift compacted by mechanical means. Fills should consist of approved earth material, free of trash or debris, roots, vegetation, or other materials determined to be unsuitable by our soil technicians or project geologist. Fill material should be free of rocks or lumps of soil in excess of twelve inches in maximum dimension; however, this should be reduced to six inches within four feet of finish grade.

All utility trench backfill should be compacted to a minimum of 90 percent of its maximum dry density. The upper twelve inches of subgrade beneath paved areas should be compacted to 95 percent of the materials maximum dry density. This compaction should be obtained by the paving contractor just prior to placing the aggregate base material and should not be part of the mass grading requirements or operation.

TEMPORARY CUT SLOPES: The contractor is solely responsible for designing and constructing stable, temporary excavations and will need to shore, slope, or bench the sides of trench excavations as required to maintain the stability of the excavation sides. The contractor's "competent person", as defined in the OSHA Construction Standards for Excavations, 29 CFR, Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety process. We anticipate that the existing on-site soils will consist of Type C material. Our firm should be contacted to observe all temporary cut slopes during grading to ascertain that no unforeseen adverse conditions exist. No surcharge loads such as foundation loads, or soil or equipment stockpiles, vehicles, etc. should be allowed within a distance from the top of temporary slopes equal to half the slope height.

SURFACE DRAINAGE: The ground around the proposed structure should be graded so that surface water flows rapidly away from the structure without ponding. In general, we recommend that the ground adjacent to structure slope away at a gradient of at least two percent. Densely vegetated areas where runoff can be impaired should have a minimum gradient of five percent within the first five feet from the structure. Rain gutters with downspouts that discharge runoff away from the structure into controlled drainage devices are also recommended. It is our opinion that storm water systems incorporating infiltration are not appropriate for the site due to the potential for hydro-consolidation of the site soils.

GRADING PLAN REVIEW: The final grading plans should be submitted to this office for review in order to ascertain that the geotechnical recommendations remain applicable to the final plan and that no additional recommendations are needed due to changes in the anticipated development. Our firm should be notified of changes to the proposed project that could necessitate revisions of or additions to the information contained herein.

LIQUEFACTION MITIGATION - STONE COLUMNS

As discussed previously, the subject site is underlain by soils that are potentially liquefiable under the design seismic ground motion. This will require that steps be taken to reduce the potential liquefaction-related total and differential building settlement to an amount that satisfies the minimum life-safety criteria and project requirements. Though it is possible to reduce the settlements using special foundation design, it is our opinion that the most appropriate option to mitigate the potential for liquefaction settlement is to perform ground improvement. Several ground improvement techniques are available; however, it appears that stone columns will likely be the most effective given the predominant soil types and the project conditions. Stone columns densify loose cohesionless soils by inserting of a vibrating probe into the ground and then building a column of crushed aggregate that is also compacted using the vibrating probe. For this type of system, it is customary for a specialty contractor to provide the design, including specific equipment and procedural specifications. The design should be developed by the specialty contractor using the information presented in this geotechnical report combined with the loading and settlement requirements of the planned structures. As a minimum, we recommend that the stone columns extend at least one row outside the perimeter of the buildings or to the project boundary, whichever distance is less.

Good engineering practice requires that where the evaluation indicates that liquefaction is likely (or reasonably possible), the hazards that might reasonably be caused by liquefaction, that could result in the collapse of a structure and/or loss of life be mitigated. For shallow foundation systems supporting low-rise structures, it is our opinion that this level of life safety can be achieved by reducing the estimated post-liquefaction reconsolidation settlement to 4 inches or less. As such, we recommend that the design and construction of stone columns be performed to achieve a post-liquefaction reconsolidation settlement of 4 inches or less.

We recommend that the stone columns be observed during installation and that post-installation Cone Penetration Testing (CPT) be performed to quantify the degree of improvement achieved. Prior to construction, the stone column contractor should provide a submittal describing the design and outlining the planned installation procedure, including intended installation methods, equipment, penetration depths, rock type and volumes, and the required amperage. During installation, the contractor should provide means to measure, for each column, the penetration depth and rock volume placed and to verify the amperage achieved. After installation, post-CPTs will be performed in areas representing the average condition of the improved soil to measure the increased tip resistances and to re-evaluate the liquefaction settlement potential based on the achieved values. The liquefaction potential and corresponding settlement will be evaluated for the matrix soil using the same parameters that were used in the original evaluation except that the calculated settlement in the treated zone will be reduced by an improvement factor to account for the stiffness of the in-place rock columns. As shown by Priebe (1998), improvement factors of 1.3, 1.4, and 1.5 are applicable for area replacement ratios of 8½ percent, 11 percent, and 14½ percent, respectively. The applied improvement factor will be determined based on the average replacement ratio achieved in the grid area tested. Acceptance will be based on the total liquefaction settlement calculated from the post-CPTs and the equivalent differential settlement given the performance criteria described above.

FOUNDATIONS

GENERAL: The following design recommendations are considered the minimum based on anticipated soil conditions and are not intended to be lieu of structural considerations. All foundations should be designed by a qualified structural engineer.

POST-TENSION FOUNDATIONS: It is our opinion that post-tensioned slab/foundation systems should be used to support the proposed residential structures. Post-tensioned slabs should be designed in accordance with the design procedures of the Post-Tension Institute, using the design criteria presented below in Table

III. We recommend that perimeter footings have a minimum embedment depth of 18 inches below the adjacent finish grade.

TABLE III: POST-TENSION DESIGN CRITERIA

Post-Tensioning Institute (PTI) – 3rd Edition	**Design Value**
Edge Moisture Variation, e_m	
Center Lift (ft)	9.0
Edge Lift (ft)	4.9
Differential Soil Movement, y_m	
Center Lift (in)	0.66
Edge Lift (in)	1.58

OTHER FOOTINGS: Retaining wall footings should have a minimum embedment depth of 18 inches below the lowest adjacent grade and should have a minimum width of 24 inches. Footings for miscellaneous exterior structures such as trash enclosures should have a minimum embedment depth of 12 inches below the lowest adjacent grade and a minimum width of 12 inches.

BEARING CAPACITY: Footings with a minimum embedment depth and width of 12 inches may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf) for dead plus live loads. This value may be increased by 300 psf for each additional foot of embedment depth or width, up to a maximum of 4,000 psf. The bearing value may also be increased by one-third for combinations of temporary loads such as those due to wind or seismic loads.

LATERAL LOAD RESISTANCE: Lateral loads against foundations may be resisted by friction between the bottom of the footing and the supporting soil, and by the passive pressure against the footing. The coefficient of friction between concrete and soil may be considered to be 0.35. The passive resistance may be considered to be equal to an equivalent fluid weight of 350 pounds per cubic foot. These values are based on the assumption that the footings are poured tight against undisturbed soil. If a combination of the passive pressure and friction is used, the friction value should be reduced by one-third.

SETTLEMENT CHARACTERISTICS: The anticipated total and differential foundation settlement for the static condition is expected to be less than one inch and ¾ inch in forty feet, respectively, provided the recommendations presented in this report are followed. It should be recognized that minor cracks normally occur in concrete slabs and foundations due to shrinkage during curing or redistribution of stresses, therefore some cracks should be anticipated. Such cracks are not necessarily an indication of excessive vertical movements.

Provided ground improvement is performed as discussed above, the expected total liquefaction settlement is on the order of 4 inches. Based on the presence of a 20- to 25-foot-thick layer of non-liquefiable soil below the buildings and the use of post-tension foundation systems, the expected differential liquefaction settlement is less than 2 inches across the width of the building. It is our opinion that this magnitude of total and differential liquefaction settlement will not result in the collapse of the structure or incur the loss of life; however, it does present the possibility of structural damage and the need to repair or replace the structure.

EXPANSIVE CHARACTERISTICS: The foundation soils are expected to have a "low" expansion index. The site preparation and foundation recommendations reflect this condition.

FOUNDATION PLAN REVIEW: The final foundation plan and accompanying details and notes should be submitted to this office for review. The intent of our review will be to verify that the plans used for construction reflect the minimum dimensioning and reinforcing criteria presented in this section and that no additional criteria are required due to changes in the foundation type or layout. It is not our intent to review structural plans, notes, details, or calculations to verify that the design engineer has correctly applied the geotechnical design values. It is the responsibility of the design engineer to properly design/specify the foundations and other structural elements based on the requirements of the structure and considering the information presented in this report.

FOUNDATION EXCAVATION OBSERVATION: All foundation excavations should be observed by the Geotechnical Consultant prior to placing reinforcing steel or formwork in order to determine if the foundation recommendations presented herein are followed. All footing excavations should be excavated neat, level, and square. All loose or unsuitable material should be removed prior to the placement of concrete.

CORROSIVITY

The water soluble sulfate content was determined for a representative soil sample from the site in accordance with California Test Method 417. The result, which is presented in Appendix B, indicates that the on-site soils are, in general, negligibly corrosive to concrete.

It should be understood Christian Wheeler Engineering does not practice corrosion engineering. If such an analysis is considered necessary, we recommend that the client retain an engineering firm that specializes in this field to consult with them on this matter. The results of our tests should only be used as a guideline to determine if additional testing and analysis is necessary.

ON-GRADE SLABS

INTERIOR SLAB: We expect that the design of floor systems of the proposed structure will be performed by others as part of the post-tension foundation design. The owner and the project structural engineer should determine if the on-grade slabs need to be designed for special loading conditions.

.

UNDER-SLAB VAPOR RETARDERS: Where floor coverings are installed, steps should be taken to minimize the transmission of moisture vapor from the subsoil through the interior slabs where it can potentially damage the interior floor coverings. We recommend that the owner/contractor follow national standards for the installation of vapor retarders below interior slabs as presented in currently published standards including ACI 302, "Guide to Concrete Floor and Slab Construction" and ASTM E1643, "Standard Practice for Installation of Water Vapor Retarder Used in Contact with Earth or Granular Fill Under Concrete Slabs".

EXTERIOR CONCRETE FLATWORK: Exterior concrete on-grade slabs should have a minimum thickness of four inches. Exterior slabs abutting perimeter foundations should be doweled into the footings. All slabs should be provided with weakened plane joints in accordance with the American Concrete Institute (ACI) guidelines. Alternative patterns consistent with ACI guidelines can also be used. A concrete mix with a 1-inch maximum aggregate size and a water/cement ratio of less than 0.6 is recommended for exterior slabs. Lower water content will decrease the potential for shrinkage cracks. Both coarse and fine aggregate should conform to the latest edition of the "Standard Specifications for Public Works Construction" ("Greenbook").

Special attention should be paid to the method of concrete curing to reduce the potential for excessive shrinkage and resultant random cracking. It should be recognized that minor cracks occur normally in concrete slabs due to shrinkage. Some shrinkage cracks should be expected and are not necessarily an indication of excessive movement or structural distress.

EARTH RETAINING WALLS

FOUNDATIONS: Foundations for retaining walls can be designed in accordance with the foundation recommendations previously presented.

ACTIVE PRESSURES: The active soil pressure for the design of unrestrained and restrained earth retaining structures with a level backfill surface may be assumed to be equivalent to the pressure of a fluid weighing 35 and 55 pounds per cubic foot, respectively. An additional 20 pounds per cubic foot should be added for 2:1 (H:V) sloping backfill. Thirty percent of any area surcharge placed adjacent to the retaining wall may be assumed to act as a uniform horizontal pressure against the wall. Where vehicles will be allowed within ten feet of the top of the retaining wall, a uniform horizontal pressure of 100 pounds per square foot should be added to the upper 10 feet of the retaining wall to account for the effects of adjacent traffic. If any other loads are anticipated, the Geotechnical Consultant should be contacted for the necessary increase in soil pressure. All values are based on a drained backfill condition.

If it is necessary to consider seismic pressure, it may be assumed to be equivalent to the pressure of a fluid weighing 13 pounds per cubic foot, but the pressure distribution should be inverted so that the highest value is at the top of the wall. This corresponds to an approximate pseudo-static acceleration (Kh) of 0.11 g.

PASSIVE PRESSURES: The passive pressure for the prevailing soil conditions may be considered to be 350 pounds per square foot per foot of depth for foundations in fill soil. This pressure may be increased one-third for seismic loading. The upper one foot of soil should be neglected where the footing is abutted by landscaping. The coefficient of friction for concrete to soil may be assumed to be 0.35 for the resistance to lateral movement. When combining frictional and passive resistance, the friction should be reduced by one-third.

WATERPROOFING AND SUBDRAINS: The project architect should provide (or coordinate) waterproofing details for the retaining walls. The design values presented above are based on a drained backfill condition and do not consider hydrostatic pressures. Unless hydrostatic pressures are incorporated into the design, the retaining wall designer should provide a subdrain detail. A typical retaining wall subdrain detail is presented as Plate No. 2 of this report. Additionally, outlets points for the retaining wall subdrains should be coordinated by the project civil engineer. For subterranean walls, it may be necessary to collect the subdrain water in sumps and then pump it to an appropriate outlet.

BACKFILL: All retaining wall backfill should be compacted to at least 90 percent relative compaction. It is anticipated that the on-site soils are suitable for use as backfill material provided the design parameters given herein are used in the wall design. Retaining walls should not be backfilled until the masonry/concrete has reached an adequate strength.

PRELIMINARY PAVEMENT SECTIONS

GENERAL: We expect that new pavement will be installed as part of the project. The following presents preliminary sections for asphalt concrete (AC) or Portland Cement Concrete (PCC) construction. The pavement sections provided in Table IV and Table VI should be considered preliminary and should be used for planning purposes only. Final pavement designs should be determined after R-value tests have been performed in the actual subgrade material in place after grading. Presuming the grading recommendations presented previously are followed, we estimate that the subgrade soils will have an R-Value of approximately 25. The Traffic Index and Traffic Categories shown below are assumed. The project client and/or civil engineer should determine whether these assumed values are appropriate for the traffic conditions.

ASPHALT CONCRETE: We expect that the drive aisles will primarily support passenger vehicles with heavily loaded vehicles such as garbage trucks and large moving vans on average about 5 times per day. The parking stalls are expected to support primarily passenger vehicles and occasional moving vans. The asphalt concrete pavement section was calculated using the Caltrans design method using an assumed Traffic Index of 6.0 for drive aisles and 4.5 for parking stalls.

TABLE IV: ASPHALT CONCRETE SECTIONS

Pavement Type	Traffic Index	Pavement Thickness	Base Thickness	Base Material	Subgrade Compaction
Asphalt Concrete					
Drive Aisles	6.0	3.0 in.	9.5 in.	CAB or Class II	95% in upper 12”
Parking Stalls	4.5	3.0 in.	5.0 in.	CAB or Class II	95% in upper 12”

Prior to placing the base material beneath asphalt concrete pavements, the subgrade soil should be scarified to a depth of 12 inches and compacted to at least 95 percent of its maximum dry density at a moisture content one to three percent above optimum.

The base material could consist of Crushed Aggregate Base (CAB) or Class II Aggregate Base. The Crushed Aggregate Base should conform to the requirements set forth in Section 200-2.2 of the Standard Specifications for Public Works Construction. The Class II Aggregate Base should conform to requirements set forth in Section 26-1.02A of the Standard Specifications for California Department of Transportation. Asphalt concrete should be placed in accordance with ‘Standard Specifications for Public Works Construction (Greenbook), Section 302-5. Asphalt concrete pavement should be compacted to at least 95 % of Hveem density.

CONCRETE PAVEMENTS: Portland cement concrete (PCC) pavement thickness can be determined from Table V. The PCC pavement section was determined in general accordance with the procedure recommended within the American Concrete Institute report ACI-330R-08 Guide for Design and Construction of Concrete Parking Lots using the parameters listed in Table IV. We recommend that the referenced ACI-330R Guide be used to determine the appropriate requirements for control joint configuration, reinforcing, and dowelling of the construction joints. Portland Cement Concrete pavement placed in front of trash enclosures should be reinforced with at least No. 4 bars placed at 12 inches on center each way.

TABLE V: CONCRETE PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of Subgrade Reaction, k	100 pci
Modulus of Rupture for Concrete, M_R	500 psi
Traffic Category (Main Driveways)	A (ADTT = 10)

ADTT = Average Daily Truck Traffic. Trucks defined as vehicles with at least six wheels.

Based on the design parameters summarized in Table V, the PCC pavements should have the minimum thicknesses shown in Table VI.

TABLE VI: MINIMUM CONCRETE PAVEMENT THICKNESS

Pavement Use	Thickness
Main Driveways/Aisles/Trash Enclosures	6.0 in
Parking Stalls	5.5 in

Prior to placing concrete pavement, the subgrade soils should be scarified to a depth of 12 inches and compacted to at least 95 percent of their maximum dry density at a moisture content one to three percent above optimum. Concrete pavement construction should comply with the requirements set forth in Sections 201-1.1.2 and 302-6 of the Standard Specifications for Public Works Construction (concrete Class 560-C-3250).

The outside edge of concrete slabs that will support wheel loads should have a thickened edge or integral curb. The thickened edge should be at least 2 inches thicker than the slab and should taper back to the recommended slab thickness 3 feet from the edge of the slab.

LIMITATIONS

REVIEW, OBSERVATION AND TESTING

The recommendations presented in this report are contingent upon our review of final plans and specifications. Such plans and specifications should be made available to the geotechnical engineer and engineering geologist so that they may review and verify their compliance with this report and with the California Building Code.

It is recommended that Christian Wheeler Engineering be retained to provide continuous soil engineering services during the earthwork operations. This is to verify compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

UNIFORMITY OF CONDITIONS

The recommendations and opinions expressed in this report reflect our best estimate of the project requirements based on an evaluation of the subsurface soil conditions encountered at the subsurface exploration locations and on the assumption that the soil conditions do not deviate appreciably from those encountered. It should be recognized that the performance of the foundations and/or cut and fill slopes may be influenced by undisclosed or unforeseen variations in the soil conditions that may occur in the intermediate and unexplored areas. Any unusual conditions not covered in this report that may be encountered during site development should be brought to the attention of the geotechnical engineer so that he may make modifications if necessary.

CHANGE IN SCOPE

This office should be advised of any changes in the project scope or proposed site grading so that we may determine if the recommendations contained herein are appropriate. This should be verified in writing or modified by a written addendum.

TIME LIMITATIONS

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in the Standards-of-Practice and/or Government Codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a period of two years without a review by us verifying the suitability of the conclusions and recommendations.

PROFESSIONAL STANDARD

In the performance of our professional services, we comply with that level of care and skill ordinarily exercised by members of our profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the locations where our test pits, surveys, and explorations are made, and that our data, interpretations, and recommendations be based solely on the information obtained by us. We will be responsible for those data, interpretations, and recommendations, but shall not be responsible for the interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty of any kind whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.

CLIENT'S RESPONSIBILITY

It is the client's responsibility, or its representatives, to ensure that the information and recommendations contained herein are brought to the attention of the structural engineer and architect for the project and incorporated into the project's plans and specifications. It is further their responsibility to take the necessary measures to insure that the contractor and his subcontractors carry out such recommendations during construction.

FIELD EXPLORATIONS

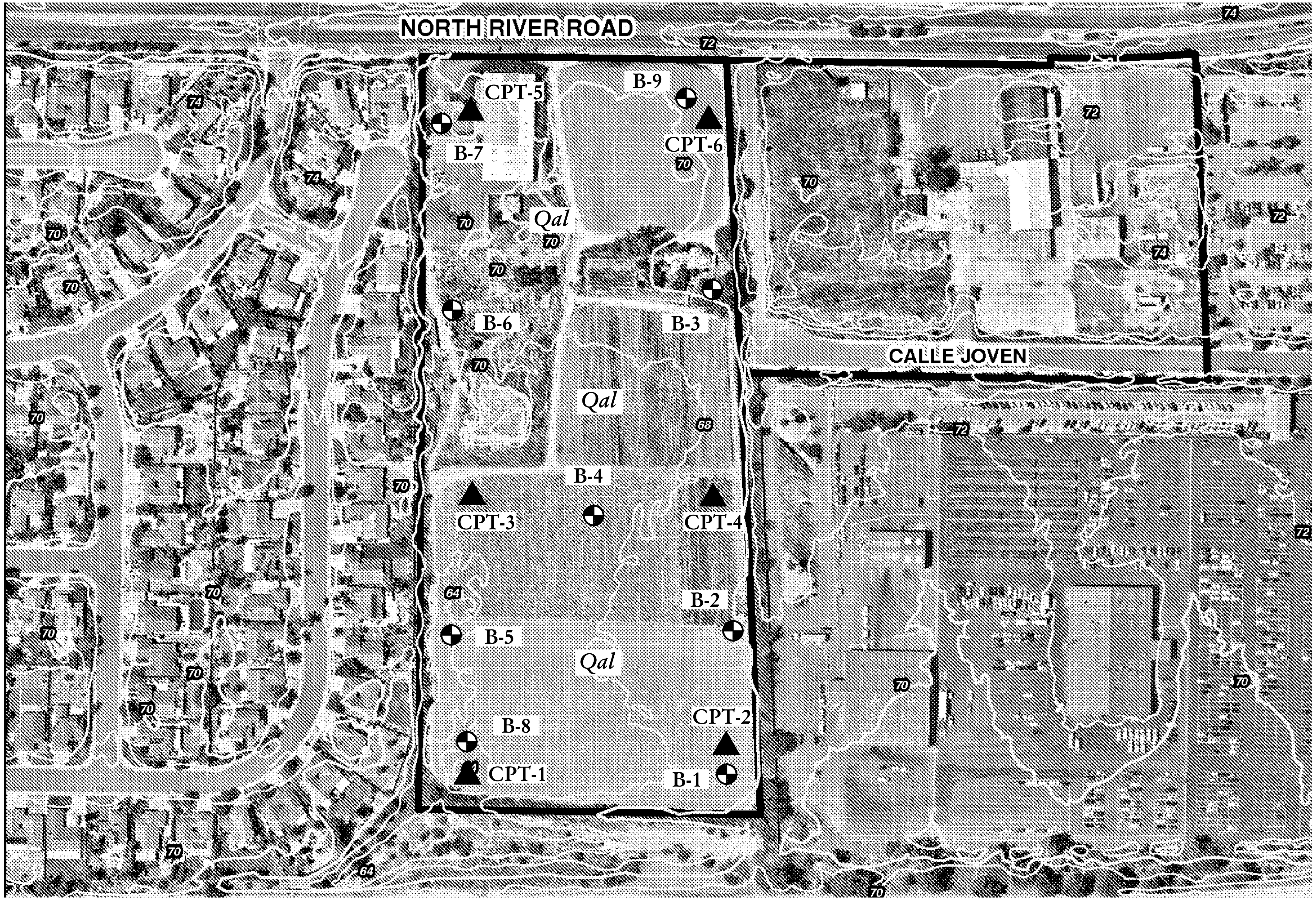
Fourteen subsurface explorations were made during this investigation at the locations indicated on the Site Plan included herewith as Plate Number 1. These explorations consisted of nine small-diameter, hollow-stem borings drilled with a truck-mounted drill rig between June 14 and July 11, 2005 and six Cone Penetration Tests conducted on October 7, 2015. The fieldwork was conducted under the observation and direction of our engineering geology personnel.

The CPT probes were performed by Kehoe Testing and Engineering, using an integrated electronic cone system. The results are presented in Appendix A. The CPT soundings were performed in accordance with ASTM Standard D3441. A thirty-ton capacity cone was used for all of the soundings. This cone had a tip area equal to 15 square centimeters and friction sleeve area of 225 square centimeters. The cone was designed with an equal end area friction sleeve and a tip end area ratio of 0.85. On the logs of the CPT soundings, the soils are described in terms of the Soil Behavior Type (SBT). The stratigraphic expression of the soil types, SBT, is based on the relationships between the measured cone bearing, sleeve friction, and penetration pore pressures measured almost continuously within each sounding.

The borings were carefully logged when made. The boring logs are presented in the attached Appendix B. The soils are described in accordance with the Unified Soils Classification. In addition, a verbal textural description, the wet color, the apparent moisture and the density or consistency are provided. The density of granular soils is given as either very loose, loose, medium dense, dense or very dense. The consistency of silts or clays is given as either very soft, soft, medium stiff, stiff, very stiff, or hard. Undisturbed samples of typical and representative soils were obtained and returned to the laboratory for testing. The undisturbed samples were obtained by driving a 2 ⅜-inch inside diameter split-tube sampler ahead of the auger using a 140-pound weight free-falling a distance of 30 inches. The number of blows required to drive the sampler each foot was recorded and this value is presented on the attached boring logs as "Penetration Resistance." Bulk samples of disturbed soil were also collected in bags from the auger cuttings during the advancement of the borings and transported to the laboratory for testing.

LABORATORY TESTING

Laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. The results are presented in Appendix B.



W+E

N W E S

0 200' 400'

SCALE: 1" = 200'

KAWANO-NAGATA

TOPOGRAPHIC AERIAL MAP

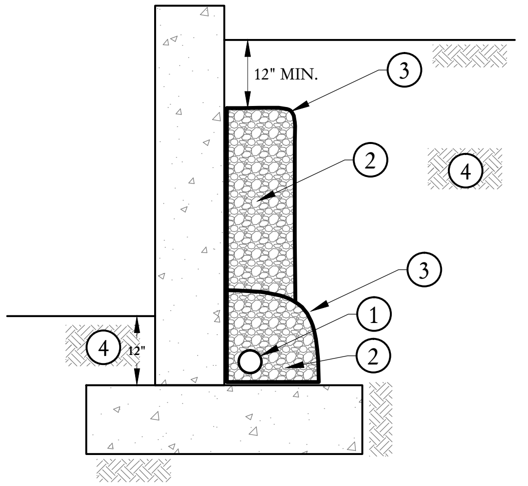
CWE LEGEND	
B-9	APPROXIMATE BORING LOCATION
CPT-6	APPROXIMATE CONE PENETROMETER TEST LOCATION
Qal	ALLUVIUM

SITE PLAN AND GEOTECHNICAL MAP

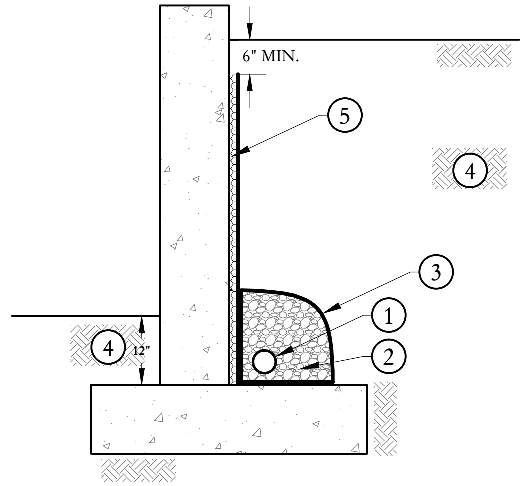
NAGATA PROPERTY
4617 NORTH RIVER ROAD
OCEANSIDE, CALIFORNIA

DATE:	DECEMBER 2015	JOB NO.:	2140692
BY:	SRD	PLATE NO.:	1

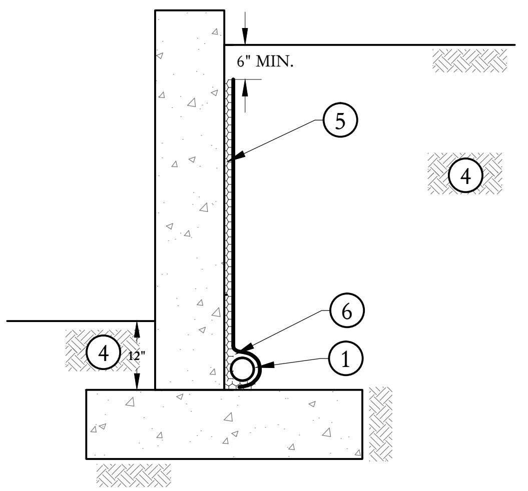
CHRISTIAN WHEELER ENGINEERING



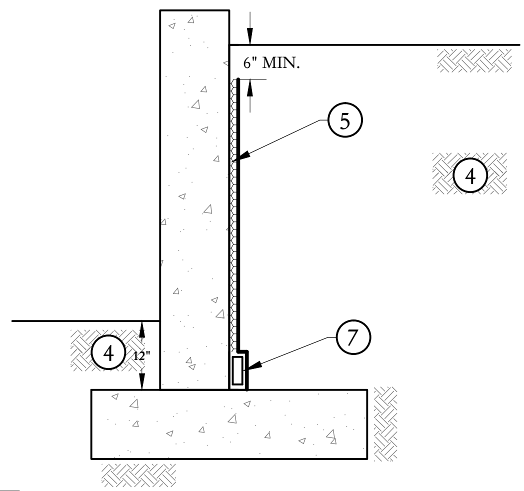
1 DETAIL



2 DETAIL



3 DETAIL



4 DETAIL

NOTES AND DETAILS

GENERAL NOTES:

1) THE NEED FOR WATERPROOFING SHOULD BE EVALUATED BY OTHERS.
2) WATERPROOFING TO BE DESIGNED BY OTHERS (CWE CAN PROVIDE A DESIGN IF REQUESTED).
3) EXTEND DRAIN TO SUITABLE DISCHARGE POINT PER CIVIL ENGINEER.
4) DO NOT CONNECT SURFACE DRAINS TO SUBDRAIN SYSTEM.

DETAILS:

1. 4-INCH PERFORATED PVC PIPE ON TOP OF FOOTING, HOLES POSITIONED DOWNWARD (SDR 35, SCHEDULE 40, OR EQUIVALENT).
2. ¾ INCH OPEN-GRADED CRUSHED AGGREGATE.
3. GEOFARBRIC WRAPPED COMPLETELY AROUND ROCK.
4. PROPERLY COMPACTED BACKFILL SOIL.
5. WALL DRAINAGE PANELS (MIRADRAIN OR EQUIVALENT) PLACED PER MANUFACTURER'S REC'S.
6. UNDERLAY SUBDRAIN WITH AND CUT FABRIC BACK FROM DRAINAGE PANELS AND WRAP FABRIC AROUND PIPE.
7. COLLECTION DRAIN (TOTAL DRAIN OR EQUIVALENT) LOCATED AT BASE OF WALL DRAINAGE PANEL PER MANUFACTURER'S RECOMMENDATIONS.

