Appendix E

Geotechnical Investigation

DESIGN-PHASE GEOTECHNICAL INVESTIGATION
PROPOSED 133-UNIT RESIDENTIAL DEVELOPMENT
(VESTING TENTATIVE TRACT NO. 72608), 4747 DAISY AVENUE
CITY OF LONG BEACH, CALIFORNIA.

INTEGRAL PARTNERS FUNDING, LLC

March 25, 2014

J.N. 13-443

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PETRA

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Environmental Scientists

March 25, 2014
J.N. 13-443

Mr. Ed Galigher
INTEGRAL PARTNERS FUNDING, LLC
888 San Clemente Drive, Suite 100
Newport Beach, California 92660

Subject: Design-Phase Geotechnical Investigation, Proposed 133-Unit Residential Development (Vesting Tentative Tract No. 72608), 4747 Daisy Avenue, City of Long Beach, California

References: See attached list

Dear Mr. Galigher:

Petra Geotechnical, Inc. (Petra) is pleased to submit herewith our design-phase geotechnical investigation report for the proposed residential development to be located at 4747 Daisy Avenue in the city of Long Beach. This work was performed in general accordance with the scope of services outlined in our most recent contract amendment dated February 9, 2014. This report presents the results of our field investigation, laboratory testing and our engineering and geologic judgment, opinions, conclusions and recommendations pertaining to geotechnical design aspects of the proposed development.

It is a pleasure to be of continued service to you on this project. Should you have any questions regarding the contents of this report, or should you require additional information, please do not hesitate to contact us.

Respectfully submitted,

PETRA GEOTECHNICAL, INC.

Siamak Jafroudi, Ph.D., GE
President

SJ/lm

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DESIGN-PHASE GEOTECHNICAL INVESTIGATION
PROPOSED 133-UNIT RESIDENTIAL DEVELOPMENT
(VESTING TENTATIVE TRACT NO. 72608), 4747 DAISY AVENUE
CITY OF LONG BEACH, CALIFORNIA

PURPOSE AND SCOPE OF SERVICES

Petra Geotechnical, Inc. (Petra) is pleased to present the results of our geotechnical investigation of the subject property. The purposes of this investigation were to 1) obtain information regarding surface and subsurface geologic conditions within the project area, 2) evaluate the engineering properties of the onsite soil materials, and 3) provide conclusions and recommendations for design and construction of the proposed residential development. To accomplish these objectives, our scope of services included the following:

1. Review of available published and unpublished literature and maps pertaining to regional faulting, seismic hazards and soil and geologic conditions within and adjacent to the site that could have an impact on the proposed development.

2. Review of the referenced EIR-level study and seismic hazards evaluation prepared by our firm, as well as the current vesting tentative tract map for the site prepared by Kimley-Horn and Associates, Inc. (dated December 12, 2013).

3. Cursory reconnaissance of the subject site and surrounding areas.

4. Drilling, sampling and logging of five exploratory borings to depths ranging from 16.5 to 41.5 feet to supplement previous field work conducted by our firm during our EIR-level investigation.

5. Laboratory testing and analyses of representative samples of earth materials (bulk and relatively undisturbed) obtained from the borings to determine their engineering properties.

6. Performing a pilot soil infiltration study to support the preliminary design of an onsite storm water infiltration system.

7. Engineering and geologic analyses of the field and laboratory data as they pertain to the proposed construction.

8. Evaluation of faulting and seismicity of the region and the possible impact of regional seismicity on the site and the proposed construction.

9. Analysis of liquefaction and its potential impact on the site and proposed construction.

10. Preparation of this report presenting our findings, conclusions and recommendations for site grading and design of building foundation systems.

LOCATION AND SITE DESCRIPTION

Site Location

The area considered under the purview of this report consists of an approximately 10½-acre parcel located immediately east of the Los Angeles River within the westerly portion of the city of Long Beach. The location of the site with respect to nearby roadways and other landmarks is shown on Figure 1. The irregularly-shaped parcel is bounded by an earthen levy, a settling basin and the Los Angeles River to the west, existing single-family residential properties and associated access streets to the north and northeast, and concrete channel and an elevated Union Pacific Railroad easement to the southeast. An aerial image depicting the property and these boundary areas is shown on Figure 2.

Current Site Usage

The subject site is currently occupied by a recently decommissioned Will J. Reid Scout Park camp facility that was originally founded in the 1940's. Existing facilities associated with the former camp operations include the following structures located within the extreme easterly portion of the site:

- A 2,400 square foot, single-story scout hall
- A 3,300 square foot, single-story classroom building
- A 2,025 square foot, single-story aquatics room
- A 2,400 square-foot, single-story ranger's residence
- A 3,500 square foot, single-story storage barn
- A 75-foot-long by 38-foot-wide, concrete-lined swimming pool (currently empty and non-operational)

All of the building structures listed above are wood-framed with either raised wood floors or concrete floor slabs constructed on grade. According to building permit information obtained from the City of Long Beach Development Services Department, all were constructed during the period between 1946 and 1984. In addition to these existing improvements, the following structures are present within the central and westerly portions of the property:

- Two 500 square foot, single-story restroom buildings
- A 150 square foot, single-story camp master's quarters
- A 400 square foot, single-story modular food service building
- A 55-foot by 50-foot, tiered concrete amphitheater

It appears that most, if not all, of the structures listed above were built during the same time period as the remainder of the structures within the site. Appurtenant non-building structures within the site include approximately 30,000 square feet of asphalt-paved parking area, concrete pedestrian walkways and patio-type slabs, an existing water supply well (located within the east-central portion of the site), an approximately 4-foot-high masonry block retaining wall along the southeasterly property boundary, an electrical transformer enclosure, and buried utility infrastructure.

Vegetation within the vacant areas of the property consists of a low growth of weeds and grasses, as well as numerous tree stumps that remain following initial site clearing activities that were recently performed. Small landscaped planter areas are located adjacent to the existing ranger's residence.

Topography

The topographic information included on the vesting tentative tract map shows that the majority of the subject site is located within a closed depression with surface drainage generally directed toward the center of the site. In general, the property exhibits a variable topography with surface elevations ranging from a high of approximately 38 feet above mean sea level in the area of the existing scout hall building (within the extreme easterly corner of the site), to a low of approximately 30 feet at the lowest level of the amphitheater. A relatively gentle northwest-southeast-trending slope approximately 8 feet in height traverses the westerly one-third of the site.

As mentioned previously, the site is bounded to the west by an approximately 8-foot-high levy that parallels the adjacent concrete-lined Los Angeles River channel. In addition, and approximately 15- to 20-foot-high earthen embankment is located just to the south of the southerly boundary of the subject property. Tracks belonging to the Union Pacific Railroad are located atop this embankment. Offsite and approximately 30 feet to the south of the existing scout activity buildings, to toe of this embankment, is supported by an approximately 15-foot-high concrete retaining wall.

PROPOSED CONSTRUCTION AND GRADING

General Project Design

The current vesting tentative tract map for the site (dated December 12, 2013) indicates that the proposed development will consist of a 133-unit, single-family residential tract that will occupy approximately 10 acres of the 10½-acre site. The remaining ½ acre will be set aside for use as a recreation area. Associated exterior improvements are expected to include asphalt-paved access streets, concrete driveways and pedestrian sidewalks,

surface and subsurface drainage controls, perimeter fencing, common landscaped areas, extensive underground infrastructure, and required storm water quality devices (possibly including a water quality basin to be located within the recreation area). A sewer lift station will also be constructed within the extreme northerly portion of the site. Primary vehicular access to the development will tentatively be provided by means of a paved road that will enter the tract from Daisy Avenue. An auxiliary maintenance and emergency entrance will also be provided from the Oregon Street terminus.

Structural details for the proposed dwellings are unknown at the present time; however, it is anticipated that the buildings will be one to two stories in height and of wood frame construction with floor slabs constructed on grade. For this type of construction, it is anticipated that relatively light foundation loads will be imposed on the subgrade soils.

Proposed Grading

Although no definitive grading plan is currently available for the proposed development, comparison of the existing surface elevation contours and proposed pad grades shown on the current vesting tentative tract map (dated December 12, 2013) suggests that mass grading of the site will generally involve placement of between 1 and 8 feet of compacted fill as required to establish the proposed finished grade elevations. Minor cuts of 2 feet or less will be required within the existing parking lot area along the northeast site boundary. Ultimate fill thicknesses throughout the site will be greater due to the required remedial grading (i.e., removal and recompaction of existing unsuitable surficial soils) as recommended in subsequent sections of this report. Local grade changes will likely be accommodated by low-height graded slopes and possibly retaining walls. No graded slopes of significant height are anticipated at this time.

INVESTIGATION PROGRAM

Prior to performing our investigation of the site, we researched and reviewed published geotechnical reports and maps for the area of the subject site and conducted a surface reconnaissance. Following completion of this initial research, a subsurface exploration and laboratory-testing program was initiated in order to characterize soil and geologic conditions within the project site. Details pertaining to our field methodology and laboratory test procedures are presented in the following sections.

Aerial Photograph Review

We performed an aerial photograph review as part of our EIR-level geotechnical study to assess previous land use and determine whether geomorphic features are present within or adjacent to the site that would be suggestive of active faulting, gross instability, or significant previous grading activity. A list of the photographs reviewed is provided in Table I, below. All photos were black-and-white orthographic images, and most were stereo-paired.

Table I
Aerial Photographs Reviewed

Date	Flight Series	Frame No.
10-19-53	AXJ-13K	217, 218
10-6-67	4-38	24 (single)
1-31-70	61-6	240, 241
3-17-78	78049	152, 153
5-12-79	FCLA 10	113, 114
7-7-88		19211, 19211A
1-27-86	F	338, 339
6-12-90	C84-15	7, 8
1-24-92	C85-5	28, 29
6-9-93	C93-14	144, 145
1-29-95	C103-34	145, 146
10-16-97	C118-34	114 (single)
2-24-99	C134-34	10, 11

Based on the results of our review, it appears that the subject site has been occupied by the recently decommissioned Will J. Reid Scout Park facility since its original construction during the mid 1940's. The photograph images did not reveal any obvious evidence of major earthwork activities, active faulting, landsliding, or other significant geotechnical constraints at the site.

Subsurface Exploration

Our initial subsurface exploration was performed on August 26, 2013 and included advancing four cone penetrometer (CPT) soundings to a maximum depth of approximately 50 feet below the surface using standard truck-mounted CPT equipment provided by Kehoe Testing and Engineering of Huntington Beach, California. The information obtained in this manner was later supplemented by drilling five exploratory borings within the site to depths of 16½ and 41½ feet below the surface on March 4, 2014. The exploratory borings were drilled utilizing a truck-mounted, hollow-stem auger drill rig provided by 2R Drilling of Chino, California. One additional small-diameter soil boring was drilled to a depth of 3 feet within the proposed recreation area at the extreme easterly corner of the site for purposes of conducting a pilot soil percolation study.

Earth materials encountered in our exploratory borings were classified and logged in accordance with Unified Soil Classification System procedures. The approximate locations of the CPT soundings (identified herein as CPT-1 through CPT-4) and exploratory borings (B-1 through B-5) are shown on the attached exploration map (Plate 1). Descriptive CPT and boring logs are presented in Appendix A of this report.

Our subsurface exploration included the collection of bulk (disturbed) and relatively undisturbed samples of subsurface soil materials from borings B-1 through B-4 for laboratory testing purposes. Bulk samples consisted of a composite of earth materials retrieved at selected depth intervals from the borings. Relatively undisturbed samples were collected using a 3-inch outside-diameter, modified California split-spoon soil sampler lined with 1-inch high brass rings. The sampler was driven to a depth of 18 inches with successive 30-inch drops of a hydraulically operated, 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the exploration logs. The central portions of the driven core samples were placed in sealed containers and transported to our laboratory for testing.

Where deemed appropriate based on the CPT data collected, Standard Penetration (SPT) tests were also performed at selected depth intervals in accordance with the American Society for Testing Materials (ASTM) Standard Procedure D 1586. This method consists of mechanically driving an unlined standard split-barrel sampler 18 inches into the soil with successive 30-inch drops of the 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the exploration logs. The number of blows required to drive the standard split-spoon sampler for the last 12 of the 18 inches was identified as the uncorrected standard penetration resistance (N). Disturbed soil samples from the unlined standard split-spoon samplers were placed in plastic bags and transported to our laboratory for testing.

Laboratory Testing

To evaluate the engineering properties of the onsite soils, a number of laboratory tests were performed on selected samples considered representative of the materials encountered during our investigation. Laboratory tests included the determination of in-place moisture content and unit dry density, maximum dry density and optimum moisture content, expansion potential, consolidation characteristics, and general soil corrosivity screening including soluble sulfate and chloride content, soil pH and minimum resistivity. The data generated during our laboratory testing has been incorporated into the findings and conclusions presented in subsequent sections of this report.

FINDINGS

Regional and Local Geologic Setting

Regional Physiographic Setting

The subject site is located within the Los Angeles Basin, northwest-trending alluviated lowland situated at the north end of the Peninsular Ranges geomorphic province of coastal southern California. This basin, which is the surface expression of a deep structural trough, has been subdivided into four primary structural blocks that are distinguished from one another by contrasting basement rock types and stratigraphy. These structural blocks are generally separated by zones of faulting along which movement has been occurring intermittently since middle Miocene time (Yerkes et al., 1965).

More specifically, the subject property is located within the southerly portion of the Downey Plain, a broad lowland area that comprises a large portion of the Central Block of the Los Angeles Basin. This plain is bounded by the Santa Monica Mountains to the north, the Puente Hills and Santa Ana Mountains to the northeast and east, and a northwest-trending alignment of hills and mesas to the west and southwest which represent surface expressions of uplift along the Newport-Inglewood fault. In the area of the subject site, the soils that form this extensive alluvial plain are composed primarily of alluvial materials deposited as a result of sedimentation along the Los Angeles River.

Area Geology

The distribution, thicknesses and characteristics of near-surface soils in Los Angeles County have been previously mapped by other investigators at a scale of 1:48,000 for purposes of seismic zonation. Based on our review of published maps, the northern portion of the city of Long Beach is underlain by unconsolidated, generally fine-grained, Holocene-age alluvial fan and valley deposits. These geologically young materials extend locally to depths in excess of 150 feet, and are underlain by semi-consolidated older alluvium or sedimentary bedrock of the late Quaternary-age Lakewood formation. Relatively minor thicknesses of artificial fill are likely to occur locally where the previously existing natural ground surfaces have been modified during urbanization of the area.

Local Geology and Subsurface Conditions

As shown on the exploration logs included in Appendix A, our subsurface investigation revealed that the area of proposed development is underlain predominantly by late Quaternary-age alluvial fan deposits that extend below the maximum depth explored (41.5 feet). These materials consist of discontinuous, interlayered medium-dense to

dense, dry to slightly moist, fine-grained sands and silty sands, and firm to very stiff, dry to very moist silts, sandy silts, silty clays and low-plasticity clays. Laboratory testing of representative samples of the more granular materials (i.e., sands and silty sands) yielded in-place dry densities ranging from 85.0 to 110.6 pounds per cubic foot and moisture contents ranging from 1.3 to 26.0 percent. The fine-grained alluvial soils (including silts, sandy silts, silty clays and clays) exhibited in-place dry densities ranging from 76.8 to 110.9 pounds per cubic foot and moisture contents ranging from 5.2 to 23.3 percent.

In all five of our exploratory borings, the native alluvial materials described above were found to be capped by an approximately 1½- to 3-foot-thick mantle of artificial fill that was presumably placed during original grading operations within the site. Given the previous usage of the site, it is unlikely that the onsite fill materials were placed in accordance with current grading standards and certified by a geotechnical professional. For this reason, all existing onsite fill is classified as "undocumented" for purposes of this investigation.

Groundwater

Groundwater was not encountered in our boring during at the time of our field investigation in the area of the subject site to the maximum depth of 41.5 feet below ground surface. The extent of shallow groundwater in the vicinity of the subject site is described in general terms in the referenced Seismic Hazard Zone report for the Long Beach quadrangle published by the California Division of Mines and Geology (CDMG, 1998). Based on information provided in that report, the subject property is located within an area where shallow groundwater (i.e., groundwater existing at a depth of 40 feet or less below the ground surface) would typically be expected to occur. The historical highest groundwater depth for the site is approximately 30 feet below the surface based on our review of Plate 1.2 of the referenced CDMG seismic hazard zone report.

A review of local area well data maintained by the California Department of Water Resources indicates that one water supply well (Well No. 04S13W01N003S) is located approximately 0.8 kilometers north of the subject site has been included in a groundwater depth monitoring program since March, 2000. During the monitoring period, the shallowest groundwater reading obtained was 4.8 feet above sea level. Extrapolated to the subject site, this represents a 14-year groundwater high of approximately 29.2 feet below the surface. This correlates well with the anticipated historical high groundwater level published in the referenced CDMG seismic hazard zone report.

Tectonic Setting

Regional Surface Fault Systems

The geologic structure of Southern California is dominated by northwest-trending faults associated with the San Andreas system. Faults such as the Newport-Inglewood, the Whittier-Elsinore, the San Jacinto, and various segments of the San Andreas Fault itself are all major faults associated with this system. They are all known to be seismically active, and most are known to have ruptured the ground surface in historic time. Also within the southern California region are a number of west-trending, low-angle reverse (thrust) faults that are similarly active. The majority of these faults occur as north-dipping planes which trend along the south-facing flanks of the Transverse Ranges. Among the known active thrust faults in the region include the Cucamonga, Sierra Madre, Santa Monica, and Hollywood faults.

Concealed Faults

Another category of fault known as the "blind thrust" became recognized as a significant seismic hazard as a result of the 1987 moment magnitude (Mw) 6.0 Whittier Narrows earthquake. Blind thrusts are concealed beneath the earth's surface and are defined as dip-slip faults that tend to fold and/or uplift the near surface sediments during moderate to large magnitude earthquakes (Shaw and Suppe, 1996). In 1994 the Mw 6.7 Northridge earthquake occurred along what researchers have interpreted as a south-dipping thrust ramp beneath the San Fernando Valley. Together, these events caused more than $25 billion in property damage and clearly demonstrate the risks that blind thrusts pose to the greater Los Angeles metropolitan area.

Recent structural models of the Los Angeles basin suggest that deep-seated, blind thrust sheets underlie portions of Orange and Los Angeles Counties. These structures are apparently accommodating north-south compression with slip rates of several millimeters per year (Hauksson, 1992; Petersen and Wesnouski, 1994). The Puente Hills and Upper Elysian Park blind thrust systems represent two such blind thrusts that are reported to extend below and in close proximity to the site (Dolan et al., 2003, Shaw et al., 2002, and Oskin et al. 2000). A similar system underlies the San Joaquin Hills (Grant et al., 1999). Structural models and seismicity values for these three blind thrust systems and the Northridge blind thrust have been incorporated into the California Geological Survey seismic model, which was updated in April 2003 (Cao, et al., 2003).

Nearby Seismic Sources

Published geologic maps and literature indicate that the site lies within 45 kilometers of a number of significant active and potentially active faults that, in addition to the various segments of the more distant San Andreas Fault

zone, are considered capable of generating strong ground motion at the subject site. The names and locations of these faults relative to the subject property are provided in Table II. The locations of these faults are graphically depicted on Figure 3.

Table II
Significant Nearby Seismic Sources

Fault Name	Approximate Distance/ Direction From Site	Source Type[1]	Slip Rate (mm/yr)[2]	Maximum Magnitude[2,3]
Newport-Inglewood	0.7 kilometers southwest	B	1.0	6.9 (7.2 – 7.5)[4]
Puente Hills Blind Thrust	9.7 kilometers northeast	B	0.7	7.1
Palos Verdes	11.5 kilometers southwest	B	3.0	7.1 (7.3-7.7)[5]
Whittier	24.2 kilometers northeast	B	2.5	6.8 (7.0)[5]
Upper Elysian Park	25.0 kilometers north-northwest	B	1.3	6.4 (6.7)[5]
San Joaquin Hills Thrust	31.0 kilometers southeast	B	0.5	6.6 (7.1)[5]
Raymond	31.9 kilometers north	B	0.5	6.5 (6.8)[5]
Hollywood	32.2 kilometers northwest	B	1.0	6.5 (6.7)[5]
Santa Monica	32.6 kilometers northwest	B	1.0	6.6 (6.8 – 7.4)[5]
Verdugo	35.3 kilometers northwest	B	0.5	6.7 (6.9)[5]
San Jose	37.2 kilometers northeast	B	0.5	6.5 (6.7)[5]
Sierra Madre	42.3 kilometers northwest	B	3.0	7.0 (7.3)[5]
Chino-Central Ave.	42.9 kilometers northeast	B	1.0	6.7
Clamshell-Sawpit	43.4 kilometers north-northeast	B	0.5	6.5 (6.7)[5]

Notes:
1) As classified according to 2001 California Building Code Table 16-U.
2) Per CGS 2002 fault data file (Cao et al, 2003).
3) Moment Magnitude (M_W).
4) The expected magnitude on the Newport-Inglewood fault according to the 2008 USGS fault files ranges from 7.2 to 7.5 depending on the cascade models chosen (EZ-FRISK 2010).
5) 2008 USGS fault file (EZ-FRISK 2010)

Based on a review of published geotechnical maps and literature pertaining to regional faulting, the closest known fault considered capable of causing strong ground motion at the subject site is the onshore segment of the Newport-Inglewood fault. Located approximately 0.7 kilometer southwest of the subject site, the Newport Inglewood fault consists of a series of parallel and en-echelon, northwest-trending faults and folds that extend from the southern edge of the Santa Monica Mountains southeast to the offshore area of Newport Beach. This zone has a history of moderate to high seismic activity and has produced numerous earthquakes greater than magnitude 4.0, including the March 11, 1933 magnitude 6.3 Long Beach earthquake (which was actually centered near the city of Newport Beach). At the time of the 1933 earthquake, secondary effects of strong ground shaking including sand boils, ground fissures, and liquefaction were noted in the city of Long Beach, as well as in the city of Huntington Beach along Pacific Coast Highway near the Huntington Beach Pier and in the Bolsa Chica area.

In addition, subsurface fault displacement of a few inches was documented following the October 21, 1941 earthquake (magnitude 4.9) and the June 18, 1944 earthquake (magnitude 4.5), both of which occurred along the Newport-Inglewood fault in the Dominguez Hills area (Barrows, 1974). Various segments of the Newport-Inglewood fault have been included within the boundaries of an Alquist-Priolo fault rupture hazard zone.

Five additional faults that are considered to be significant seismogenic sources are located in relatively close proximity to the subject site and thus warrant mention in this report. These include the San Joaquin Hills thrust fault, the Palos Verdes fault, the Puente Hills blind thrust and the Whittier fault. Descriptions of these faults are provided in the following paragraphs:

Puente Hills Blind Thrust Fault

Located approximately 9.7 kilometers northeast of the site, the Puente Hills blind thrust lies buried about two miles beneath the surface and dips to the north at approximately 25 degrees (Shaw et al., 2002; Dolan et al, 2003). The fault extends approximately 40 kilometers from the City of Brea to downtown Los Angeles and consists of the Coyote Hills, Santa Fe Springs and Los Angeles segments. According to research, this fault generated the 1987 Mw 5.9 Whittier Narrows earthquake (Hauksson and Stein, 1989), which caused an estimated $358 million in property damage. This earthquake occurred at a depth of approximately six miles and was followed by an aftershock of slightly lower magnitude three days later.

