GEOTECHNICAL STUDY | INVESTIGATION

CLARIANA HOTEL ADDITION
PROPOSED NEW SIX-STORY ADDITION
PODIUM DECK WITH FIVE LEVELS ABOVE
AN EXISTING FIVE-STORY BUILDING

10 South Third Street
San Jose, California

Submitted to:
Ms. Tina Phan
C/O RSTP Investments, LLC
100 East Santa Clara Street
San Jose, California

File No. 17364-S
August 2018

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File No. 17364-S
August 01, 2018

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Subject: Proposed New Five-Story Addition
Clariana Hotel Addition | Existing Five-Story Building
Podium Deck with Five-Levels Above
10 South Third Street | San Jose, California
Geotechnical Study | Investigation

Dear Ms. Pham -

Advance Soil Technology, Inc. ("AST") is pleased to present herein the results of our geotechnical study/investigation for the Proposed New Six-Story Addition to the existing five-story Clariana Hotel Structure/Facility located at 10 South Third Street in San Jose, California.

In the proceeding sections of this report, enclosed please find the results of our geotechnical investigation/evaluation of the site of proposed improvement, results of subsurface exploration, laboratory testing and analysis, which formed the basis of our conclusions, considerations and recommendations related to the geotechnical and foundation design aspects of this project.

Based on the results of our investigation and analysis, it is our professional opinion that the site is suitable for the proposed development and construction of the above-mentioned structures, provided the recommendations presented in our report are incorporated in the design and during the construction phase of the project.

Please note that the conclusions and recommendations presented in this report are based on the subsurface soil investigation, variations between anticipated and actual soil conditions may occur in localized areas during the construction phase of the project. It is recommended that Advance Soil Technology, Inc. (AST) be retained during construction phase of the project to observe earthwork operations, and installation of foundations/verification to make changes and provide additional recommendations as deemed necessary, due to varying subsurface soil conditions. Furthermore, it is also recommended that AST review the plans and specifications pertaining to the grading and foundation aspects of the project, prior to completion of the final construction documents to assure compliance to the recommendations presented in this

report. Do not over-rely on the construction recommendations included in this report. These recommendations are not final, because they were developed principally from AST's professional judgment and opinion. Recommendations can be finalized only by observing actual subsurface conditions revealed during the construction phase of the project. AST cannot assume responsibility or/ liability for the recommendations reflected in this report, if we do not perform construction observation.

Sufficient monitoring, testing and consultation by AST should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining AST for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions. In the event conclusions or recommendations based on these data are made by others, such conclusions and recommendations are not our responsibility, unless we have been given an opportunity to review and concur with such conclusions and recommendation in writing.

We appreciate the opportunity for providing our services to you on this project and trust this report meets your needs at this time. If you have any questions concerning the information presented, please feel free to contact this office at (408)-261-1155 at your convenience.

Very truly yours,
ADVANCE SOIL TECHNOLOGY, INC.

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GEOTECHNICAL STUDY | INVESTIGATION
CLARIANA HOTEL ADDITION

PROPOSED NEW ADDITION OT THE EXISITNG BUILDING
CONCRETE PODIUM DECK WITH FIVE-LEVELS ABOVE
10 SOUTH THIRD STREET
SAN JOSE, CALIFORNIA

1.0 INTRODUCTION

Advance Soil Technology, Inc. ("AST") conducted geotechnical study/investigation at above-mentioned property for the proposed development will include a concrete podium deck with five-levels above the podium deck along with other surrounding improvements associated with them. At the time of this geotechnical study, the site of proposed development was covered with asphalt being utilized as a part of the parking lot for the existing hotel facility. The purpose of this geotechnical investigation was to evaluate the subsurface soil conditions and provide recommendations for the proposed improvements.

This report presents the results of our geotechnical investigation for the proposed addition to be associated with the existing five-story Clariana Hotel Facility with a partial below-grade basement on the southwest corner of the building located at the southeast corner of South Third Street and East Santa Clara Street in downtown San Jose, California. It is an irregular shaped parcel surrounded with existing commercial and retail developments. The property has a gated access from Third and Fourth Streets respectively.

Based on the results of our investigation and analysis, it is our professional opinion that the site is suitable for the proposed development and construction of the above-mentioned structures, provided the recommendations presented in our report are incorporated in the design and during the construction phase of the project. The subsurface materials encountered at the site generally consist of a layer of expansive and moderately compressible clays and deposits, which are underlain by interbedded layers of stiff clays and silts and medium dense to dense silty sands and gravelly sands. Some of the isolated sandy and gravelly sandy layers that are medium dense could be considered potentially liquefiable. Details regarding these subsurface conditions and their effect on foundation design are contained in this report; therefore, anyone relying on this report should read it in its entirety.

In the proceeding sections of this report, please find the recommendations for the earthwork operation/grading and foundation design, based on the results of this investigation that are to be incorporated in the design and during the construction phase of the project. Please refer to the site plan enclosed in Appendix "A" of this report for information pertaining to the site of proposed development and the locations of the cone penetrometer test (CPTs)/exploratory borings.

2.0 PROJECT DESCRIPTION

The site of proposed development is multi-parcel property owned by RSTP Investments, LLC and the portion of the parking lot in the back of the property is being developed as part of the addition to the existing hotel. The parcel (APN Parcel No. 467-23-88) currently under consideration for this development has an approximate combined square footage of ±10,072-square feet. It is being developed for as part of an addition to the existing hotel with a concrete podium deck with five-story structure above.

Based on our discussion with the Architect of Record (AOR) TCA Architects, it is our understanding that the following options are being explored:

- Steel Frame over Post Tensioned Deck
- Wood Frame Floors over Concrete Podium
- Column Spacing: 30'X30' Grid

The anticipated structural loads (dead plus live loads) for the proposed structural options are as stated and reflected in the table below: *these structural column and lateral loads need to be confirmed by the Project Structural Engineer, prior to the final design, so we can revise our recommendations accordingly.*

TABLE 1 – BUILDING LOADS & INFORMATION

Option	Building	No. of Stories	Column Loads (DL+LL) kips
1	Wood & Steel	6	585
2	Concrete	6	810

LATERAL LOADS -

- Assumed Estimated Lateral Load: 15 to 20 kips

Furthermore, it is our understanding that the topographic information, building plans and structural loads have not yet been finalized and are still considered to be schematic and conceptual in nature. Upon completion, the report needs to be updated for elevation, type of structure and structural loads, following the completion of the construction documents.

Based on the site reconnaissance, the site of proposed is a flat parcel of land and so is the natural topography of the area in general. Review of the existing available geological maps, aerial photographs and Google Earth Maps, the elevation at the site has been determined to be approximately in the range of (±84) to (±85)-feet above mean sea level (msl). Other associated improvements at the site will include underground utilities, exterior flatwork and landscape areas along with other appurtenant improvements associated with them.

3.0 PURPOSE AND SCOPE OF SERVICES

The purpose of our investigation has been to evaluate the existing subsurface soil conditions at the site as necessary to characterize subsurface strata, geologic hazards and develop geotechnical recommendations for the structural design and construction of the proposed development. The scope of our services for this study included the following:

- Reconnaissance of the site of proposed development, location of the existing underground utility lines and subsurface storage units (if any) with respect to the Exploratory Boring and CPT locations, prior to commencement of the drilling operation.
- Mark boring/CPT locations and notify Underground Service Alert at least (72)-hours prior to the planned exploration activity.
- Exploration of subsurface soil conditions at the site of proposed development included advancing (3)-Cone Penetrometer Tests and Drilling (1)-Exploratory Boring within the footprint for evaluating the subsurface soil conditions.
- Research and review pertinent geotechnical and geological maps and reports, relevant to the site regarding seismic and geologic history of the site and the immediate vicinity.
- Review of Existing Geotechnical Reports by other consultants for various projects in the area and historic aerial photographs for topographic changes and information pertaining to tonal variations (if any) at the subject property.
- Evaluation of the potential local and regional geologic hazards at the site, including liquefaction and resulting seismic settlements as per the requirement of the California Geological Survey (CGS)
- Perform laboratory testing and analysis of the soil samples to evaluate the engineering characteristics of the subsurface materials. Laboratory tests included Soil Classification, Gradation, Atterberg Limits Tests; In-situ Moisture Density, Corrosivity Analysis and other tests as deemed necessary.

Engineering analyses based on the results of laboratory testing and strength characteristics of the subsurface soils included the following:

- Soil classification and seismic design parameters based on ASCE 7-10 and CBC 2016 (California Building Code).
- Define specific site conditions and subsurface soil profile encountered in the cone penetrometer test and exploratory borings.
- Review of published geology and seismology reports and fault maps pertinent to the site area regarding the existing conditions in the vicinity of the site.
- Geology and seismicity of the project, including the appropriate soil profile type and other seismic parameters per 2016 Edition of CBC/ASCE 7-10.

- Seismic evaluation of the site including seismic compaction, ground shaking, distance from the earthquake faults, seismic coefficients per the requirements of 2016 CBC, site specific response spectra and future earthquake probability.
- Potential for anticipated settlement due to liquefaction during a seismic event, impact and mitigation.
- Impact of groundwater or/ seepage on below grade structures and retaining walls, etc. based on the depth, design and construction of the proposed improvements (if any).
- Overall assessment of the general surface and subsurface soil conditions and impact of the site settlement under structural loads.
- Recommendations for the type of foundation system based on anticipated structural loads/type of structures and proposed design requirements for Deep Foundation System (Auger Cast Pressure Grouted Displacement Piles), Structural Slab, Estimated total and differential settlements etc.
- Recommendations for retaining walls including lateral earth pressures (active and at-rest), seismic increments, drainage behind walls etc.
- Design criteria for structurally supported concrete slabs, non-structural slabs and modulus of subgrade reaction.
- Impact of soil corrosion on buried elements, including foundation, concrete, buried steel and underground utilities.
- Recommendations for overall site grading, engineered fill materials, lime treatment or/ chemical treatment of subgrade soils, surface and subsurface drainage.
- Recommendations for Flexible, Rigid and Permeable Pavement Design Sections for Low Impact Development (LID) areas.

4.0 SITE CONDITIONS

4.1 SURFACE CONDITIONS

The site of proposed development under this investigation is located in a commercial/ retail/business part of City of San Jose in the County of Santa Clara, California (Please refer to Plates (1) to (9) in Appendix "A" of this report). The site of proposed development is an irregular shaped parcel, bound by East Santa Clara Street and existing Clariana Hotel Building to the north, South Third Street to the West and other commercial/retail parcels to the east and west. It is a relatively flat parcel of land located to the southeast of East Santa Clara Street and South Third Street in the City of San Jose, California.

The site was identified in the United States Geological Survey (USGS), San Jose West Quadrangle Map 7.5-minute series, which was reviewed for this study. It is located approximately at the following co-ordinates: Latitude 37.33694° North and

121.88784° West Longitude respectively. According to the contour lines on the topographic map, ground surface elevations at the site range approximately from (±84) to (±85)-feet above mean sea level (msl). It is a relatively flat terrain, and so are the surrounding areas and the general vicinity with street section.

Based on our review of the FEMA electronic floodplain data (Federal Emergency Management Agency) Map No. 06085C0234H, the site is located in the Flood Zone D with a possibility of flood risk due to a levee. We recommend that the Project Civil Engineer be retained to confirm this information and verify the base flood elevation (if appropriate).

4.1 RECENT HISTORY

Historical aerial photographs from 1948 to 2017 and existing historical topographic maps from 1897 to 2012 were reviewed online to determine the usage of the property, site conditions and the vicinity in general. The subject property and the vicinity mostly appear be developed parcels with changes over the years. The existing building that exists on-site was originally constructed in 1913 with full basement and is now considered as a historical building and is part of City of San Jose's Historic resource Inventory.

4.2 FIELD INVESTIGATION

Middle Earth Geo-Testing and Exploration Geo-Services were subcontracted to provide CPT sounding and drilling services respectively to evaluate the subsurface soil conditions and the potential for liquefaction at the site. The subsurface soil investigation at the site was performed on February 15 and March 02, 2018. Prior to commencement of the drilling operation, the approximate exploratory boring and CPT locations were identified and marked as selected by AST based on the conceptual architectural site plan and the proposed location of the structures. All CPT locations were cleared by a private underground utility service locator and USA, prior to commencement of the drilling operation.

A total of (3)-CPT Soundings (Cone Penetrometer Tests) were advanced and at the site of proposed development. The CPT soundings were advanced using a limited access integrated electronic cone system to the depth of refusal and the exploratory borings were drilled to a depth of approximately (±60)-feet below the existing ground surface. The CPTs was performed in general accordance with ASTM D3441 and ASTM D5778. The information gathered from CPTs was used for identifying the potential for liquefaction, soft compressible soils and to evaluate the foundation support design.

As mentioned above, the cone penetrometer tests (CPTs) and the exploratory boring were advanced and drilled respectively to a depth of approximately (±60)-feet below the existing ground surface and within the areas of proposed development. The log for the CPTs showing tip resistance and friction ratio by depth, as well as interpreted SPT N-values, friction angle, soil shear strength parameters and an interpreted soil classification as presented in the following

sections of this report. Generally, cohesive soils (clays) have high friction ratios and low cone bearing and generate large pore pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate small excess pore water pressures. The interpretation of the soil properties from the cone data has been carried out using recent correlations developed by Robertson et al, 1986 and Olsen, 1988. It should be noted that it is not always possible to clearly identify a soil type based on cone bearing (Qc) and sleeve friction (Fs). In these situations, experience and judgment and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type. The CPT data collected from the sounding (cone bearing, sleeve friction, friction ratio and equivalent standard penetration test blow counts (N) versus penetration depth below the subsurface is presented in the attached report in Appendix "B" of this report.

The stratigraphic interpretation of the collected CPT data was performed based on relationships between cone bearing and sleeve friction versus depth of penetration. The friction ratio (Rf), which is sleeve friction divided by cone bearing is a calculated parameter which is used to infer the type of soil behavior.

Upon completion, the boreholes were grouted as per the requirements of the Santa Clara Valley Water District (SCVWD) and the drill cuttings were placed in drums and left on-site at the designated location. Please note that the location of the boring and CPTs were estimated by AST based on rough measurements from the existing features at the site and Google Earth. Topographic information needs to be updated upon completion of the site survey and the plans by the Civil Engineer.

4.4 SUBSURFACE SOIL CONDITIONS AT THE SITE

The subsurface soil conditions encountered at the location of the CPTs (cone penetrometer tests) and the Exploratory Boring that were advanced and drilled at the site of proposed development were as follows:

CPT-01: At this location, sands (mostly fill material) was encountered on the surface and the below this depth mostly clays and silty clays were encountered and extended to the bottom of the CPT. The soil layers were stiff to very stiff and the CPT was terminated at a depth of approximately (±60)-feet below the existing ground surface.

CPT-02: At this location, sands were encountered on the surface and below this depth mostly clays and silty clays with interbedded layers of very thin lenses of silty sand was encountered and extended to a depth of approximately (±42)-feet below the existing ground surface. At this depth, silty sand to sand and gravelly sand was encountered and extended to a depth of approximately (±56)-feet below the existing ground surface. These layers were medium dense to dense. At the above depth, clayey silty to silty clay was encountered and extended to the bottom of the CPT. The soil layer was stiff and the CPT was terminated at a depth of approximately (±60)-feet below the existing ground surface.

CPT-03: At this location, silty sand to clayey sand was encountered on the surface

and below this depth mostly clays and silty clays with interbedded layers of very thin lenses of silty sand was encountered and extended to a depth of approximately (±27.5)-feet below the existing ground surface. At this depth, silty sand to sand and gravelly sand was encountered and extended to a depth of approximately (±28.8)-feet below the existing ground surface. These layers were medium dense. At the above depth, clayey silty to silty clay was encountered and extended to a depth of approximately (±58.5)-feet below the existing ground surface. At this depth, dense sandy layers were encountered and extended to the bottom of the CPT and the CPT was terminated at a depth of approximately (±60)-feet below the existing ground surface.

Exploratory Boring EB-01: At the location of this exploratory boring, (3.0)-inches of asphalt over (5.0)-inches of concrete over (1.0)-inch of asphalt over two to three-inches of loose sandy gravel. Below the surface, black to dark brown lean fat clay to silty clay was encountered and extended to a depth of approximately (±8.0) to (±8.5)-feet below the existing ground surface. It was moist and stiff to very stiff. At the above depth, sandy silty clay to silty sandy clay was encountered and extended to a depth of approximately (±13.0) to (±13.5)-feet below the existing ground surface. It was moist and medium stiff. At this depth, dark to medium/tan brown to bluish gray silty clay was encountered and extended to a depth of approximately (±42.0) to (±42.5)-feet below the existing ground surface. It was moist and medium stiff to very stiff. At the above depth, silty sandy gravel to gravelly silty coarse sand was encountered and extended to a depth of (±52.0) to (±52.5)-feet below the existing ground surface. These sands were dense to very dense. Below this sandy layer, lean sandy clay to silty sandy clay was encountered and extended to a depth of (±58.0) to (±58.5)-feet below the existing ground surface. It was moist, very stiff and hard. At the above depth, clayey silty coarse sand was encountered and extended to the bottom of the boring. It was dense and the boring was terminated at a depth of approximately (±60.0)-feet below the existing ground surface.

Please note that the above information depicts the existing subsurface soil conditions at the specific boring location as reflected on the site plan. Stratification lines represent the approximate boundaries between the material types. The actual transitions between the materials may be gradual. Subsurface soil and groundwater conditions may vary at other locations from the conditions that were encountered at the boring locations with the passage of time. For subsurface soil information, please refer to the boring logs on Plates (15) thru (26) enclosed in Appendix “B” of this report.

4.5 GROUNDWATER

Based on the pore pressure data that was collected at the site from the CPTs, free groundwater elevation was determined to be at a depth of approximately (14) to (15)-feet below the existing ground surface as reflected in the table below. However, based on the review of the CGS Seismic Hazard Zone Map Report (058) for the San Jose West Quadrangle, it is our understanding that the historic groundwater elevation in the vicinity of this site is being considered to be at a depth of approximately (±10.0)-feet below the existing ground surface.

TABLE 2 - GROUNDWATER ELEVATIONS

Piezometer	Date	Depth to Groundwater (bgs ± feet)	Ground Elevation (msl) (± feet)	Groundwater Elevation (msl) (± feet)
CPT-01	2/15/2018	14.5	85.0	70.5
CPT-03	2/15/2018	15.9	85.0	69.1
EB-01	3/02/2018	14.0	85.0	71.0

*bgs – below existing ground surface
*msl - mean sea level (NAVD88)
*Elevation/Depth in feet (Civil Engineer)

Please note that seasonal groundwater studies were beyond the scope of this investigation, it shall be noted that groundwater level and elevation may fluctuate due to variations in rainfall, geological changes, temperature, natural springs, pumping water from the wells and other factors that were not evident at the time of this investigation. For information pertaining to the depth of groundwater elevation, please refer to the boring logs for information pertaining to the groundwater elevation enclosed in Appendix "C" of this report.

5.0 LABORATORY TESTING

Laboratory testing program performed on the soil samples collected from the site was directed towards a quantitative determination of the physical and engineering properties of the soils underlying the site. To evaluate the strength characteristics of the soil for the foundation engineering design, tests were performed on relatively undisturbed soil samples collected from various depths and from different boring locations at the site. In addition to the above, Unit Weight, Moisture Content, Atterberg Limits Tests to determine the Liquid Limit and Plasticity Index of the soil samples and Particle Size Distribution were performed on various samples.

- **Moisture Content:** The natural water content was determined (ASTM D2216) on all samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

- **Dry Densities:** In place dry density determinations (ASTM D2937) were performed on all samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

- **Plasticity Index:** Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which these materials, exhibit plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification

System and to evaluate the soil expansion potential. Results of these tests are shown on the boring logs at the appropriate sample depths and are attached in the appendix.

- **Washed Sieve Analyses:** The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on selected samples of the subsurface soils to aid in the classification of these soils.

The laboratory analyses were conducted in accordance with the criteria and guidelines set-forth by CGS Special Publication 117A - Guidelines for Evaluating and Mitigating Seismic Hazards in California, SCEC (1999) Southern California Earthquake Center Report. The results of these laboratory testing are presented on Plates (15) thru (29) in Appendix "B" of this report.

5.1 PLASTICITY & EXPANSION POTENTIAL

Atterberg Limits test performed on the selected samples from the upper layers of the soil revealed moderate to high plasticity index (PI) and expansion potential, when subjected to moisture fluctuations with plasticity indexes (PI) ranging between 20 to 22. For information please refer to Plate No. (27) In Appendix "B" of this report.

5.2 CORROSIVITY ANALYSIS

The National Association of Corrosion Engineers (NACE) defines corrosion as "deterioration of a substance or its properties because of a reaction with its environment." For inland construction, the "environment is the surrounding soil and the "substances" are reinforced concrete foundation or/ various types of steel substructures such as piers, pipes, metal fittings etc. that are in contact with the soil. The results of the analysis have been presented in the table below.

TABLE 3 – RESULTS OF CORROSIVITY ANALYSIS

ASTM Method	Chemical Analysis	Range of Results	Corrosion Classification
D512C	Chlorides	5 mg/kg	Non-corrosive
D516A (SM 4500)	Sulfates	86 mg/kg	Non-Corrosive
D2976/D4972/G51	pH	7.9	Non-Corrosive
G57	Saturated Resistivity	3477 ohm-cm	Non-Corrosive
D1498	Sulfide	Negative	PG&E
D516A (SM 4500)	Redox	538	Non- Corrosive

Furthermore, we recommend retaining a corrosion consultant to provide specific design recommendations for corrosion protection for buried metals and concrete

elements. The design team should also consider City of San Jose's specific requirements for underground improvements constructed in such environment. For example, City Standard Section 02661 requires cathodic protection such as epoxy coatings for buried metallic improvements. The 2016 CBC references the 2008 American Concrete Institute Manual, ACI 318 (Chapter 4, Sections 4.2 and 4.3) for concrete requirements. ACI Tables 4.2.1 and 4.3.1 provide sulfate exposure categories and classes, and concrete requirements in contact with soil based upon the exposure risk as excerpted below.

Please note that AST Inc. does not practice the field of corrosion engineering. It is recommended that a competent corrosion engineer could be retained to evaluate the corrosion potential on-site, if needed to provide recommendations and mitigation measures to minimize the impact of soil corrosion on buried concrete, metal pipes, fittings, post tensioning strands, anchors etc.

5.3 ENVIRONMENTAL SERVICES FOR SOIL OFFHAUL

Environmental services were not requested by the client or/ a part of this project scope. Therefore, AST has no environmental responsibility associated with this project. All existing and future environmental concerns associated with this site should be addressed by the project environmental consultant. In addition to the above, the environmental consultant should review and incorporate geotechnical recommendations provided in this report for compatibility, as part of the mitigation measures for existing environmental concerns that as they may exist at the present time due to subsurface soil conditions on-site.

6.0 GEOLOGIC SETTING

6.1 REGIONAL GEOLOGY

Geologically, the site under evaluation is located within the physiographic region known as the San Francisco Bay Area, which itself lies within the Coast Range geomorphic province of California, which consists of a series of northwest trending mountains and valleys along the western edge of the North American Continent. The San Francisco Bay Area itself lies within the Coast Range Geomorphic Province, a more or less discontinuous series of northwest trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting.

Geologic and Geomorphic structures within the San Francisco Bay Area are dominated by tectonic deformation and along the San Andreas Fault system. This right-lateral strike-slip fault extends on land from the Gulf of California in Mexico, to Cape Mendocino, on the Coast of Humboldt County in northern California. It forms a portion of the boundary between two independent tectonic plates on the surface of the earth. To the west of the San Andreas Fault, the Pacific plate moves north relative to the North American plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is distributed across the

San Andreas Fault and a number of other faults including the Hayward, Calaveras, and San Gregorio. Together, these faults are referred to as the San Andreas Fault system. The general trend of the faults within this system is responsible for the strong northwest-southeast structural grain of geologic and geomorphic features in the San Francisco Bay Area.

For most of the length of the San Andreas Fault, basement rock on the east generally consists of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. The Franciscan rocks are generally considered Jurassic and Cretaceous age (about 65 to 205 million years old). Overlying the basement rocks are Cretaceous marine, as well as Tertiary (about 65 to 1.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks typically have been extensively folded and faulted largely as a result of movement along the San Andreas Fault System over about the last 25 million years.

6.2 LOCAL GEOLOGY

The Santa Clara Valley is a broad alluvial plane between the Santa Cruz Mountains to the southwest and west, and the Diablo Range to the northeast. The site is located toward the eastern flanks of the Santa Clara Valley within the Evergreen Basin, an elongated trough between the Silver Creek Fault and the Hayward Fault. Based on the review of the regional geologic map enclosed in Appendix "A" of this report, Surficial mapping indicates that the site and adjacent areas are underlain by Latest Holocene alluvial fan levee deposits (Qhly), which are subdivided based on geomorphic relations (Knudsen et al., 2000, Witter et al., 2006, CDMG, 2000). Knudsen et al. describe the Qhly unit as: "Sediment deposited by streams emanating from canyons onto alluvial valley floors or alluvial plains. Alluvial fan levee sediment includes CL 63%; and other 37% with sand, silt, clay that are moderately to poorly sorted, and moderately to poorly bedded soil layers".

In the San Jose West Quadrangle there are 16 Quaternary units mapped by Knudsen and others (2000). Coalescing late Pleistocene and Holocene alluvial fans form a northeastward sloping bajada that covers much of the western and southern parts of the quadrangle. From west to east, the creeks that supply sediment to the alluvial fans are Calabazas, Saratoga, Tomas Aquinas, Los Gatos, and Ross. Near the heads of the fans, creeks have incised large, latest Pleistocene alluvial fan deposits (Qpf) consisting of coarse sand and gravel. Farther upstream, a few small upland valleys containing undifferentiated late Pleistocene to Holocene alluvium (Qa) are mapped in the foothills of the Santa Cruz Mountains. Stream terrace deposits (Qhty, Qht, and Qt) are mapped in the upper reaches of San Tomas Aquinas and Los Gatos creeks.

Sediment along the eastern margin of the quadrangle has been deposited near the axis of the Santa Clara Valley by the Guadalupe River, and in the northeastern corner of the quadrangle, by Coyote Creek. The site is in the Santa Clara Valley

Basin, which is bound on the north by San Francisco Bay, to the west and south by the Santa Cruz Mountains, and to the east by the Mount Diablo Foothills. The valley basin is a down-thrown block structure filled in with thousands of feet of marine and continental sediments derived from the surrounding mountains. These sediments consist primarily of marine, alluvial fan, and stream deposits that extend to great depths. They typically consist of discontinuous, interbedded layers of silt, clay, sand, and gravel. In the vicinity of the site, fine grained basin and marine deposits are predominant and consist of silt and clay. Thin interbedded, discontinuous and buried channel deposits of sand and gravel are within these fine-grained units. The silt and clay layers, interbedded with sand and gravel, extend to a depth greater than 100 feet beneath the ground surface

7.0 SEISMIC HAZARD ASSESSMENT

7.1 SEISMICITY/GROUND SHAKING

The San Francisco Bay Area is recognized by geologists and seismologists as one of the most seismically active regions in the United States. A broad system of inter-related northwest-southeast trending strike slip faults represents a segment boundary between the pacific and North American crustal plates. For 15 million years, the Pacific Plate has been slipping northwest ward with respect to the North American Plate (Atwater, 1970; Graham, 1978). The majority of the movement has been along the San Andreas Fault System; however, there are other faults within this broad system that have experienced movement at one time or another. The faults of Santa Clara County are characterized by both strike-slip and dip-slip components of displacement. There are three major fault systems that display large right-lateral offsets, the San Andreas, the Pilarcitos and the San Gregorio fault zones. These fault systems trend roughly N30W.

Movement on the many splays of the San Andreas Fault system has produced the dominant northwest-oriented structural and topographic trend seen throughout the Coast Ranges today. This trend reflects the boundary between two of the Earth's major tectonic plates: The North American plate to the east and the Pacific plate to the west. The San Andreas Fault system and its major branching faults is about 40 miles wide in the Bay area and extends from the San Gregorio Fault near the coastline to the Coast Ranges-Central Valley blind thrust at the western edge of the Great Central Valley as shown on the historical Fault Map.

The San Andreas Fault is the dominant structure in the system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Within the region, the San Andreas Fault system, which distributes shearing across a complex assemblage of primarily right lateral, strike-slip, parallel and sub-parallel faults that includes the Hayward, Calaveras, Monte-Vista Shannon, Sargent, Silver Creek and Cascade Faults. In general, the site will experience strong to severe seismic shaking during the lifetime of the proposed structures from the above and other faults mentioned in the following sections of this report. These faults that are capable of generating significant earthquakes are

generally associated with well defined areas of crustal movement, which generally trend in the northwesterly direction. Table III reflected below presents the considered active faults within a 100 km (62 miles) radius of the site of proposed improvements and they are as follows:

TABLE 4 - NEAREST SEISMIC SOURCE WITHIN 100 KILOMETERS

Fault Site	Distance (km)	Source Type	Maximum Magnitude	Slip Rate mm/yr
Silver Creek Fault	1.34	B	6.9	0.1
Cascade Fault	7.46	B	6.7	0.2
Hayward (SE Extension)	8.8	B	6.5	3.0
Monte – Vista Shannon	12.1	B	6.5	0.40
Calaveras Fault (South)	13.2	B	6.2	15.0
Hayward (Total Length)	14.0	A	7.1	9.0
Calaveras Fault (North)	14.0	B	6.8	6.0
San Andreas (1906)	19.6	A	7.9	24.0
Sargent Berrocal Fault	22.5	B	6.8	3.0
Zayante-Vergeles	28.6	B	7.0	0.1
Greenville	35.1	B	6.9	2.0
San Gregorio	43.1	A	7.3	5.0
Monterey Bay – Tularcitos	50.7	B	7.1	0.5
Ortigalita	54.3	B	6.9	1.0
Concord-Green Valley Fault	61.8	B	6.9	6.0
Paolo Colorado - Sur	63.9	B	7.0	3.0
Quien Sabe	65.1	B	6.5	1.0
San Andreas (Creeping)	66.3	B	5.0	34.0
Rinconada	74.5	B	7.3	1.0
Healdsburg Rogers Creek Fault	92.6	A	7.0	9.0
West Napa Fault	97.3	B	6.8	1.0
Point Reyes	107.4	B	6.8	0.30

7.2 HISTORICAL EARTHQUAKES

The earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2014 are reflected on Plate (7), enclosed in Appendix "A" of this report. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Plate 13) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, Mw, for this earthquake is about 6.25.

In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an Mw of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an Mw of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake occurred on 17 October 1989, in the Santa Cruz Mountains with a magnitude of Mw of 6.9.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated Mw for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably Mw of about 6.5) was reported on the Calaveras fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (Mw= 6.2). The most recent earthquake to affect the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, with an approximate magnitude MW of 6.0.

7.3 FUTURE EARTHQUAKE PROBABILITY

The presence of faults in the San Francisco Bay Area Region and the seismic activity in the recent past has led USGS to constantly upgrade the predictions for the possibility of the next major earthquake in the Bay Area. Per the information received from the review of the documents, it is our understanding that The Working Group on California Earthquake Probabilities (2008) has concluded that the probability of a magnitude 7.0+ earthquake in the San Francisco Bay Area over the next 30 years is 63 percent. This probability is a low estimate since only three active faults in the area; the Hayward Fault, San Andreas Fault and Rodgers Creek Fault were included in the study.

Schwartz (1994) concludes that the probability of occurrence of one or more magnitude 6.7+ earthquakes in the Bay Area is substantially higher than 63 percent, possibly as high as 99.7 percent. It shall be noted that significant earthquakes could occur on an active fault or a potentially active fault for which probabilities might not have been estimated. Even though research on earthquake predictions has increased tremendously in recent years, seismologists still cannot predict when or where an earthquake of that magnitude will occur. Based on the

information that is available, it is our understanding that the site will likely be subjected to at least one moderate to severe earthquake within 50 years of the proposed construction. During such an earthquake, the possibility of fault offset could be perceived to be low. However, strong ground shaking will be experienced at the site. The probability of the maximum moment magnitude earthquake occurring during the next (30) year period are presented in the table below and they are as follows:

TABLE 5 – EARTHQUAKE PROBABILITIES

Fault Segment	(30)–Year Probability (%) Earthquake Magnitude M = 6.7
Hayward (North & South)	31
San Andreas (Peninsula Segment)	21
Calaveras	7
San Gregorio	6
Concord-Green Valley	3
Greenville	3
Mount Diablo Thrust	1

8.0 GEOLOGICAL HAZARDS

This section presents our geologic hazards review per the requirements of the California Geological Survey (CGS) for the proposed improvement located at 10 South Third Street in San Jose, California. The site is located approximately at Latitude 37.33694° N and Longitude -121.88784° W. Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, liquefaction, and round lurching. In the proceeding sections, potential geologic hazards related to the site are addressed as below:

8.1 SURFACE FAULT RUPTURE

Earthquakes generally are caused by a shift or displacement along a discrete zone of weakness, termed a fault, in the Earth's crust. Surface fault rupture, which is a manifestation of the fault displacement at the ground surface, usually is associated with moderate- to large-magnitude earthquakes (magnitudes of about 6 or larger). Generally, primary surface fault rupture occurs on active faults having mapped traces or zones at the ground surface. In other words, major faults tend to rupture on pre-existing planes of weakness. The amount of surface fault displacement can be as much as (20)-feet, depending on the earthquake magnitude and other factors. Large earthquakes also can “trigger” slip on adjacent

faults, which may or may not be mapped at the surface, causing co-seismic ground deformation (*e.g.,* Lawson 1908; Schmidt and Others, 1995).

Potential surface fault rupture hazards exist along the known active faults in the greater San Francisco Bay Region. The faults which have been identified as potential surface rupture hazards by the California Geologic Survey in close proximity to the site include the Hayward, Calaveras, San Andreas, San Gregorio and Monte Vista-Shannon Faults, which are located in close proximity to the site. These faults show historic (last 200 years) displacement associated with mapped surface rupture or/ surface creep.

The site is not located within a currently designated California Alquist-Priolo Earthquake Fault Zone (CDMG, 1982), formerly known as a Special Studies Zone, City of San Jose Fault Hazard Zone (CSJ, 1983), or a Santa Clara County Fault Rupture Hazard Zone (SCC, 2002). No indications of active faulting were observed in aerial photographs or in the field, nor have any surface fault expressions been mapped across the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

8.2 HISTORIC GROUND FAILURES

Many historical earthquakes have occurred on active faults and fault branches throughout coastal California. Hayward and San Andreas Fault are considered to be some of the major active faults of the region generating damaging earthquakes in 1836 and 1868, as well as the great San Francisco Earthquake of 1906, which had an approximate Richter Magnitude of 8.3, and the Loma Prieta Earthquake of 1989.

Lawson (1908) reported considerable damage from the 1906 earthquake in the Bay Area. Very few observations of the 1868 Hayward earthquake record specific evidence for liquefaction in the region. However, Lawson (1908) reports that water spurted up in the streets of San Jose, and out in the road between Milpitas and San Jose, to the height of several feet. Lawson (1908) described damage resulting from the 1906 earthquake in the downtown area of San Jose as; severe structural damage too many brick and mortar buildings and many chimneys were toppled. The main part of San Jose and surrounding areas shook at Rossi-Forel intensity VIII (many chimneys fall) to IX (partial or complete collapse of some buildings). Witter et. al., (2006) indicate four historical occurrences of ground deformation within the "Qhf" mapping unit within the region. These ground failures were concentrated close to the bay several miles north of the site.

Youd and Hoose (1978) report that following the 1906 earthquake numerous cracks were observed on both sides of Coyote Creek from Milpitas "all the way to San Jose." They also report that within the valley no cracking was observed along Alum Rock Road east of Coyote Creek. No observations of liquefaction were recorded in the San Jose West Quadrangle following the 1989 Loma Prieta earthquake (Tinsley and others, 1998). Co-seismic contractional deformation occurred within the south bay area during the 1989 Loma Prieta earthquake in an

area traversed by several northwest-trending reverse faults, including the Monte Vista, Shannon, and Berrocal faults (Haugerud and Ellen, 1990). These breaks coincide with the projections of vegetation lineaments and linear depressions on both the northwestern and southeastern sides of Los Gatos Creek. Schmidt et al., (1995) documented cases of co-seismic damage in the region however they did not document any evidence of such damage at or/ near the site of proposed improvement.

8.3 LIQUEFACTION POTENTIAL

Liquefaction is a soil behavior phenomenon in which a soil loses a substantial amount of strength due to high excess pore-water pressure generated by strong earthquake ground shaking. Recently-deposited (i.e., within about the past 11,000 years) and relatively un-consolidated soils and artificial fills located below the ground water surface are considered susceptible to liquefaction (Youd and Perkins, 1978). Typically, the soils that are most susceptible to liquefaction include relatively clean, loose, uniformly graded sand, silty sand, and non-plastic silt deposits (e.g., National Research Council, 1985). Liquefaction is the transformation of clean, loose saturated sand and silt (cohesionless soil) and some sensitive clayey silt soil from a solid state to a semi-liquid state. This transformation occurs during a seismic event/ground shaking, when soil is subjected to cyclic stresses that cause increase in hydrostatic pressure that induces liquefaction. The resulting upward flow of water will often turn cohesionless soil into a liquefied condition (loss of density).

At the ground surface, liquefaction is manifested by the formation of sand boils, ground cracking, and lateral spreading and in some cases development of quicksand like conditions, which results in the settlement or movement of the structures. Cohesive soils are generally not considered susceptible to soil liquefaction. However, the soils that are most susceptible to liquefaction are loose to moderately dense, saturated granular soils. Sensitive clays are vulnerable to significant strength loss under relatively minor strains during seismic event. The site of proposed development is located within the liquefaction zone as outlined by CGS Seismic Hazard Zone Report (058) for the San Jose West Quadrangle and could experience moderate to low vertical settlement during a seismic event.

8.3.1 Liquefaction Evaluation and Analysis Criteria

The site is within a zone designated with the potential for liquefaction, as identified by the California Geologic Survey (formerly the California Division of Mines and Geology) in a map titled, State of California Seismic Hazard Zones, San Jose West Quadrangle 7.5-Minute Quadrangle, Santa Clara County prepared by the California Geologic Survey. Specifically, the map shows the site is in an area "where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693 (c) would be required." We performed our liquefaction analysis in accordance with the State of California Special Publication 117A, Guidelines for Evaluation and Mitigation of

Seismic Hazards in California and following the procedures presented in the 1996 NCEER and the 1998 NCEER/NSF workshops on the Evaluation of Liquefaction Resistance of Soils (Youd and Idriss 2001). The NCEER methods are updates of the simplified procedures developed by Seed et al. (1971). To estimate volumetric strain and associated liquefaction-induced settlement, we used the procedure developed by Tokimatsu and Seed (1987). These methods are used to estimate a factor of safety against liquefaction triggering by taking the ratio of soil strength (resistance of the soil to cyclic shaking) to the seismic demand that can be expected from a design level seismic event. Specifically, two distinct terms are used in the liquefaction triggering analyses.

- Cyclic Resistance Ratio (CRR), which quantifies the soil's resistance to cyclic shaking; a function of soil depth, density, depth of groundwater, earthquake magnitude, and overall soil behavior

- Cyclic Stress Ratio (CSR), which quantifies the stresses that are anticipated to develop during cyclic shaking and is dependent on the duration, magnitude, and peak ground acceleration associated with a design seismic event.

The factor of safety (FS) against liquefaction triggering can be expressed as the ratio of CRR over CSR. For our analyses, if the FS for a soil layer is less than 1.3, it is considered possible that the soil layer may liquefy during a large seismic event. For our calculations of estimated liquefaction-induced settlement, we assumed layers with a FS equal to or greater than 1.3 will not experience liquefaction-induced settlement.

The primary design parameters used in our liquefaction triggering calculations are summarized in the table below. Our CRR calculations are based on the CPT tip resistance. The CPT tip pressures were normalized and corrected for overburden pressure; fines content, CPTs tip resistance were also modified to account for thin layers, where appropriate. The CPT method also utilizes the soil behavior type index (IC) and the exponential factor "n" applied to the Normalized Cone Resistance "q" to evaluate the cohesive nature of the soil. According to Bray and Sancio (2006) and Seed et al. (1982), soil with these properties is not susceptible to liquefaction. The lower gravelly deposits have less than 5 percent fines and low SPT blow counts (N).

Our CRR calculations are based on Standard Penetration Test (SPT) blow counts in the borings and CPT tip resistance. The SPT blow counts and CPT tip pressures were normalized and corrected for overburden pressure, fines content, CPTs tip resistance were also modified to account for thin layers, where appropriate. The CPT method also utilizes the soil behavior type index (IC) and the exponential factor "n" applied to the Normalized Cone Resistance "q" to evaluate the cohesive nature of the soil. All of these are included in our analyses.

The CSR is obtained using the equations presented in NCEER paper and is based on the density of the soil, the depth to design groundwater, the estimated peak horizontal acceleration at the ground surface (PGAM), and a stress reduction

coefficient (rd). However, for this liquefaction study, the data collected from CPTs (CPT-01 thru CPT-03) were utilized for the liquefaction settlement and lateral spreading analyses.