CANTILEVER RETAINING WALL DRAINAGE SYSTEMS

NAGATA PROPERTY
4617 NORTH RIVER ROAD
OCEANSIDE, CALIFORNIA

DATE:	DECEMBER 2015	JOB NO.:	2140692
BY:	SRD	PLATE NO.:	2

CHRISTIAN WHEELER
ENGINEERING

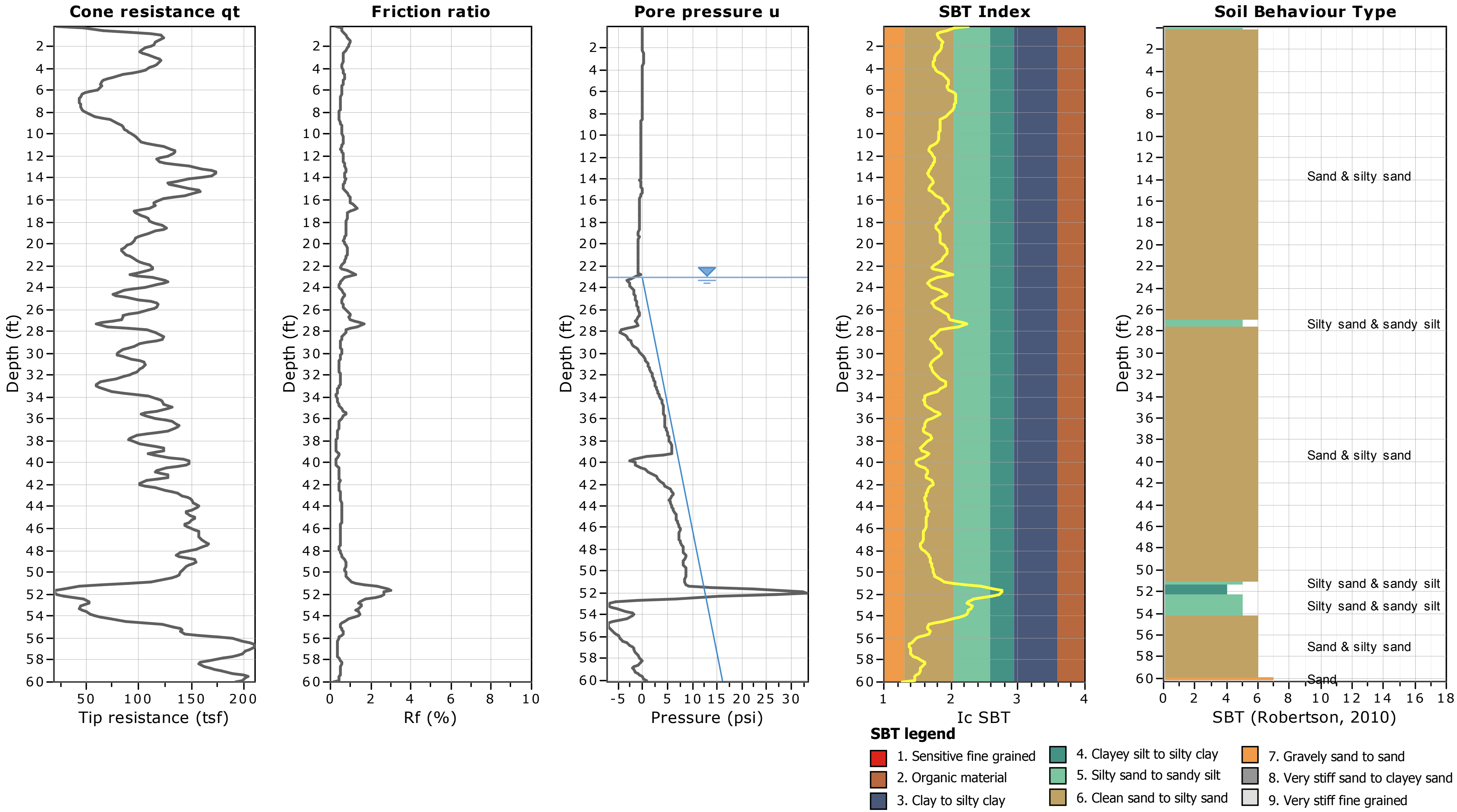
Appendix A

Cone Penetration Test Results

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-1
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 65.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

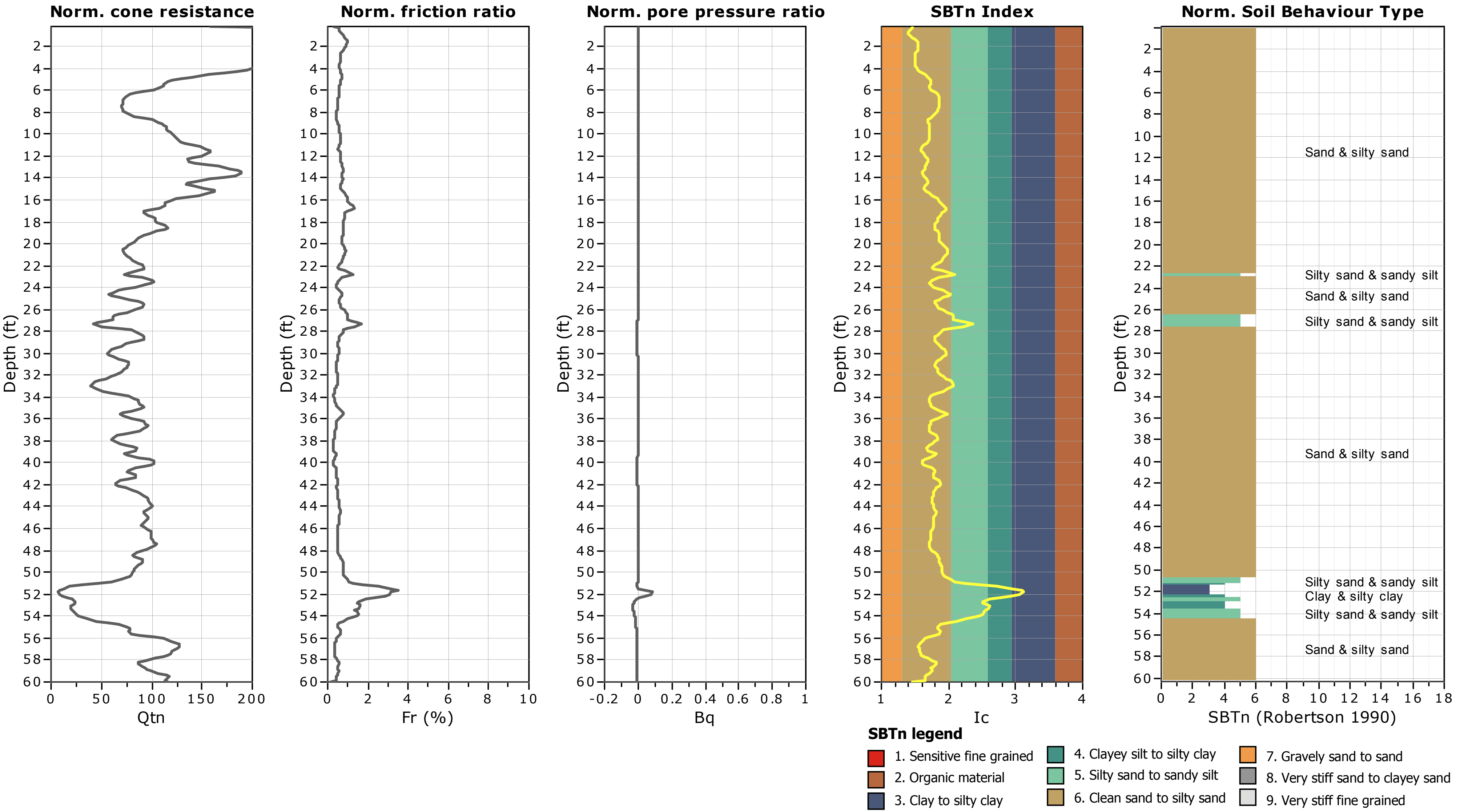
Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-1
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 65.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

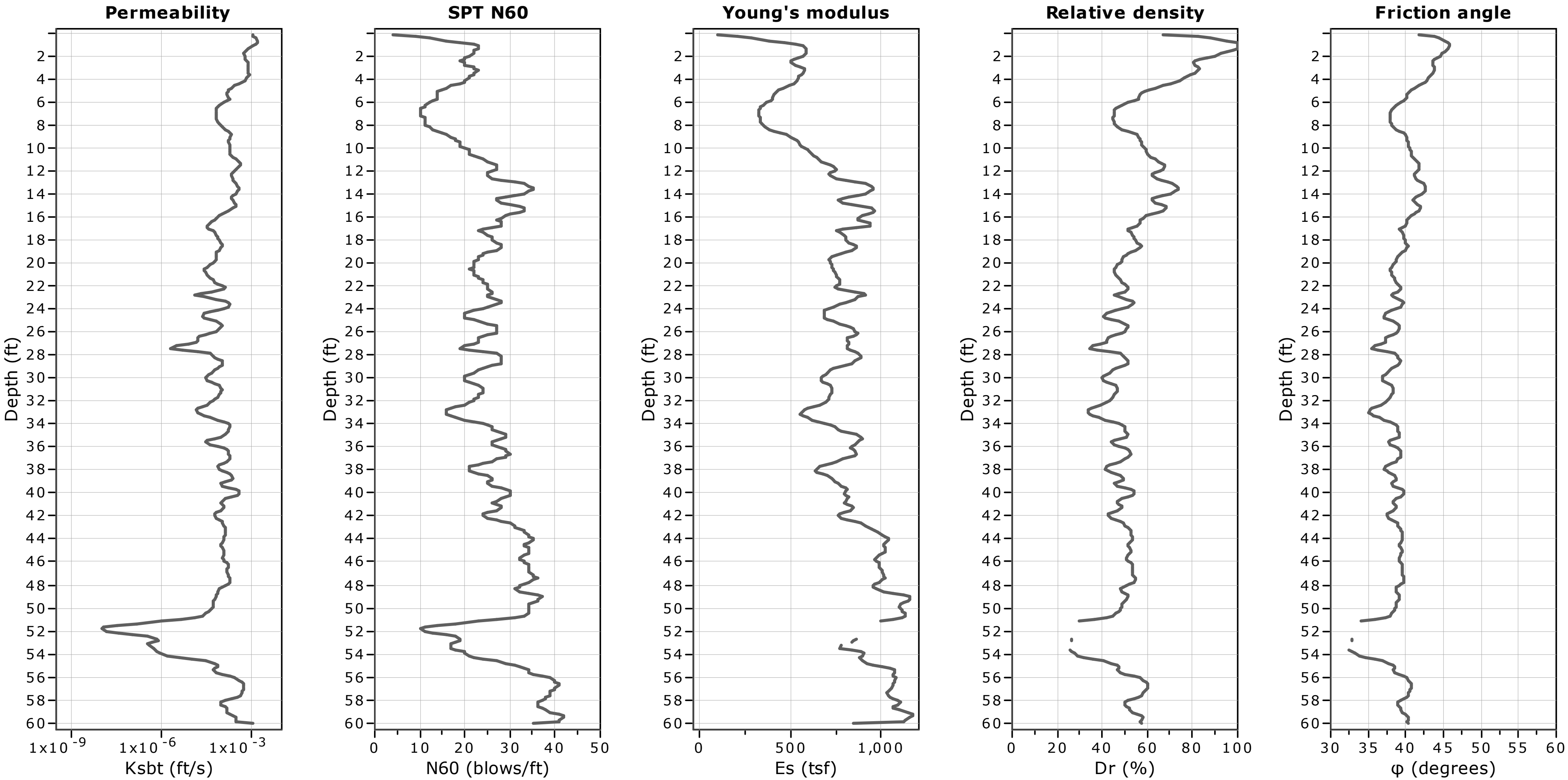
Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-1
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 65.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**

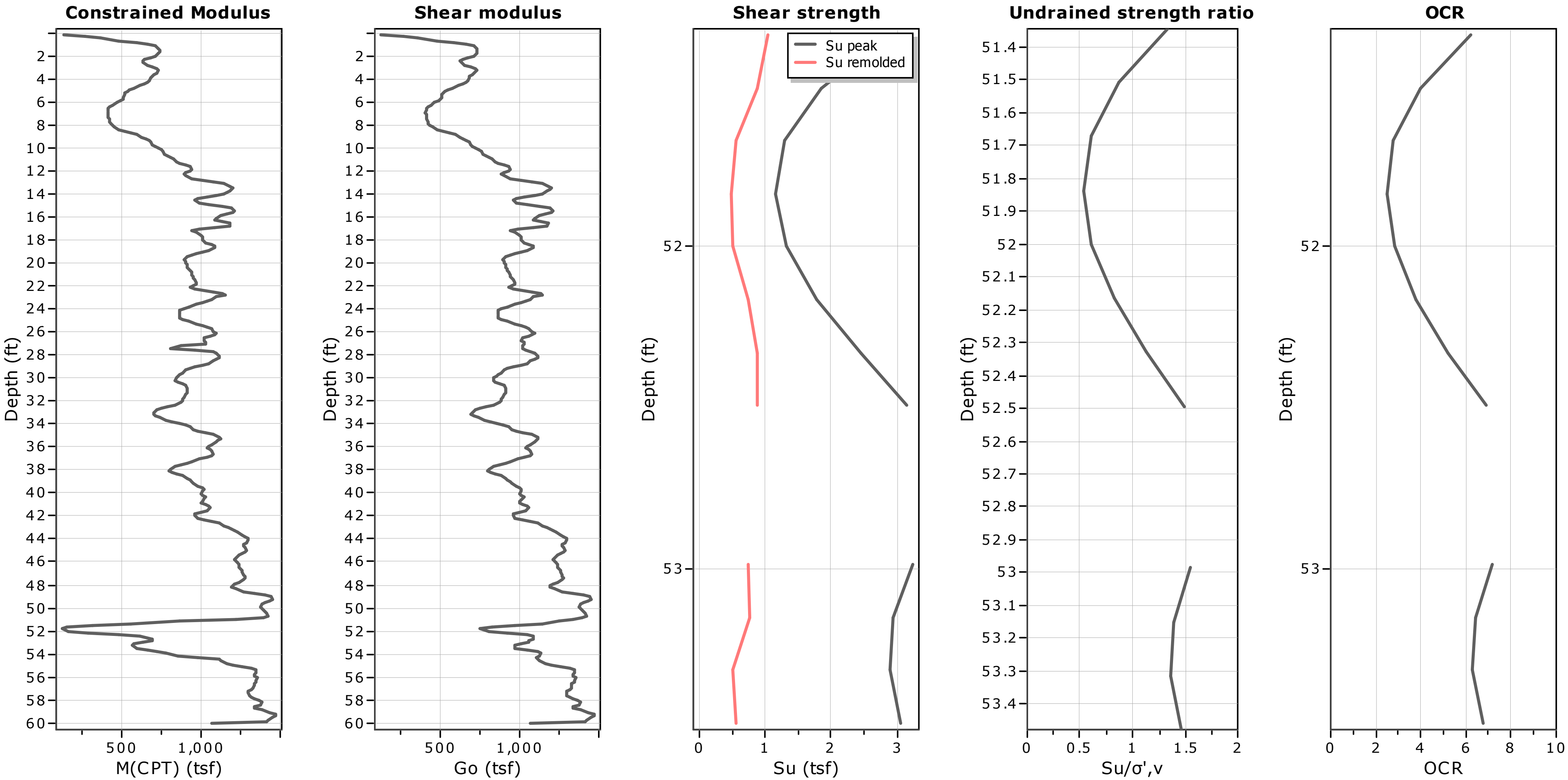


Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative desnisty constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-1
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 65.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**

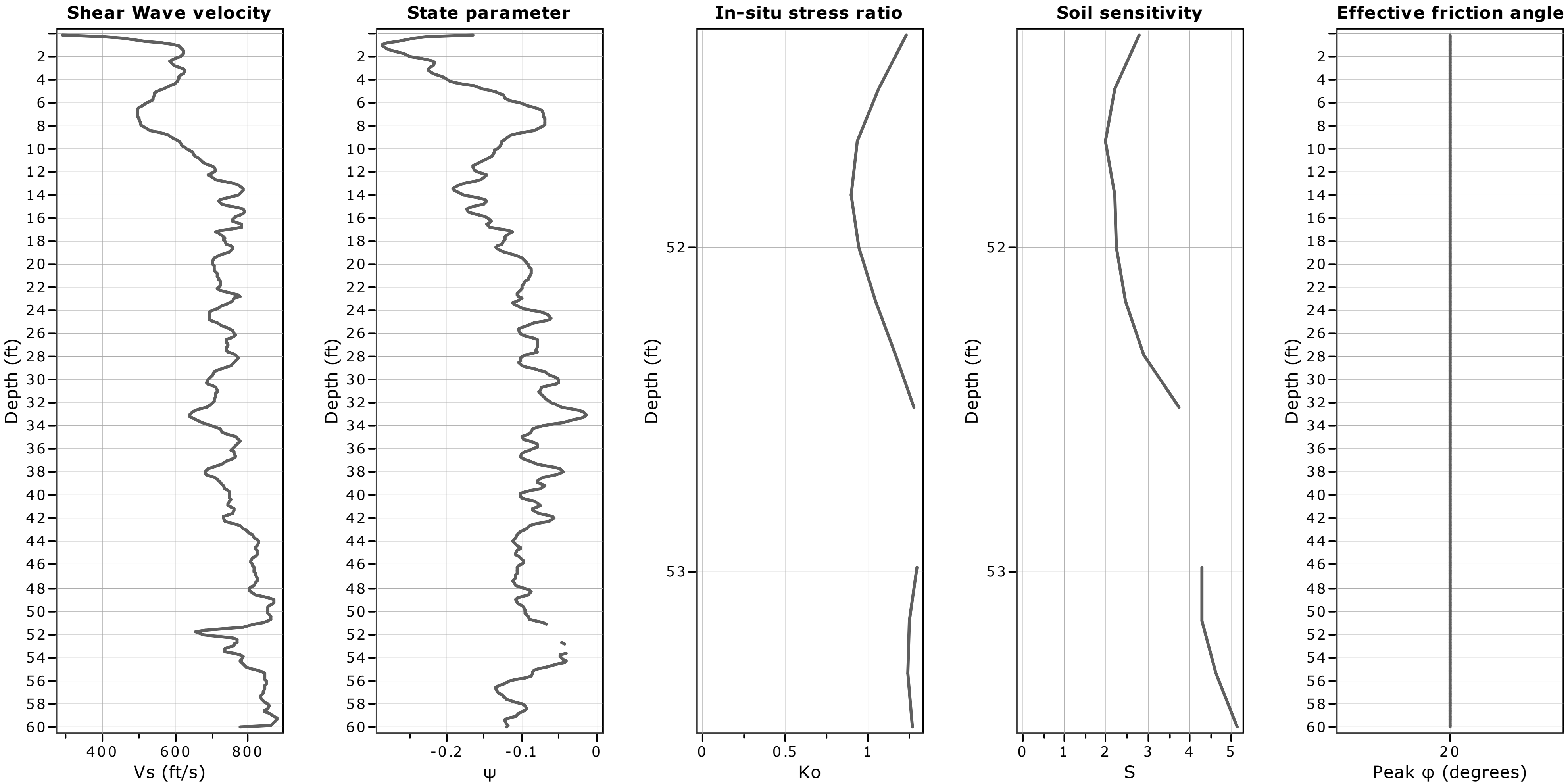


Calculation parameters
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14
OCR factor for clays, N_{kt}: 0.33
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-1
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 65.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: Nagata Site
Location: 4617 North River Road, Oceanside, California



Calculation parameters

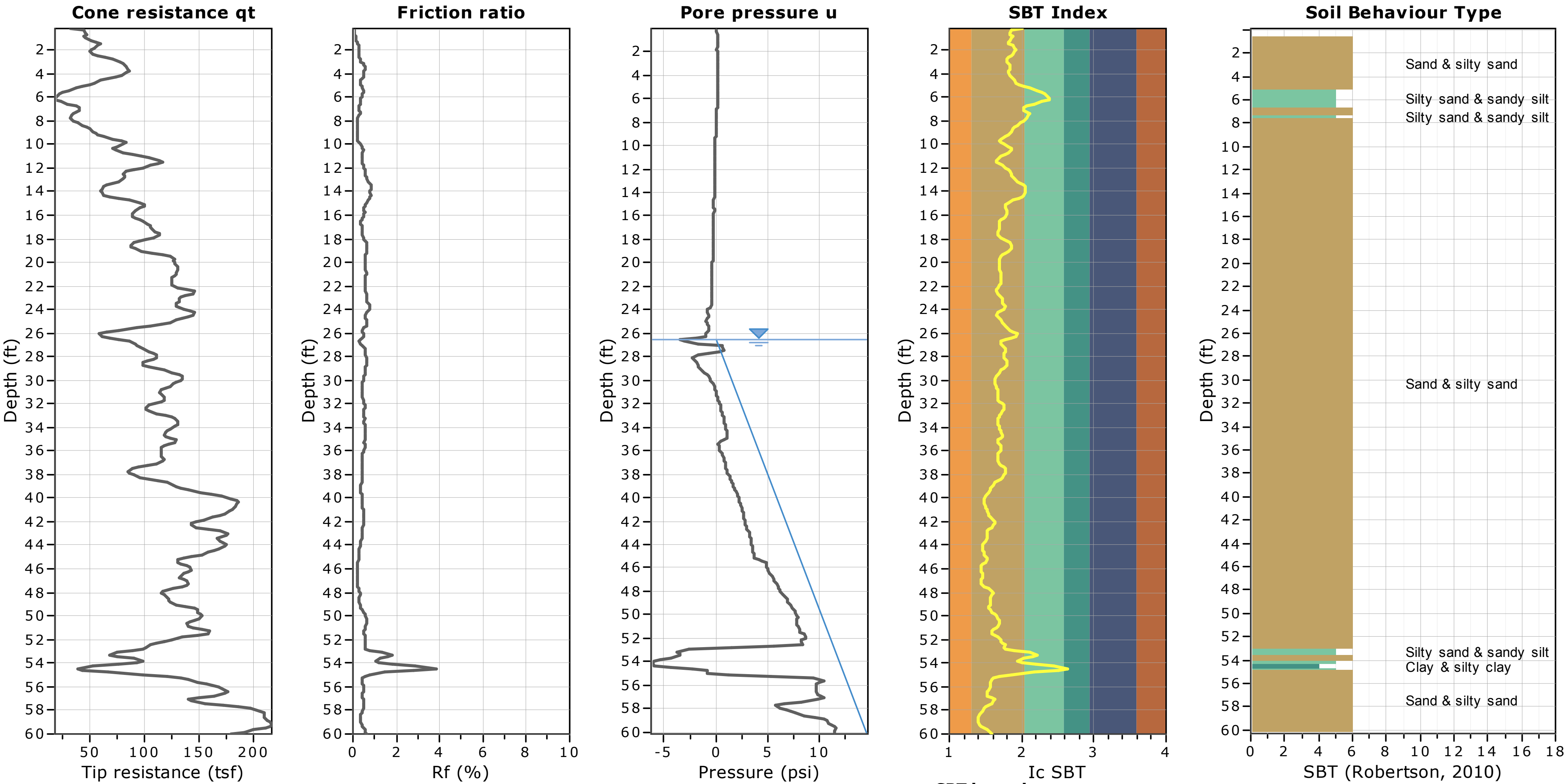
Soil Sensitivity factor, N_S: 7.00

User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-2
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



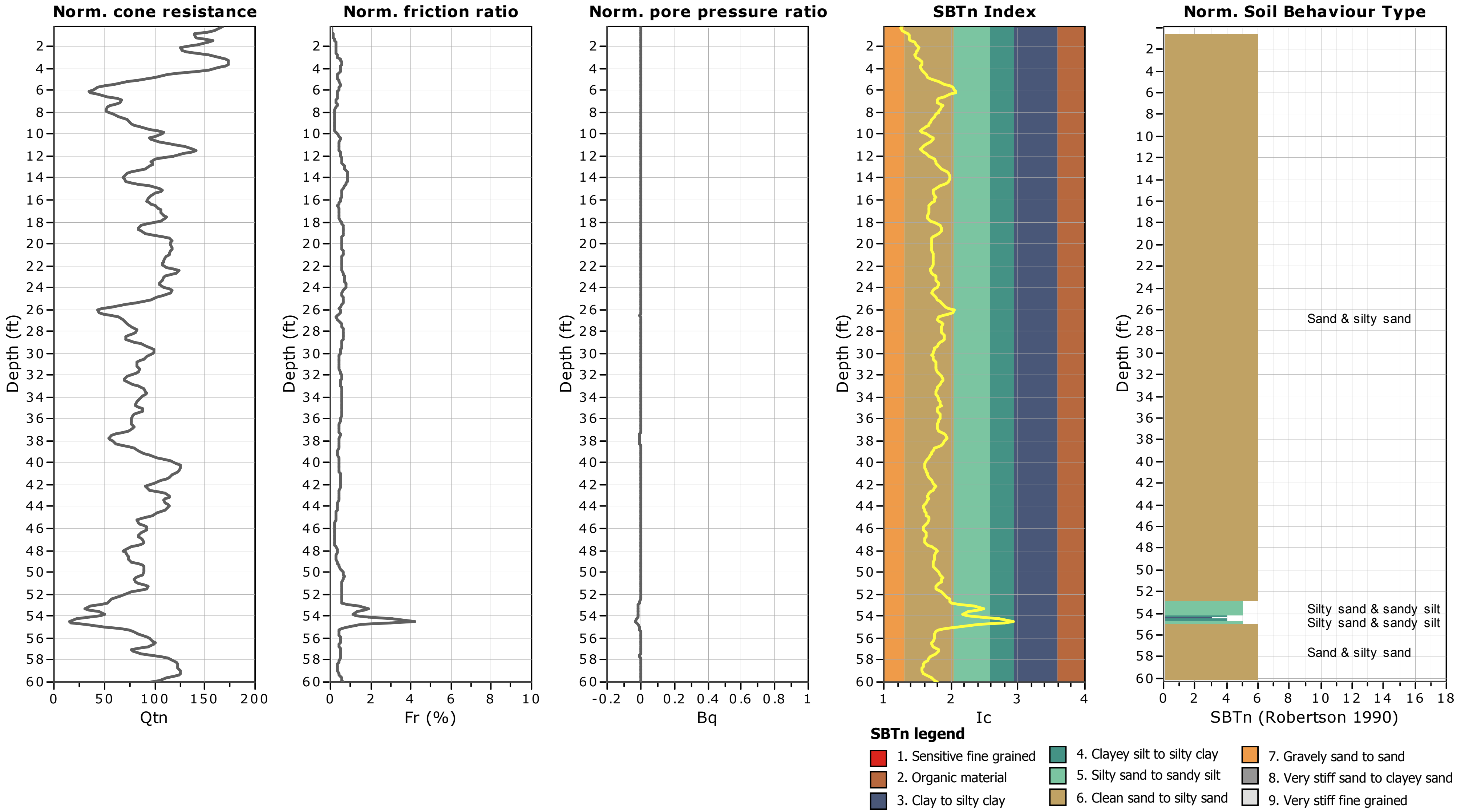
SBT legend

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

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-2
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

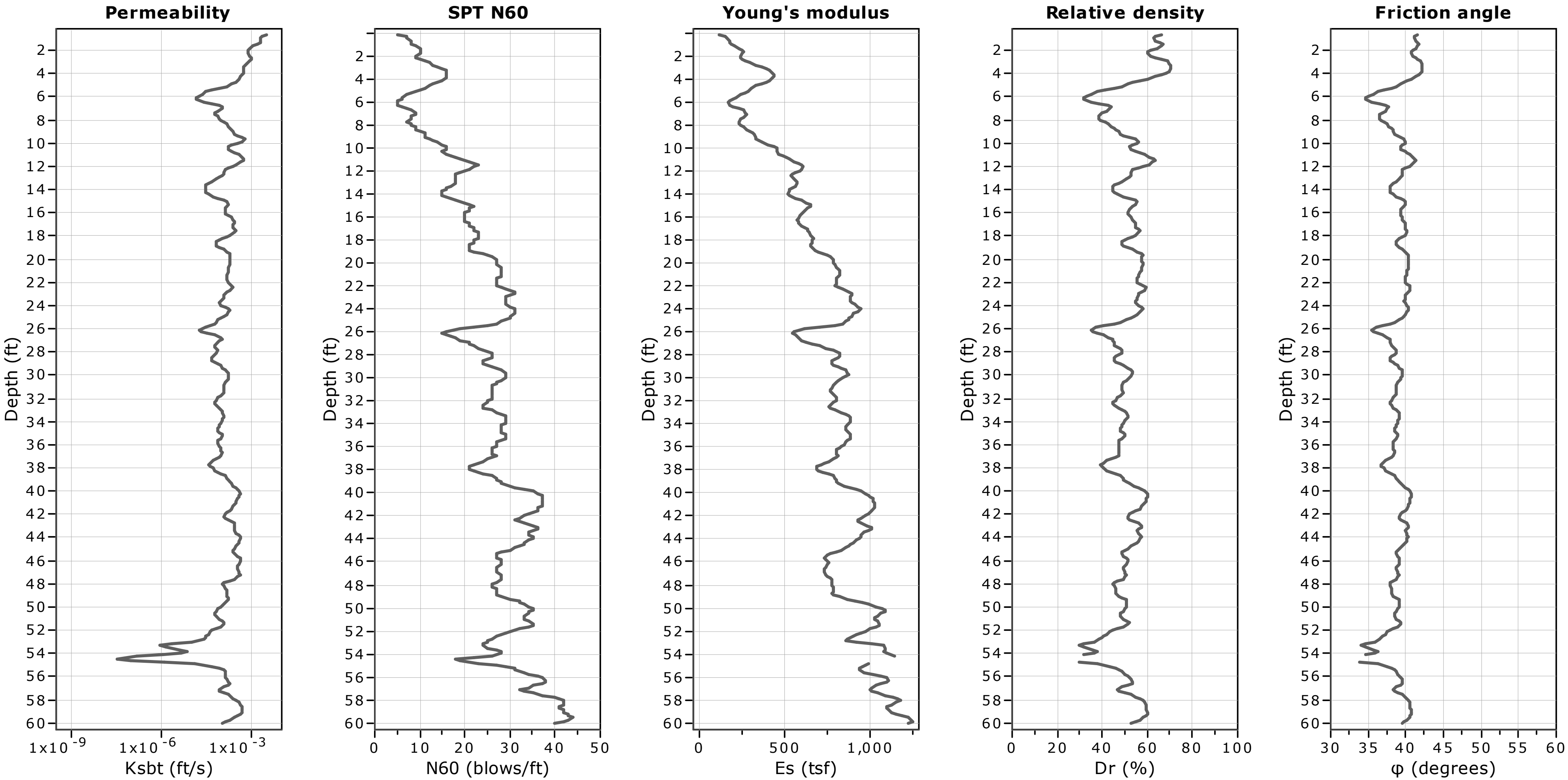
Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-2
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**

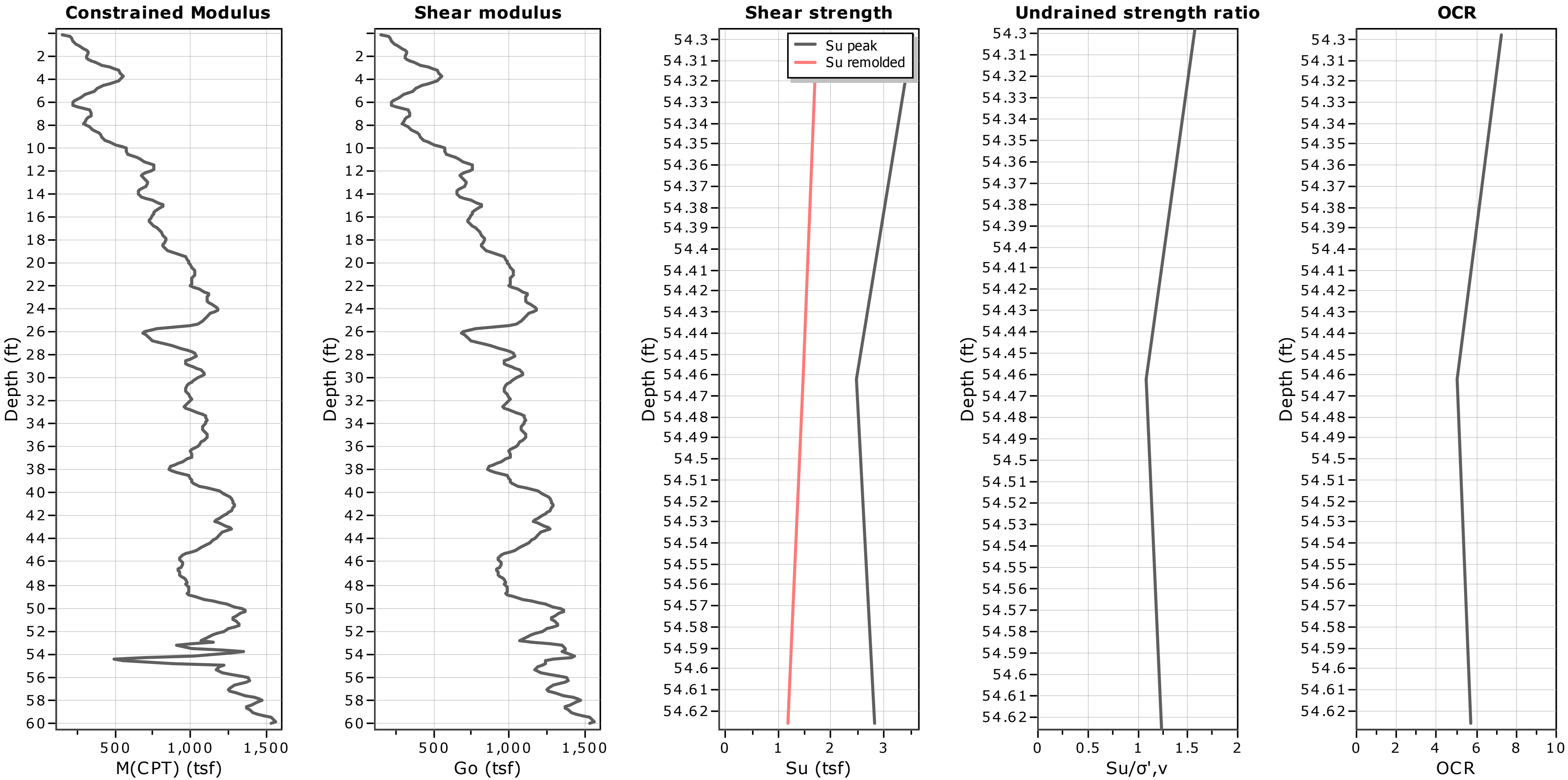


Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative desnisty constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-2
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



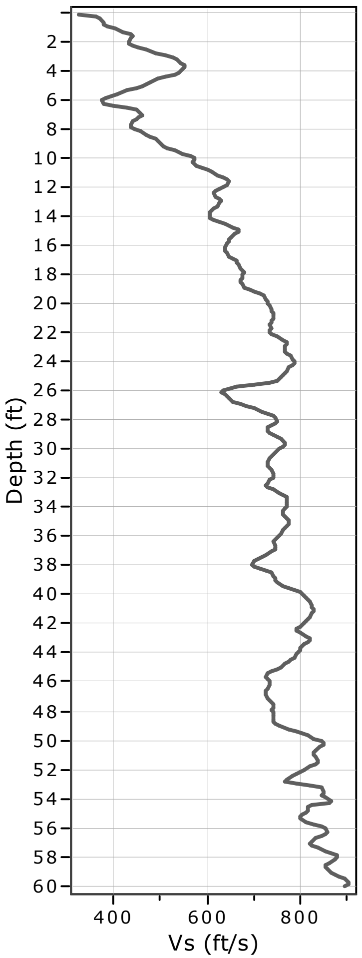
Calculation parameters
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14
OCR factor for clays, N_{kt}: 0.33
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