Palos Verdes Fault

The Palos Verdes fault is located approximately 11.5 kilometers to the southwest of the subject site and is generally described in terms of three individual segments, namely the San Pedro Bay, the onshore, and the Santa Monica Bay segments (Ziony, 1985). All segments are believed to possess a reverse or reverse right oblique sense of motion. References reviewed as part of this report indicate that the San Pedro Bay portion of the fault has been shown to displace Holocene sedimentary materials; however, evidence for Holocene activity along the onshore and Santa Monica Bay segments is currently in dispute. Nonetheless, in light of the increased amount of seismicity that has been attributed to the Santa Monica Bay segment, the Palos Verdes Hills fault has been classified as active.

Whittier Fault

At its closest approach, the Whittier fault is located approximately 24.2 kilometers northeast of the subject site. It is one of the most prominent structural features in the Los Angeles Basin and occurs as three subparallel strands that form a zone approximately 1.2 kilometers wide and about 74 kilometers long. Topographic expression of this zone is marked by a distinct linear valley with offset drainages along the valley margins. Published investigations reveal that this fault offsets Holocene stratigraphy just east of the city of Whittier, as well as to the northwest of Brea Canyon (Leighton and Associates, 1990). For this reason, this fault is considered active and is included within the boundaries of an Alquist-Priolo Earthquake Fault zone.

Most sources report a relatively low level of seismic activity along the eastern portion of the Whittier fault with earthquake magnitudes rarely exceeding Richter Magnitude 5.0. However, on September 3, 2002, a magnitude

4.6 earthquake occurred northeast of Yorba Linda in Orange County that has been attributed to a small conjugate fault related to the Whittier fault zone (Hauksson and Hutton, 2002). Another moderate earthquake having a Richter Magnitude of 5.4 occurred in the same general area on July 28, 2008 and was also initially attributed to the Whittier fault; however, subsequent analysis suggests that this seismic event was associated with a newly postulated feature that has been referred to as the "Yorba Linda Trend." Researchers currently suspect that this feature consists of a one- to two-mile-wide fault system that traverses the area where the Whittier, Elsinore and Chino Hills faults intersect near the northern end of the Santa Ana Mountains.

Upper Elysian Park Seismic Zone

Although generally not classified as an active fault by the common definition, published literature indicates that the subject site is located roughly 25 kilometers southeast of the Elysian Park seismic zone which, as mentioned previously, belongs to a group of features known as "buried (or blind) thrust faults" due to the fact that they are expressed at the surface as broad uplifted folds rather than as distinct scarps or surface traces. The seismic risk posed by buried faults, in terms of recurrence interval and maximum credible magnitude, has not yet been well established; however, it is generally accepted that the Elysian Park seismic zone is responsible for the moderate-sized 1987 Whittier Narrows earthquake. Consequently, the potential for future earthquakes along this fault with magnitudes larger than 6.0 cannot be precluded.

San Joaquin Hills Thrust Fault

Recent studies by various researchers have suggested that the hilly terrain that characterizes the San Joaquin Hills in central and southern Orange County is the result of late Quaternary folding associated with tectonic uplift along an active thrust fault. Recognition of this potentially seismogenic blind thrust extends the known area of active blind thrusts and fault-related folding present in Los Angeles County southward into coastal Orange County (Grant et al., 1999). Recent blind thrust earthquakes, including the 1987 magnitude 5.9 Whittier Narrows and the 1994 magnitude 6.7 Northridge events, have demonstrated the significance of these features with respect to the tectonic setting of southern California. Although the San Joaquin Hills thrust has not been observed directly at the surface, structural modeling indicates that this fault has a slip rate of approximately 0.5 millimeters per year that yields a recurrence interval of 1,650 to 3,100 years for moderate-sized earthquakes.

Historical Seismicity

As is the case with most locations in Southern California, the subject site is located in a region that is characterized by moderate to high seismic activity. The project site and vicinity have experienced strong ground shaking due to earthquakes on a number of occasions in historic time. Some of the more significant historic seismic events for which detailed ground motion data are available are listed in Table III, along with the corresponding approximate epicentral distances to the subject site, the calculated moment magnitude, and the approximate peak horizontal site accelerations based on various published earthquake databases. The locations of selected earthquake epicenters with respect to the subject site are shown graphically on Figure 4.

Table III
Significant Historic Earthquakes

Earthquake Events	Approximate Epicentral Distance From Site (kilometers)	Moment Magnitude (Mw)	Approximate Site Acceleration (g)[1]	Approximate Modified Mercalli Intensity[2]
Calexico/Sierra El Mayor (April 4, 2010)	319	7.2	0.01	IV
Inglewood (May 18, 2009)	16.4	4.7	0.18	VII
Chino Hills (July 29, 2008)	42.3	5.4	0.06	V
Hector Mine (October 16, 1999)	197.1	7.1	0.02	IV
Northridge (Jan. 17, 1994)	51.6	6.7	0.08	V
Whittier Narrows (Oct. 1, 1987)	26.8	5.9	0.25	VII
Sylmar (Feb. 9, 1971)	65.9	6.4	0.05	V
Landers (June 28, 1992)	167.3	7.6	0.04	V
Big Bear (June 28, 1992)	132.7	6.7	0.03	IV
Kern County (July 21, 1952)	148.8	7.7	0.05	V
Long Beach (March 11, 1933)	33.0	6.3	0.10	VI
Glen Ivy Hot Springs (May 15, 1910)	75.6	6.0	0.03	IV
Lytle Creek (July 30, 1894)	75.2	6.0	0.03	IV
Los Angeles (July 11, 1855)	30.1	6.3	0.12	VI
Wrightwood (Dec. 8, 1812)	77.5	7.0	0.06	V

Notes: [1] Maximum free-field site accelerations for the Calexico/Sierra El Mayor, Inglewood, Chino Hills, Northridge, Whittier Narrows, Landers and Big Bear earthquakes are based on CDMG Office of Strong Motion Studies published accelerogram data for CSMIP Station No. 14242, located approximately 0.4 kilometer southeast of the subject site. For the Hector Mine earthquake, the maximum site acceleration is based on the published accelerogram data for CGS CSMIP Station No. 14560, located approximately 7.7 kilometers south of the subject site. Site accelerations for all other listed are estimated based on the results of a computerized database search using a program developed by T.F. Blake (Eqsearch V3.0, 2000). For purposes of the computerized site acceleration estimates, the attenuation relationship developed by Bozorgnia, Campbell and Niazi (1999) for Holocene soil sites was considered appropriate.

[2] Based on Wald, D.J. et. al., 1999.

Active Fault Zonation

No portion of the subject site is located within the boundaries of an Earthquake Fault Zone as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act (Hart and Bryant, 1997). The site is; however, located approximately ½ of a mile to the north-northeast of the earthquake fault zone that has been established around the known active traces of the Newport-Inglewood fault.

On the basis of our review of the current revision of the Seismic Safety Element of the City of Long Beach General Plan, the subject site is not located within a City-designated "Caution Zone" wherein additional subsurface investigation would be required to determine the presence and level of activity of suspected active branches of the Newport-Inglewood fault (City of Long Beach, 1988).

Secondary Seismic Hazards – Ground Failure

Secondary effects of seismic activity that are typically considered as possible hazards to a particular site include several types of ground failure. The general types of ground failure that can occur at a particular site as a consequence of severe ground shaking include landsliding, ground subsidence, ground lurching, shallow ground rupture, lateral spreading, liquefaction, and soil strength loss. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from the causative fault, topography, soil and groundwater conditions, in addition to other factors.

Of the seismically induced ground failure modes listed above, liquefaction and liquefaction-related surface phenomena appear to be the primary concerns with respect to the subject site. Based on our review of the published Seismic Hazard Zone Report for the U.S.G.S. Long Beach 7.5-minute quadrangle (CDMG, 1998), the subject site lies within a designated Liquefaction Hazard Zone (see Figure 5). Given the essentially flat topography that characterizes the northern portion of the city of Long Beach, the site has not been included within a State-designated seismically-induced landslide hazard zone.

Areas of potential liquefaction have also been identified in the Seismic Safety Element of the City of Long Beach General Plan and categorized in terms of general liquefaction susceptibility (i.e., minimal, low, moderate and significant). Review of Plate 7 of the Seismic Safety Element indicates that the subject site is located within a City-designated area of "Minimal" liquefaction potential (City of Long Beach Department of Planning and Building, 1988).

Site-Specific Liquefaction Analysis

Liquefaction occurs when dynamic loading of a saturated sand or silt causes pore-water pressures to increase to levels where grain-to-grain contact is lost and material temporarily behaves as a viscous fluid. Liquefaction can cause settlement of the ground surface, settlement and tilting of engineered structures, flotation of buoyant buried structures and fissuring of the ground surface. A common surface manifestation of liquefaction is the formation of sand boils, short-lived fountains of soil and water that emerge from fissures or vents and leave freshly deposited conical mounds of sand or silt on the ground surface.

Assessment of liquefaction potential for a particular site requires knowledge of a number of regional as well as site-specific parameters including the estimated design earthquake magnitude, the distance to the assumed causative fault and the associated probable peak horizontal ground acceleration at the site, subsurface stratigraphy

and soil characteristics. Parameters such as distance to causative faults and estimated probable peak horizontal ground acceleration were determined using published references and by utilizing online computer programs by the U.S. Geological Survey (USGS). Stratigraphy and soil characteristics were determined by means of a site-specific subsurface investigation combined with appropriate laboratory analysis of representative samples of onsite soils. The site-specific ground motion analysis is attached in Appendix C of this report.

As noted previously herein, groundwater did not encounter at the time of our field investigation in the area of proposed construction to a maximum depth of 41.5 feet below ground surface. Groundwater depths according with published maps which indicate that the historic high groundwater level in the vicinity of the site is less than 40 feet below the ground surface (CDMG, 1998). In accordance with current standards of practice, we have assumed a historical high groundwater level of 30 feet below the surface for purposes of our analysis.

A variety of computer programs are available that were developed specifically for liquefaction and seismic settlement analyses. For purposes of this study, we selected the commercially available software program Cliq Version 1.7.1.14 (Geologismiki, 2012) that implements updated versions of the NCEER procedure as recommended by Dr. Peter Robertson (2010). The procedures were based on the methods originally recommended by Seed and Idriss (1982). Our analysis was performed solely using CPT data due to the fact that the CPT provides *continuous* penetration resistance data as opposed to than borehole data that must be averaged over discrete sampling increments (e.g., 5 or 10 feet).

Surface Manifestation of Liquefaction

Using the methods outlined by Ishihara (1985), and considering the depth of the liquefiable layers identified by the results of our CPT testing and a historic high groundwater depth of 30 feet, the thickness of the non-liquefiable layers above the liquefiable zone appears to be sufficient to prevent surface manifestation of liquefaction (such as sand boils, ground fissures, etc).

Seismically Induced Settlement

Based on the results of our site-specific study, the maximum estimated total dynamic vertical settlement was calculated to vary from approximately ½ inches to 2½ inches across the site. Supporting calculations are provided in Appendix D of this report. It should be noted owing to the existing groundwater table level and the local groundwater usage, it is expected that the actual ground settlement during a major ground shaking to be lower than these maximum predicted values.

The maximum differential settlement is estimated to be on the order of less than 1½ inches over a horizontal span of 40 feet. Predicted values of maximum seismically induced settlement are provided in Table IV, below.

Table IV
CPT-Based Maximum Seismically Induced Settlement

CPT Location	Estimated Settlement, in
CPT-1	¾
CPT-2	½
CPT-3	1¾
CPT-4	2½

Lateral Spreading

Lateral spreading is an additional hazard associated with liquefaction and refers to the movement of liquefied soil in a downslope direction or toward an open face. This site is located in close proximity to a settling basin, the Los Angeles River and a concrete channel. Therefore, it may be plausible to assume lateral spreading to occur at the site, especially, in the settling basin/Los Angeles River direction.

Based on the exploration and sounding data, the site is underlain by potentially liquefiable sands and silty sands that are discontinuous and interlayered between non-liquefiable clay layers. Our analysis indicates that the site, at an approximate elevation that corresponds to Mean Sea Level, is susceptible to as much as 12 inches of lateral spreading most likely in the direction of the adjacent Los Angeles River. Table V below details the results of our analysis.

As shown in Table V, lateral spreading/movement of the site during or immediately after a strong earthquake is estimated to be on the order of 12 inches at shallow depths. It should be noted that the bottom of the adjacent Los Angeles River is at an approximate elevation of 25 feet, MSL with the depth of the adjacent settling basin somewhat below the river bottom. As such, a majority of the lateral spreading zones are significantly (15± feet) deeper the bottom of the river.

Table V
CPT-Based Maximum Predicted Lateral Spreading

CPT Location	Surface Elevation*	Spreading Elevation	Estimated Lateral Spread, in
CPT-1	31	-13 to -14	3
		-18 to -19	1½
CPT-2	38	-3 to -6	8½
CPT-3	38	8 to 6	12
		2 to 1	5
		-3 to -5	8
CPT-4	33	2 to -1	7
		-2 to - 7	12

* Note: The Bottom of Adjacent Los Angeles River is at Approximate Elevation of 25 Feet, MSL

It should be noted owing to the existing groundwater table level and the local groundwater usage, it is expected that the actual lateral spreading during a major ground shaking to be lower than these maximum predicted values. Further, as stated earlier, Plate 7 of the Seismic Safety Element indicates that the subject site is located within a City-designated area of "Minimal" liquefaction potential. As such, the probability of the lower layers to contribute to the surface lateral spreading during a strong ground shaking is considered very low. Additionally, CGS Special Publication 117A suggests that it is possible to design foundations of sufficient strength to withstand lateral displacements on the order of 0.5 meter (approximately 20 inches) or less. Post-tensioned slab foundations or strengthened conventional foundations are considered to be viable alternatives given the anticipated level of lateral movement at the subject site. Critical utility service connections should also be designed to accommodate the potential lateral movement.

Seismically-Induced Flooding

The types of seismically induced flooding which may be considered as potential hazards to a particular site normally includes flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major reservoir or other water retention structure upstream of the site. Since the site lies 6 miles inland from the Pacific Ocean at an average elevation of approximately 35 feet above sea level, and since it does not lie in close proximity to an enclosed body of water, the probability of flooding from a tsunami or seiche is considered to be very low. In addition, the site is not located within a designated tsunami inundation area as identified on the published Tsunami Inundation Map for the Long Beach Quadrangle (CEMA, 2009).

Four major flood control dams lie between 11 and 30 miles upstream of the city of Long Beach. These include the Sepulveda Basin, Hansen Basin, Whittier Narrows Basin and Santa Fe Basin flood control facilities. In the event that a seismically-induced failure of either the Sepulveda or Hansen facilities were to occur when these facilities were filled to capacity, most if not all of the resulting flood waters would be expected to dissipate prior to reaching the Long Beach city limits. However, flood inundation maps prepared by the Army Corps of Engineers indicate that a failure of the Hansen Dam could cause extensive shallow flooding in the northern and western portions of Long Beach (City of Long Beach, 1988). In addition, failure of the Whittier Narrows Dam, when filled to capacity, could cause flooding along the easterly and westerly perimeter areas of the city of Long Beach.

The potential for seismically-induced flooding within the boundaries of the city of Long Beach is addressed in Section 6.7 of the Seismic Safety Element of the City's General Plan. That section includes a low-resolution map showing the limits of anticipated maximum flood inundation based on an assumed breach of the afore-mentioned major upstream flood control facilities. Based on our review of that map, no portion of the subject property is located within any of the established seismically-induced flood inundation limits.

Flooding Not Related to Seismicity

As part of this investigation, we conducted an independent review of the applicable FEMA flood insurance rate map for the area of the subject site (Map No. 06037C1955F, effective September 26, 2008). This map indicates that the site of the proposed construction is located within an area that is designated as "Zone X." This designation corresponds to an area that is located within the 0.2 percent annual chance floodplain, an area with a 1 percent annual chance flood with an average depth of less than 1 foot or with a drainage area of less than 1 square mile, and areas protected by levees from the 1 percent annual chance flood.

Strong Ground Motion

The site is located in a seismically active area of southern California and will likely be subjected to strong seismically-related ground shaking during the anticipated life span of the project. Structures within the site should therefore be designed and constructed to resist the effects of strong ground motion in accordance with the most current edition of the California Building Code (CBC).

CONCLUSIONS AND RECOMMENDATIONS

General Feasibility

From a soils engineering and engineering geologic point of view, the subject property is considered suitable for the proposed development provided the following recommendations are incorporated into the design criteria and project specifications. In addition, the proposed grading and construction are not expected to affect the stability of adjoining properties in an adverse manner provided grading and construction are performed in accordance with current standards of practice, all applicable grading ordinances, and the recommendations presented in this report.

Grading Plan Review

This report has been prepared without the aid of a finalized grading plan, a foundation plan, or detailed specifications pertaining to the proposed grading and construction. In the absence of these documents, the proposed construction concept has been assumed based on the preliminary grading plan that was provided to our firm for purposes of this investigation. **As such, the recommendations provided in this report should be considered tentative until a finalized precise grading plan and foundation plan are available and reviewed by our firm.** Additional recommendations and/or modification of the recommendations provided herein might be necessary depending upon the results of our precise grading and foundation plan review.

Primary Geotechnical Considerations

There are several geotechnical conditions inherent to the property that may adversely impact the gross stability of the building sites if not mitigated as part of site grading and building design. These adverse conditions are discussed in the following paragraphs.

Compressible Near-Surface Soils

Our laboratory program included selective testing of representative samples of onsite fill and native soils for in-place dry density and compressibility. The results of these tests indicate that near-surface soils are subject to variable degrees of compressibility under saturated conditions. Due to the inherent nature of pressure distribution through particulate media (e.g., soils), the compressibility of the near-surface soil layers is expected to contribute significantly to the overall foundation settlement. Therefore, in order to reduce the potential for building distress as a result of excessive differential settlement, it is recommended that the existing fill and near-surface alluvial soils in all proposed structural areas be overexcavated and replaced as property compacted fill in order to provide

a more uniform bearing condition. Recommendations for remedial grading are provided in the "Earthwork and Grading" section of this report.

Seismically-Induced Settlement and Related Surface Effects

The presence of granular soil layers, together with a shallow historic high groundwater level and nearby fault systems, which are considered capable of causing strong ground motion, render the site susceptible to liquefaction and dynamic settlement. However, given the fact that maximum estimated dynamic angular distortion ratio (approximately 1:240) is within commonly accepted construction tolerances for contemporary residential foundation systems, it is the opinion of our firm that the potential detrimental effects of liquefaction-induced differential settlement can be reduced to a less than significant level for engineering purposes through the use of properly designed and constructed, post-tensioned or strengthened conventional concrete foundation systems for the proposed dwelling structures.

Foundations for residential structures may lose a portion of the available bearing capacity during a strong seismic event that results in surface manifestation of liquefaction; however, it is the opinion of this firm that the detrimental effects of potential bearing failure can also be reduced to a less than significant level through proper remedial grading combined with the use of a properly designed post-tensioned or strengthened conventional concrete foundation system. Specific recommendations for site grading and building foundation design should be provided in the comprehensive design-phase geotechnical report.

Boundary Conditions

Maximum remedial removal depths within the subject site are anticipated to be approximately 4 feet below the existing ground surface. Based on the relatively non-cohesive nature of on-site soils, temporary backcut slopes adjacent to these boundaries will generally be restricted to a slope ratio of 1:1 (horizontal to vertical) or flatter. If encroachment into these adjacent properties is not possible during grading, a relatively narrow wedge of compressible soil will be left in place along the tract perimeter that will extend into the site to a horizontal distance equal to the vertical depth of the required remedial removals. If unsuitable soils are left in place, some degree of distress may result to the proposed improvements if they are constructed within the zone of influence of these unsuitable soils (generally regarded as a 1:1 projection upward from the bottom of the temporary backcut slope). In areas where existing offsite structures are to be protected in place, temporary backcut slopes should start at a minimum distance of 2 feet from the edge of existing footings.

If unsuitable soils are left in place, some degree of distress may result to improvements proposed along the boundaries that will be located within the zone of influence of these soils. Considering the location of the proposed dwelling structures as shown on the conceptual site plan, the foundations of these structures are anticipated to be located beyond the zone of influence of unsuitable soils that will be left in-place. However, the foundations for masonry block walls that may be proposed along the property boundaries may be underlain by unsuitable soils left in place along the property boundaries. Specific recommendations for these conditions are provided in the subsequent sections of this report.

Strong Ground Motion

The subject site is located in a seismically active area of southern California. The type and magnitude of seismic hazards that may affect the site are dependent on both the distance to causative faults and the intensity and duration of the seismic event. Although the probability of primary surface rupture is considered low, ground shaking hazards posed by earthquakes occurring along regional active faults do exist and should be taken into account in the design and construction of the proposed facilities within the subject site.

Subsurface Obstructions

Buried pipelines associated with present and previous utility services may exist within the areas of proposed grading and construction. In addition, other unknown subsurface structures may be encountered during grading. All surface and buried structures encountered during grading within the limits of proposed grading and construction should be removed in their entirety, and the resulting cavities backfilled as described in the "Earthwork and Grading" section of this report.

Earthwork and Grading

General Specifications

All earthwork should be performed in accordance with current industry standards of practice in the area, with all applicable requirements of the City of Long Beach municipal code, as well as with current standards of practice in the industry and the recommendations provided in this report.

Site Clearing

All structural materials associated with the existing buildings and appurtenant exterior improvements (including the existing swimming pool, hardscape features and buried utilities) should be demolished and removed from the

site. Clearing operations should also include the removal of all landscape vegetation. Trees and large shrubs, when removed, should be grubbed out to include their stumps and major root systems. During site grading, laborers should be provided to clear from fill soils any roots, tree branches, and other deleterious materials missed during initial clearing and grubbing operations.

Although none were encountered within the site during the subsurface investigation by our firm, any seepage pits, cisterns, leach lines or similar structures that may exist within the areas of proposed grading and construction should be cleaned out, backfilled with concrete slurry, gravel or clean sand that is jetted into place, and then capped with filter fabric and a minimum of 5 feet of compacted onsite soils. Any concrete septic tanks or leach lines should likewise be excavated and removed from the site.

Our firm should be notified at the appropriate times to observe general clearing operations. Should any unusual soil conditions or buried structures be encountered during demolition operations or grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of our firm for corrective recommendations.

Water Well Abandonment

As described previously in this report, an active water well is located within the central portion of the project site. As part of clearing operations, this well should be located and abandoned in accordance with the requirements of the local oversight agencies including the City of Long Beach Department of Health and Human Services and the Los Angeles County Department of Public Health (as applicable).

Contaminant-Affected Soils

If hydrocarbon-affected soils or soils affected by other potentially hazardous materials are encountered during grading, it is recommended that the earthwork within this area be terminated pending further evaluation by the project environmental consultant.