In our analyses soil that has significant amount of plastic fines, Ic greater than 2.6 were considered too cohesive to liquefy; a corrected cone tip resistance qc1N greater than 160 tons per square foot (tsf) or a Standard Penetration Test blow count, (N1)60 greater than 30 blows/ft were considered too dense to liquefy. Because the predominant earthquake is a moment magnitude 7.9, the cyclic resistance ratio (CRR) has been scaled to a moment magnitude of 7.5 using magnitude scaling factors developed by Idriss (Youd and Idriss, 2001).

TABLE 6 - PARAMETRS USED IN LIQUEFACTION EVALUATION

Parameter	Value
Estimated High Groundwater elevation	±10.0-Feet (bgs)
Peak Ground Acceleration (PGAM)	0.50g
Predominant Earthquake Moment magnitude, Mw	7.9
Minimum Factor of Safety	1.3
Correction for SPT Sampler blow count to SPT N-Value	1.4
Blow Count Conversion Factor S&H to SPT N-Value	0.84
Hammer Efficiency	0.84
CPT conversion factor for tip resistance to SPT N-value	4 to 5

AST understands that the State guidelines (SCEC, 1999) recommend a minimum depth of (50)-feet below lowest proposed finished grade for evaluation of liquefaction potential. We performed liquefaction hazard analysis in accordance with the current standard of practice, as outlined in the *Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction in California* requires liquefaction analysis to a depth of (±50)-feet below the existing ground surface. However, the CPTs/exploration points at this site were advanced to a minimum depth of (±60)-feet below the existing ground surface.

We have used the computer software program CLiq (Version 2.2.0.37, Geologismiki) and the in-situ soil parameters measured in the CPT soundings. The software utilized the 1998 National Center for Earthquake Engineering Research (NCEER) method of analysis which was developed with the broad consensus of national geotechnical earthquake engineering experts. We have evaluated the potential for liquefaction and resultant settlements at the site using the CPT data and the methodology of Youd et. al. (2001). In this liquefaction analysis, we have

incorporated an earthquake moment magnitude (Mw) of 7.9 and a groundwater depth of (±10.0)-feet *(In-situ groundwater elevation varying from (14) to (15)-feet (bgs)* below the existing ground surface). The groundwater elevation used in our analysis was obtained from an historic high groundwater exhibit within the CGS Seismic Hazard Report (058) for the San Jose West Quadrangle. Our evaluation was performed in accordance with the methodology of Idriss and Boulanger (2008) and considered cyclic softening in clayey soils. The 2016 CBC indicates a ground motion/Peak Ground Acceleration (PGAM) of 0.50g is acceptable for design, which was obtained from the USGS Design Maps Summary Report.

In estimating post-liquefaction settlement at the site, we have implemented a depth weighting factor proposed by Cetin (2009). Following evaluation of 49-high quality cyclically induced ground settlement case histories from seven different earthquakes, Cetin proposed the use of a weighting factor based on the depth of layers. The weighting procedure was used to tune the surface observations at liquefaction sites to produce a better model fit with measured data.

Aside from the better model fit it produced, the rationale behind the use of a depth weighting factor is based on the following: 1) upward seepage, triggering void ratio redistribution, and resulting in unfavorably higher void ratios for the shallower sub-layers of soil layers; 2) reduced induced shear stresses and number of shear stress cycles transmitted to deeper soil layers due to initial liquefaction of surficial layers; and 3) possible arching effects due to non-liquefied soil layers. All these may significantly reduce the contribution of volumetric settlement of deeper soil layers to the overall ground surface settlement (Cetin, 2009).

On the basis of the results of our analyses, we conclude several of these layers could potentially liquefy during a major earthquake and may experience liquefaction induced settlement. A summary of our CPT subsurface data for the exploration points, as well as other pertinent parameters regarding liquefaction triggering and associated settlement are presented in our liquefaction analysis enclosed in Appendix “D” of this report.

8.3.2 Settlement Due to Liquefaction

Based on the results of the analysis, the potential for liquefaction and/or cyclic softening at the site varies with respect to the location of the boring and CPTs with the maximum total settlement being (2.13)-inches with a differential settlement (2/3 of the total settlement) to (1.42)-inch. The likely consequence of potential liquefaction at the site is ground surface settlement, if liquefaction were to occur. Please refer to the results of our analysis in the table below. Therefore, we recommend that all structural elements be designed to accommodate (±1.5)-inches of differential seismic settlement across a horizontal span of (30)-feet. Results of liquefaction analysis are presented as attachment of this report and in the reflected in the table below.

Table 7 – SETTLEMENT DUE TO LIQUEFACTION

Location	Total Settlement (inches)	Differential Settlement (inches)
CPT-01	0.4	0.27
CPT-02	2.13	1.42
CPT-03	0.62	0.42

8.3.3 Lateral Spreading

Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported down slope or in the direction of a free face by earthquake and gravitational forces. As the failure tends to propagate as block failures, it is difficult to analyze and estimate, where the first tension crack will form. In general, lateral spreading is considered when an open face (Height = D) is within about 40D of a site. Since the project site is not located within the established criteria and the surrounding areas are generally level with no open face in the immediate vicinity of the site. The nearest open face channel (Guadalupe River Channel) is located approximately more than (≥3800)-feet) to the west of the site of proposed development. The potential for lateral spreading to occur at a site is typically evaluated using an empirical relationship developed by Youd, Hansen, and Bartlett (2002). This relationship incorporates the thickness, fines content, mean grain-size diameter, and relative density of the liquefiable layer, the magnitude and distance of the earthquake from the site, the slope of the ground surface, and boundary conditions (such as a free face or/ edge of shoreline), to estimate the horizontal ground movement.

The site of proposed development and the surrounding areas are relatively flat. We used the CPT data and the Revised Multi-Linear Regression Equations for Prediction of Lateral Spread Displacements (Youd et al. 2001) to evaluate the potential for lateral spreading. These regression equations indicate that sandy soil layers with (N1)60 values greater than 15 blows per foot are sufficiently dense to resist the potential for lateral spreading (Youd et al 2001). In addition to the above, the potentially liquefiable layers are generally discontinuous and there is no historical evidence of lateral spreading that has been recorded in the vicinity of the site. Considering these conditions, we judge the potential for lateral spreading is low.

8.4 SAND BOILS

Ishihara (1985) has shown that the presence of a sufficient thickness of a non-liquefiable surface layer may prevent the observable effect of at-depth liquefaction from reaching the surface. A more recent study by Youd and Garris (1995) expanded on the work of Ishihara to include data from over 300 exploratory borings, 15 different earthquakes, and several ranges of recorded peak ground

acceleration. Considering the capping effects from overlying non-liquefiable layers and additional engineered fill to be placed to raise site grades, the soil above the potentially liquefiable soils are thick enough to resist upward pressure and the liquefiable lenses are thin enough to provide only a limited reservoir of water. Accordingly, we do not anticipate the occurrence of sand boils at the site should liquefaction occur.

8.5 SEISMIC SETTLEMENT/CYCLIC DENSIFICATION

Seismic densification can occur during strong ground shaking in loose, clean granular deposits above the water table and over the hard / dense material, resulting in ground surface settlement. The soil layers below the subsurface mostly consist of clays that are too cohesive with thin layers of sand and are usually not susceptible to densification. We judge the settlement due to seismic densification to be low.

8.6 DIFFERENTIAL COMPACTION

If near surface soil varies in composition both vertically and laterally, strong earthquake shaking can cause non-uniform compaction of soil strata, resulting in movement of near surface soils. Based on the nature of the soils encountered at the site, we judge the probability of differential compaction at the site to be low.

8.7 GROUND LURCHING & LANDSLIDING

Seismically Induced Ground Lurching/Landsliding is a lateral movement of portions of the ground normally accompanied by fissuring perpendicular to the direction of lurching. It usually occurs along steep slopes and embankments, such as unconsolidated and unsupported stream banks. However, there are no open channels or/ banks located in immediate vicinity of the site. Our interpretation of the landslide potential is based on our geologic reconnaissance, aerial photographs interpretation and review of the previous geotechnical investigation, research of published maps and reports and our subsurface exploration. Since the site is located in a flat area, we judge the possibility of Ground Lurching/Landsliding to be low.

8.8 SEISMICALLY INDUCED WAVES – TSUNAMIS & SEICHES

Tsunamis are seismic sea waves that are typically, an open ocean phenomenon caused by underwater landslides, volcanic eruptions and seismic events, which primarily impact low-lying coastal areas. Seiches are earthquake-generated waves or oscillations (sloshing) of the water surface in restricted bodies of water, such as the San Francisco Bay. The 1868 earthquake on the Hayward fault is reported to have generated Seiches activity in the bay.

Seiches are extremely rare in the Bay, which generally attenuates such activity due to its irregular shape and shallow shoreline. Ritter and Dupre (1972) indicate that the coastal lowland areas, immediately adjacent to San Francisco Bay and the

Pacific Ocean Coastline are subject to possible inundation from tsunami with a run up height of 20-feet at the Golden Gate Bridge. Based on the review of the Tsunami Inundation Maps for Emergency Planning for the Santa Clara County, it is our understanding that the site is not located within the tsunami hazard zone mapped by the California Emergency Management Agency (CalEMA). Furthermore, the site is not located next to any major drainage areas that would be affected by or generate a seismically induced wave. Therefore, this potential is judged to be low.

8.9 FLOODING AND RESERVOIR INUNDATION

Based on our internet search of the Federal Emergency Management Agency (FEMA) and the review of the available FEMA Flood Zone Maps, it is our understanding that the site is not located in a flood zone. However, we recommend that the Project Civil Engineer be retained to confirm this information and verify the base flood elevation (if appropriate).

9.0 CBC - CODE BASED SEISMIC DESIGN CRITERIA -

9.1 2016 CBC SITE CLASSIFICATION & SEISMIC COEFFICIENTS

The following table summarizes site specific design criteria obtained from 2016 California Building Code based on 2015 International Building code (IBC) and ASCE 7-10. The data was calculated using U.S. Seismic Design Maps web application provided by the USGS. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613A.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented below are for the risk-targeted maximum considered earthquake (MCER).

Based on the results of the CPTs and exploratory boring advanced/drilled at the site and review of local geology, the site is underlain with medium stiff to very stiff clayey soils and there are several thin interbedded layers of potentially liquefiable material throughout subsurface profile. However, these layers are relatively thin, are separated by either thick clay or/ medium dense to dense sandy layers with typical SPT “N” values being $15 \leq N \leq 50$ blows per foot. Therefore, even accounting for the softening of the potentially liquefiable material, we have classified the site with the soil classification “D” with alluvium soil conditions.

The mapped spectral acceleration parameters Ss and S1 were calculated using the USGS computer Program Earthquake Ground Motion Parameters, version 5.10 revision date February 10, 2011 based on the site coordinates presented in table below and the site classification along with other factors used to determine the seismic coefficients and parameters. The mapped acceleration parameters were adjusted for local site conditions based on the average soil conditions for the upper 100-feet of the soil profile. We estimated the average shear wave velocity of the soil profile to be approximately 270 m/s based on the alluvium soil conditions encountered in our explorations and available geologic data.

TABLE 8 – ASCE 7-10 SEISMIC DESIGN PARAMETERS

Parameter	Values
Site Class	D
Site Latitude	37.33695°N
Site Longitude	-121.88785°W
Risk Category	I, II or III
Seismic Category	E
0.2-Second Period Mapped Spectral Acceleration, Class B (short), Ss	1.500g
1-Second Period Mapped Spectral Acceleration, Class B, S1	0.600g
Short Period Site Coefficient - Fa	1.0
Long Period Site Coefficient - Fv	1.5
0.2-second Period Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects (short), SMS	1.500g
1.0-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for the Site Effects, SM1	0.900g
0.2-second Period, Design Earthquake Spectral Response Acceleration, SDS	1.000g
1.0-second Period, Design Earthquake Spectral Response Acceleration, SD1	0.600g
Long Period Transition - TL	12-secs
Site Coefficient FPGA	1.0
Mapped Geometric Mean PGAM	0.5g

Conformance to the criteria reflected in the tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive. Table above presents the mapped maximum considered geometric mean (MCEG) seismic design parameters for the projects located in Seismic Design Categories of D through F in accordance with CBC 2016.

10.0 DISCUSSION & CONCLUSIONS

Based on the results of our investigation/study, it is our professional opinion that the site of proposed improvement could be developed as planned; provided if the conclusion and recommendations of this report are incorporated in the design and during the construction phase of the project. However, if there are any changes that are being proposed to the original scope of work or/ to the nature of development, location, design changes, type of structures or/ addition of any subsurface structures, then additional CPTs or/ exploratory boring may be required to provide additional recommendations.

The primary geotechnical issues for this site are the presence of soft, compressible clay with interbedded layers of medium dense to very dense sands, seismically-induced settlement, undocumented fills, buried structures, impact on existing adjacent shallow building foundation and basement structures and potential for significant settlements and proposed below grade swimming pool. Our conclusions and recommendations regarding these issues are discussed in the sections below:

- Shallow groundwater Elevation
- Potential for Liquefaction
- Presence of high expansive soils
- Undocumented fill and Buried Structures
- Impact to Existing Adjacent Foundation and Basements
- Foundation and Potential for significant settlements
- Plans and Specifications

10.1 SHALLOW GROUNDWATER

Shallow groundwater was encountered in the CPTs ranging approximately (14) to (15)-feet below the existing ground surface (bgs) or/ to an elevation of (71.0) to (70.0)-feet (msl) depending upon the location of the CPTS. However, based on the review CGS Seismic Zone maps, it is our understanding that the historic groundwater elevation at the site is being considered to be at a depth of (±10)-feet below the existing ground surface (bgs).

10.2 POTENTIAL FOR LIQUEFACTION

Due to the presence of existing shallow groundwater elevation at the site and the presence of medium dense sand and silty layers, the potential for liquefaction is considered to be high. Settlement due to liquefaction ranges form (0.4) to (2.13)-inches across the site. This settlement is in addition to the settlement due to static loads and downdrag. Therefore, the foundation should be designed to resist liquefaction-induced in addition to the settlement due to static loads.

10.3 PRESENCE OF HIGH EXPANSIVE SOILS

Highly expansive clays were also encountered in the upper layers of the subsurface soils. These expansive near-surface soils are subject to high volume changes

during seasonal fluctuations in moisture content, which can cause cracking of shallow foundations, floor slabs, concrete flatwork and pavements, exterior concrete flatwork such as sidewalks. These effects can be mitigated by moisture conditioning the expansive soil and placing a minimum of (18)-inches of non-expansive imported fill material or/ a minimum of (18)-inches of lime treated material, below all slabs below the zone of severe moisture change or/ designing foundations and slabs to resist ground movements associated with the volume changes. Recommendations for subgrade preparation, engineered fill, placement and compaction of non-expansive material are presented in the "Recommendation Section" of this report.

10.3 FOUNDATION SUPPORT

Generally, the site is underlain with moderate to high compressible alluvial subsurface soils (Holocene deposits) with some variable zones of sand that are susceptible to liquefaction. During the site investigation, buried structures were encountered at the location of the borings. Furthermore, the demolition and excavation of the surface and subsurface structures that was carried out in the past and backfilled will result in excessive settlement. If the structures are constructed at grade on shallow foundations, the new stresses from the structural loads would result in excessive total and differential settlement of the proposed structures. Therefore, we would recommend that heavily loaded six-story hotel structure addition be supported on a deep foundation system such as Auger Cast Piles (ACP) or/ Auger Cast Displacement Piles (ACDP) that gain support in the stiff clay/silt and medium dense to dense sands below the soft compressible and potentially liquefiable soils. Please refer to the foundation section for information pertaining to the ACP/ACDP.

10.4 AUGER CAST DISPLACEMENT PILES (ACDP)

ACDPs are installed by drilling to the required depth with a hollow-stem continuous-flight auger. The auger has a reverse flight near the tip which results in displacement and densification of the surrounding soil and results in little to no spoils. When the auger reaches the required depth, cement grout or concrete is injected through the bottom port of the hollow stem auger. Grout or concrete is injected continuously as the augers, still rotating in a forward direction, are slowly withdrawn, replacing the displaced soil. While the grout is still fluid, a steel reinforcing cage is inserted into the shaft. ACDPs can also range in diameter; however, (14) to (18)-inch-diameter ACDPs are commonly used in the Bay Area.

10.5 AUGER CAST PILES (ACP)

ACPs are installed by drilling to the required depth with a hollow-stem, continuous-flight auger. When the auger reaches the required depth, cement grout or concrete is injected through the bottom part of the hollow stem auger. Grout or concrete is injected continuously as the augers are slowly withdrawn. While the grout is still fluid, a steel reinforcing cage is inserted into the shaft. ACPs can range in diameter; however, (18) and (24)-inch-diameter ACPs are common in this

area. Because these piles are not displacement piles they generate a significant amount of drilled soil cuttings.

10.6 PLANS & SPECIFICATIONS

To verify our established recommendations have been properly implemented, we recommend that AST be retained to review the final construction plans and specifications and to observe the earthwork and foundation depth during the construction phase of the project. These recommendations are not final, because they were developed principally from AST's professional judgment and opinion. AST's recommendations can be finalized only by observing actual subsurface conditions revealed during the construction phase of the project. Retaining AST for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions. In the event conclusions or recommendations based on the submitted data are made by others, then such conclusions and recommendations are not our responsibility, unless we have been given an opportunity to review and concur with such conclusions and recommendation in writing.

11.0 RECOMMENDATIONS -

11.1 EXISITNG UTILITIES

Existing utilities located within the areas of proposed improvements should be removed in their entirety. Utilities within the proposed areas of improvements could be considered for in-place abandonment, provided they do not conflict with new improvements, that the ends and all laterals are located and completely grouted, and the previous fills associated with the utility trenches do not pose a risk to the proposed improvements. Utilities outside the areas of proposed improvements should be removed or abandoned in-place by grouting or plugging ends with concrete. Fills associated with utilities abandoned in place could pose some risk of settlement; utilities that are plugged could also pose some risk of future collapse or erosion should they leak or become damaged. The potential risks are relatively low for smaller diameter pipes abandoned in place and increasingly higher with increase in diameter.

11.2 EARTHWORK SECTION|DEMOLITION, CLEARING & SITE PREPARATION

Any areas to be graded should initially be cleared of all obstructions, including the buried foundation system (if any), footings, underground utility pipes, including drain lines, landscape areas, brush, trees not designated to remain, debris, stumps, root balls, existing pavements, rubble and debris should be removed and hauled off from the site. Roots greater than ½-inches diameter shall be removed completely. All active or inactive utilities within the construction area should be protected, relocated or abandoned. Any pipes that are abandoned in place should be filled/pressure grouted with non-shrink grouts/cement slurry. Depressions/ Holes/excavated areas resulting from the removal of the existing foundation or/

underground obstructions, trees/root balls etc. below the existing or/ proposed finished subgrade levels should be cleared for engineered fill given below. Prior to the start of grading and subgrade preparation operations, the site should first be cleared and stripped to remove all surface vegetation, organic laden topsoil, existing underground utilities and other subsurface structures (if any). Over-excavation and re-compaction should extend laterally a minimum of (5)-feet beyond the limits of structures, and (2)-feet beyond the flatwork, vertical curbs/curb and gutter areas.

Please note that all earthwork operations at the site including demolition and fill placement shall be observed by Advance Soil Technology, Inc. ("AST") or/ by its representative. It is important that during the demolition, removal of buried footings, buried structures, underground utilities, below grade structures, stripping and scarification process our representative be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during our geotechnical/field investigation.

11.3 PAD PREPARATION

Following the clearing and grubbing operation, demolition, excavation, removal of the existing fills and disturbed soils areas/backfill of the existing subsurface structures and underground utilities, the existing exposed grade shall be scarified to a minimum depth of (12)-inches, moisture conditioned as needed and compacted to a minimum of 95% relative compaction as per ASTM D1557, prior to any placement of any fill material. Any additional fill placed on the pads shall be compacted to a minimum of 95% relative compaction as per ASTM D1557.

After the completion of the above, the entire building pad area shall be graded/excavated to a depth of the subgrade (native soil) to accommodate a structural section under the concrete slab for a minimum of (6)-inch reinforced concrete slab or/designed thickness or/ structural slab, vapor membrane (Perminator), (4.0)-inches of free draining gravel and a minimum (18)-inches of non-expansive soil or/ a minimum of (18)-inches of lime treated material. The bottom of the excavation shall be uniformly graded to an even elevation; scarified to a depth of (12)-inches below the excavated bottom, moisture condition to ±2 to 3% over optimum moisture content and compacted to a minimum of 95% relative compaction as per afore-mentioned procedure. The soil generated from the excavation should be considered for off-haul from the site.

Soft, unstable, pumping and over-saturated areas may be encountered during the initial grading operation may require additional excavation and stabilization of these areas by means over-excavation and usage of stabilization fabric and dry material. Upon achieving the compaction of the existing bottom, the entire site shall then be graded uniformly (high and low areas) by means of "Cut and Fill" operation to achieve a uniform condition and to minimize differential settlement, prior to placement of any an additional fill material or/ importing any material to the site.

Please note that all areas of proposed improvements/graded areas, including at

grade building pad areas, walkways, patios and trash enclosure areas etc. shall be scarified to a depth of (12)-inches; moisture conditioned and compacted to a minimum of 95% relative compaction as per ASTM D1557.

11.4 GEOTECHNICAL REQUIREMENTS FOR IMPORT MATERIALS

Any and all imported non-expansive fill material required from an off-site source shall comply with the following geotechnical criteria for evaluation and acceptability of the material. This is to include but not limit the materials in the pad areas, under concrete slab-on-construction and for backfill behind walls or/ retained structures shall be primarily granular material with low plasticity and expansion potential and shall comply with the following:

- Resistance R-Value Not less than 25
- Plasticity Index 12 or/ less
- Liquid Limit 30% or/ less
- Expansion Index 20% or/ less
- Passing Sieve #200 Between 10 and 20%
- Maximum rock size ≤ (3)-inches

11.5 LIME/CEMENT TREATMENT

Due to the presence of moderate to high expansion potential in the soil at the site, the subgrade soils and areas with excessive moisture could be lime (HiCal Lime) treated to lower the moisture content and expansion potential. The lime treatment shall penetrate the proposed subgrade/the bottom of the excavation to a minimum depth of (18)-inches, below the exposed ground surface.

Lime/Cement treatment shall be conducted with appropriate equipment, such that a uniform mix (5 to 6% by weight of the soil approximately (120.0)-pounds per cubic feet) is achieved over the entire area. Upon achieving the uniform mix, the lime treated material shall then be compacted to a minimum of 95% relative compaction as per ASTM D1557. After the initial mix and hydration, the treated material shall be allowed to mellow a minimum of (36)-hours, prior to the re-mixing the material.

Upon completion of the mellowing time, the material shall be moisture conditioned as needed, remixed and re-hydrated to the full depth of treatment, prior to achieving the desired degree of 95% relative compaction as per ASTM D1557. The lime treatment contractor shall discuss the treatment procedure with the Soil Engineer and submit the process for review and recommendations for the type of equipment usage.

11.6 WEATHER/MOISTURE CONSIDERATIONS

All imported soil/fill material shall be approved by the Geotechnical Engineer, prior to hauling the material to the site. Based on our experience in the area, grading during the rainy season may be difficult due to the type of soil at the site. If earthwork operations and construction for this project are scheduled to be

performed during the rainy season or in areas containing saturated soils, provisions may be required for drying of soil or providing admixtures to the soil prior to compaction. If desired, we can provide supplemental recommendations for wet weather earthwork and alternatives for drying the soil prior to compaction. Conversely, additional moisture may be required during dry months. Water trucks should be made available in sufficient numbers to provided adequate water during earthwork operations. If site grading is performed during the rainy months, the site soils could become very wet and difficult to compact without undergoing significant drying. This may not be feasible without delaying the construction schedule. For this reason, drier import soils could be required or lime treating may be needed if construction takes place during winter months.

11.7 TEMPORARY SLOPES & TRENCH EXCAVATIONS

The contractor should be responsible for all temporary excavations, slopes and trenches excavated at the site and the design of any required shoring system. Shoring, bracing and benching shall be performed by the contractor in accordance with the strict governing safety standards. Temporary shoring is usually considered as a construction issue (means and methods) and has to be addressed by the contractor.

Shoring and bracing should be provided in accordance with all applicable local, state and federal safety regulations, including but not limited to the current OSHA excavation and trench safety standards. Surface water inflows into excavations must be prevented from causing caving and running ground conditions. Field conditions must be carefully assessed before excavations are made so that appropriate measures can be taken to prevent sloughing, caving and excessive ground movement during the construction phase.

11.8 ON-SITE UNDERGROUND UTILITY TRENCHING

Bedding and embedment materials around the underground utility lines should be well-graded sand or gravel and should be placed and compacted in accordance with the project specifications, local requirements and governing jurisdictions.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered with a minimum of six-inches with sand or fine gravel above the pipe, which should then be mechanically tamped or/ in accordance with the requirements of City of San Jose. Where trenches extend below the groundwater level, it will be necessary to temporarily dewater them to allow for placement of the pipe and/or conduits and backfill.

All underground utility trenches on-site must be compacted to a minimum of 95% relative compaction per requirements of the local governing agency or/ as recommended by the Soils Engineer and in accordance with the test procedure ASTM D1557. Utility trenches located adjacent to the existing or proposed structural foundation shall be no closer than the required 2:1 slope criteria. This

means that no trenches should be located within an area, which would intercept the hypothetical slope line drawn from the bottom edge of the footing at a 2:1 (horizontal to vertical) slope. The trenches could be backfilled with base rock, quarry fines, cement slurry or/ with concrete densified fill all the way up to the required subgrade elevation. The material shall be moisture conditioned (±2 to 3% over optimum) and shall be placed in (8) inch un-compacted lifts and each lift shall be compacted to a minimum of 95% relative compaction as per ASTM D1557. Locally, flexible utility connections may be needed to accommodate up to several inches of vertical ground settlement.

Utility Trenches that are crossing the building foundation shall be backfilled with concrete/cement slurry, a minimum of four (4.0)-feet on either side of the footing. Trenches located in the landscape areas should be compacted to a minimum of 90% relative compaction with the exception of the top-foot, compacted to a minimum of 85% relative compaction as per ASTM D1557. Please note that jetting of the trenches will not be permitted (without any exception) at any time during the backfill operation.

12.0 CONCRETE SLAB-ON-GRADE FLOORS

12.1 STRUCTURAL/INTERIOR CONCRETE SLAB-ON-GRADE CONSTRUCTION

All interior structural concrete slabs on grade shall be a minimum of six (6.0)-inches thick, reinforced with a minimum of #4 rebar spaced (16)-inches on center both ways for shrinkage control to minimize the impact of expansion and shall be designed as per the latest edition of ACI and supported as follows:

- Vapor membrane for capillary-break, Perminator (min. 15 mil-Visquine)
- Four (4) inches of ¾-inch clean crushed rock compacted properly to achieve locking action. (No recycled rock shall be used on the building pads). Areas where movement of traffic is anticipated, the slab should be supported on minimum (6.0)-inches of clean crushed rock compacted properly to achieve locking action or/ Class II Aggregate Base Compacted to 95% relative compaction (No recycled material shall be used on the building pad).
- (18)-inches of non-expansive import material or/ a minimum of (18)-inches of lime treated material as per the requirements reflected in Section (11.4) of the grading recommendations. (Please note that (4.0)-inches of rock and (2.0)-inches of sand shall not be considered as a part of the non-expansive material).

Furthermore, we would recommend casting the floor slabs as slabs-on-grade after the full building weight has been applied to the foundations, to reduce the potential for slab cracking. The slabs should be poured as late as possible in the construction process.

The slab reinforcing mentioned above could exceed the minimum requirement depending on the anticipated usage and loading conditions. The Project Structural Engineer shall determine the final thickness and reinforcing). Post-construction

cracking of concrete slabs is inherent on any project especially where soil has high expansion potential. To minimize the cracks in the slabs, we recommended that proper spacing of expansion and contraction joints.

Where the risk of moisture penetration through interior floor slabs is to be reduced, the slab should be constructed on a layer of capillary break material covered by a continuous impermeable membrane vapor barrier. The capillary break material should be at least (4)-inches thick and should consist of free-draining crushed rock or gravel graded such that 100 percent will pass the 1-inch sieve and none will pass the No. 4 sieve.

A capillary moisture break should consist of at least four inches of clean, free-draining gravel or crushed rock. It would be appropriate for the vapor retarder to meet the requirements for Class C vapor retarders stated in ASTM E1745 and for the vapor retarder to be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by (12)-inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder may be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction.

It should be emphasized that we are not floor moisture proofing experts. While the current industry standard is to place a vapor barrier over a gravel layer as described above, this system may not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor covering manufacturing standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building.

All elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction may have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excess moisture in a building and affect indoor air quality. Recommendations for slab-on-grade or slab-on-grade construction presented above are based on ACI 302.2R-06, “Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials,” published in 2006.

The architect and design engineer should review that reference for background on moisture vapor penetration through concrete slabs and issues regarding protection from delamination of flooring, blistering, staining, mold growth, and other problems related to performance of moisture-sensitive flooring. Since 1999, water-based flooring adhesives have replaced solvent-based adhesives because of restrictions by the EPA, which has lead to an increase in moisture-related problems. The performance of flooring is complicated as described in ACI 302.2R-06 and depends on many factors including sub-slab relative humidity, concrete materials and water-cement ratio, internal relative humidity, and construction aspects, such as curing, length of drying, environmental conditions, pH, etc. As noted above, the architect and design engineer should review pertinent

background materials and decide what measures are needed depending on the type of flooring that will be used.

Please note that the recommendations presented above are not intended to resolve every issue regarding moisture vapor penetration through on-grade concrete slabs. If additional concerns need to be addressed, then additional information needs to be provided and reviewed by the geotechnical engineer and probably by an expert in vapor moisture transmission through concrete slabs.

12.2 HARDSCAPE/FLATWORK AREAS

Due to the anticipated settlement surrounding structures and pile-supported structure, hardscape areas surrounding the building (including vertical curbs, curb and gutter areas) and flatwork connecting to pile-supported building entrance areas, should be designed as a hinged slab to prevent separation at the joint with the building. Hardscape Areas and Flatwork should be reinforced to allow for the appropriate span in the event of settlement. Where the flatwork forms a corner with the building, dowels should only be placed on the side where separation between the foundation and the flatwork is not desirable to provide for flexible movements in the remaining portion of the flatwork. Maintenance or replacement of the hardscape areas and entry slabs should be expected as the ground settles at the perimeter of pile-supported structure.

All hardscape areas exterior concrete slabs-on-grade/flatwork should be a minimum of (5.0)-inch thick reinforced with a minimum of #3 rebar spaced (12)-inches on center both ways, supported on (4)-inches of gravel, (12)-inches of non-expansive import material over properly prepared subgrade. The subgrade soils and non-expansive import material shall be scarified to a minimum depth of (12)-inches below the surface and compacted to a minimum of 95% relative compaction as per ASTM D1557. Areas where vehicle loading or movement traffic is anticipated, we recommend a minimum of (6)-inches of Caltrans Class 2 Base rock/Aggregate Base under the flatwork compacted to a minimum of 95% relative compaction as per ASTM D1557.

13.0 FOUNDATION

Provided the subgrade is prepared as described above, the proposed six-story hotel structure shall be supported on a deep foundation system consisting of auger cast displacement piles. The piles will primarily gain capacity from friction in clayey soil and in end bearing where the medium dense to very dense sands are encountered below the subsurface depending upon the location. Recommendations for this type of foundation system have been outlined and discussed in the following sections of this report.

13.1 DEEP FOUNDATION

The proposed building shall be supported on deep foundation system. We

recommend pile foundations consist of auger-cast displacement piles (ACDP). Auger cast piles should derive their axial capacity through friction in the stiff clays and medium dense to dense sands and gravels beneath the uppermost fill/marine deposits at the site. ACDP piles are typically designed and installed by design-build contractors. As such, the final design capacity of the piles should be provided by the contractor.

Based on our experience with similar sites, we estimate a minimum of (60)-foot long ACDP piles could have an allowable compression capacity on the order of 150 kips for dead plus live loads. The actual design capacities, developed by the design-build contractor, should be verified by a load test program. The preliminary allowable compressive capacities are based on analyses by the contractors with experience installing these pile types in the Bay Area and who have reviewed the site soil conditions. Final design axial pile capacities should be determined by the selected specialty/design building contractor and verified by a test program as discussed below.

TABLE 9 – PRELIMINARY AXIAL PILE CAPACITIES

Pile Type	Approximate Length (feet)	Allowable Compressive Axial Capacity Dead Plus Live Loads (kips)	Allowable Uplift Capacity (kips)
(16)-inch ACDP	60	155	91
(18)-inch ACDP	60	175	100

Piles should be spaced no closer than three pile-diameters center-to-center, to prevent vertical capacity reductions due to pile interaction effects; the outer auger-tip diameter should be used when determining the pile spacing.

13.2 LATERAL LOAD RESISTANCE

The piles should develop lateral resistance from the passive pressure acting on the upper portion of the piles and their structural rigidity. The allowable lateral capacity of the piles depends on:

- Strength of the Surrounding Soils
- Stiffness of the Pile
- Axial Load on the Pile
- Allowable Moment Capacity of the Pile.
- Allowable Deflection at the Pile top and the ground surface

At some locations the upper soil strata are liquefiable; the lateral capacity of the piles has considered the softening of potentially liquefiable layers. Based on our preliminary evaluation, the lateral capacity of the two pile types for lateral

deflection at the pile head or/ the maximum allowable moment for the ACDP. For a free-head condition, the pile top is free to move laterally and rotate. For a fixed-head condition, the pile top is restrained from rotating but free to move laterally. The results of our analyses are presented in the table below.

TABLE 10 – PRELIMINARY AXIAL PILE CAPACITIES

Pile Type	Connection Type	Axial Load (kips)	Lateral Load (kips)	Maximum Moment (kip-in)	Depth to Maximum Moment (feet)	Pile Head Deflection (inches)
16-inch ACDP	Free	155	15	800.8	6.6	0.64
	Fixed	155	15	699.9	0.0	0.15
	Free	155	20	1189.8	7.2	1.12
	Fixed	155	20	1009.7	0.0	0.24
18-inch ACDP	Free	175	15	793.4	7.2	0.43
	Fixed	175	15	720.4	0.0	0.11
	Free	175	20	1169.6	7.2	0.72
	Fixed	175	20	1039.1	0.0	0.17

Once the final pile type and loads has been determined, the appropriate deflection and moment profiles for a single pile should be developed. The lateral capacities in the table above are for single piles only. To account for group effects, the lateral load capacity of a single pile should be multiplied by the appropriate reduction factors as reflected in the table below:

TABLE 11 – LATERAL GROUP REDUCTION FACTORS

Number of Piles Within the Pile Cap	Lateral Group Reduction Factor
2	0.9
3 to 5	0.8
≥6	0.7

However, the maximum moment for a single pile with an unfactored load should be used to check the design of individual piles in a group. The reduction factors are based on a minimum center-to-center spacing of three pile widths. Where piles are spaced at least six pile diameters in all directions, no group reduction factors need to be applied. Reduction for other pile group spacing can be provided once the number and arrangement of piles are finalized.

Lateral load also resistance can be mobilized by the individual piles in combination with other foundation elements embedded below the ground surface. Lateral resistance of piles will depend on the stiffness of the pile, the strength of the surrounding soil, the allowable deflection of the pile top, and the bending moment capacity of the pile. If ACDP piles are used, we can provide lateral capacities once the pile type, length, diameter, and pile cap thickness have been determined.

Additional lateral resistance can be provided by a combination of passive pressure on the vertical faces of the pile caps. To compute lateral resistance, we recommend using an allowable uniform pressure of 1,100 psf (rectangular distribution) in the native soil and an allowable equivalent fluid weight (triangular distribution) of 280 pounds per cubic foot (pcf) in select or lime treated fill. The upper foot of soil should be ignored unless confined by concrete slabs. These values include a factor of safety of 1.5.

If there is insufficient lateral capacity from the piles, pile caps, and grade beams, the lateral capacity of the system can be increased by adding piles, deepening grade beams.

13.3 INDICATOR PILE PROGRAM

If auger cast displacement piles (ACDP) are used, we recommend an indicator pile program be performed to provide data for production pile installation. This data will help with the following information:

1) Provide torsional and down-pressure resistance data to correlate with information obtained from the field investigation;
2) Evaluate pre-drilling requirements; and
3) Estimate production pile lengths.

The foundation contractor should evaluate the potential for variations throughout the site and given their ability to accommodate these variations, determine the appropriate number of indicator piles to install to evaluate these variations; we recommend a minimum of four indicator piles be installed at each of the building location and within the footprint of the building.

Indicator piles may be installed at column locations, selected by the geotechnical engineer and approved by the structural engineer, and can be used for support of the future structures. They should also be installed with the same equipment and procedures that will be used to install the production piles. We expect the indicator piles can be used for support of the proposed structure if installed in the proper location and are not damaged during installation or testing. If indicator piles will not be used as production piles, then the indicator piles should be located at least seven pile diameters (center-to-center) from the closest production pile locations. In addition, we recommend load tests of ACDP piles be performed to confirm the axial compression and tensile pile capacities. We recommend a minimum of two compression and one uplift load tests be performed for each

proposed production pile installation methodology (i.e. rig type, pre-drilling depth and diameter, pile length, etc.) The test pile locations should be selected by the geotechnical engineer and approved by the structural engineer. Compression load tests should be performed in accordance with ASTM D1143-07, *Standard Test Method for Piles Under Static Axial Compressive Load*, and the tension tests should be performed in accordance with ASTM D3689-07. Equipment used for the test (load frame, jacks, and reaction piles) should be capable of applying at least 2.0 times the allowable dead plus live design load and at least 1.5 times the total load. The Davisson Method or other accepted criteria per the 2016 California Building Code should be used to interpret the ultimate capacities of the piles.

13.4 PILE INSTALLATION WORK PLAN

The contractor should submit a work plan describing the proposed pile installation equipment and methodology, including, but not limited to, pre-drilling depth, diameter of auger used for pre-drilling, pile diameter and pile length, as well as the proposed pile load test set-up and procedure should be submitted to the Geotechnical Engineer for review and approval at least five working days prior to the indicator pile and pile load test programs.

The work plan should include a site plan showing the locations of indicator test and reaction piles relative to permanent foundation elements and a drawing showing the layout of the load test set up. Following the completion of pile load tests, the Geotechnical Engineer will require at least seven working days to review and evaluate the load test results and propose recommendations for production pile installation. Additional pile load tests will be required if, during production pile installation, the equipment or/ installation procedure deviates from the approved work plan and indicator pile load test program.

13.5 FOUNDATION SETTLEMENT

Settlements under static loads are expected to be primarily elastic with majority of the settlement occurring immediately upon application of the structural loads. Post construction total settlements have been anticipated to be in the range of approximately (±¾)-inch to one-inch with a differential settlement of (±½)-inch between adjacent columns over a horizontal span of (30)-feet. In addition, we estimate that differential seismic movement due to liquefaction will be on the order of (1¾)-inches, resulting in a total estimated differential footing movement of less than (±1¾)-inches between foundation elements, assumed to be on the order of (40)-feet.

13.6 LATERAL LOAD RESISTANCE

Lateral load resistance shall be provided by the friction between the foundation and the supporting subgrade. The lateral load resistance could also be provided by the passive pressures acting against the sides of the foundation, provided they are constructed neatly against the undisturbed native soil. It is recommended that an allowable equivalent fluid pressure of 280 pounds per cubic foot be used for design

purposes. This passive pressure has been assumed to act at a depth of one-foot below the proposed subgrade elevation. The allowable passive pressure may be increased by one-third for resistance to lateral loading due to wind or seismic forces. A coefficient of 0.3 may be utilized for the above, provided the combination of both friction and passive pressure may be used provided that one of them is reduced by 50 percent.

13.7 Swimming Pool Recommendations

13.7.1 Pool Foundation

As mentioned above, due to the variability/nature of the fill material a significant variation in material properties may occur in the pool excavation. It is our experience that there is a potential for significant differential swell and settlement characteristics at the site. Such situations can be detrimental to structure performance. Therefore, in order to minimize the uplift / shifting of the swimming pools during a seismic event, we recommend that the pool shell be designed and structurally supported on the pile foundation. Please refer to the AGPD design recommendations reflected in the sections above. Settlements should be within tolerable limits with total post-construction and differential settlement to be less than ½-inch to ¾-inch.

13.7.2 Swimming Pool Shell

The pool reinforcing should be designed by a structural engineer. The final pool plans should be submitted to the Geotechnical Engineer for review when they become available. The following items may be used in the pool design.

- Active equivalent fluid pressure of 65 pcf and an at-rest equivalent fluid pressure of 90 pcf of the equivalent fluid density as the retaining walls are restrained from rotation. These values are for level conditions and do not include additional loads due to hydrostatic conditions or/ additional surcharge loads.
- The pool bottom should be underlain by an eight-inch gravel blanket of clean washed ¾-inch drain rock and be equipped with hydrostatic pressure relief valves to prevent excess hydrostatic pressure from damaging the pool in the event it is drained.
- Passive pressures may be assumed as 280 pcf provided that the area in front of the swimming pool shell is level for a distance of at least 10 feet. The upper 12 inches should be neglected in passive pressure design.
- A Coefficient of Friction of 0.3
- An approximate Soil Unit Weight of 125 pcf.