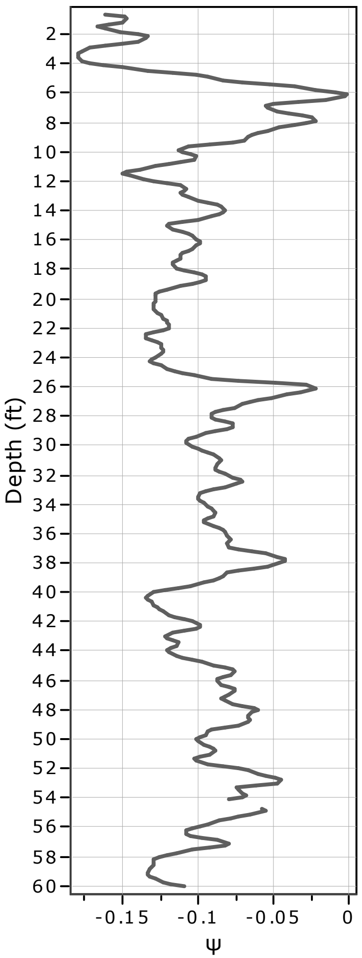
CPT: CPT-2
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**

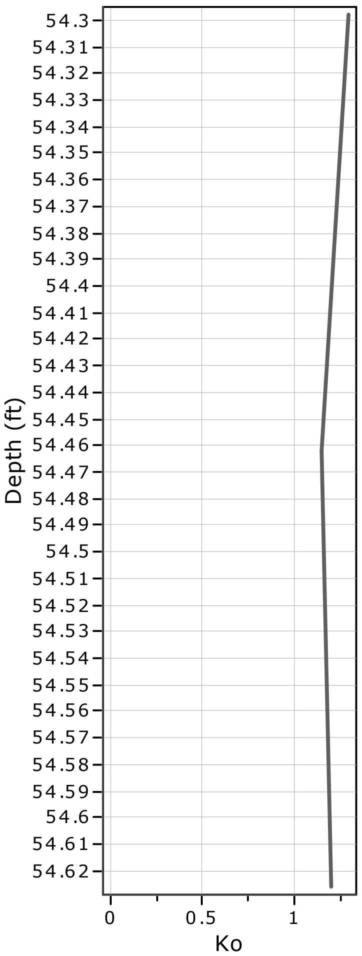
Shear Wave velocity



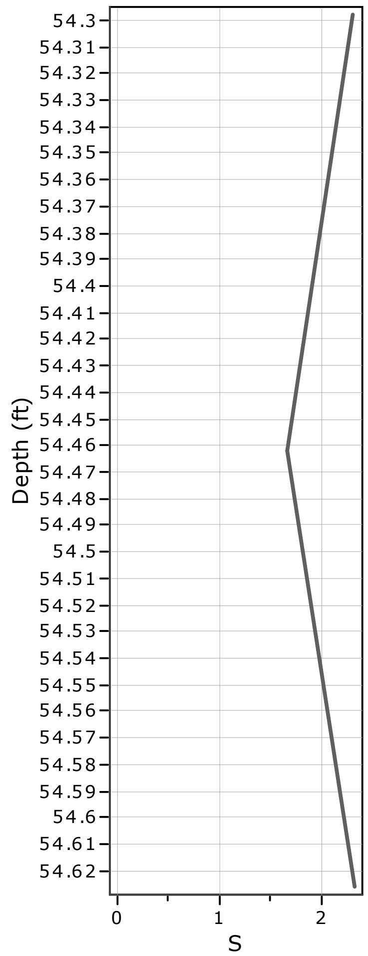
State parameter



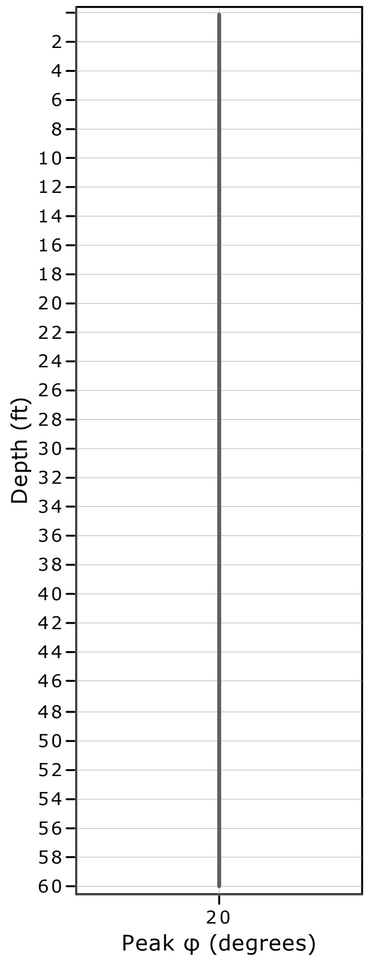
In-situ stress ratio



Soil sensitivity



Effective friction angle



Calculation parameters

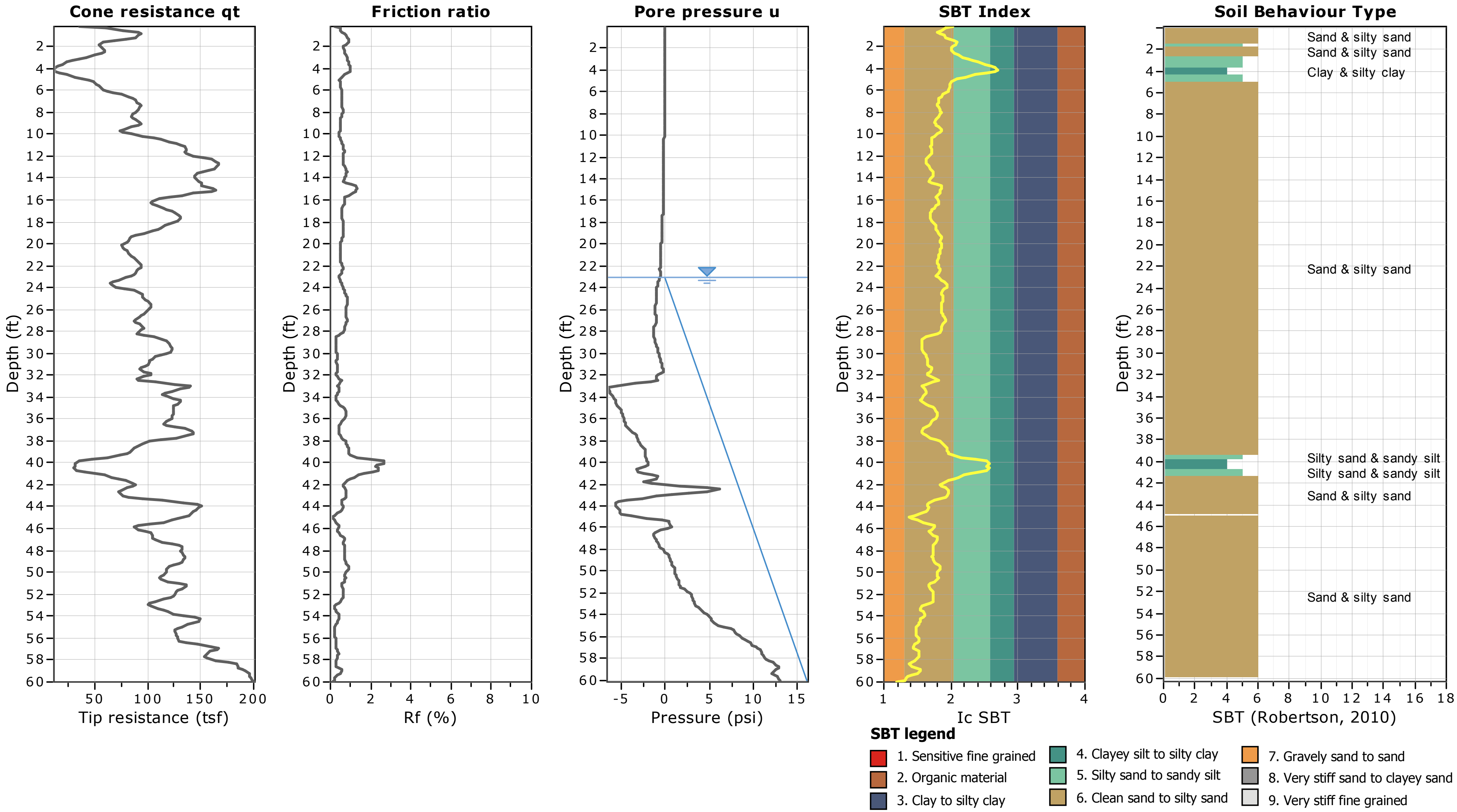
Soil Sensitivity factor, N_S: 7.00

User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-3
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 66.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

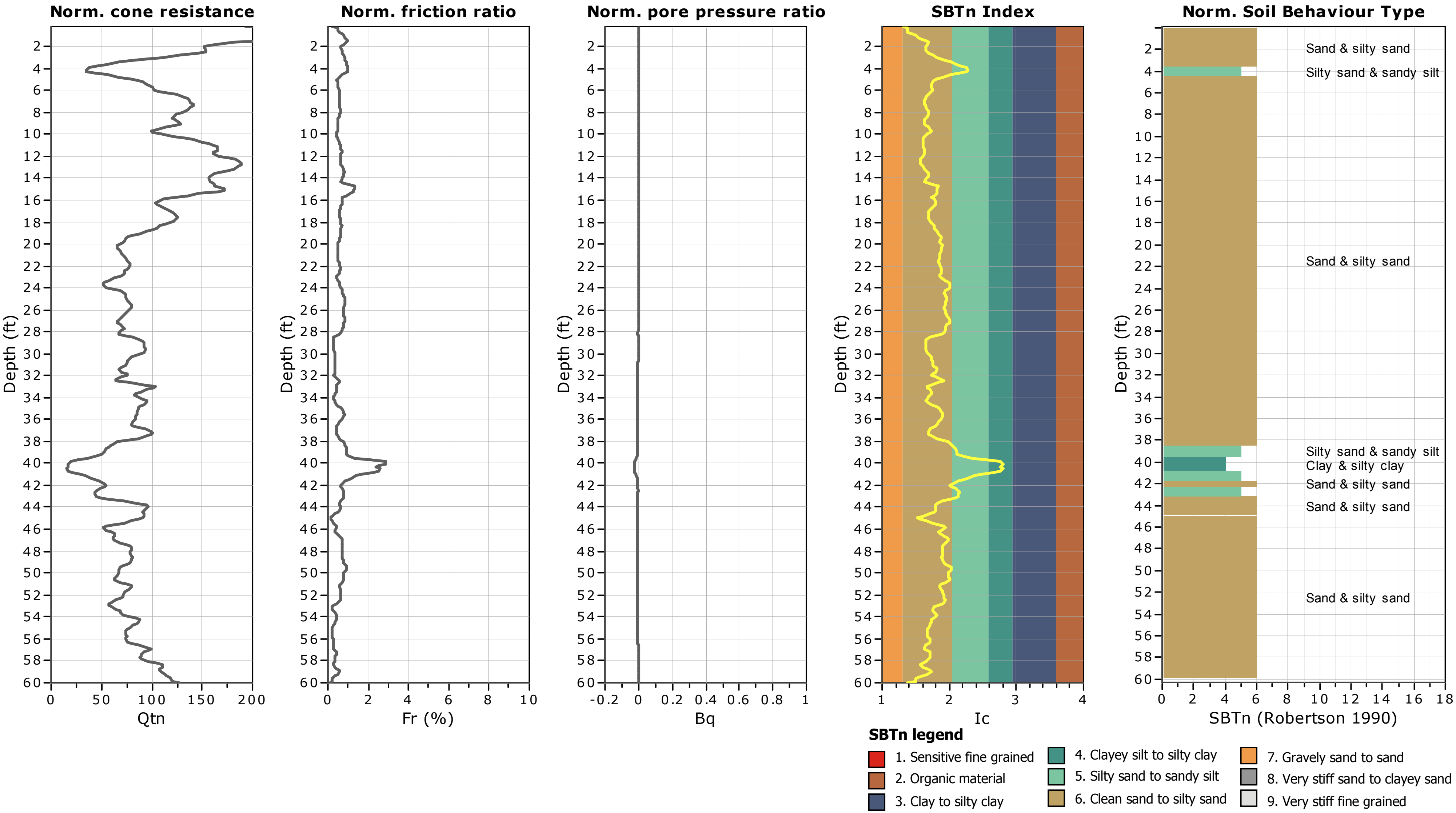
Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-3
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 66.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

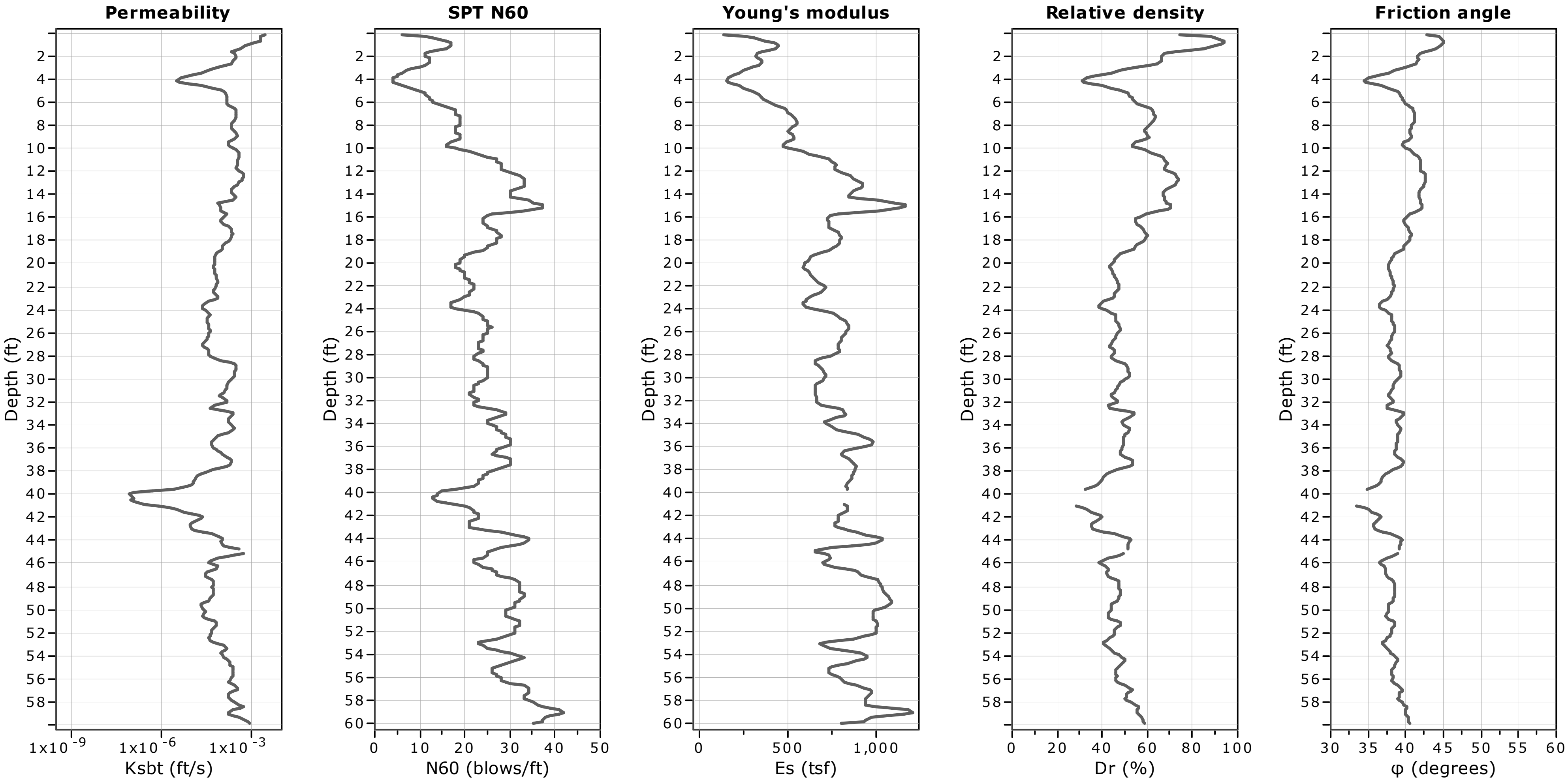
Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-3
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 66.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**

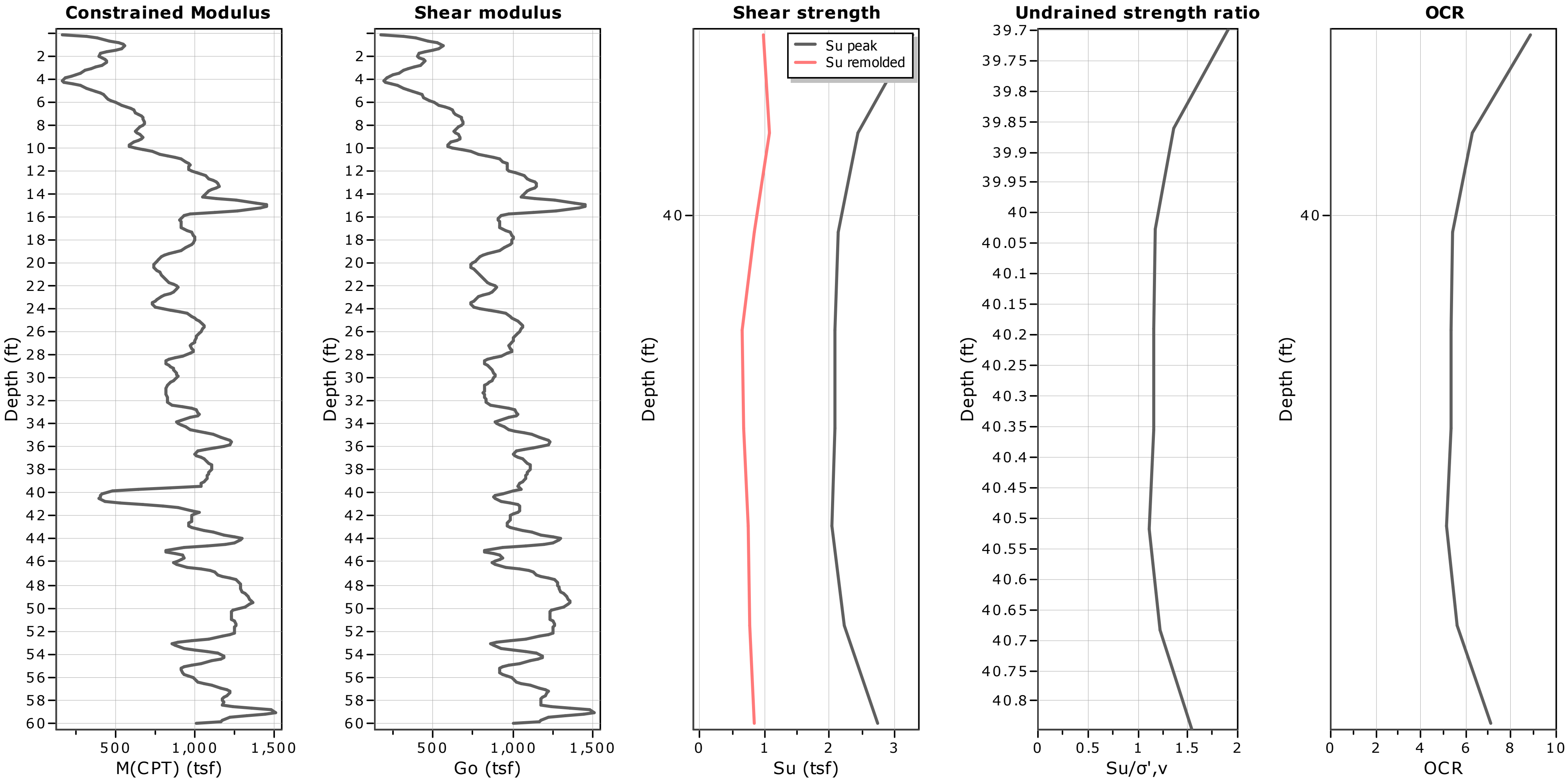


Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative desnisty constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-3
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 66.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**

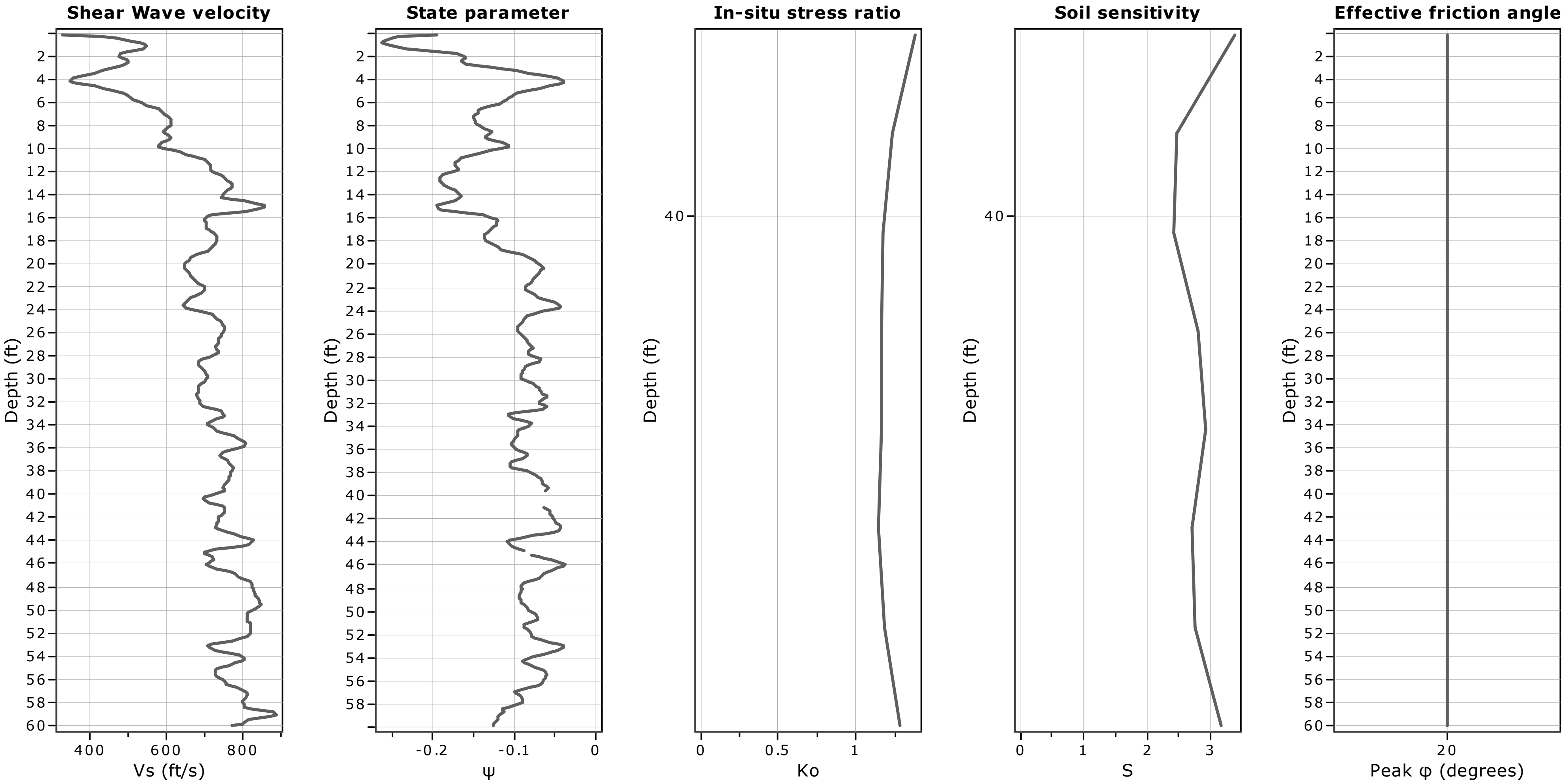


Calculation parameters
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14
OCR factor for clays, N_{kt}: 0.33
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**

CPT: CPT-3
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 66.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering



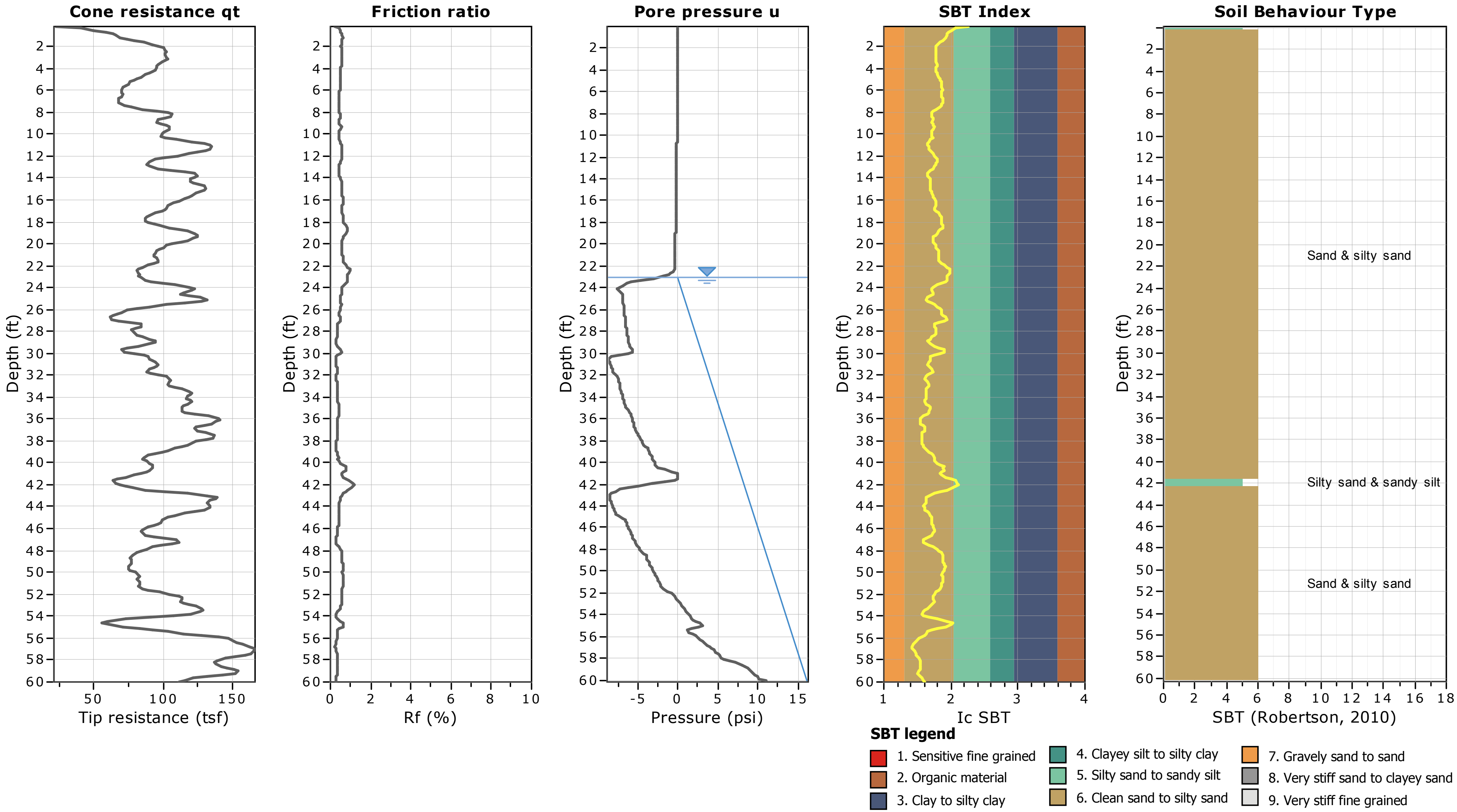
Calculation parameters

Soil Sensitivity factor, N_S: 7.00
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-4
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

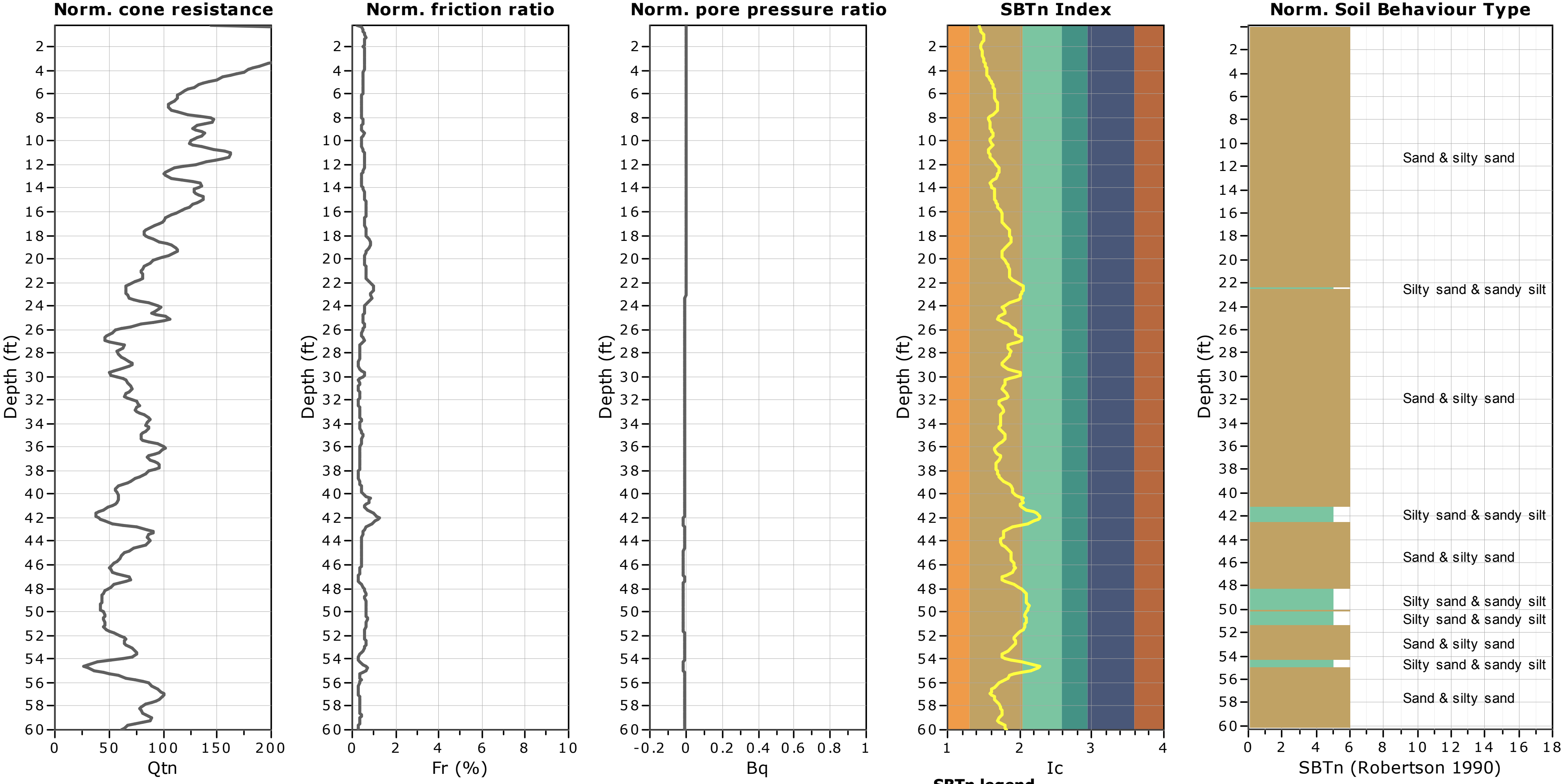
Project: Nagata Site
Location: 4617 North River Road, Oceanside, California



Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-4
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