Processing of Existing Ground

Our subsurface investigation revealed that near-surface soils within the areas of proposed construction exhibit relatively low in-place densities and contain locally abundant organic material (i.e., roots) associated with the existing turf and other landscape vegetation. These soils are subject to compression under the proposed loadings and, if unmitigated, may result in adverse differential settlement beneath the proposed residences and exterior

hardscape features. In addition, it is expected that the near-surface soils will be considerably disturbed during removal of existing site improvements and landscaping. Therefore, in order to create a uniform compacted fill mat across the site and reduce the potential for settlement-related distress to the proposed building foundations and exterior improvements, it is recommended that the surficial soil materials be over-excavated and replaced as properly compacted fill.

Based on field observations and laboratory test results, removal depths on the order of 4 feet below existing grades, or 2 feet below the bottoms of proposed structural footings, **whichever is deeper,** should generally be anticipated. In order to provide adequate support for perimeter improvements such as hardscape, sidewalks and paved streets, the limits of overexcavation and recompaction should essentially extend from tract boundary to tract boundary (exclusive of park areas where no rigid, settlement-sensitive improvements are proposed); however, consideration should be given to the protection of adjacent offsite improvements such as existing walls to be protected in place, sidewalks and active underground utilities. Remedial grading and ground preparation should be performed prior to placing any new fills.

It must be emphasized that the anticipated depths of remedial grading provided in the above paragraph are estimates only and are based on conditions observed at the boring locations. Subsurface conditions can and usually do vary between points of exploration. For this reason, the actual removal depths will have to be determined on the basis of in-grading observations and testing performed by a representative of the project geotechnical consultant.

Excavation Characteristics

Based on the results of our subsurface investigation, the existing fill and native materials within the site are expected to be readily excavatable using conventional earthmoving equipment; however, as indicated on the exploration logs, soils exhibiting relatively high moisture contents and relatively low in-place densities were encountered at depths as shallow as 6.5 feet below existing grades during our subsurface investigation. Therefore, since remedial excavations extending to a depth of approximately 4 feet below existing grades will be required, excavation and soil compaction may become difficult using conventional rubber-tired earthmoving equipment. Under these conditions, track-mounted equipment may be required.

Stability of Temporary Excavation Sidewalls

During remedial grading of the site, temporary excavations with sidewalls varying up to approximately 4 feet in height are expected to be created. Based on the non-cohesive nature of near-surface soils within the site,

temporary backcut slopes adjacent to the property boundaries will generally be restricted to a slope ratio of 1:1 or flatter. In areas where existing offsite structures are to be protected in place, temporary backcut slopes should be maintained at a minimum distance of 2 feet from the edge of existing footings.

Temporary excavation sidewalls that are cut to the above configurations are expected to remain sufficiently stable during grading; however, all temporary excavations should be observed by a representative of the project geotechnical consultant for any evidence of potential instability. Depending upon the results of these observations, revised temporary slope configurations may become necessary. Other factors that should be considered with respect to the stability of temporary excavation sidewalls include construction traffic and storage of materials on or near the tops of the slopes, construction scheduling, and weather conditions at the time of construction. All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should also be followed. No temporary excavations along the property lines should be left open without proper protections to mitigate safety hazards. **The grading contractor is solely responsible for ensuring the safety of construction personnel and the general public.**

Mitigation of Boundary Conditions

Where the horizontal limits of remedial grading are constrained by existing offsite improvements, it is likely that a narrow wedge of compressible soil material will be left in place along the perimeter tract boundaries that will extend into the site to a horizontal distance equal to the vertical depth of remedial grading. If unsuitable soils are left in place, some degree of distress may result to the proposed perimeter improvements if they are constructed within the zone of influence of these soils (generally regarded as a 1:1 projection from the outside bottom edge of these soils up to the surface). Taking into account the anticipated depth of remedial grading, the zone of influence if these materials may be on the order of 7 feet wide as measured inward from the site perimeter.

To facilitate removals and protect offsite improvements along the tract boundary, slot-cutting techniques may be used by the grading contractor. Alternatively, or in conjunction with slot cutting, deepened foundations or shallow caissons could be used to transfer the foundation load of the proposed structures or walls below the projected influence zone of the unsuitable soil that is left in place.

In areas where foundations for proposed tract perimeter walls will be underlain entirely by compacted fill soils but the lateral extent of remedial grading is limited due to perimeter constraints, reduced allowable bearing pressure and passive pressure may be utilized to provide adequate foundation support without the use of deepened

foundations. The actual width of the zone of influence of unsuitable soil to be left in place depends on the required depth of removals during rough grading and should be evaluated on a lot-by-lot basis. Based on this evaluation, the appropriate mitigative measures for improvements within the zone of influence should be provided after completion of rough grading and during the geotechnical review of the precise grading plans.

Fill Placement

Following removal of unsuitable surficial materials, exposed bottom surfaces in areas approved for placement of fill should be first scarified to a depth of 6 inches, watered or air dried as necessary to achieve optimum moisture conditions, and compacted to a minimum relative compaction of 90 percent. All fills should be placed in 6- to 8-inch-thick maximum lifts, watered or air dried as necessary to achieve slightly above-optimum (but preferable no more than 2 percent over optimum) moisture conditions, and then compacted to a minimum relative compaction of 90 percent. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with ASTM Test Method D 1557.

Fill Slope Construction and Stability

As previously mentioned, low-height fill slopes may be constructed within the tract to accommodate elevation changes between individual lots, or between specific lots and the adjacent access streets and neighboring residential properties. If planned, slope inclinations should be maintained at 2:1 (horizontal to vertical) or flatter. The finish surfaces of the fill slopes should be compacted to a minimum relative compaction of 90 percent. To achieve this, we recommend that the slopes be overfilled and backrolled during construction and then trimmed back to the compacted inner core. This procedure is typically provides uniformly compacted slope surfaces. Properly maintained fill slopes constructed in accordance with the recommendations presented above are expected to be both grossly and surficially stable and are expected to remain so under normal climatic conditions.

Imported Soils

If imported soils are required to complete the planned grading, these soils should consist of clean materials devoid of rock exceeding a maximum dimension of 12 inches, as well as organics, trash and similar deleterious materials. Imported soils should also exhibit an expansion index no greater than 21. Prospective import soils should be observed, tested and approved by the geotechnical consultant **prior to importing the soils to the site.** It is recommended that the project environmental consultant should also be notified so that they can confirm the suitability of the proposed import material from an environmental standpoint.

Geotechnical Observations and Testing During Grading

A representative of our firm should observe exposed bottom surfaces in each remedial removal area **prior to placing fill** in order to document adequate bearing conditions. In addition, a representative of our firm should be present onsite during grading operations to observe proper placement and adequate compaction of all fills, as well as to document compliance with the other recommendations presented herein.

Volumetric Changes - Bulking, Shrinkage and Subsidence

Volumetric changes in earth quantities will occur when onsite soils are excavated and replaced as properly compacted fill. Based on in-place densities of earth materials encountered during our investigation, a shrinkage factor on the order of 6 to 10 percent may be anticipated. The actual shrinkage that will occur during grading will depend on the average degree of relative compaction achieved. A maximum subsidence of approximately 0.15 feet may be anticipated as a result of the scarification and recompaction of the exposed bottom surfaces within the removal areas.

The above estimates of shrinkage and subsidence are intended for use by project planners in estimating earthwork quantities and should not be considered absolute values. Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that will occur during grading.

Post-Grading Considerations

Utility Trenches

All utility trench backfill should be compacted to a minimum relative compaction of 90 percent. Onsite earth materials cannot be densified adequately by flooding and jetting techniques. Therefore, trench backfill materials should be placed in lifts no greater than approximately 12 inches in thickness, watered or air-dried as necessary to achieve near optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. A representative of the project geotechnical consultant should probe and test the backfills to verify adequate compaction.

As an alternative for shallow trenches where pipe or utility lines may be damaged by mechanical compaction equipment, such as under building floor slabs, imported clean sand having a sand equivalent (SE) value of 30 or greater may be utilized. The sand backfill materials should be watered to achieve near optimum moisture conditions and then tamped into place. No specific relative compaction will be required; however, observation,

probing, and if deemed necessary, testing should be performed by a representative of the project geotechnical consultant to document that an adequate degree of compaction has been achieved.

If clean, imported sand is to be used for backfill of exterior utility trenches, it is recommended that the upper 12 inches of trench backfill materials consist of properly compacted onsite soil materials. This is to mitigate infiltration of irrigation and rainwater into granular trench backfill materials.

Where an exterior and/or interior utility trench is proposed in a direction parallel to a building footing, the bottom of the trench should not extend below a 1:1 (horizontal to vertical) plane projected downward from the bottom edge of the adjacent footing. Where this condition occurs, the adjacent footing should be deepened or the utility constructed and the trench backfilled and compacted prior to footing construction. Where utility trenches cross under a building footing, these trenches should be backfilled with on-site soils at the point where the trench crosses under the footing to reduce the potential for water to migrate under the floor slabs.

Precise Grading and Site Drainage

It is likely that surface drainage systems consisting of sloping concrete flatwork and graded earth swales will be constructed on the subject site to collect and direct all surface water to the adjacent streets. In addition, the ground surface around the proposed buildings should be sloped to provide a positive drainage gradient away from the structures. The purpose of the drainage systems is to prevent ponding of surface water within the level areas of the site and against building foundations and associated site improvements. It is recommended that the following recommendations be implemented during construction:

1. Area drains should be extended into all planters and landscape areas that are located within 10 feet of building foundations, retaining walls, and masonry block walls to mitigate excessive infiltration of water into the foundation soils.

2. Section 1804.3 of the 2013 California Building Code requires that "the ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5 percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall." Further, "swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet (3048 mm) of the building foundation." These provisions fall under the purview of the design civil engineer. However, exceptions to allow modifications to these criteria are provided within the same section of the code as "Where climatic or soil conditions warrant, the slope of the ground away from the building foundations is permitted to be reduced to not less than one unit in 48 units horizontal (a 2 percent slope)." This exemption provision appears to fall under the purview of the Geotechnical Engineer-of-Record.

3. It is our understanding that the state-of-the-practice for projects in various cities and unincorporated areas of Los Angeles County, as well as throughout Southern California, has been to construct earthen slopes at 2 percent gradient away from the foundations and at 1 percent minimum for earthen swale gradients. Structures constructed and properly maintained under those criteria have performed satisfactorily. Therefore, considering the semi-arid climate, site soil conditions and an appropriate irrigation regime, **Petra** considers that the implementation of 2 percent slopes away from the structures and 1 percent swales to be suitable for the subject lots.

4. It should be emphasized that the all surface drainage controls must be properly maintained and unobstructed, and that future improvements not alter established gradients unless replaced with suitable alternative drainage systems. Further, where the flowline of any swale exists within five feet of a building structure, the adjacent footings shall be deepened appropriately to maintain minimum embedment requirements as measured from the flowline elevation of the swale.

5. Concrete flatwork surfaces located within 10 feet of building foundations should be inclined at a minimum gradient of 2 percent away from building foundation and similar structures. Concrete flatwork surfaces located more than 10 feet from building foundations may be inclined at a minimum gradient of 1 percent away from building foundation and similar structures. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations.

6. For the landscape areas, a watering program should be implemented that maintains a uniform, near optimum moisture condition in the soils. Overwatering and subsequent saturation of the soils will cause excessive soil expansion and heave and, therefore, should be avoided. However, allowing the soils to dry out will cause excessive soil shrinkage. As an alternative to a conventional irrigation system, drip irrigation systems are strongly recommended for all planter areas.

7. Although no grading plan is currently available for review, it is assumed that the proposed finished grade elevations around the site perimeter will closely match existing offsite grades. No slopes of significant height are currently anticipated. This, combined with the fact that onsite soils are somewhat cohesive in nature, would preclude substantial soil erosion or loss of topsoil within the developed site. There is the potential for localized erosion during grading operations; however, it is expected that this will be mitigated through the implementation of a Storm Water Pollution Prevention Plan (SWPPP) for the site as required by the oversight agencies.

Ground Acceleration and Seismic Design

Earthquake loads on earthen structures and buildings are a function of ground acceleration which may be determined from the site-specific acceleration response spectrum. To provide the design team with the parameters necessary to construct the site-specific acceleration response spectrum for this project, we used two computer applications that are available on the United States Geological Survey (USGS) website, http://geohazards.usgs.gov/

Specifically, the Design Maps website http://geohazards.usgs.gov/designmaps/us/application.php was used to calculate the ground motion parameters. And, the 2008 PSHA Interactive Deaggregation website http://geohazards.usgs.gov/deaggint/2008/ was used to determine the appropriate earthquake magnitude.

To run the above computer applications, site latitude, longitude, risk category and knowledge of "Site Class" are required. The site class definition depends on the average shear wave velocity, Vs_{30}, within the upper 30 meters (approximately 100 feet) of site soils. A shear wave velocity of 285 meters per second for the upper 100 feet was used for the site based on engineering experience and judgment and the CPT data.

The following table, Table VI, provides parameters required to construct the site-specific acceleration response spectrum based 2013 CBC guidelines. A summary of Code approach and a printout of the computer output are attached in Appendix E.

Table VI
Seismic Design Parameters

Ground Motion Parameters	Reference	Parameter Value	Unit
Latitude (North)	-	33.84194	°
Longitude (West)	-	-118.20085	°
Site Class Definition	Table 20.3-1, ASCE 7-10	D	-
Assumed Risk Category	Table 1604.5, CBC 2013	II	-
M_w - Earthquake Magnitude	Section 1803.5.12.2, CBC 2013	6.75	-
S_s - Mapped Spectral Response Acceleration	Figure 1613.3.1(1), CBC 2013	1.654	g
S_1 - Mapped Spectral Response Acceleration	Figure 1613.3.1(2), CBC 2013	0.613	g
F_a - Site Coefficient	Table 1613.3.3(1), CBC 2013	1.0	-
F_v - Site Coefficient	Table 1613.3.3(2), CBC 2013	1.5	-
S_{MS} - Adjusted Maximum Considered Earthquake Spectral Response Acceleration	Equation 16-37, CBC 2013	1.654	g
S_{M1} - Adjusted Maximum Considered Earthquake Spectral Response Acceleration	Equation 16-38, CBC 2013	0.920	g
S_{DS} - Design Spectral Response Acceleration	Equation 16-39, CBC 2013	1.103	g
S_{D1} - Design Spectral Response Acceleration	Equation 16-40, CBC 2013	0.613	g
T_o - (0.2 S_{D1}/ S_{DS})	Section 11.3, ASCE 7-10	0.111	s
T_s - (S_{D1}/ S_{DS})	Section 11.3, ASCE 7-10	0.556	s
T_L - Long Period Transition Period	Figure 22-12, ASCE 7-10	8	s
F_{PGA} - Site Coefficient	Figure 22-7, ASCE 7-10	1.0	-
PGA_M - Peak Ground Acceleration at MCE [1]	Equation 11.8-1, ASCE 7-10	0.627	g
PGA – Design Level – (0.4 S_{DS} [2])	Equation 11.4-5, ASCE 7-10	0.44	g
C_{RS} - Short Period Risk Coefficient	Figure 22-17, ASCE 7-10	0.966	-
C_{R1} - Long Period Risk Coefficient	Figure 22-18, ASCE 7-10	0.984	-
Seismic Design Category [3]	Section 1613.3.5, CBC 2013	D	-

[1] PGA Calculated at the MCE return period of 2475 years (2 percent chance of exceedance in 50 years).

[2] PGA Calculated at the Design Level of 2/3 of MCE which is approximately equivalent to a return period of 475 years (10 percent chance of exceedance in 50 years).

[3] Seismic Design Category may be calculated by the structural engineer in accordance with the alternate design procedures of Section 1613.3.5.1 based on structural characteristics in addition to the ground motion parameters, this may supersede the category listed herein.

References: USGS Seismic Design Web Application – http://geohazards.usgs.gov/designmaps/us/application.php
USGS 2008 Interactive Deaggregation Tool - https://geohazards.usgs.gov/deaggint/2008/

Allowable Bearing Capacity, Estimated Settlement and Lateral Resistance

Allowable Soil Bearing Capacity

Continuous Footings

An allowable soil bearing capacity of 1,500 pounds per square foot may be utilized for design of continuous footings founded at a minimum depth of 12 inches below the lowest adjacent final grade. This value may be increased by 20 percent for each additional foot of depth and by 10 percent for each additional foot of width, to a maximum value of 2,500 pounds per square foot. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for short duration wind and seismic forces.

Pad Footings

An allowable soil bearing capacity of 1,500 pounds per square foot may be utilized for design of isolated 24-inch-square footings founded at a minimum depth of 12 inches below the lowest adjacent final grade. This would apply to pad footings that are not a part of the building slab system and which are intended for support of such features as roof overhangs, second-story decks, patio covers, etc. This value may be increased by 20 percent for each additional foot of depth and by 10 percent for each additional foot of width, to a maximum value of 2,500 pounds per square foot. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for short duration wind and seismic forces.

Footing Settlement Estimates

Based on the allowable bearing values provided above, total static settlement of the footings under the anticipated loads is expected to be on the order of 1 inch. Static differential settlement is expected to be less than ¾ of an inch over a horizontal span of 40 feet. The majority of settlement is likely to take place as footing loads are applied or shortly thereafter.

It should be noted that the settlement estimates provided above do not take into consideration the settlement that may occur as a result of seismically-induced liquefaction. As was stated previously in this report, the results of our site-specific study indicate that the maximum estimated total dynamic vertical settlement for the site is approximately 2½ inches. The maximum dynamic differential settlement between exploratory points was estimated to be approximately 1½ inches over a horizontal span of approximately 40 feet, with a corresponding equivalent angular distortion ratio of less than 1:240. The project architect and/or structural engineer should determine whether these seismically-induced settlement values should be considered as additive to the static settlement estimates provided in the previous paragraph.

Lateral Resistance

A passive earth pressure of 150 pounds per square foot per foot of depth, to a maximum value of 1,500 pounds per square foot, may be used to determine lateral bearing resistance for footings. In addition, a coefficient of friction of 0.25 times the dead load forces may be used between concrete and the supporting soils to determine lateral sliding resistance. The above values may be increased by one-third when designing for transient wind or seismic forces. It should be noted that the above values are based on the condition where footings are cast in direct contact with compacted fill or competent native soils. In cases where the footing sides are formed, all backfill placed against the footings upon removal of forms should be compacted to at least 90 percent of the applicable maximum dry density.

Footing and Slab-on-Grade Design and Construction Considerations

Given the granular nature of near-surface soils within the site, it is likely that these materials exhibit expansion potentials that are within the Very Low range (Expansion Index from 0 to 20). As such, the design of slabs-on-grade is considered to be exempt from the procedures outlined in Sections 1803.5.3 and 1808.6.2 of the 2013 CBC and may be performed using any method deemed rational and appropriate by the project structural engineer. However, given the potential for earthquake-induced liquefaction and associated settlement at the site, the following minimum guidelines are presented for design and construction of footings and slabs on-grade the project site.

The design and construction recommendations that follow are based on the above soil conditions and may be considered for reducing the effects of variability in composition and behavior within the site soils and long-term differential settlement. These recommendations have been developed on the basis of the previous experience of this firm on projects with similar soil conditions. Although construction performed in accordance with these recommendations has been found to reduce post-construction movement and/or distress, they generally do not positively eliminate all potential effects of variability in soils characteristics and future settlement.

It should also be noted that the recommendations for reinforcement provided herein are performance-based and intended only as guidelines to achieve adequate performance under the anticipated soil conditions. The project structural engineer, architect and/or civil engineer should make appropriate adjustments to reinforcement type, size and spacing to account for internal concrete forces (e.g., thermal, shrinkage and expansion) as well as external forces (e.g., applied loads) as deemed necessary. Consideration should also be given to minimum design criteria as dictated by local building code requirements.

Strengthened Conventional Slabs-on-Grade System

Given the potential for liquefaction-induced total and differential settlement within the site, we recommend that footings and floor slabs be designed and constructed in accordance with the following minimum criteria.

Footings

1. Exterior continuous footings supporting one- and two-story structures should be founded at a minimum depth of 18 inches below the lowest adjacent final grade. Interior continuous footings may be founded at a minimum depth of 12 inches below the tops of the adjacent finish floor slabs.

2. All continuous footings should have minimum widths of 12 and 15 inches for one-story and two-story construction, respectively. All continuous footings should be reinforced with a minimum of four No. 4 bars, two top and two bottom.

3. A minimum 12-inch-wide grade beam founded at the same depth as adjacent footings should be provided across garage entrances or similar openings (such as large doors or bay windows). The grade beam should be reinforced in a similar manner as provided above.

4. Interior isolated pad footings, if required, should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the bottoms of the adjacent floor slabs. Pad footings should be reinforced with a minimum of No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings.

5. Exterior isolated pad footings intended for support of roof overhangs such as second-story decks, patio covers and similar construction should be a minimum of 24 inches square, and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings. Exterior isolated pad footings may need to connect to adjacent pad and/or continuous footings via tie beams at the discretion of the project structural engineer

6. The spacing and layout of the interior concrete grade beam system required below floor slabs should be determined by the project architect or structural engineer in accordance with the WRI publication using the effective plasticity index value provided previously.

7. The minimum footing dimensions and reinforcement recommended herein may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2013 CBC) by the structural engineer responsible for foundation design based on his/her calculations and engineering experience and judgment.

Building Floor Slabs

1. Concrete floor slabs should be a minimum 4 inches thick and reinforced with a minimum No. 3 bars spaced a maximum of 18 inches on centers, both ways. All slab reinforcement should be continued and bent into the footings and supported on concrete chairs or brick to ensure the desired placement near mid-depth.

2. Living area concrete floor slabs should be underlain with a moisture vapor retarder consisting of a minimum 10-mil-thick polyethylene or polyolefin membrane that meets the minimum requirements of ASTM E96 and ASTM E1745 for vapor retarders (such as Husky Yellow Guard®, Stego® Wrap, or equivalent). All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch-thick leveling course of sand across the pad surface prior to the placement of the membrane.

   At the present time, some slab designers, geotechnical professionals and concrete experts view the sand layer below the slab (blotting sand) as a place for entrapment of excess moisture that could adversely impact moisture-sensitive floor coverings. As a preventive measure, the potential for moisture intrusion into the concrete slab could be reduced if the concrete is placed directly on the vapor retarder. However, if this sand layer is omitted, appropriate curing methods must be implemented to ensure that the concrete slab cures uniformly. A qualified materials engineer with experience in slab design and construction should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing. Additional steps would also need to be taken to prevent puncturing of the vapor retarder during concrete placement.

3. Garage floor slabs should be a minimum 4 inches thick and reinforced in a similar manner as living area floor slabs. Garage slabs should also be poured separately from adjacent wall footings with a positive separation maintained using ¾-inch-minimum felt expansion joint materials. To control the propagation of shrinkage cracks, garage floor slabs should be quartered with weakened plane joints.

4. Prior to placing concrete, the subgrade soils below living area floor slabs should be prewatered to achieve a moisture content that is at least 1.3 times the optimum moisture content. This moisture should penetrate to a depth of approximately 18 inches into the subgrade.

5. The minimum dimensions and reinforcement recommended herein for building floor slabs may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2013 CBC) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience and judgment.

Post-Tensioned Slab-on-Grade System

Given the very low expansion potential exhibited by onsite soils, any rational and appropriate procedure may be chosen by the project structural engineer for the design of post-tensioned slabs-on-grade; however, consideration should be given to the potential for earthquake-induced soil liquefaction and associated total and differential settlement. Should the design engineer choose to follow the most current procedure published by the Post-Tensioning Institute (PTI), the minimum design criteria are provided in Table VII.