Furthermore, we recommend that efforts be made to reduce the probability of leakage from the pool and to detect the leakage should it occur. It is important to provide a gunite shell with a high degree of certainty against leakage. Routine pool maintenance should be scheduled to repair any leakage that may develop. Excavations that will be deeper than five feet and will be entered by workers should be shored or/ sloped in

accordance with the OSHA standards (29 CFR Part 1926). Because the soil through which the pools will extend is highly expansive, we recommend that the sides and bottom of the pool excavation be kept wet following excavation and their moist condition maintained until concrete is placed. We should check the condition of the pool excavations just prior to concrete placement to confirm that the excavations are sufficiently moist.

The pool excavation should be observed by AST's field representative prior to steel installation. The soils exposed in the pool excavation should be kept moist at all times prior to placement of concrete.

13.7.3 Pool Deck

Pool deck slabs should have a minimum of 5.0-inches of concrete reinforced with No. 4 bars spaced at (12)-inches on center each way supported over with (12)-inches of Class II aggregate base and compacted subgrade. The concrete deck should have a minimum slope of 2 percent away from the pool edge in all directions. A liberal number of control joints should be incorporated into the pool deck. All joints should be filled with elastomeric sealant and should be examined regularly to ensure that no openings have developed.

The subgrade material under the pool deck should be reworked to a depth of at least (24)-inches and compacted as per the grading recommendations reflected in this report. The subgrade should be graded to slope away from the pool at a minimum slope of 2 percent. The compacted subgrade should be overlain by a tough vapor proof membrane at least 20 mil thick. The 12-inch-thick aggregate base and a perimeter drain system should then be installed at the base of the rock section on the outside face of the deck. The aggregate base should be compacted to a minimum of 95 percent relative compaction with a moisture content of at least optimum. Subgrade soil material should not be allowed to become dry prior to completion of landscaping. Minor cracking on concrete slabs should be expected in the future.

13.7.4 Pool Sub-drainage

It is imperative that excess water from either irrigation or leakage from the pools be prevented from saturating the soils below. Excess water could have a detrimental effect on the stability of the foundation soils for nearby improvements and the pool shells. We recommend that efforts be made to reduce the probability of over-irrigation and leakage and to detect the leakage should it occur. Routine maintenance should be scheduled to repair any leakage that may develop, since leakage of water into the soils may cause foundation instability associated with shrinking and swelling of saturated soils.

A subsurface drainage system should be installed at the base of the pool excavation to collect and rapidly remove any water that may pond at the base of the pool excavations. Subsurface drainage should consist of a minimum of (6)-inch-thick layer of either clean crushed rock, underlain by filter fabric (Mirafi 140N minimum), or Class 2 permeable material placed at the base of the pool to intercept and drain any

subsurface water or leaks. The base of the pool excavation should be sloped at least 2 percent towards the centerline of the pool, and a shallow trench for a subdrain should be constructed along the centerline of the pool for the length of the pools. AST should observe the materials and prepared surfaces prior to installation.

A 4-inch-diameter perforated sub-drain pipe (SDR 35 or approved equivalent) encapsulated in either crushed rock wrapped in filter fabric (6-ounce minimum) or/ Caltrans Class 2 permeable material should be placed at the base of the subdrain trench with a minimum slope of 1 percent and directed to an appropriate discharge point that can be periodically observed for leakage. Solid discharge pipe should be used once the subdrain pipe leaves the pool excavations. If grades permit, discharging the pool subdrain into a solid collection system that would carry water to a storm drain catch basin or manhole is the preferred option. If this is not possible, then a sump pit could be installed and frequently monitored and pumped.

14.0 SOIL RETAINING STRUCTURES

14.1 RETAINING WALLS

Retaining structures that are free to rotate or translate laterally (e.g. cantilevered retaining walls) through a horizontal distance to wall height ratio of no less than 0.004 are referred to as unrestrained or yielding retaining structures. Such walls can generally move enough to develop active earth pressure conditions. Retaining structures that are unable to rotate or deflect laterally (e.g. restrained basement walls) are referred to as restrained or non-yielding walls and subject to at-rest earth pressure conditions. Backfill materials behind the wall and within a 1h: 1v projection up from the foundation should consist of granular soils meeting the requirements of Section (11.4).

14.2 STATIC EARTH PRESSURES

Cantilevered walls with granular soil backfill can be designed for active earth pressures using an equivalent fluid weight of 45 pcf for horizontal backfill and drained conditions (no hydrostatic loading).

Restrained walls with granular soil backfill should be designed for at-rest earth pressures estimated using an equivalent fluid weight of 55 pcf assuming drained and horizontal backfill conditions plus an additional uniform lateral pressure of 8H psf, where H is the height of the backfill above the top of the wall footing in feet. Wherever, walls are subjected to surcharge loads, they should be designed for an additional uniform lateral pressure equal to ½ or 1/3 the anticipated surcharge loads for restrained or unrestrained walls, respectively.

Retaining walls with sloping backfill should be designed for an additional uniform lateral pressure of 1 pcf for 3 degrees of slope inclination. The lateral earth pressure distributions should be applied along a vertical line through the heel of the wall between the intersection of the vertical line with the ground surface above

the wall and a point defined by the elevation of the lowest structural member of the wall. Surcharge loads induce additional pressures on earth retaining structures. Uniform area surcharge pressures for retaining walls may be assumed equal to 0.5 of the applied surcharge pressures. Lateral pressures for other surcharge loading conditions can be provided, if required.

14.3 DYNAMIC LATERAL EARTH PRESSURE

The increase in lateral earth pressure on walls from earthquake loading can be estimated using the Mononobe-Okabe theory, as described by Seed and Whitman (1970). The theory is based on assumption that sufficient wall movement occurs during seismic shaking to allow active earth pressure conditions to develop. The increase in lateral earth pressure resulting from earthquake loading can also be estimated using the Mononobe-Okabe theory. Because that theory is based on assumption that sufficient movement occurs such that active earth pressure conditions develop during seismic shaking, the applicability of the theory to restrained or basement walls is not direct. However, Nadim and Whitman (1992) suggest the theory that can be used for such walls.

In the Mononobe-Okabe approach, the total dynamic pressure can be divided into static and dynamic components. For the proposed project, the estimated dynamic lateral force increase (based on seismic loading conditions) for either unrestrained or restrained walls with level backfill surfaces may be taken as 45 x PHGA x H^2 in pounds per linear foot of wall. In the above formula, peak horizontal ground acceleration (PHGA) equals the horizontal ground acceleration and H is the height of wall in feet. The centroid of that dynamic lateral force increase should be applied at a distance of 0.6H above the base of the wall, where H is equal to the below-grade portion of the wall height in feet. In this procedure, the seismic increment is a function of the anticipated peak ground acceleration at the project site.

Table Below presents the at-rest and seismic pressures (active plus seismic pressure increment) for two code levels of shaking, the DE and the MCE event. All of these values assume the soil upslope of the permanent walls is relatively flat. All parameters are presented as equivalent fluid weights (triangular distribution).

TABLE 12 – RETAINING WALL EARTH PRESSURES (DRAINED CONDITION)

Backfill	Static Condition	Seismic Condition DE (PGA 0.4g)	Seismic Condition MCE (PGA 0.6g)
Backfill	(At Rest Pressure) (EFP, pcf)	Active plus Seismic Pressure increment	Active plus Seismic Pressure increment
Level Backfill	55 pcf+8H psf	45 pcf+18H^2 plf	45 pcf+27H^2 plf

14.4 DRAINAGE PROVISIONS

Drainage measures should be provided behind the walls to help collect groundwater seepage and prevent the buildup of hydrostatic pressures. Drainage measures can consist of constructing a vertical drainage system behind the wall by placing free-draining backfill (meeting the requirements of Section 11.4) directly behind the wall. The free-draining material should be at least (1.0)-foot wide and a perforated pipe should be placed at the base of the material to collect and convey water to an outlet location.

Depending on the type of free-draining material that is used, filter fabric may be required to separate the drainage material from the adjacent soils or backfill. The backside of retaining walls should be waterproofed to prevent potential effervescence (salt buildup) from forming on the front side of the wall.

In lieu of using a (1.0)-foot-wide zone of free-draining backfill material to provide drainage behind the wall, geo-composite drains (for example, Miradrain, manufactured by Mirafi, Inc., or similar) can be used to control groundwater and prevent hydrostatic pressures on the walls. If drainage panel products are used, they should be appropriate for the proposed use and installed in accordance with the manufacturer's recommendations. In either case, a perforated pipe should be placed at the base of the material to collect and convey water to an outlet.

14.5 COMPACTION ADJACENT TO WALLS

Backfill placed within (5.0)-feet of the retaining structures (measured horizontally behind the wall) should be compacted with lightweight, hand-operated compaction equipment to reduce the potential for developing compaction-induced stresses. If large or heavy compaction equipment is used, lateral earth pressures could exceed those presented previously. If larger or heavier compaction equipment is to be used, further evaluation of the potential for compaction-induced stresses in the walls is recommended. Backfill material should be brought up uniformly behind below-grade walls (i.e., the backfill should be at about the same elevation all around the wall as the backfill is placed). That is, the elevation difference of the backfill surface around the wall should not be greater than about (2.0)-feet, unless the wall is designed for the potential for differential backfill heights. The backfill material shall be placed and compacted to a minimum of 90% relative compaction as per ASTM D1557. However, if the backfills are deeper than (5.0)-feet then they should be compacted to 95% relative compaction as per ASTM D1557.

14.6 FOUNDATION FOR ON-SITE RETAINING WALLS

Retaining walls could be supported foundation system, as designed in accordance with the recommendations outlined in the "Foundation Section" of this report.

15.0 PAVEMENT DESIGN

15.1 SUBGRADE PREPARATION FOR PARKING LOT AREAS

The proposed areas of improvements including parking lot, vertical curbs and curb and gutter areas shall be graded uniformly, scarified to a depth of (12)-inches (ripped and cross ripped); moisture conditioned, mixed thoroughly to achieve a uniform mix, prior to being compacted. Upon achieving a uniform mix, the soil shall then be compacted to a minimum of 95% relative compaction according to ASTM D1557 test procedure, prior to placement of any additional fill material.

Additional fill material, if required shall be placed in lifts and each lift shall then be compacted to a minimum of 95% relative compaction all the way up to the required/proposed subgrade elevation. The subgrade preparation for the proposed pavement areas shall extend a minimum of two (2)-feet beyond the curb line and shall also be compacted to not less than 95% relative compaction, using the aforementioned procedure. The material shall be moisture conditioned slightly over the optimum moisture content and shall be spread in lifts not exceeding (8) inches (un-compacted thickness) and compacted to not less than 95% relative compaction using the ASTM D1557 test procedure.

Upon achieving the desired subgrade elevation and relative compaction, the required base rock section shall be placed in lifts and each lift shall be compacted to 95% relative compaction as per ASTM D1557.

15.2 PAVEMENT CUT-OFF/SEEPAGE CONTROL

Concrete slabs around the landscaping areas should be protected from water seepage. The water seepage from these areas usually creates over-saturation of the base rock and the subgrade, thereby causing unstable conditions. Henceforth, we recommend the following:

- Provide vertical cut-off or a deep vertical curb section all along the proposed pavement section and the landscape areas. The vertical cut-off should extend through the base rock and a minimum of four inches into the subgrade. The vertical cut-off will limit the moisture intrusion / water seepage around the foundation, into the pavement section and thereby extending the life of the pavement.
- Utility trenches (irrigation lines, electrical conduits, plumbing, etc.) shall not be placed close to the foundation especially, parallel to the building. This means that no trenches should be located within an area, which would intercept the hypothetical slope line drawn from the bottom edge of the footing at a 2:1 (horizontal to vertical) slope. If the trenches are excavated close to the foundation, then 2:1(horizontal to vertical) slope criteria shall be achieved at all times. If the above-mentioned criteria are not honored or utilized, then the trenches become a pathway for water intrusion into the footing and slab areas, resulting in soil distress and settlement problems.

- All the utility trenches in the concrete slabs shall be capped with at least one foot of native material or concrete or cement slurry. We recommend that the utility lines located close to the foundations and along the side of the buildings be inspected to make sure they are installed correctly and compacted properly.
- In landscape areas, to minimize moisture changes in the natural soils and fills, we recommend the usage of drought resistant plants with a drip irrigation watering system.

15.3 PERMEABLE PAVEMENT

Permeable pavements may be considered as a potential component of the site's surface water management system. Our evaluation of potential site constraints such as subsurface infiltration rate at the site suggests low permeability of the subsurface soils. According to Jackson (2003), the best locations for permeable pavement are parking lots and low-volume roads. Areas with high frequency and/or heavy truck loading should not be considered.

Cahill et al. (2004) recommend permeable pavements be designed to a ratio of 5:1 impervious area to infiltration area and be laid on flat slopes with inclinations of 6 percent or less. A typical permeable pavement section, from top to bottom, includes a porous asphalt course, a top filter course, a reservoir course, an optional bottom filter course, and filter fabric overlying a level base of native un-compacted soil. The thickness of the reservoir course is typically designed to allow complete drainage within 72 hours; however, capacity should be designed by an engineer proficient in hydrology and storm water design and should comply with local regulations.

Permeable pavements should be maintained to promote unobstructed drainage and prevent the accumulation of fines within the system. Pavement edges are usually lined with unpaved stone or catch basins to provide additional drainage pathways to the reservoir course if the asphalt course is repaved or becomes impermeable.

Additionally, the section bottom is typically designed with positive overflow elements to prevent saturation of the pavement if the native soil subgrade becomes impermeable. Our review of relevant literature suggests that the long-term performance of permeable pavement systems is generally unknown and may require periodic maintenance. Pavers, if being proposed for the site could be designed and supported as follows:

- (2)-inch of No. 8 clean washed gravel and compacted to achieve locking action
- (4)-inches of No. 57 clean washed gravel, compacted to achieve locking action
- (10)-inches of No. 2 clean washed gravel, compacted to achieve locking action
- Four-inch Perforated Pipe in the middle
- Mirafi Fabric RS580i or Equal
- (12)-inches of subgrade compacted to 90% compaction as per ASTM D1557

15.4 RIGID CONCRETE PAVEMENT/PORTLAND-CEMENT CONCRETE PAVEMENT

Rigid Concrete Pavement (Portland Cement Concrete Pavement section) will be required at truck loading dock ramps, stress pads at the trash enclosure areas and where movement of heavy traffic is anticipated. Portland cement concrete pavements are typically better able to resist the intense stresses induced in pavements by the turning motions of vehicles - particularly delivery and garbage trucks. Concrete pavements should be used in areas frequented by such vehicles as well as in driveway and entry aprons. Concrete pavement sections presented in the table below are based on current Portland Cement Association (PCA) design procedures and the assumptions reflected below:

- Modulus of subgrade reaction = 50 psi/in
- Modulus of rupture of concrete = 550 psi
- Aggregate Interlock Joints
- No concrete shoulders
- 20-year design life
- Load Safety Factor = 1.0

Table 14 – RECOMMENDED PORTLAND CEMENT-CONCRETE PAVEMENT

Proposed Usage	Average Daily Truck Traffic	Portland Cement Concrete		Aggregate Base		Subgrade Scarification (inches)
		Inches	Feet	Inches	Feet	
Light Duty	15	7.0	0.58	8.0	0.67	12
Heavy Duty	30	7.5	0.63	8.0	0.67	12

Portland cement concrete pavement sections provided above are contingent on the following recommendations being implemented during construction phase of the project as follows:

- Pavement areas shall be supported on a subgrade, scarified to a depth of (12)-inches; moisture conditioned to 2 to 3% over optimum moisture content and compacted to a minimum of 95% relative compaction as per testing procedure ASTM D1557, prior to placement of the base rock section. The base rock shall be placed and compacted to a minimum of 95% relative compaction.
- Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils are not allowed to become wet.
- Over-finishing of concrete pavements should be avoided. Typically, a broom or burlap drag finish should be used.
- Construction and/or control joint spacing should not exceed (12)-feet.
- Concrete pavement should have a minimum 28-day compressive strength of 4,000 psi. Concrete slumps should be from 3 to 4 inches. The concrete should

be properly cured in accordance with PCA recommended procedures and vehicular traffic should not be allowed for 3 days (automobile traffic) or a minimum of 7 days (truck traffic).

- Thickened edges should be used along outside edges of concrete pavements. Edge thickness should be at least (4)-inches greater than the concrete pavement thickness and taper to the actual concrete pavement thickness (36)-inches inward from the edge. Integral curbs may be used in lieu of thickened edges.
- To help offset plastic shrinkage, concrete pavement may be reinforced with at least No. 4 bars, 16 inches on-center, both ways (located 1/3 of the slab thickness from the top of the slab).

The above pavement recommendations should be incorporated into project plans and specifications by the project architect and/or engineer.

15.5 FLEXIBLE PAVEMENT

The following pavement section design is based on an estimated laboratory resistance "R" value of 5 for the near surface soil samples and for the assumed traffic indices ranging between 4.5, 5.5 and 6.5 for parking areas and automobile drive thru areas have been presented in the following sections of this report.

TABLE 14 - RECOMMENDED ASPHALT PAVEMENT SECTIONS

General Traffic Condition	Traffic Index	Asphalt (inches)	Class II Base Rock R = 78 min. (inches)	Total Pavement Thickness (inches)	Subgrade Native Material (inches)	% Relative Compaction
Automobile Parking	4.5	3.0	8.0	11.0	12	95%
Automobile Driveway Aisles	5.5	3.0	11.0	14.0	12	95%
Truck Traffic & Access Areas	6.5	4.0	14.5	18.5	12	95%

16.0 SITE DRAINAGE

16.1 SURFACE DRAINAGE

Bio-swales if proposed for the site, then they shall be located at a minimum of (10.0)-foot offset from the exterior face of the building foundation / footing to the top of slope of the bio-swale and a minimum of (5.0)-feet from any concrete slabs

on grade or/ any pavement areas. Positive surface drainage should be provided around the building to direct surface water away from the foundations.

To reduce the potential for water ponding adjacent to the building, we recommend the ground surface within a horizontal distance of five feet from the building slope down away from the building with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations. As mentioned above, Infiltration basins or/ bio-swales should not be placed within (10)-feet of the foundations. Because the subgrade soil consists predominantly of clay, it will have a relatively low permeability. If infiltration basins or bio-swales are planned, drains should be provided that direct the water away to an appropriate outlet.

16.2 STORMWATER TREATMENT DESIGN CONSIDERATIONS

If storm water treatment improvements, such as shallow bio-retention swales, basins or/ pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction. General Bioswale Design Guidelines are as follows:

- If possible, avoid placing bioswales or/ basins within (10.0) feet of the building perimeter foundation or/ within (5.0)-feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with a heavy-duty liner to reduce water infiltration into the surrounding expansive clays.

- Bioswales constructed within (5.0)-feet of proposed buildings may be within the foundation or/ from the exterior face of the footings zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the "foundation plane of influence," an imaginary 2:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.

- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

16.3 BIOSWALE CONSTRUCTIONS ADJACENT TO PAVEMENTS

If bioswales or/ bio-detention areas are located adjacent to proposed parking lots or exterior flatwork, we recommend that mitigation measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement,

depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should have an offset from the vertical edge of a bioswale such that there is a greater than or/ a minimum of (2.0)-foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth, or/
- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the "Retaining Walls" section of this report, or/ concrete curbs or edge restraint should be adequately keyed into the native soil or/ engineered fill material to reduce the potential for lateral movement of the curbs.

16.4 IRRIGATION AND LANDSCAPING LIMITATIONS

The use of water-intensive landscaping around the perimeter of the buildings should be avoided to reduce the amount of water introduced to the expansive clay subgrade. In addition, irrigation of landscaping around the buildings should be limited to drip or bubbler-type systems. The purpose of these recommendations is to avoid large differential moisture changes adjacent to the foundations, which has been known to cause large differential settlement over short horizontal distances in expansive soil, resulting in cracking of slabs and architectural damage.

Moderately to highly expansive native clay is expected to be present at or near the subgrade level. For this condition, prior experience and industry literature indicate some species of high water-demand trees can induce ground surface settlement by drawing water from the expansive soil and causing it to shrink. Where these types of trees are planted adjacent to structures, the ground-surface settlement may result in damage to structure. This problem usually occurs ten or more years after project completion as the trees reach mature height.

To reduce the risk of tree-induced, ground-surface settlement, we recommend trees of the following genera shall not be planted within a horizontal distance from the building equal to the mature height of the tree such as Eucalyptus, Populus etc. This limited list does not include all genera that could induce ground-surface settlement. Therefore, the project landscape architect should exercise proper judgment in limiting other types or trees with similar properties in the vicinity or close proximity to the building foundation.

17.0 ADDITIONAL SERVICES

17.1 PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING

All conclusions and recommendations presented in this report are contingent upon

Advance Soil Technology, Inc. (AST) being retained to review the grading plans, prior to construction. The general contractor/grading contractor/sub-contractors shall comply with the recommendations of the soil engineer at all times. Appropriate field adjustments will be made as deemed necessary during the construction phase of the project. If any unforeseen circumstances are encountered during the grading operation, the engineer shall be notified immediately for recommendations to minimize the chance of the grading work not being approved by the engineer.

In addition to the above, we shall observe and perform compaction test as deemed necessary during the grading (earthwork) operation at the site. It is the responsibility of the owner or his representative to schedule the inspections for the purpose of documentation. The site preparation and grading, excavation, cutting and backfilling shall be carried out under the observation of the Geotechnical Engineer. AST will perform appropriate field and laboratory tests to evaluate the suitability of the fill material, proper moisture content for compaction and the degree of compaction as needed per the requirements of this report. Any fill material that does not meet the specification requirements shall be removed or replaced and reworked until the requirements are completely satisfied. Grading, shaping, excavating, conditioning, backfilling and compacting procedures require approval of AST as they are performed in the field.

17.2 CONSTRUCTION ACCEPTANCE

A representative from AST shall be present during the entire grading operation, so that he can provide recommendations as deemed necessary during the construction phase of the project. Unobserved and unapproved grading work will not be accepted under any circumstances. The grading operation shall be performed under the supervision of the soil engineer and in accordance with the requirements of the specifications of this report.

17.3 SEASONAL LIMITS

No fill material shall be placed, spread or rolled during unfavorable weather conditions. If the grading operation is interrupted due to heavy rain, fill operations shall not be resumed until field density / moisture test have been taken and indicate that the moisture content of the fill is as previously specified or approved / directed by the soil engineer.

17.4 UNUSUAL/WET CONDITIONS

In the event that any unusual conditions, not covered by the special provisions, are encountered during grading operations, the soil engineer shall be immediately notified for supplemental recommendations.

18.0 SUMMARY OF COMPACTION RECOMMENDATIONS

Compaction requirements for the earthwork activities during the grading operation

at the site shall be as follows:

Areas	Compaction Recommendation
General Engineered Fill	Compact to a minimum of 95% compaction at a minimum of 2% over the optimum moisture content. Where fills are deeper than (5.0)-feet, the portion below (5)-feet should be compacted to a minimum of 95% relative compaction.
Non-Expansive Fill	NEF or Lime Treated Material shall be compacted to a minimum of 95% compaction at near or/ close to the optimum moisture content.
Utility Trenches	Compact to a minimum of 90% compaction at a minimum of 2% over the optimum moisture content with the exception of upper (3.0)-feet compacted to a minimum of 95% relative compaction.
Exterior Flatwork	Compact to a minimum of 90% compaction at a minimum of 2% over the optimum moisture content. Areas where exterior flatwork is subjected to vehicular traffic; compact upper 12 inches of subgrade to a minimum of 95% relative compaction at a minimum of ±2% over the optimum moisture content. Compact base rock to a minimum of 95% relative compaction at or/ near the optimum moisture content.
Parking and Driveways	Compact upper (12)-inches of subgrade to a minimum of 95% relative compaction at a minimum of ±2% over the optimum moisture content, Compact base-rock to a minimum of 95% relative compaction at or/ near optimum moisture content. Asphalt compact to a minimum of 98% relative compaction.
Landscape Areas	Compact to a minimum of 90% relative compaction with the exception of upper (12)-inches compacted to 85% relative compaction.

General Notes:

- Depths are below finished subgrade elevation.
- All compacted surfaces should be firm, stable, and unyielding under the weight of the compaction equipment and during the movement of construction traffic.

- All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D1557. All lifts to be compacted shall be a maximum of (8)-inch loose thickness, unless otherwise recommended.

19.0 LIMITATIONS

The recommendations made in this report are based on assumption that subsurface soil and groundwater conditions do not deviate from those disclosed at the location of the cone penetrometer test locations and exploratory borings drilled at this site. If any variations or/ undesirable conditions are encountered during construction, the effects of these conditions on the recommendations presented herein should be evaluated again and if necessary, supplemental recommendations developed and provided as deemed necessary. The report and recommendations have been for the exclusive use of our client Ms. Tina Phan of RSTP Investment, LLC for the proposed Addition/development of Clariana Hotel located at 10 South Third Street in San Jose, California as described above in this report.

In the performance of our professional services, AST, its employees, and its agents will comply with the standards of care and skill ordinarily exercised by members of our profession practicing in the same or similar localities. This report may not provide all of the subsurface information that may be needed by a contractor to construct the project. No warranty, either expressed or/ implied, is made or intended in connection with the work performed by us, or/ by the proposal for consulting or/ other services, or/ by the furnishing of oral or written reports or findings. We are responsible for the conclusions and recommendations provided in this report, which are based on data related only to the specific project and locations discussed herein.

This report is for the sole use of the Client and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than one year from the date of issuance of this report. The work performed was based on project information provided by Client. If Client does not retain AST to review any plans and specifications, including any revisions or modifications to the plans and specifications, AST assumes or/ has no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field conditions or/ to the plans and specifications, Client must obtain written approval from AST that such changes do not affect our recommendations. Failure to do so will invalid recommendations and the report in its entirety. Do not over-rely on the construction recommendations included in this report. These recommendations are not final, because they were developed principally from AST's professional judgment and opinion.

AST's recommendations can be finalized only by observing actual subsurface conditions revealed during the construction phase of the project. AST cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation. Sufficient monitoring, testing and consultation

by AST should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether the earthwork activities were completed in accordance with the recommendation reflected in the geotechnical report. Retaining AST for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions. In the event conclusions or/ recommendations are made or/ provided by others, based on these data reflected in this report, then such conclusions and recommendations are not our responsibility unless we have been given an opportunity to review and concur with such conclusions and recommendation in writing.

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Important Information About Your

Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

Rely on Your ASFE-Member Geotechnical Engineer For Additional Assistance

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

ASFE
The Best People on Earth

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

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Appendix "A"

Plate 1	Site Location Map
Plate 2	Site Location Map – Aerial View
Plate 3	Topographic Map
Plate 4	Geologic Map
Plate 5	California Fault Map
Plate 6	Peak Ground Acceleration
Plate 7	Historical Groundwater
Plate 8	Overall Site Plan
Plate 9	Site Plan (CPT & Boring Locations)
Plate 10	Geological Cross Section A-A'
Plate 11	Peak Ground Acceleration
Plate 12	Historical Groundwater Elevations
Plate 13	Site Plan with Boring & CPT Locations
Plate 14	Geological Cross Section A-A

Site Location Map

Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **Clariana Hotel Addition** New Concrete Podium Deck with Five Levels Above
343 So. Baywood Avenue | San Jose, California	10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 1

Site Location Map | Aerial View

Advance Soil Technology, Inc.

Geological , Geotechnical, Environmental
Consulting & Construction Services

Geotechnical Study | Investigation

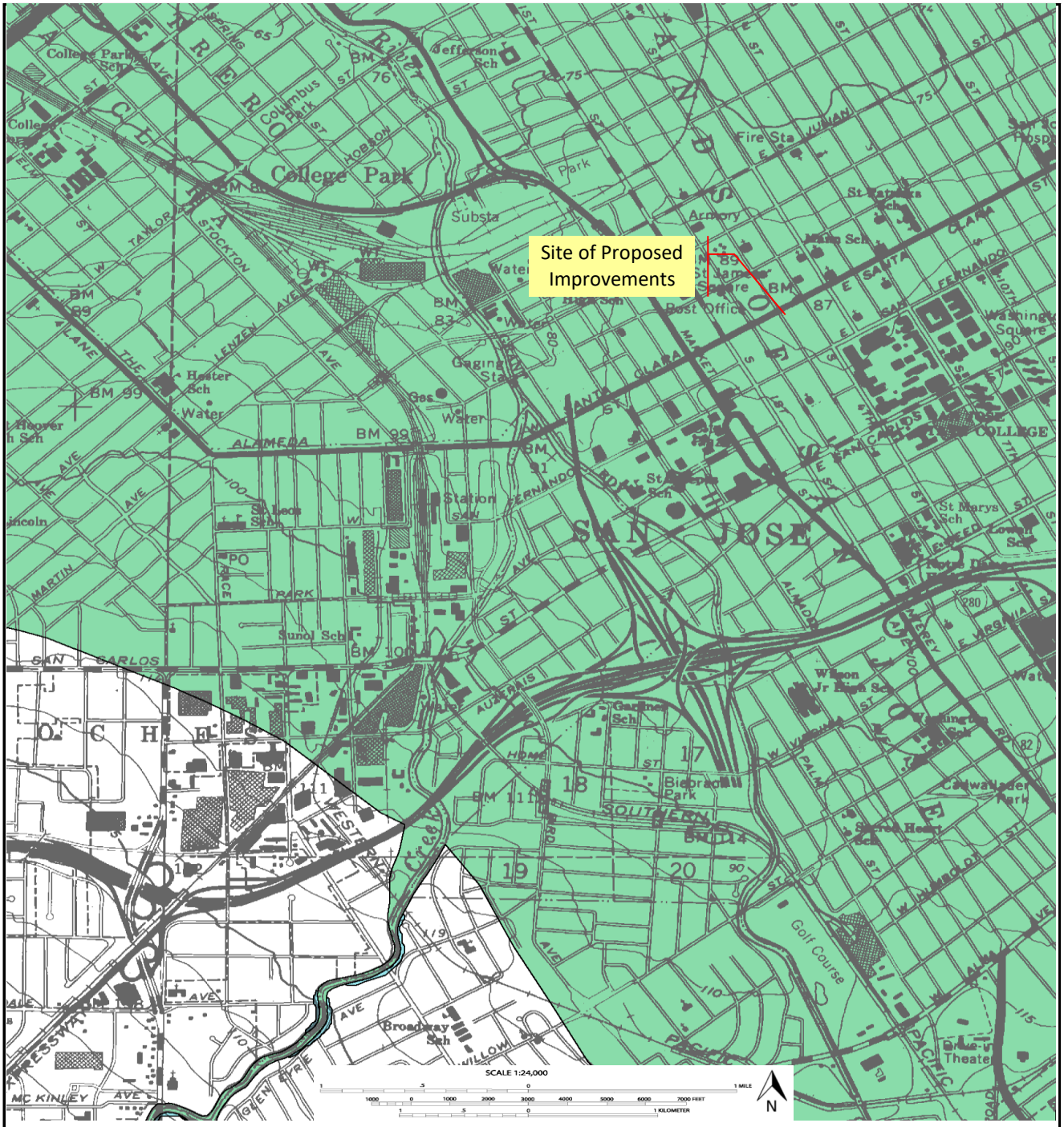
Clariana Hotel Addition

New Concrete Podium Deck with Five-Story Levels Above

343 So. Baywood Avenue I San Jose, California

10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 2



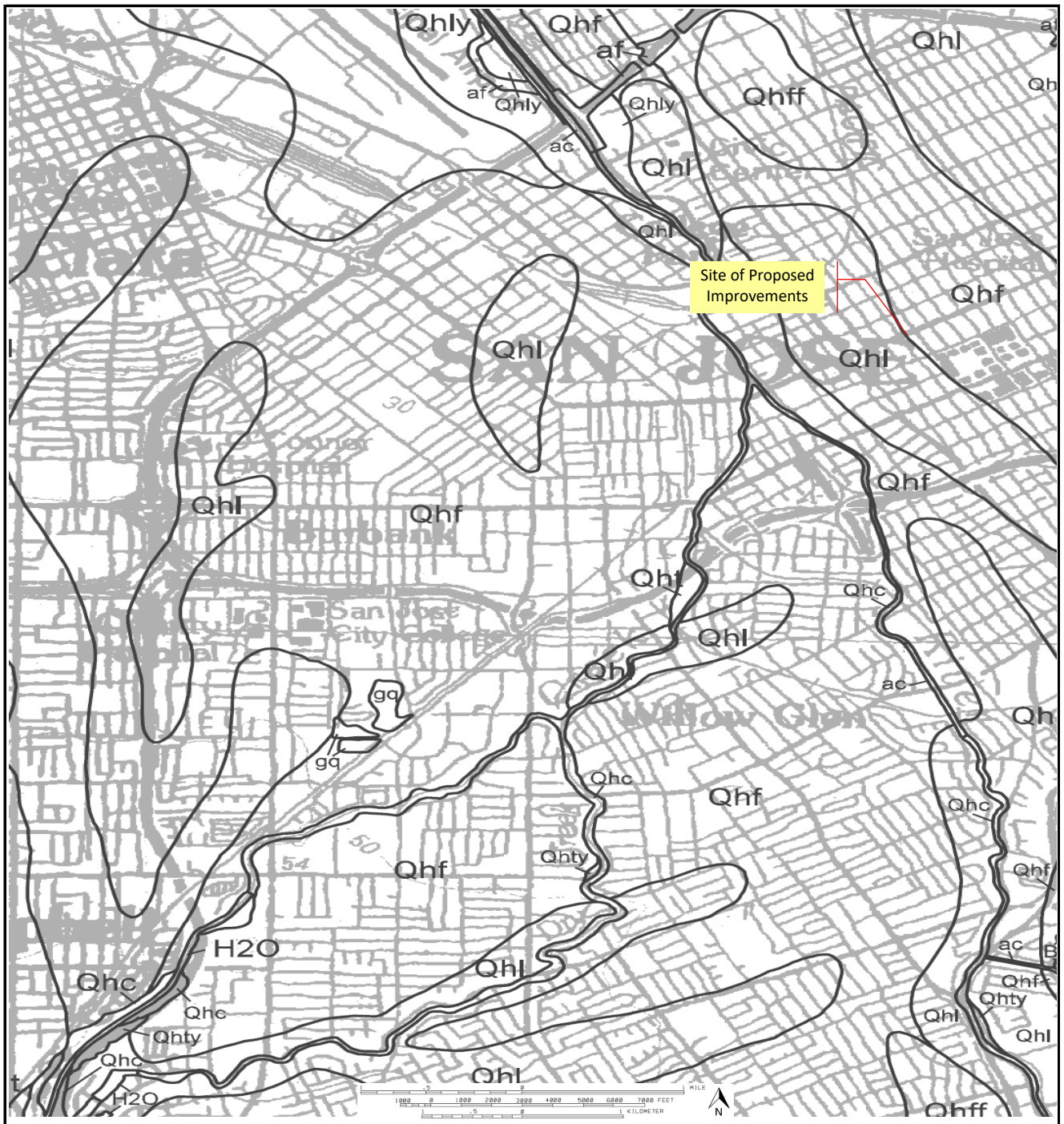
CGS official Map - San Jose West Quadrangle Map - Dated February 2002

Seismic Hazard Zone Report 058 by Kevin B. Clahan, Ellise Mattison, Anne Rosinski

Topographic Map

Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **Clariana Hotel Addition** New Concrete Podium Deck with Five Levels Above
343 So. Baywood Avenue | San Jose, California	10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 3



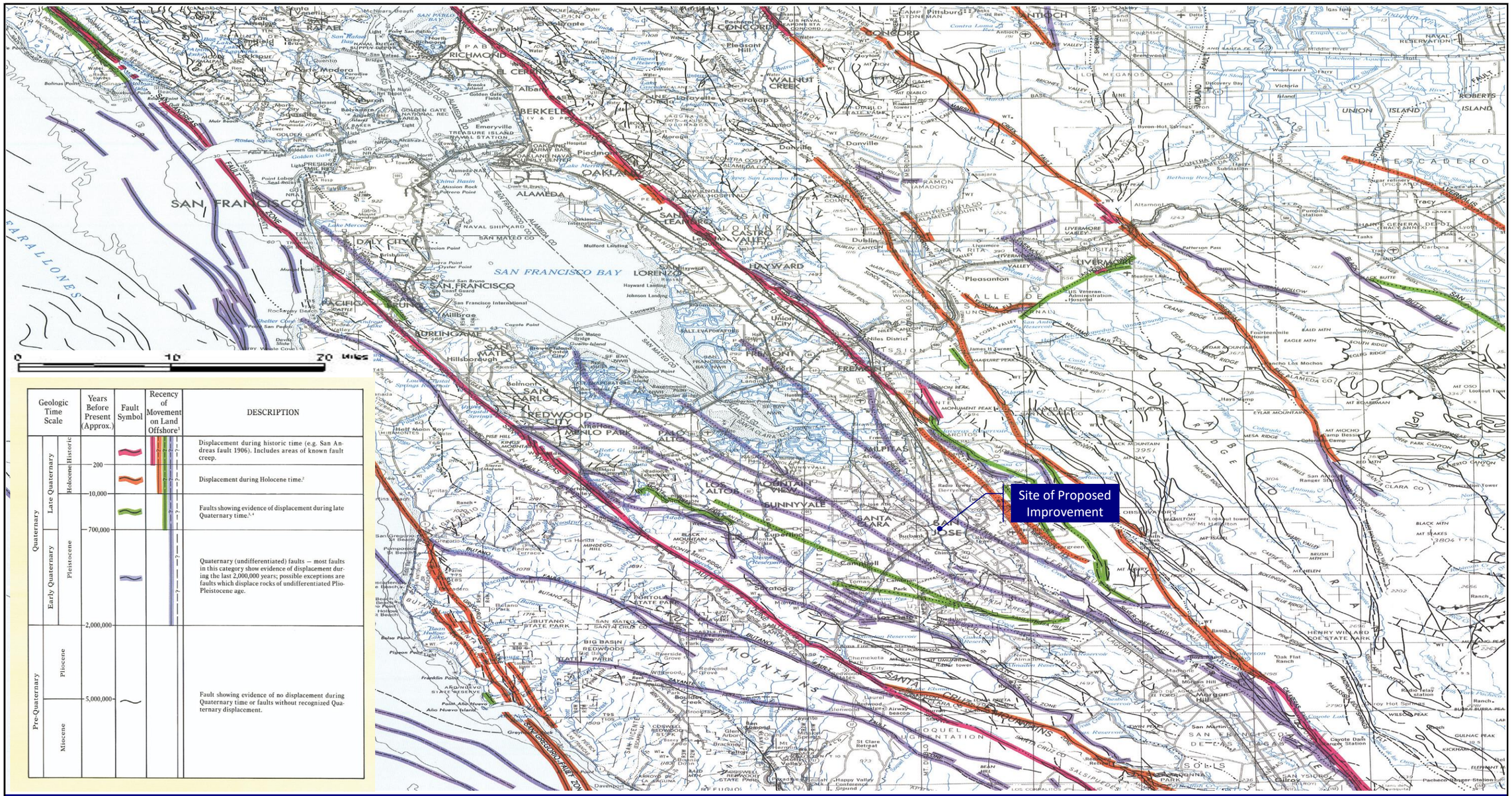
CGS official Map - San Jose West Quadrangle Map - Dated February 2002

Seismic Hazard Zone Report 058 by Kevin B. Clahan, Ellise Mattison, Anne Rosinski

Geological Map

Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **Clariana Hotel Addition** New Concrete Podium Deck with Five Levels Above
343 So. Baywood Avenue | San Jose, California	10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 4



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	10,000			Displacement during Holocene time.[2]
	Early Quaternary	Pleistocene	700,000			Faults showing evidence of displacement during late Quaternary time.[3, 4]
			2,000,000			Quaternary (undifferentiated) faults — most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary	Pliocene		5,000,000			Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.
	Miocene					

California Fault Map

Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study | Investigation **Clariana Hotel Addition** New Concrete Podium With Five-Levels Above	RSTP Investments, LLC
343 So. Baywood Avenue | San Jose, California	10 So. Third Street | San Jose, California	100 E. Santa Clara Street | San Jose, California
AST File No. 17364-S	Date: July 2018	Plate No. 5

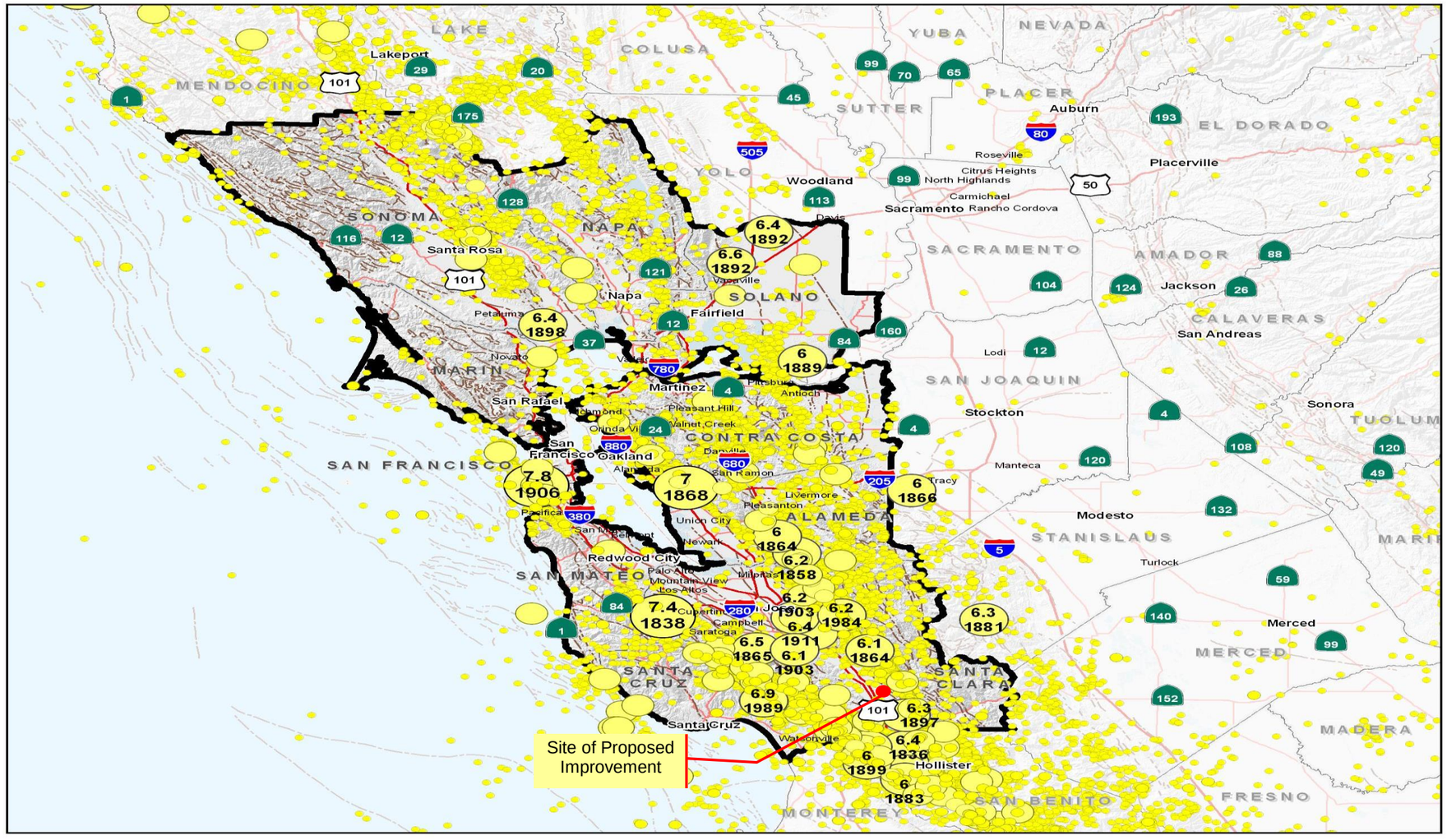
ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A,B,C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (R,SS,DS,BT)
Silver Creek	1.3	B	6.9	----	SS
Cascade	7.4	B	6.7	----	R
HAYWARD (SE Extension)	8.8	B	6.5	3.00	SS
MONTE VISTA - SHANNON	12.1	B	6.5	0.40	DS
CALAVERAS (So. of Calaveras Res)	13.2	B	6.2	15.00	SS
HAYWARD (Total Length)	14.0	A	7.1	9.00	SS
CALAVERAS (No. of Calaveras Res)	14.0	B	6.8	6.00	SS
SAN ANDREAS (1906)	19.6	A	7.9	24.00	SS
SARGENT	22.5	B	6.8	3.00	SS
ZAYANTE-VERGELES	28.6	B	6.8	0.10	SS
GREENVILLE	35.1	B	6.9	2.00	SS
SAN GREGORIO	43.1	A	7.3	5.00	SS
MONTEREY BAY - TULARCITOS	50.7	B	7.1	0.50	DS
ORTIGALITA	54.3	B	6.9	1.00	SS
CONCORD - GREEN VALLEY	61.8	B	6.9	6.00	SS
PALO COLORADO - SUR	63.9	B	7.0	3.00	SS
QUIEN SABE	65.1	B	6.5	1.00	SS
SAN ANDREAS (Creeping)	66.3	B	5.0	34.00	SS
RINCONADA	74.5	B	7.3	1.00	SS
RODGERS CREEK	92.6	A	7.0	9.00	SS
WEST NAPA	97.3	B	6.5	1.00	SS
POINT REYES	107.4	B	6.8	0.30	DS
HUNTING CREEK - BERRYESSA	127.1	B	6.9	6.00	SS
HOSGRI	133.0	B	7.3	2.50	SS
MAACAMA (South)	155.0	B	6.9	9.00	SS
COLLAYOMI	175.0	B	6.5	0.60	SS
BARTLETT SPRINGS	185.7	A	7.1	6.00	SS
SAN ANDREAS - 1857 Rupture	189.2	A	7.8	34.00	SS
MAACAMA (Central)	194.8	A	7.1	9.00	SS
SAN JUAN	231.9	B	7.0	1.00	SS
GENOA	236.1	B	6.9	1.00	DS
LOS OSOS	243.6	B	6.8	0.50	DS
ROBINSON CREEK	247.0	B	6.5	0.50	DS
ANTELOPE VALLEY	249.4	B	6.7	0.80	DS
SAN LUIS RANGE (S. Margin)	250.6	B	7.0	0.20	DS
MONO LAKE	252.0	B	6.6	2.50	DS
MAACAMA (North)	255.0	A	7.1	9.00	SS
HARTLEY SPRINGS	255.1	B	6.6	0.50	DS
ROUND VALLEY (N. S.F.Bay)	268.4	B	6.8	6.00	SS

Regional Faults

Bay Area ~ ShakeOut Area
Historical Epicenters

Earthquake Country Alliance
We're all in this together.