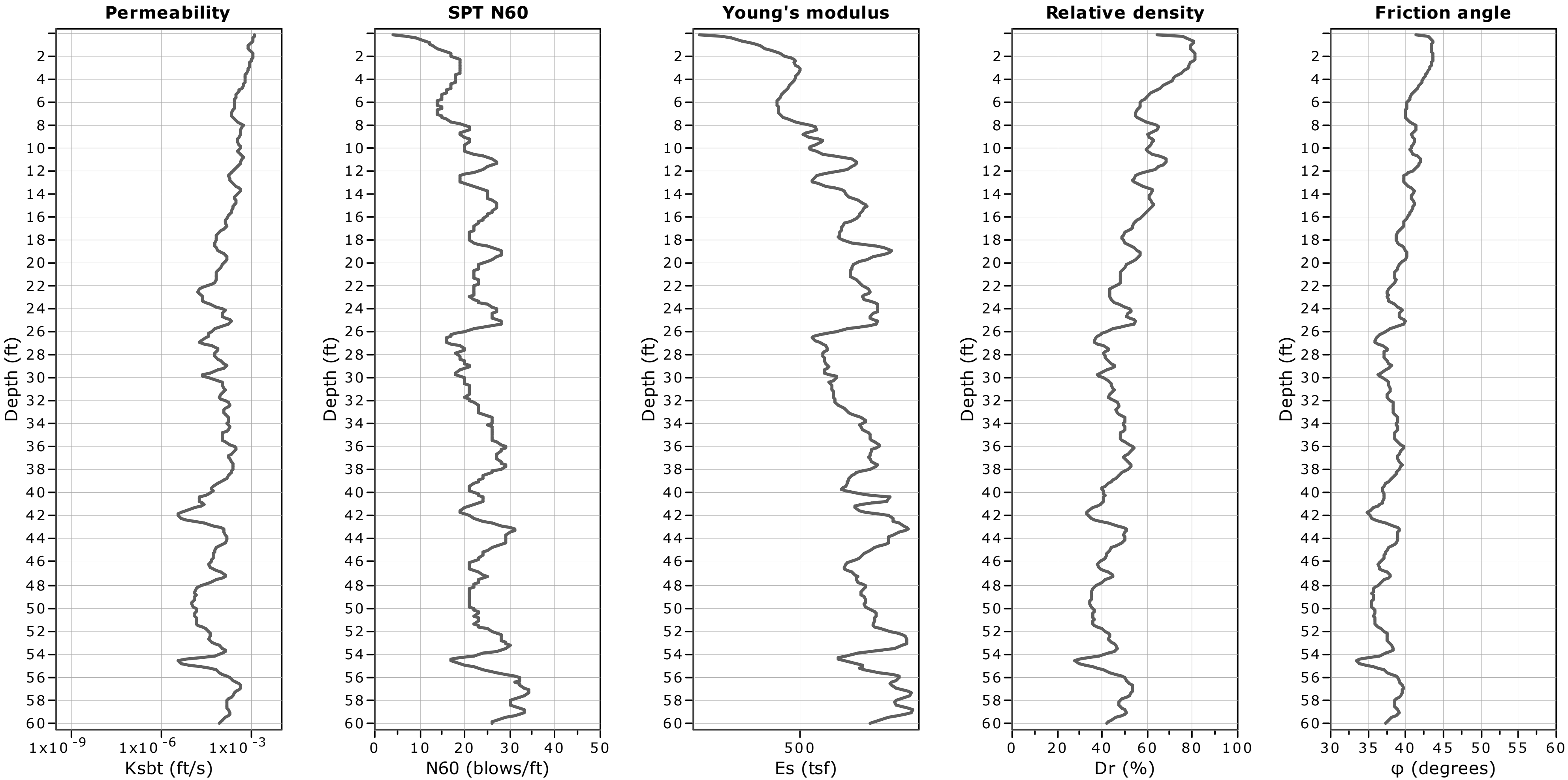
SBTn legend

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

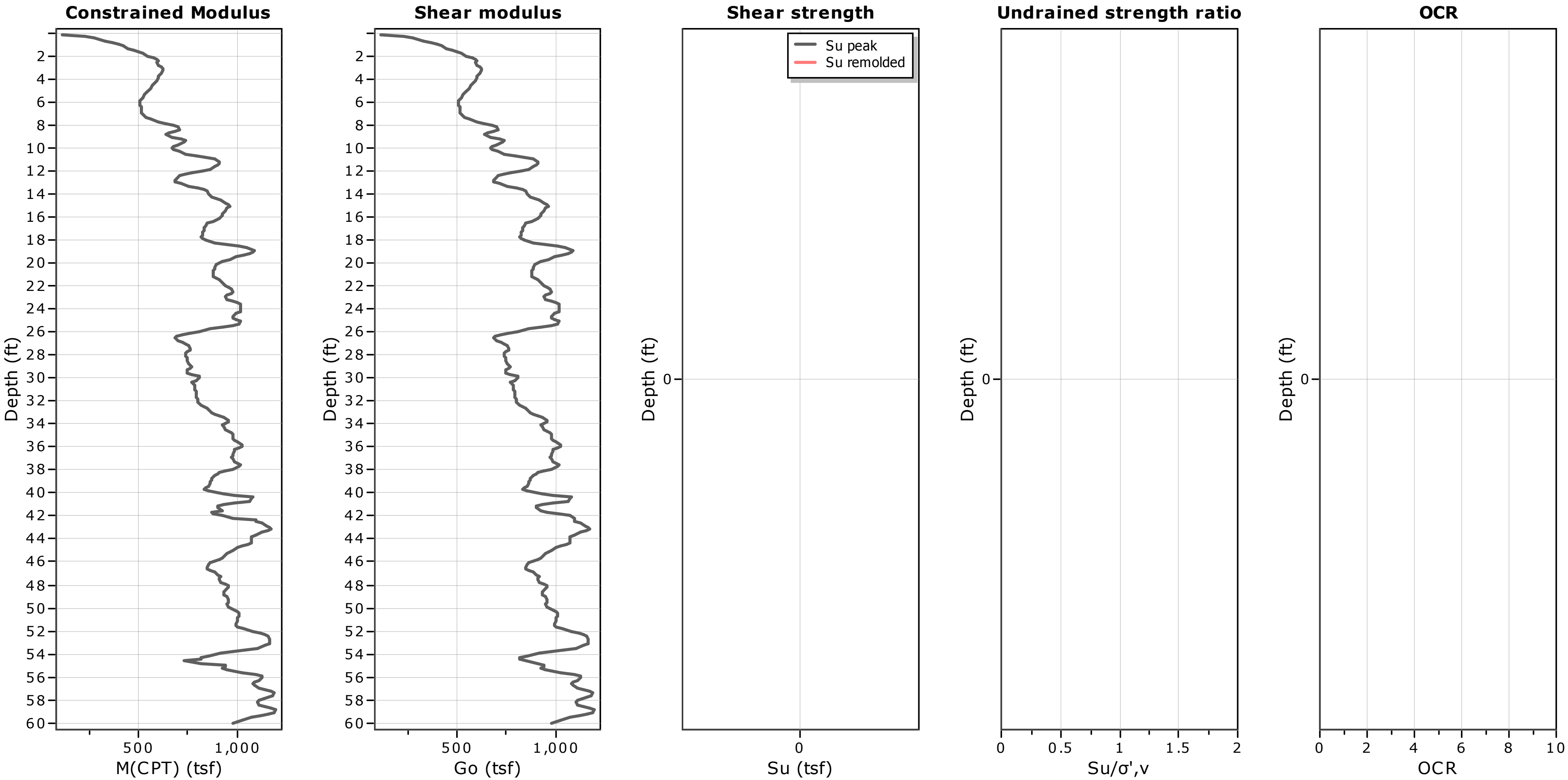
Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-4
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



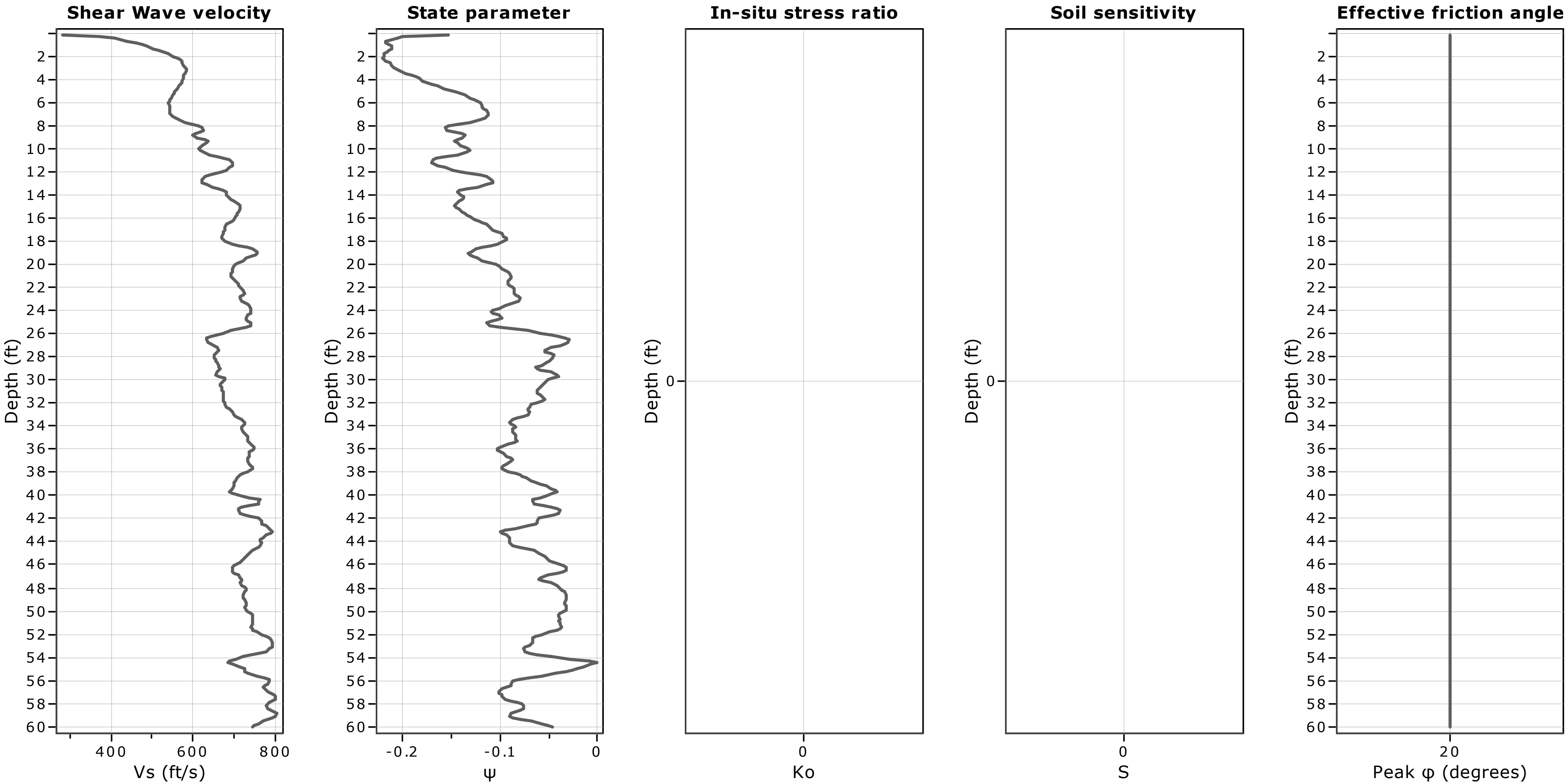
Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative desnisty constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data



Calculation parameters

Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14

OCR factor for clays, N_{kt}: 0.33
User defined estimation data



Calculation parameters

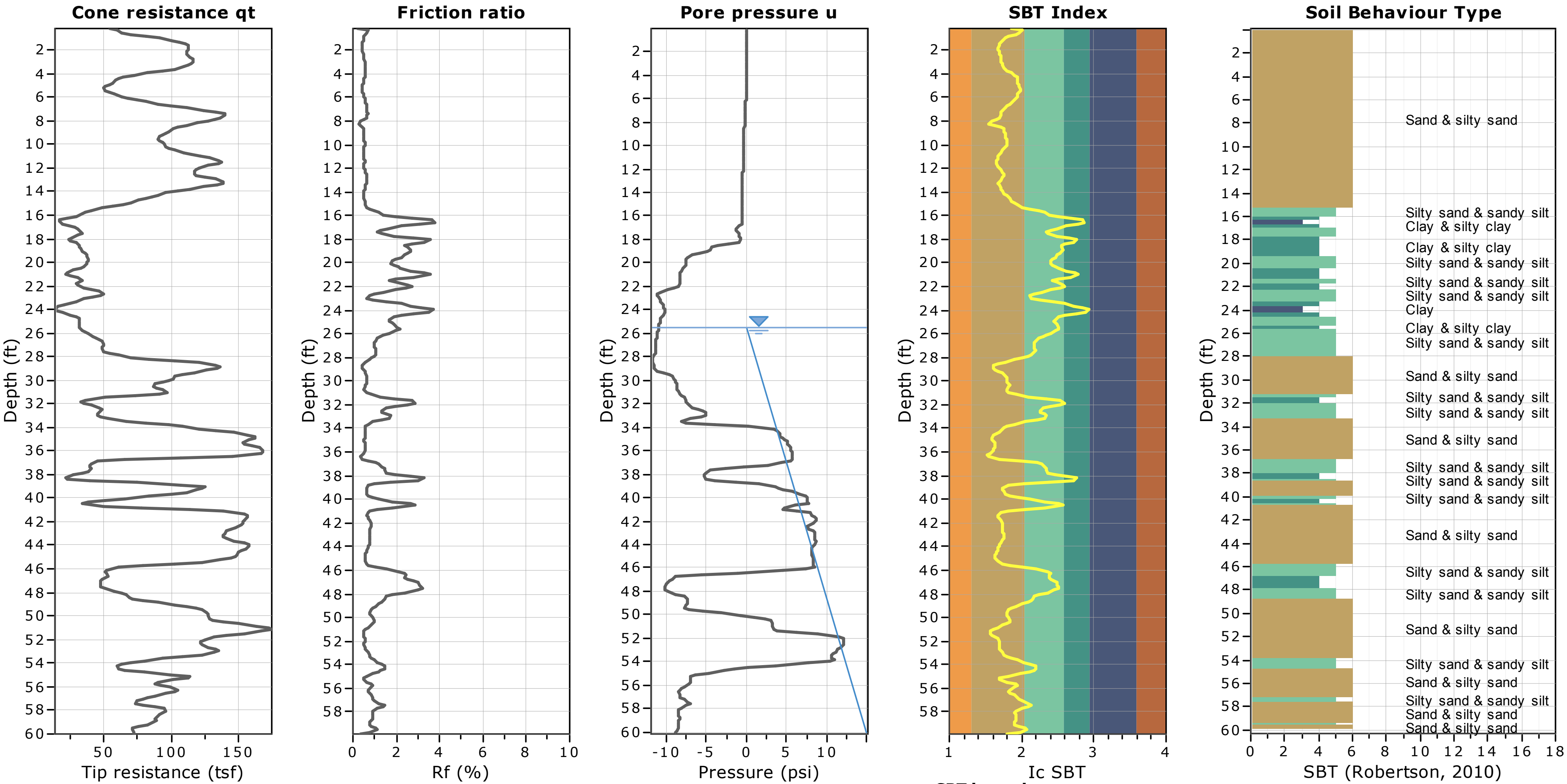
Soil Sensitivity factor, N_S: 7.00

User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-5
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



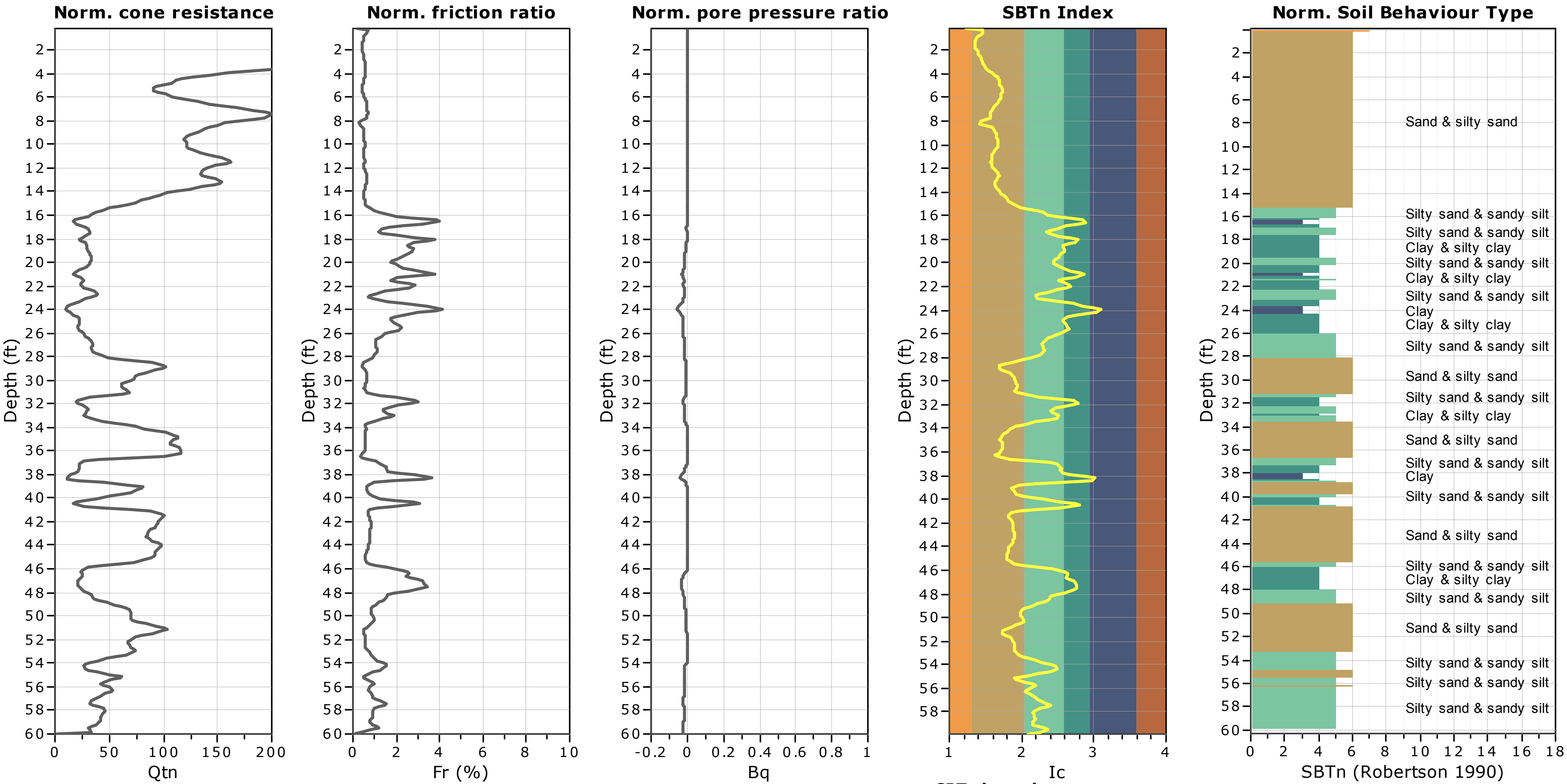
SBT legend

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

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-5
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



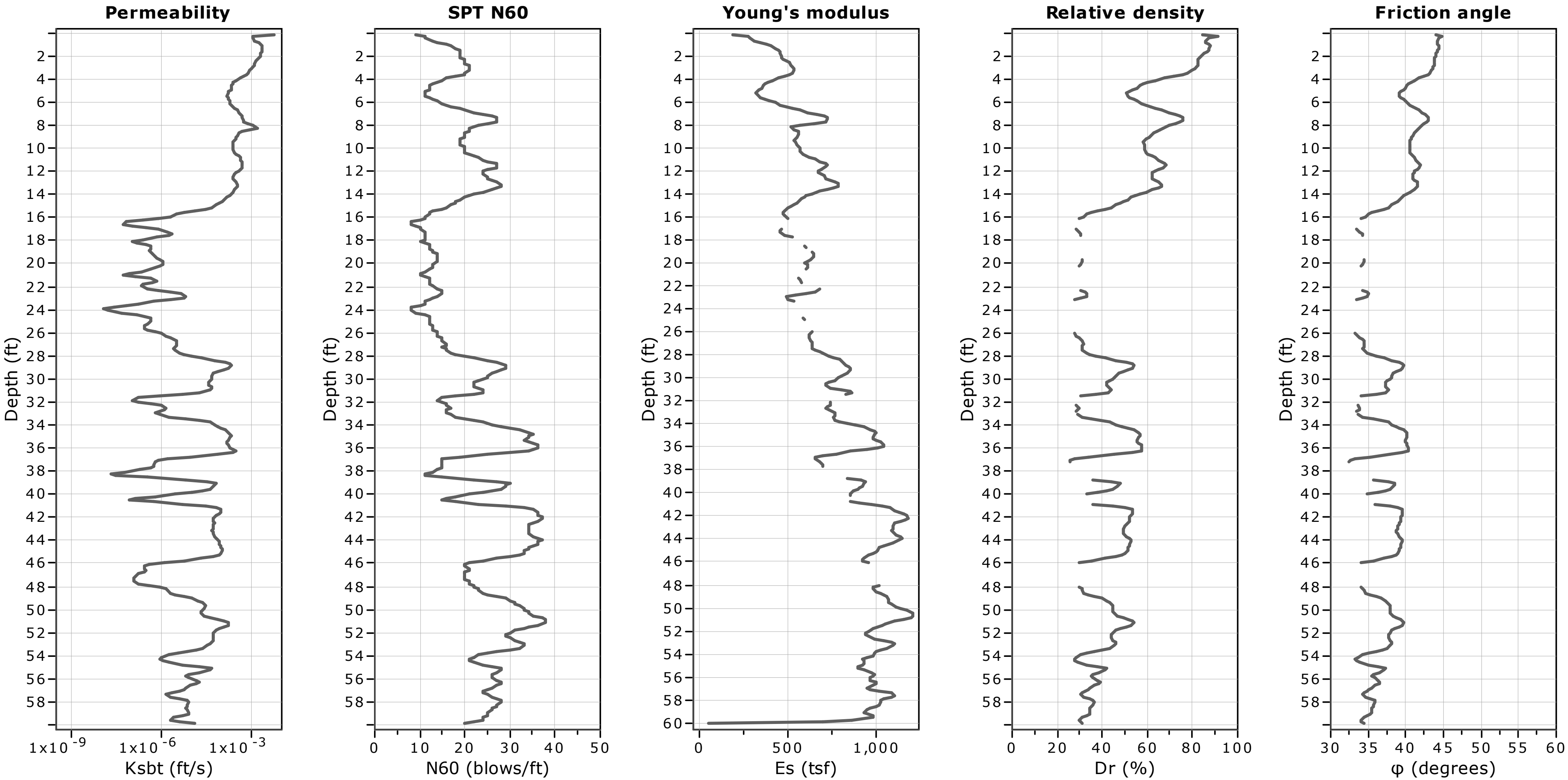
SBTn legend

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

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-5
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**

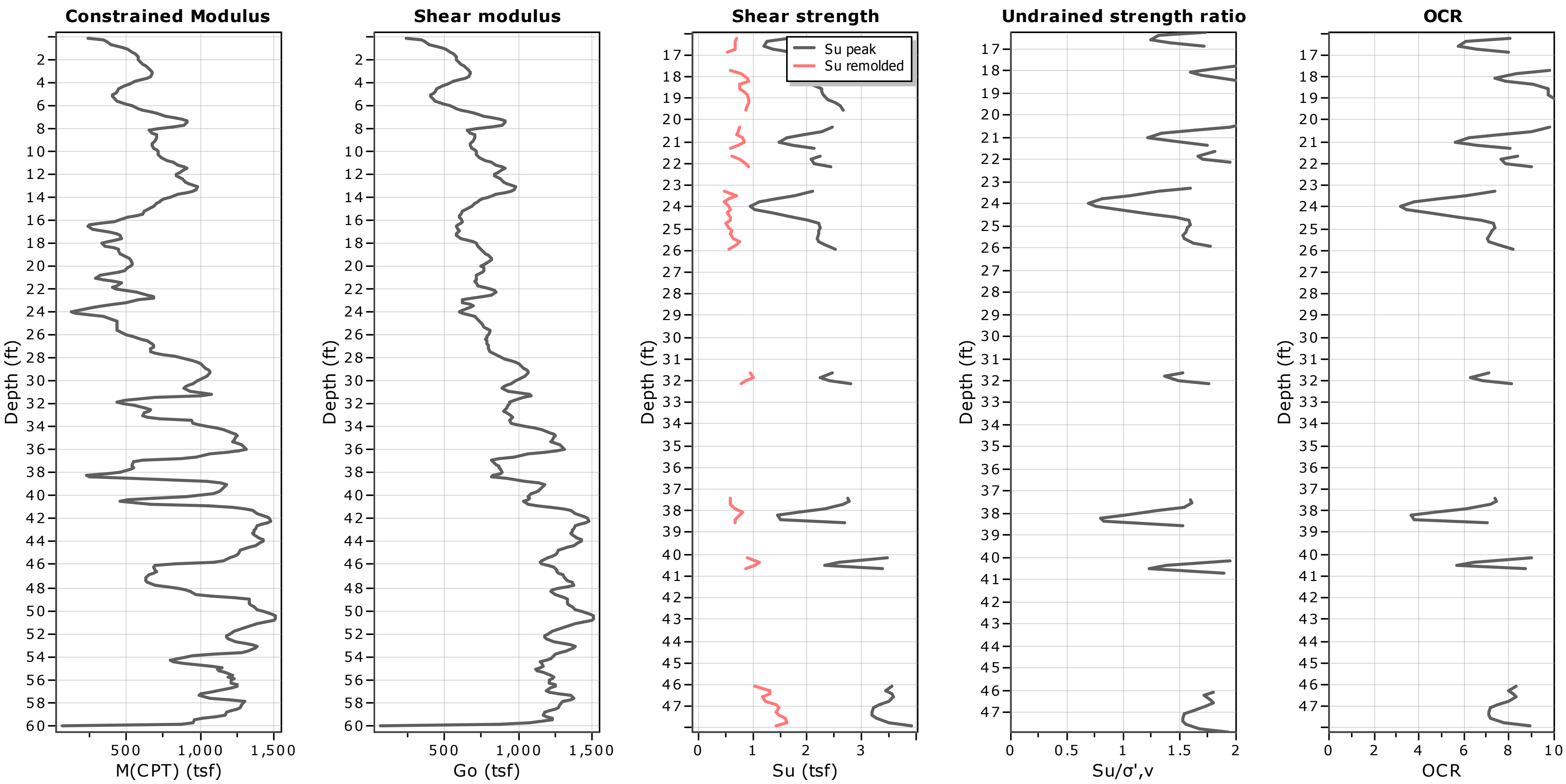


Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative desnisty constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-5
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: Nagata Site
Location: 4617 North River Road, Oceanside, California

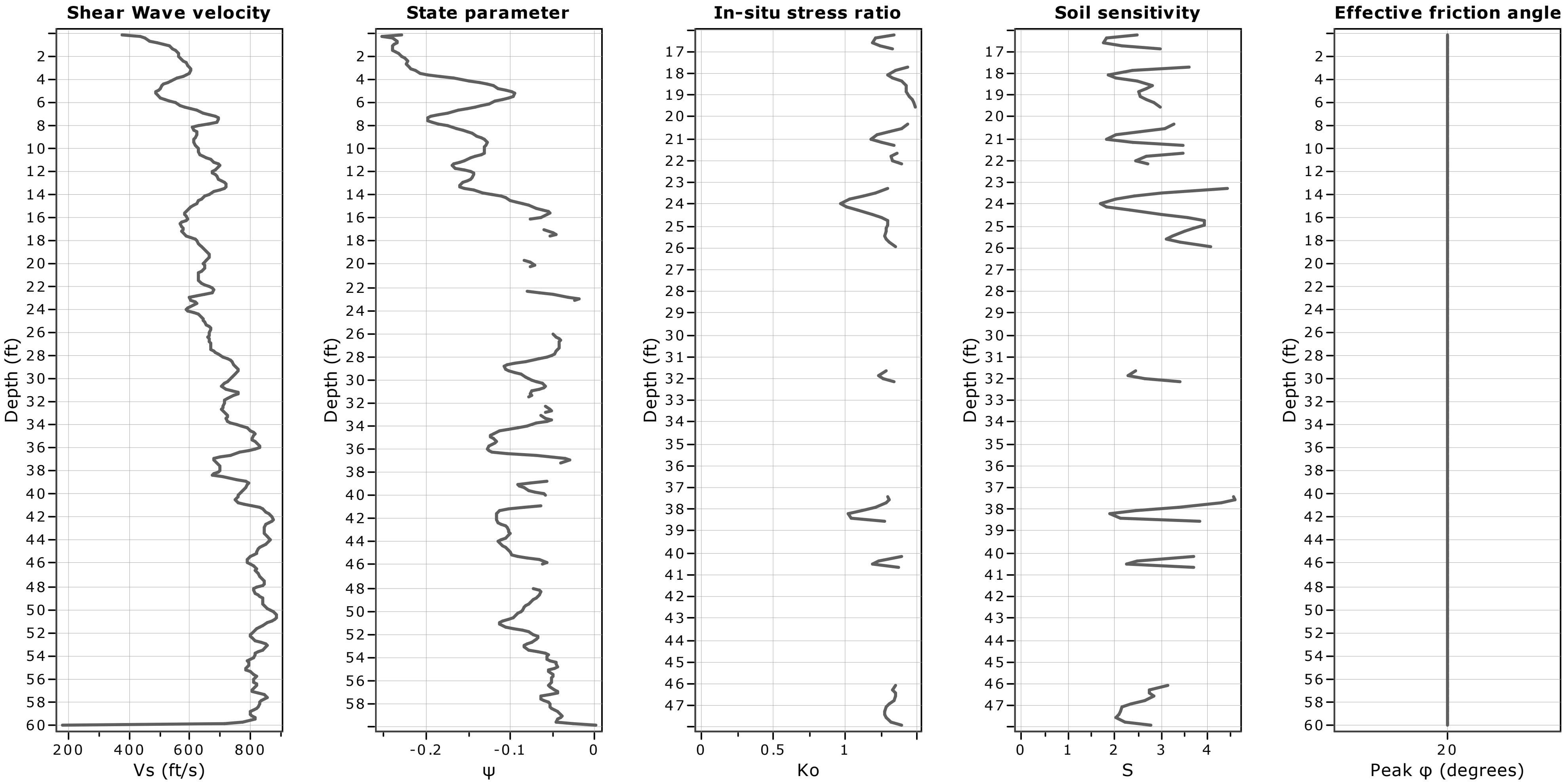


Calculation parameters
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14
OCR factor for clays, N_{kt}: 0.33
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-5
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 68.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



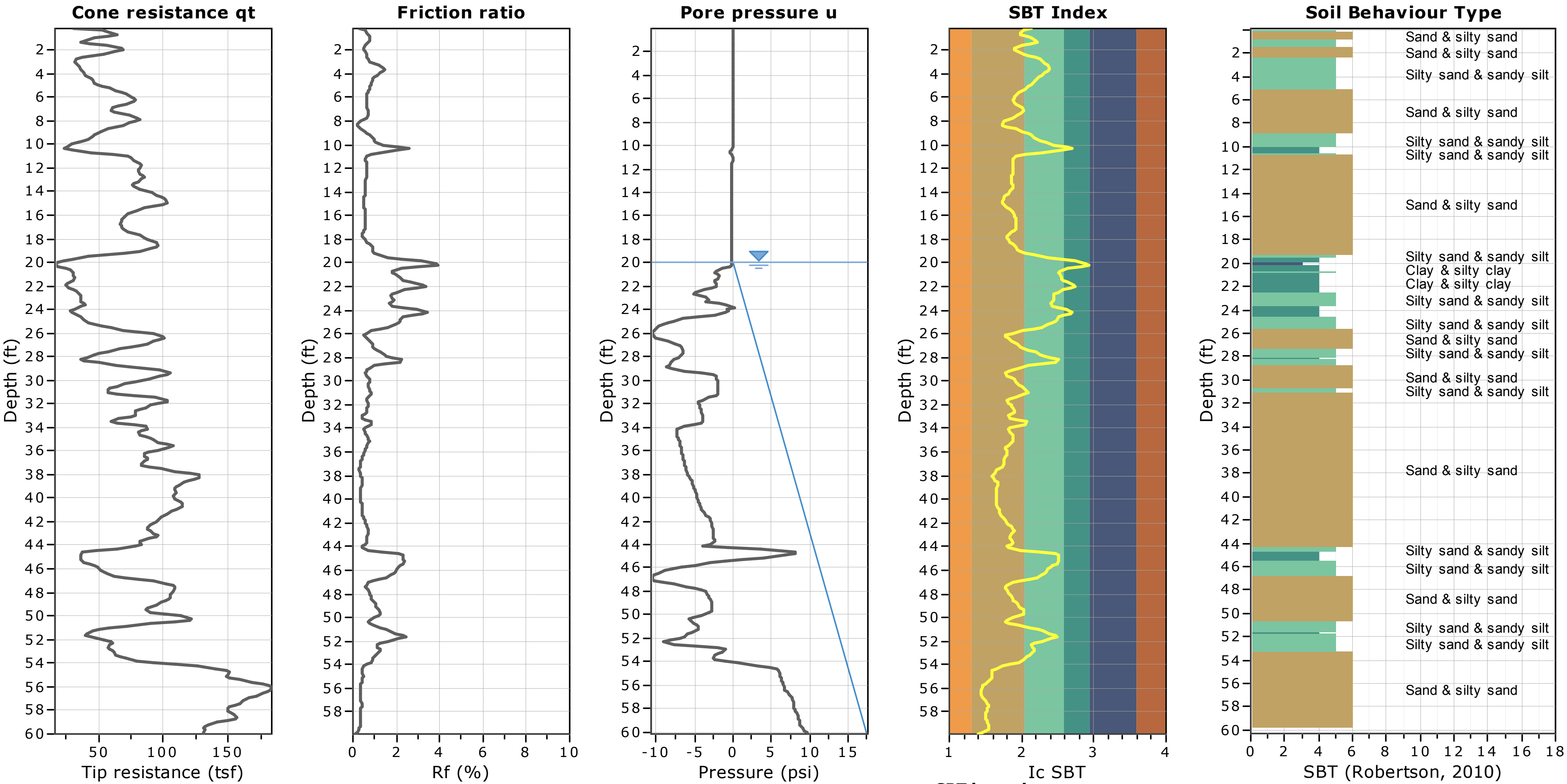
Calculation parameters

Soil Sensitivity factor, N_S: 7.00
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**

CPT: CPT-6
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 71.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering



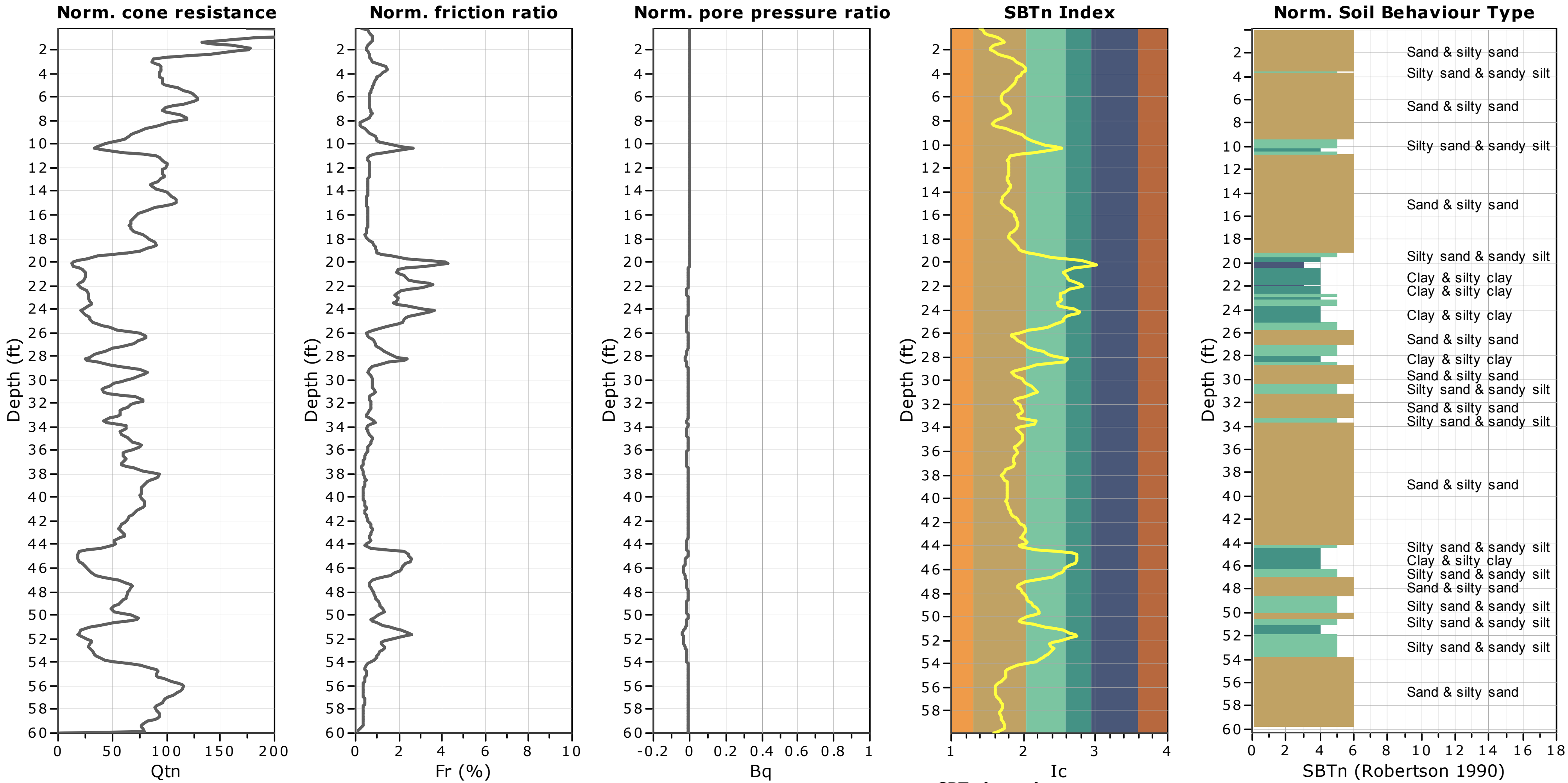
SBT legend

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

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-6
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 71.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



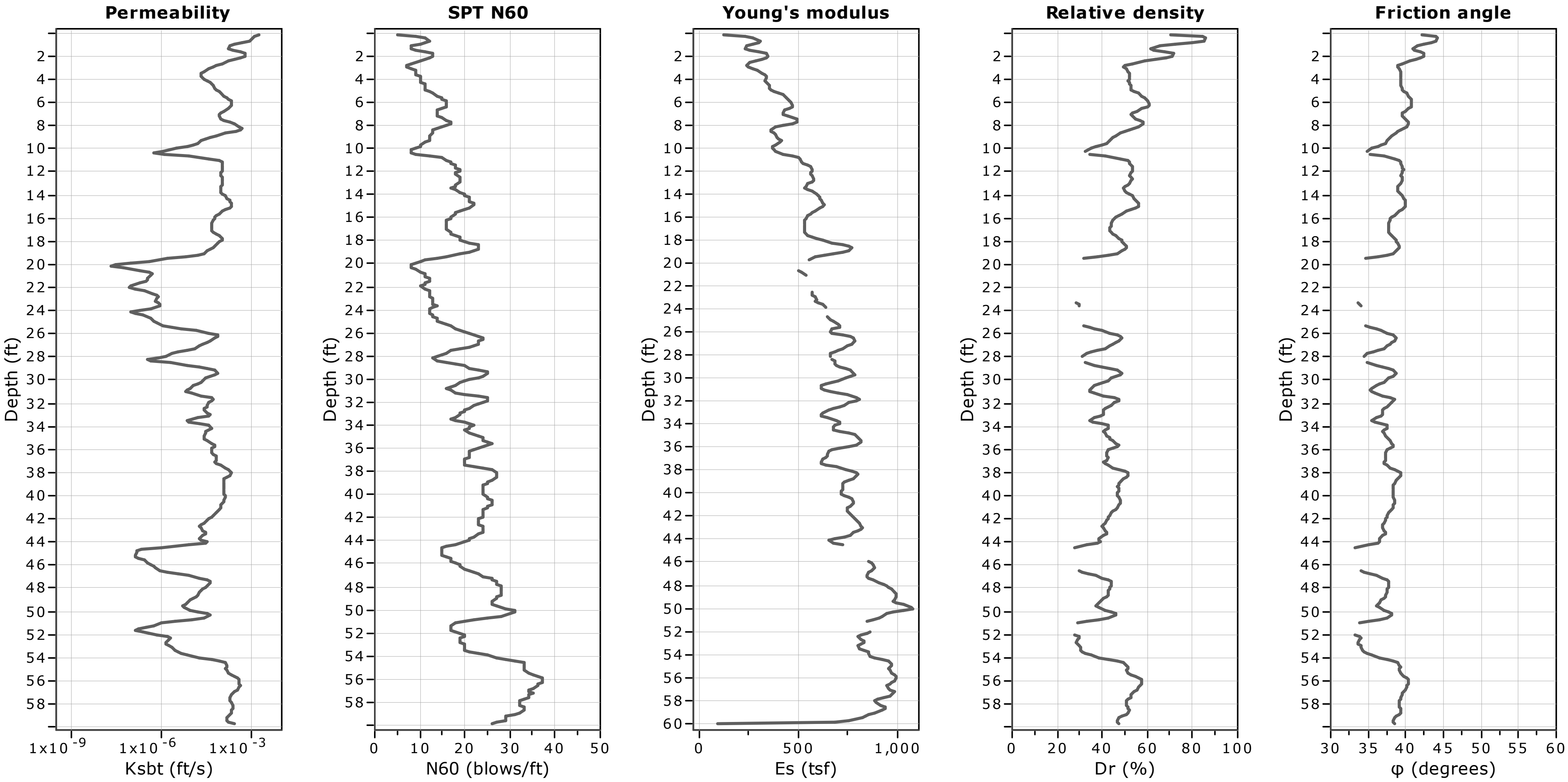
SBTn legend

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

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**

CPT: CPT-6
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 71.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

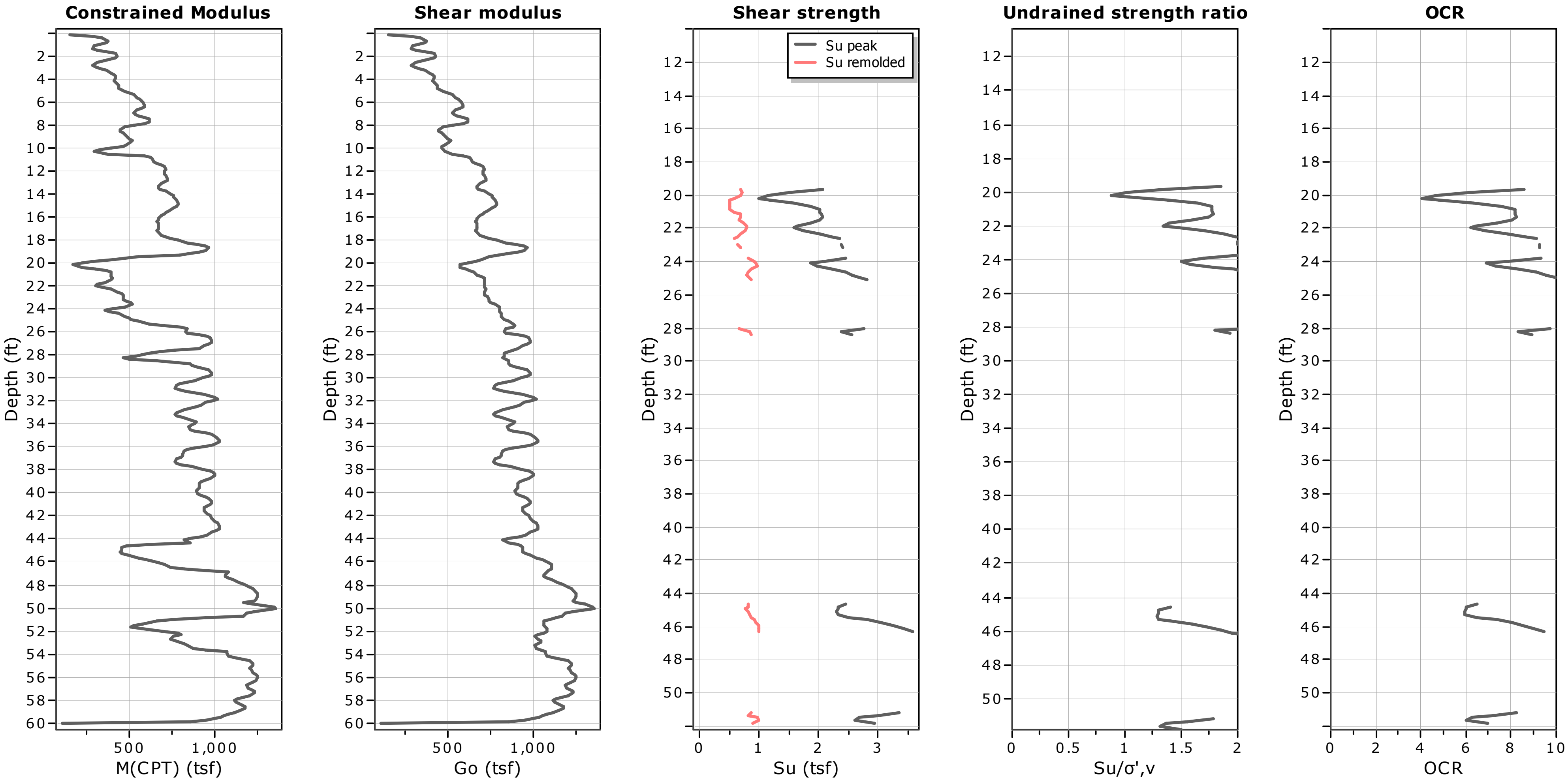


Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative desnisty constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-6
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 71.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**

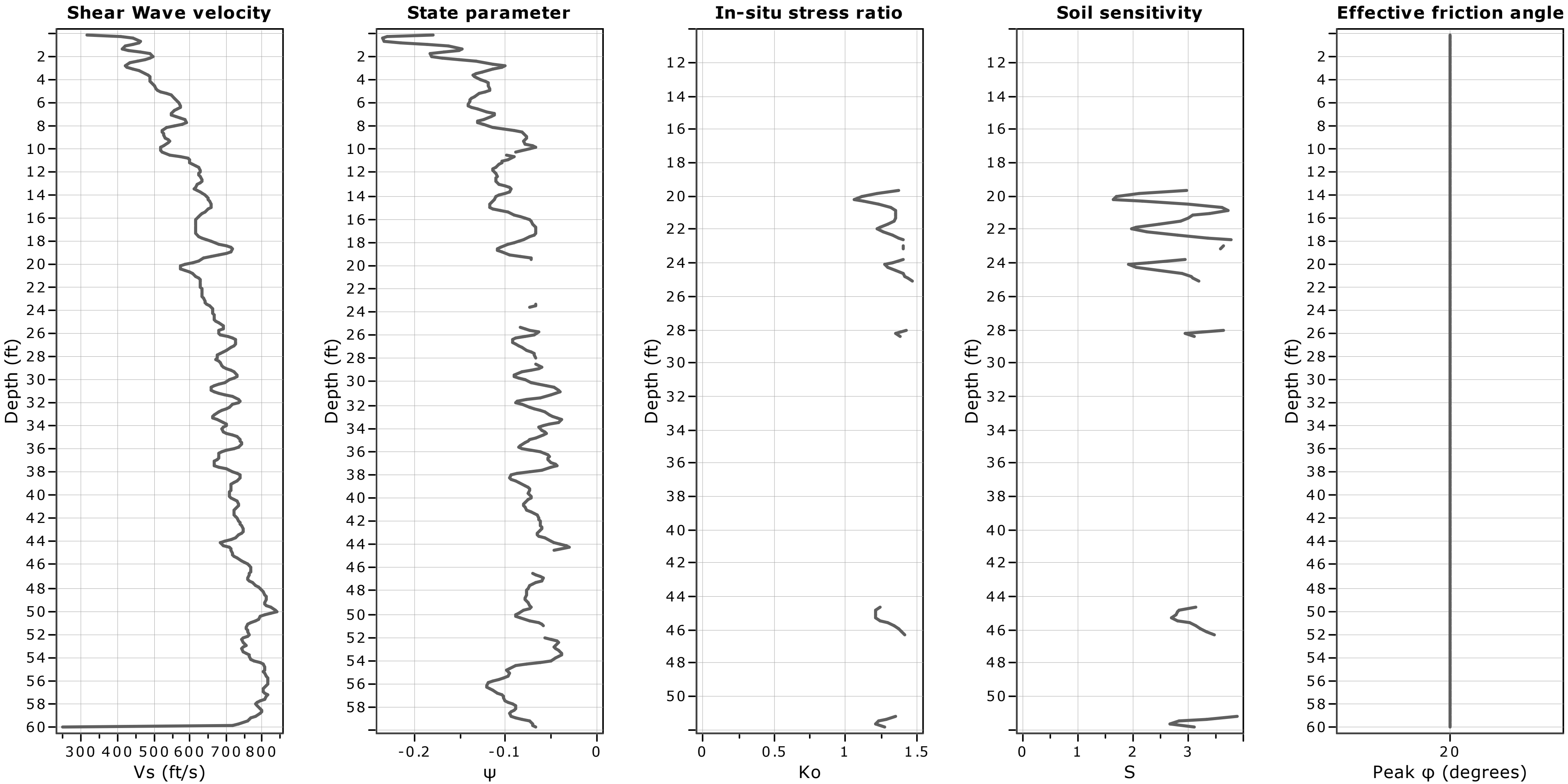


Calculation parameters
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14
OCR factor for clays, N_{kt}: 0.33
User defined estimation data

Christian Wheeler Engineering
3980 Home Avenue
San Diego, California 92105

CPT: CPT-6
Total depth: 60.04 ft, Date: 10/7/2015
Surface Elevation: 71.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator: Kehoe Testing & Engineering

Project: **Nagata Site**
Location: **4617 North River Road, Oceanside, California**



Calculation parameters

Soil Sensitivity factor, N_S: 7.00
User defined estimation data

Appendix B

Data from Previous Studies (CWE, 2005)

LOG OF TEST BORING NUMBER B-1

Date Excavated:	6/14/2005	Logged by:	AKN
Equipment:	CME-55	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	19 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		Artificial Fill (Qaf): Light brownish-gray, dry to damp, loose, SILTY SAND (SM), with gravels.						SA, SO_4
2		**Topsoil:** Light to medium brown, damp to moist, loose to medium dense, SILTY SAND (SM), fine-grained.						
			Cal		24			
4		**Alluvium (Qal):** Light gray, damp, medium dense, POORLY-GRADED SAND (SP), medium-grained, friable, micaceous, with slight iron staining.	Cal		28	2.5	104.2	
6								SA, MD, SO_4
8								
10			Cal		22	3.1	101.1	
12								
14			Cal		31	4.5	96.5	
16		At 15 feet becomes moist.						
18		At 18 feet becomes wet.						
20		At 19 feet becomes saturated.	Cal		63	2.8	87.4	

Boring terminated at 20 feet. Boring properly backfilled with 6.5 cubic feet of bentonite grout mix.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT

4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	2

LOG OF TEST BORING NUMBER B-2

Date Excavated:	6/14/2005	Logged by:	AKN
Equipment:	CME-55	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	N/A
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
2		**Artificial Fill (Qaf):** Light brownish-gray, dry to damp, loose, SILTY SAND (SM). Concrete pieces present from 1-2 feet.						
4		**Topsoil:** Light to medium brown, dry, loose, SILTY SAND (SM), fine-grained.	Cal		24	2.8	87.4	
6		**Alluvium (Qal):** Light to medium gray, damp, loose to medium dense, POORLY-GRADED SAND (SP), medium-grained, friable.	Cal		18	4.4	96.9	
8								
10		At 9 feet becomes moist, medium dense.	Cal		27	6.3	104.7	
12								
14			Cal		26	10.4	97.3	
16		At 15 feet becomes very moist.						
18			Cal		41	18.6	108.4	
20		Boring terminated at 19 feet.						

CHRISTIAN WHEELER
ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT
4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	3

LOG OF TEST BORING NUMBER B-3

Date Excavated:	6/14/2005	Logged by:	AKN
Equipment:	CME-55	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	N/A
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		Artificial Fill (Qaf): Light brownish-gray, damp to moist, loose, SILTY SAND (SM), with minor trash.						R-Val
2		**Topsoil:** Light to medium brown, moist, loose, SILTY SAND (SM), fine to medium-grained, micaceous.	Cal		18	8.4	100.2	
4		**Alluvium (Qal):** Light to medium gray, damp, medium dense, SILTY SAND-POORLY-GRADED SAND (SM-SP), medium-grained, friable, micaceous, with trace gravels and slight iron staining.	Cal		20	3.6	103.5	SA
6								
8								
10			Cal		25	3.4	104.9	
12								
14			Cal		27	3.4	101.4	
16								
18		At 18 feet grades to wet.	Cal		37	17.2	106.3	
20		Boring terminated at 19 feet.						

CHRISTIAN WHEELER ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT

4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	4

LOG OF TEST BORING NUMBER B-4

Date Excavated:	6/14/2005	Logged by:	AKN
Equipment:	CME-55	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	17½ feet
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0–2		**<u>Topsoil:</u>** Light grayish-brown, damp, loose to medium dense, SILTY SAND (SM), micaceous, with plastic trash.						
2		**<u>Alluvium (Qal):</u>** Light to medium gray, damp, medium dense, POORLY-GRADED SAND (SP), medium-grained, micaceous, friable.	Cal		26	2.4	97.8	
4			Cal		25	4.2	101.5	MD, SA, SO_4, DS
7		At 7 feet becomes moist.						
10			Cal		30	5.1	98.6	
14½		At 14½ feet becomes very moist.	Cal		42	7.7	106.6	
16		At 16 feet becomes wet.						
17½	▽	At 17½ feet becomes saturated.						
20			Cal		38	18.6	109.8	

Boring terminated at 20 feet. Boring properly backfiled with 6.5 cubic feet of bentonite grout mix.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT

4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	5

LOG OF TEST BORING NUMBER B-5

Date Excavated:	6/14/2005	Logged by:	AKN
Equipment:	CME-55	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	17½ feet
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0–2		**Topsoil:** Light brownish-gray, damp, loose, SILTY SAND (SM).						
2		**Alluvium (Qal):** Light to medium gray, dry to damp, loose to medium dense, POORLY-GRADED SAND (SP), medium-grained, micaceous, friable.	Cal		26	1.9	95.6	
4			Cal		13	2.7	100.5	
6								
8								
10			Cal		13	3.8	95.9	
12								
14		At 13 feet becomes moist.	Cal		28	13.7	111.5	
16	▽							
18		At 17½ feet becomes saturated.						
20			Cal		47	15.6	111.9	

Boring terminated at 20 feet. Boring properly backfilled with 6.5 cubic feet of bentonite grout mix.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT
4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	6

LOG OF TEST BORING NUMBER B-6

Date Excavated:	6/15/2005	Logged by:	AKN
Equipment:	CME-55	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	N/A
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
2		**Artificial Fill (Qaf):** Light to medium brownish-gray, dry, medium dense, SILTY SAND (SM), with trace gravels and minor metal debris. At 1 foot becomes damp.	Cal		38	3.2	101.6	R-Val
4		**Topsoil:** Light to medium brown, dry, loose, SILTY SAND (SM), fine to medium-grained.	Cal		23	0.8	99.6	
6		**Alluvium (Qal):** Light to medium gray, dry, medium dense, POORLY-GRADED SAND (SP), medium-grained, friable.						
8		At 8 feet becomes damp.						
10			Cal		38	2.5	99.8	
12								
14			Cal		29	2.9	102.8	
16		At 15 feet becomes moist.						
18		At 18 feet becomes wet.	Cal		46	20.6	102.8	
20		Boring terminated at 19 feet.						

CHRISTIAN WHEELER
ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT

4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	7

LOG OF TEST BORING NUMBER B-7

Date Excavated:	6/15/2005	Logged by:	AKN
Equipment:	CME-55	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	N/A
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0–3		**Artificial Fill (Qaf):** Light to medium brown, moist, medium dense, SILTY SAND (SM), with AC debris.	Cal		30	13.0	114.4	SA, MD
3–19		**Alluvium (Qal):** Light to medium gray, moist, medium dense, POORLY-GRADED SAND (SP), medium-grained, friable.	Cal		23	5.6	99.3	
6								SA, SO_4
10			Cal		21	3.1	98.6	
15			Cal		45	3.9	105.8	
16		At 15 feet becomes moist.						
19		At 18 feet becomes wet.	Cal		35	20.0	105.2	
20		Boring terminated at 19 feet.						

PROPOSED RESIDENTIAL DEVELOPMENT

4617 North River Ranch Road, Oceanside, California

CHRISTIAN WHEELER
ENGINEERING

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	8

LOG OF TEST BORING NUMBER B-8 (Continued)

Date Excavated:	7/11/2005	Logged by:	AKN
Equipment:	IR-300	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	19 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
22		**<u>Alluvium (Qal):</u>** Light to medium gray, saturated, medium dense, POORLY-GRADED SAND (SP), medium to coarse to very coarse-grained, friable.						
24			SPT		19			SA
26								
28								
30		At 29 feet becomes medium to coarse-grained.	SPT		27			
32								
34			SPT		29			
36								
38								
40			SPT		29			SA

Boring continued on Plate No. 11.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT

4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	10

LOG OF TEST BORING NUMBER B-8 (Continued)

Date Excavated:	7/11/2005	Logged by:	AKN
Equipment:	IR-300	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	19 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
42		**Alluvium (Qal):** Light to medium gray, saturated, medium dense, POORLY GRADED SAND (SP), medium to coarse-grained, friable.						
44, 46, 48		Medium gray, saturated, dense, POORLY-GRADED SAND-SILTY SAND (SP-SM), medium-grained, slightly micaceous, friable.	SPT		35			
50, 52		Medium brownish-gray, saturated, dense, SILTY SAND (SM), fine to medium-grained, micaceous.	SPT		30			SA
54, 56, 58		Light to medium gray, saturated, very dense, POORLY GRADED SAND (SP), coarse-grained, friable.	SPT		77			SA
60		Boring properly backfilled with 20.5 cubic feet of bentonite grout mix.	SPT		60			SA

Boring terminated at 60 feet.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT

4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	11

LOG OF TEST BORING NUMBER B-8

Date Excavated:	7/11/2005	Logged by:	AKN
Equipment:	IR-300	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	19 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0–2		**Artificial Fill (Qaf):** Light brownish-gray, damp, loose to medium dense, SILTY SAND (SM), with minor plastic trash.						
2–4		**Topsoil:** Light to medium brown, moist, medium dense, SILTY SAND (SM), fine to medium-grained.	Cal		55			
4		**Alluvium (Qal):** Light gray, moist, medium dense, POORLY-GRADED SAND (SP), medium to coarse-grained, friable, with iron staining.	Cal		33			
10			Cal		18			
14			Cal		42			
18	▽	At 18 feet becomes very moist, medium dense.						
20		At 19 feet becomes saturated.	SPT		15			

Boring continued on Plate No. 10.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT
4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	9

LOG OF TEST BORING NUMBER B-9 (Continued)

Date Excavated:	7/12/2005	Logged by:	AKN
Equipment:	IR-300	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	23½ feet
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		<u>Alluvium (Qal):</u> Light to medium gray, very moist, loose, POORLY GRADED SAND (SP), medium to coarse-grained, friable.						
22			SPT		8			
	▽	Medium to dark gray, very moist, loose, SILTY SAND (SM), very fine to fine-grained, micaceous.						
24		At 23½ feet becomes saturated.	SPT		8			SA
26								
28		Medium gray, saturated, medium dense, SILTY SAND-POORLY-GRADED SAND (SM-SP), medium-grained, slightly micaceous, friable.						
30			SPT		14			
32								
34			SPT		16			SA
36								
38								
40			SPT*		23			

Boring continued on Plate No. 14.

* No sample recovery.

CHRISTIAN WHEELER ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT
4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	13

LOG OF TEST BORING NUMBER B-9 (Continued)

Date Excavated:	7/12/2005	Logged by:	AKN
Equipment:	IR-300	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	23½ feet
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
42–44		**<u>Alluvium (Qal):</u>** Medium gray, saturated, medium dense, , SILTY SAND-POORLY-GRADED SAND (SM-SP), medium-grained, friable, slightly micaceous.	SPT		10			SA
46–52		Medium to dark gray, saturated, medium dense, SILTY SAND-SANDY SILT (SM-ML), very fine to fine-grained, micaceous.	SPT		12			
54–58		Light to medium gray, saturated, medium dense to dense, SILTY SAND-POORLY-GRADED SAND (SM-SP), medium to coarse-grained, slightly micaceous.	SPT		29			SA
60		Boring properly backfilled wth 20.5 cubic feet of bentonite grout mix.	SPT*		32			

Boring terminated at 60 feet.

* No sample recovery.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT

4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	14

LOG OF TEST BORING NUMBER B-9

Date Excavated:	7/12/2005	Logged by:	AKN
Equipment:	IR-300	Project Manager:	CHC
Existing Elevation:	N/A	Depth to Water:	23½ feet
Finish Elevation:	N/A	Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		Artificial Fill (Qaf): Medium grayish-brown, damp, loose, SILTY SAND (SM), with trace gravels. At 1 foot becomes moist.						
2		**Topsoil:** Medium grayish-brown, moist, medium dense, SILTY SAND (SM).	Cal		19			
4		**Alluvium (Qal):** Medium to dark gray, moist, medium dense, SILTY SAND (SM), very fine to fine-grained, micaceous.	Cal		17			
6		Medium gray, moist, medium dense, SILTY SAND-POORLY-GRADED SAND (SM-SP), medium-grained, slightly micaceous, friable.						
8								
10		Medium to dark gray, moist, loose, SILTY SAND-SANDY SILT (SM-ML), very fine to fine-grained, micaceous.	Cal		12			
12								
14		Light to medium gray, moist, medium dense, POORLY GRADED SAND (SP), medium to coarse-grained, friable.	Cal		25			
16								
18								
20			Cal*		24			

Boring continued on Plate No. 13.

* No sample recovery.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED RESIDENTIAL DEVELOPMENT

4617 North River Ranch Road, Oceanside, California

BY:	HF	DATE:	July 2005
JOB NO. :	2050567	PLATE NO.:	12

LABORATORY TEST RESULTS

PROPOSED NAGATA RANCH RESIDENTIAL DEVELOPMENT

4617 NORTH RIVER ROAD

OCEANSIDE, CALIFORNIA

MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT (ASTM D1557)

Sample Location	**Boring B-1 @ 3'-10'**	**Boring B-4 @ 1½'-10'**	**Boring B-7 @ 0'-3½'**
Sample Description	Dark brown, silty sand	Gray, silty sand	Brown, silty sand
Maximum Density	114.2 pcf	113.3 pcf	120.3 pcf
Optimum Moisture	10.5 %	13.6 %	10.6 %

DIRECT SHEAR (ASTM D3080)

Sample Location	**Boring B-4 @ 1½'-10'**
Sample Type	Remolded to 90 %
Friction Angle	30 °
Cohesion	25 psf

GRAIN SIZE DISTRIBUTION (ASTM D422)

Sample Location	**Boring B-1 @ 0'-3'**	**Boring B-1 @ 3'-10'**	**Boring B-3 @ 3'-10'**
Sieve Size	*Percent Passing*	*Percent Passing*	*Percent Passing*
½"	100		
3/8"	99	100	100
#4	99	100	99
#8	98	98	97
#16	94	89	82
#30	78	58	52
#50	40	26	18
#100	14	10	6
#200	4	3	3

Sample Location	**Boring B-4@ 1½-10'**	**Boring B-7 @ 0'-3½'**	**Boring B-7 @ 3½'-10'**
Sieve Size	*Percent Passing*	*Percent Passing*	*Percent Passing*
3/8"	100	100	
#4	100	99	100
#8	99	99	99
#16	92	97	94
#30	70	86	75
#50	31	52	34
#100	10	23	11
#200	4	13	5

LABORATORY TEST RESULTS (Continued)

GRAIN SIZE DISTRIBUTION (ASTM D422)

Sample Location	**Boring B-8@ 24-25'**	**Boring B-8 @ 39'-40'**	**Boring B-8 @ 49'-50'**
Sieve Size	*Percent Passing*	*Percent Passing*	*Percent Passing*
3/8"	100	100	
#4	100	100	100
#8	100	99	100
#16	97	95	98
#30	82	82	92
#50	35	49	72
#100	12	17	42
#200	6	6	23

Sample Location	**Boring B-8@ 54-55'**	**Boring B-8@ 59-60'**	**Boring B-9 @ 24'-25'**
Sieve Size	*Percent Passing*	*Percent Passing*	*Percent Passing*
3/8"	100	100	
#4	100	100	100
#8	97	97	100
#16	82	84	100
#30	54	59	99
#50	22	31	92
#100	9	15	66
#200	4	8	45

Sample Location	**Boring B-9 @ 34'-35'**	**Boring B-9@ 44-45'**
Sieve Size	*Percent Passing*	*Percent Passing*
3/8"		100
#4	100	99
#8	100	97
#16	99	89
#30	95	69
#50	64	36
#100	24	15
#200	10	7

SOLUBLE SULFATES (CALIFORNIA TEST 417)

Sample Location	**Boring B-1 @ 0'-3'**	**Boring B-1 @ 3'-10'**	**Boring B-4 @ 1½-10'**	**Boring B-7 @ 3½'-10**
Soluble Sulfate	0.009 % (SO_4)	0.007 % (SO_4)	0.005 % (SO_4)	0.007 % (SO_4)

RESISTANCE VALUE (CALIFORNIA TEST 301)

Sample Location	**Boring B-3@ 0'-3**	**Boring B-6 @ 0'-3.5'**
By Exudation	73	69
By Expansion	N/A	N/A
By Equilibrium	73	69