Table VII
Post-Tensioned Slab-on-Grade Design Parameters for PTI Procedure
Very Low Expansion Potential

Soil Information	
Approximate Depth of Constant Suction, feet	9
Approximate Soil Suction, pF	3.9
Inferred Thornthwaite Index:	-20
Average Edge Moisture Variation Distance, e_m in feet: Center Lift Edge Lift	 9.0 4.7
Anticipated Swell, y_m in inches: Center Lift Edge Lift	 0.20 0.40

Modulus of Subgrade Reaction

The modulus of subgrade reaction for design of load bearing partitions may be assumed to be 125 pounds per cubic inch.

Minimum Design Recommendations

The soil values provided above may be utilized by the project structural engineer to design post-tensioned slabs-on-ground in accordance with Section 1808.6.2 of the 2013 CBC and the PTI publication. Thicker floor slabs and larger footing sizes may be required for structural reasons and should govern the design if more restrictive than the minimum recommendations provided below:

1. Perimeter footings for both one-story and two-story structures should be founded at a minimum depth of 15 inches below the lowest adjacent finished ground surface. Interior footings may be founded at a minimum depth of 12 inches below the tops of the finish floor slabs. All continuous footings should be reinforced with a minimum of four No. 4 bars, two top and two bottom. Alternatively, post-tensioned tendons may be utilized in perimeter continuous footings in lieu of the reinforcement bars.

2. A 12-inch-wide grade beam founded at the same depth as adjacent footings should be provided across the garage entrances or similar openings (such as large doors or bay windows). The grade beam should be reinforced in a similar manner as provide above.

3. Exterior isolated pad footings intended for support of roof overhangs such as second-story decks, patio covers and similar construction should be a minimum of 24 inches square, and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings. Exterior isolated pad footings may need to connect to adjacent pad and/or continuous footings via tie beams at the discretion of the project structural engineer.

4. The thickness of the floor slabs should be determined by the project structural engineer with consideration given to the expansion potential of the on-site soils; however, we recommend that a minimum slab thickness of 4 inches to be considered.

5. As an alternative to designing 4-inch-thick post-tensioned slabs with perimeter footings as described in Items 1 and 2 above, the structural engineer may design the foundation system using a thickened slab design. The minimum thickness of this uniformly thick slab should be 10 inches. The engineer in charge of post-tensioned slab design may also opt to use any combination of slab thickness and footing embedment depth as deemed appropriate based on their engineering experience and judgment.

6. Living area concrete floor slabs should be underlain with a moisture vapor retarder consisting of a minimum 10-mil-thick polyethylene or polyolefin membrane that meets the minimum requirements of ASTM E96 and ASTM E1745 for vapor retarders (such as Husky Yellow Guard®, Stego® Wrap, or equivalent). All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch-thick leveling course of sand across the pad surface prior to the placement of the membrane.

   At the present time, some slab designers, geotechnical professionals and concrete experts view the sand layer below the slab (blotting sand) as a place for entrapment of excess moisture that could adversely impact moisture-sensitive floor coverings. As a preventive measure, the potential for moisture intrusion into the concrete slab could be reduced if the concrete is placed directly on the vapor retarder. However, if this sand layer is omitted, appropriate curing methods must be implemented to ensure that the concrete slab cures uniformly. A qualified materials engineer with experience in slab design and construction should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing. Additional steps would also need to be taken to prevent puncturing of the vapor retarder during concrete placement.

7. Presaturation of the subgrade below floor slabs will not be required; however, prior to placing concrete, the subgrade below all dwelling and garage floor slab areas should be thoroughly moistened to achieve a moisture content that is at least equal to or slightly greater than optimum moisture content to a minimum depth of 18 inches below the bottoms of the slabs.

8. The minimum footing dimensions and reinforcement recommended herein may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2013 CBC) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience and judgment.

General Corrosivity Screening

As a screening level study, limited chemical and electrical tests were performed on representative samples of onsite soils to identify potential corrosive characteristics of these soils. The following sections present the test results and an interpretation of current codes and guidelines that are commonly used in our industry as they relate

to the adverse impact of chemical contents of the site soils and their associated moisture on various components of the proposed structures in contact with site soils.

A variety of test methods are available to quantify corrosive potential of soils for various elements of construction materials. Depending on the test procedures adopted, characteristics of the leachate that is used to extract the target chemicals from the soils and the test equipment; the results can vary appreciably for different test methods in addition to those caused by variability in soil composition. The testing procedures referred to herein are considered to be typical for our industry and have been adopted and/or approved by many public or private agencies. In drawing conclusions from the results of our chemical and electrical laboratory testing and providing mitigation guidelines to reduce the detrimental impact of corrosive site soils on various components of the structure in contact with site soils, heavy references were made to 2013 CBC and American Concrete Institute, 2011 Structural Concrete Building Code (ACI 318-11). Where relevant information was not available in these codes, references were made to guidelines developed by California Department of Transportation (Caltrans), mainly because their risk tolerance for highway bridges are considered comparable to those for residential or commercial structures and that Post Tensioning Institute (PTI), in part, accepts and uses Caltrans' relevant corrosivity criteria for post-tensioned slabs on-grade.

It should be noted that **Petra** does not practice corrosion engineering; therefore, the test results, opinion and engineering judgment provided herein should be considered as general guidelines only. Additional analyses would be warranted, especially, for cases where buried metallic building materials (such as copper and cast or ductile iron) in contact with site soils are planned for the project. In many cases, the project geotechnical engineer is not informed of these choices. Therefore, for conditions where such elements are considered, we recommend that the project design professionals (i.e., the architect and/or structural engineer) consider recommending a qualified corrosion engineer to conduct additional sampling and testing of near-surface soils during the final stages of site grading to provide a complete assessment of soil corrosivity. Recommendations to mitigate the detrimental effects of corrosive soils on buried metallic and other building materials that may be exposed to corrosive soils should be provided by the corrosion engineer as deemed appropriate.

Concrete in Contact with Site Soils

Soils containing soluble sulfates beyond certain threshold levels as well as acidic soils are considered to be detrimental to long-term integrity of concrete placed in contact with such soils. For the purpose of this study, soluble sulfates (SO_4) concentration in soils determined in accordance with California Test Method No. 417. The

soil soluble sulfate severity rating is adopted from ACI 318 publication. Soil acidity, as indicated by hydrogen-ion concentration (pH), was determined in accordance with California Test Method No. 643. The soil acid severity rating is adopted from The United States Department of Agriculture, Natural Resources Conservation Service classification.

The results of our limited in-house laboratory tests indicate that on-site soils contain a water-soluble sulfate content of 0.03 percent by weight. Based on Article 1904.1 of Section 1904 of the 2013 CBC, concrete that will be exposed to sulfates in site soil should be assigned exposure classes in accordance with the durability requirements of ACI 318.

Based on the test results and in reference to Table 4.2.1 of ACI 318-11, an exposure class of **S0** is appropriate for onsite soils. Accordingly, a severity level of **Not Applicable** for exposure to sulfate may be expected for concrete placed in contact with the onsite soil materials. As such, Table 4.3.1 of ACI 318-11 provides that no restriction for cement type or maximum water-cement ratio for the fresh concrete would be required. However, this table indicates that the concrete minimum unconfined compressive strength should not be less than 2,500 psi.

The results of limited in-house testing of representative samples indicate that soils within the subject site are neutral with respect to pH (a pH of 7.2). Based on this finding and according to Section 8.22.2 of Caltrans' 2003 Bridge Design Specifications (2003 BDS) requirements (which consider the combined effects of soluble sulfates and soil pH), a commercially available Type V or Type II Modified cement may be used.

The guidelines provided herein should be evaluated and confirmed, or modified, in its entirety by the project structural engineer and the contractor responsible for concrete placement for concrete used in exterior and interior footings, interior slabs on-ground, garage slabs walls foundation and concrete exposed to weather such as driveways, patios, porches, walkways, ramps, steps, curbs, etc.

Metals Encased in Concrete

Soils containing a soluble chloride concentration beyond a certain threshold level are considered corrosive to metallic elements such as reinforcement bars, tendons, cables, bolts, etc. that are encased in concrete that, in turn, is in contact with such soils. For the purpose of this study, soluble chlorides (Cl) in soils were determined in accordance with California Test Method No. 422.

Based on Article 1904.1 of Section 1904 of the 2013 CBC, concrete that will be exposed to chlorides from "*deicing chemicals, salt, saltwater, brackish water, seawater or spray from these sources, where concrete has steel reinforcement*" should be assigned exposure classes in accordance with the durability requirements of ACI 318. Table 4.2.1 of ACI 318-11 states that an exposure class of **C0** with a severity designation of **Not Applicable** is appropriate for reinforced concrete that remains dry or protected from moisture. Similarly, an exposure class of **C1** with a severity designation of **Moderate** is appropriate for reinforced concrete that is exposed to moisture but not to external sources of chlorides. And, lastly, an exposure class of **C2** with a severity designation of **Severe** is appropriate for reinforced concrete that is exposed to moisture and external sources of chlorides as enumerated above.

Based on our understanding of the project, it is our professional opinion that an exposure class of **C1** with a severity designation of **Moderate** is appropriate for a majority of reinforced concrete, to be placed at the site, that are in contact with site soils. It should be noted, however, that an exposure class of **C2** with a severity designation of **Severe** is more appropriate for reinforced concrete that is planned for pool walls and decking, should such features be considered for the project.

The results of our limited laboratory tests performed indicate that onsite soils contain a water-soluble chloride concentration of 83 parts per million (ppm). Article 1904.2 of Section 1904 of the 2013 CBC requires that concrete mixtures conform to the most restrictive maximum water-cementitious material ratios, maximum cementitious admixture, minimum air-entrainment and minimum specified concrete compressive strength requirements of ACI 318 based on the exposure classes assigned in Article 1904.1. No maximum water/cement ratio for the fresh concrete is prescribed by ACI 318 for class **C1** (or **Moderate** severity) exposure condition. However, Table 4.3.1 of ACI 318-11indicates that concrete minimum unconfined compressive strength, f'_c, should not be less than 2,500 psi. For class **C2** (or **Severe**) exposure condition, Table 4.3.1 of ACI 318-11requires that the maximum water/cement ratio of the fresh concrete should not exceed 0.40 and concrete minimum unconfined compressive strength, f'_c, should not be less than 5,000 psi.

Metallic Elements in Contact with Site Soils

Elevated concentrations of soluble salts in soils tend to induce low level electrical currents in metallic objects in contact with such soils. This process promotes metal corrosion and can lead to distress to building metallic components that are in contact with site soils. The minimum electrical resistivity measurement provides a simple indication of relative concentration of soluble salts in the soil and, therefore, is widely used to estimate soil

corrosivity with regard to metals. For the purpose of this investigation, the minimum resistivity in soils is measured in accordance with California Test Method No. 643. The soil corrosion severity rating is adopted from the Handbook of Corrosion Engineering by Pierre R. Roberge.

The minimum electrical resistivity for onsite soils was found to be 2,500 ohm-cm based on limited testing. The result indicates that on-site soils are **Highly Corrosive** to ferrous metals and copper. As such, any ferrous metal or copper components of the subject buildings (such as cast iron or ductile iron piping, copper tubing, etc.) that are expected to be placed in direct contact with site soils should be protected against detrimental effects of highly corrosive soils based on recommendations provided by a qualified corrosion engineer.

Masonry Block Screen Walls

Where there is sufficient space to perform remedial grading beyond the property boundaries, the footings for the masonry block screen walls may be designed in accordance with the bearing and lateral resistance values provided previously for building footings. However, where remedial grading cannot encroach into the adjacent properties, a reduced bearing value of 1,200 pounds per square foot may be used for 12-inch-wide continuous footings founded at a minimum depth of 12 inches below the lowest adjacent final grade. No increase in bearing value may be used for wider or deeper footings for this condition. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for short duration wind and seismic forces. In addition, a reduced passive earth pressure of 100 pounds per square foot per foot of depth, to a maximum value of 1,000 pounds per square foot, may be used to resist lateral loads.

A coefficient of friction of 0.30 times the dead load forces may still be used between concrete and the supporting soils to determine lateral sliding resistance. The above values may be combined without reduction provided the lateral sliding resistance does not exceed one-half the dead load. An increase of one-third of the above values may also be used when designing for short duration wind or seismic forces.

As a minimum, the wall footings should be embedded at a minimum depth of 12 inches below the lowest adjacent final grade. The footings should also be reinforced with a minimum of two No. 4 bars, one top and one bottom. In order to reduce the potential for unsightly cracking related to the possible effects of differential settlement and/or expansion, positive separations (construction joints) should also be provided in the block walls at each corner and at horizontal intervals of approximately 20 to 25 feet. The separations should be provided in the blocks and not extend through the footings. The footings should be poured monolithically with continuous rebars to serve as effective "grade beams" below the walls.

Retaining Wall Design Recommendations

Allowable Bearing and Lateral Resistance

Retaining wall footings or masonry block screen wall footings retaining soil may be designed using the allowable bearing and lateral resistance values recommended previously for design of masonry block walls; however, when calculating the passive resistance, the resistance of the upper six inches of supporting soils should be ignored in areas where the footings will not be covered with concrete flatwork, or where the thickness of soil covering the footings is less than 12 inches.

Active and At-Rest Earth Pressures

As of the date of this report, it is uncertain whether retaining walls will be backfilled with on-site soils or imported granular materials. For this reason, active and at-rest pressures are provided below for both conditions.

1. Onsite Soils Used for Wall Backfill

   Onsite earth materials have a low expansion potential. Therefore, if these onsite materials are used as backfill, active earth pressures equivalent to fluids having densities of 40 and 63 pounds per cubic foot should be used for design of cantilevered walls retaining a level backfill and ascending 2:1 backfill, respectively. For walls that are restrained at the top, at-rest earth pressures of 60 and 95 pounds per cubic foot (equivalent fluid pressures) should be used. The above values are for retaining walls that have been supplied with a proper subdrain system (see Figure RW-1). All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the active and at-rest earth pressures.

2. Imported Sand, Pea Gravel, or Rock Used for Wall Backfill

   Imported clean sand exhibiting a sand equivalent value (SE) of 30 or greater, pea gravel, or crushed rock may be used for wall backfill, to reduce the lateral earth pressures provided these granular backfill materials extend behind the walls to a minimum horizontal distance equal to one-half the wall height. In addition, the sand, pea gravel, or rock backfill materials should extend behind the walls to a minimum horizontal distance of 2 feet at the base of the wall or to a horizontal distance equal to the heel width of the footing, whichever is greater (see Figures RW-2 and RW-3). For the above conditions, cantilevered walls retaining a level backfill and ascending 2:1 backfill may be designed to resist active earth pressures equivalent to fluids having densities of 30 and 41 pounds per cubic foot, respectively. For walls that are restrained at the top, at-rest earth pressures equivalent to fluids having densities of 45 and 62 pounds per cubic foot are recommended for design of restrained walls supporting a level backfill and ascending 2:1 backfill, respectively. These values are also for retaining walls supplied with a proper subdrain system. Furthermore, as with native soil backfill, the walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the recommended active and at-rest earth pressures.

It is recommended that retaining wall plans and structural details be provided to this firm for review prior to commencement of the grading and construction phases of the project.

Earthquake Loads on Retaining Walls

For sites that fall under Seismic Design Categories D, E and F, Section 1803.5.12 of the 2013 CBC requires the determination of seismically-induced lateral loads on retaining walls that will support more than 6 feet of backfill. It should be noted that, although both the 2013 CBC and 2009 IBC can be interpreted as requiring seismic design for these retaining walls, many municipalities consider seismic design for retaining walls within residential projects to be overly conservative. As a result, some building code subcommittees are enforcing revised policies. For example, the City of Los Angeles requires that only walls greater than 12 feet high need to be designed for lateral earthquake loads. Similarly, the County of Los Angeles requires that walls greater than 8 feet high (for non R-3) or 12 feet high (for R-3) shall be designed for lateral earthquake loads. Therefore, the project structural engineer should consult with the City of Long Beach building official to confirm the City's policy regarding new retaining walls that may be proposed within the site.

The 2013 CBC allows that the peak ground acceleration (PGA) may be assumed equal to $S_{DS}/2.5$. This gives a PGA value of 0.44g for this site (1.103g/2.5). This value was used in the Seed and Whitman (1970) simplified calculation for level conditions behind retaining structures. According to the research of Sitar, et al. (2012), the simplified Seed and Whitman calculation is appropriate for use for cantilever retaining walls.

From the County of Los Angeles Department of Public Works Manual for the Preparation of Geotechnical Reports (Dec., 2006), the horizontal ground acceleration value (k_h) for cantilever retaining walls may be assumed to be equal to half of the peak ground acceleration. Thus, $k_h = ½ (a_g) = (0.5) (0.44g) = 0.22g$.

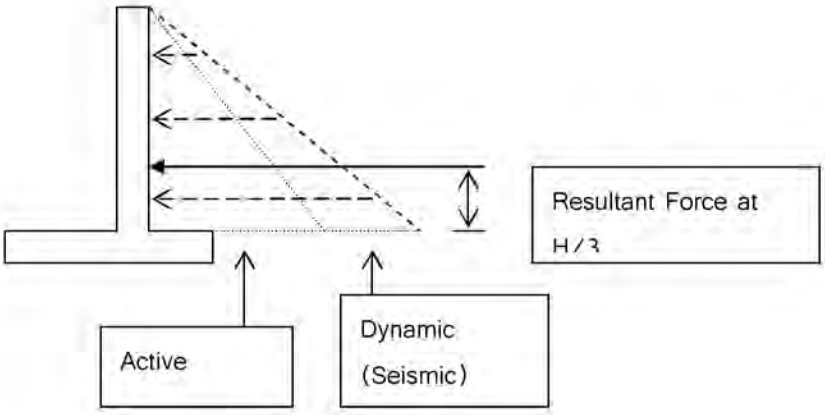
From Seed and Whitman (1970), the lateral load on a retaining structure can be determined by the following equation:

$P_D = \Upsilon (¾) K_h$

where P_D = Dynamic Lateral Earth Pressure,
Υ = weight of soil = 120 pcf, and
K_h = horiz. ground acceleration

thus, $P_D = (120 \text{ pcf}) (¾) (0.22) \approx 20 \text{ pcf}$.

For cantilever retaining walls, Sitar, et al. (2012) indicates that the seismic earth pressures have a triangular distribution with the largest load occurring at the bottom of the wall. The distribution of the seismic lateral load for both types of walls is as follows:



Geotechnical Observation and Testing

All grading and construction phases associated with retaining wall construction, including backcut excavations, footing trenches, installation of the subdrainage systems, and placement of backfill should be observed and tested by a representative of this firm.

Subdrainage

Perforated pipe and gravel subdrains should be installed behind all retaining walls to prevent entrapment of water in the backfill (see Figures RW-1 through RW-3). Perforated pipe should consist of 4-inch-minimum diameter PVC Schedule 40, or SDR-35, with the perforations laid down. The pipe should be encased in a 1-foot-wide column of ¾-inch to 1½-inch open-graded gravel. If on-site soils are used as backfill, the open-graded gravel should extend above the wall footings to a minimum height equal to one-third the wall height or to a minimum height of 1.5 feet above the footing, whichever is greater. If imported sand, pea gravel, or crushed rock is used as backfill, subdrain details shown on Figures RW-2 and RW-3 should be utilized. The open-graded gravel should be completely wrapped in filter fabric consisting of Mirafi 140N or equivalent. Solid outlet pipes should be connected to the subdrains and then routed to a suitable area for discharge of accumulated water.

If a limited area exists behind the walls for installation of a pipe and gravel subdrain, a geotextile drain mat such as Mirafi Miradrain, or equivalent, can be used in lieu of drainage gravel. The drain mat should extend the full height and lengths of the walls and the filter fabric side of the drain mat should be placed up against the backcut. The perforated pipe drain line placed at the bottom of the drain mat should consist of 4-inch minimum diameter PVC Schedule 40 or SDR-35. The filter fabric on the drain mat should be peeled back and then wrapped around the drain line.

Waterproofing

The portions of retaining walls supporting backfill should be coated with an approved waterproofing compound or covered with a similar material to inhibit infiltration of moisture through the walls.

Wall Backfill

Recommended active and at-rest earth pressures for design of retaining walls are based on the physical and mechanical properties of the on-site soil materials. However, since the on-site soil materials are locally silty, they may be difficult to compact when placed in the relatively confined areas located between the walls and temporary backcut slopes. Therefore, to facilitate compaction of the backfill, consideration should be given to using sand, pea gravel, crushed rock, or select imported or onsite granular soils that exhibit a **Very Low** expansion potential (Expansion Index of less than 20) behind the proposed retaining walls. For this condition, the reduced active and at-rest pressures provided previously for sand, pea gravel, or crushed rock backfill may be considered in wall design provided they are installed as shown on Figures RW-2 and RW-3.

Where the onsite soils materials are used as backfill behind the proposed retaining walls, the backfill materials should be placed in approximately 6- to 8-inch-thick maximum lifts, watered as necessary to achieve near optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. Flooding or jetting of the backfill materials should be avoided. A representative of the project geotechnical consultant should observe the backfill procedures and test the wall backfill to document that adequate compaction has been achieved.

If imported pea gravel or rock is used for backfill, the gravel should be placed in approximately 2- to 3-foot-thick lifts, thoroughly wetted but not flooded, and then mechanically tamped or vibrated into place. A representative of the project geotechnical consultant should observe the backfill procedures and probe the backfill to determine that an adequate degree of compaction is achieved.

To mitigate the potential for the direct infiltration of surface water into the backfill, imported sand, gravel, or rock backfill should be capped with at least 12 inches of on-site soil. Filter fabric such as Mirafi 140N or equivalent, should be placed between the soil and the imported gravel or rock to prevent fines from penetrating into the backfill.

Exterior Concrete Flatwork

As is indicated on the exploration logs included in Appendix A, near-surface soils within the site consist of interlayered granular (sandy) and fine-grained soils (silty or clayey) which tend to exhibit variable expansion potentials. Upon completion of precise grading operations within proposed flatwork areas, the exposed subgrade soils will consist of a blend of these materials and thus are likely to exhibit locally variable expansive characteristics. Due to project scheduling constraints, it is not typically feasible to collect additional samples of subgrade soils for testing to verify expansion index immediately prior to pouring concrete. For this reason, it is recommended that all the exterior concrete flatwork including pedestrian sidewalks, vehicular access roads, and large decorative slabs within quad areas be designed by the project architect with consideration given to reducing the potential for cracking and uplift that can develop in expansive soils.

The guidelines that follow are based on the assumption that no time allowance will be made for the collection and testing of verification samples of flatwork subgrade soils prior to placement of steel reinforcement and concrete pouring. These guidelines should be considered as minimums and are subject to review and revision by the project architect, structural engineer and/or landscape consultant as deemed appropriate. If time will be allowed in the project schedule for verification sampling and testing prior to the concrete pour, the test results generated may dictate that a somewhat less conservative design could be used.