Cal EMA
CALIFORNIA EMERGENCY MANAGEMENT AGENCY

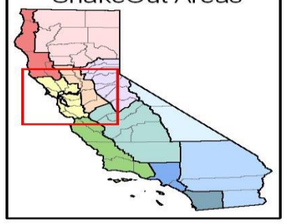


Historical Epicenters By Magnitude

- 3.00 - 3.99
- 4.00 - 4.99
- 5.00 - 5.99
- 6.00 - 6.99
- 7.00 - 7.90

- Known Earthquake Faults
- Freeway
- State Highways

ShakeOut Areas



Sources:
Earthquake Faults - Ca Geologic Survey
Epicenters - Ca Geologic Survey and ANSS
ANSS is comprised of NCEDC and various member organizations
http://www.ncedc.org/acknowledge.html

0 5 10 20
Miles

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Created by Cal-EMA GIS Unit, Kris Higgs
September 8, 2009, Revised June 8, 2010
\\data\admin\shakeout_regions\projects\June_10 Bay-Area_epicenters_10.mxd

Historical Seismicity

AST **Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study | Investigation **Clariana Hotel Addition** New Concrete Podium With Five-Levels Above	RSTP Investments, LLC
343 So. Baywood Avenue | San Jose, California	10 So. Third Street | San Jose, California	100 E. Santa Clara Street | San Jose, California
AST File No. 17364-S	Date: July 2018	Plate No. 7

Magnitude / Intensity Comparison

Magnitude and Intensity measure different characteristics of earthquakes. Magnitude measures the energy released at the source of the earthquake. Magnitude is determined from measurements on seismographs. Intensity measures the strength of shaking produced by the earthquake at a certain location. Intensity is determined from effects on people, human structures, and the natural environment. The following table gives intensities that are typically observed at locations near the epicenter of earthquakes of different magnitudes.

Magnitude / Intensity Comparison	
Magnitude	Typical Maximum Modified Mercalli Intensity
1.0 - 3.0	I
3.0 - 3.9	II - III
4.0 - 4.9	IV - V
5.0 - 5.9	VI - VII
6.0 - 6.9	VII - IX
7.0 and higher	VIII or higher

Abbreviated Modified Mercalli Intensity Scale

I. Not felt except by a very few under especially favorable conditions.

II. Felt only by a few persons at rest, especially on upper floors of buildings.

III. Felt quite noticeably by persons indoors, especially on upper floors of buildings. Many people do not recognize it as an earthquake. Standing motorcars may rock slightly. Vibrations similar to the passing of a truck. Duration estimated.

IV. Felt indoors by many, outdoors by few during the day. At night, some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motorcars rocked noticeably.

V. Felt by nearly everyone; many awakened. Some dishes, windows broken. Unstable objects overturned. Pendulum clocks may stop.

VI. Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.

VII. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken.

VIII. Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, and walls. Heavy furniture overturned.

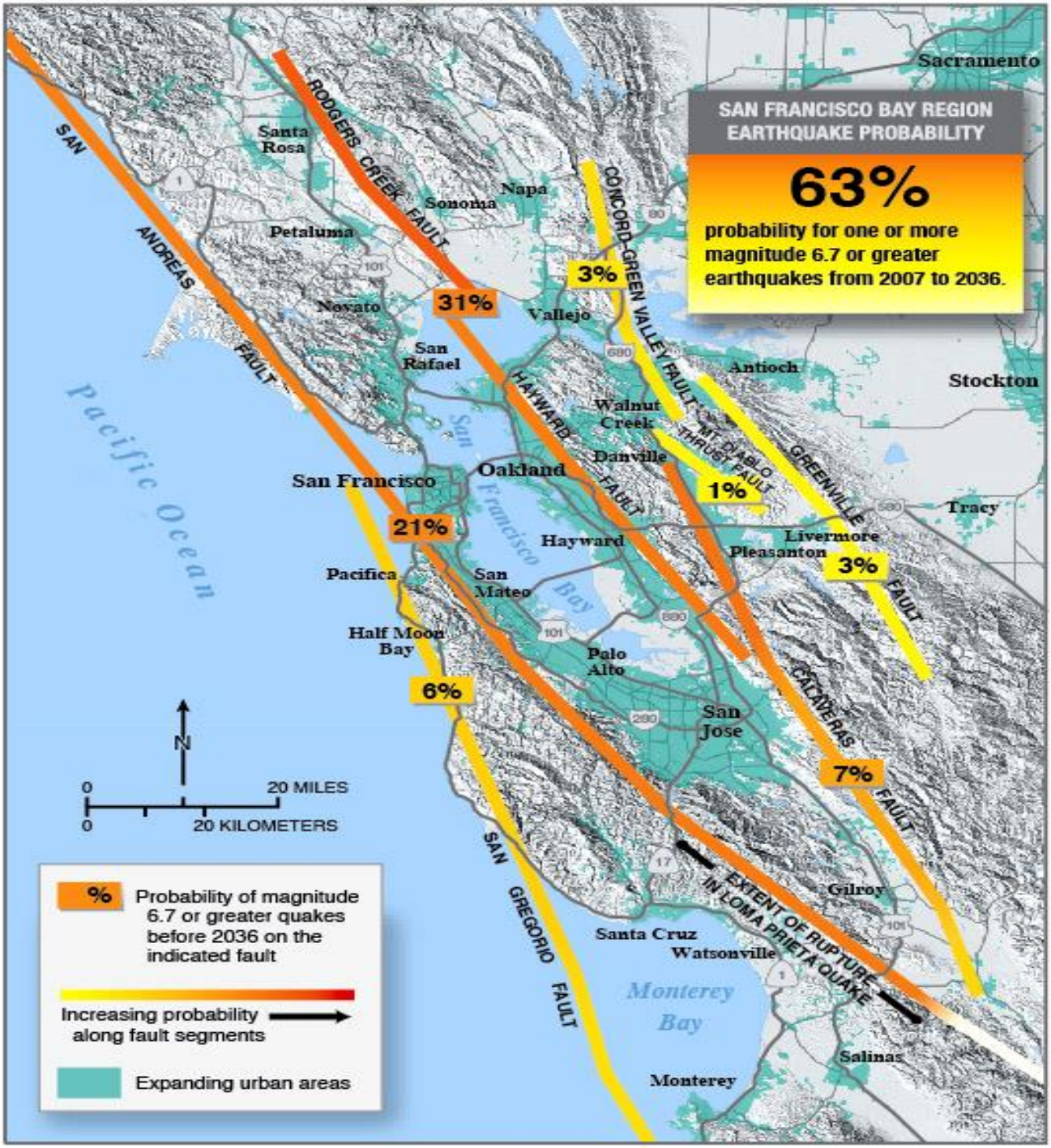
IX. Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations.

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent.

XI. Few, if any (masonry) structures remain standing. Bridges destroyed. Rails bent greatly.

XII. Damage total. Lines of sight and level are distorted. Objects thrown into the air.

Modified Mercalli Intensity Scale

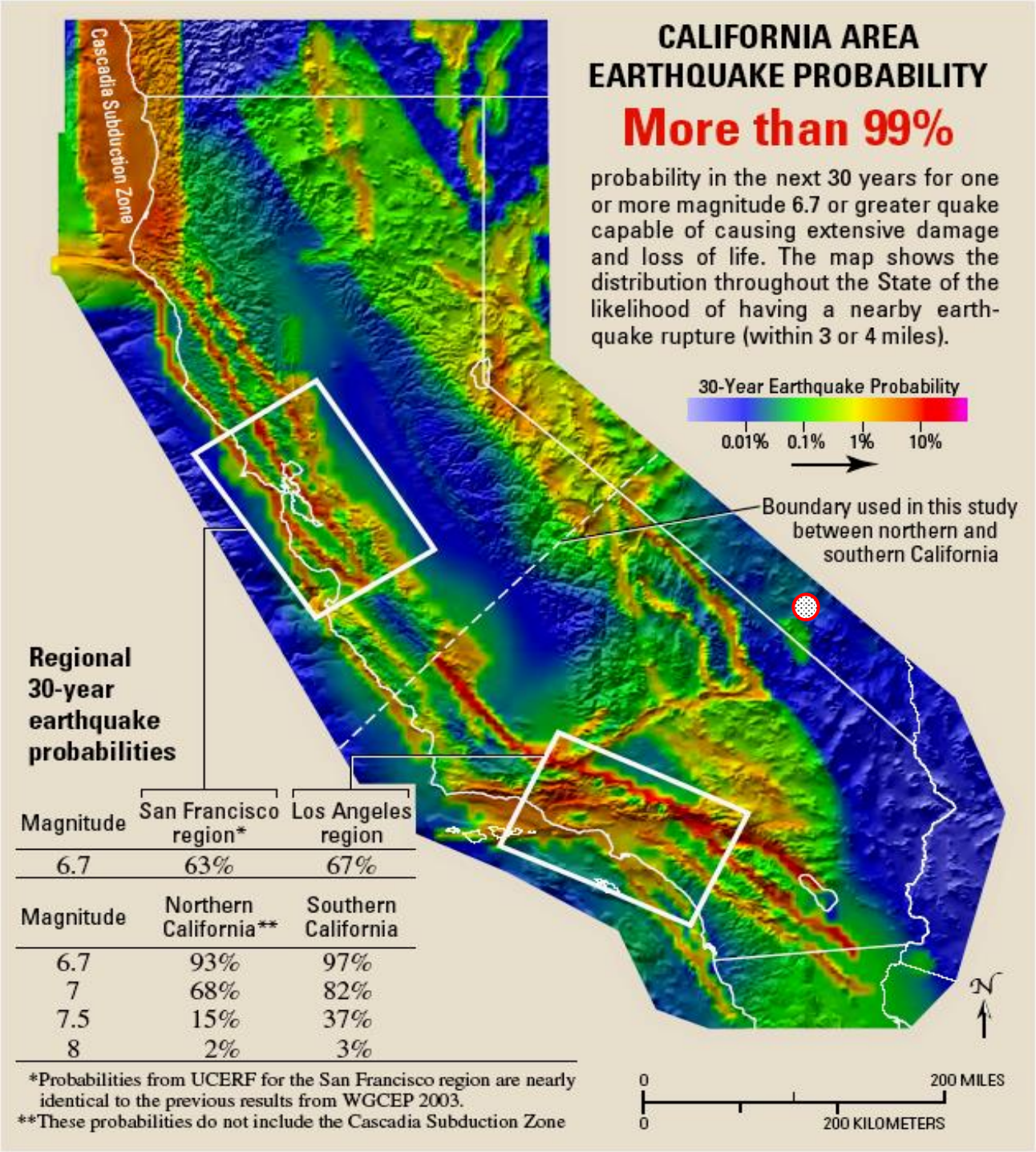


Source USGS Open File Report 2007-1437 CGS Special Report 203 (WGCEP 2008)

San Francisco Bay Area Region Earthquake Probabilities

Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **Clariana Hotel Addition** New Concrete Podium Deck with Five Levels Above
343 So. Baywood Avenue | San Jose, California	10 South Third Street | San Jose, California

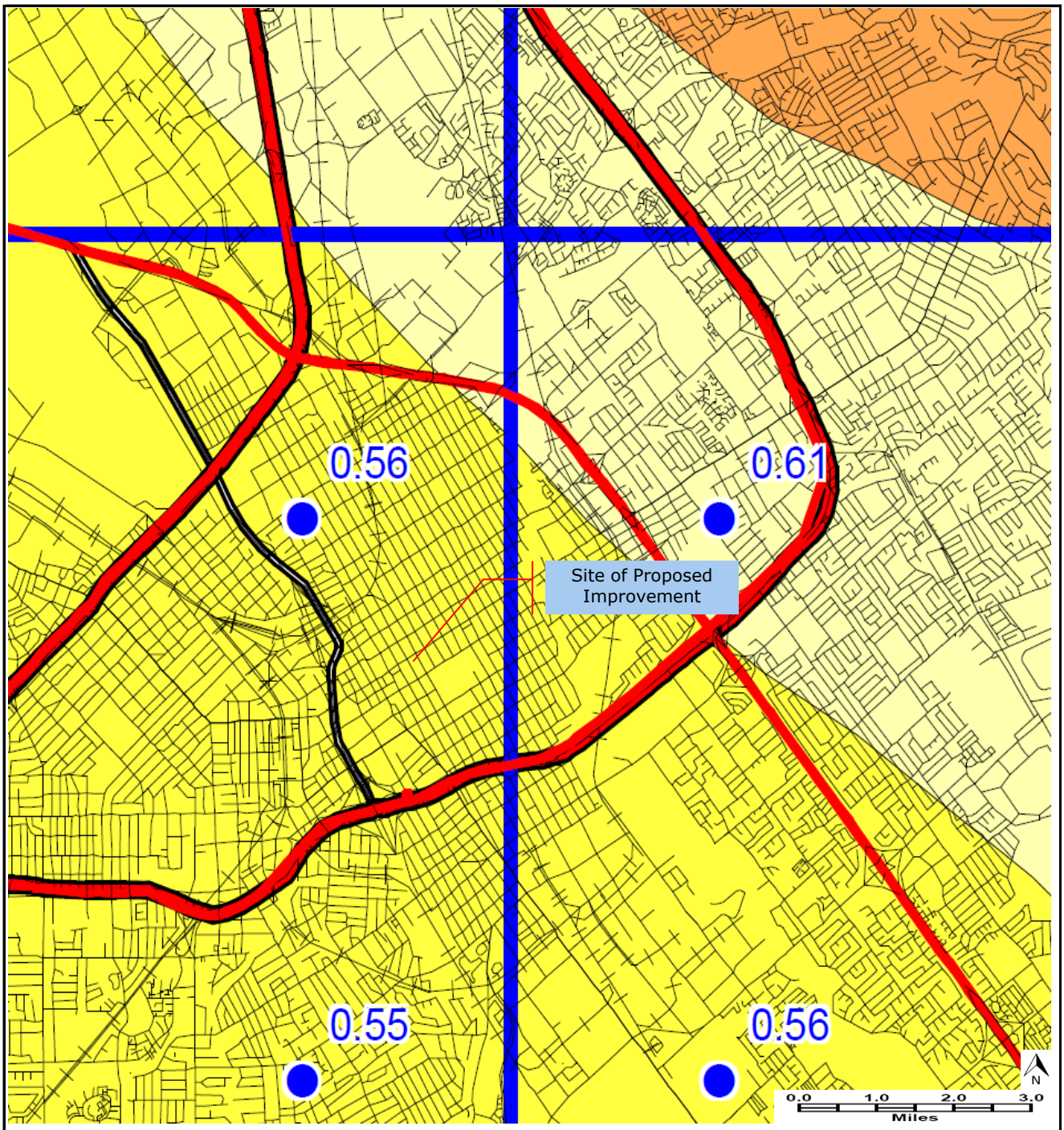
AST File No. 17364-S	Date: July 2018	Plate No. 9



California Earthquake Probabilities

Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **Clariana Hotel Addition** New Concrete Podium Deck with Five Levels Above
343 So. Baywood Avenue | San Jose, California	10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 10



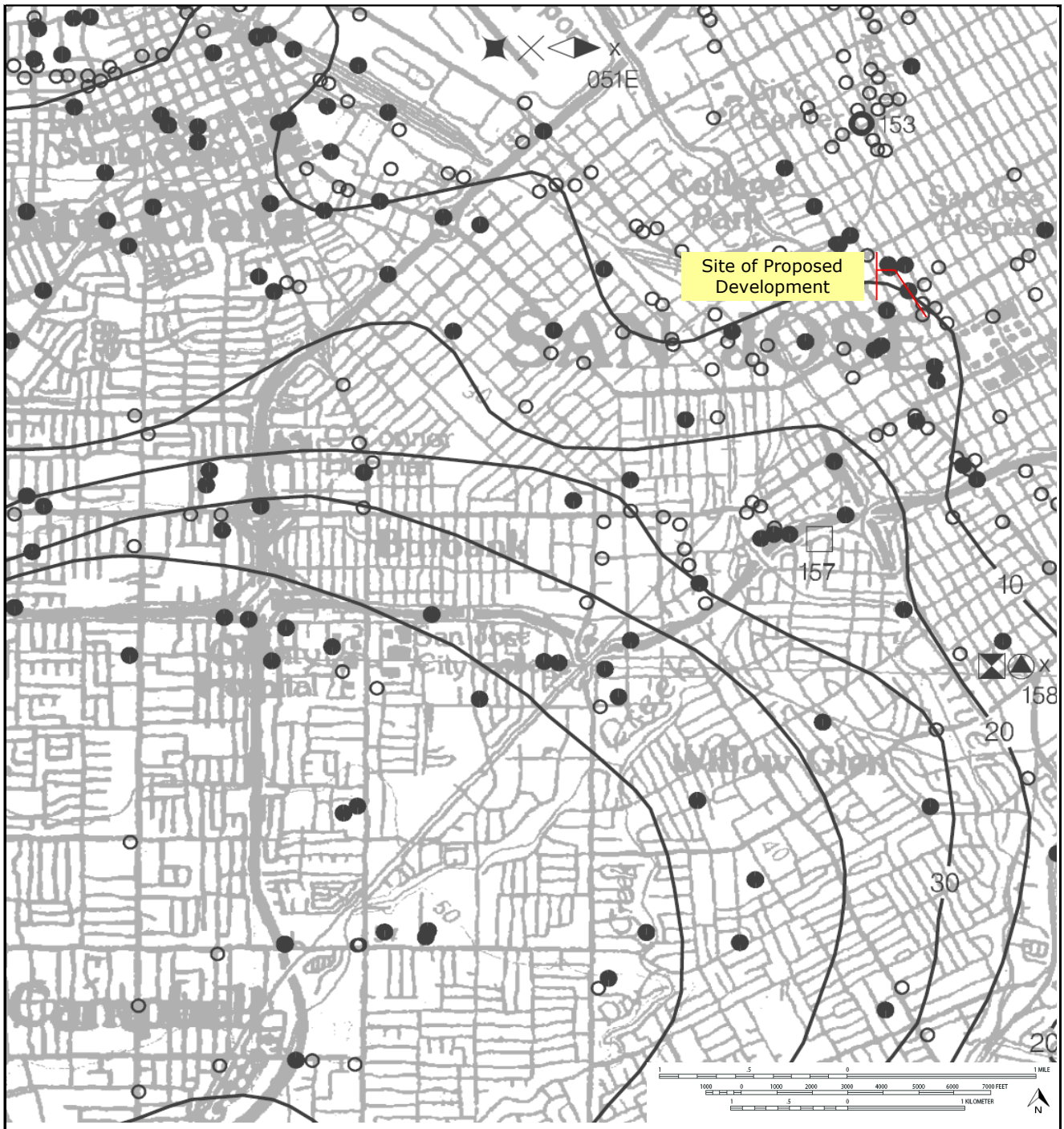
CGS Official Map - San Jose West Quadrangle Map - Dated February 2002

Seismic Hazard Zone Report 058 by Kevin B. Clahan, Ellise Mattison, Anne Rosinski

Peak Ground Acceleration

Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **Clariana Hotel Addition** New Concrete Podium Deck with Five Levels Above
343 So. Baywood Avenue | San Jose, California	10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 11



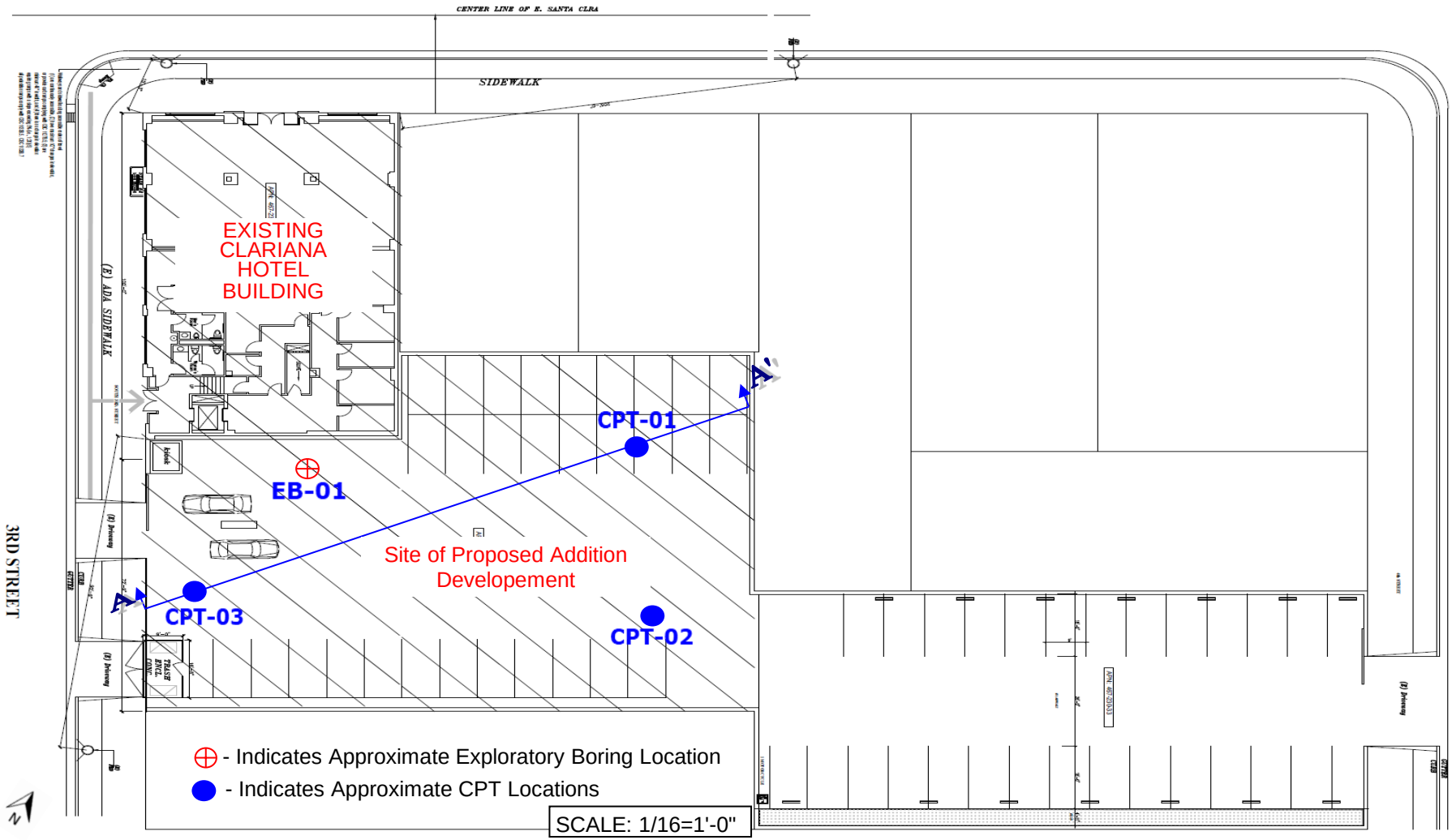
CGS Official Map - San Jose West Quadrangle Map - Dated February 2002

Seismic Hazard Zone Report 058 by Kevin B. Clahan, Ellise Mattison, Anne Rosinski

Historical Groundwater

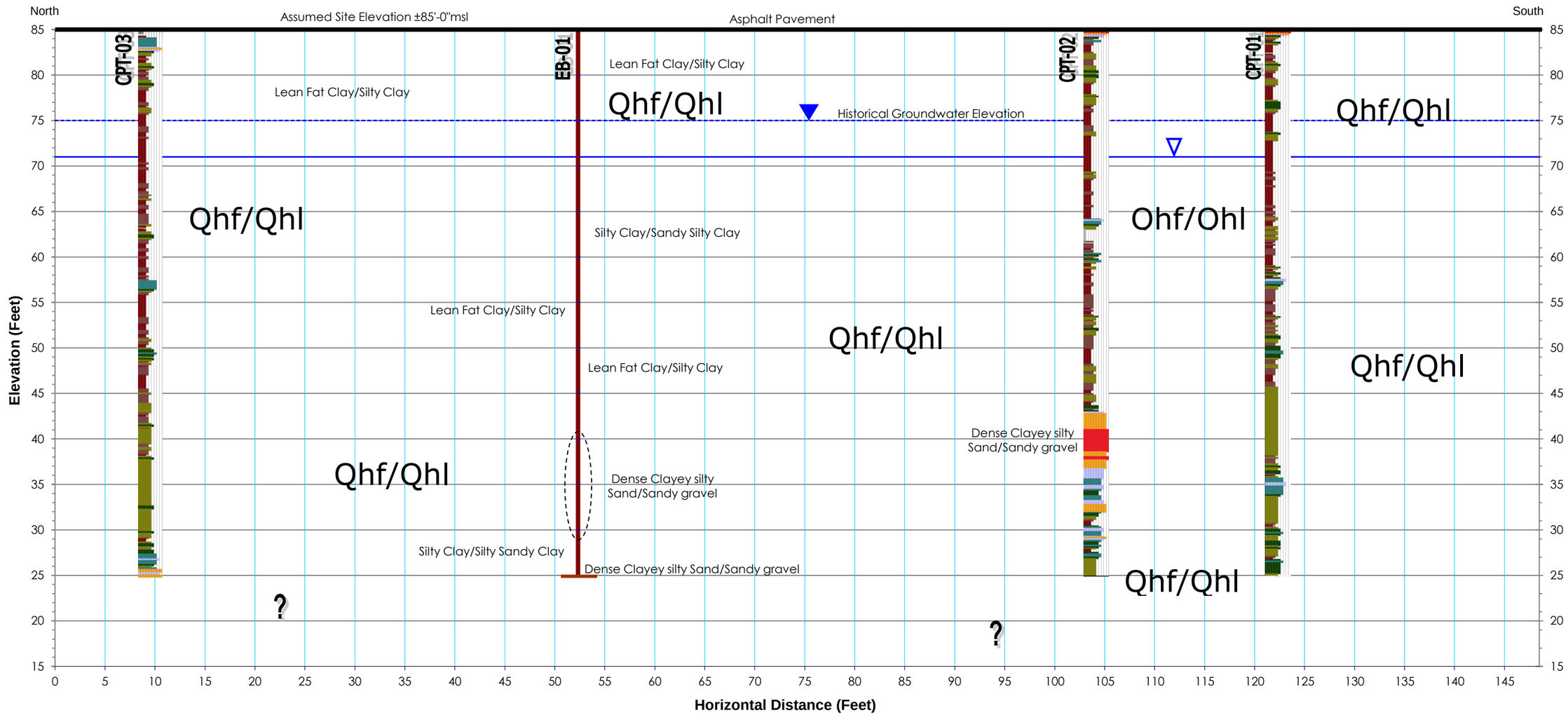
Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **Clariana Hotel Addition** New Concrete Podium Deck with Five Levels Above
343 So. Baywood Avenue | San Jose, California	10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 12



Clariana Hotel Addition

AST **Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services		Geotechnical Study | Investigation **Clariana Hotel Addition** New Concrete Podium With Five Levels Above
343 So. Baywood Avenue I San Jose, California		10 South Third Street | San Jose, California
AST File No. 17364-S	Date: July 2018	Plate No. 13



Legend:

EB-01 PROJ. ±24.0-FEET EAST CPT-02 PROJ. ±52.3-FEET WEST	**Qhf** Holocene Alluvial Fan Deposits	**Qhl** Holocene Alluvial Fan Levee Deposits	1 - sensitive fine grained; 2 - organic material; 3 - clay; 4 - silty clay to clay; 5 - clayey silt to silty clay; 6 - sandy silt to clayey silt; 7 - silty sand to sandy silt; 8 - sand to silty sand; 9 - sand; 10 - gravelly sand to sand; 11 - very stiff fine grained (*); 12 - sand to clayey sand (*)

Idealized Geological Cross Section A-A'

AST **Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study | Investigation **Clariana Hotel Addition** New Concrete Podium With Five-Levels Above	**RSTP Investments, LLC**
343 So. Baywood Avenue | San Jose, California	10 So. Third Street | San Jose, California	100 E. Santa Clara Street | San Jose, California
AST File No. 17364-S	Date: July 2018	Plate No. 14

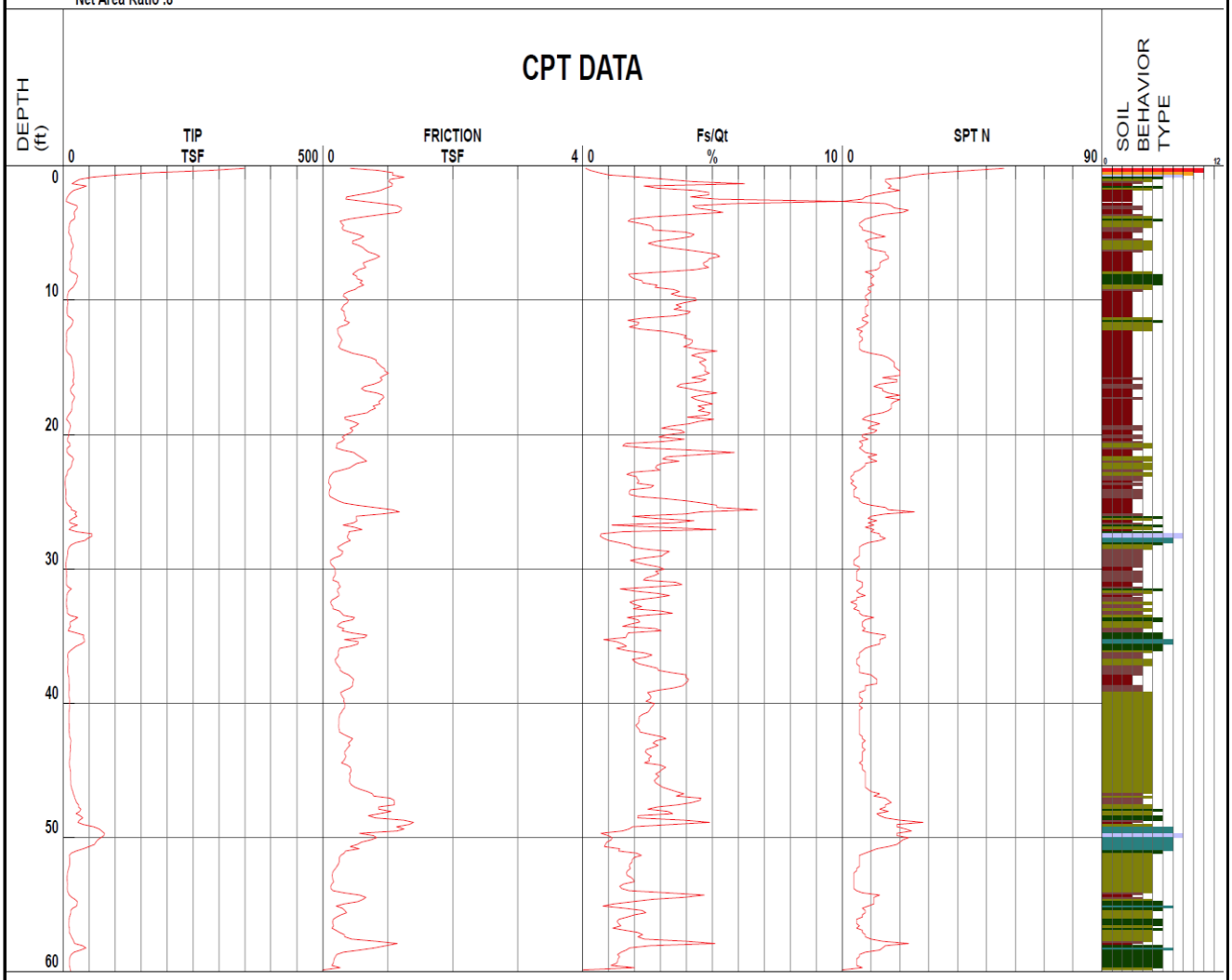
Appendix "B"

Plate 15-20 Cone Penetrometer Tests (CPT) Logs
Plate 21-23 Soil Classification, Terminology & Abbreviations
Plate 24-26 Exploratory Boring Log (EB-1)

Advance Soil Technology, Inc.

Project	Clariana Hotel	Operator	BH-RB-JO	Filename	SDF(058).cpt
Job Number	17364-S	Cone Number	DDG1418	GPS	
Hole Number	CPT-01	Date and Time	2/15/2018 9:01:19 AM	Maximum Depth	60.20 ft
EST GW Depth During Test		14.50 ft			

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

*Soil behavior type and SPT based on data from UBC-1983

CPT-01

Advance Soil Technology, Inc.

Geological , Geotechnical, Environmental
Consulting & Construction Services

Geotechnical Study/Investigation

Clariana Hotel Addition

New Concrete Podium Deck with Five Levels Above

343 So. Baywood Avenue | San Jose, California

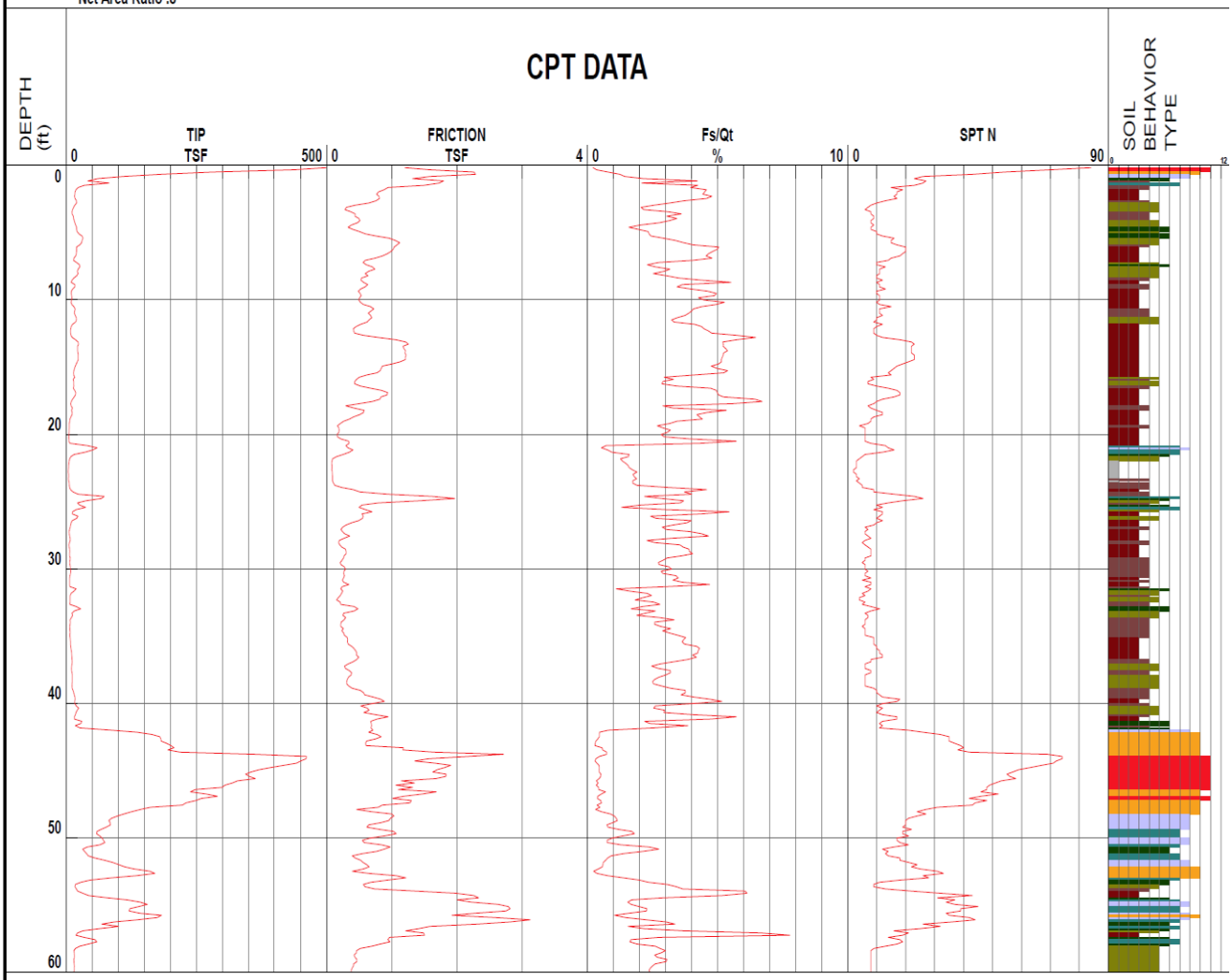
10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 15

Advance Soil Technology, Inc.