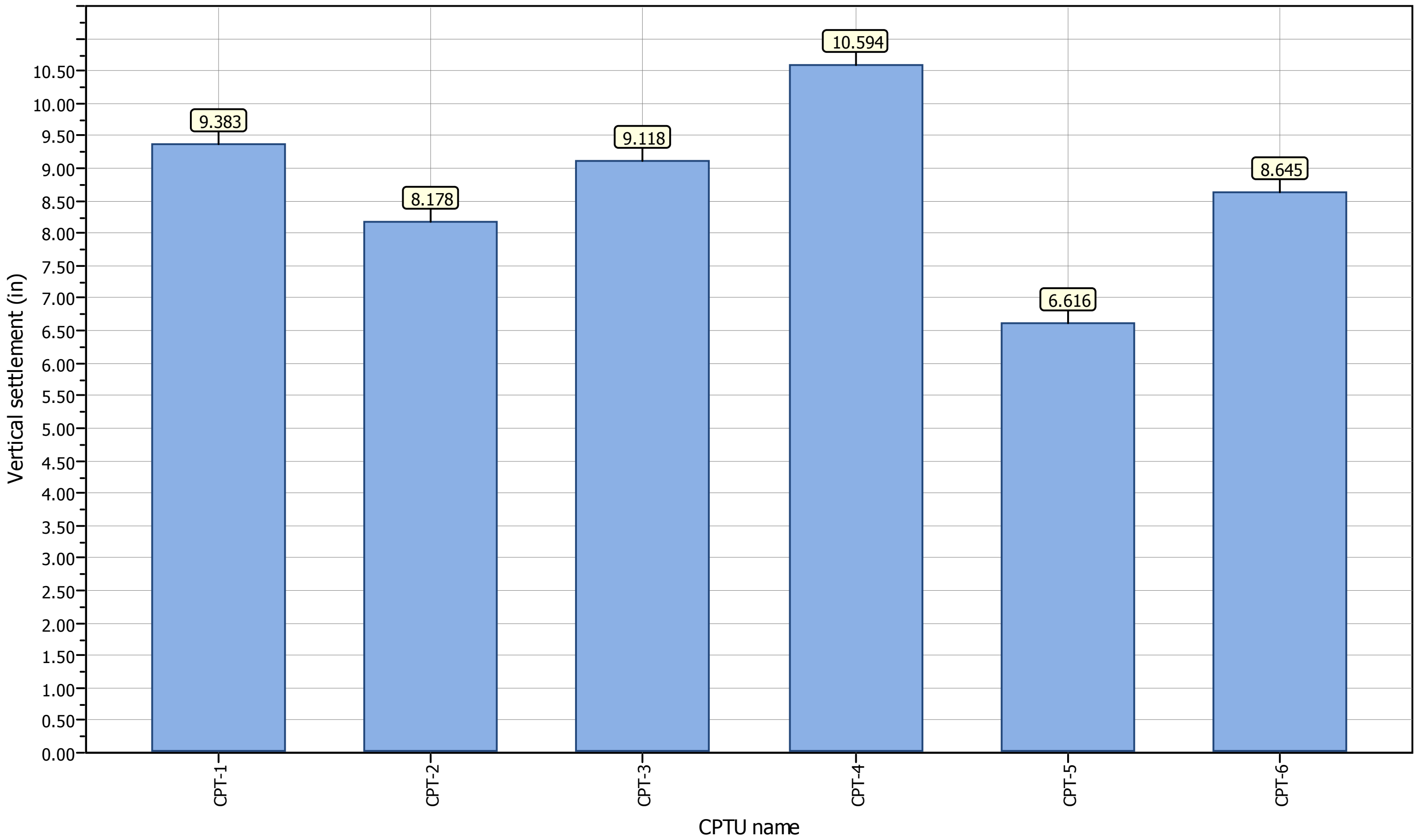
Appendix D

Liquefaction Analyses

Project title : Nagata Site

Location : 4617 North River Road, Oceanside

Overall vertical settlements report



Christian Wheeler Engineering
3980 Home Avenue
San Diego, CA 92105

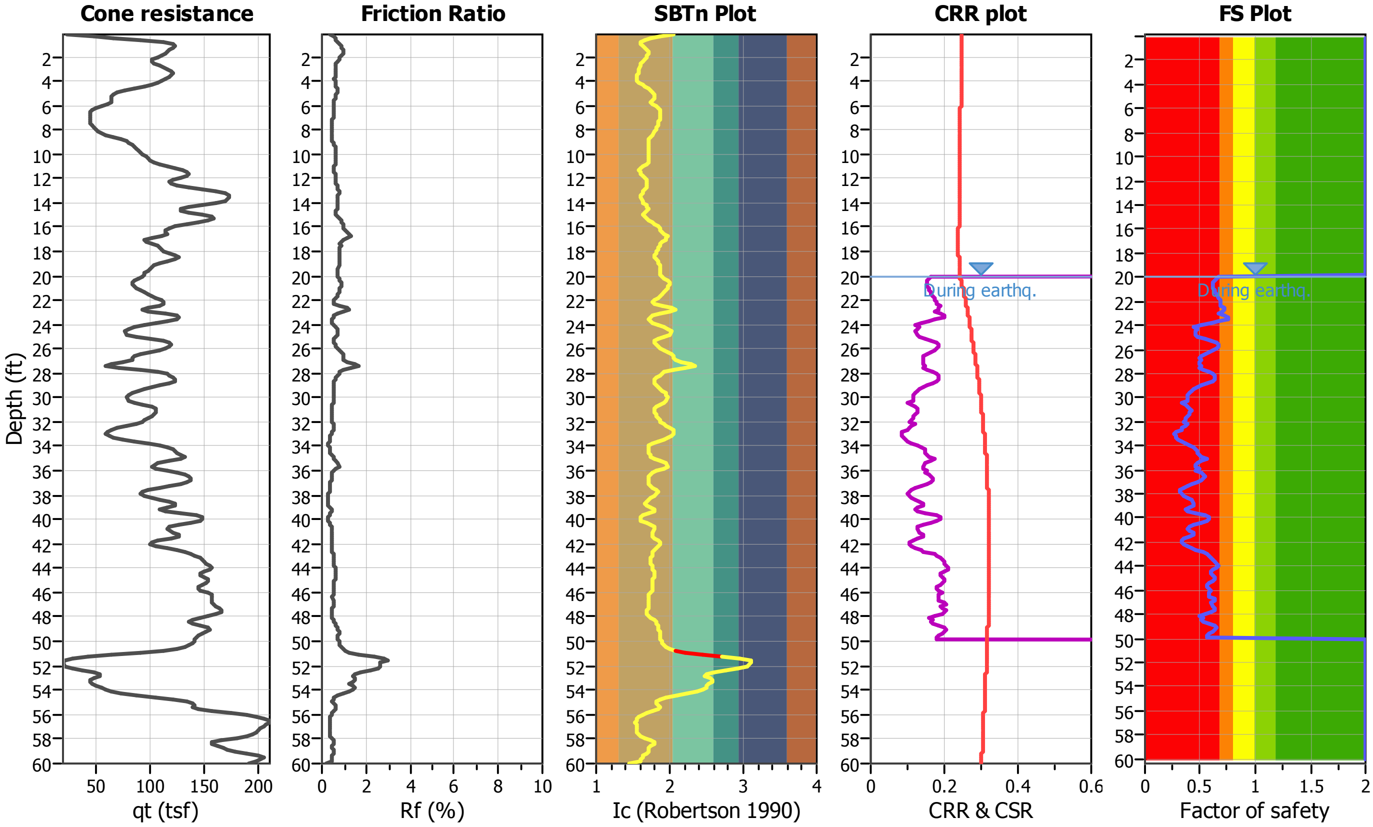
LIQUEFACTION ANALYSIS REPORT

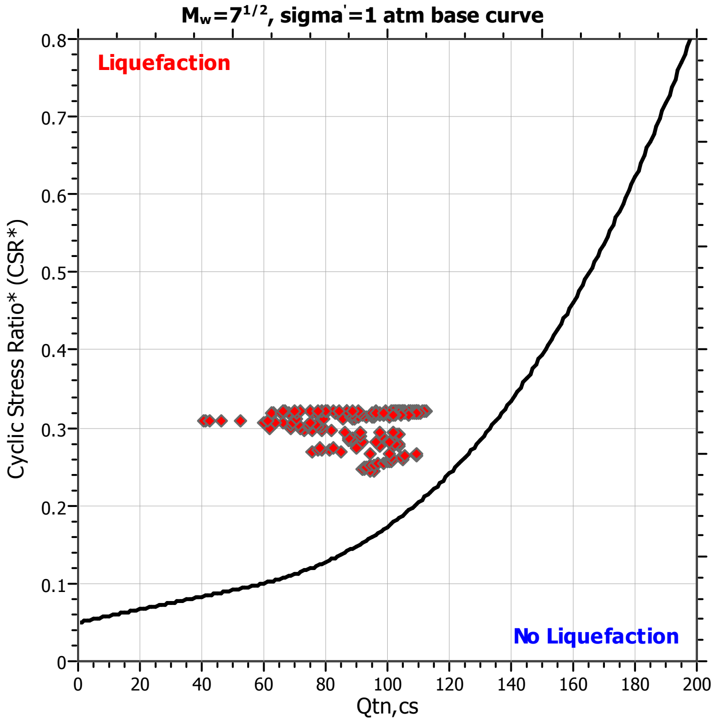
Project title : Nagata Site **Location : 4617 North River Road, Oceanside**

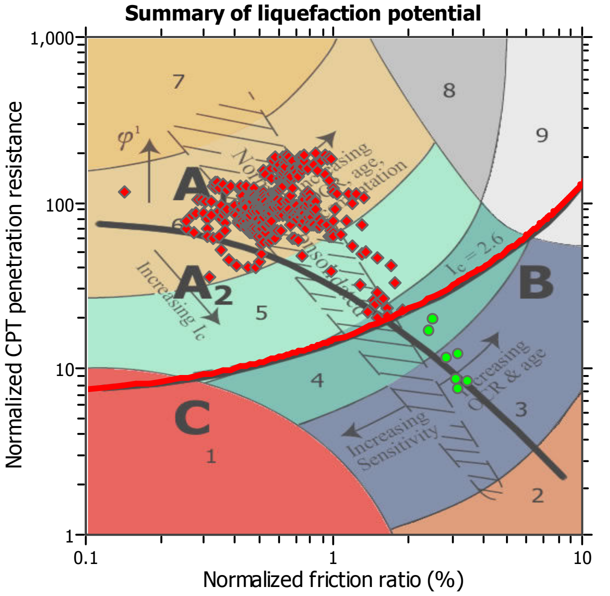
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	23.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.40	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

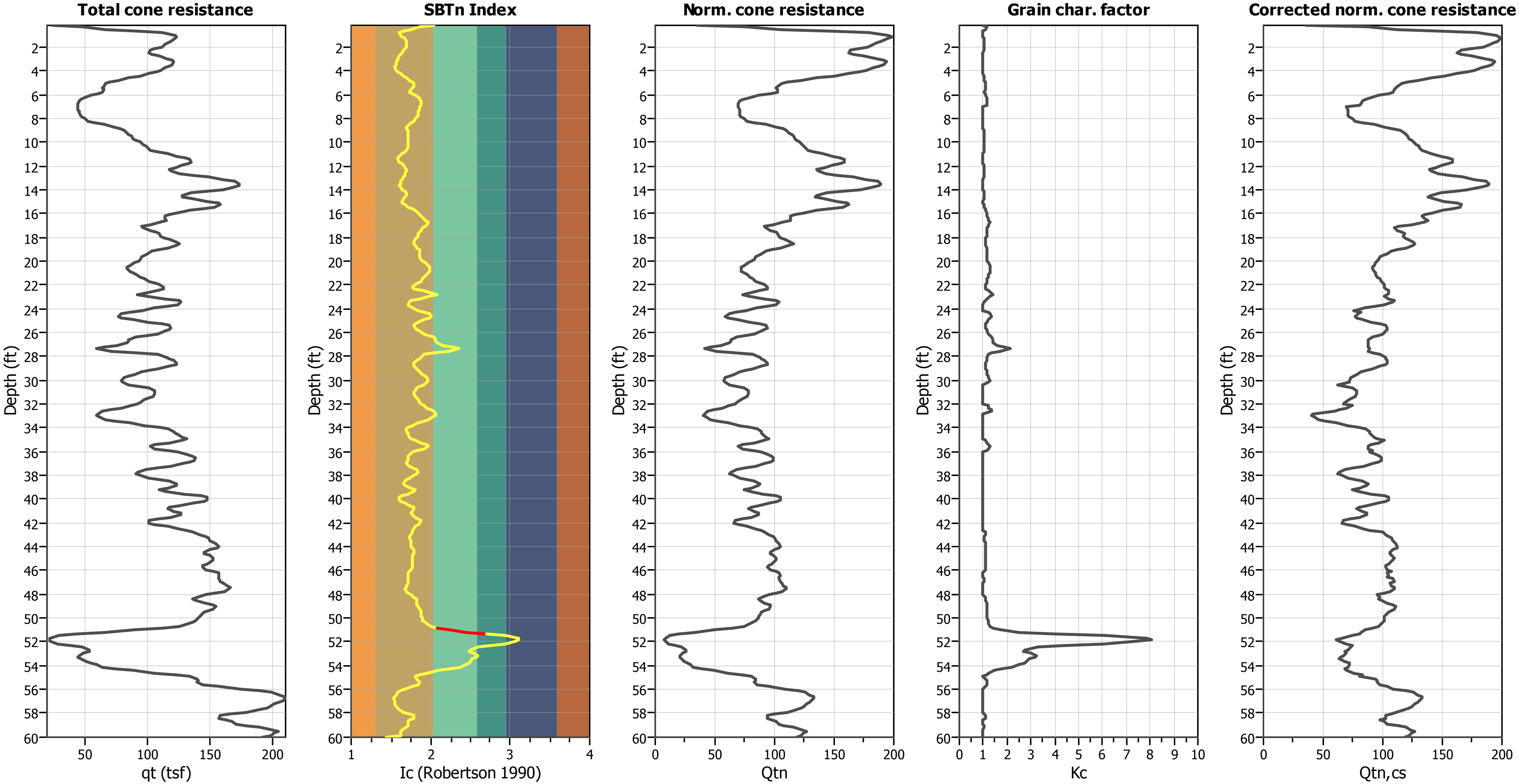






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

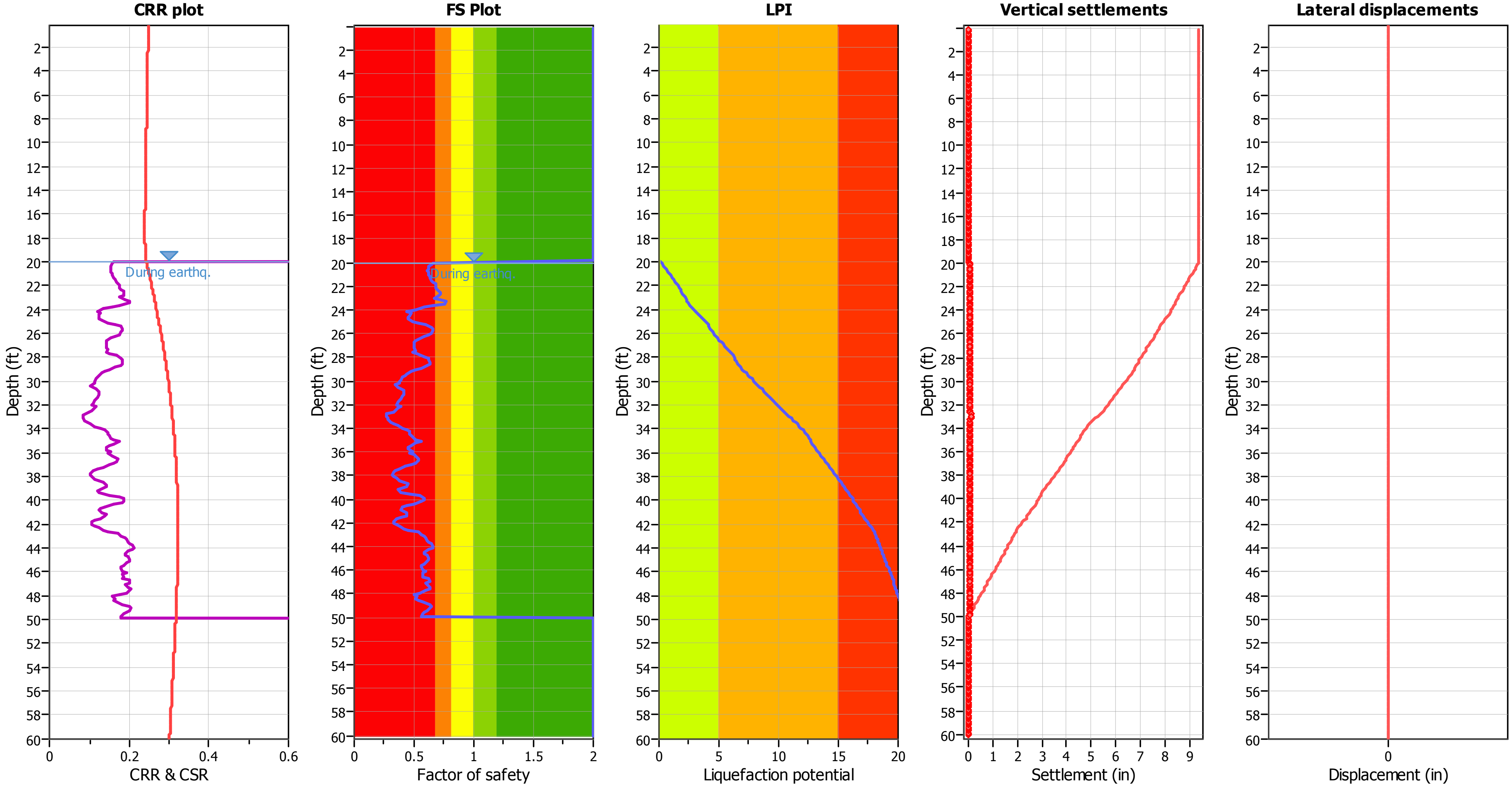
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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

LPI color scheme
- Very high risk
- High risk
- Low risk

Christian Wheeler Engineering
3980 Home Avenue
San Diego, CA 92105

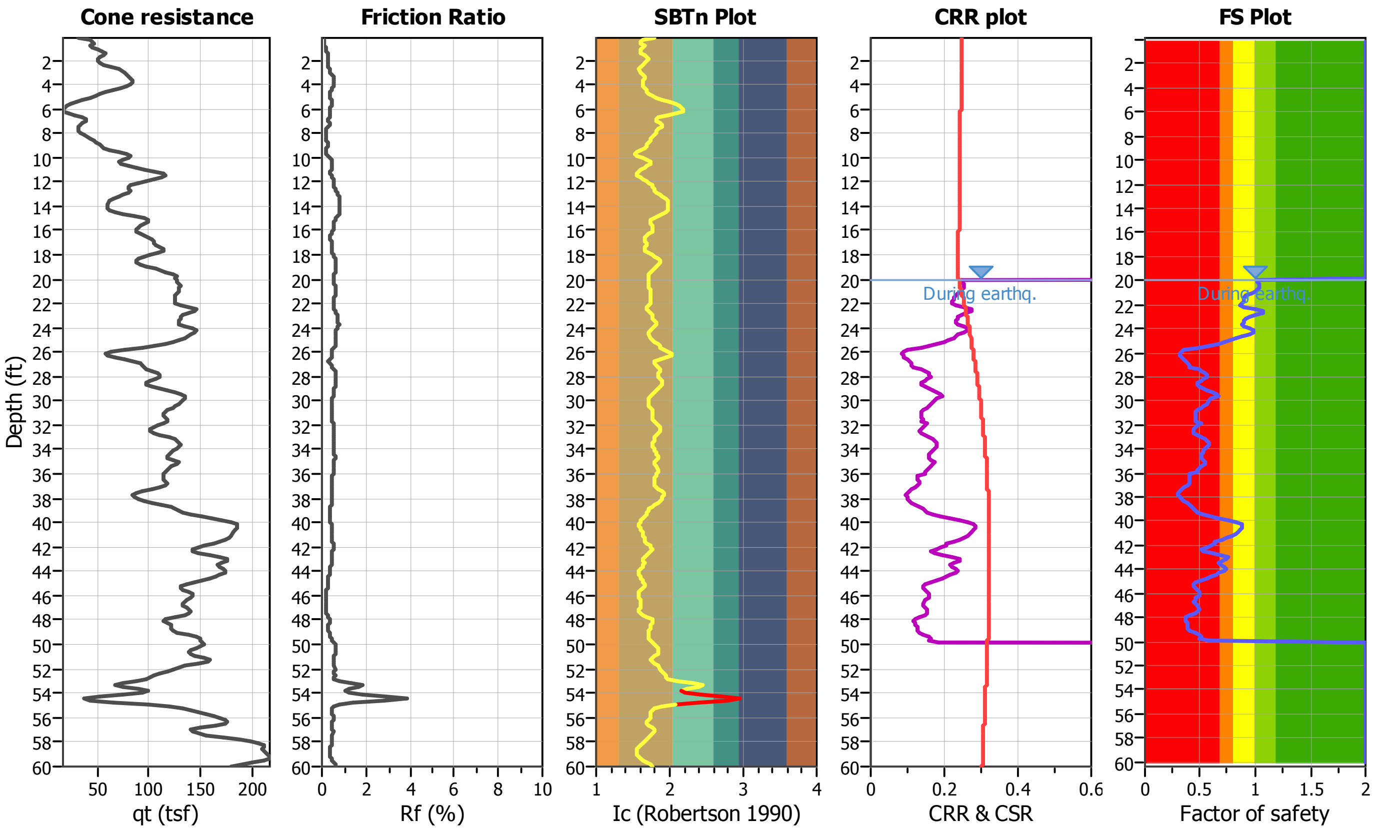
LIQUEFACTION ANALYSIS REPORT

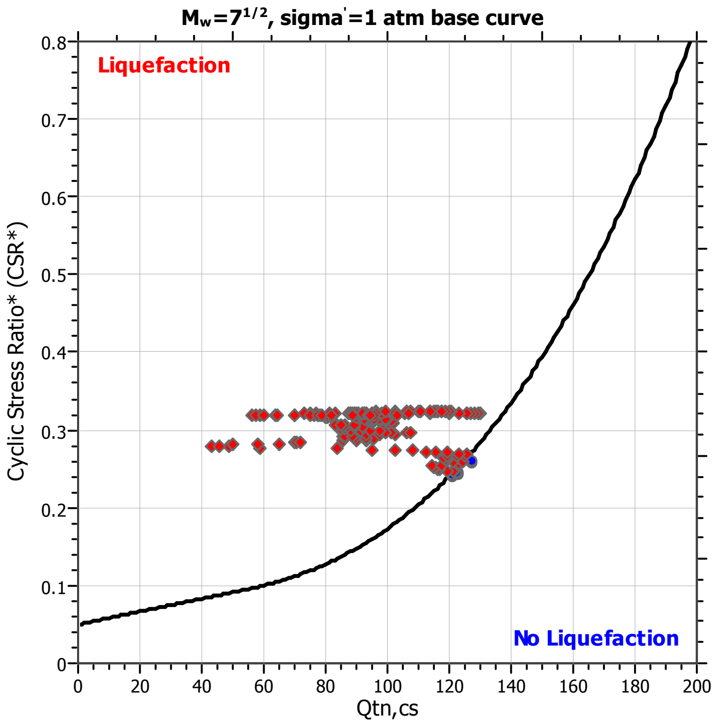
Project title : Nagata Site **Location : 4617 North River Road, Oceanside**

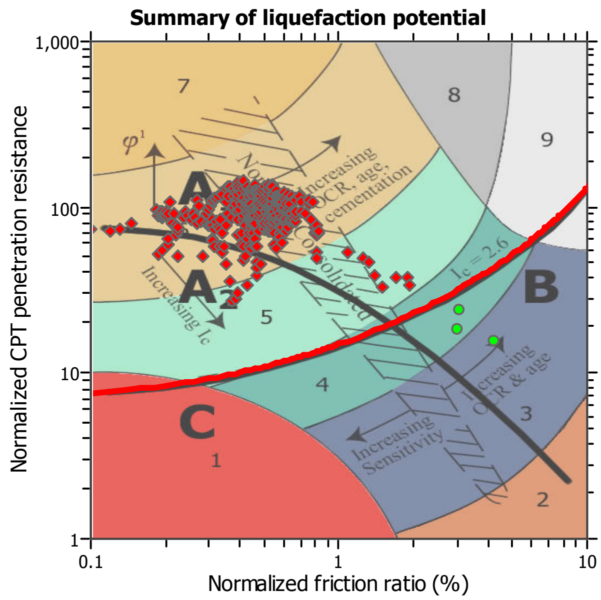
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	26.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.40	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

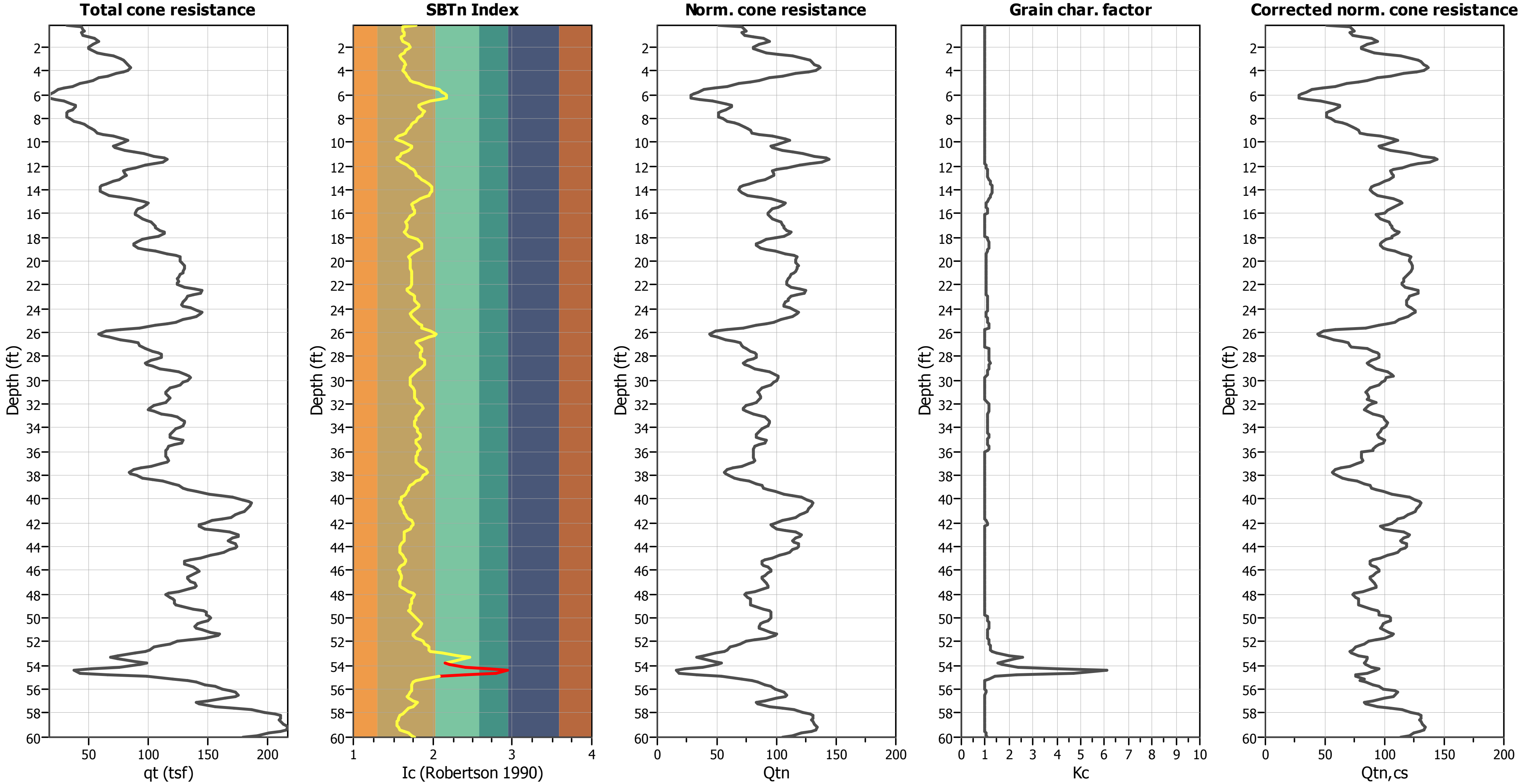






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

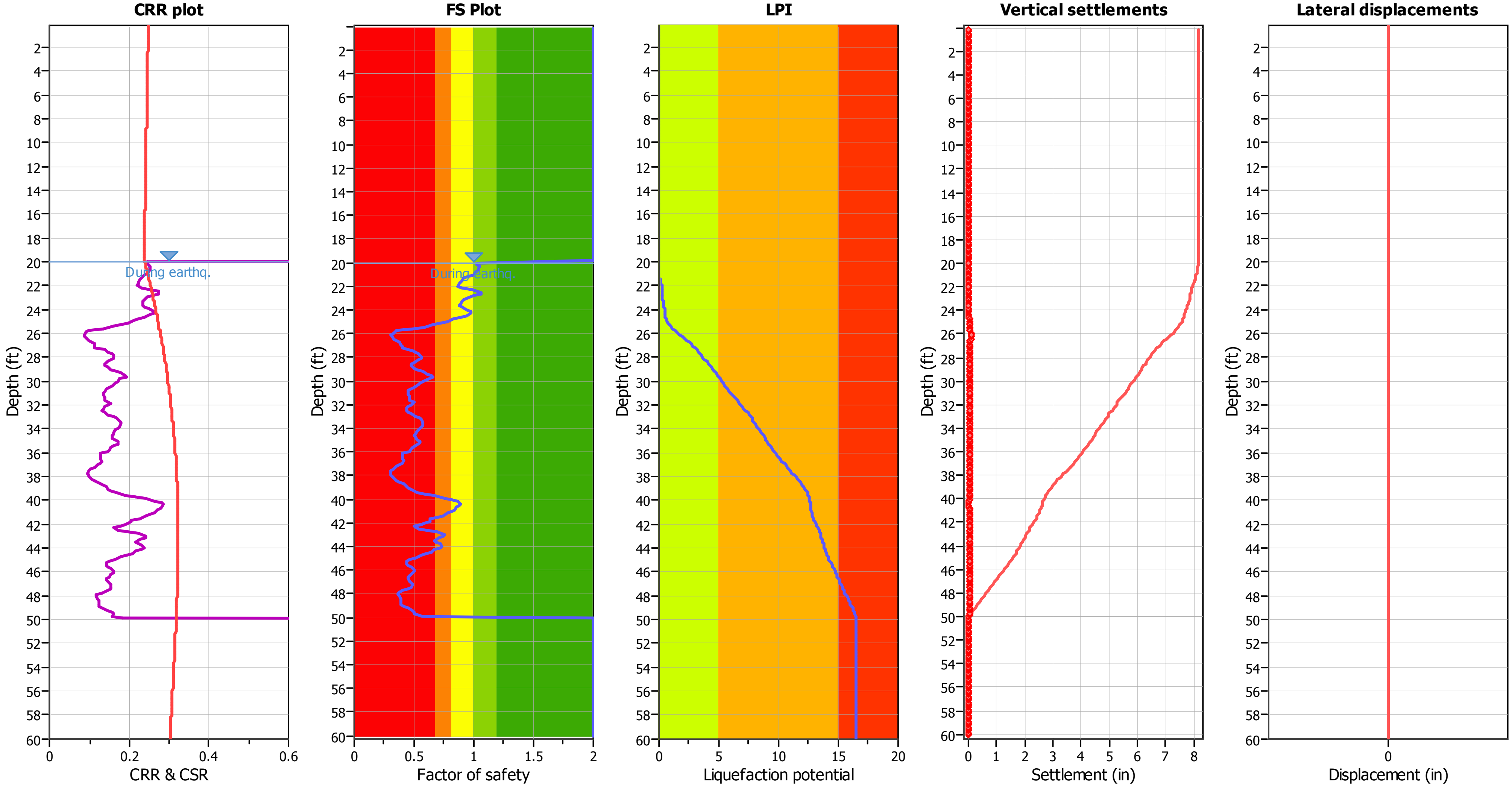
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	26.50 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	26.50 ft	Fill height:	N/A	Limit depth:	50.00 ft

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Christian Wheeler Engineering
3980 Home Avenue
San Diego, CA 92105

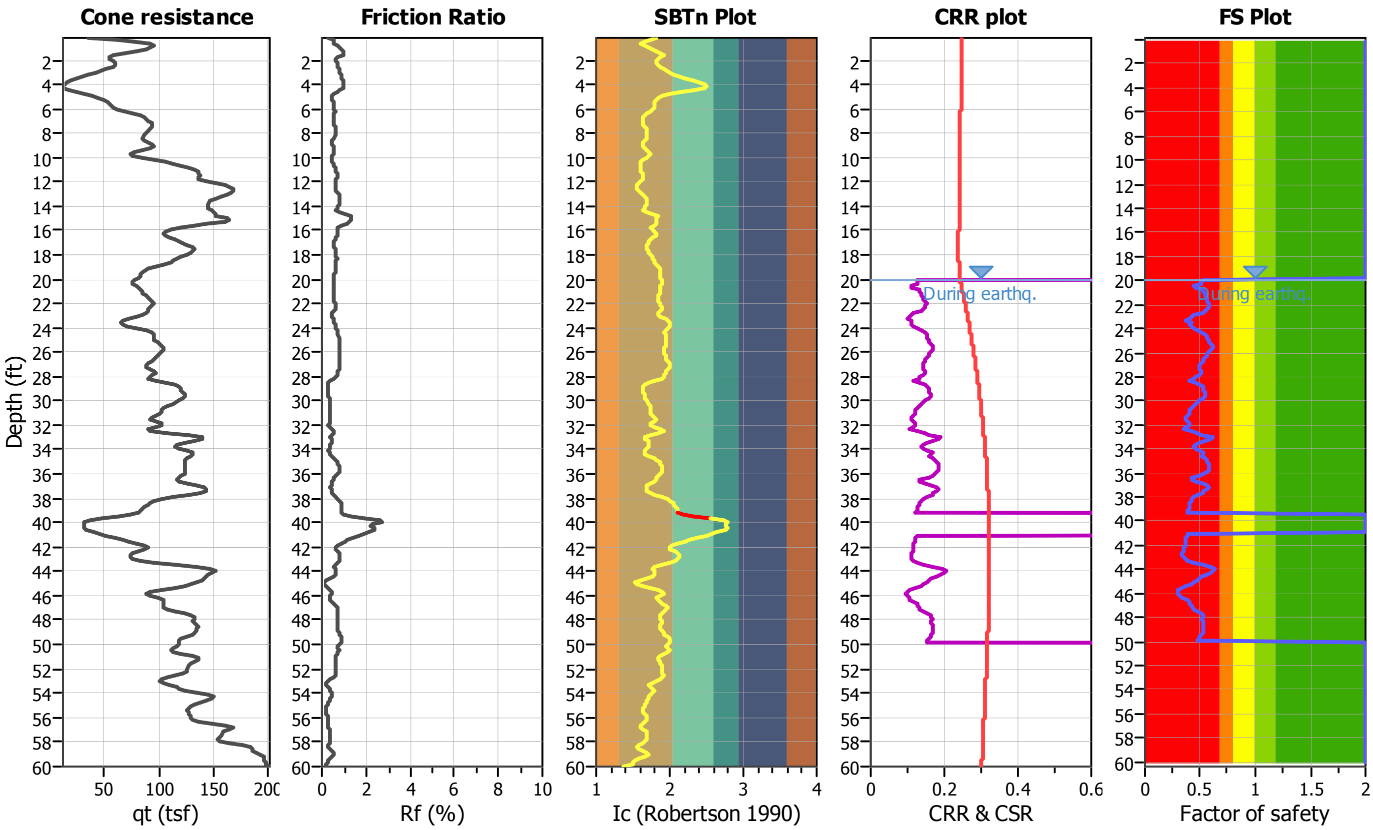
LIQUEFACTION ANALYSIS REPORT

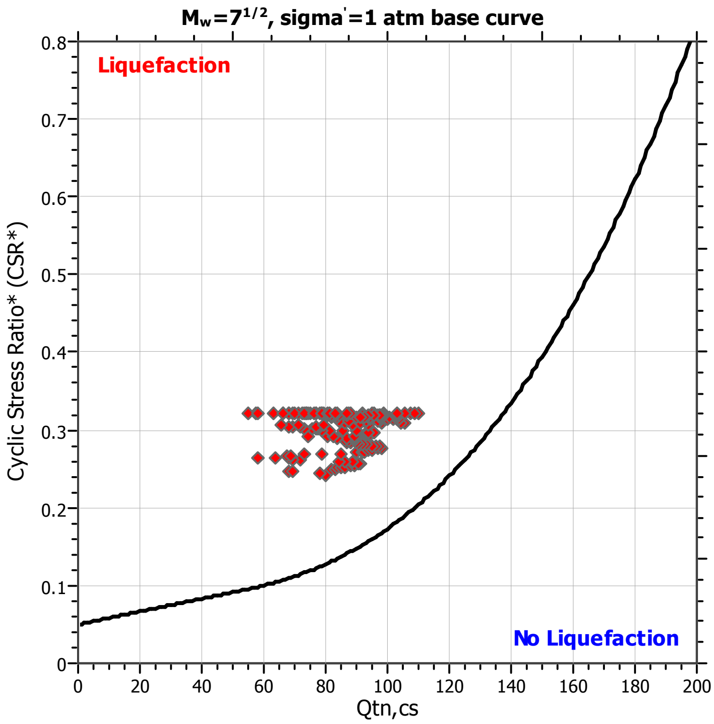
Project title : Nagata Site **Location : 4617 North River Road, Oceanside**