Pedestrian Flatwork Areas

To reduce the potential of unsightly cracking related to the effects of locally expansive soils, it is suggested that concrete sidewalks, patio-type slabs and subslabs to be covered with decorative pavers be a minimum of 4 inches thick and provided with saw cuts or expansion joints every 10 feet or less. It is further suggested that concrete slabs and sidewalks be reinforced with a minimum of No. 3 bars spaced 18 inches on centers, both ways. Alternatively, the slab reinforcement may consist of welded wire mesh of the sheet type (not rolled) with 6x6/W2.9xW2.9 designation in accordance with the Wire Reinforcement Institute (WRI). All reinforcement should be positioned near the middle of the slabs by means of concrete chairs or brick. *Note: The minimum reinforcement suggested in this report does not account for thermal and shrinkage characteristics of reinforced concrete. Additional steel reinforcement may be required by the project architect and/or structural engineer to account for these characteristics.*

Edge Beams

Where the outer edges of concrete flatwork are to be bordered by landscaping, it is recommended that consideration be given to the use of edge beams (thickened edges) to prevent excessive infiltration and accumulation of water under the slabs. Edge beams, if used, should be 6 to 8 inches wide, extend 8 inches below the tops of the finish slab surfaces, and be reinforced with a minimum of two No. 4 bars, one top and one bottom. Edge beams are not mandatory; however, their inclusion in flatwork construction adjacent to landscaped areas is expected to significantly reduce the potential for vertical and horizontal movements and subsequent cracking of the flatwork related to the effects of high uplift forces that can develop in expansive soils.

Concrete Driveways and Pavement

Concrete pavement may be desirable at site entry points and trash collection areas. For concrete pavement that will be designed based on an unlimited number of applications of an 18-kip single-axle load, it is suggested that the pavement have a minimum thickness of 6 inches and a minimum 28-day compressive strength of 3,000 pounds per square inch. In addition, the pavement should have a minimum reinforcement of No. 3 bars spaced a maximum of 16 inches on center (each way), and individual slabs should be doweled together at control joints. Control joints should be spaced at maximum 10-foot intervals. A modulus of subgrade reaction of 125 pounds per cubic foot may be used for design purposes. Concrete pavement should be placed on 6 inches of aggregate base compacted to 95 percent relative compaction (ASTM D 1557).

The R-value and traffic index that form the basis of these recommendations have been assumed based on the predominant engineering characteristics of near-surface soils. We recommend that bulk samples of the actual subgrade materials be retrieved for laboratory analysis after rough grading is completed. Once actual as-graded conditions are confirmed, modified design recommendations may be required.

Subgrade Preparation

As a further measure to mitigate cracking and/or shifting of concrete flatwork, the subgrade soils below concrete flatwork areas should be compacted to a minimum relative compaction of 90 percent per ASTM D1557 and then thoroughly moistened prior to placing concrete. Where concrete driveways are proposed, the upper 6 inches of subgrade soil should be compacted to a minimum 95 percent relative compaction. The moisture content of the soils should be at least 1.3 times the optimum moisture content and penetrate to a minimum depth of 12 inches into the subgrade. Flooding or ponding of the subgrade is not considered feasible to achieve the above moisture conditions since this method would likely require construction of numerous earth berms to contain the water.

Therefore, moisture conditioning should be achieved with sprinklers or a light spray applied to the subgrade over a period of several days just prior to pouring concrete. A representative of the project geotechnical consultant should observe and document the density and moisture content of the soils, and the depth of moisture penetration prior to pouring concrete.

Drainage

Drainage from patios and other flatwork areas should be directed to local area drains and/or graded earth swales designed to carry runoff water to the adjacent streets or other approved drainage structure. The concrete flatwork should also be sloped at a minimum gradient of one percent away from building foundations, retaining walls, masonry garden walls and slope areas.

Tree Wells

Tree wells are not recommended in concrete flatwork areas since they introduce excessive water into the subgrade soils or allow root invasion, both of which can cause heave of the flatwork.

Asphalt Pavement Design

Due to the remedial grading that will be performed within the site, the existing near-surface soils will be processed and blended with deeper soils resulting in a compacted fill material that may exhibit considerably different subgrade strength characteristics than those that presently exist at the surface within the site. In addition, imported soils may also be required to establish the proposed subgrade elevations. Therefore, representative samples of the subgrade soils within street areas should be obtained for R-Value testing at or near the completion of grading. A separate letter providing recommendations for structural pavement sections within the site will then be submitted by our firm based on the results of these tests. It should be noted that the City of Long Beach minimum pavement section thickness might supersede the calculated sections provided by our firm.

Field Percolation Testing

Storm Water Infiltration System

It is our understanding that an onsite storm water percolation system may be constructed in order to achieve the required best management practices for the project. To support the design of the proposed storm water dissipation system, this investigation included performing a pilot percolation study within the proposed recreation area at the extreme easterly corner of the site. This study was performed concurrently with the previously

described field exploration program on March 4, 2014 and included drilling one percolation test boring (identified herein as B-2A). The percolation test boring was drilled to an approximate depth of 3 feet below the existing ground surface using a hollow-stem drilling rig. Test data are summarized in Appendix E of this report. The approximate location of the percolation test is shown on Plate 1. **It should be noted that the percolation rate provided below is representative of the location tested and may vary considerably over relatively small horizontal distances. Once the finalized location and depth of the proposed storm water infiltration system have been determined, it is recommended that supplemental testing be performed to assess percolation rates at that specific location.**

Following drilling, a four-inch-diameter (I. D.) perforated P.V.C. pipe was placed in the test hole. Gravel ($^3/_8$- to ¾-inch) was then placed within the annular space between the pipe and boring wall. Clean tap water was subsequently added to the boring to pre-soak the sidewall soils prior to commencement of the percolation testing.

The percolation test was conducted by filling the test boring with clean tap water to a depth of approximately 12 inches below the surface. The water level was measured at approximately 30-second to 10-minute intervals (due to the relatively fast infiltration rate), and then refilled to approximately the initial water elevation. From these readings, the percolation characteristics of the underlying native alluvial soils were estimated. The percolation test results are summarized in Table VIII, below:

Table VIII
Preliminary Percolation Test Data

Test Boring No.	Soil Type (USCS)	Depth of Borehole (feet)	Absorption Rate (gallons/ft^2/day)
B-2A	Sand/(SP)	3.0	95

Summary of Findings and Recommendations

1. One percolation test was conducted for this study to provide preliminary information to support the design of a storm water infiltration system that may be located within the proposed recreation area at the easterly corner of the site.

2. Approximately 2½ feet of artificial fill was encountered at the borehole location. Beneath the fill, native alluvial deposits were encountered to the maximum borehole depth (3 feet).

3. It should be noted that the granular native soils existing below a depth of 2½ feet were targeted for this test. Therefore, the test data provided above is representative of this sandy native soil unit only and should not be used for the design of any storm water infiltration system that does not penetrate into this soil layer.

4. Based on information obtained from this boring and the borings drilled in other areas of the subject site, the uppermost 5 feet of native materials consist predominantly of interlayered medium-dense sands and silty sands, and firm sandy silts and silty clays.

5. Groundwater was not encountered in any of the exploratory borings drilled within the subject site to the maximum depth explored (41½ feet).

6. An absorption rate of 95 gallons/day/ft^2 was obtained at the time of our study. Using the conversion factor published in the County of Orange Water Quality Management Plan, Technical Appendix VII, this data yields a minimum tested infiltration rate of approximately 13 inches per hour for the site.

7. It should be noted that the percolation test was conducted with relatively clean water. Nuisance water, which contains sediments and other impurities, may reduce the soil absorption rate.

8. The storm water system should be designed according to the standards set by the City of Long Beach, the County of Los Angeles, or other applicable jurisdictional agency. An appropriate safety factor should be used for preliminary design calculations.

9. All storm water infiltration areas should be maintained at a minimum horizontal distance of 10 feet away from any existing or proposed structural footings.

10. It may be considered prudent to designate a backup area for conditions where the main storm water absorption area has either lost its full absorption capacity, or an overflow of storm water takes place.

GRADING AND FOUNDATION PLAN REVIEW

It must be emphasized that the recommendations provided throughout this report are based solely on the preliminary grading plan no finalized grading plans, structural plans or details were available for review as of the date of this report. As such, the conclusions and recommendations provided herein should be considered as tentative. Once such plans and details become available, our firm should be retained to review these documents to determine the applicability of our recommendations to the actual construction proposed. Additional recommendations and/or modification of the recommendations provided herein will be provided if necessary depending on the results of the grading plan and/or structural plan review.

FUTURE IMPROVEMENTS AND GRADING

If additional improvements are considered in the future, our firm should be notified so that we may provide design recommendations to mitigate movement, settlement and/or tilting of the structures. Potential problems can develop when drainage on the pads is altered in any way such as placement of fill and construction of new walkways, patios, landscape walls, or planters. Therefore, it is recommended that we be engaged to review the final design drawings, specifications and grading plan prior to any new construction. If we are not provided the opportunity to review these documents with respect to the geotechnical aspects of new construction and grading, it should not be assumed that the recommendations provided herein are wholly or in part applicable to the proposed construction.

REPORT LIMITATIONS

This report is based on the proposed project and geotechnical data as described herein. The materials encountered on the project site, described in other literature, and utilized in our laboratory investigation are believed representative of the project area, and the conclusions and recommendations contained in this report are presented on that basis. However, soil materials can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect the conclusions and recommendations contained herein. As such, observation and testing by a geotechnical consultant during the grading and construction phases of the project are essential to confirming the basis of this report.

This report has been prepared consistent with that level of care being provided by other professionals providing similar services at the same locale and time period. The contents of this report are professional opinions and as such, are not to be considered a guarantee or warranty. This report should be reviewed and updated after a period of one year or if the project concept changes from that described herein.

The information contained herein has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

This report is subject to review by the controlling authorities for this project. Should you have any questions, please do not hesitate to call.

Respectfully submitted,

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SJ/AS/SW/lm

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FIGURES

SITE LOCATION MAP

SITE AERIAL VIEW MAP

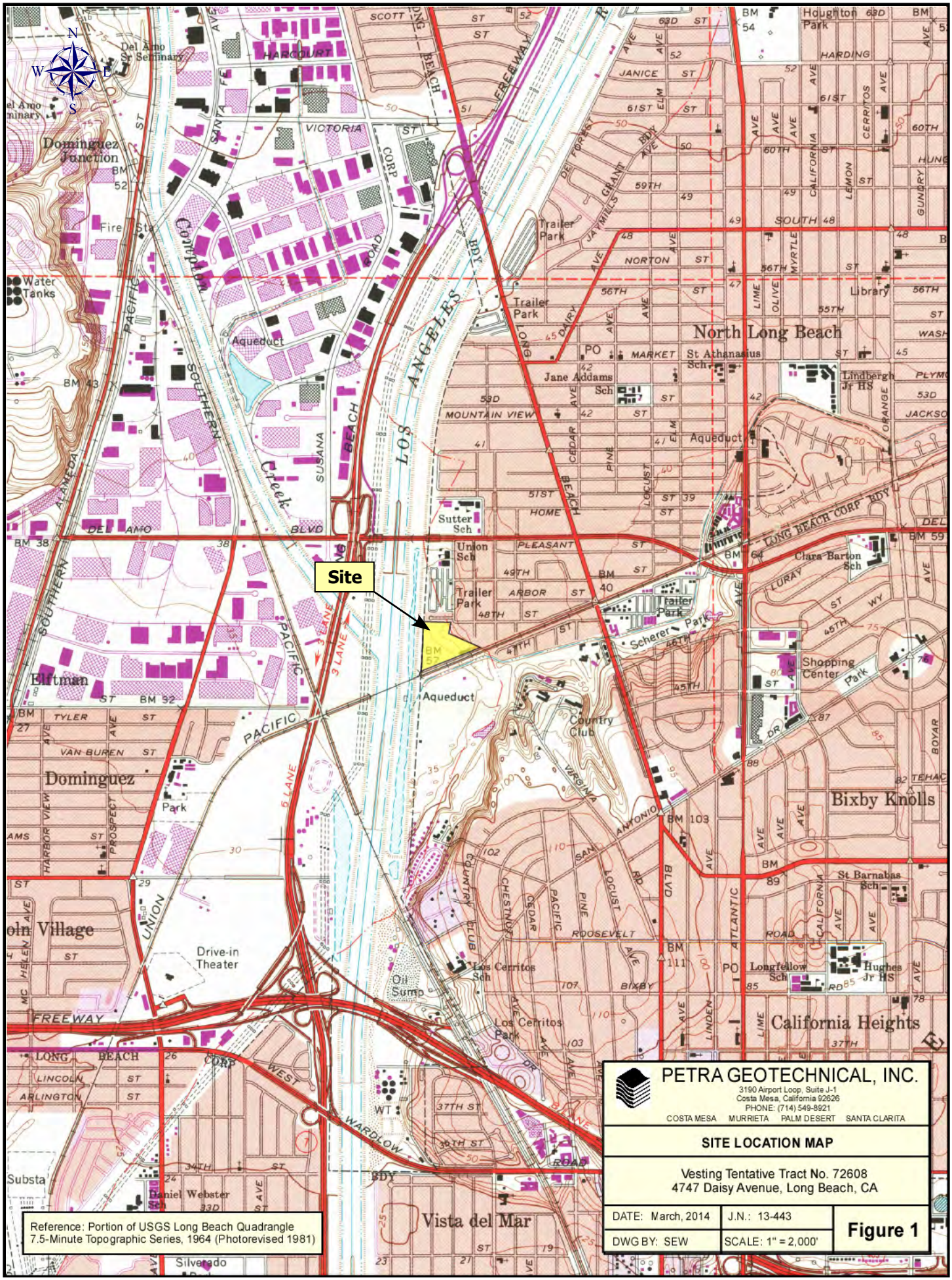
REGIONAL FAULT ACTIVITY MAP

EARTHQUAKE EPICENTERS MAP

SEISMIC HAZARD ZONES MAP

RETAINING WALL

PETRA


Site
Dominguez Junction
Compton Creek
Aqueduct
Water Tanks
Fire Sta
Del Amo Sr Seminary
North Long Beach
Trailer Park
Jane Addams Sch
St Athanasius Sch
Lindbergh Jr HS
Library
Sutter Sch
Union Sch
Clara Barton Sch
Scherer Park
Shopping Center
Country Club
Bixby Knolls
St Barnabas Sch
Longfellow Sch
Hughes Jr HS
California Heights
Los Cerritos Sch
Los Cerritos Park
Oil Sump
Drive-in Theater
Elftman
Dominguez
Park
Lincoln Village
Daniel Webster Sch
Vista del Mar
Silverado
Substa
Houghton Park
LOS ANGELES
LONG BEACH CORP BDY
PACIFIC
SOUTHERN
FREEWAY
WEST
WARDLOW
ROOSEVELT
BIXBY
ANTONIO
ATLANTIC
BEACH
DEL AMO
BLVD
MOUNTAIN VIEW
PLEASANT
ARBOR
HOME
NORTON
MARKET
JANICE
HARDING
SOUTH
VICTORIA
HARCOURT
SCOTT
TYLER
VAN BUREN
LINCOLN
ARLINGTON
37TH ST
36TH ST
34TH ST
WT
Reference: Portion of USGS Long Beach Quadrangle
7.5-Minute Topographic Series, 1964 (Photorevised 1981)
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COSTA MESA MURRIETA PALM DESERT SANTA CLARITA
SITE LOCATION MAP
Vesting Tentative Tract No. 72608
4747 Daisy Avenue, Long Beach, CA
DATE: March, 2014
J.N.: 13-443
DWG BY: SEW
SCALE: 1" = 2,000'
Figure 1

PETRA GEOTECHNICAL, INC.
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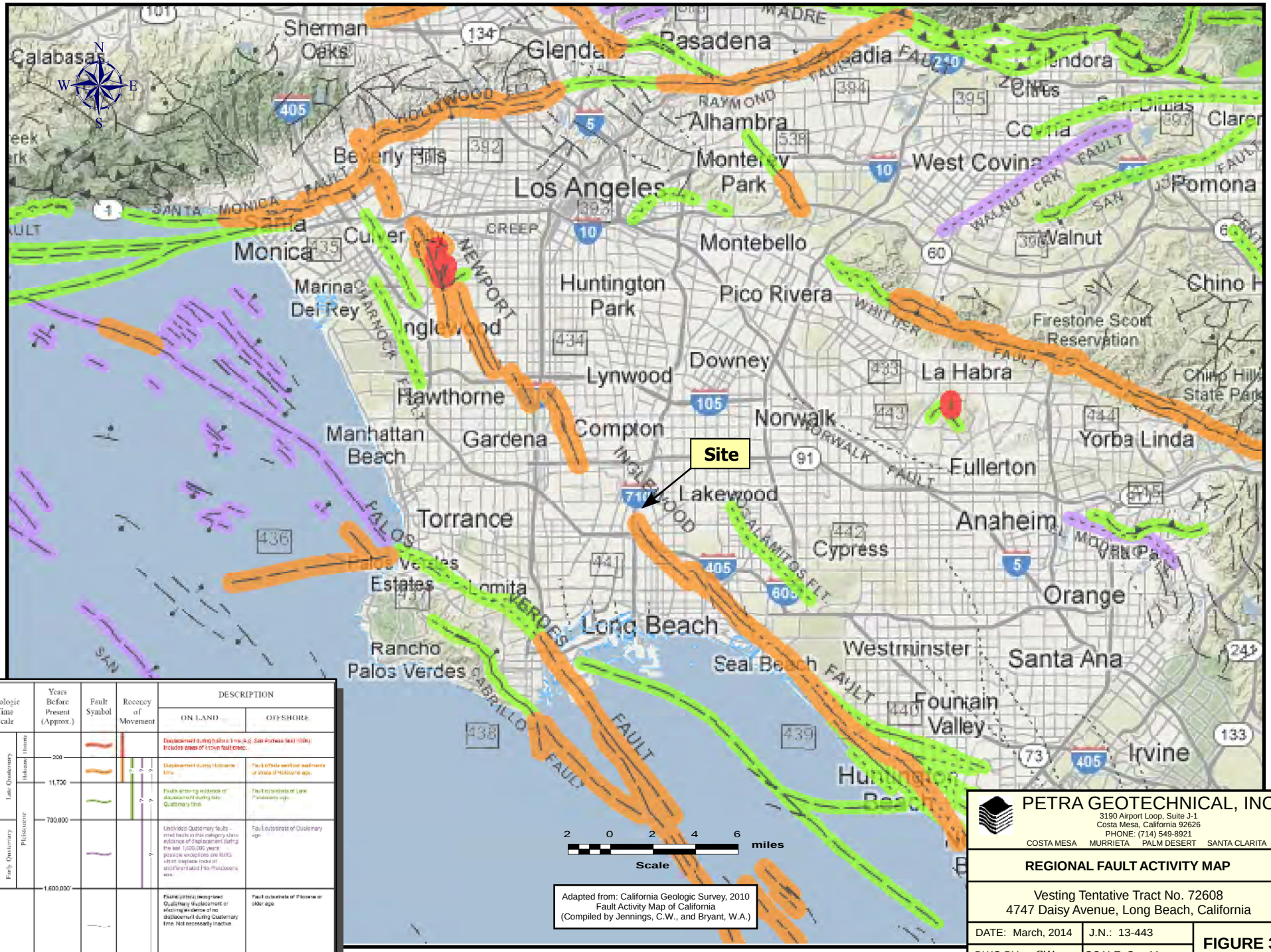
Costa Mesa, California 92626
PHONE: (714) 549-8921
COSTA MESA TEMECULA PALM DESERT VALENCIA

SITE AERIAL VIEW

Vesting Tentative Tract No. 72608
4747 Daisy Avenue, Long Beach, CA

DATE: March, 2014	J.N.: 13-443	**Figure 2**
DWG BY: SW	SCALE: 1" = 200'	



Adapted from: California Geologic Survey, 2010
Fault Activity Map of California
(Compiled by Jennings, C.W., and Bryant, W.A.)

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Holocene	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			11,700			Displacement during Holocene time.	Fault offsets seafloor sediments of areas of Holocene age.
		Pleistocene	700,000			Faults showing evidence of displacement during late Quaternary time.	Fault offsets of Late Pleistocene age.
	Early Quaternary		1,600,000			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of uncertain related Pliocene/Pleistocene age.	Fault offsets of Quaternary age.
Pre-Quaternary			4.5 billion (Age of Earth)			Faults which recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault offsets of Pliocene or older age.