Project	Clariana Hotel	Operator	BH-RB-JO	Filename	SDF(059).cpt
Job Number	17364-S	Cone Number	DDG1418	GPS	
Hole Number	CPT-02	Date and Time	2/15/2018 9:52:37 AM	Maximum Depth	60.53 ft
EST GW Depth During Test		16.00 ft			

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

CPT-02

Advance Soil Technology, Inc.
Geological , Geotechnical, Environmental
Consulting & Construction Services

Geotechnical Study/Investigation
Clariana Hotel Addition
New Concrete Podium Deck with Five Levels Above

343 So. Baywood Avenue | San Jose, California

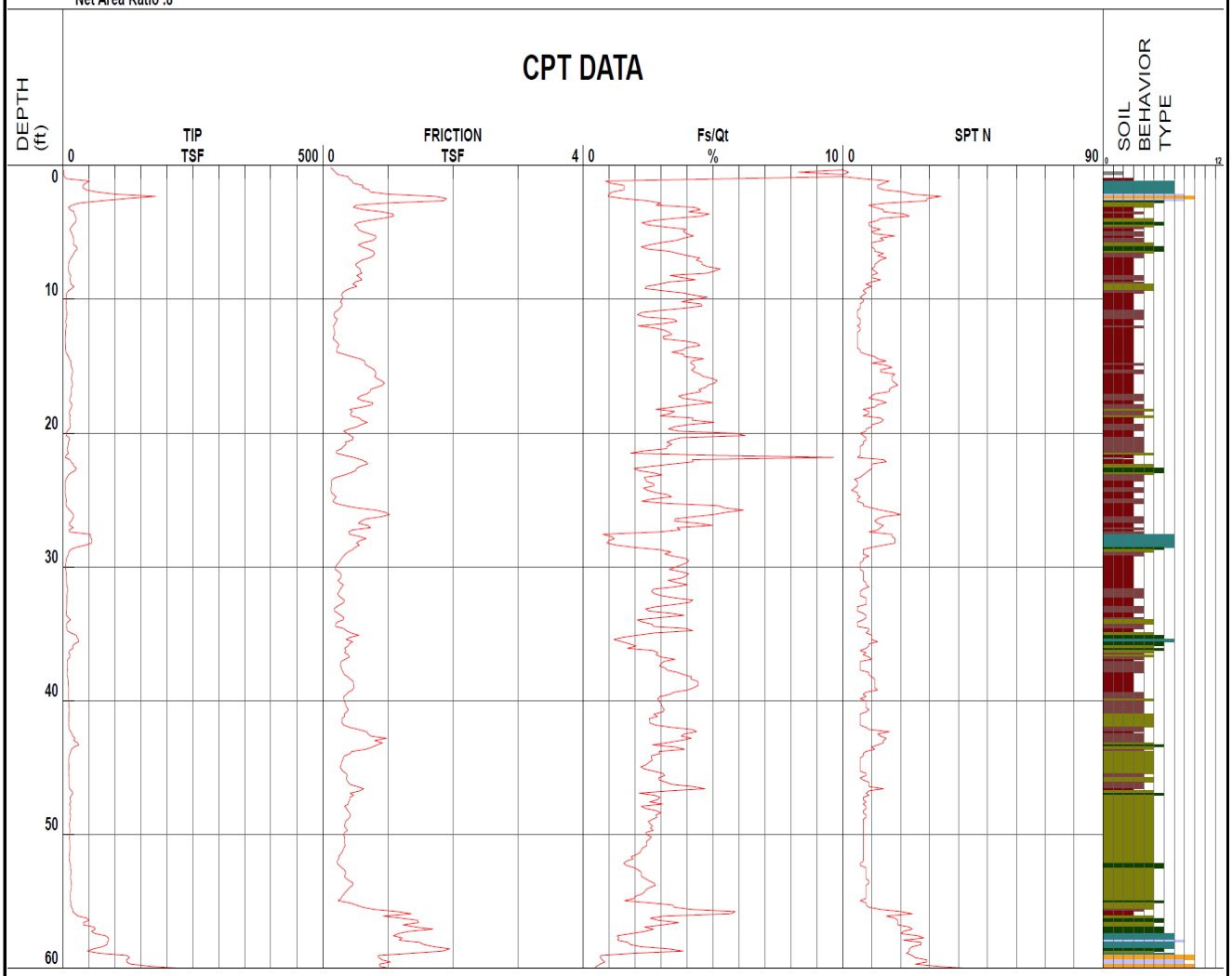
10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 16

Advance Soil Technology, Inc.

Project	Clariana Hotel	Operator	BH-RB-JO	Filename	SDF(057).cpt
Job Number	17364-S	Cone Number	DDG1418	GPS	
Hole Number	CPT-03	Date and Time	2/15/2018 8:13:18 AM	Maximum Depth	60.53 ft
EST GW Depth During Test		15.90 ft			

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

*Soil behavior type and SPT based on data from UBC-1983

CPT-03

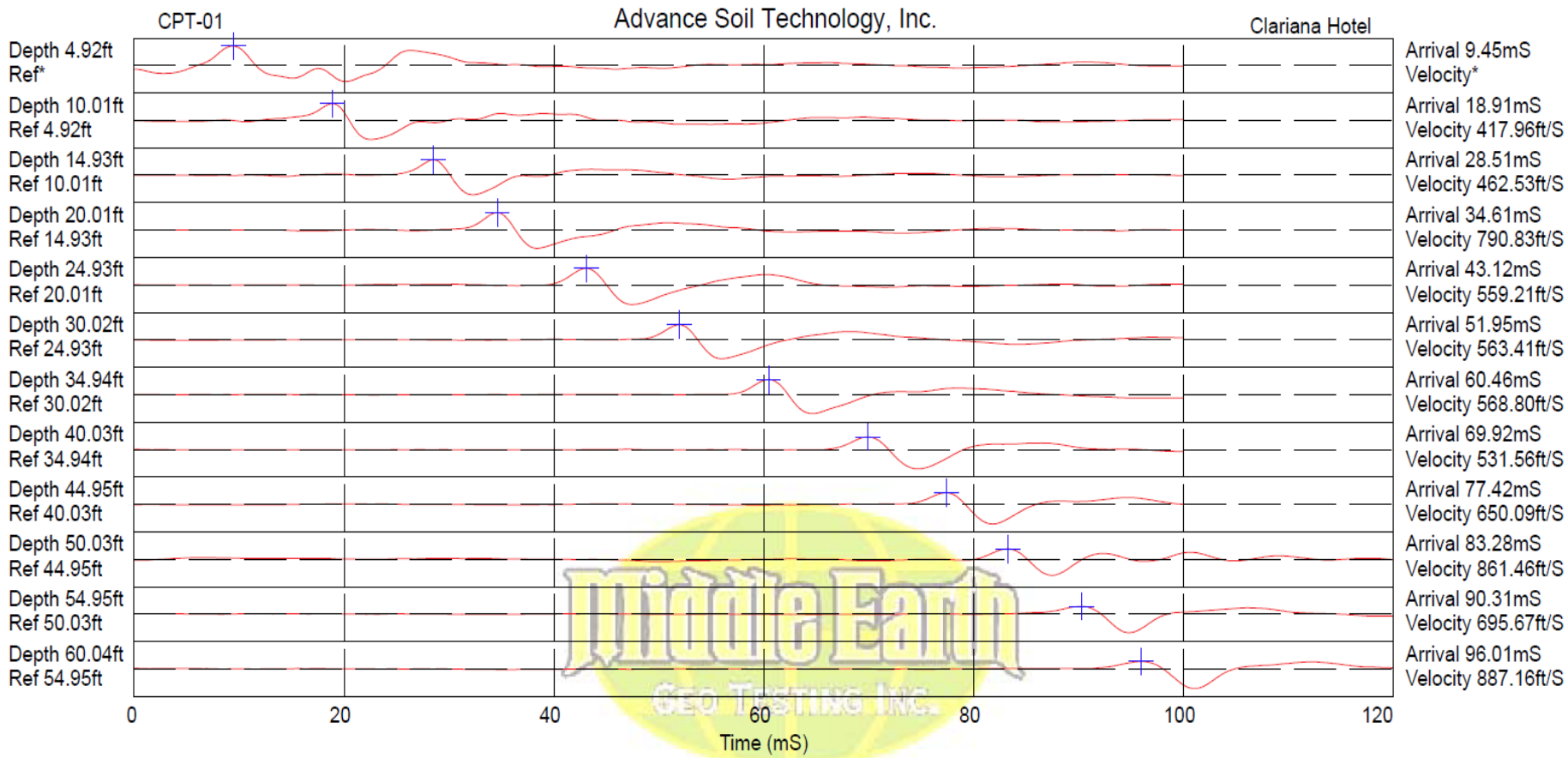
Advance Soil Technology, Inc.
Geological , Geotechnical, Environmental
Consulting & Construction Services

Geotechnical Study/Investigation
Clariana Hotel Addition
New Concrete Podium Deck with Five Levels Above

343 So. Baywood Avenue | San Jose, California

10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 17



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

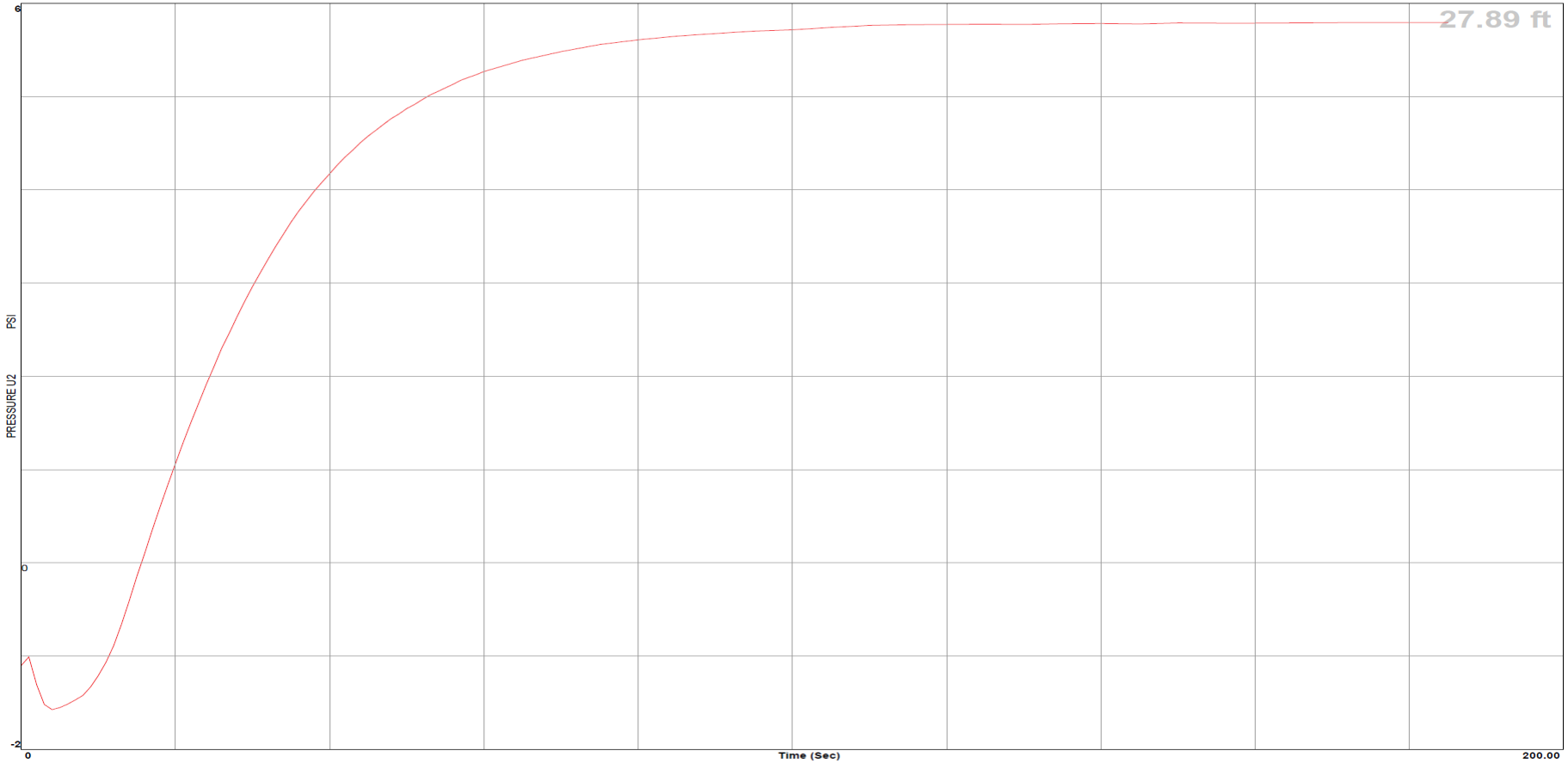
HOLE NUMBER: CPT-01

Shear Velocity

AST **Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services		Geotechnical Study | Investigation **Clariana Hotel Addition** New Concrete Podium Deck with Five-Story Levels Above
343 So. Baywood Avenue I San Jose, California		10 South Third Street | San Jose, California
AST File No. 17364-S	Date: July 2018	Plate No. 18

Advance Soil Technology, Inc.

Location	Clariana Hotel	Operator	BH-RB-JO	GPS	
Job Number	17364-S	Cone Number	DDG1418		
Hole Number	CPT-01	Date and Time	2/15/2018 9:01:19 AM		
Equilized Pressure	5.7	EST GW Depth During Test	14.5		

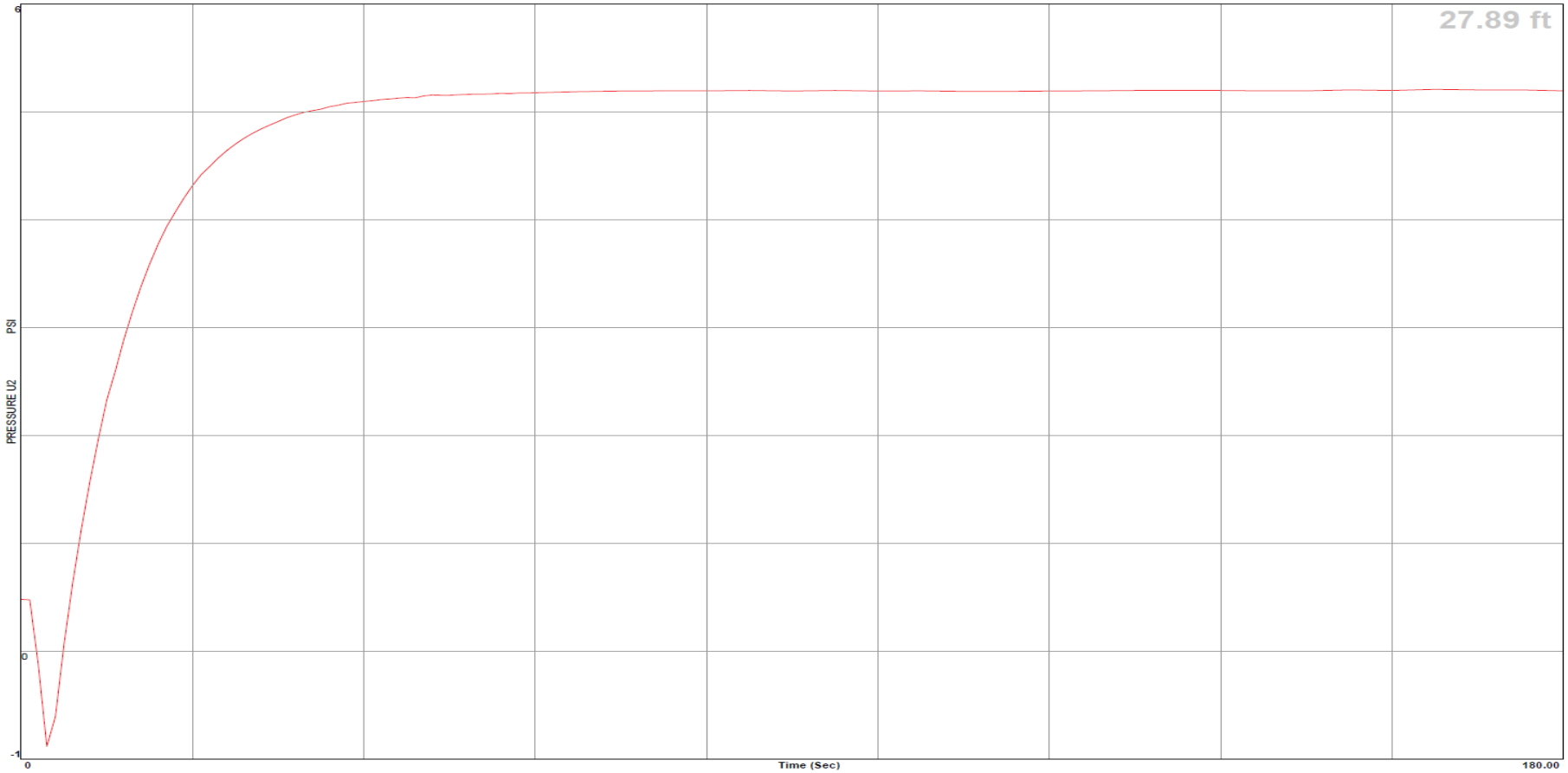


Pore Pressure Reading CPT-01

AST AST	**Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study | Investigation **Clariana Hotel Addition** New Concrete Podium Deck with Five-Story Levels Above
343 So. Baywood Avenue I San Jose, California		10 South Third Street | San Jose, California
AST File No. 17364-S	Date: July 2018	Plate No. 19

Advance Soil Technology, Inc.

Location	Clariana Hotel	Operator	BH-RB-JO	GPS	
Job Number	17364-S	Cone Number	DDG1418		
Hole Number	CPT-03	Date and Time	2/15/2018 8:13:18 AM		
Equilized Pressure	5.1	EST GW Depth During Test	15.9		



Pore Pressure Reading CPT-03

AST AST	**Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study | Investigation **Clariana Hotel Addition** New Concrete Podium Deck with Five-Story Levels Above
343 So. Baywood Avenue I San Jose, California		10 South Third Street | San Jose, California
AST File No. 17364-S	Date: July 2018	Plate No. 20

COARSE-GRAINED SOILS

LESS THAN 50% Fines

GROUP SYMBOLS	ILLUSTRATED GROUP NAMES	MAJOR DIVISIONS
GW	Well graded gravel Well graded gravel with sand	GRAVELS More than Half of coarse fraction is larger than No. 4 sieve size
GP	Poorly graded gravel Poorly graded gravel with sand	
GM	Silty gravel Silty gravel with sand	
GC	Clayey gravel Clayey gravel with sand	
SW	Well graded Sand Well graded sand with gravel	SANDS More than half of coarse fraction is smaller than No. 4 sieve size
SP	Poorly graded sand Poorly graded sand with gravel	
SM	Silty Sand Silty Sand with gravel	
SC	Clayey sand Clayey sand with gravel	

Note: Coarse grained soils receive dual symbols if:
* The fines are CL-ML or GC-GM or SC-SM
* If they contain 5-12% fines

FINE-GRAINED SOILS

MORE THAN 50% FINES*

GROUP SYMBOLS	ILLUSTRATED GROUP NAMES	MAJOR DIVISIONS
CL	Lean clay Sandy clay with gravel	SILTS & CLAYS liquid limit less than 50
ML	Silt Sandy, Clayey silt with fine sand	
OL	Organic clay Sandy organic clay with gravel	
CH	Clayey gravel Clayey gravel with sand	SILTS & CLAYS liquid limit more than 50
MH	Elastic Silt Sandy elastic silt with gravel	
OH	Organic clay Sandy organic clay with gravel	
PT	Peat Highly organic silt	HIGHLY ORGANIC SOILS

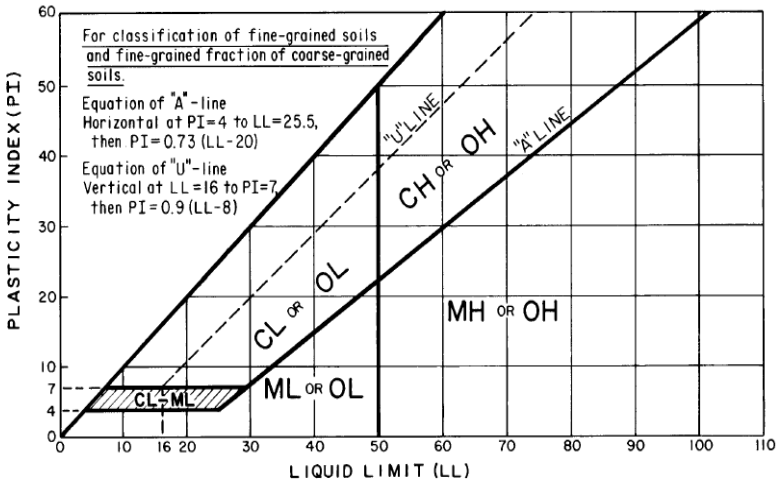
Note: Fine Grained soils will receive dual symbols if their limits in the hatches zone on the Plasticity Chart

SOIL SIZES

COMPONENT	SIZE RANGE
BOULDERS	Above 12 in.
COBBLES	3 in. to 12 in.
GRAVEL	No. 4 to 3 in.
Coarse	3/4 in. to 3 in.
Fine	No. 4 to 3/4 in.
SAND	No. 200 to No. 4
Coarse	No. 10 to No. 4
Medium	No. 40 to No. 10
Fine	No. 200 to No. 40
FINES	Below No. 200

Note: Classification is based on the portion of a sample that passes the 3-inch sieve

PLASTICITY CHART



Reference: ASTM D2487-00. Standard Classification of Soils for Engineering purposes (Unified Soil Classification System)

General Notes: The tables list 30 out of a possible 110 group names, all of which are assigned to unique proportions of constituent soils. Flow Charts in ASTM D2487 aid assignment of the group names. Some general rules for fine grained soils are: less than 15% sand or gravel is not mentioned; 15% to 25% sand or gravel is termed "with sand or with gravel, and 30 to 49% sand or gravel is termed as sandy or gravelly. Some general rules for coarse-grained soils are uniformly-graded or gap-graded soils are poorly graded (SP or GP); 15% or more sand or gravel is termed "with sand" or "with gravel". 15% to 25% clay and silt is termed clayey and silty and any cobbles or boulders are termed "with cobbles" or "with boulders".

Unified Soil Classification System

Advance Soil Technology, Inc.
Geological , Geotechnical, Environmental
Consulting & Construction Services

343 So. Baywood Avenue | San Jose, California

Geotechnical Study/Investigation

Clariana Hotel Addition

New Concrete Podium Deck with Five Levels Above

10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 21

Soil Types

Boulders	Particles of rock that will not pass a 12-inch screen
Cobbles	Particles of rock that will pass a 12-inch screen; but not a 3-inch sieve
Gravel	Particles of rock that will pass a 3-inch sieve; but not a #4 sieve
Sand	Particles of rock that will pass a #4 sieve; but not a #200 sieve
Silt	soil that will pass a #200 sieve; that is non-plastic or very slightly plastic, and that exhibits little or no strength when dry
Clay	soil that will pass a #200 sieve; that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength, when dry

Measures of Consistency of Cohesive Soils (Clays)

Very Soft	N=0-1*	C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by Strong finger pressure
Very Stiff	N=16-30	C=2000-4000 psf	Dented by slight finger pressure
Hard	N=30	C=4000 psf	Dented slightly by pencil point

*N=blows per foot in the Standard Penetration Test. In cohesive soils, with the 3-inch diameter ring sampler. 140-pound weight, divide the blow by 1.2 get N

Moisture & Density

Moisture Condition	an observation term, dry, moist, wet or saturated
Moisture Content	the weight of water in a sample divided by the weight of dry soil in the soil sample, expressed as a percentage
Dry Density	pounds of dry soil in a cubic foot

Measures of Relative Density of Granular Soils (Gravels, Sands and Silts)

Very Loose	N=0-4*	RD=0-30	Easily push a 1/2-inch Reinf Bar by hand
Loose	N=5-10	RD=30-50	Push a 1/2-inch Reinf Bar by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a 1/2-inch Reinf Bar
Dense	N=31-50	RD=70-90	Drive a 1/2-inch Reinf Bar one-foot
Very Dense	N=50	RD=90-100	Drive a 1/2-inch Reinf Bar a few inches

*N=blows per foot in the Standard Penetration Test. In granular soils, with the 3-inch diameter ring sampler, 140-pound weight, divide the blow by 2 get N

Reference

*ASTM Designation D2487. Standard Classification of soils for Engineering Purposes

*Means R.E. and Parcher J.V. Physical Properties of Soils (1963)

*Terzaghi, Karl and Peck Ralph B. Soil Mechanics Engineering Practice (1967)

*Das B.M. (1994) Principles of Geotechnical Engineering

*Sivakugan N. Soil Classification (2000)

Soil Terminology

Sampler Symbols

- Split Spoon Sampler 3.0-inch O.D.
- Shelby Tube
- Grab Sample
- Standard Penetration Test Sampler (Terzaghi Sampler)
- Rock Core

Groundwater Symbols

- Indicates historic groundwater elevation
- Indicates final groundwater elevation
- Indicates initial groundwater elevation

msl - mean sea level
bgs - below ground surface

Geotechnical Abbreviations

*	- Stratifications shown on borings are approximate, transition between materials may be gradual
ppt	- Pocket Penetrometer Test (tsf)
UC	- Unconfined compressive strength ASTM D2166
TXUU	- Triaxial Compression test Unconsolidated Undrained ASTM D2850
TXCU	- Triaxial Compression test Consolidated Undrained ASTM D4767
TXCD	- Triaxial Compression test Consolidated Drained ASTM (USACE)
consol	- Consolidation Test ASTM D2435
% Fines Content	- Passing Sieve #200
PI	- Plasticity Index
LL	- Liquid Limit
PL	- Plastic Limit
Blows/foot	- No. of blows per foot of the driven sampler
SPT	- N value, Standard Penetration Test
C	- Cohesion
Ø	- Friction Angle

Notes & Abbreviations

AST	Date: March 02, 2018	Boring No.: EB-1
	AST Project No. 17364-S	Plate No. 24
	Client: Clariana Hotel	Driller: Exploration Geo-Services Inc.
	Location: 10 South Third Street | San Jose, CA	Drilling Method: Hollow Stem Auger
	- Indicates Seepage	Sampler: Modified California & SPT
	- Indicates Final Groundwater Elevation	Boring Diameter (in): 8.0
	- Indicates Initial Groundwater Elevation	Boring Depth: (60)-feet below existing grade

Elevation (feet)	Depth (feet)	Sample Description & Soil Type	Sample No.	Sample	Penetration Resistance	SPT Blows/Foot	Liquid Limit	Plasticity Index	% Fines (# 200)	Dry Density (pcf)	(%) Water Content	Friction Angle	Cohesion (psf)	Remarks
±84	0	(3.0)-inches of Asphalt over (5.0)-inches of Concrete over (1.0)-inch of Asphalt over silty sand with some gravel, loose												
	1													
	2	Lean Fat Clay, Silty Clay, Black to Dark Brown (CH-CL) moist, very stiff `	1-1	X	11 14 17	26	40	21		105.6	16.6			PI
	3													
	4													
±79	5	moist and stiff	1-2	X	5 7 7	12				110.6	14.7			
	6													
	7	getting soft....												
	8													
	9	Sandy Silty Clay to Silty Sandy Clay, Medium Brown (CL) moist, medium stiff	1-3	X	5 5 5	8				82.2	33.4			
±74	10													
	11													
	12													
	13													
	14	Lean Fat Clay, Silty Clay, Dark Brown (CL) moist, very stiff												
±69	15		1-4	X	7 14 20	29				110.0	24.8			
	16													
	17	getting soft....												
	18													
	19		1-5	X	4 5 6	9				95.0	27.9			
±64	20	Silty Clay, Sandy Silty Clay, Medium to Tan Brown (CL) with Grayish mottling, moist, medium stiff												
	21													
	22	getting stiff.... color change....												
	23													
	24		1-6	X	5 5 8	11				102.6	24.9			
±59	25	Silty Clay, Grayish to Bluish Gray (CL) with iron oxide stains, moist, stiff												
	26													
	27													

Date: March 02, 2018	Boring No.:	EB-1 (cont)
AST Project No. 17364-S	Plate No.	25
Client: Clariana Hotel	Driller: Exploration Geo-Services Inc.	
Location: 10 South Third Street | San Jose, CA	Drilling Method: Hollow Stem Auger	
- Indicates Seepage	Sampler: Modified California & SPT	
- Indicates Final Groundwater Elevation	Boring Diameter (in): 8.0	
- Indicates Initial Groundwater Elevation	Boring Depth: (60)-feet below existing grade	

Elevation	Depth (feet)	Sample Description & Soil Type	Sample No.	Sample	Penetration Resistance	SPT Blows/Foot	Liquid Limit	Plasticity Index	% Fines (# 200)	Dry Density (pcf)	Water Content (%)	DS Friction Angle	DS Cohesion (psf)	Remarks
	28													
	28													
±54	30	Lean Fat Clay, Silty Clay, Bluish Gray (CL) moist, medium stiff	1-7	X	6 5 5	8				98.1	29			
	31													
	32													
	33													
	34													
±49	35	moist, stiff	1-8	X	3 6 8	12	41	14		81.9	40			PI
	36													
	37													
	38													
	39													
±44	40	moist, stiff	1-9	X	9 10 15	21	33	15		106.7	22			PI
	41													
	42													
	43	Silty Sandy Gravel, Gravely Silty Coarse Sand (SP-SC) moist, very dense												
	44													
±39	45		1-10	X	26 40 50/6"	45				133.0	11			
	46													
	47													
	48													
	49													
±34	50	moist, medium dense to dense	1-11		7 10 19	29			8.9	SPT	15			#200
	51													
	52													
	53	Lean Sandy Clay, Silty Sandy Clay, Olive Green (CL) moist, very stiff, hard												
	54													
±29	55	moist, hard	1-12		9 15 22	37	30	12		SPT	24			PI

AST	Date: March 02, 2018	Boring No.: EB-1 (cont)
	AST Project No. 17364-S	Plate No. 26
	Client: Clariana Hotel	Driller: Exploration Geo-Services Inc.
	Location: 10 South Third Street | San Jose, CA	Drilling Method: Hollow Stem Auger
	- Indicates Seepage	Sampler: Modified California & SPT
	- Indicates Final Groundwater Elevation	Boring Diameter (in): 8.0
	- Indicates Initial Groundwater Elevation	Boring Depth: (60)-feet below existing grade

Elevation	Depth (feet)	Sample Description & Soil Type	Sample No.	Sample	Penetration Resistance	SPT Blows/Foot	Liquid Limit	Plasticity Index	% Fines (# 200)	Dry Density (pcf)	Water Content (%)	DS Friction Angle	DS Cohesion (psf)	Remarks
	56	Lean Sandy Clay, Silty Sandy Clay, Olive Green (CL) moist, very stiff, hard												
	57													
	58													
	59	Clayey Silty Coarse Sand (SC) moist, medium dense	1-13		7 10 35	38			5.9	105.6	17			
±24	60													
		Exploratory Boring terminated at a depth of (60)-feet below the existing ground surface (bgs)												
	61													
	62													
	63													
	64													
	65													
	66													
	67													
	68													
	69													
	70													
	71													
	72													
	73													
	74													
	75													
	76													
	77													
	78													
	79													
	80													
	81													
	82													
	83													

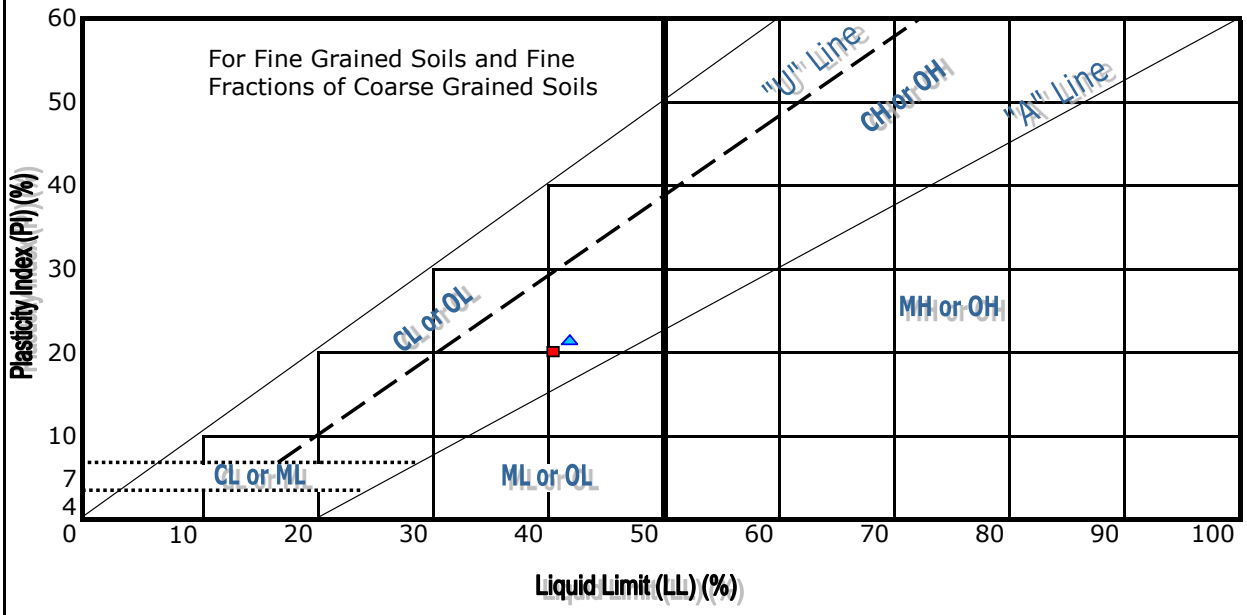
Appendix "C"

Plate 27 Plasticity Index
Plate 28 Grain Size Distribution
Plate 29 Results of Corrosivity Analysis

Plasticity Data

Key Symbol	Boring No.	Depth (feet)	Natural Water Content (Wc) %	Wc/LL	Liquid Limit %	Plasticity Index %	Unified Soil Classification & Description of Soil	
△	EB1-1	2	12.5	0.29	43	22	CL	Clay
■	EB1-2	5	14.7	0.36	41	20	CL	Clay

Plasticity Chart



Plasticity Index

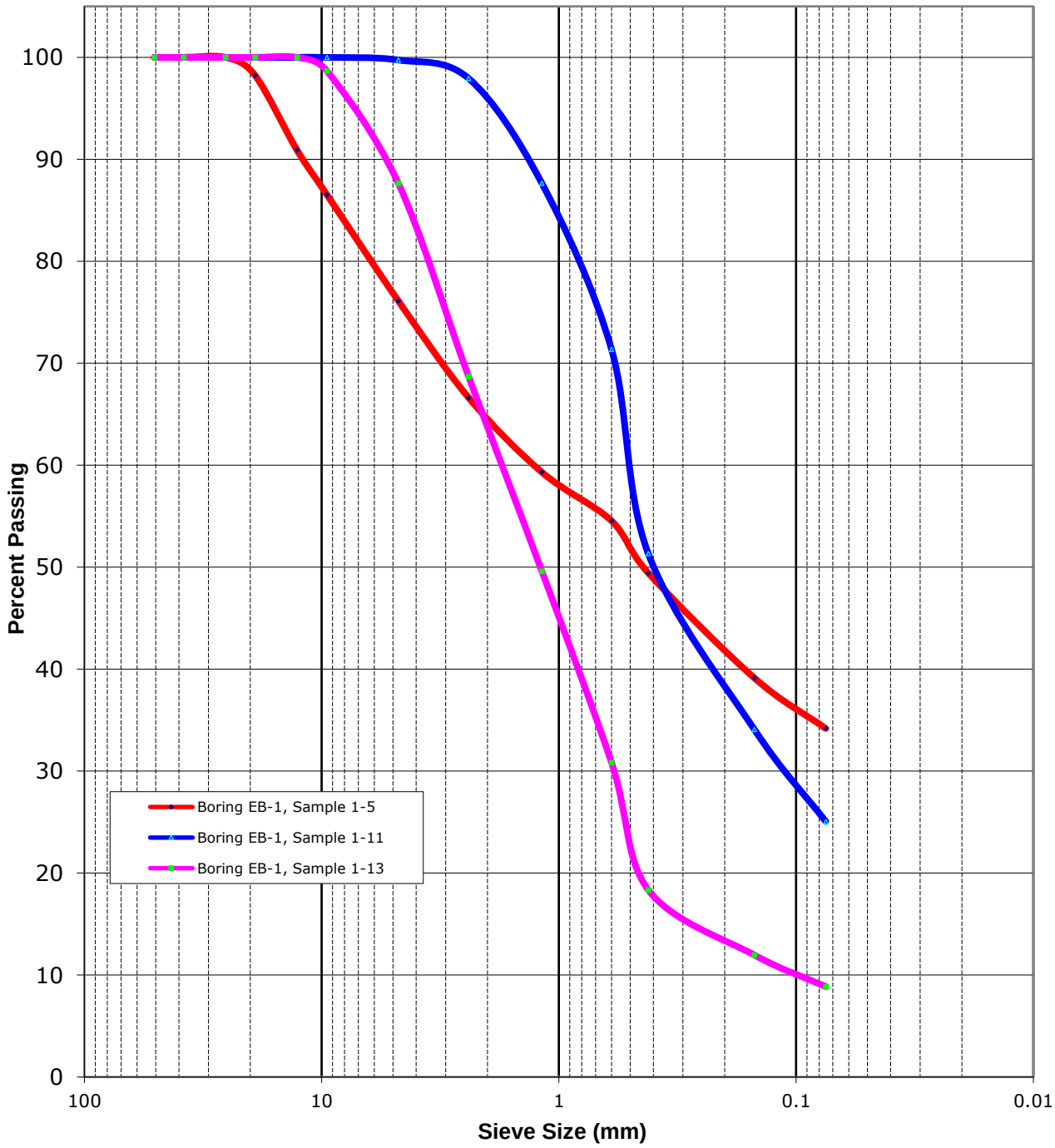
Advance Soil Technology, Inc.
Geological , Geotechnical, Environmental
Consulting & Construction Services

Geotechnical Study/Investigation
Clariana Hotel Addition
New Concrete Podium Deck with Five Levels Above

343 So. Baywood Avenue | San Jose, California

10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 27



Grain Size Distribution

Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **Clariana Hotel Addition** New Concrete Podium Deck with Five Levels Above
343 So. Baywood Avenue | San Jose, California	10 South Third Street | San Jose, California

AST File No. 17364-S	Date: July 2018	Plate No. 28

COOPER TESTING LABORATORY

Corrosivity Tests Summary

PLATE NO. 29

CTL #	132-104	**Date:**	3/12/2018	**Tested By:**	PJ	**Checked:**	PJ
Client:	Advanced Soil Technology	**Project:**	Clariana Hotel, San Jose			**Proj. No:**	17364-S
Remarks:							

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	Soil Visual Description
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	**Sample, No.**	**Depth, ft.**	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
EB-1	-	2-5	-	-	3,477	5	86	0.0086	7.9	538	21	Negative	13.1	Olive Brown CLAY w/ Sand & Gravel

Corrosivity Analysis

AST

Advance Soil Technology, Inc.