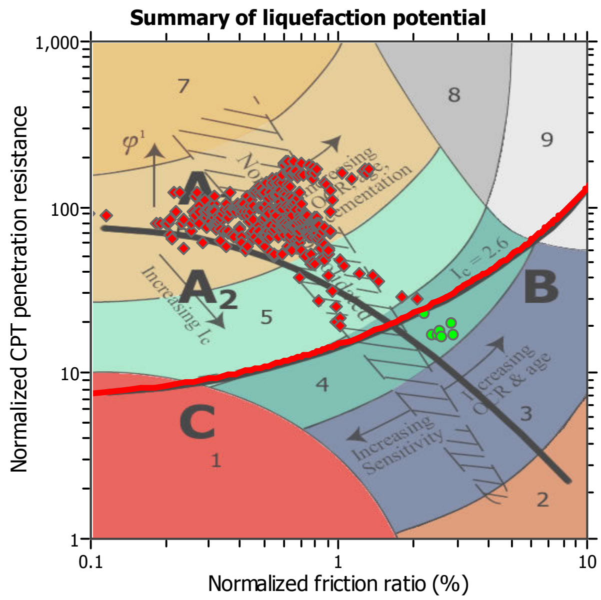
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	23.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.40	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

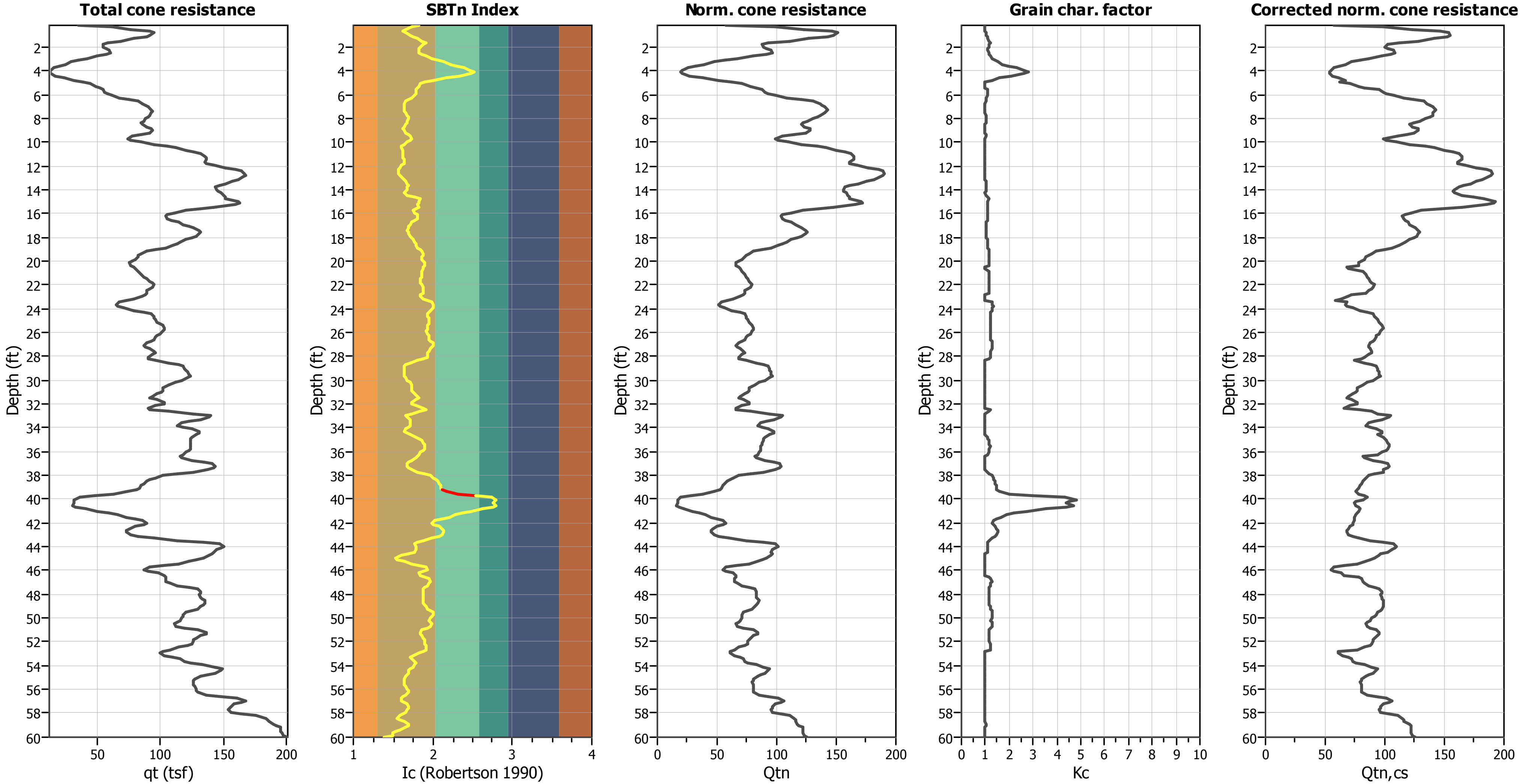






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

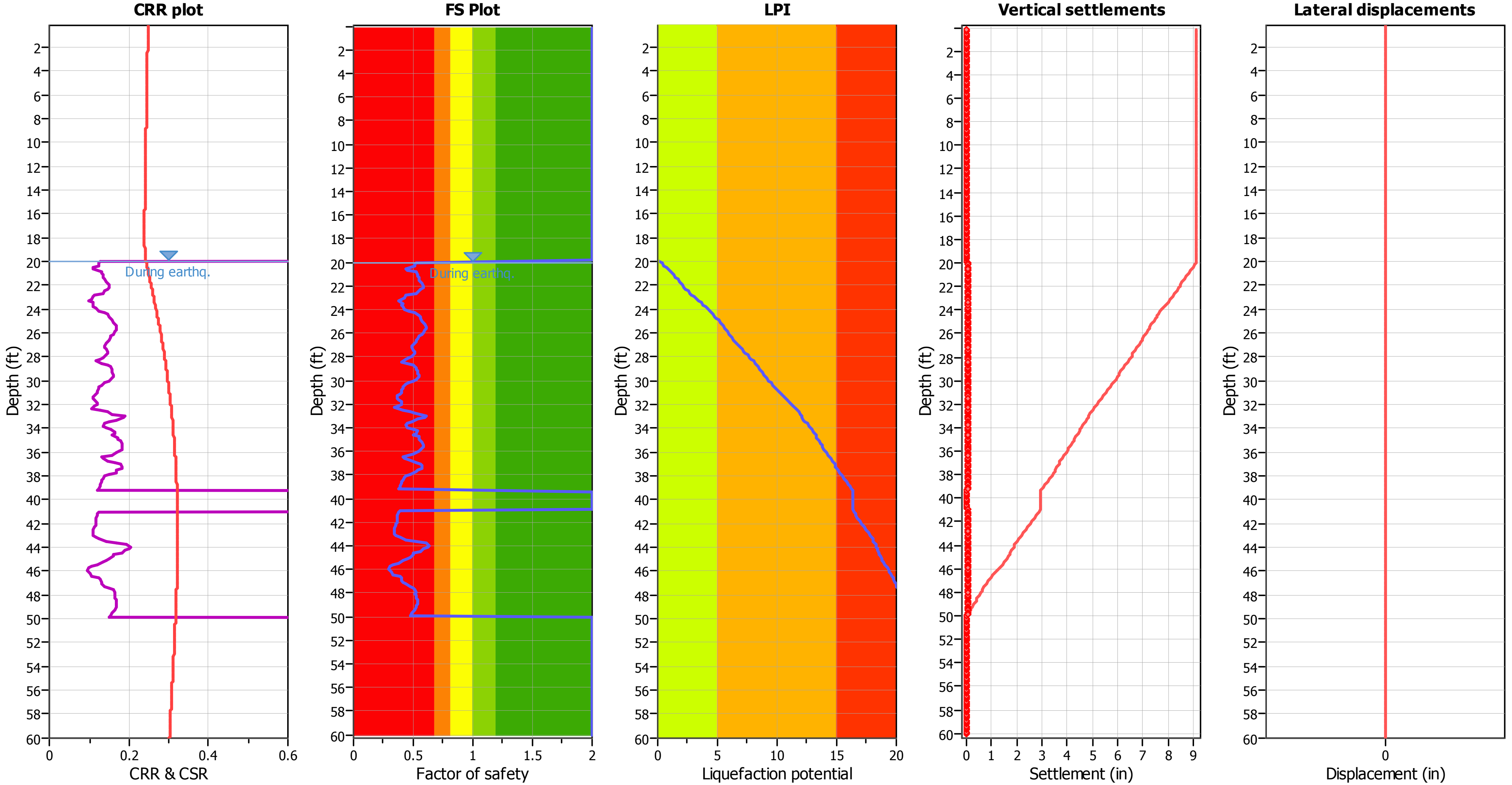
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Christian Wheeler Engineering
3980 Home Avenue
San Diego, CA 92105

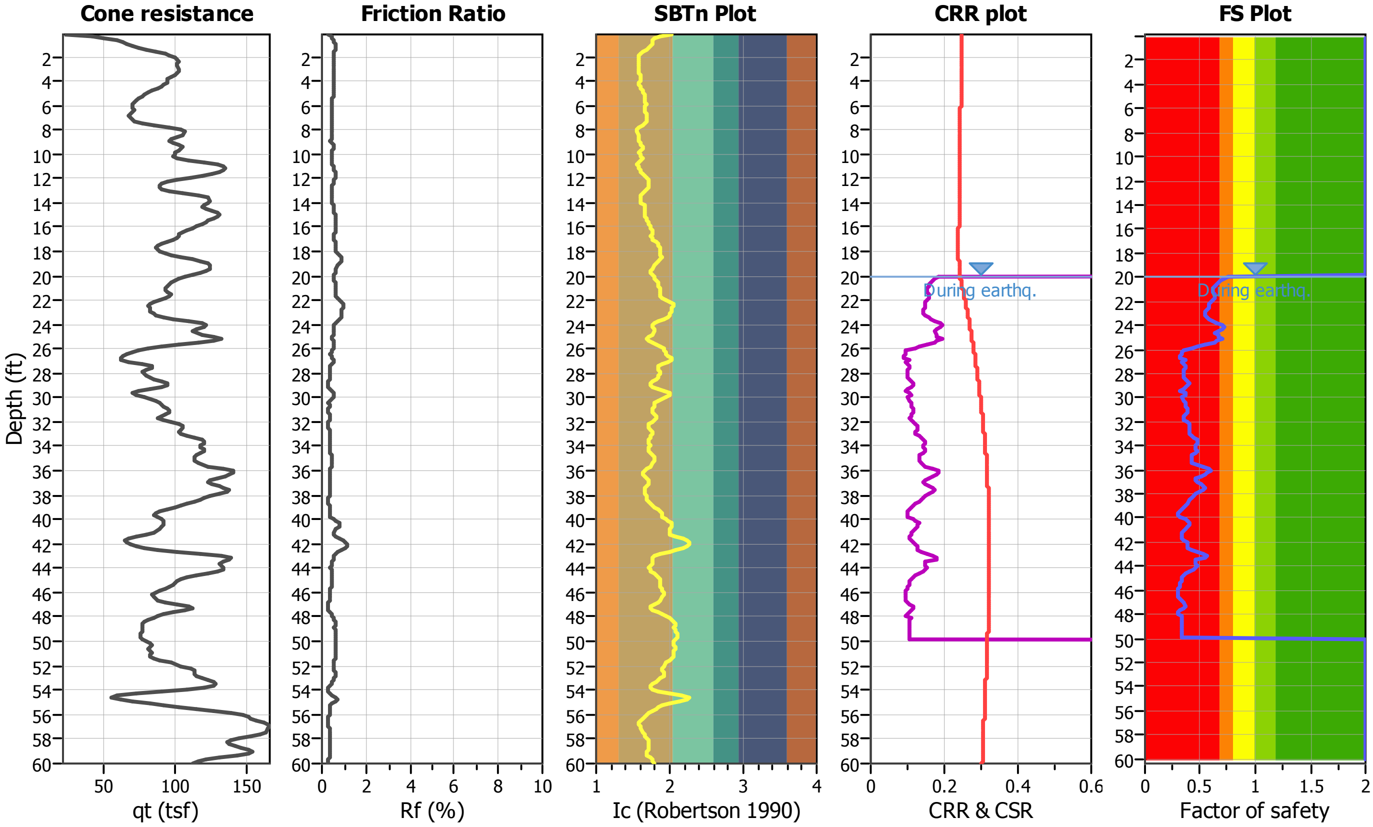
LIQUEFACTION ANALYSIS REPORT

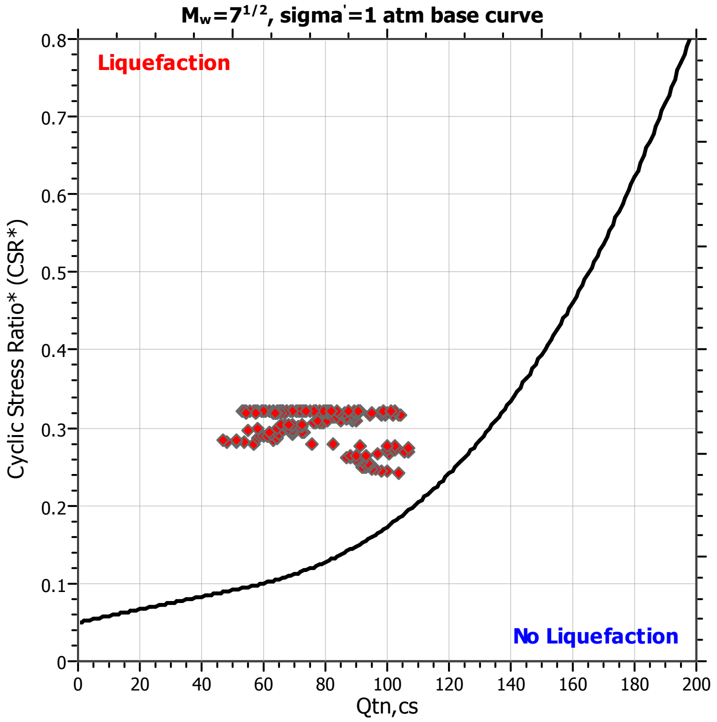
Project title : Nagata Site **Location : 4617 North River Road, Oceanside**

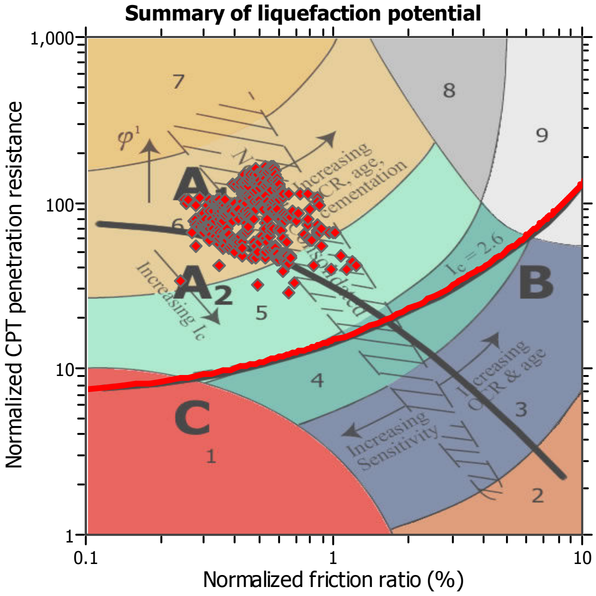
CPT file : CPT-4

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	23.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.40	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

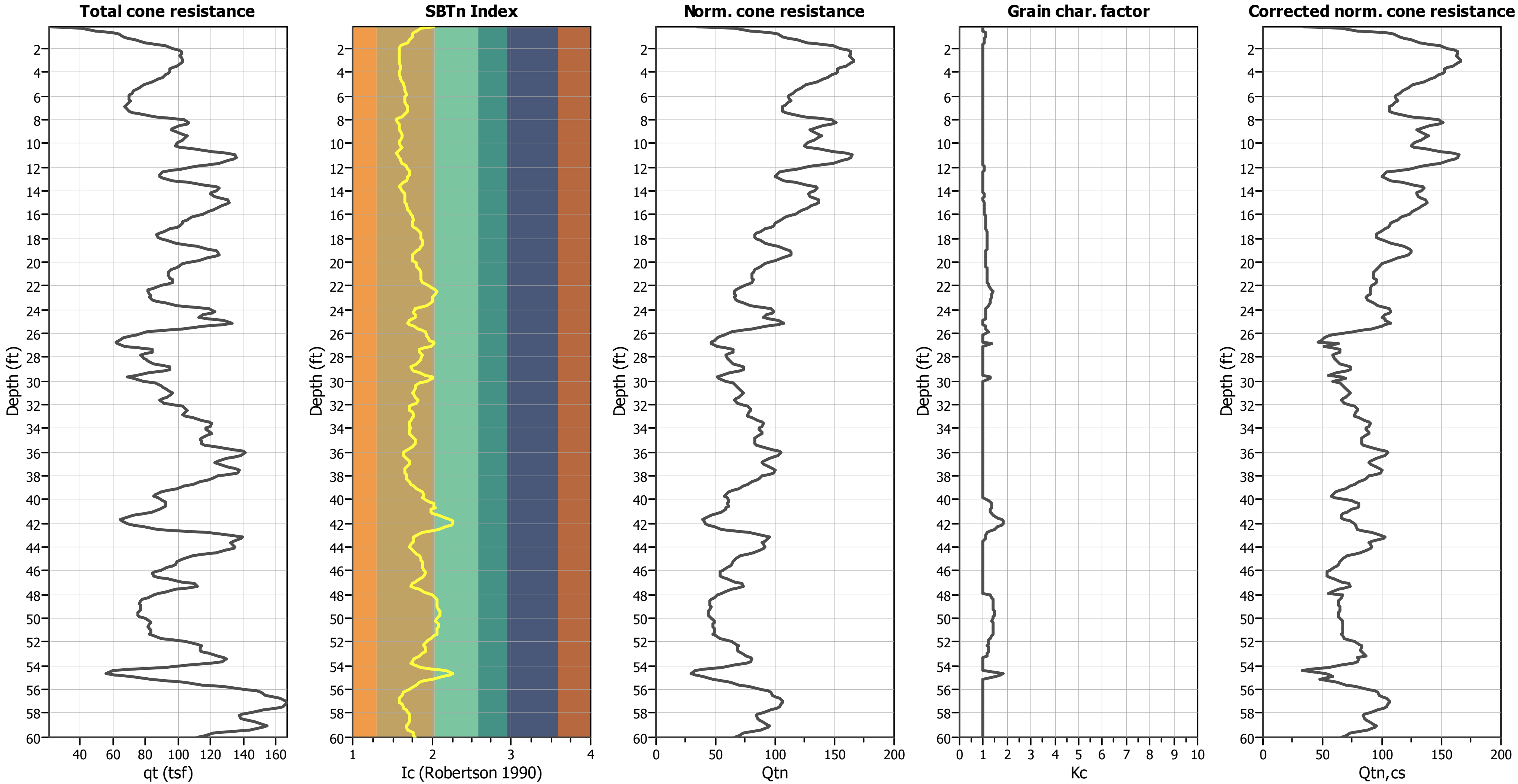






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

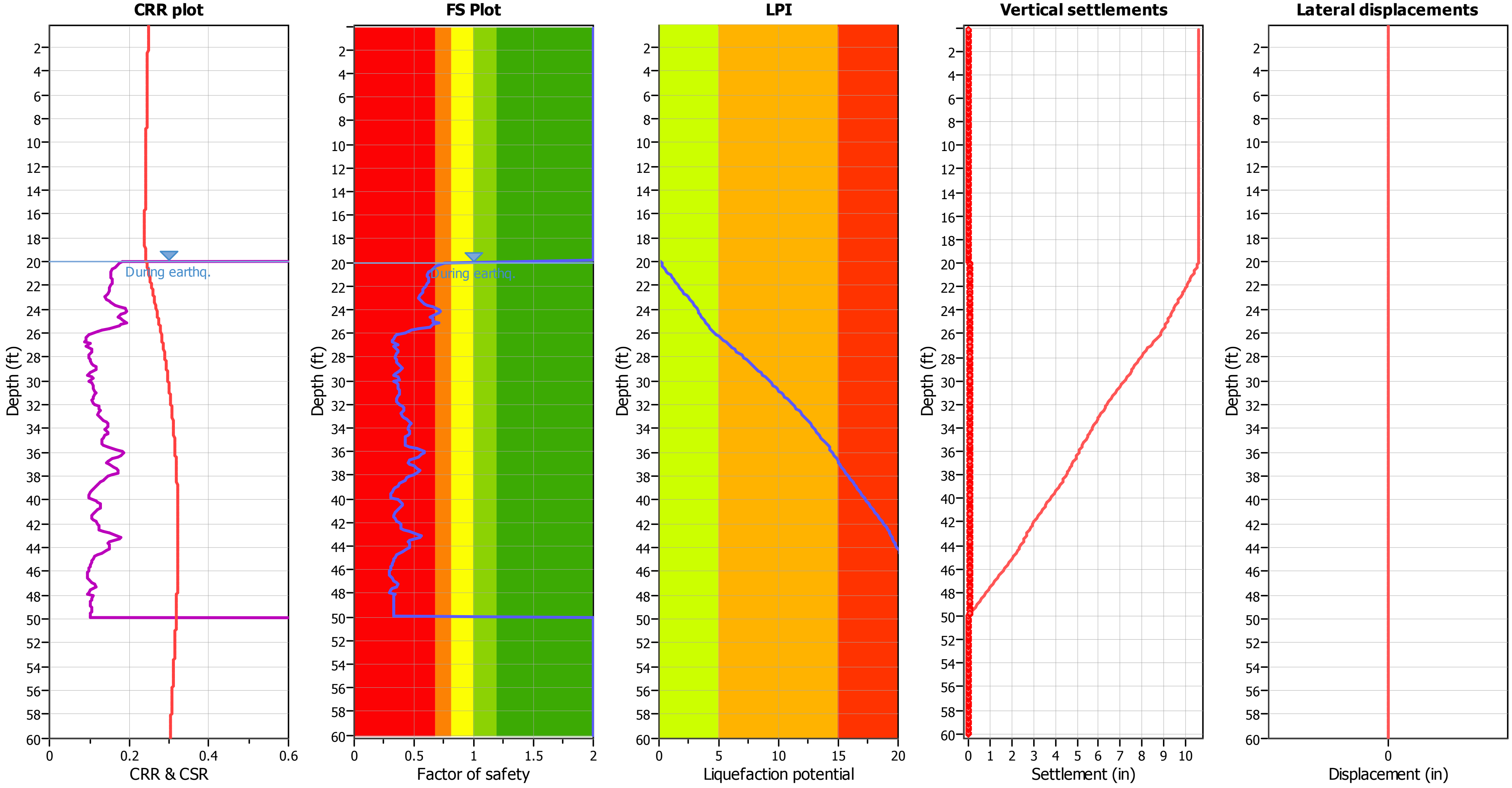
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	23.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Christian Wheeler Engineering
3980 Home Avenue
San Diego, CA 92105

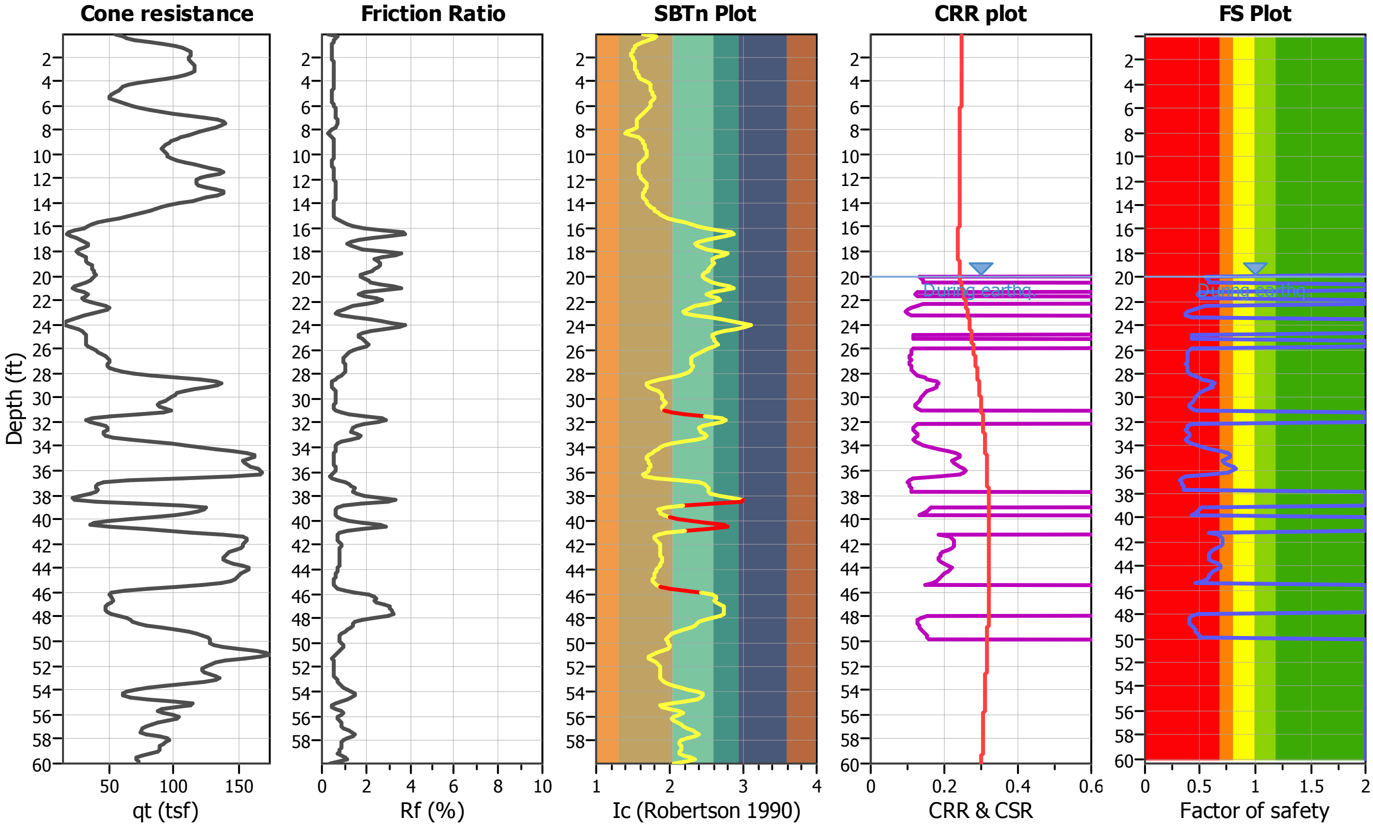
LIQUEFACTION ANALYSIS REPORT

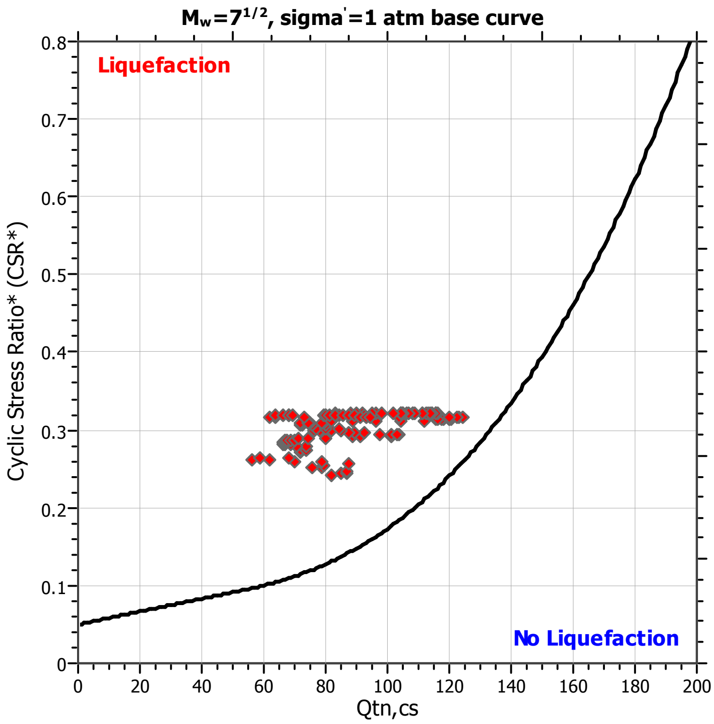
Project title : Nagata Site **Location : 4617 North River Road, Oceanside**

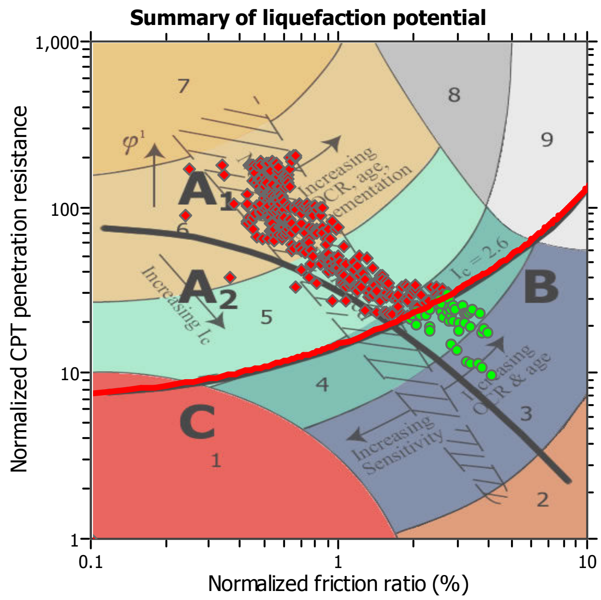
CPT file : CPT-5

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	25.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.40	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