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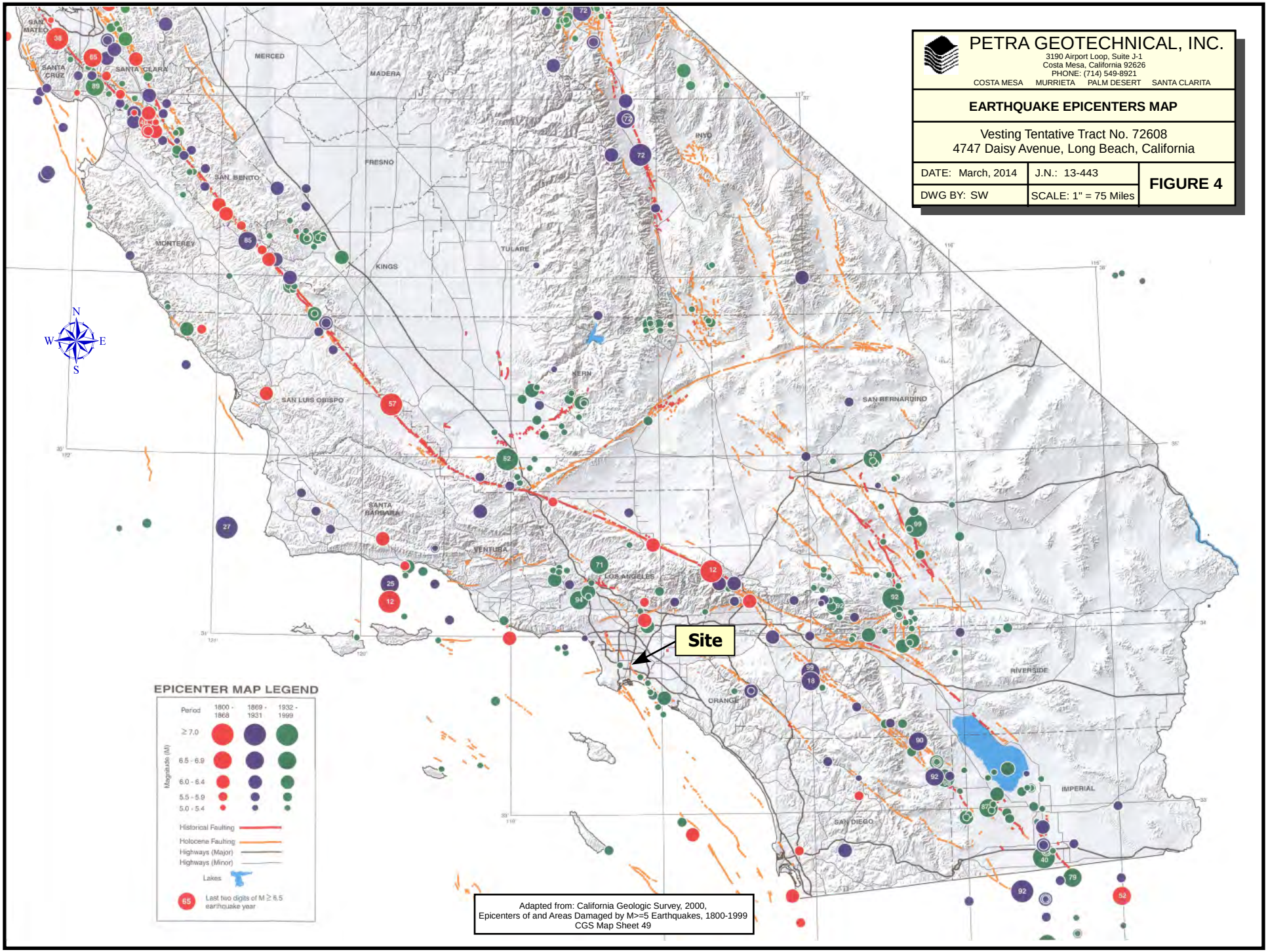
COSTA MESA MURRIETA PALM DESERT SANTA CLARITA

REGIONAL FAULT ACTIVITY MAP

Vesting Tentative Tract No. 72608
4747 Daisy Avenue, Long Beach, California

DATE: March, 2014	J.N.: 13-443	**FIGURE 3**
DWG BY: SW	SCALE: See Map	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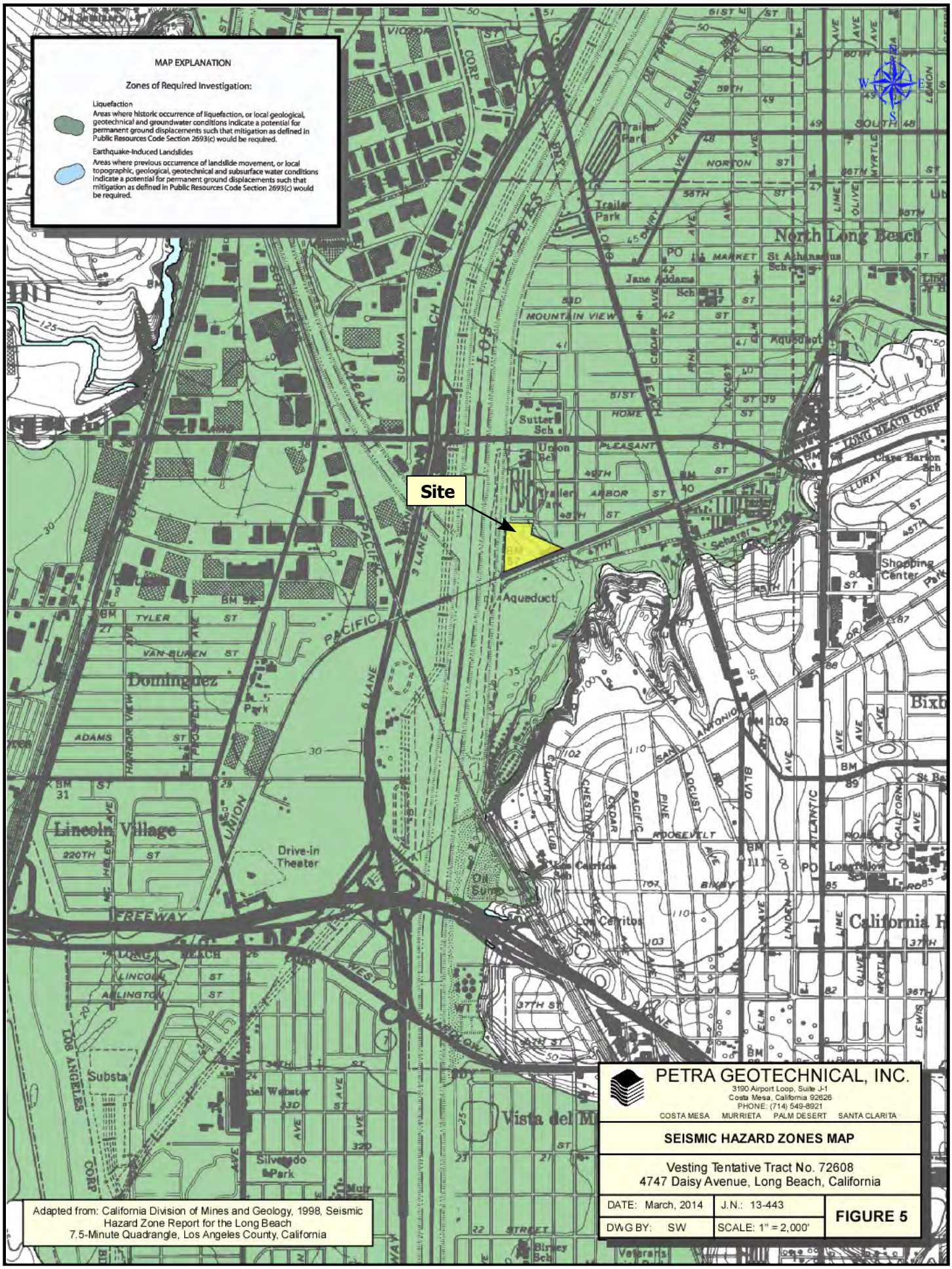
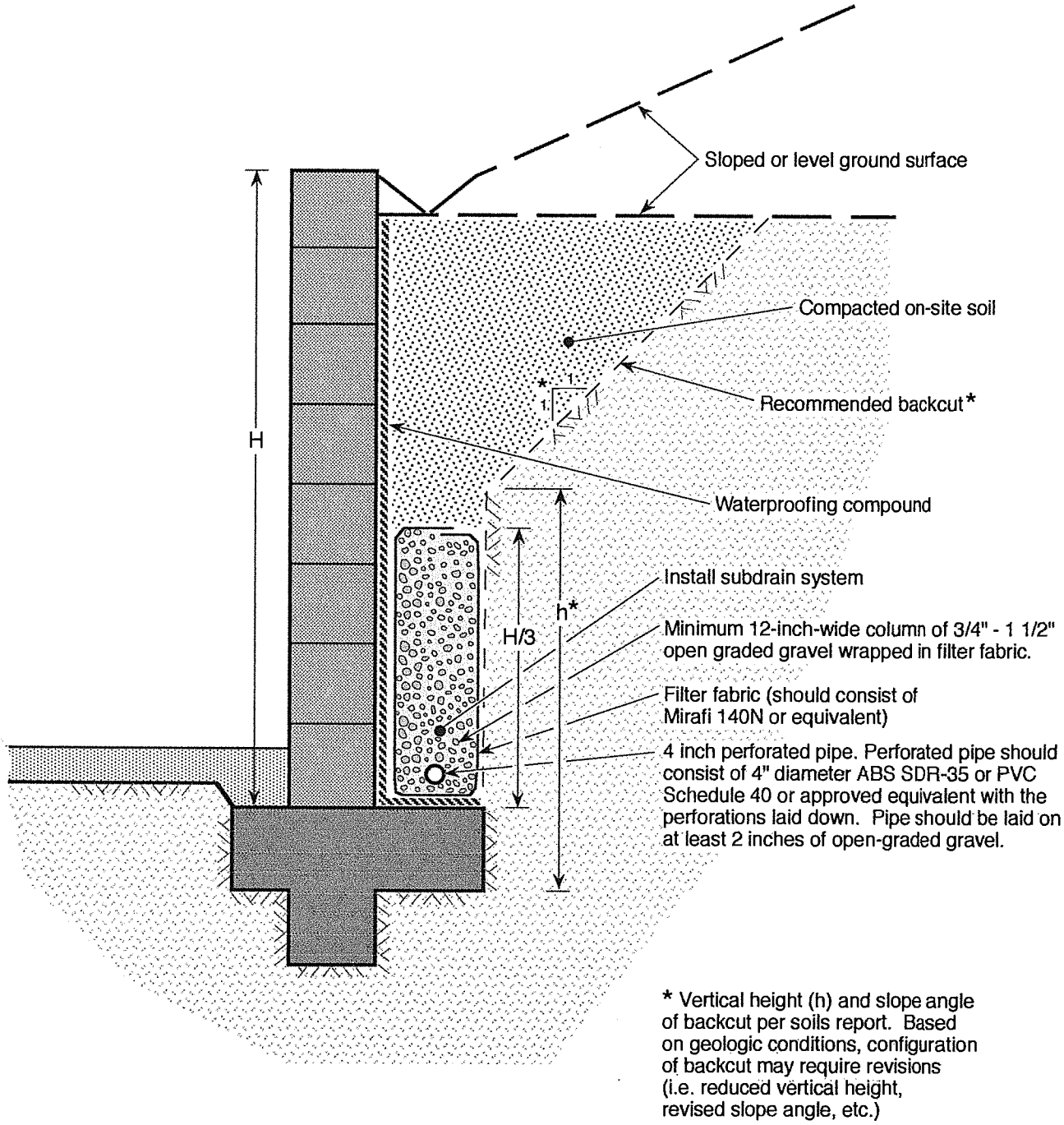
Costa Mesa, California 92626
PHONE: (714) 549-8921
COSTA MESA MURRIETA PALM DESERT SANTA CLARITA
EARTHQUAKE EPICENTERS MAP
Vesting Tentative Tract No. 72608
4747 Daisy Avenue, Long Beach, California
DATE: March, 2014
J.N.: 13-443
DWG BY: SW
SCALE: 1" = 75 Miles
FIGURE 4
SAN MATEO
SANTA CRUZ
SANTA CLARA
MERCED
MADERA
FRESNO
INYO
SAN BENITO
MONTEREY
KINGS
TULARE
KERN
SAN LUIS OBISPO
SAN BERNARDINO
SANTA BARBARA
VENTURA
LOS ANGELES
ORANGE
RIVERSIDE
IMPERIAL
SAN DIEGO
Site
N
S
E
W
EPICENTER MAP LEGEND
Period
1800 - 1868
1869 - 1931
1932 - 1999
Magnitude (M)
≥ 7.0
6.5 - 6.9
6.0 - 6.4
5.5 - 5.9
5.0 - 5.4
Historical Faulting
Holocene Faulting
Highways (Major)
Highways (Minor)
Lakes
65 Last two digits of M ≥ 6.5 earthquake year
Adapted from: California Geologic Survey, 2000,
Epicenters of and Areas Damaged by M>=5 Earthquakes, 1800-1999
CGS Map Sheet 49


MAP EXPLANATION
Zones of Required Investigation:
Liquefaction
Areas 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.
Earthquake-Induced Landslides
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.
Site
North Long Beach
Lincoln Village
Drive-in Theater
Aqueduct
Shopping Center
Silverado Park
Adapted from: California Division of Mines and Geology, 1998, Seismic Hazard Zone Report for the Long Beach 7.5-Minute Quadrangle, Los Angeles County, California
PETRA GEOTECHNICAL, INC.
3190 Airport Loop, Suite J-1
Costa Mesa, California 92626
PHONE: (714) 549-8921
COSTA MESA MURRIETA PALM DESERT SANTA CLARITA
SEISMIC HAZARD ZONES MAP
Vesting Tentative Tract No. 72608
4747 Daisy Avenue, Long Beach, California
DATE: March, 2014
J.N.: 13-443
DWG BY: SW
SCALE: 1" = 2,000'
FIGURE 5

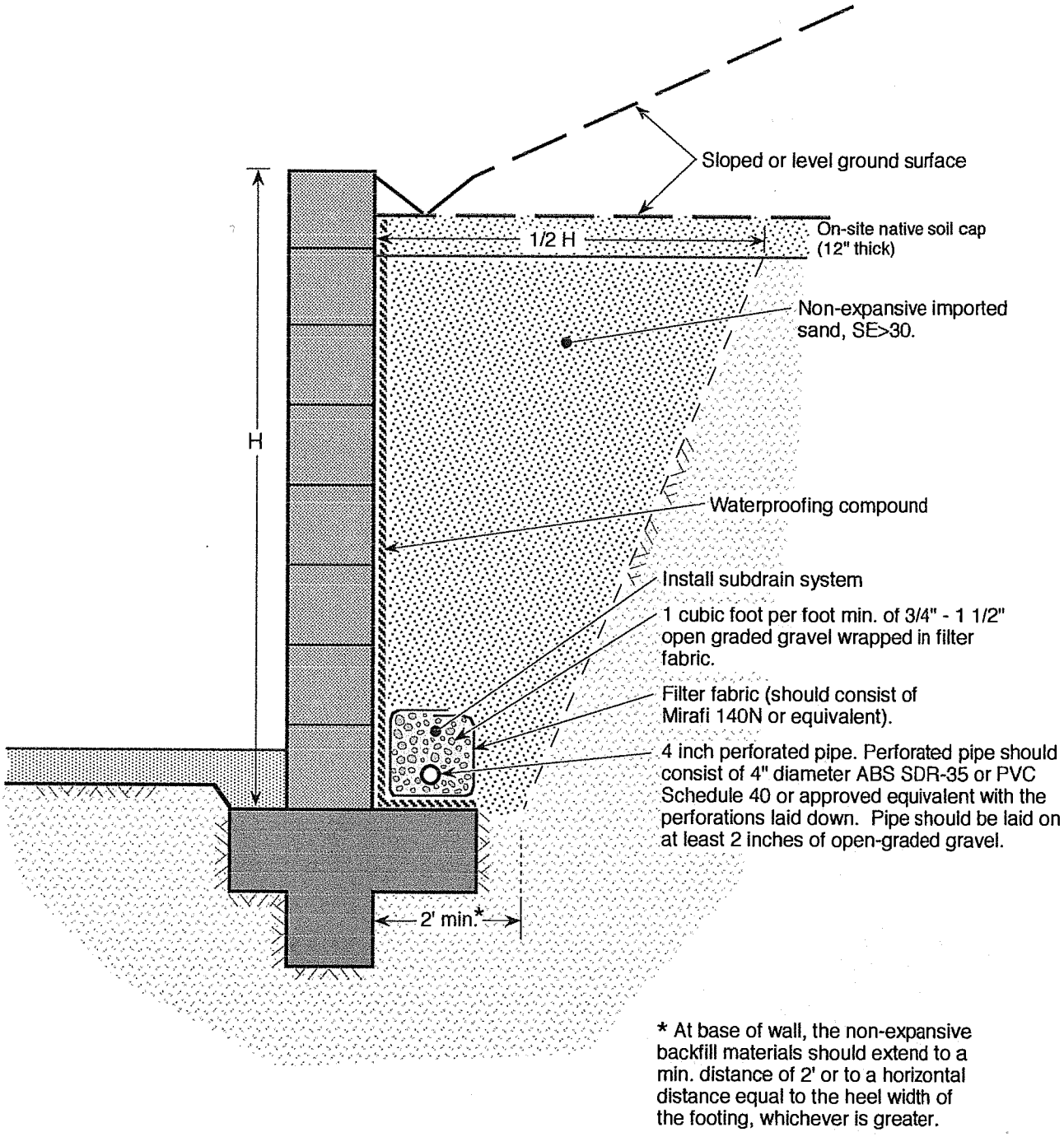
NATIVE SOIL BACKFILL



* Vertical height (h) and slope angle of backcut per soils report. Based on geologic conditions, configuration of backcut may require revisions (i.e. reduced vertical height, revised slope angle, etc.)

PETRA	RETAINING WALL BACKFILL AND SUBDRAIN DETAILS	FIGURE RW-1

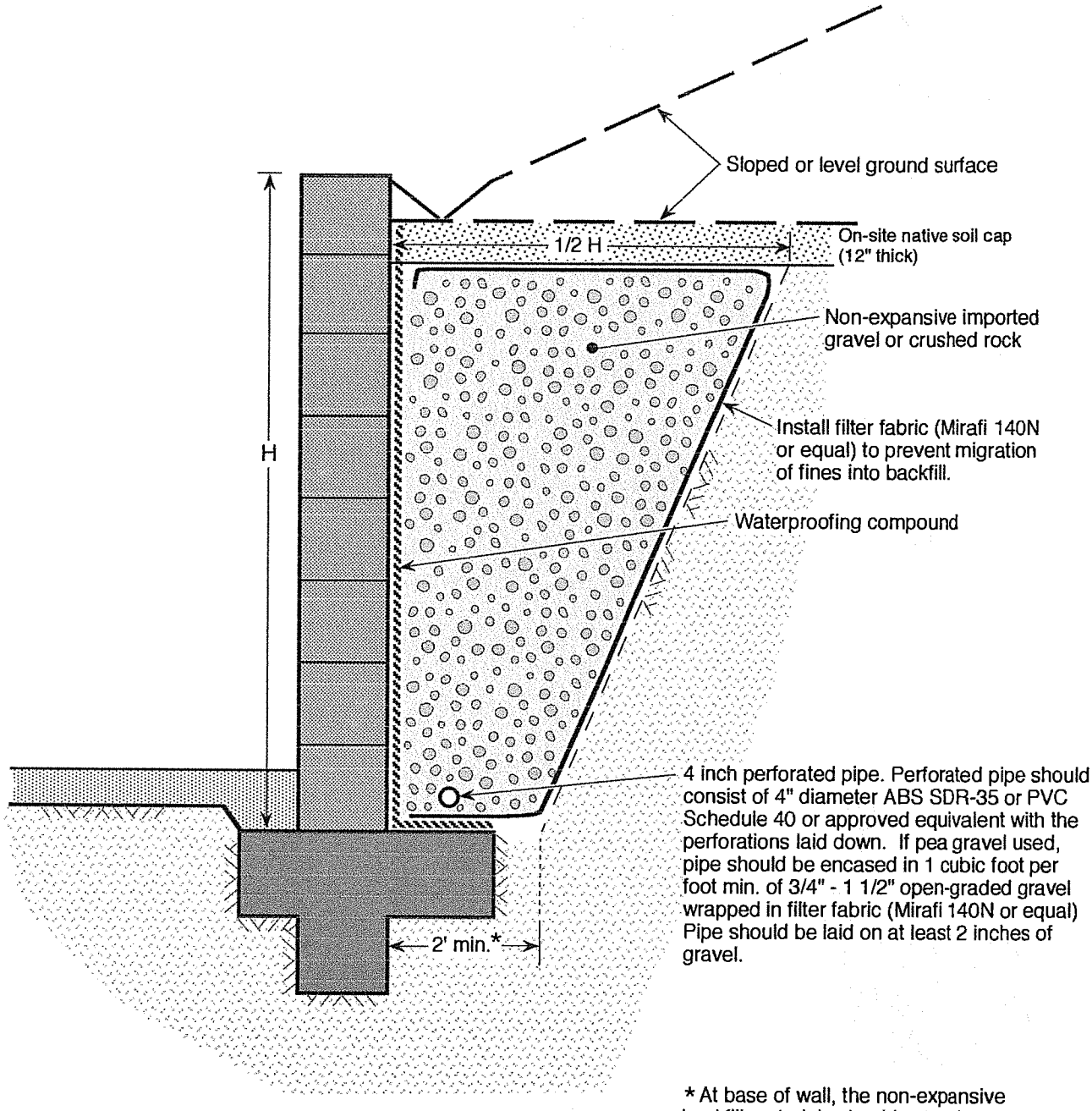
IMPORTED SAND BACKFILL



* At base of wall, the non-expansive backfill materials should extend to a min. distance of 2' or to a horizontal distance equal to the heel width of the footing, whichever is greater.

PETRA	RETAINING WALL BACKFILL AND SUBDRAIN DETAILS	FIGURE RW-2

IMPORTED GRAVEL OR CRUSHED ROCK BACKFILL



* At base of wall, the non-expansive backfill materials should extend to a min. distance of 2' or to a horizontal distance equal to the heel width of the footing, whichever is greater.

PETRA	RETAINING WALL BACKFILL AND SUBDRAIN DETAILS	FIGURE RW-3

APPENDIX A

EXPLORATION LOGS

CPT TEST DATA

PETRA

Key to Soil and Bedrock Symbols and Terms

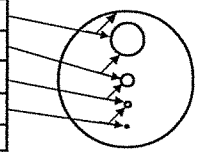
PETRA

Unified Soil Classification System

Coarse-grained Soils > 1/2 of materials is larger than #200 sieve (The No. 200 U.S. Standard Sieve is about the smallest particle visible to the naked eye)	GRAVELS more than half of coarse fraction is larger than #4 sieve	Clean Gravels (less than 5% fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines	GM	Silty Gravels, poorly-graded gravel-sand-silt mixtures
			GC	Clayey Gravels, poorly-graded gravel-sand-clay mixtures
	SANDS more than half of coarse fraction is smaller than #4 sieve	Clean Sands (less than 5% fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sands, little or no fines
		Sands with fines	SM	Silty Sands, poorly-graded sand-gravel-silt mixtures
			SC	Clayey Sands, poorly-graded sand-gravel-clay mixtures
Fine-grained Soils > 1/2 of materials is smaller than #200 sieve	SILTS & CLAYS Liquid Limit Less Than 50		ML	Inorganic silts & very fine sands, silty or clayey fine sands, clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts & clays of low plasticity
	SILTS & CLAYS Liquid Limit Greater Than 50		MH	Inorganic silts, micaceous or diatomaceous fine sand or silt
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic silts and clays of medium-to-high plasticity
Highly Organic Soils			PT	Peat, humus swamp soils with high organic content

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		>12"	>12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19"	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized to
Fines		Passing #200	<0.0029"	Flour-sized and smaller



Laboratory Test Abbreviations

MAX	Maximum Dry Density	MA	Mechanical (Particle Size) Analysis
EXP	Expansion Potential	AT	Atterberg Limits
SO4	Soluble Sulfate Content	#200	#200 Screen Wash
RES	Resistivity	DSU	Direct Shear (Undisturbed Sample)
pH	Acidity	DSR	Direct Shear (Remolded Sample)
CON	Consolidation	HYD	Hydrometer Analysis
SW	Swell	SE	Sand Equivalent
CL	Chloride Content	OC	Organic Content
RV	R-Value	COMP	Mortar Cylinder Compression

Modifiers

Trace	< 1 %
Few	1 - 5%
Some	5 - 12 %
Numerous	12 - 20 %

Sampler and Symbol Descriptions

- Approximate Depth of Seepage
- Approximate Depth of Standing Groundwater
- Modified California Split Spoon Sample
- Standard Penetration Test
- Bulk Sample
- Shelby Tube
- No Recovery in Sampler

Bedrock Hardness

Soft	Can be crushed and granulated by hand; "soil like" and structureless
Moderately Hard	Can be grooved with fingernails; gouged easily with butter knife; crumbles under light hammer blows
Hard	Cannot break by hand; can be grooved with a sharp knife; breaks with a moderate hammer blow
Very Hard	Sharp knife leaves scratch; chips with repeated hammer blows

Notes:

Blows Per Foot: Number of blows required to advance sampler 1 foot (unless a lesser distance is specified). Samplers in general were driven into the soil or bedrock at the bottom of the hole with a standard (140 lb.) hammer dropping a standard 30 inches unless noted otherwise in Log Notes. Drive samples collected in bucket auger borings may be obtained by dropping non-standard weight from variable heights. When a SPT sampler is used the blow count conforms to ASTM D-1586

EXPLORATION LOG

Project: **VTT 72608**		Boring No.: **B-1**	
Location: **4747 Daisy Ave, Long Beach**		Elevation: **+/- 31**	
Job No.: **13-443**	Client: **Integral Communities**	Date: **3/4/14**	
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **SHW**	

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per 6 Inches	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (af) Silty Sand (SM): Grayish-brown; very moist; loose; very fine- to fine-grained sand; with some clay. @2': becomes brown; moist.							
		YOUNG ALLUVIAL FAN DEPOSITS (Qyf) Sand (SP): Off-White; dry to slightly moist; medium dense; fine-grained sand; with no fines.		6 11 11			1.3	99.5	MAX
5		@5': becomes slightly moist.		4 4 8			2.0	91.6	
		@7': becomes off-white to light gray.		6 12 16			1.8	105.2	
		Silt (ML): Light gray; dry to slightly moist; stiff; with limonite staining.							
10				8 10 14			18.7	95.1	
15		Sand (SP): Light gray light brown; slightly moist; dense; very fine-grained sand.		7 20 30			2.3	101.4	
		Total Depth = 16.5 Feet No Groundwater Encountered Boring Backfilled with Cuttings.							
20									

EXPLORATION LOG - V3 13-443.GPJ PETRA.GDT 3/19/14

PLATE A-1

Petra Geotechnical, Inc.