Geological , Geotechnical, Environmental
Consulting & Construction Services

Geotechnical Study | Investigation

Clariana Hotel Addition

An Existing Five-Story Hotel Facility

343 So. Baywood Avenue I San Jose, California

10 South Third Street | San Jose, California

AST File No. 17364-S | Date: July 2018 | Plate No. 29

Appendix "D"

- Plate 30 Liquefaction & Lateral Spreading Analysis with CPT

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Transition layer aglorithm data report 19
Vertical settlements summary report 20

CPT-03 results
Summary data report 21
Transition layer aglorithm summary report 28
Transition layer aglorithm data report 29
Vertical settlements summary report 30

Advance Soil Technology, Inc.
Geotechnical | Environmental Engineers
San Jose, California
http://advancesoil.com

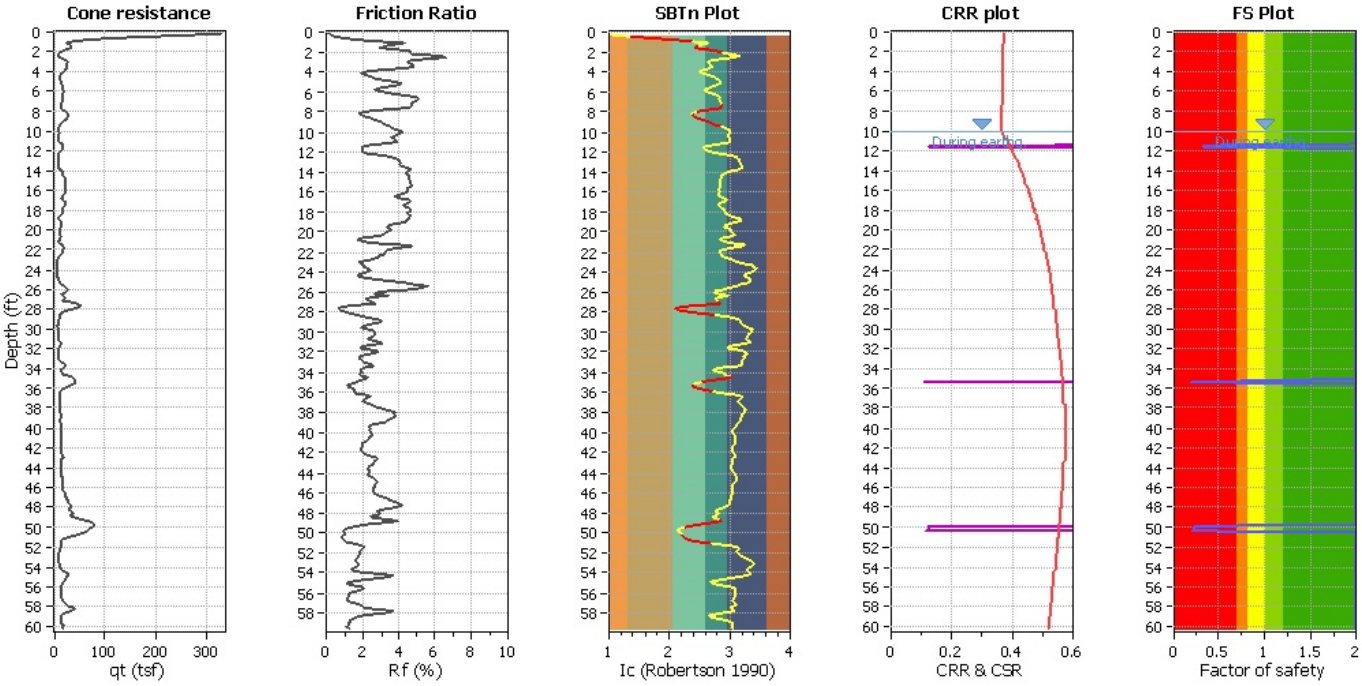
LIQUEFACTION ANALYSIS REPORT

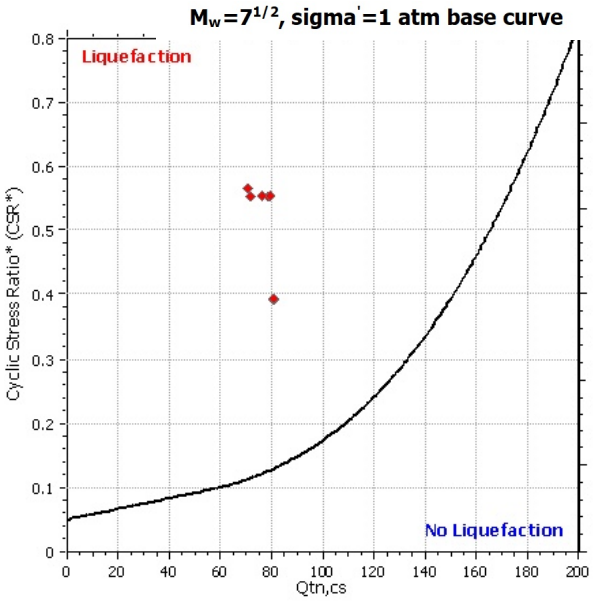
Project title : Clariana Hotel **Location : 100 E. Santa Clara Street, San Jose, CA**

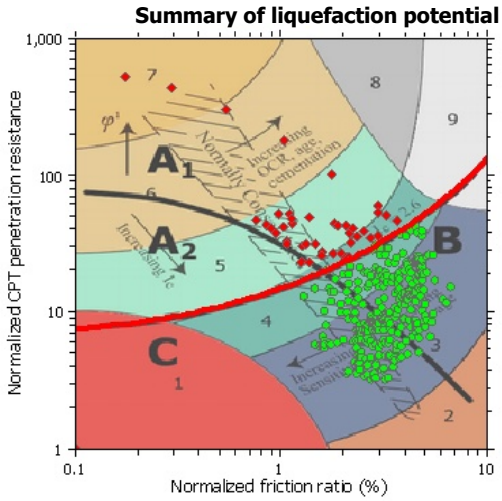
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	14.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

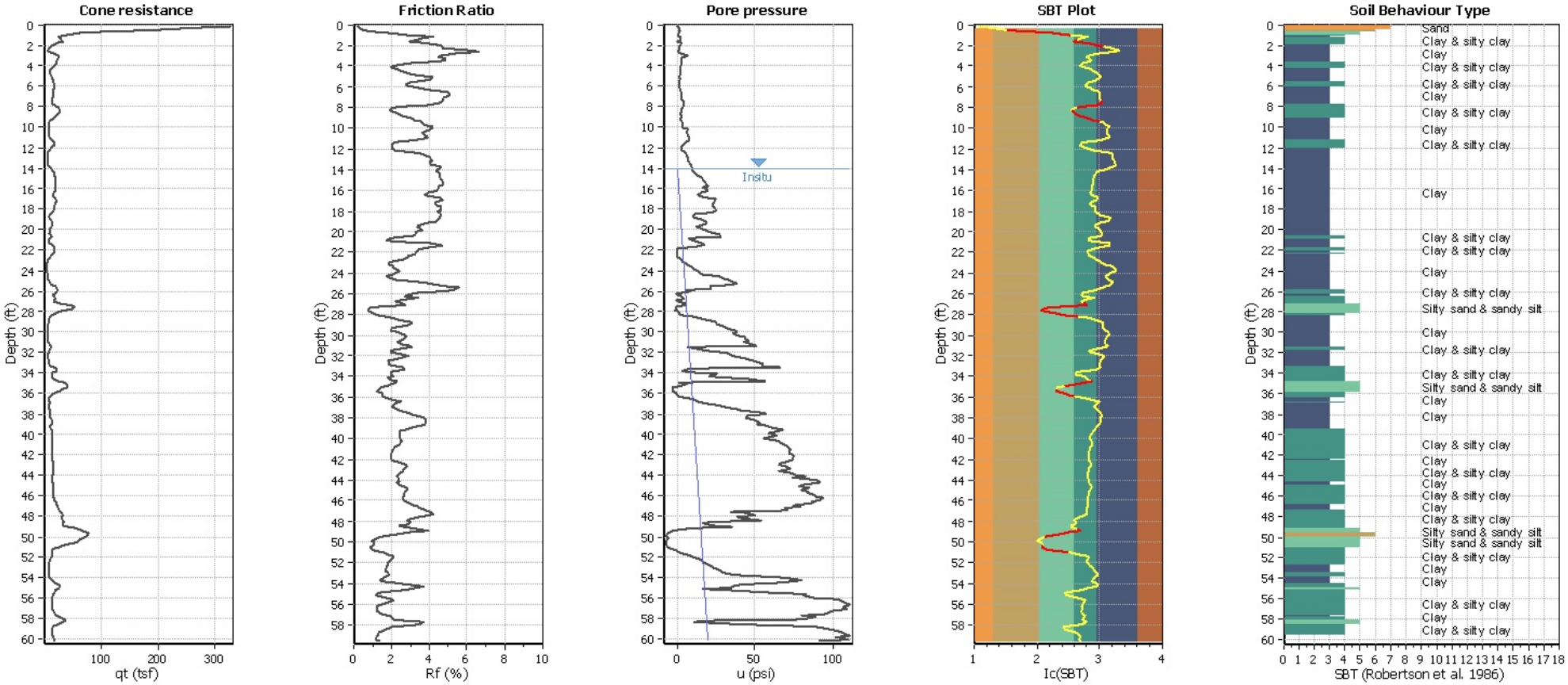






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plo



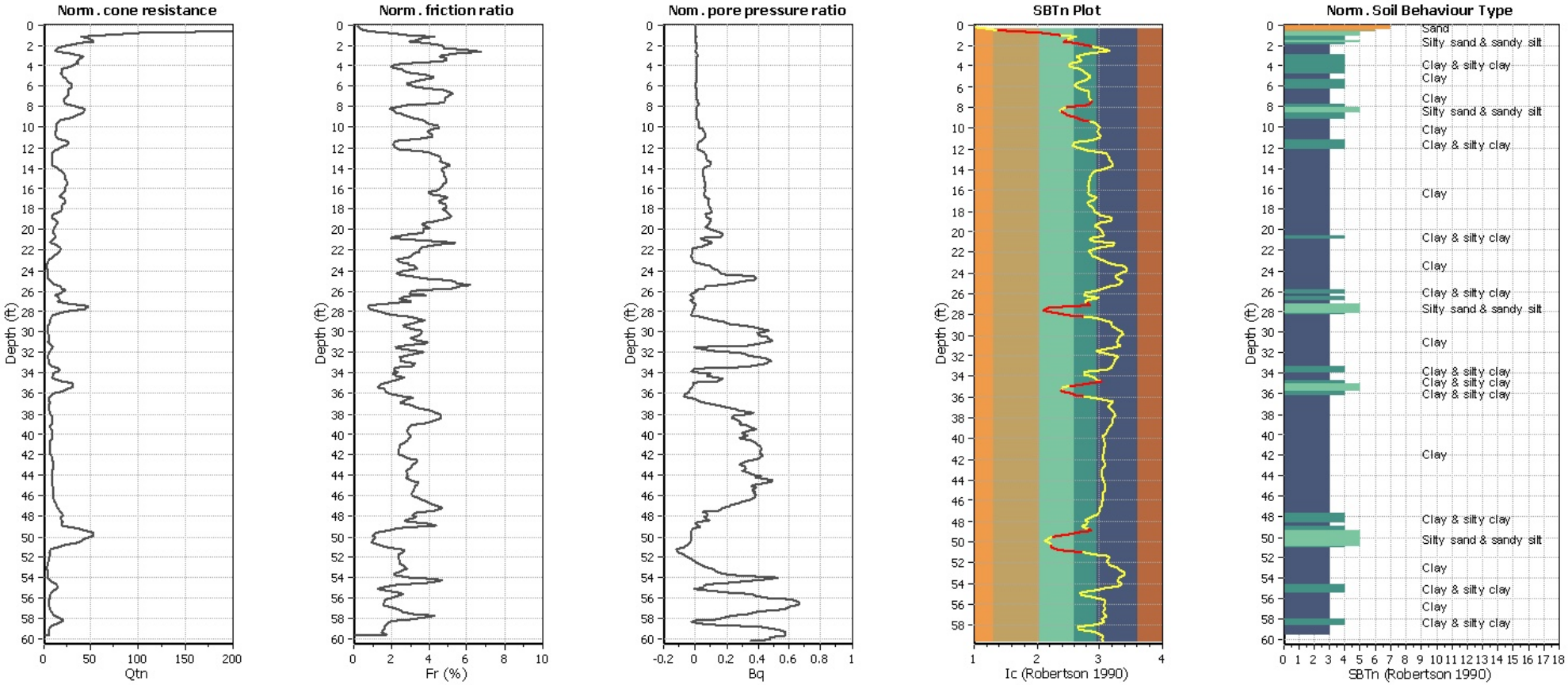
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normaliz



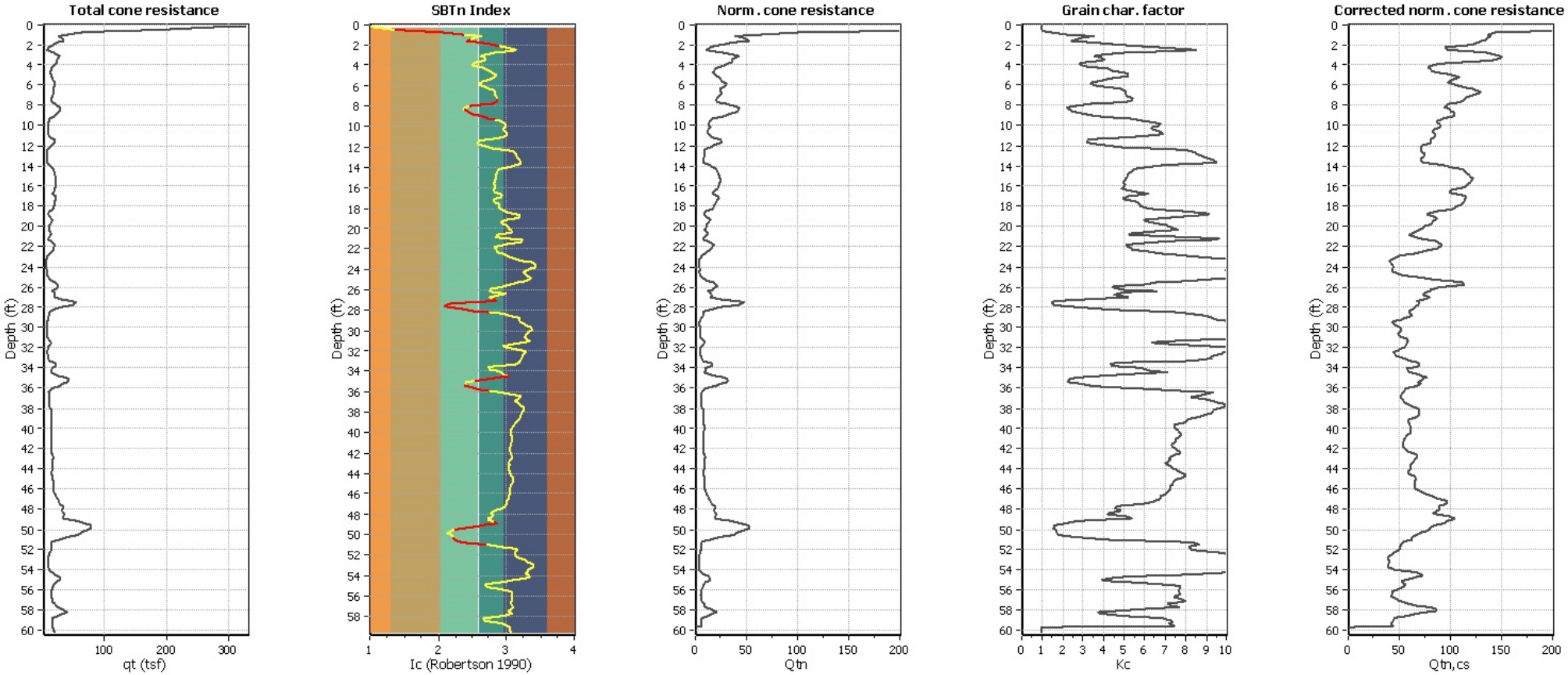
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty 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

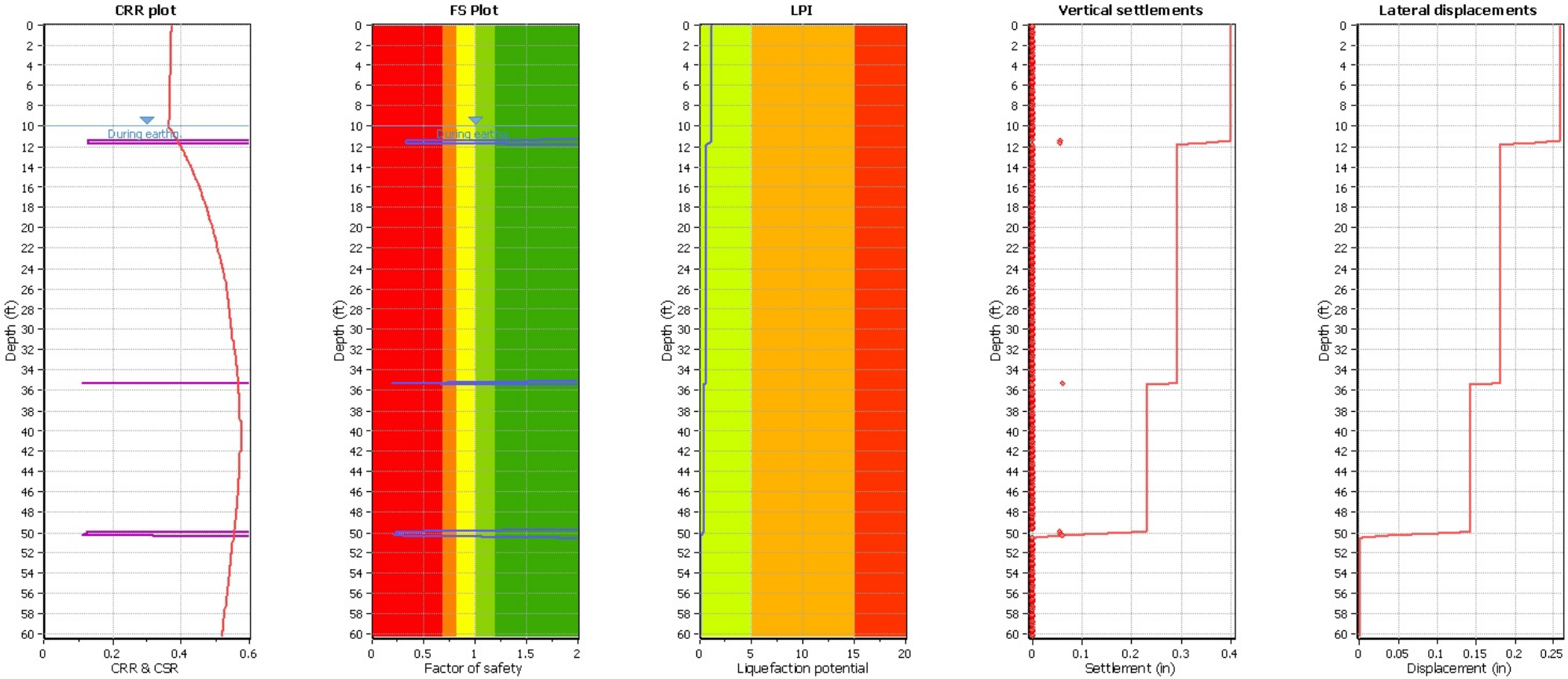
Liquefaction analysis overall plots (intermediate resu



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

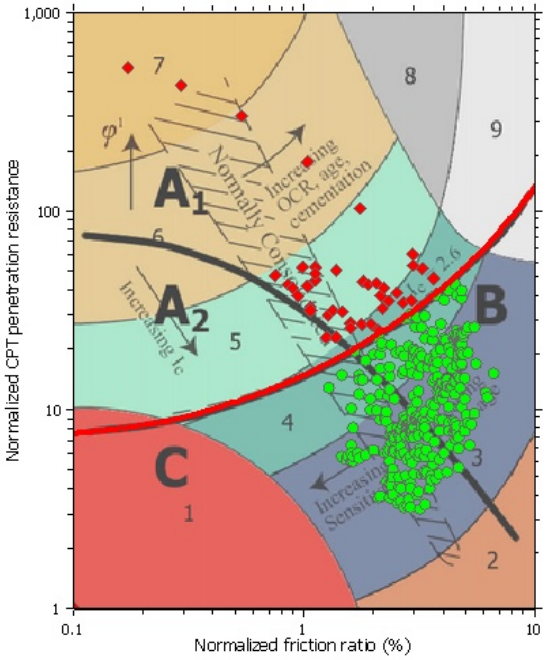
F.S. color scheme

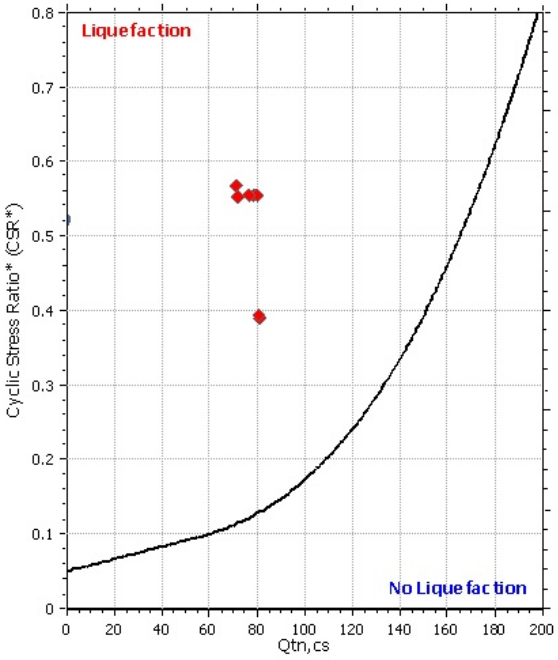
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

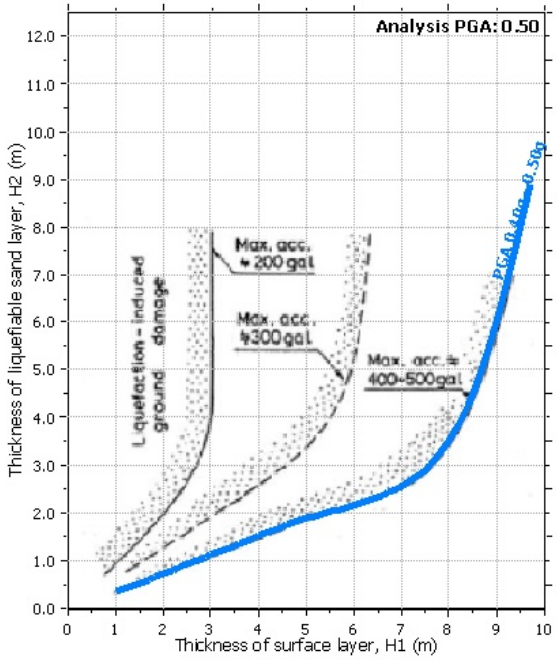
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo



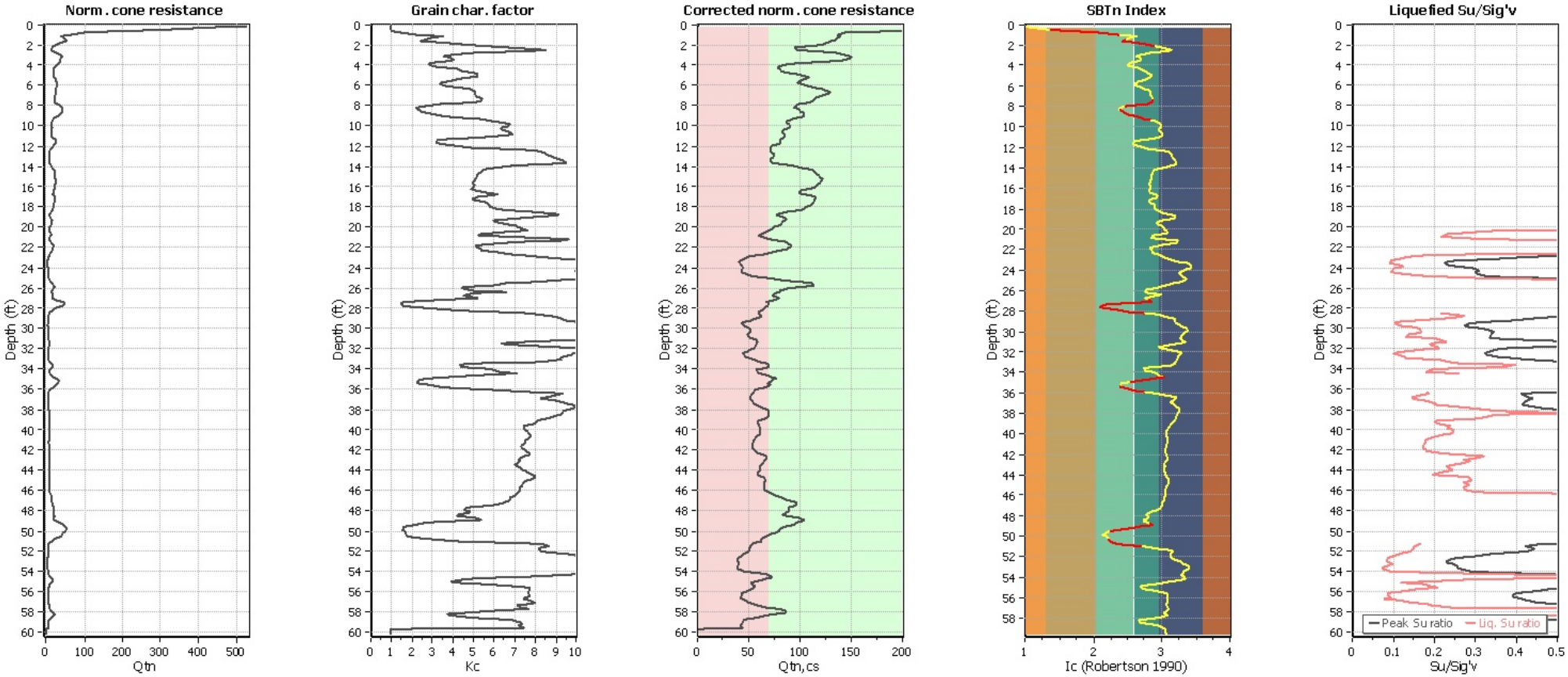




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

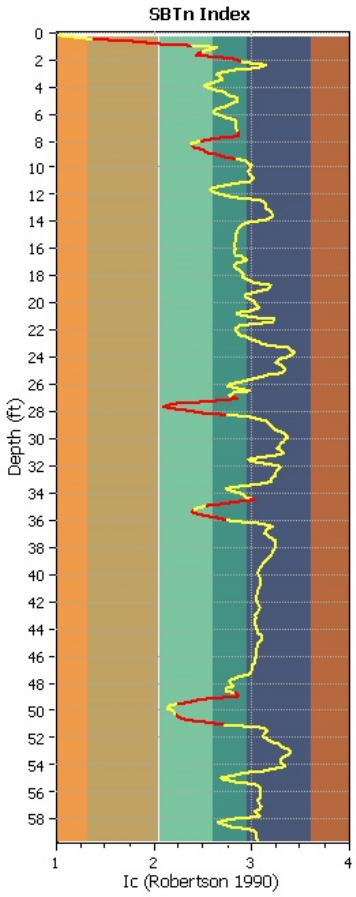
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

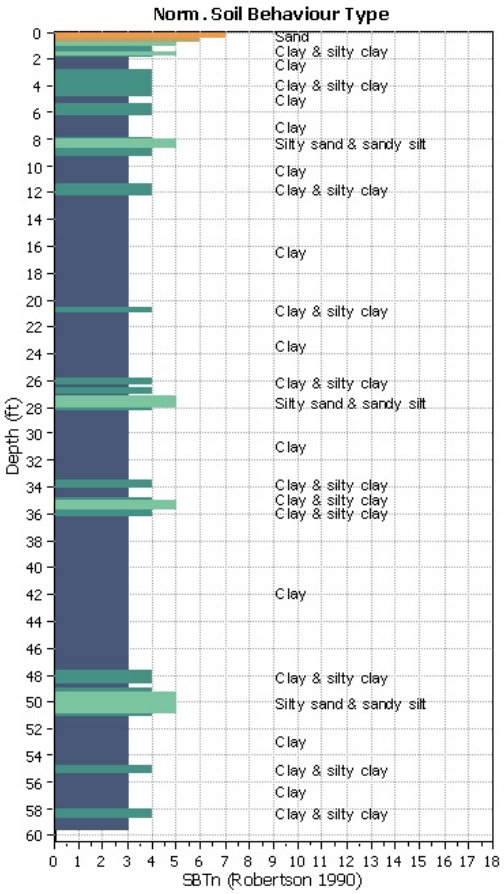
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.



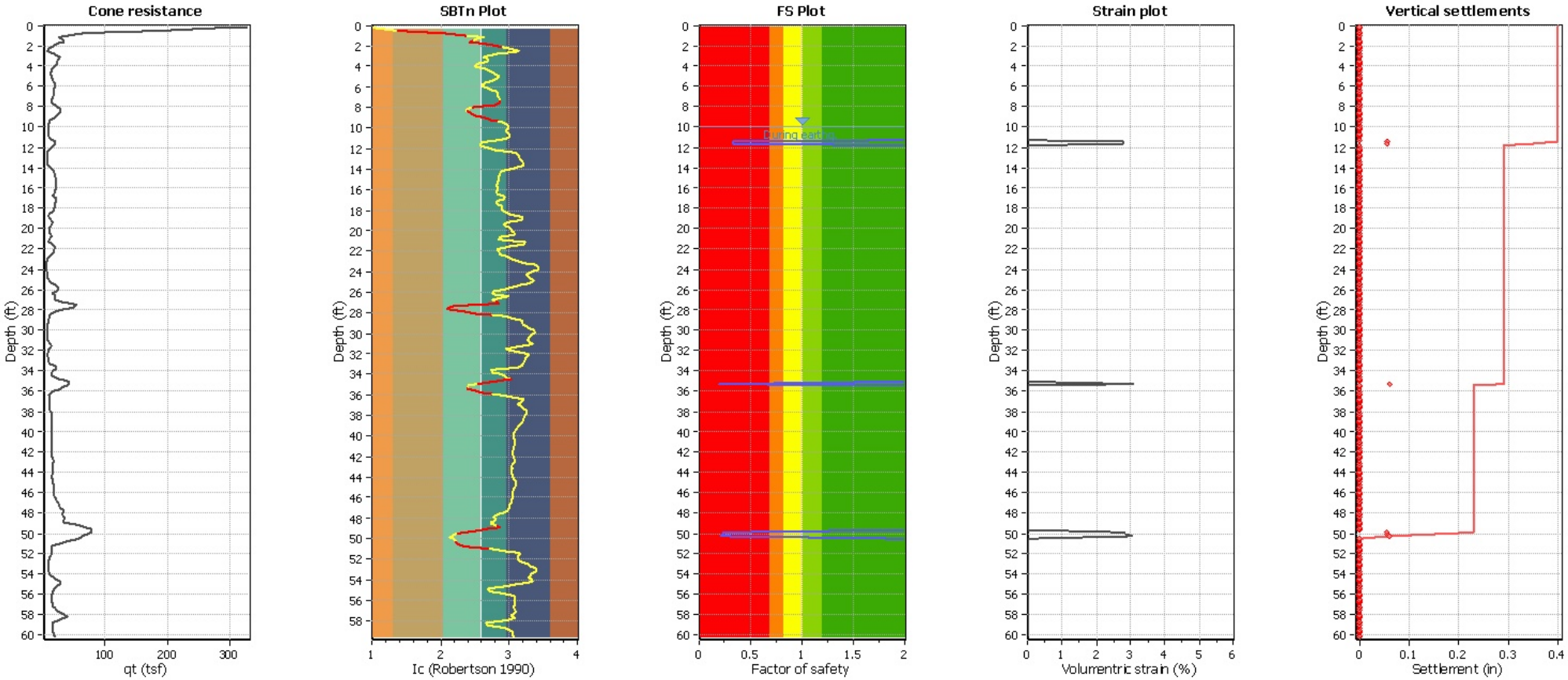


Transition layer algorithm properties		General statistics	
I_c minimum check value:	1.70	Total points in CPT file:	367
I_c maximum check value:	3.00	Total points excluded:	51
I_c change ratio value:	0.0250	Exclusion percentage:	13.90%
Minimum number of points in layer:	4	Number of layers detected:	10

Transition layer No	Number of points	Depth	SBT_n number	SBT_n description
Transition layer 1	4	Start depth: 0.66 (ft)	6	Sand & silty sand
		End depth: 1.15 (ft)	4	Clay & silty clay
Transition layer 2	5	Start depth: 1.64 (ft)	5	Silty sand & sandy silt
		End depth: 2.30 (ft)	3	Clay
Transition layer 3	5	Start depth: 7.55 (ft)	3	Clay
		End depth: 8.20 (ft)	5	Silty sand & sandy silt
Transition layer 4	7	Start depth: 8.53 (ft)	5	Silty sand & sandy silt
		End depth: 9.51 (ft)	3	Clay
Transition layer 5	4	Start depth: 27.07 (ft)	3	Clay
		End depth: 27.56 (ft)	5	Silty sand & sandy silt
Transition layer 6	6	Start depth: 27.56 (ft)	5	Silty sand & sandy silt
		End depth: 28.38 (ft)	3	Clay
Transition layer 7	4	Start depth: 34.61 (ft)	3	Clay
		End depth: 35.10 (ft)	5	Silty sand & sandy silt
Transition layer 8	5	Start depth: 35.43 (ft)	5	Silty sand & sandy silt
		End depth: 36.09 (ft)	4	Clay & silty clay
Transition layer 9	6	Start depth: 48.88 (ft)	3	Clay
		End depth: 49.70 (ft)	5	Silty sand & sandy silt
Transition layer 10	5	Start depth: 50.52 (ft)	5	Silty sand & sandy silt
		End depth: 51.18 (ft)	3	Clay

Start depth: Depth where the transition layer begins
End depth: Depth where the transition layer ends

Estimation of post-earthquake settlements



Abbreviations

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

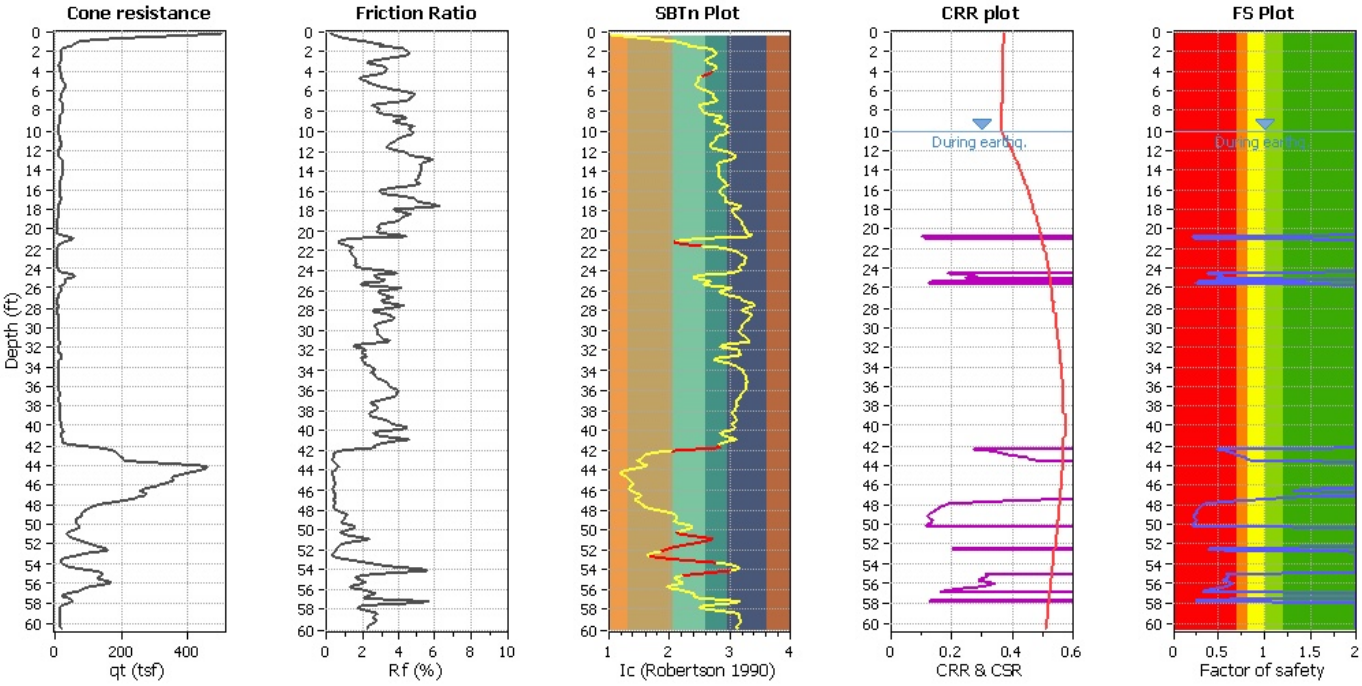
LIQUEFACTION ANALYSIS REPORT

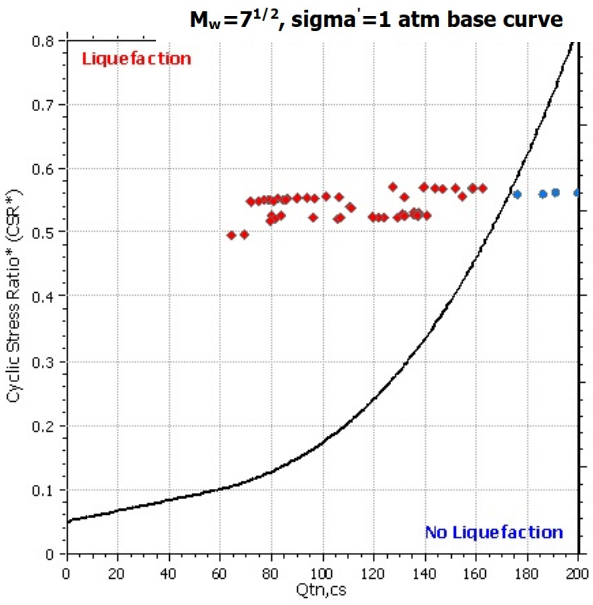
Project title : Clariana Hotel **Location : 100 E. Santa Clara Street, San Jose, CA**

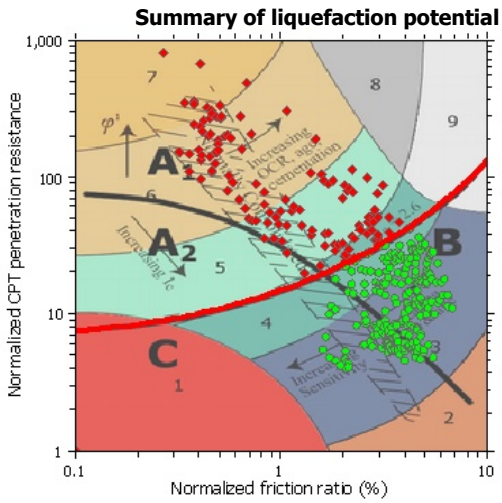
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	14.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