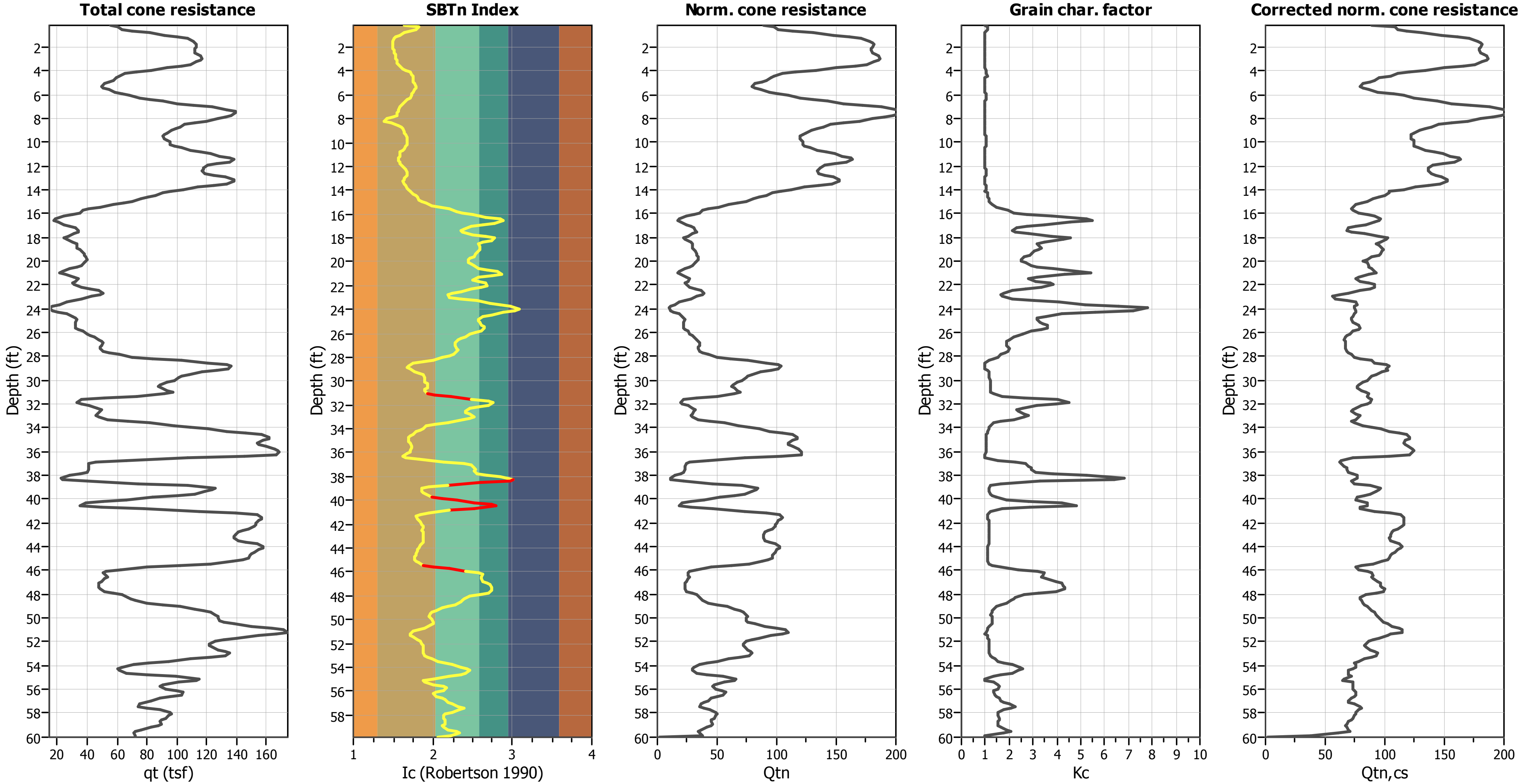






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

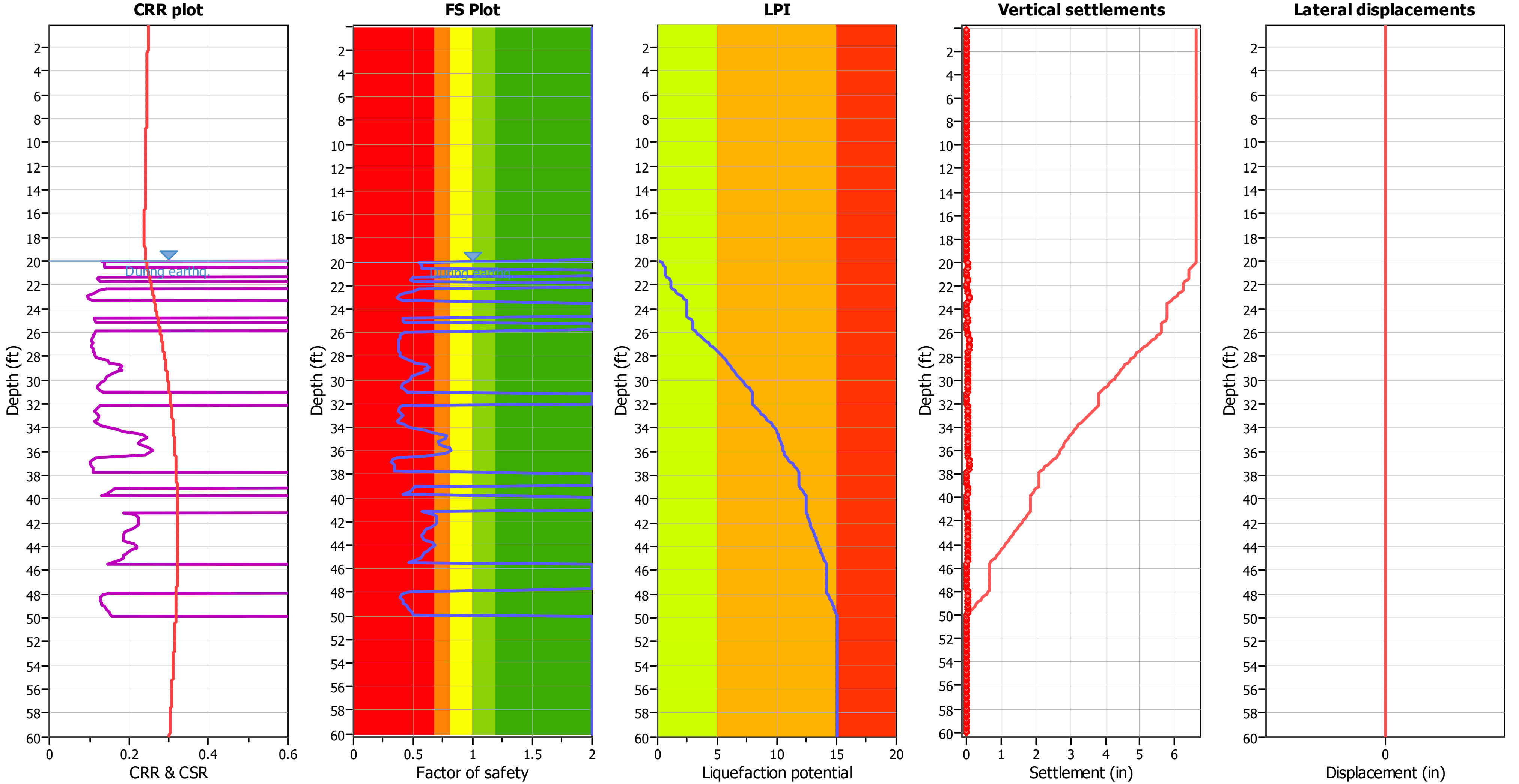
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.50 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.50 ft	Fill height:	N/A	Limit depth:	50.00 ft

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Christian Wheeler Engineering
3980 Home Avenue
San Diego, CA 92105

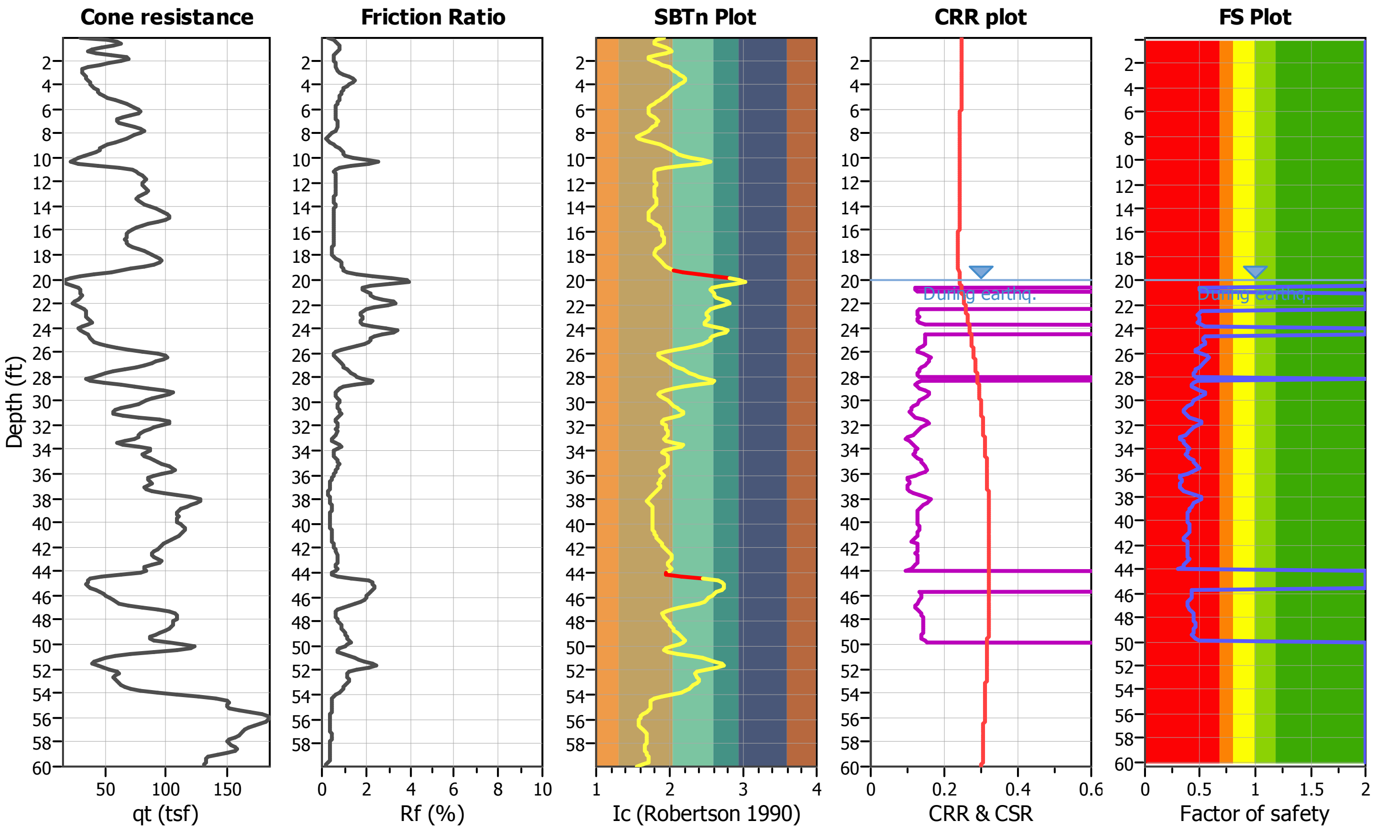
LIQUEFACTION ANALYSIS REPORT

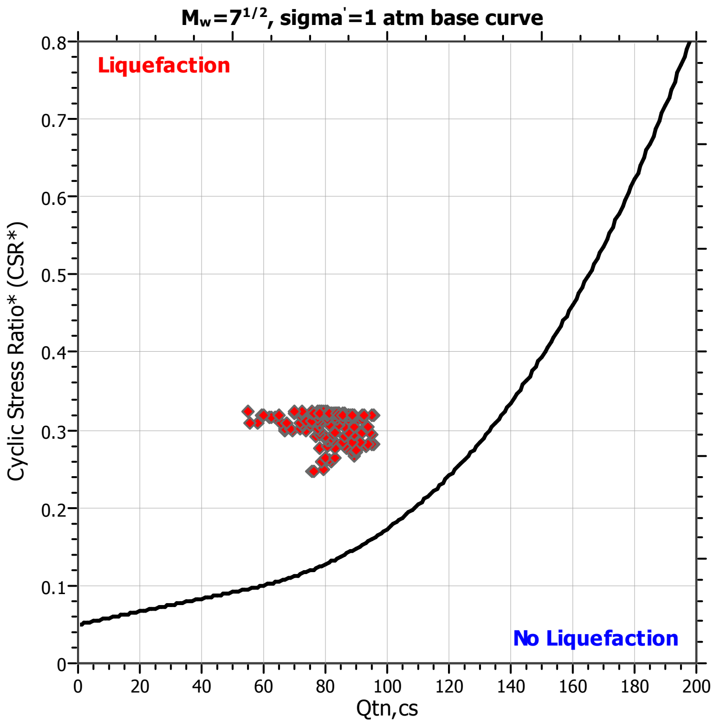
Project title : Nagata Site **Location : 4617 North River Road, Oceanside**

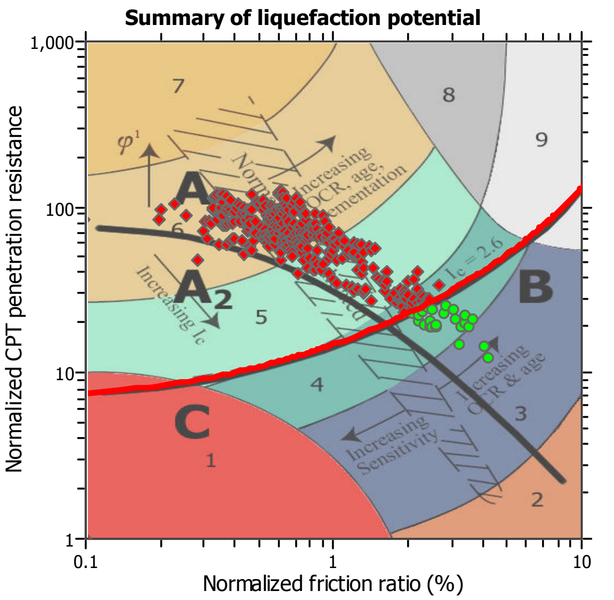
CPT file : CPT-6

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	20.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.40	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

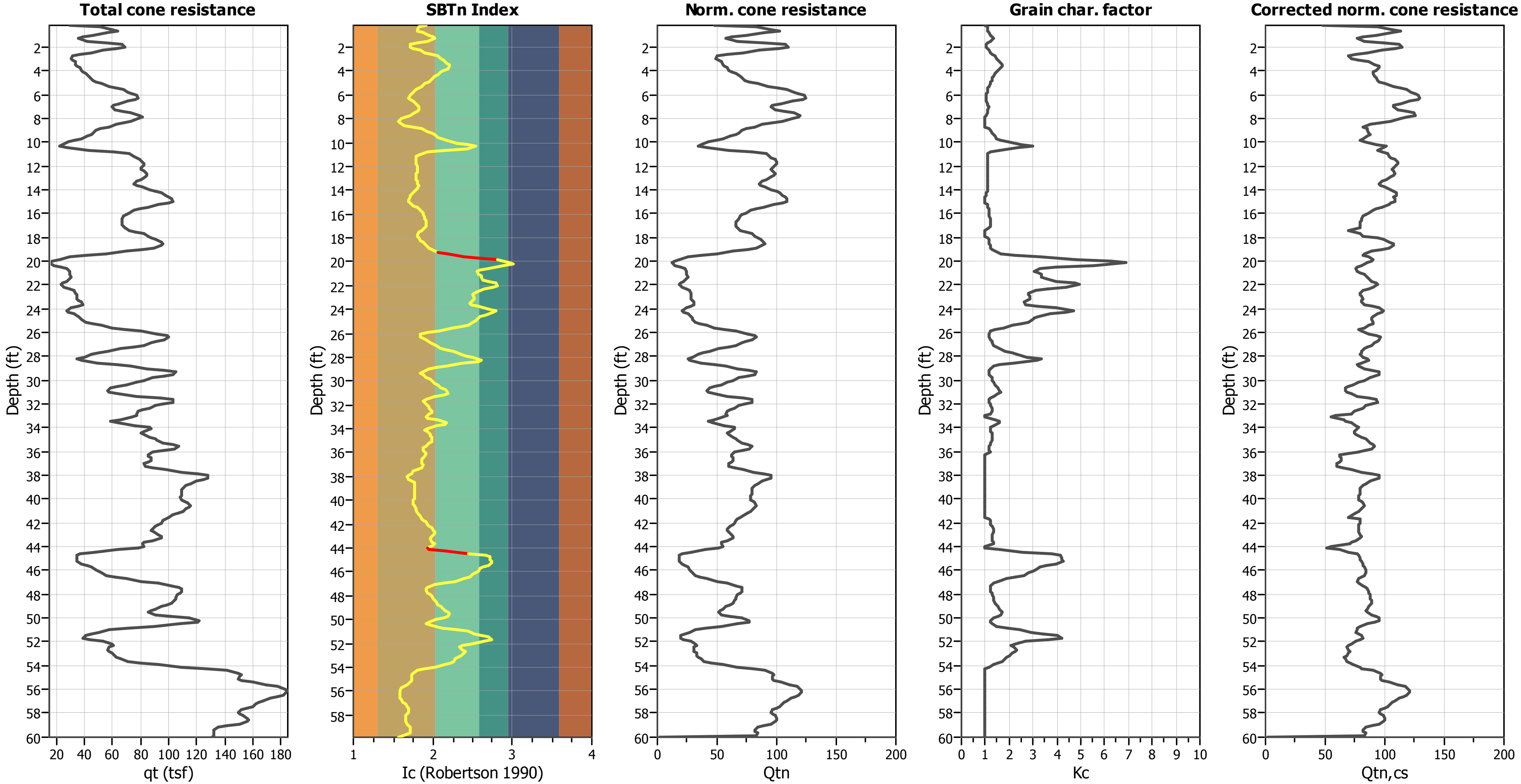






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

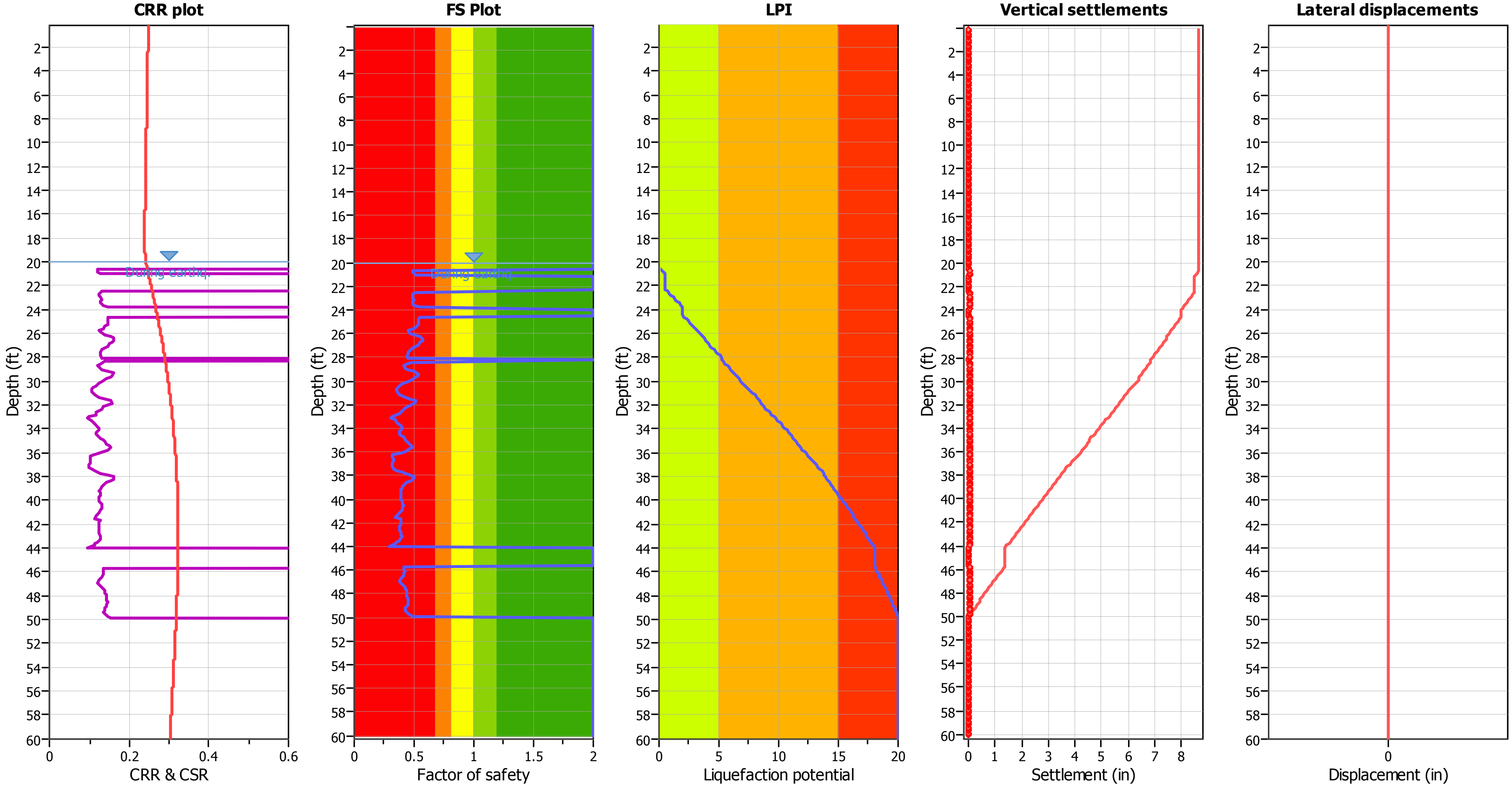
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	20.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	20.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Appendix D

References

REFERENCES

Bartlett, S.F. and Youd, T.L., 1995, Empirical Prediction of Liquefaction-Induced Lateral Spread, American Society of Civil Engineers, Journal of Geotechnical Engineers, v. 121, no. 4, p. 316-329.

Bryant, W. A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: California Geological Survey Web Page, http://www.consrv.ca.gov/CGS/information/publications/QuaternaryFaults_ver2.htm

California Division of Mines and Geology, 1998, Maps of Known Active Fault Near Source-Zones in California and Adjacent Portions of Nevada.

California Mining and Geology Board, 1996, Guidelines For Evaluating The Hazard of Surface Fault Rupture, adopted May 9, 1996.

California Division of Mines and Geology, 1997, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117.

California Division of Mines and Geology, 1999, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, organized through the Southern California Earthquake Center, University of Southern California.

California Emergency Management, 2009, Tsunami Inundation Map For Emergency Planning.

California Geological Survey, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117.

Christian Wheeler Engineering, 2005, Report of Preliminary Geotechnical Investigation, Proposed Nagata Ranch Residential Development, 4617 North River Road, Oceanside, California, CWE Report 2050567.02, dated July 20, 2005.

ConeTec Investigations Ltd., 2002, CPT Liquefaction Analysis Spreadsheet, LQCPTV2, Application of the Integrated CPT Method (Version 2) for Estimating Cyclic Resistance Ratio (CRR) and Liquefaction Induced Soil Deformations, Release 1.00, Revision C, October 31, 2002.

Federal Emergency Management Agency, 2012, San Diego County, California and Incorporated Areas Flood Insurance Rate Map, Map Number 06073C1611G.

Idriss, I. M. and Boulanger, R. W., 2008, Soil Liquefaction During Earthquakes, Oakland, CA: Earthquake Engineering Research Institute.

Ishihara, K., 1985, "Stability of Natural Deposits During Earthquakes," Theme Lecture, Proceeding of the XI ICSMFE, Vol. 2, pp. 321-376.

Hart, E.W., 1994, Fault-Rupture Hazard Zones in California, California Division of Mines and Geology Special Publication 42.

Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map, California Geological Survey, Geologic Data Map No. 6, http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html

Kennedy, Michael P. and Tan, Siang S., 2008, Geologic Map of the San Diego 30'x60' Quadrangle, California, California Geologic Survey, Map No. 3.

Kern, P., 1989, Earthquakes and Faults in San Diego County, Pickle Press, 73 pp.

Tan, S.S., 1995, Landslide Hazards in the Southern Part of the San Diego Metropolitan Area, San Diego County, California, California Division of Mines and Geology Open-File Report 95-03.

Robertson, P.K. and Wride, C.E., 1998. Cyclic Liquefaction and its Evaluation based on the CPT Canadian Geotechnical Journal, 1998, Vol. 35, August.

Seed et al, 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework, Keynote Presentation, 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Long Beach, CA.

U.S. Geological Survey, U.S. Seismic Design Maps Web Application, http://geohazards.usgs.gov/designmaps/us/application.php

U.S. Geological Survey, Quaternary Faults in Google Earth, http://earthquake.usgs.gov/hazards/qfaults/google.php

Yoshimine, M., Robertson, P.K. and Wride, C.E., 1999, Undrained Shear Strength of Clean Sands to Trigger Flow Liquefaction, Canadian Geotechnical Journal, Vol.36, No.5, pp.891-906

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J., Liao, S., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R., and Stokoe, K.H., Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 127, October, pp 817-833.

Zhang, G., Robertson. P.K., Brachman, R., 2002, Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168-1180

Zhang, G., Robertson. P.K., Brachman, R., 2004, Estimating Liquefaction Induced Lateral Displacements using the SPT and CPT, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 130, No. 8, 861-871

Appendix E

Recommended Grading Specifications – General Provisions

RECOMMENDED GRADING SPECIFICATIONS - GENERAL PROVISIONS

NAGATA PROPERTY
4617 NORTH RIVER ROAD
OCEANSIDE, CALIFORNIA

GENERAL INTENT

The intent of these specifications is to establish procedures for clearing, compacting natural ground, preparing areas to be filled, and placing and compacting fill soils to the lines and grades shown on the accepted plans. The recommendations contained in the preliminary geotechnical investigation report and/or the attached Special Provisions are a part of the Recommended Grading Specifications and shall supersede the provisions contained hereinafter in the case of conflict. These specifications shall only be used in conjunction with the geotechnical report for which they are a part. No deviation from these specifications will be allowed, except where specified in the geotechnical report or in other written communication signed by the Geotechnical Engineer.

OBSERVATION AND TESTING

Christian Wheeler Engineering shall be retained as the Geotechnical Engineer to observe and test the earthwork in accordance with these specifications. It will be necessary that the Geotechnical Engineer or his representative provide adequate observation so that he may provide his opinion as to whether or not the work was accomplished as specified. It shall be the responsibility of the contractor to assist the Geotechnical Engineer and to keep him appraised of work schedules, changes and new information and data so that he may provide these opinions. In the event that any unusual conditions not covered by the special provisions or preliminary geotechnical report are encountered during the grading operations, the Geotechnical Engineer shall be contacted for further recommendations.

If, in the opinion of the Geotechnical Engineer, substandard conditions are encountered, such as questionable or unsuitable soil, unacceptable moisture content, inadequate compaction, adverse weather, etc., construction should be stopped until the conditions are remedied or corrected or he shall recommend rejection of this work.

Tests used to determine the degree of compaction should be performed in accordance with the following American Society for Testing and Materials test methods:

Maximum Density & Optimum Moisture Content - ASTM D-1557-91
Density of Soil In-Place - ASTM D-1556-90 or ASTM D-2922

All densities shall be expressed in terms of Relative Compaction as determined by the foregoing ASTM testing procedures.

PREPARATION OF AREAS TO RECEIVE FILL

All vegetation, brush and debris derived from clearing operations shall be removed, and legally disposed of. All areas disturbed by site grading should be left in a neat and finished appearance, free from unsightly debris.

After clearing or benching the natural ground, the areas to be filled shall be scarified to a depth of 6 inches, brought to the proper moisture content, compacted and tested for the specified minimum degree of compaction. All loose soils in excess of 6 inches thick should be removed to firm natural ground which is defined as natural soil which possesses an in-situ density of at least 90 percent of its maximum dry density.

When the slope of the natural ground receiving fill exceeds 20 percent (5 horizontal units to 1 vertical unit), the original ground shall be stepped or benched. Benches shall be cut to a firm competent formational soil. The lower bench shall be at least 10 feet wide or 1-1/2 times the equipment width, whichever is greater, and shall be sloped back into the hillside at a gradient of not less than two (2) percent. All other benches should be at least 6 feet wide. The horizontal portion of each bench shall be compacted prior to receiving fill as specified herein for compacted natural ground. Ground slopes flatter than 20 percent shall be benched when considered necessary by the Geotechnical Engineer.

Any abandoned buried structures encountered during grading operations must be totally removed. All underground utilities to be abandoned beneath any proposed structure should be removed from within 10 feet of the structure and properly capped off. The resulting depressions from the above described procedure should be backfilled with acceptable soil that is compacted to the requirements of the Geotechnical Engineer. This includes, but is not limited to, septic tanks, fuel tanks, sewer lines or leach lines, storm drains and water lines. Any buried structures or utilities not to be abandoned should be brought to the attention of the Geotechnical Engineer so that he may determine if any special recommendation will be necessary.

All water wells which will be abandoned should be backfilled and capped in accordance to the requirements set forth by the Geotechnical Engineer. The top of the cap should be at least 4 feet below finish grade or 3 feet below the bottom of footing whichever is greater. The type of cap will depend on the diameter of the well and should be determined by the Geotechnical Engineer and/or a qualified Structural Engineer.

FILL MATERIAL

Materials to be placed in the fill shall be approved by the Geotechnical Engineer and shall be free of vegetable matter and other deleterious substances. Granular soil shall contain sufficient fine material to fill the voids. The definition and disposition of oversized rocks and expansive or detrimental soils are covered in the geotechnical report or Special Provisions. Expansive soils, soils of poor gradation, or soils with low strength characteristics may be thoroughly mixed with other soils to provide satisfactory fill material, but only with the explicit consent of the Geotechnical Engineer. Any import material shall be approved by the Geotechnical Engineer before being brought to the site.

PLACING AND COMPACTION OF FILL

Approved fill material shall be placed in areas prepared to receive fill in layers not to exceed 6 inches in compacted thickness. Each layer shall have a uniform moisture content in the range that will allow the compaction effort to be efficiently applied to achieve the specified degree of compaction. Each layer shall be uniformly compacted to the specified minimum degree of compaction with equipment of adequate size to economically compact the layer. Compaction equipment should either be specifically designed for soil compaction or of proven reliability. The minimum degree of compaction to be achieved is specified in either the Special Provisions or the recommendations contained in the preliminary geotechnical investigation report.

When the structural fill material includes rocks, no rocks will be allowed to nest and all voids must be carefully filled with soil such that the minimum degree of compaction recommended in the Special Provisions is achieved. The maximum size and spacing of rock permitted in structural fills and in non-structural fills is discussed in the geotechnical report, when applicable.

Field observation and compaction tests to estimate the degree of compaction of the fill will be taken by the Geotechnical Engineer or his representative. The location and frequency of the tests shall be at the Geotechnical Engineer's discretion. When the compaction test indicates that a particular layer is at less than the required degree of compaction, the layer shall be reworked to the satisfaction of the Geotechnical Engineer and until the desired relative compaction has been obtained.

Fill slopes shall be compacted by means of sheepsfoot rollers or other suitable equipment. Compaction by sheepsfoot roller shall be at vertical intervals of not greater than four feet. In addition, fill slopes at a ratio of two horizontal to one vertical or flatter, should be trackrolled. Steeper fill slopes shall be over-built and cut-back to finish contours after the slope has been constructed. Slope compaction operations shall result in all fill material six or more inches inward from the finished face of the slope having a relative compaction of at least 90 percent of maximum dry density or the degree of compaction specified in the Special Provisions section of this specification. The compaction operation on the slopes shall be continued until the Geotechnical Engineer is of the opinion that the slopes will be surficially stable.

Density tests in the slopes will be made by the Geotechnical Engineer during construction of the slopes to determine if the required compaction is being achieved. Where failing tests occur or other field problems arise, the Contractor will be notified that day of such conditions by written communication from the Geotechnical Engineer or his representative in the form of a daily field report.

If the method of achieving the required slope compaction selected by the Contractor fails to produce the necessary results, the Contractor shall rework or rebuild such slopes until the required degree of compaction is obtained, at no cost to the Owner or Geotechnical Engineer.

CUT SLOPES

The Engineering Geologist shall inspect cut slopes excavated in rock or lithified formational material during the grading operations at intervals determined at his discretion. If any conditions not anticipated in the preliminary report such as perched water, seepage, lenticular or confined strata of a potentially adverse nature, unfavorably inclined bedding, joints or fault planes are encountered during grading, these conditions shall be analyzed by the Engineering Geologist and Geotechnical Engineer to determine if mitigating measures are necessary.

Unless otherwise specified in the geotechnical report, no cut slopes shall be excavated higher or steeper than that allowed by the ordinances of the controlling governmental agency.

ENGINEERING OBSERVATION

Field observation by the Geotechnical Engineer or his representative shall be made during the filling and compaction operations so that he can express his opinion regarding the conformance of the grading with acceptable standards of practice. Neither the presence of the Geotechnical Engineer or his representative or the observation and testing shall release the Grading Contractor from his duty to compact all fill material to the specified degree of compaction.

SEASON LIMITS

Fill shall not be placed during unfavorable weather conditions. When work is interrupted by heavy rain, filling operations shall not be resumed until the proper moisture content and density of the fill materials can be achieved. Damaged site conditions resulting from weather or acts of God shall be repaired before acceptance of work.

RECOMMENDED GRADING SPECIFICATIONS - SPECIAL PROVISIONS

RELATIVE COMPACTION: The minimum degree of compaction to be obtained in compacted natural ground, compacted fill, and compacted backfill shall be at least 90 percent. For street and parking lot subgrade, the upper twelve inches should be compacted to at least 95 percent relative compaction.

EXPANSIVE SOILS: Detrimentally expansive soil is defined as clayey soil which has an expansion index of 50 or greater when tested in accordance with the American Society of Testing Materials (ASTM) Laboratory Test D4829-95.

OVERSIZED MATERIAL: Oversized fill material is generally defined herein as rocks or lumps of soil over six inches in diameter. Oversized materials should not be placed in fill unless recommendations of placement of such material is provided by the Geotechnical Engineer. At least 40 percent of the fill soils shall pass through a No. 4 U.S. Standard Sieve.

TRANSITION LOTS: Where transitions between cut and fill occur within the proposed building pad, the cut portion should be undercut a minimum of one foot below the base of the proposed footings and recompacted as structural backfill. In certain cases that would be addressed in the geotechnical report, special footing reinforcement or a combination of special footing reinforcement and undercutting may be required.