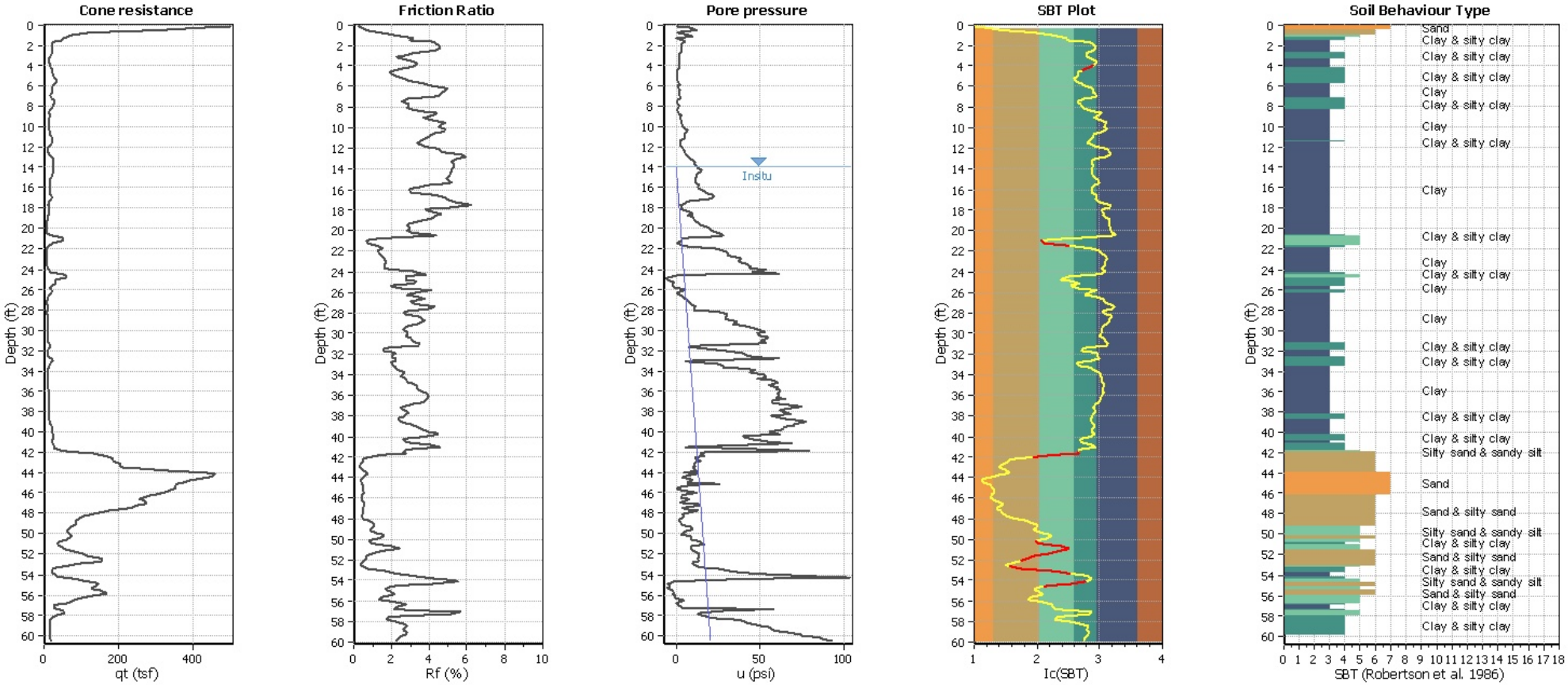






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plo



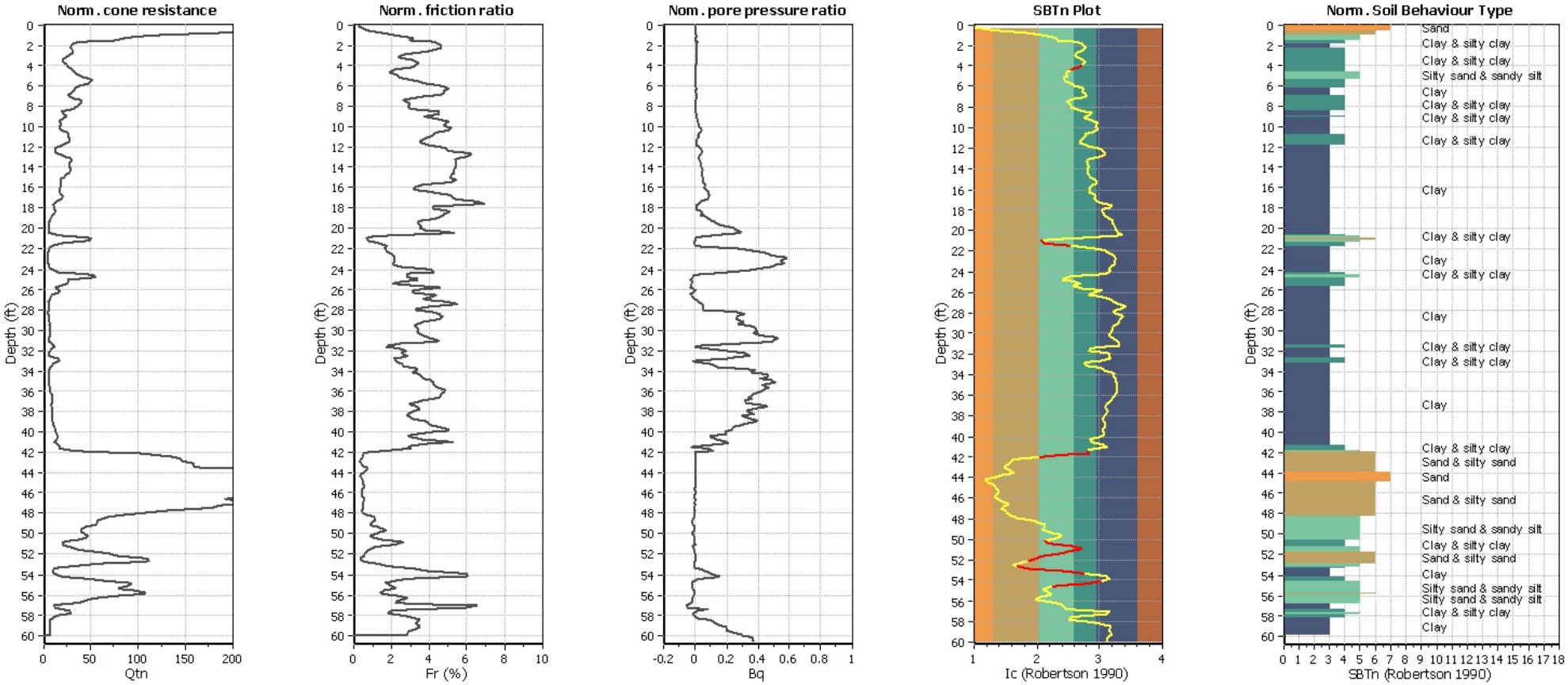
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normaliz



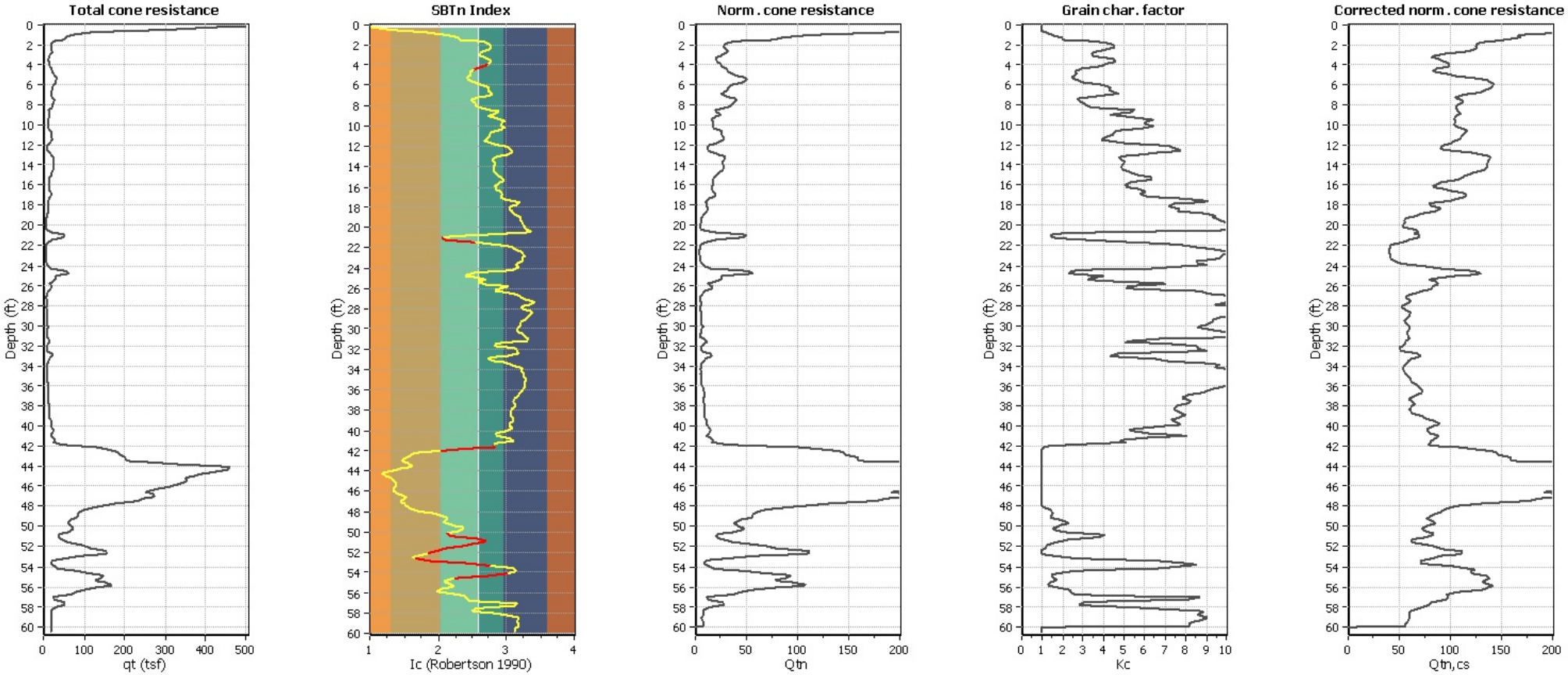
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

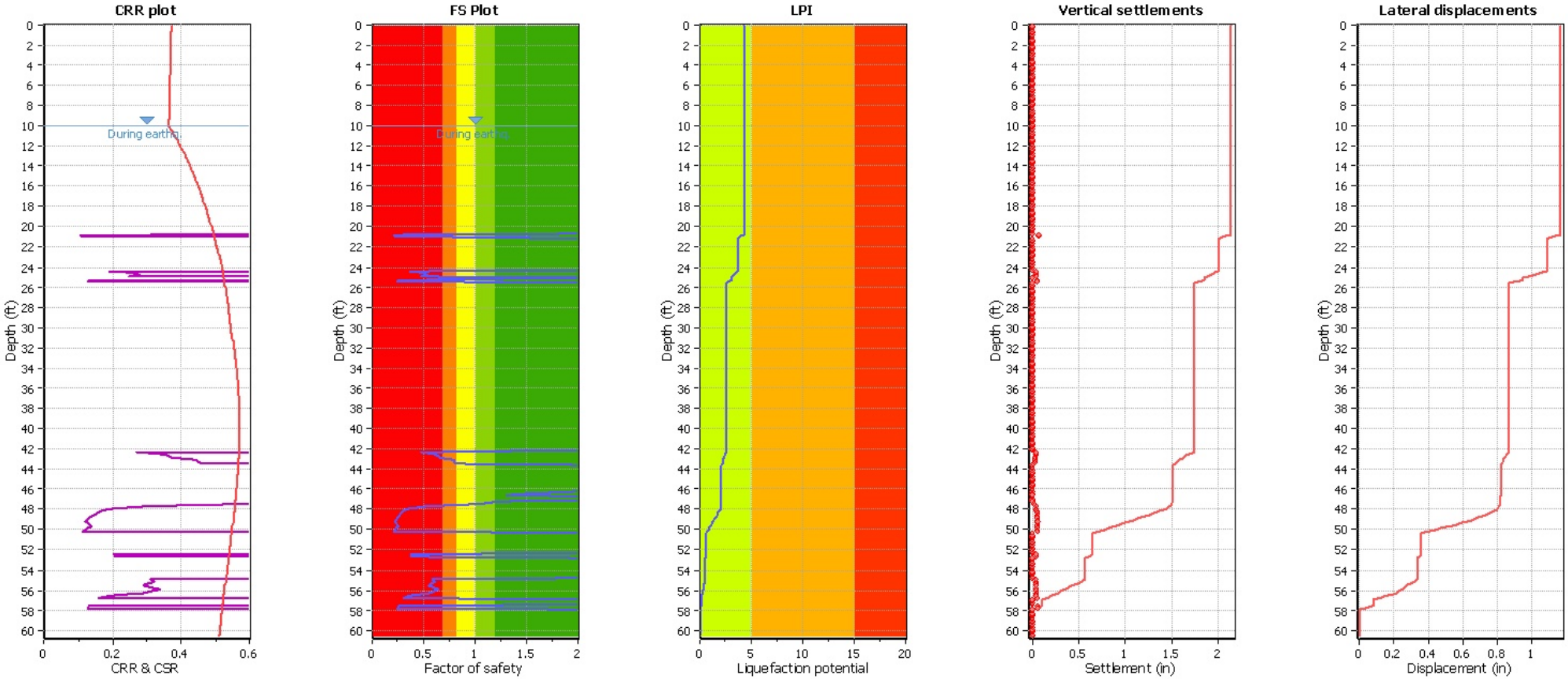
Liquefaction analysis overall plots (intermediate resu



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

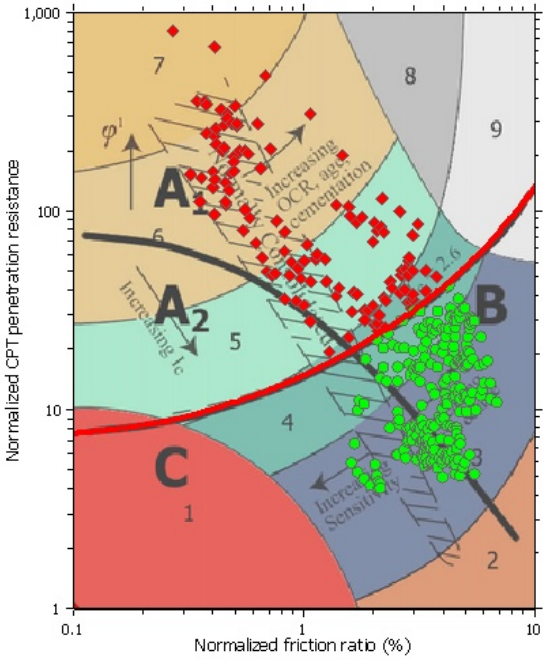
F.S. color scheme

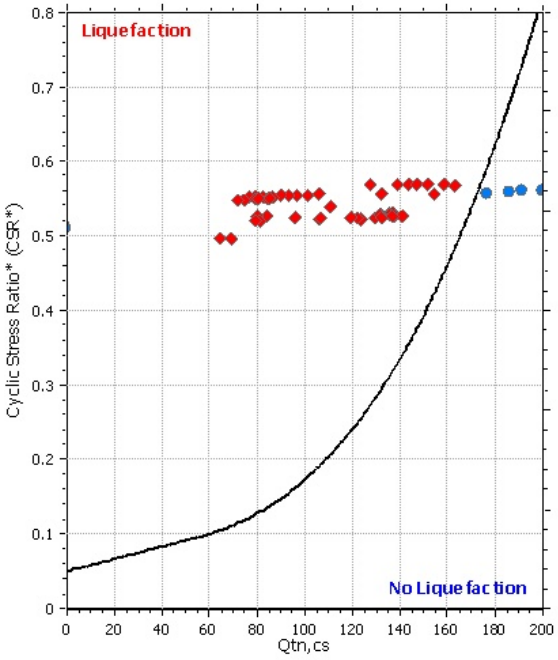
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

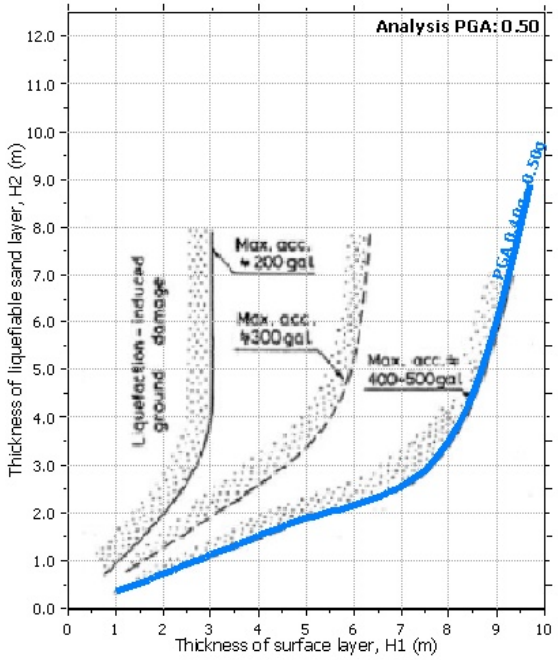
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo





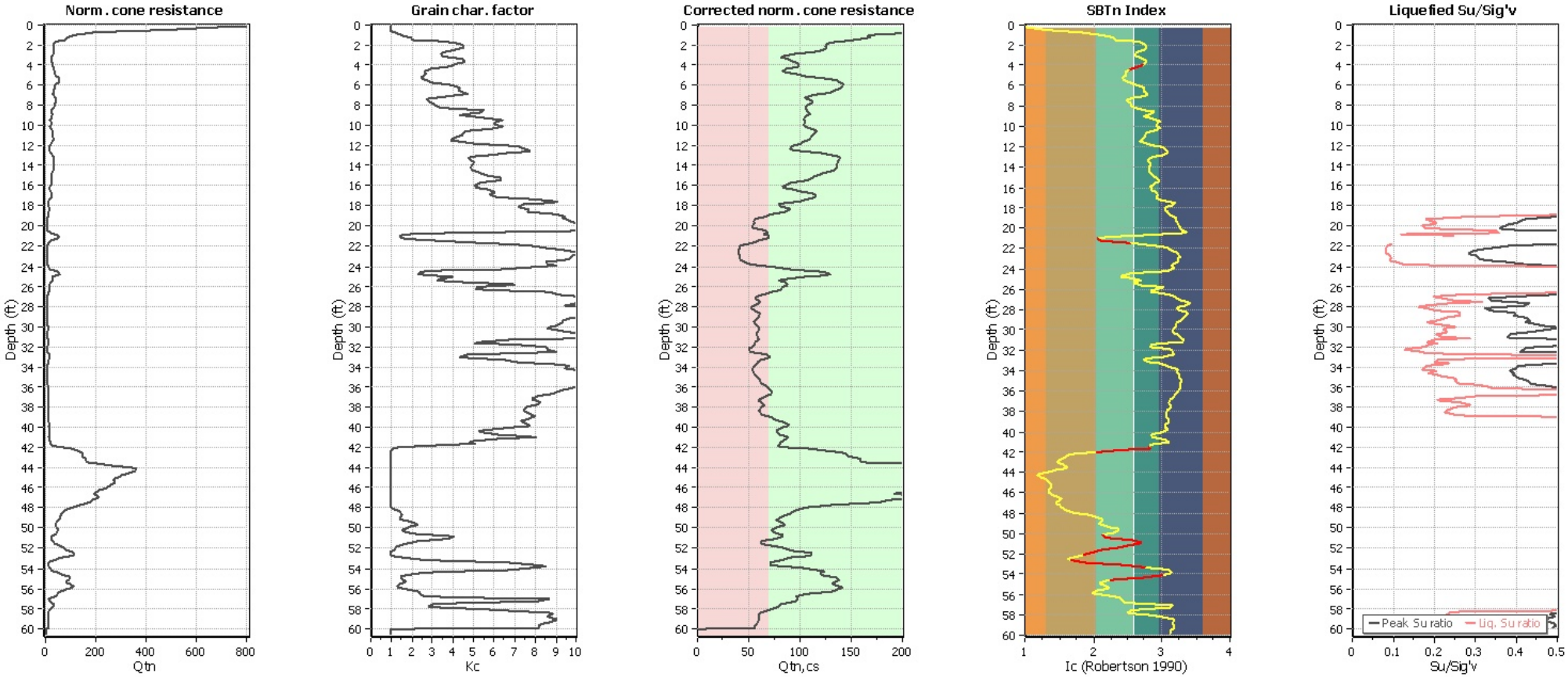


CPT-02 (4

Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

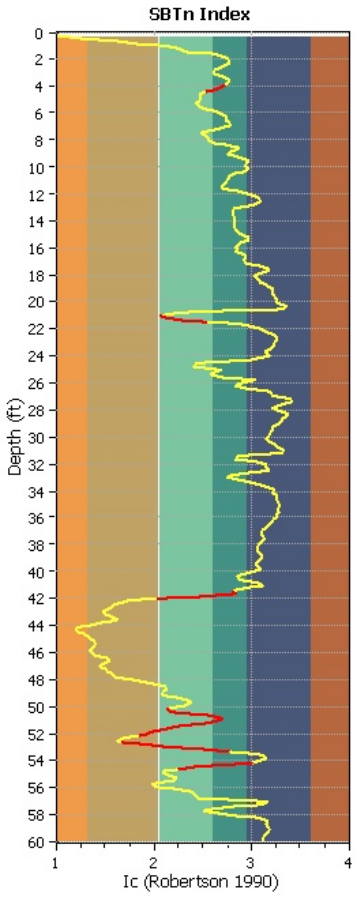
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

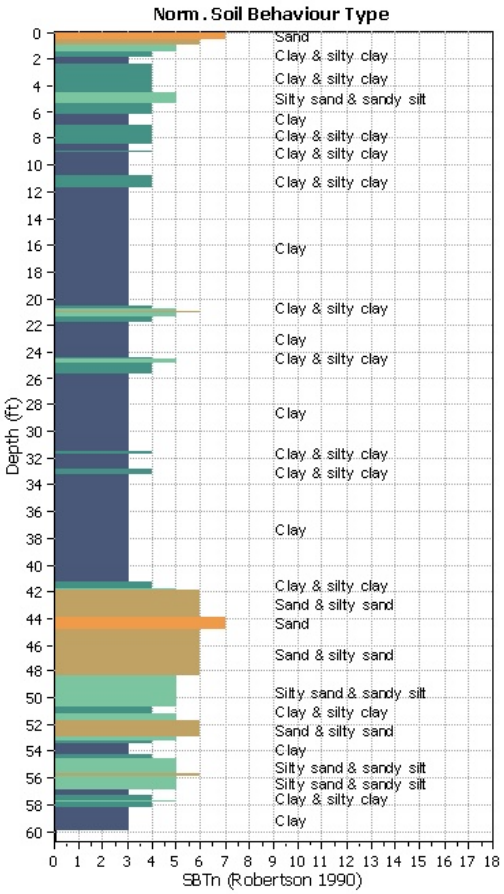
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.



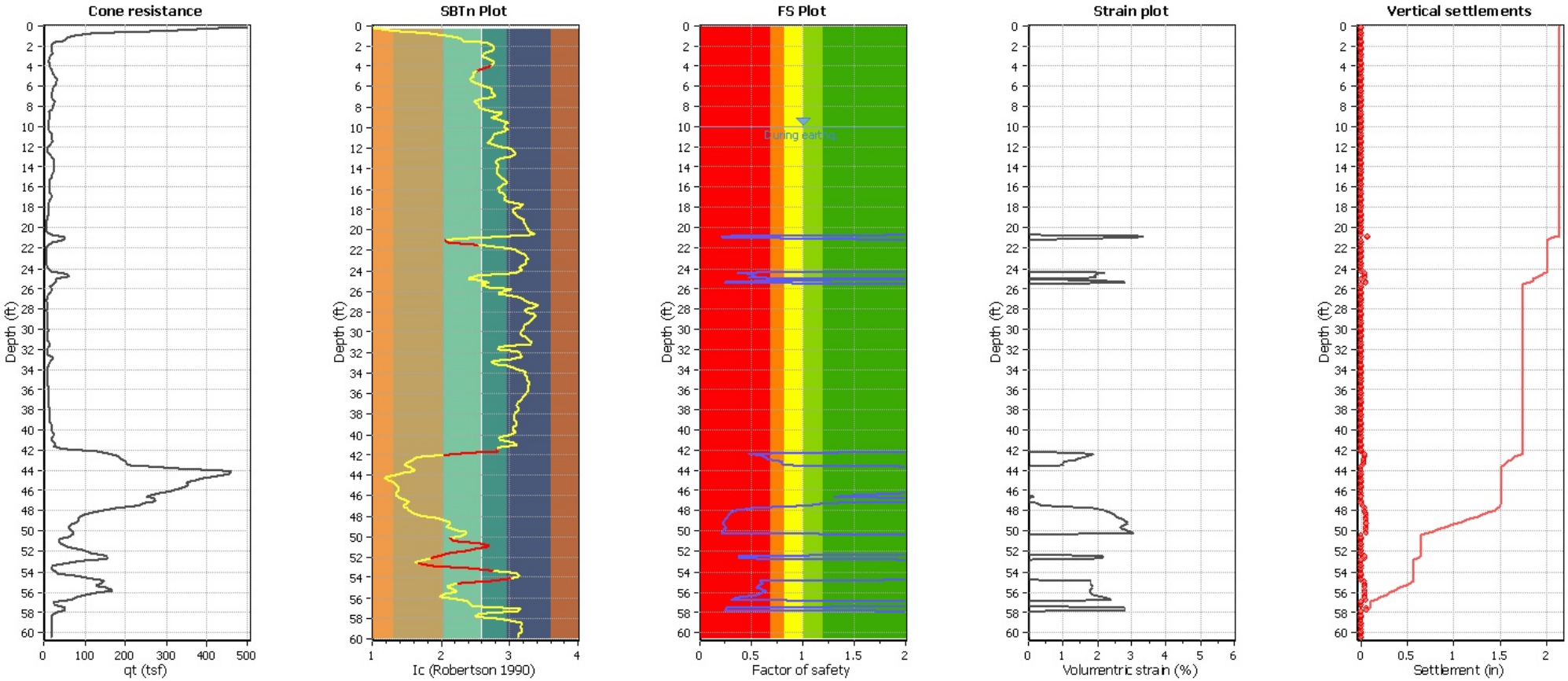


Transition layer algorithm properties		General statistics	
I_c minimum check value:	1.70	Total points in CPT file:	369
I_c maximum check value:	3.00	Total points excluded:	34
I_c change ratio value:	0.0250	Exclusion percentage:	9.21%
Minimum number of points in layer:	4	Number of layers detected:	7

Transition layer No	Number of points	Depth	SBT_n number	SBT_n description
Transition layer 1	4	Start depth: 4.10 (ft)	4	Clay & silty clay
		End depth: 4.59 (ft)	5	Silty sand & sandy silt
Transition layer 2	4	Start depth: 21.16 (ft)	5	Silty sand & sandy silt
		End depth: 21.65 (ft)	4	Clay & silty clay
Transition layer 3	4	Start depth: 41.67 (ft)	4	Clay & silty clay
		End depth: 42.16 (ft)	6	Sand & silty sand
Transition layer 4	4	Start depth: 50.36 (ft)	5	Silty sand & sandy silt
		End depth: 50.85 (ft)	4	Clay & silty clay
Transition layer 5	9	Start depth: 51.02 (ft)	4	Clay & silty clay
		End depth: 52.33 (ft)	6	Sand & silty sand
Transition layer 6	5	Start depth: 52.82 (ft)	6	Sand & silty sand
		End depth: 53.48 (ft)	3	Clay
Transition layer 7	4	Start depth: 54.30 (ft)	4	Clay & silty clay
		End depth: 54.79 (ft)	5	Silty sand & sandy silt

Start depth: Depth where the transition layer begins
End depth: Depth where the transition layer ends

Estimation of post-earthquake settlements



Abbreviations

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

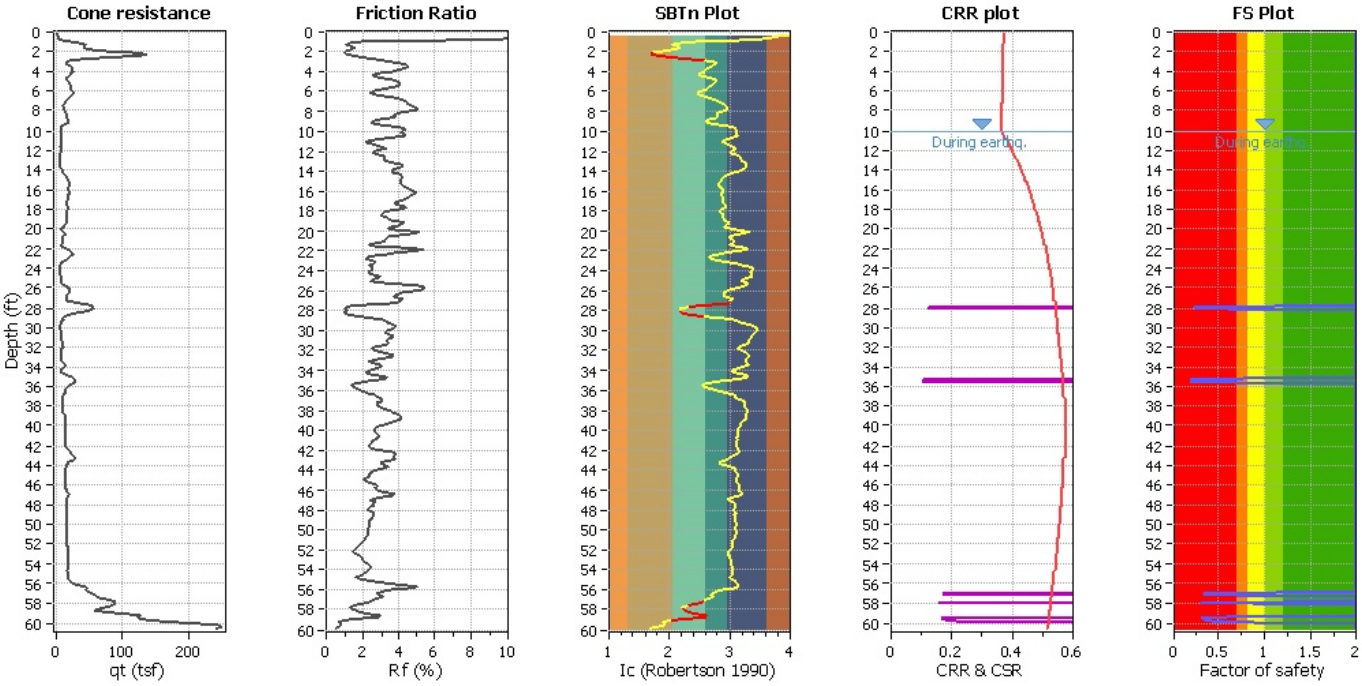
LIQUEFACTION ANALYSIS REPORT

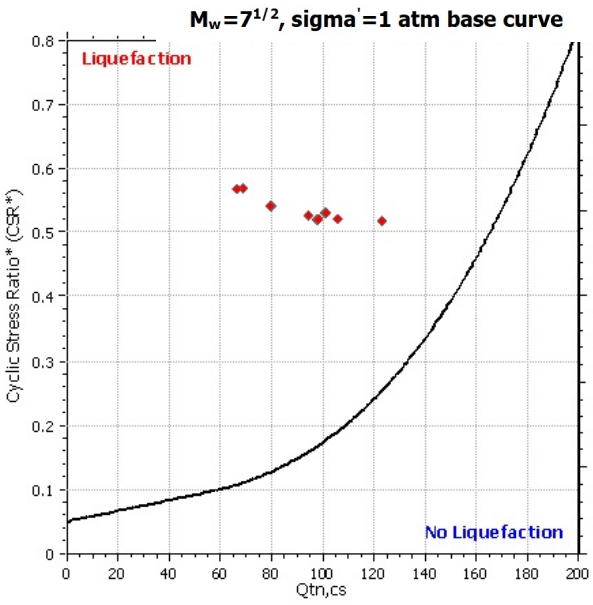
Project title : Clariana Hotel **Location : 100 E. Santa Clara Street, San Jose, CA**

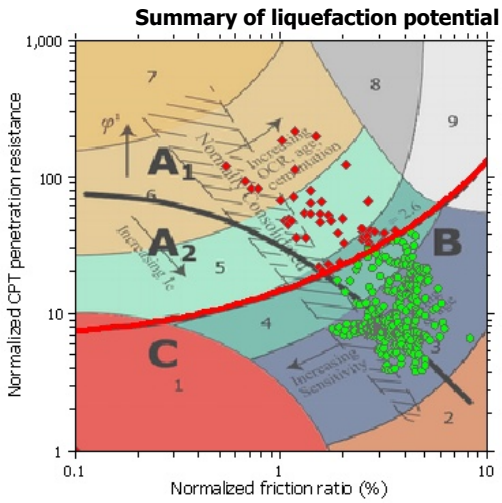
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	14.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

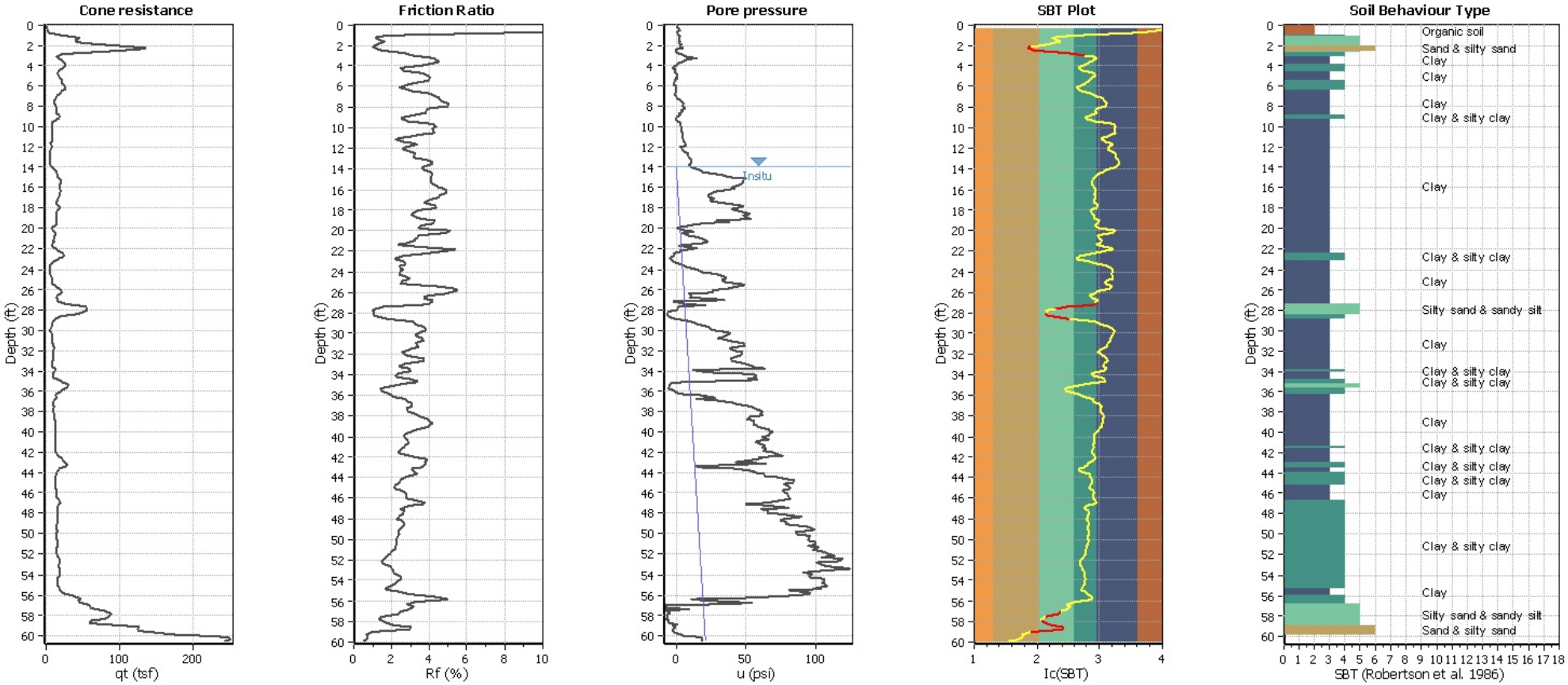






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plo



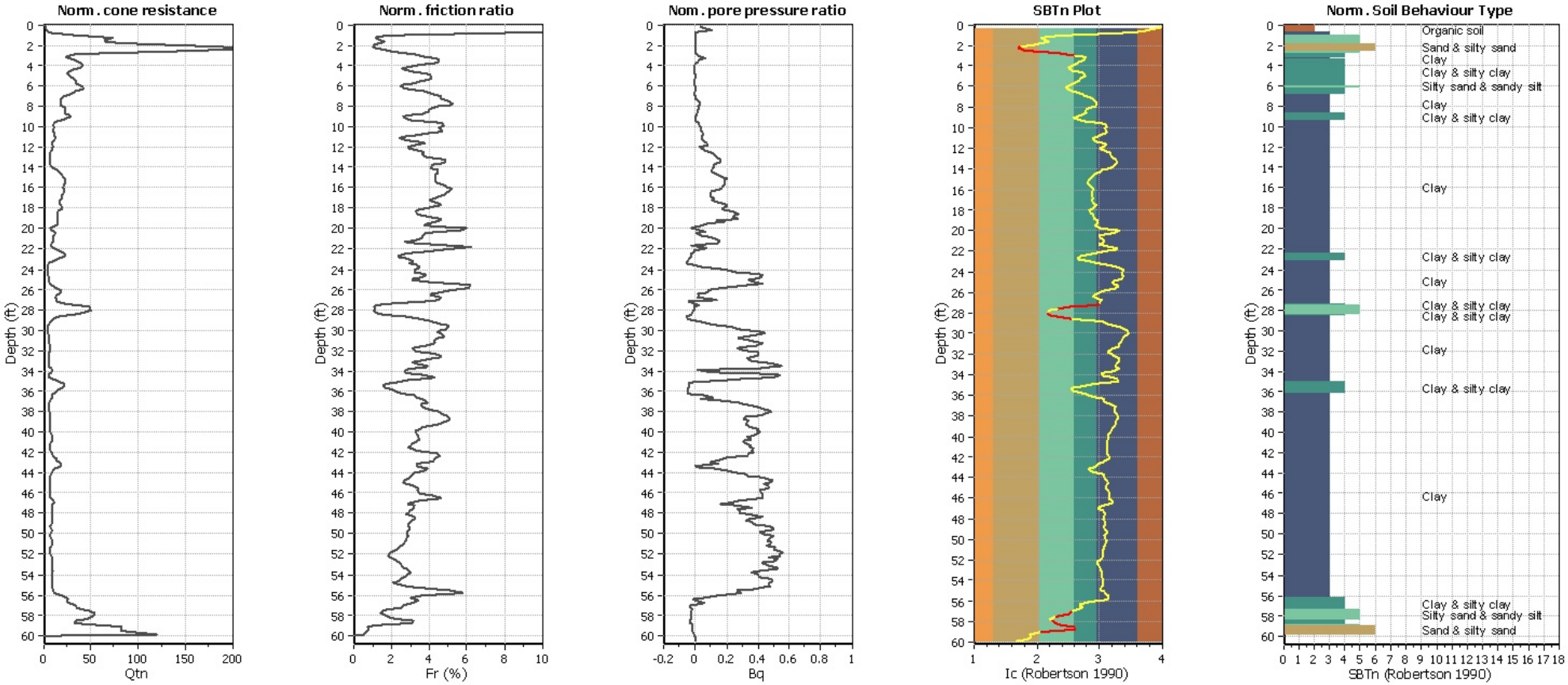
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normaliz



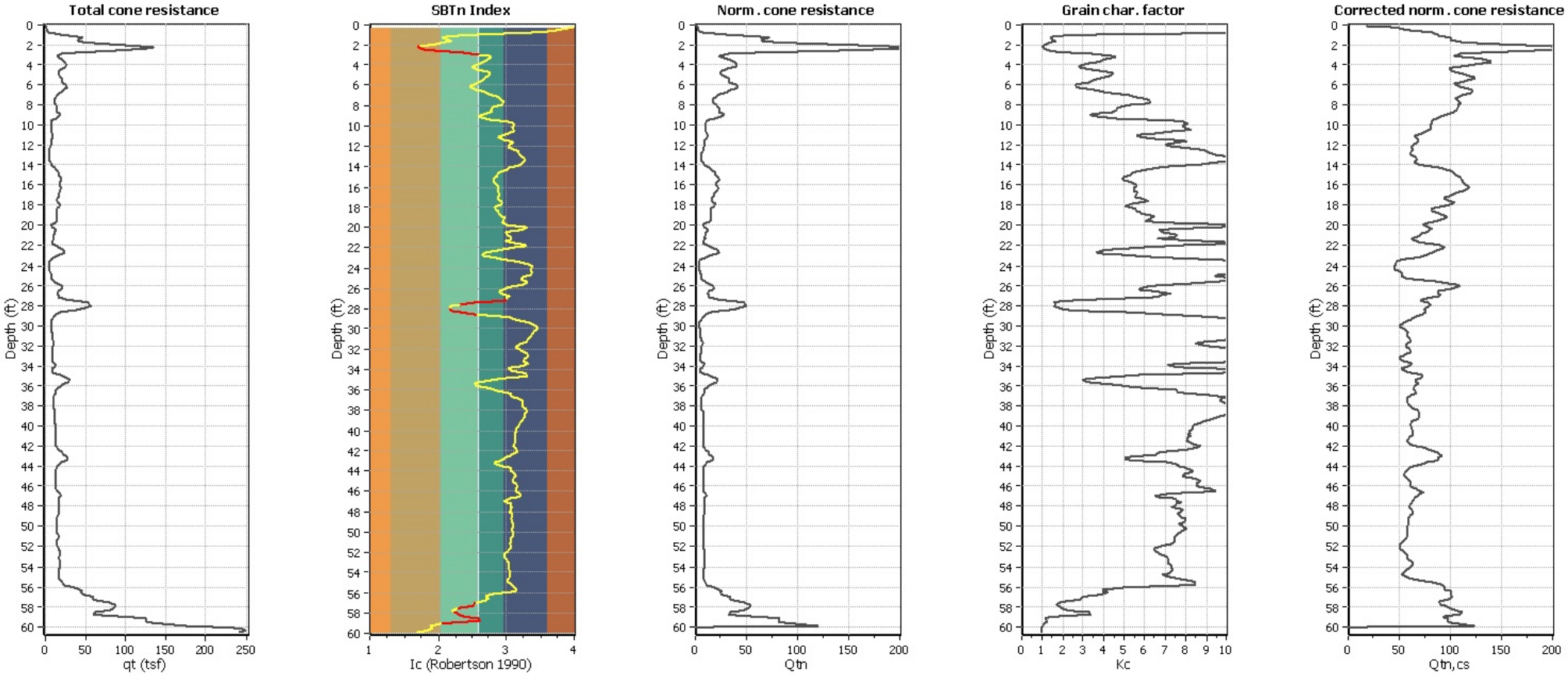
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

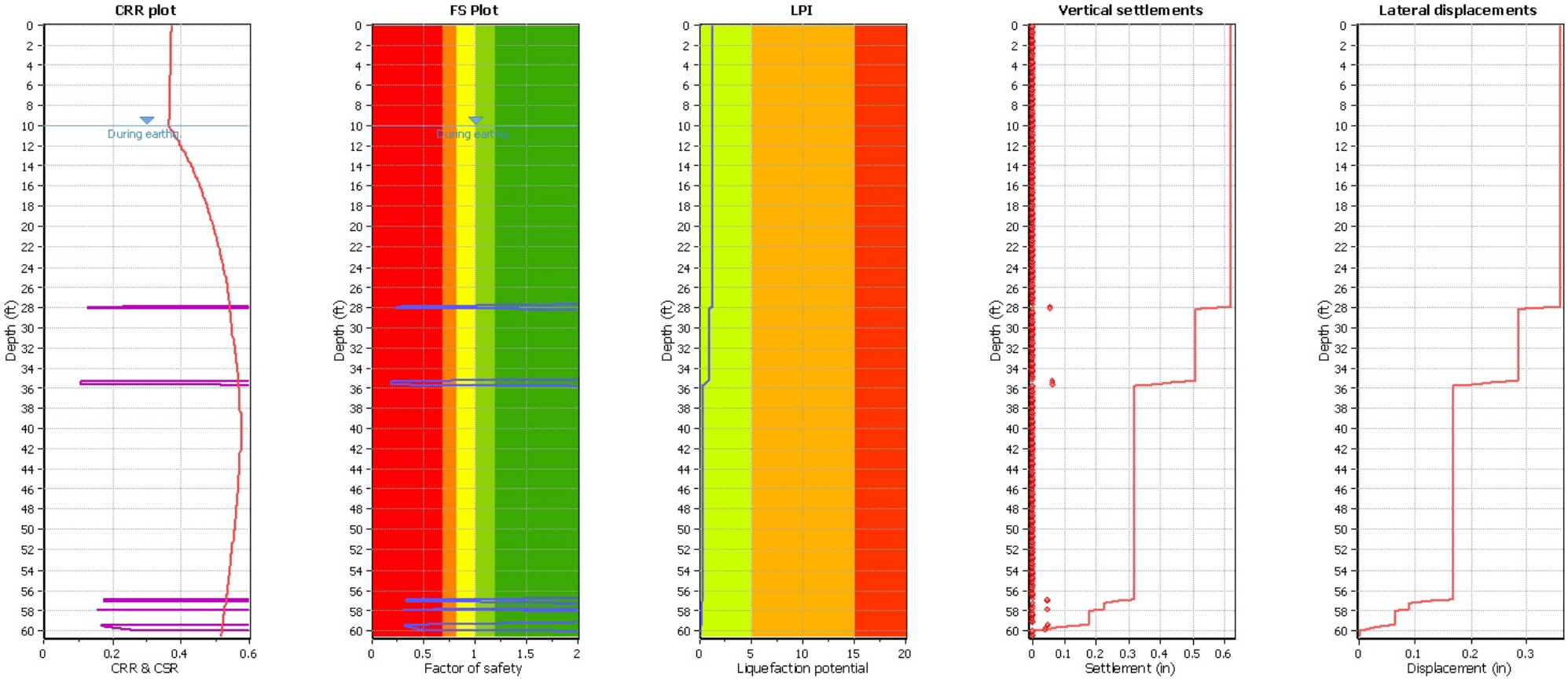
Liquefaction analysis overall plots (intermediate resu



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

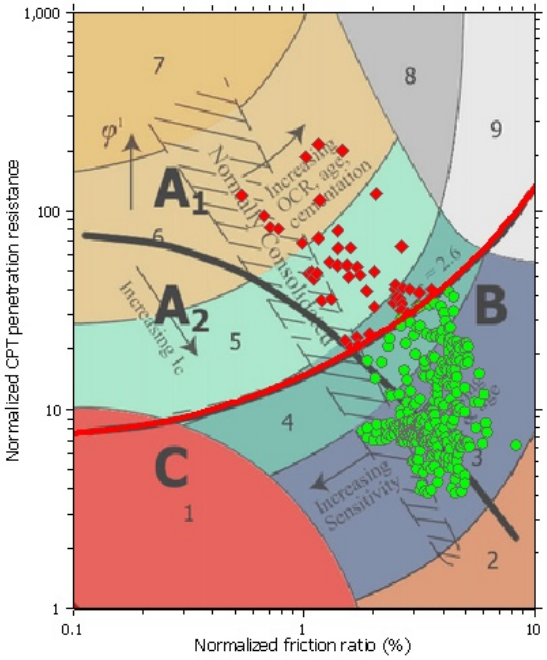
F.S. color scheme

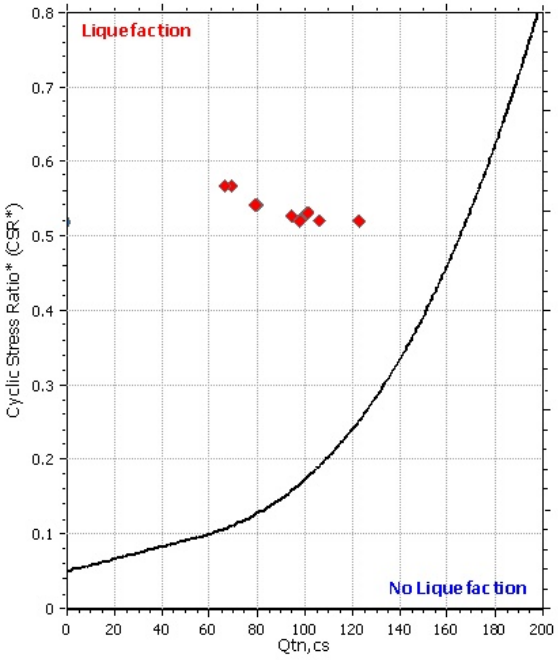
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

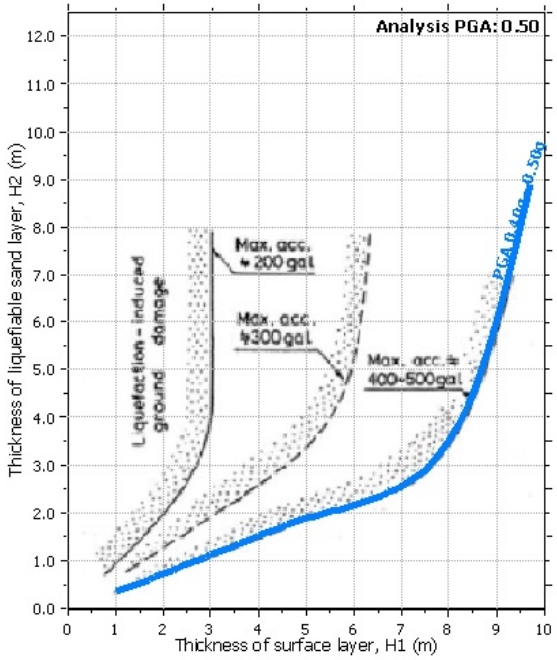
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo



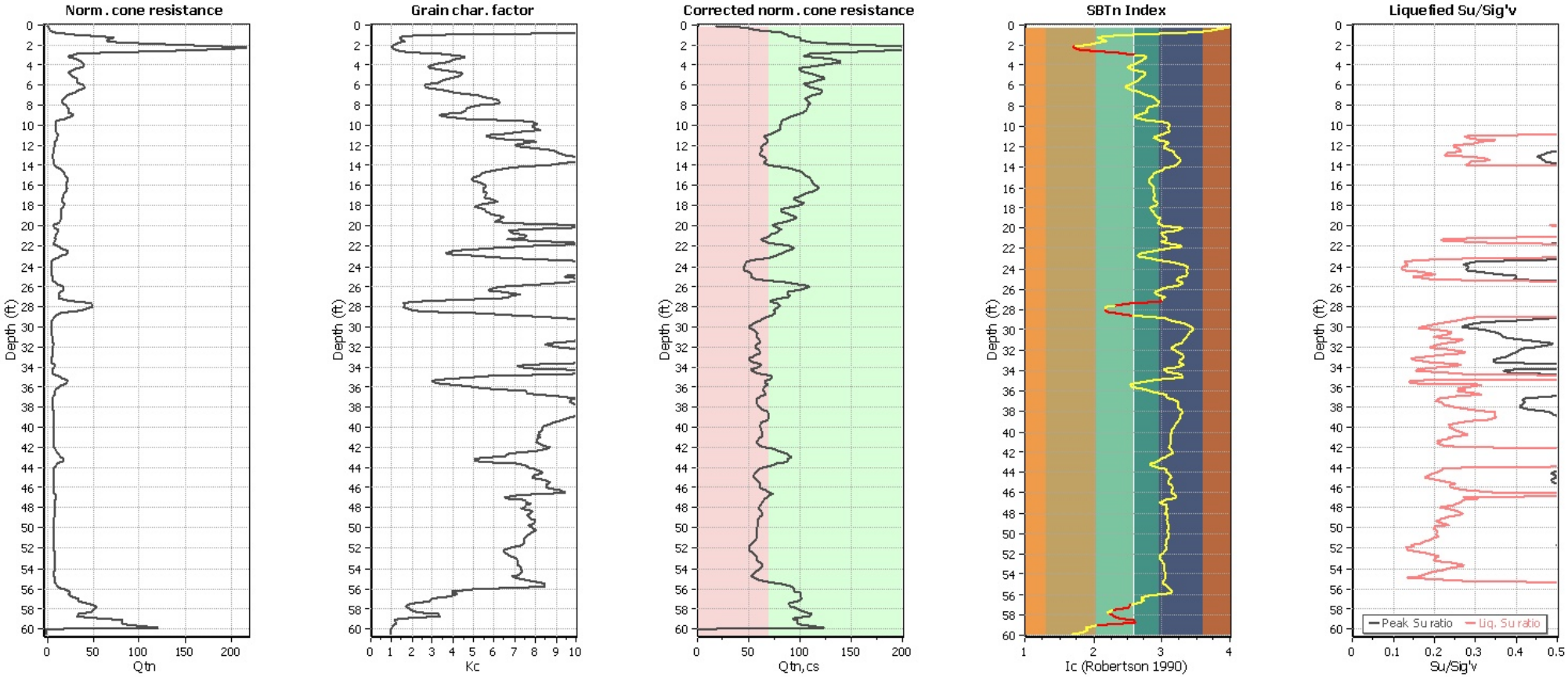




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 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:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

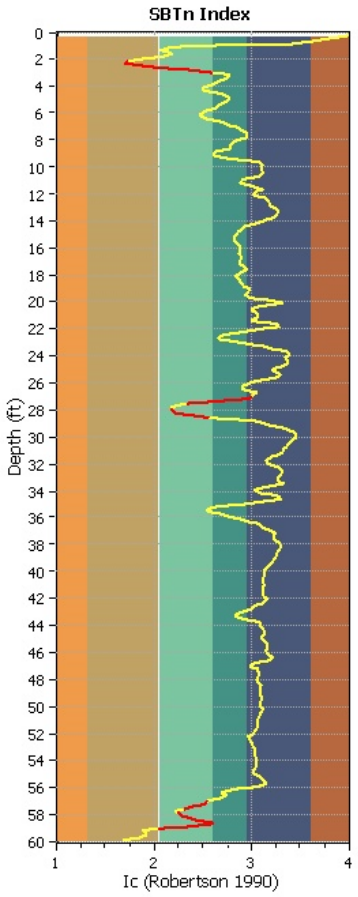
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

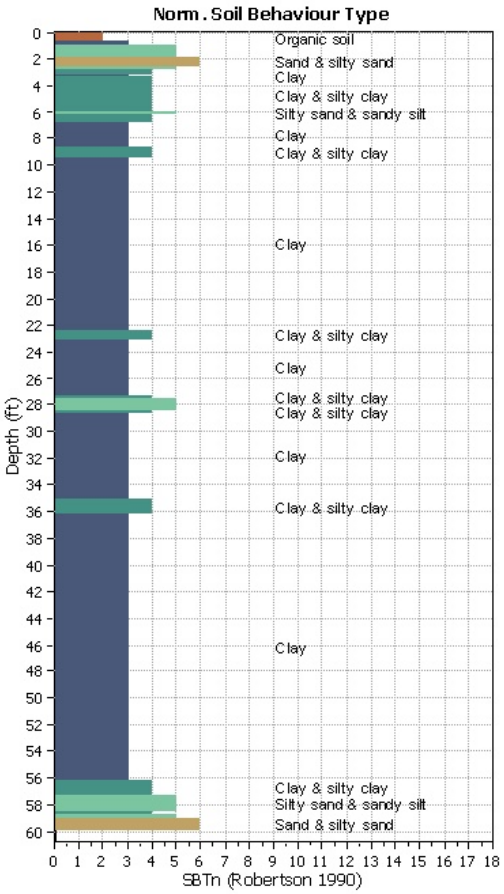
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.



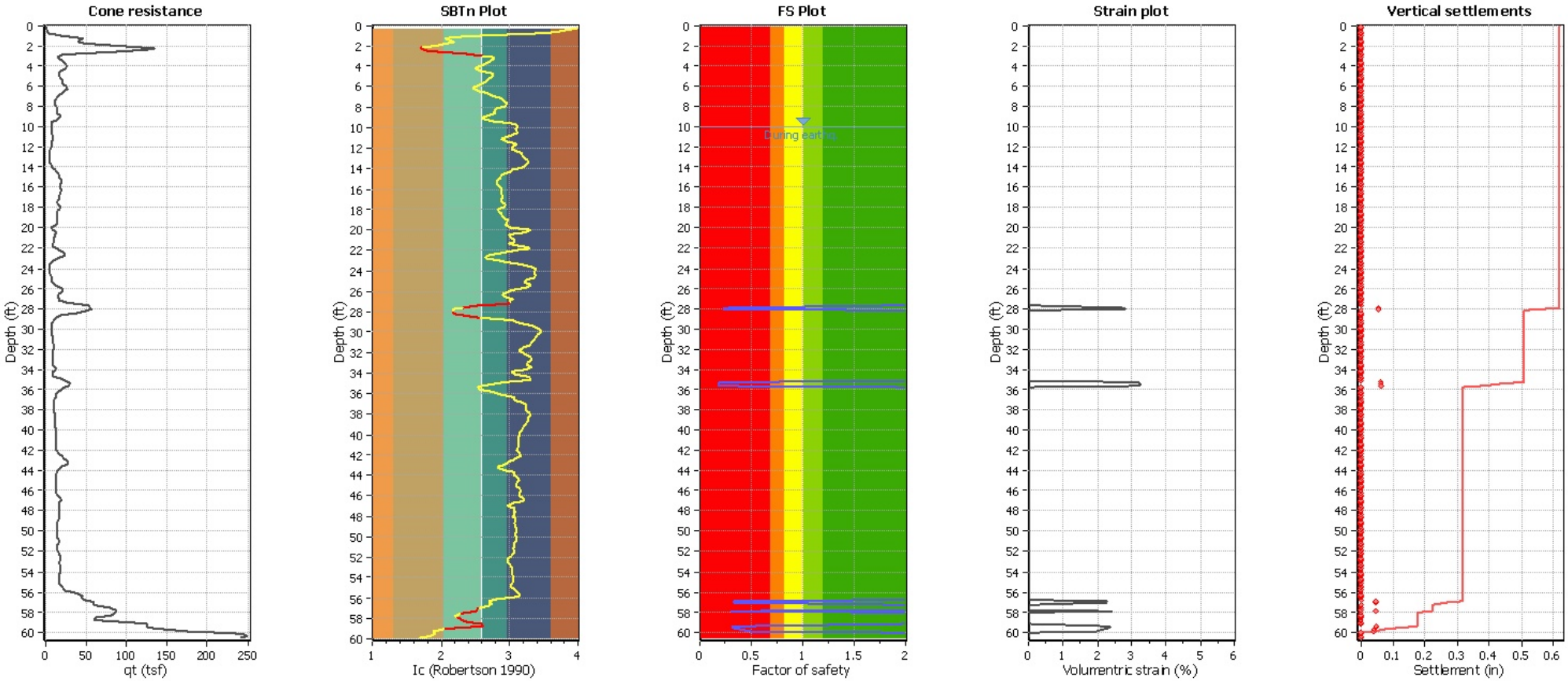


Transition layer algorithm properties		General statistics	
I_c minimum check value:	1.70	Total points in CPT file:	369
I_c maximum check value:	3.00	Total points excluded:	26
I_c change ratio value:	0.0250	Exclusion percentage:	7.05%
Minimum number of points in layer:	4	Number of layers detected:	6

Transition layer No	Number of points	Depth	SBT_n number	SBT_n description
Transition layer 1	6	Start depth: 2.30 (ft)	6	Sand & silty sand
		End depth: 3.12 (ft)	4	Clay & silty clay
Transition layer 2	4	Start depth: 27.23 (ft)	3	Clay
		End depth: 27.72 (ft)	5	Silty sand & sandy silt
Transition layer 3	4	Start depth: 28.22 (ft)	5	Silty sand & sandy silt
		End depth: 28.71 (ft)	3	Clay
Transition layer 4	4	Start depth: 57.25 (ft)	4	Clay & silty clay
		End depth: 57.74 (ft)	5	Silty sand & sandy silt
Transition layer 5	4	Start depth: 58.07 (ft)	5	Silty sand & sandy silt
		End depth: 58.56 (ft)	4	Clay & silty clay
Transition layer 6	4	Start depth: 58.73 (ft)	4	Clay & silty clay
		End depth: 59.22 (ft)	6	Sand & silty sand

Start depth: Depth where the transition layer begins
End depth: Depth where the transition layer ends

Estimation of post-earthquake settlements

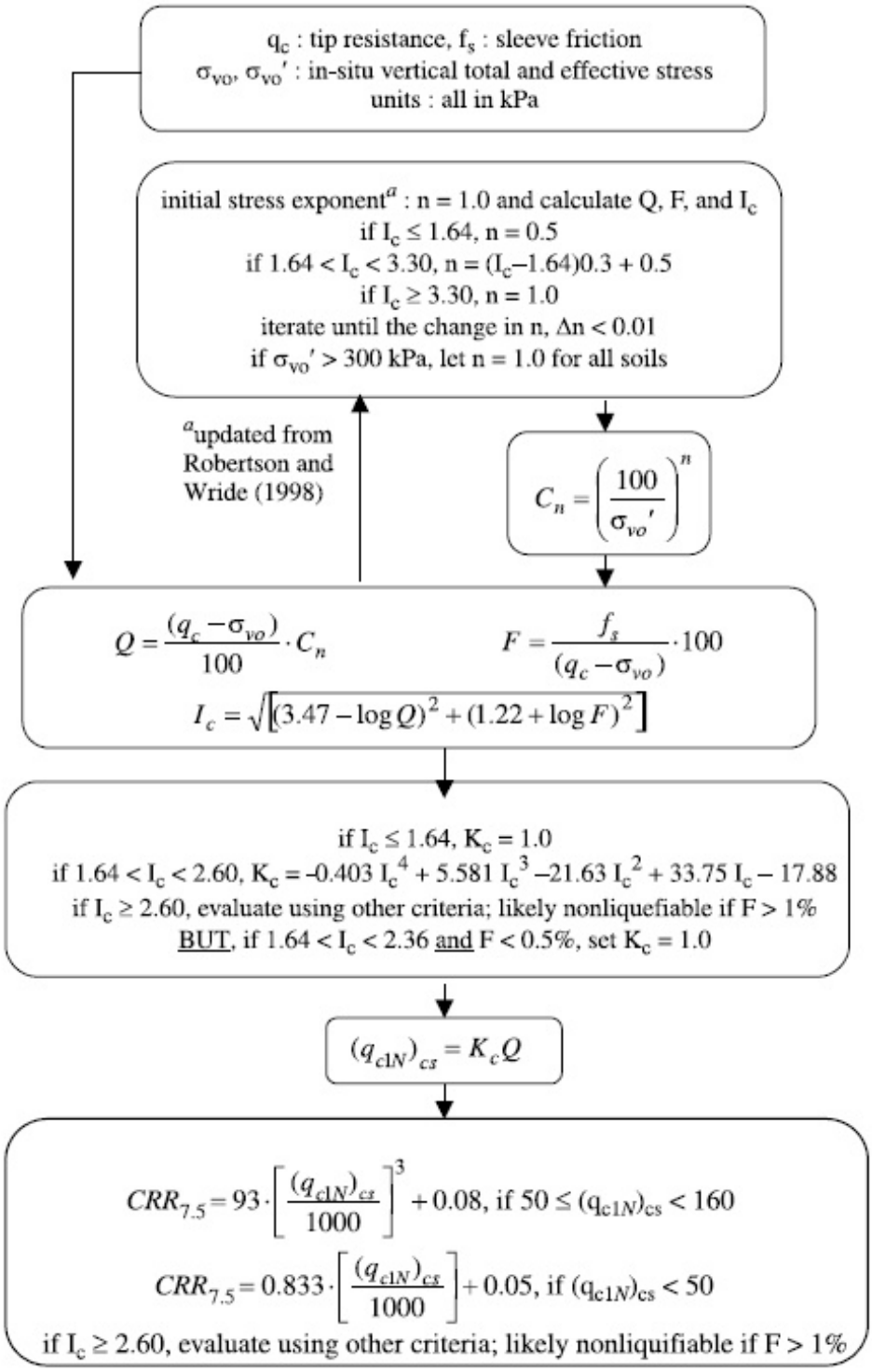


Abbreviations

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

Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

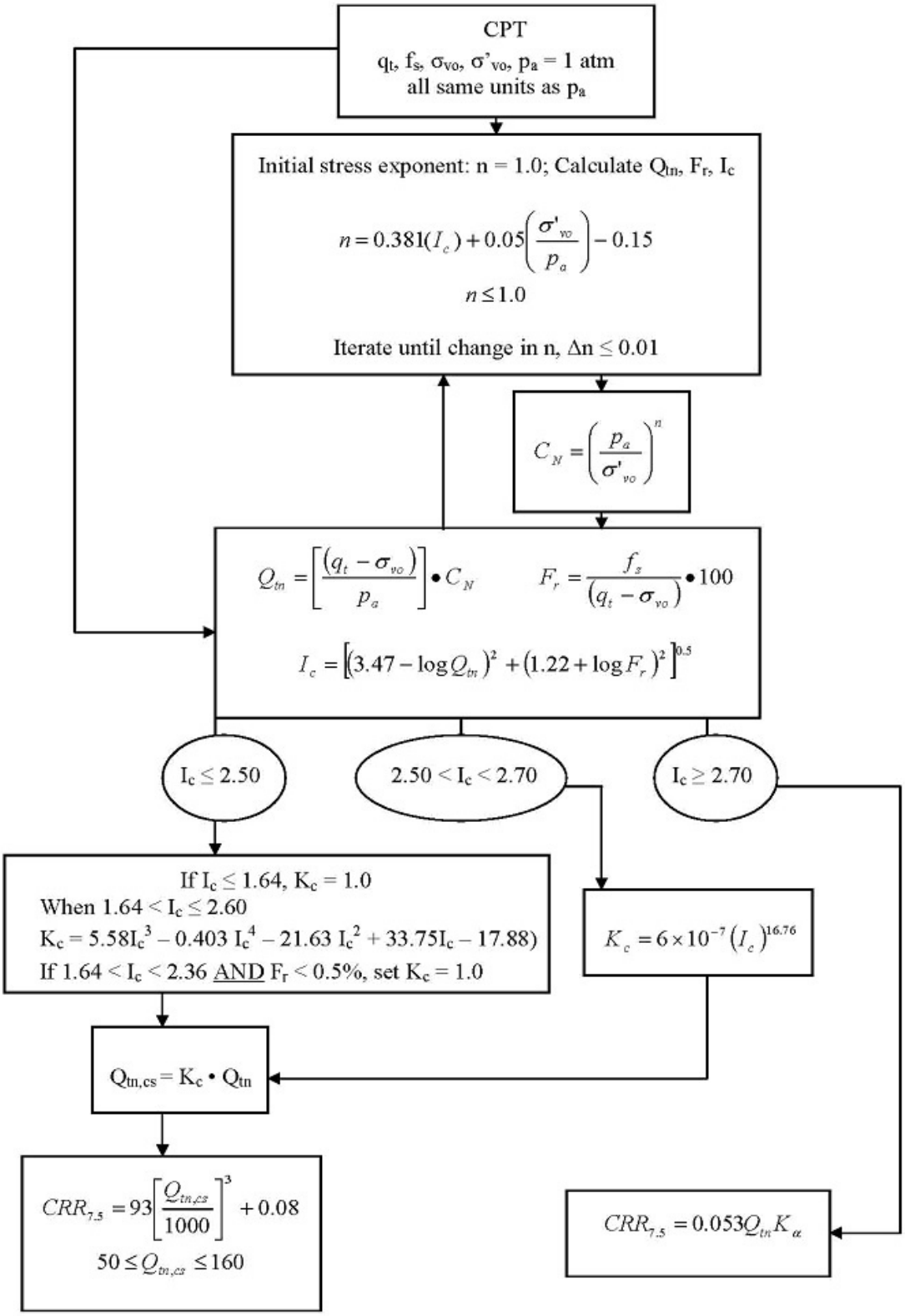
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

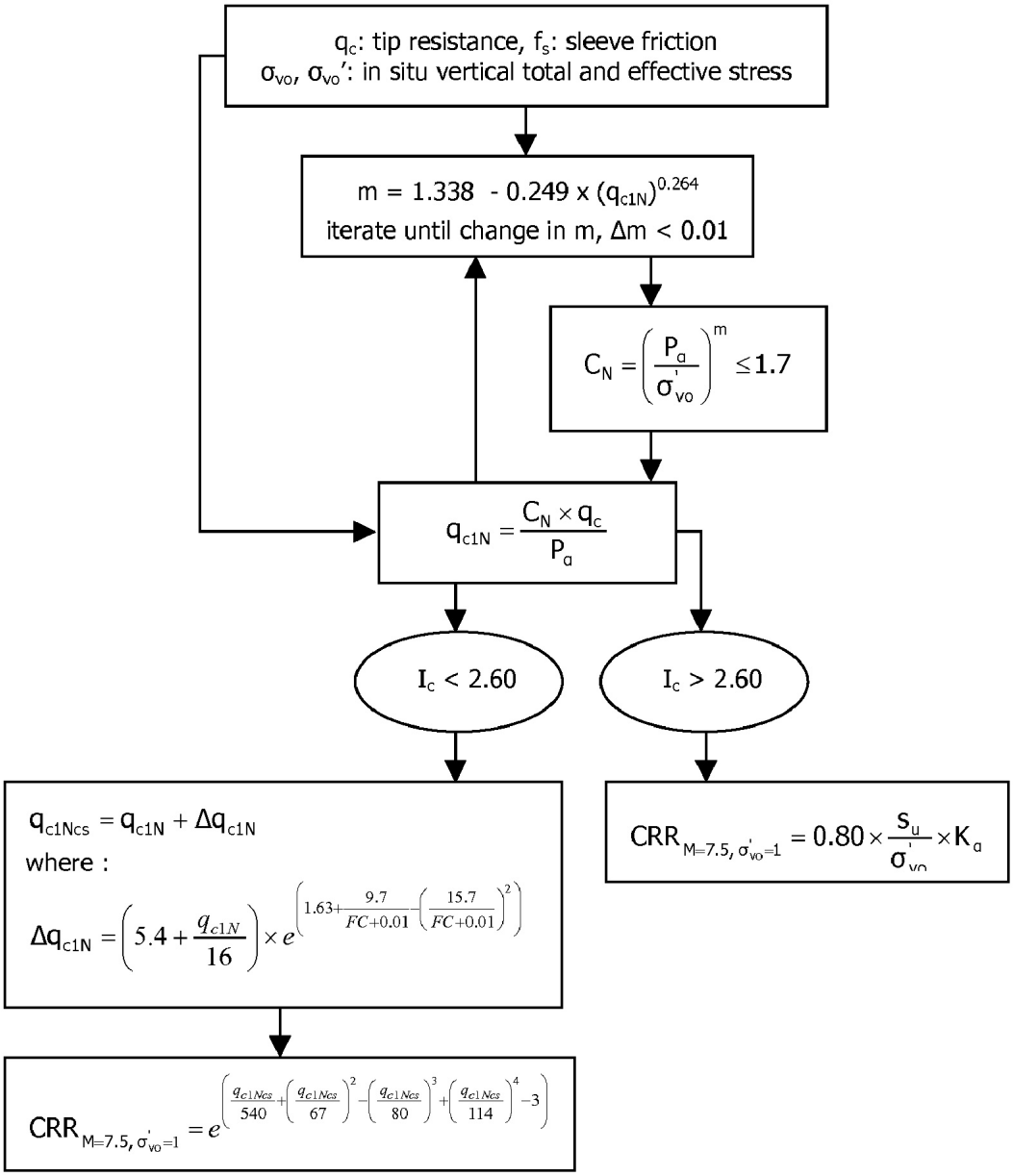
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:

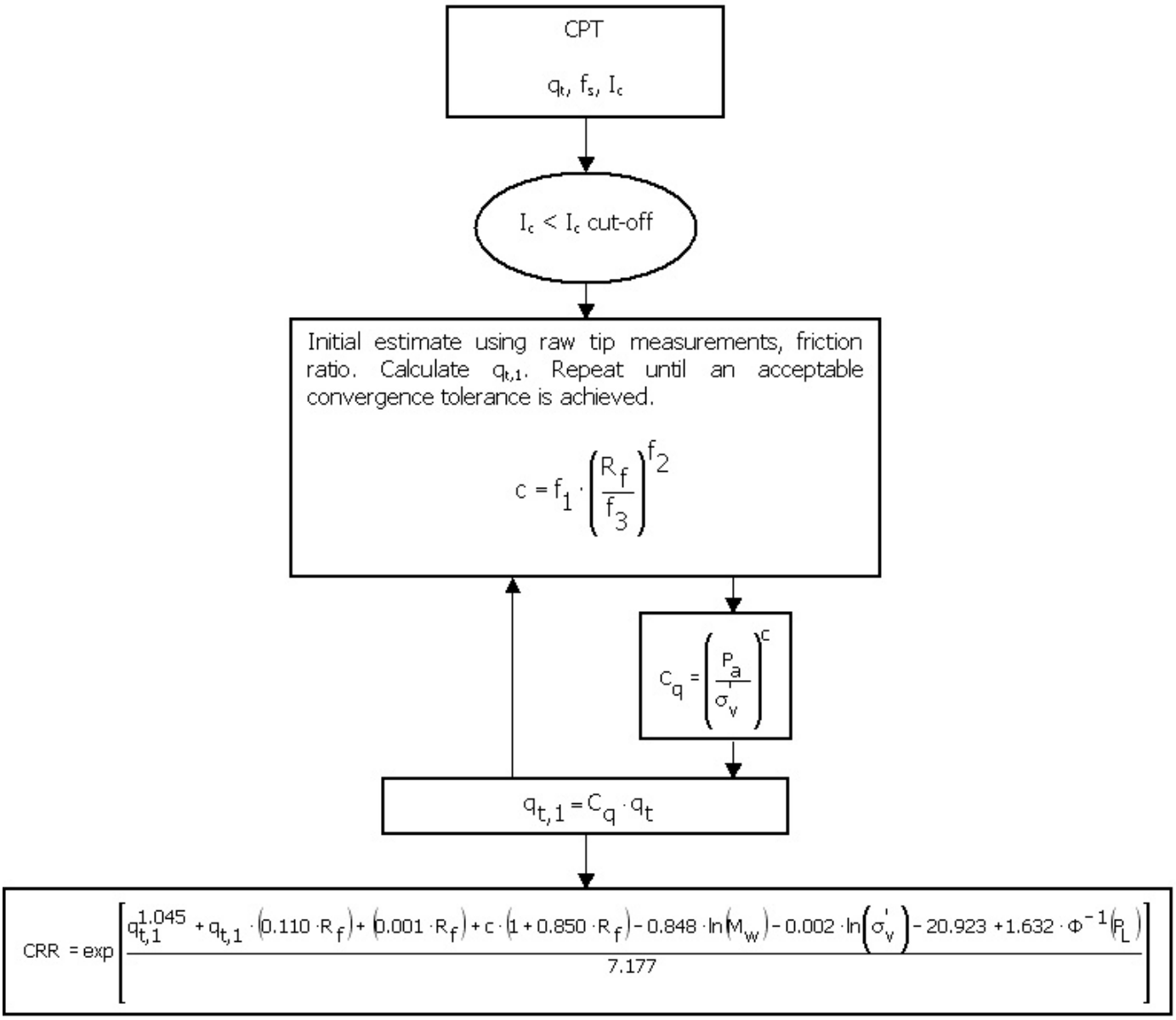


[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)



Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)



Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)

$$FS_{liq} = \frac{CRR_{M=7.5,\sigma'_v=1atm}}{CSR_{M=7.5,\sigma'_v=1atm}}$$

$$CSR_{M=7.5,\sigma'_v=1atm} = 0.65\frac{\sigma_v}{\sigma'_v}\frac{a_{\max}}{g}r_d\frac{1}{MSF}\frac{1}{K_\sigma}$$

$$r_d = \exp\left[\alpha(z)+\beta(z)\cdot M\right]$$

$$\alpha(z) = -1.012-1.126\sin\left(\frac{z}{11.73}+5.133\right)$$

$$\beta(z) = 0.106+0.118\sin\left(\frac{z}{11.28}+5.142\right)$$

$$K_\sigma = 1-C_\sigma\ln\left(\frac{\sigma'_v}{P_a}\right)\le 1.1$$

$$C_\sigma = \frac{1}{37.3-8.27\left(q_{c1Ncs}\right)^{0.264}}\le 0.3$$

$$MSF = 1+\left(MSF_{max}-1\right)\left(8.64\exp\left(\frac{-M}{4}\right)-1.325\right)$$

$$MSF_{max} = 1.09+\left(\frac{q_{C1Ncs}}{180}\right)^3\le 2.2$$

σ'_v *and* σ_v *at start of earthquake shaking*

$$CRR_{M=7.5,\sigma'_v=1atm} = \exp\left(\frac{q_{c1Ncs}}{113}+\left(\frac{q_{c1Ncs}}{1000}\right)^2-\left(\frac{q_{c1Ncs}}{140}\right)^3+\left(\frac{q_{c1Ncs}}{137}\right)^4-2.80\right)$$

$$q_{c1Ncs} = q_{c1N}+\Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9+\frac{q_{c1N}}{14.6}\right)\exp\left(1.63-\frac{9.7}{FC+2}-\left(\frac{15.7}{FC+2}\right)^2\right)$$

$$q_{c1N} = C_{Ne}\frac{q_c}{P_a}$$

$$C_{Ne} = \left(\frac{P_a}{\sigma'_v}\right)^m\le 1.7$$

$$m = 1.338-0.249\left(q_{c1Ncs}\right)^{0.264} \quad with \quad 0.264\le m\le 0.782$$

$$FC = 80\left(I_C+C_{FC}\right)-137 \quad with \quad 0\%\le FC\le 100\%$$

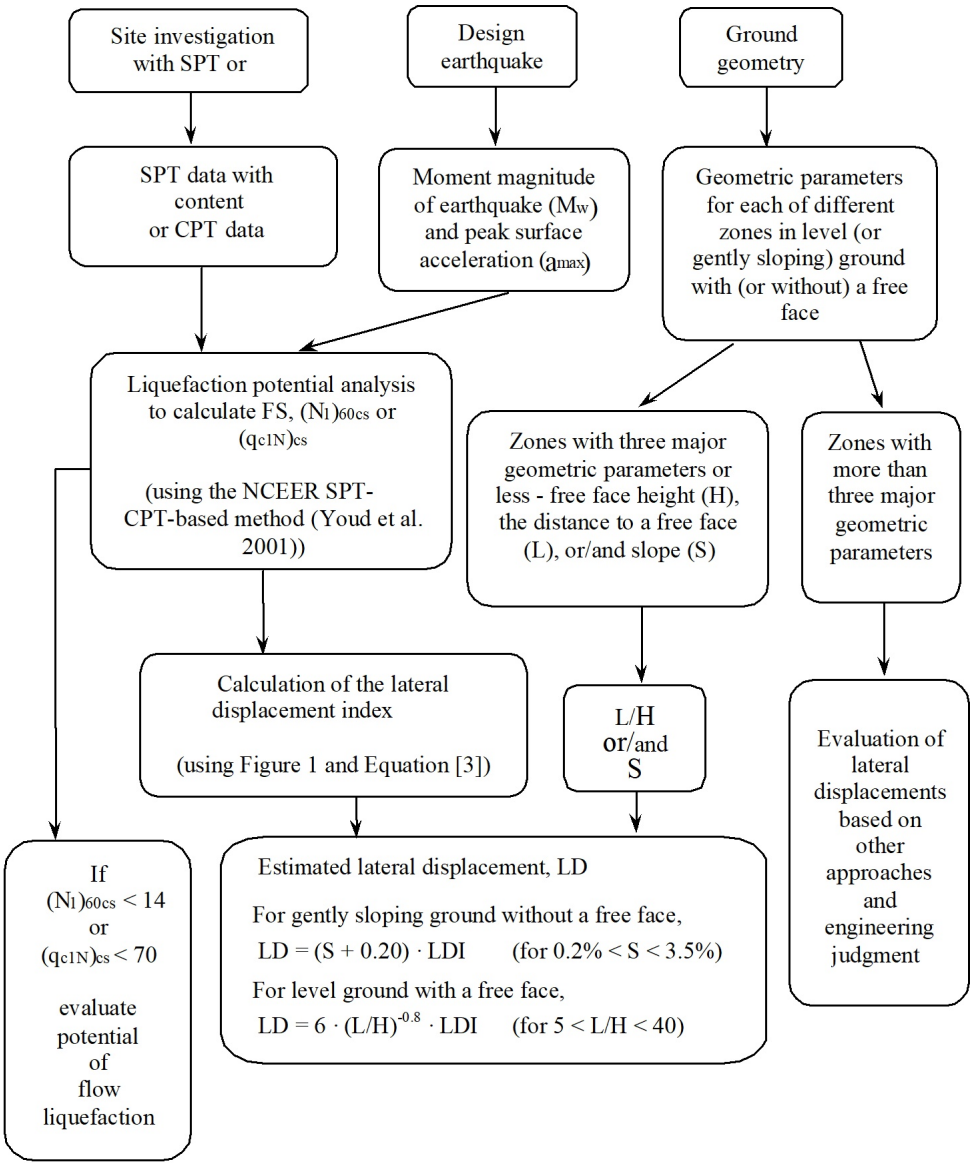
$$I_c = \left[\left(3.47-\log(Q)\right)^2+\left(1.22+\log(F)\right)^2\right]^{0.5}$$

$$Q = \left(\frac{q_c-\sigma_{vc}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vc}}\right)^n \quad with\ 0.5\le n\le 1.0\ per\ Robertson\ \&\ Wride\ (1998)$$

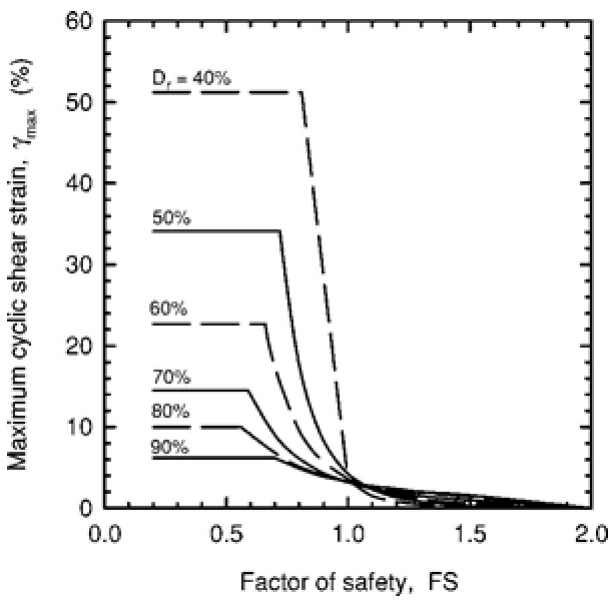
$$F = \left(\frac{f_s}{q_c-\sigma_{vc}}\right)\cdot 100\%$$

σ'_v *at time of CPT sounding*

Procedure for the evaluation of liquefaction-induced lateral spreading displacements



[1] Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



[1] **Figure 1**

$$LDI = \int_0^{z_{max}} \gamma_{max} dz$$

[1] **Equation [3]**

[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the estimation of seismic induced settlements in dry sands

Average shear stress, τ_{av}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{a_{max}}{g} \cdot \sigma_{v0} \cdot r_a$$

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c + 1.68)}\right] \cdot (q_t - \sigma_v)$$

Estimate shear strain amplitude, γ

(based on Pradel (1998))

$$\gamma = \left[\frac{1 + \alpha \cdot e^{bR}}{1 + \alpha}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$\alpha = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

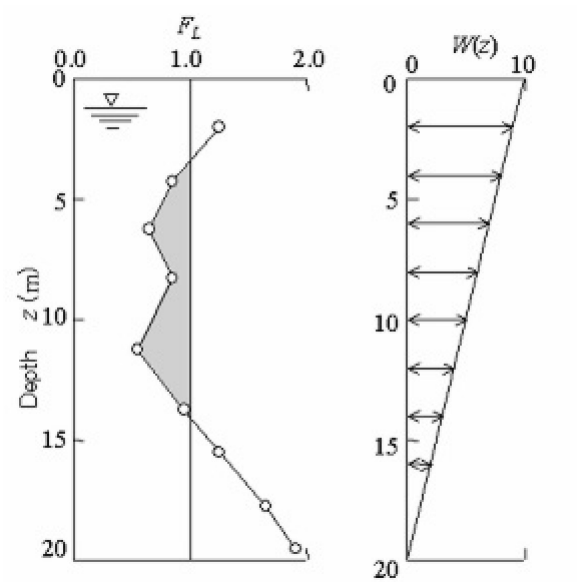
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_0^{20} (10 - 0{,}5_Z) \times F_L \times d_z$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

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