EXPLORATION LOG

Project: **VTT 72608**		Boring No.: **B-2**	
Location: **4747 Daisy Ave, Long Beach**		Elevation: **+/- 38**	
Job No.: **13-443**	Client: **Integral Communities**	Date: **3/4/14**	
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **SHW**	

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per 6 Inches	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ASPHALTIC CONCRETE (A/C) 4" thick, no aggregate base.							MAX EXP AT HYD SO4 pH RES CL
		ARTIFICIAL FILL (af) Clayey Silt (ML): Brown; moist; firm; slightly micaceous; with few very fine-grained sand.		3 4 7			7.1	92.2	
		YOUNG ALLUVIAL FAN DEPOSITS (Qyf) Sand (SP): Light gray; moist; medium dense; very fine-grained sand; with few silt.		3 6 9			26.0	87.0	
5		Silty Clay (CL): Dark olive brown; very moist; firm.							
		Sand (SP): Light gray light brown; moist; medium dense; very fine-grained sand; with some silt.		4 7 6			2.6	91.8	
10		Silty Clay (CL): Dark olive gray olive brown; very moist; stiff; with limonite staining and some very fine-grained sand.		6 11 16			17.0	110.9	
15		Sand (SP): Light gray light brown; slightly moist; dense; very fine-grained sand; with few silt.		12 19 23			6.0	100.6	
		Total Depth = 16.5 Feet No Groundwater Encountered Boring Backfilled with Cuttings.							
20									

EXPLORATION LOG - V3 13-443.GPJ PETRA.GDT 3/19/14

PLATE A-2

Petra Geotechnical, Inc.

EXPLORATION LOG

Project: **VTT 72608**		Boring No.: **B-2A**	
Location: **4747 Daisy Ave, Long Beach**		Elevation: **+/- 38**	
Job No.: **13-443**	Client: **Integral Communities**	Date: **3/4/14**	
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **SHW**	

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per 6 Inches	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ASPHALTIC CONCRETE (A/C) 3" thick; no aggregate base.							
		ARTIFICIAL FILL (af) Clayey Silt (ML): Olive gray; moist; firm; micaceous; with few very fine-grained sand.							
		YOUNG ALLUVIAL FAN DEPOSITS (Qyf) Sand (SP): Light gray; moist; medium dense; very fine-grained sand; with few to some silt.							
5		Total Depth = 3 Feet No Groundwater Encountered Boring Converted to Perc Hole and A/C Patched.							
10									
15									
20									

EXPLORATION LOG - V3 13-443.GPJ PETRA.GDT 3/19/14

PLATE A-3

Petra Geotechnical, Inc.

EXPLORATION LOG

Project: **VTT 72608**		Boring No.: **B-3**	
Location: **4747 Daisy Ave, Long Beach**		Elevation: **+/- 38**	
Job No.: **13-443**	Client: **Integral Communities**	Date: **3/4/14**	
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **SHW**	

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per 6 Inches	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (af) Silty Sand (SM): Dark olive gray; very moist; loose; very fine-grained sand; with some clay.							
		YOUNG ALLUVIAL FAN DEPOSITS (Qyf) Sand (SP): Light gray off-white; dry to slightly moist; medium dense; very fine-grained sand; with few silt. @3': with increasing silt.		5 11 11			1.4	110.6	
5		Sandy Silt (ML): Light grayish-brown; slightly moist; firm; very fine-grained sand.		7 5 10			15.0	81.4	
		@6': becomes light gray; with limonite staining; increasing sand.		7 9 13			9.7	95.6	CON
10		Sand (SP): Off-White to light gray; dry to slightly moist; medium dense; very fine-grained sand; with limonite staining; no fines.		10 13 18			2.0	100.4	
15		@15': becomes dense.		12 17 24			1.8	99.6	
20		@20': becomes white to light gray; slightly moist; very fine- to fine-grained sand.		12 16 23			3.7	94.1	
		Total Depth = 21.5 Feet No Groundwater Encountered Boring Backfilled with Cuttings.							

EXPLORATION LOG - V3 13-443.GPJ PETRA.GDT 3/19/14

PLATE A-4

Petra Geotechnical, Inc.

EXPLORATION LOG

Project: **VTT 72608**		Boring No.: **B-4**	
Location: **4747 Daisy Ave, Long Beach**		Elevation: **+/- 38**	
Job No.: **13-443**	Client: **Integral Communities**	Date: **3/4/14**	
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **SHW**	

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per 6 Inches	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (af) Silty Sand (SM): Brown; very moist; loose; very fine-grained sand; with roots to 1/2".							MAX DSR
		YOUNG ALLUVIAL FAN DEPOSITS (Qyf) Sand (SP): Off-White light gray; slightly moist; medium dense; very fine-grained sand; with no fines.		4 6 7			4.3	85.0	
5		@5': with increasing silt.		7 9 12			4.6	89.7	CON
		Sandy Silt (ML): Light gray; dry to slightly moist; stiff; very fine-grained sand.							
		Silty Clay (CL): Light gray; dry to slightly moist; stiff; porous; with limonite staining; root casts.		5 6 8			23.3	76.8	
10		@10': becomes dark olive gray.		6 13 21			5.2	110.9	
		Silty Sand (SM): Olive brown; dry to slightly moist; medium dense; very fine-grained sand.							
15		@15': becomes off-white to light gray; with decreasing silt.		8 12 20			1.9	93.2	
		Sand (SP): White light gray; dry to slightly moist; dense; very fine-grained sand.							
20		@20': same as above.		18 23 30			2.0	100.3	

EXPLORATION LOG - V3 13-443.GPJ PETRA.GDT 3/19/14

PLATE A-5

Petra Geotechnical, Inc.

EXPLORATION LOG

Project: **VTT 72608**		Boring No.: **B-4**	
Location: **4747 Daisy Ave, Long Beach**		Elevation: **+/- 38**	
Job No.: **13-443**	Client: **Integral Communities**	Date: **3/4/14**	
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **SHW**	

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per 6 Inches	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		@25': becomes very fine- to fine-grained sand.		21	■		2.1	103.9	
				26	■				
				26	■				
				11	▨				
				15	▨				
				15					
30		@30': becomes dark brown; slightly moist; medium dense; very fine-grained; with abundant organic material.		5	■		15.7	92.4	
				13	■				
		@31': white; dry to slightly moist; dense; very fine-grained sand; no organics.		35	■				
35		@ 35': becomes medium dense; moist; fine- to coarse-grained sand; with limonite staining and 2-3" thick, gray clay interbeds.		5	▨				
				5	▨				
				8	▨				
		Interlayered Sand and Clay (SP/CL): Bluish-gray; very moist to wet; dense/very stiff; very fine-grained sand.							
40				5	■		20.7	99.2	
				16	■				
		@ 40': same as above.		30	■				
		Total Depth = 41.5 Feet							
		No Groundwater Encountered							
45		Boring Backfilled with Cuttings.							

EXPLORATION LOG - V3 13-443.GPJ PETRA.GDT 3/19/14

PLATE A-7

Petra Geotechnical, Inc.

EXPLORATION LOG

Project: **VTT 72608**		Boring No.: **B-5**	
Location: **4747 Daisy Ave, Long Beach**		Elevation: **+/- 34**	
Job No.: **13-443**	Client: **Integral Communities**	Date: **3/4/14**	
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **SHW**	

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per 6 Inches	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (af) Silty Sand (SM): Grayish-brown; very moist; loose; very fine-grained sand.							
		Sand (SP): Gray; dry to slightly moist; loose; very fine-grained sand; with buried wood fragments (buried fire pit) and old bricks.							
		YOUNG ALLUVIAL FAN DEPOSITS (Qyf)							
5		Sand (SP): Light gray light brown; slightly moist; loose; very fine-grained sand; with few silt.		3 4 6	■		2.7	87.4	
		@7': same as above.		3 5 7	■		3.1	88.5	
10		@10': same as above.		4 6 11	■		3.5	92.0	
15		Sandy Silt (ML): Light gray; slightly moist; very stiff; very fine-grained sand.		8 13 27	■		13.1	100.0	
		Total Depth = 16.5 Feet No Groundwater Encountered Boring Backfilled with Cuttings.							
20									

EXPLORATION LOG - V3 13-443.GPJ PETRA.GDT 3/19/14

PLATE A-7

Petra Geotechnical, Inc.

13-443

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

4747 Daisy Avenue
Long Beach, CA
August 26, 2013

Prepared for:

Mr. Scott Winslow
Petra Geotechnical, Inc.
3190 Airport Loop Drive, Ste J1
Costa Mesa, CA 92626
Office (714) 549-8921 / Fax (714) 668-3770

Prepared by:

KEHOE TESTING & ENGINEERING
5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

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2. SUMMARY OF FIELD WORK

3. FIELD EQUIPMENT & PROCEDURES

4. CONE PENETRATION TEST DATA & INTERPRETATION

APPENDIX

- CPT Plots
- CPT Classification/Soil Behavior Chart
- Interpretation Output (CPeT-IT)
- Summary of Shear Wave Velocities
- Pore Pressure Dissipation Graphs
- CPeT-IT Calculation Formulas

SUMMARY
OF
CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at 4747 Daisy Avenue in Long Beach, California. The work was performed by Kehoe Testing & Engineering (KTE) on August 26, 2013. The scope of work was performed as directed by Petra Geotechnical, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at four locations to determine the soil lithology. Groundwater measurements and hole collapse depths provided in **TABLE 2.1** are for information only. The readings indicate the apparent depth to which the hole is open and the apparent water level (if encountered) in the CPT probe hole at the time of measurement upon completion of the CPT. KTE does not warranty the accuracy of the measurements and the reported water levels may not represent the true or stabilized groundwater levels.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	50	Hole open to 30.0 ft (dry)
CPT-2	50	Hole open to 30.0 ft (dry)
CPT-3	50	Hole open to 31.0 ft (dry)
CPT-4	50	Hole open to 29.5 ft (dry)

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed
- Pore Pressure Dissipation (at selected depths)

At location CPT-3, shear wave measurements were obtained at approximately 10-foot intervals. The shear wave is generated using an air-actuated hammer, which is located inside

the front jack of the CPT rig. The cone has a triaxial geophone, which recorded the shear wave signal generated by the air hammer.

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil classification on the CPT plots is derived from the attached CPT Classification Chart (Robertson) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

Tables of basic CPT output from the interpretation program CPeT-IT are provided for CPT data averaged over one foot intervals in the Appendix. Spreadsheet files of the averaged basic CPT output and averaged estimated geotechnical parameters are also included for use in further geotechnical analysis. We recommend a geotechnical engineer review the assumed input parameters and the calculated output from the CPeT-IT program. A summary of the equations used for the tabulated parameters is provided in the Appendix.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Richard W. Koester, Jr.
General Manager

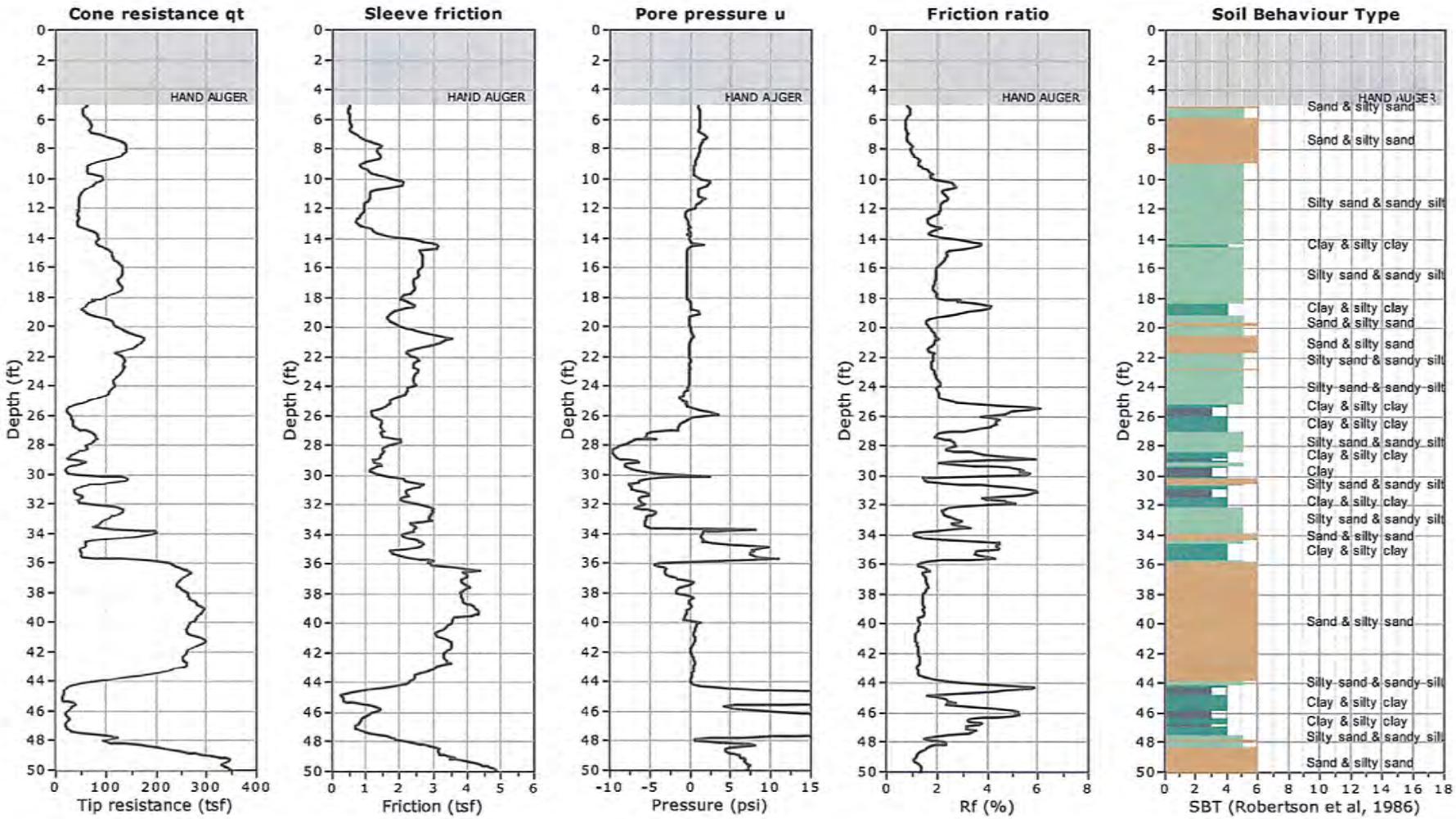
09/13/13-ds-4105

APPENDIX

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT: CPT-1
Total depth: 50.59 ft, Date: 8/26/2013
Cone Type: Vertek

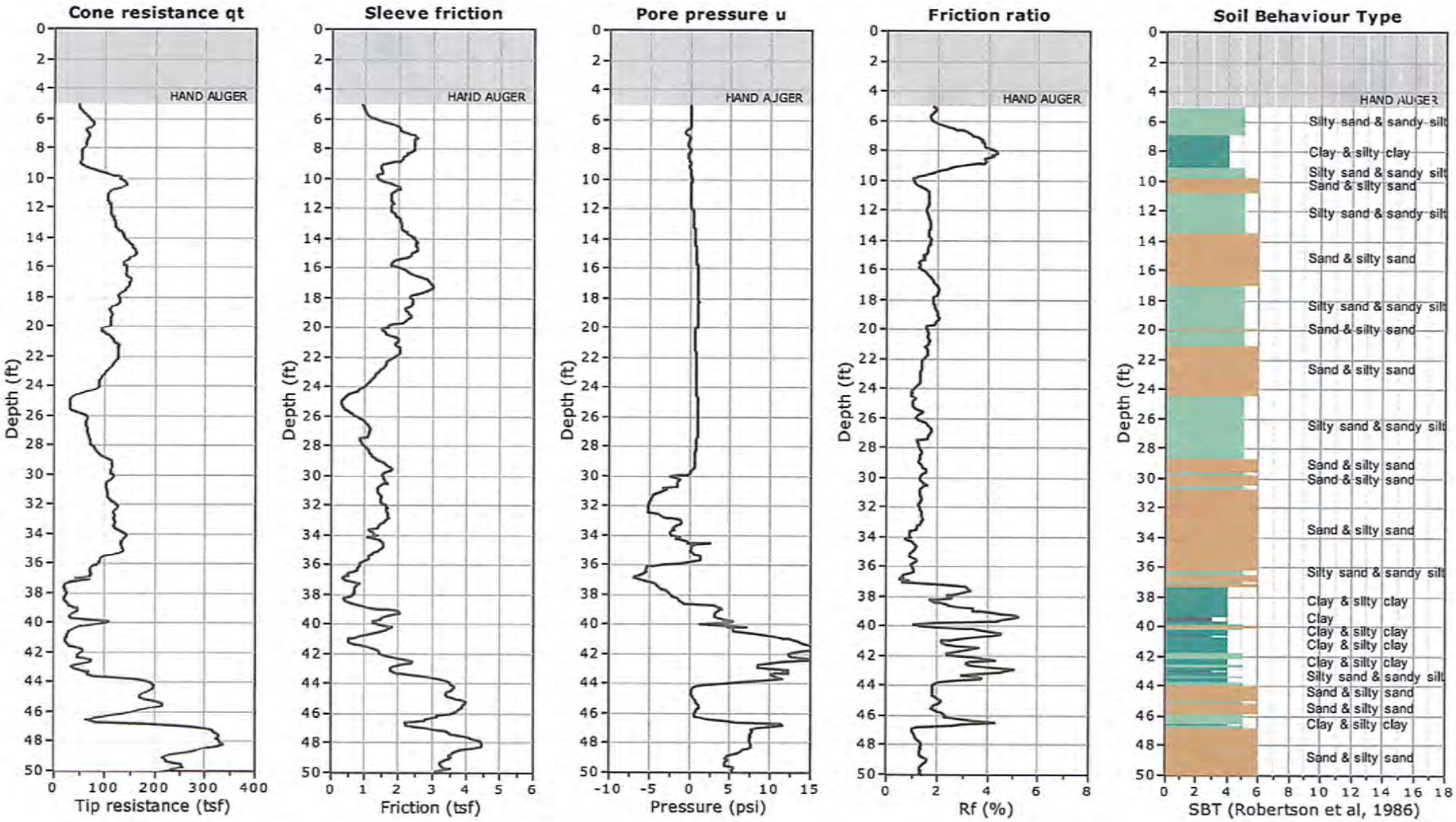
Project: Petra Geotechnical
Location: 4747 Daisy Ave. Long Beach, CA



Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

Project: **Petra Geotechnical**
Location: **4747 Daisy Ave. Long Beach, CA**

CPT: CPT-2
Total depth: 50.76 ft, Date: 8/26/2013
Cone Type: Vertek

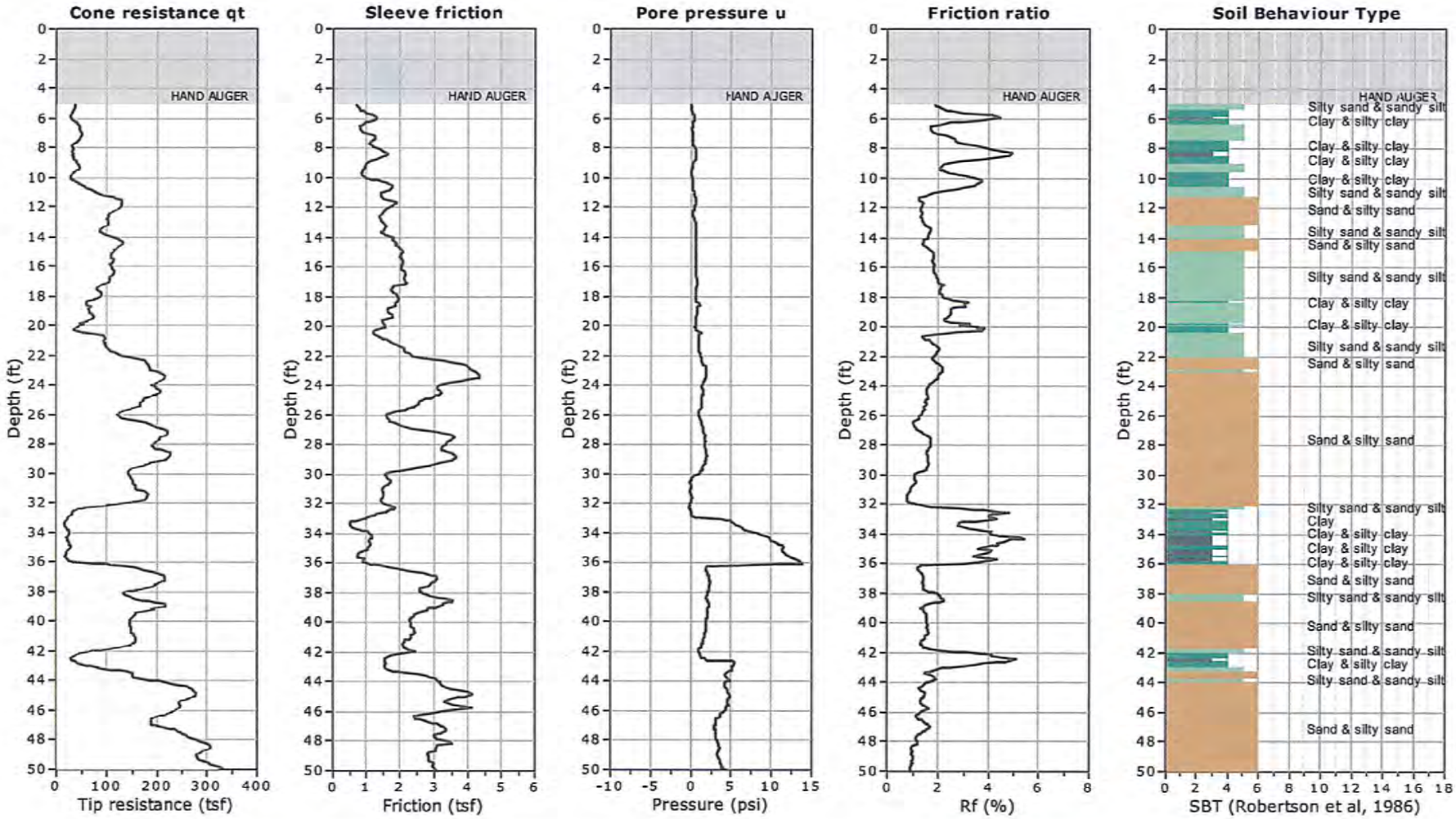


Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT: CPT-3

Project: **Petra Geotechnical**
Location: **4747 Daisy Ave. Long Beach, CA**

Total depth: 50.56 ft, Date: 8/26/2013
Cone Type: Vertek



Kehoe Testing and Engineering
714-901-7270
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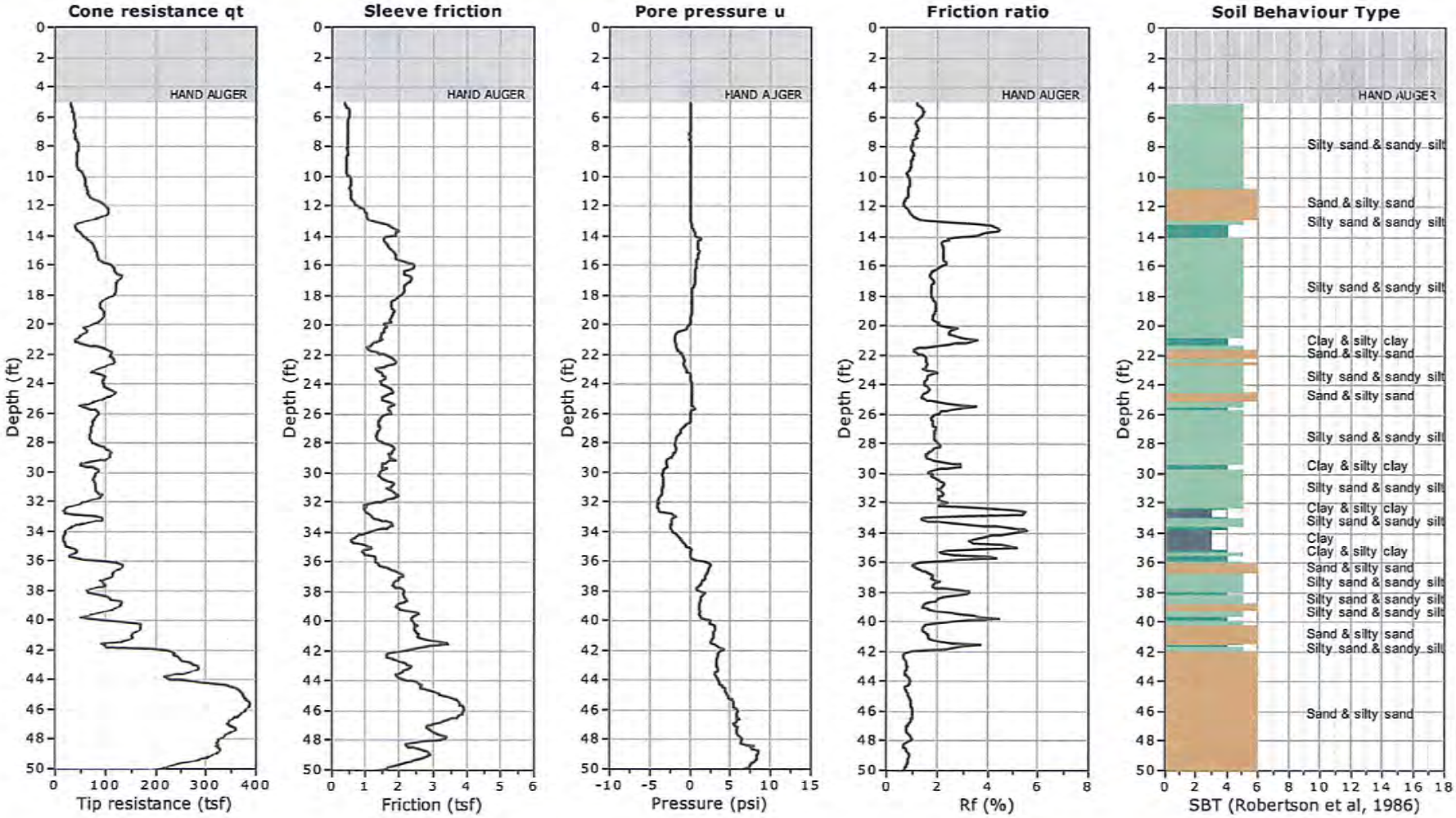
www.kehoetesting.com

CPT: CPT-4
Total depth: 50.48 ft, Date: 8/26/2013
Cone Type: Vertek

Project: **Petra Geotechnical**
Location: **4747 Daisy Ave. Long Beach, CA**



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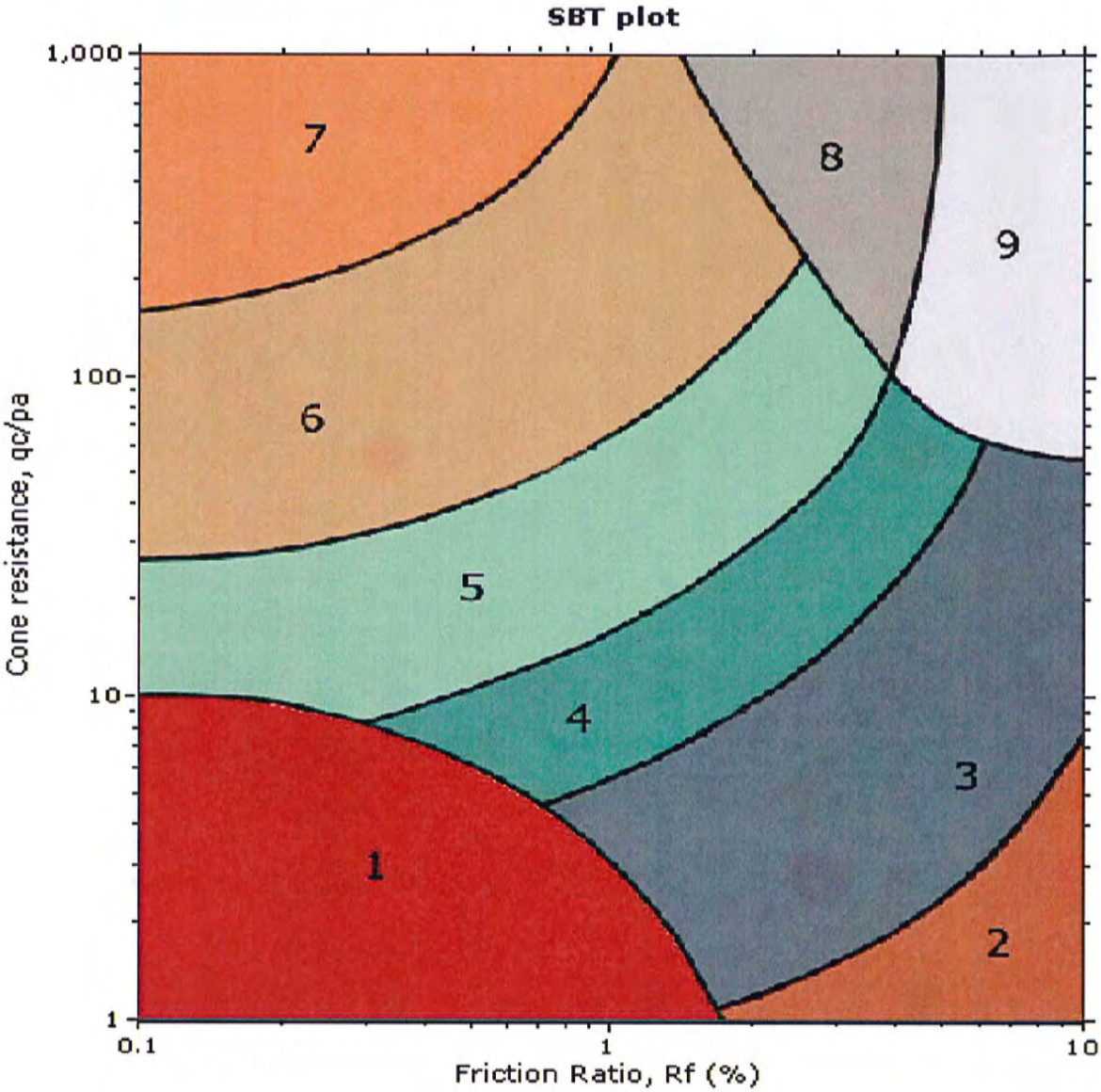


SBT legend

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

	CPT-1	In situ data										Basic output data								
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	82.1	0.64	0.47	0.78	82.1058	0.7795	6	1.9321	117.757	0.05888	0	0.0589	1393.5	0.78	0.0004	6	0.4987	2	1.6952	155.0822
2	68.7	0.38	0.32	0.55	68.7039	0.5531	6	1.91689	113.508	0.11563	0	0.1156	593.16	0.554	0.0003	6	0.4896	2	1.6645	129.6433
3	138.7	1.15	0.98	0.83	138.712	0.8291	6	1.76788	123.3241	0.17729	0	0.1773	781.38	0.8301	0.0005	6	0.4491	2	1.5506	261.8536
4	122.7	1.3	0.78	1.06	122.71	1.0594	6	1.87778	123.9222	0.23926	0	0.2393	511.88	1.0615	0.0005	6	0.4959	2	1.6656	231.4892
5	63.1	0.55	0.91	0.88	63.1111	0.8715	6	2.0536	116.0064	0.29726	0	0.2973	211.31	0.8756	0.001	6	0.556	2	1.8161	118.7287
6	64.2	0.5	1.09	0.78	64.2133	0.7787	6	2.02008	115.3512	0.35493	0	0.3549	179.92	0.783	0.0012	6	0.5558	1.8351	1.8091	110.7504
7	78.1	0.67	1.23	0.86	78.1151	0.8577	6	1.97381	117.9707	0.41392	0	0.4139	187.72	0.8623	0.0011	6	0.5545	1.6827	1.7985	123.5663
8	142.3	1.21	1.15	0.85	142.314	0.8502	6	1.76649	123.7588	0.4758	0	0.4758	298.11	0.8531	0.0006	6	0.4975	1.4883	1.6413	199.5
9	69.7	0.87	0.73	1.25	69.7089	1.2481	5	2.11165	119.6043	0.5356	0	0.5356	129.15	1.2577	0.0008	6	0.6274	1.5329	1.9747	100.2154
10	94.8	1.78	1.22	1.88	94.8149	1.8773	5	2.12931	125.5926	0.5984	0	0.5984	157.45	1.8893	0.0009	5	0.6485	1.4472	2.0219	128.8649
11	49.6	1.12	0.85	2.25	49.6104	2.2576	5	2.39011	120.6229	0.65871	0	0.6587	74.315	2.288	0.0013	5	0.7514	1.4278	2.2842	66.0532
12	44.6	0.96	0	2.14	44.6	2.1525	5	2.41171	119.2353	0.71833	0	0.7183	61.089	2.1877	0	5	0.7694	1.3472	2.3242	55.86963
13	43.5	0.72	-0.39	1.67	43.4952	1.6554	5	2.3485	117.0692	0.77686	0	0.7769	54.988	1.6855	-7E-04	5	0.7555	1.2629	2.2803	50.98689
14	84.8	2.1	-0.41	2.48	84.795	2.4766	5	2.24886	126.5298	0.84013	0	0.8401	99.931	2.5013	-4E-04	5	0.7299	1.1834	2.2047	93.89396
15	111.1	2.62	-0.46	2.36	111.094	2.3584	5	2.15303	128.8075	0.90453	0	0.9045	121.82	2.3777	-3E-04	5	0.7029	1.1165	2.1261	116.2741
16	133.5	2.61	-0.43	1.96	133.495	1.9551	5	2.03912	129.2275	0.96914	0	0.9691	136.75	1.9694	-2E-04	5	0.668	1.0604	2.0266	132.8148
17	125.3	2.37	-0.36	1.89	125.296	1.8915	5	2.04717	128.3671	1.03333	0	1.0333	120.25	1.9073	-2E-04	5	0.6789	1.0162	2.0475	119.343
18	97.8	1.92	-0.35	1.97	97.7957	1.9633	5	2.13349	126.2221	1.09644	0	1.0964	88.194	1.9855	-3E-04	5	0.7203	0.9747	2.1483	89.07615
19	57.3	1.8	1.06	3.14	57.313	3.1407	4	2.44189	124.4465	1.15866	0	1.1587	48.465	3.2055	0.0014	4	0.849	0.9258	2.4783	49.13367
20	122.1	2.13	0	1.75	122.1	1.7445	5	2.02938	127.5229	1.22242	0	1.2224	98.884	1.7621	0	5	0.695	0.9045	2.066	103.3343
21	172.9	3.29	0.05	1.9	172.901	1.9028	6	1.95642	131.5525	1.2882	0	1.2882	133.22	1.9171	2E-05	5	0.6724	0.8761	1.9984	142.0886
22	131.2	2.4	-0.13	1.83	131.198	1.8293	5	2.02292	128.5714	1.35249	0	1.3525	96.005	1.8483	-7E-05	5	0.7071	0.8407	2.0813	103.1614
23	133	2.54	-0.21	1.91	132.997	1.9098	5	2.03267	129.0195	1.41699	0	1.417	92.859	1.9304	-1E-04	5	0.718	0.8108	2.1017	100.8294
24	118.2	2.34	-0.28	1.98	118.197	1.9798	5	2.07894	128.1317	1.48106	0	1.4811	78.805	2.0049	-2E-04	5	0.7443	0.7786	2.1625	85.88193
25	64.9	1.69	-0.84	2.61	64.8897	2.6044	5	2.34646	124.288	1.5432	0	1.5432	41.049	2.6679	-1E-03	4	0.8622	0.7223	2.4645	43.24
26	30.7	1.19	2.69	3.87	30.7329	3.8721	4	2.70119	119.8986	1.60315	0	1.6032	18.17	4.0852	0.0067	3	1	0.66	2.8706	18.17029
27	56.3	1.54	-2.14	2.74	56.2738	2.7366	5	2.40597	123.2604	1.66478	0	1.6648	32.802	2.8201	-0.003	4	0.9029	0.6642	2.5561	34.27892
28	59.9	1.62	-8.72	2.71	59.7933	2.7093	5	2.38392	123.7789	1.72667	0	1.7267	33.629	2.7899	-0.011	4	0.901	0.6432	2.5435	35.29976
29	35.3	1.19	-7.44	3.39	35.2089	3.3798	4	2.6178	120.2302	1.78679	0	1.7868	18.705	3.5605	-0.016	3	1	0.5922	2.8231	18.70515
30	53.6	1.64	-0.18	3.06	53.5978	3.0598	4	2.45479	123.6019	1.84859	0	1.8486	27.994	3.1691	-3E-04	4	0.9457	0.59	2.6459	28.85504
31	38.3	2.25	-7.24	5.9	38.2114	5.8883	3	2.75991	125.0905	1.91113	0	1.9111	18.994	6.1983	-0.014	3	1	0.5537	2.9751	18.99408
32	76.7	2.66	-5.82	3.47	76.6288	3.4713	5	2.38576	128.0125	1.97514	0	1.9751	37.797	3.5631	-0.006	4	0.9259	0.5611	2.5779	39.58583
33	102.9	2.93	-5.63	2.85	102.831	2.8493	5	2.23658	129.4372	2.03986	0	2.0399	49.411	2.907	-0.004	5	0.8693	0.5652	2.4203	53.83831
34	194.8	2.02	0.82	1.04	194.81	1.0369	6	1.72594	128.2744	2.104	0.0312	2.0728	92.969	1.0482	0.0001	6	0.6616	0.6409	1.8726	116.7243
35	48.1	2.11	7.26	4.38	48.1889	4.3786	4	2.59738	125.1863	2.16659	0.0624	2.1042	21.872	4.5847	0.01	3	1	0.5029	2.842	21.87173
36	227.8	2.77	-3.98	1.21	227.751	1.2162	6	1.7309	130.9658	2.23207	0.0936	2.1385	105.46	1.2283	-0.002	6	0.6664	0.6257	1.877	133.3601
37	252	3.88	-1.64	1.54	251.98	1.5398	6	1.78212	133.6781	2.29891	0.1248	2.1741	114.84	1.554	-1E-03	6	0.6871	0.6097	1.9271	143.8669
38	264.2	3.81	-1.59	1.44	264.181	1.4422	6	1.74707	133.6602	2.36574	0.156	2.2097	118.48	1.4552	-0.001	6	0.6758	0.6079	1.893	150.4267
39	292.6	4.27	0.1	1.46	292.601	1.4593	6	1.72429	134.7434	2.43311	0.1872	2.2459	129.2	1.4716	-6E-04	6	0.668	0.6049	1.8679	165.8764
40	267.5	3.49	0.98	1.3	267.512	1.3046	6	1.7095	133.0488	2.49964	0.2184	2.2812	116.17	1.3169	-6E-04	6	0.6676	0.5988	1.8625	149.9652
41	288	3.35	0.62	1.16	288.008	1.1632	6	1.65056	132.9293	2.5661	0.2496	2.3165	123.22	1.1736	-7E-04	6	0.6463	0.6026	1.8022	162.5679
42	260.8	3.34	0.15	1.28	260.802	1.2807	6	1.71016	132.6654	2.63244	0.2808	2.3516	109.78	1.2937	-0.001	6	0.6749	0.5833	1.8728	142.3265
43	254.3	3.06	0.46	1.21	254.306	1.2033	6	1.69625	131.9633	2.69842	0.312	2.3864	105.43	1.2162	-0.001	6	0.6734	0.5783	1.8645	137.5097
44	64.9	2.25	0.07	3.47	64.9009	3.4668	4	2.43463	126.3825	2.76161	0.3432	2.4184	25.694	3.6209	-0.005	4	1	0.4375	2.7219	25.69427
45	13.2	0.31	69	2.2	14.0446	2.2073	4	2.82031	108.1462	2.81568	0.3744	2.4413	4.5996	2.7607	0.4091	3	1	0.4334	3.2619	4.59958
46	22.8	1.17	10.82	5.1	22.9324	5.1019	3	2.87584	119.0604	2.87521	0.4056	2.4696	8.1216	5.8333	0.0186	3	1	0.4285	3.2403	8.12161
47	20.7	0.62	34.47	2.94	21.1219	2.9353	4	2.74886	114.2133	2.93232	0.4368	2.4955	7.2889	3.4085	0.1124	3	1	0.424	3.1416	7.2889
48	91.2	2.29	0.34	2.52	91.2042	2.5109	5	2.23127	127.3413	2.99599	0.468	2.528	34.893	2.5961	-0.005	4	0.9248	0.4469	2.5053	37.25475
49	307.7	3.61	6.19	1.17	307.776	1.1729	6	1.6355	133.6382	3.06281	0.4992	2.5636	118.86	1.1847	-2E-04	6	0.6607	0.5573	1.809	160.4847
50	351.6	4.78	6.96	1.36	351.685	1.3592	6	1.6525	136.0176	3.13082	0.5304	2.6004	134.04	1.3714	-8E-05	6	0.6666	0.5492	1.8199	180.8972

	CPT-2	In situ data										Basic output data								
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	39.8	0.43	0.02	1.09	39.8002	1.0804	5	2.27182	113.081	0.05654	0	0.0565	702.92	1.0819	4E-05	6	0.6255	2	2.0285	75.12221
2	61.7	0.83	0.11	1.34	61.7014	1.3452	5	2.17339	118.9623	0.11602	0	0.116	530.81	1.3477	0.0001	6	0.5975	2	1.9475	116.4065
3	67.8	1.32	0.02	1.96	67.8002	1.9469	5	2.24604	122.587	0.17732	0	0.1773	381.37	1.952	2E-05	5	0.6336	2	2.0348	127.8186
4	40.3	1	-0.14	2.49	40.2983	2.4815	4	2.48526	119.2866	0.23696	0	0.237	169.06	2.4962	-3E-04	5	0.7255	2	2.2685	75.72257
5	47.2	0.86	-0.06	1.82	47.1993	1.8221	5	2.34662	118.5686	0.29624	0	0.2962	158.33	1.8336	-9E-05	5	0.6738	2	2.1255	88.65451
6	70.7	1.3	-0.06	1.84	70.6993	1.8388	5	2.2159	122.5774	0.35753	0	0.3575	196.74	1.8481	-6E-05	5	0.632	1.9852	2.0079	131.9757
7	66.9	2.26	-0.75	3.39	66.8908	3.3786	4	2.41748	126.4886	0.42078	0	0.4208	157.97	3.4	-8E-04	5	0.7205	1.9434	2.2319	122.0836
8	56.9	2.45	-0.06	4.31	56.8993	4.3059	4	2.54227	126.6847	0.48412	0	0.4841	116.53	4.3428	-8E-05	9	0.7783	1.8378	2.3745	97.9866
9	53.1	1.69	-0.06	3.18	53.0993	3.1827	4	2.46961	123.7989	0.54602	0	0.546	96.248	3.2158	-8E-05	5	0.7613	1.6547	2.3243	82.18512
10	131.9	1.37	0.22	1.04	131.903	1.0386	6	1.84864	124.4822	0.60826	0	0.6083	215.85	1.0435	0.0001	6	0.5476	1.3541	1.7558	168.0265
11	107.8	1.79	0.11	1.66	107.801	1.6605	5	2.05218	125.9466	0.67123	0	0.6712	159.6	1.6709	7E-05	6	0.6319	1.3332	1.969	134.9809
12	109.1	1.76	0.13	1.61	109.102	1.6132	5	2.03967	125.8522	0.73416	0	0.7342	147.61	1.6241	9E-05	6	0.6364	1.2619	1.9729	129.2362
13	119.9	2.07	0.44	1.73	119.905	1.7264	5	2.0316	127.2696	0.79779	0	0.7978	149.3	1.7379	0.0003	6	0.6427	1.199	1.9816	134.9667
14	139.4	2.45	0.61	1.76	139.407	1.7574	6	1.99224	128.8703	0.86223	0	0.8622	160.68	1.7684	0.0003	6	0.637	1.1393	1.9581	149.174
15	162.4	2.39	0.77	1.47	162.409	1.4716	6	1.89061	129.0614	0.92676	0	0.9268	174.24	1.48	0.0003	6	0.6066	1.0837	1.8706	165.3911
16	144.6	2.02	0.94	1.39	144.612	1.3969	6	1.90894	127.5476	0.99053	0	0.9905	144.99	1.4065	0.0005	6	0.621	1.0418	1.9008	141.4123
17	150.3	2.96	1	1.97	150.312	1.9692	5	2.00724	130.4377	1.05575	0	1.0558	141.37	1.9832	0.0005	5	0.6661	1.0015	2.0111	141.2695
18	127.8	2.31	0.98	1.8	127.812	1.8073	5	2.02684	128.228	1.11987	0	1.1199	113.13	1.8233	0.0006	5	0.6816	0.9621	2.0438	115.1934
19	112.5	2.28	1.02	2.03	112.512	2.0264	5	2.10098	127.8214	1.18378	0	1.1838	94.045	2.048	0.0007	5	0.7182	0.9226	2.1317	97.0677
20	89	1.64	0.87	1.85	89.0107	1.8425	5	2.1433	124.8391	1.2462	0	1.2462	70.426	1.8686	0.0007	5	0.7435	0.8855	2.1903	73.44425
21	119	1.98	0.61	1.67	119.007	1.6638	5	2.02242	126.926	1.30966	0	1.3097	89.869	1.6823	0.0004	5	0.7028	0.8608	2.0754	95.75003
22	127.5	1.89	0.77	1.48	127.509	1.4822	6	1.96569	126.7539	1.37304	0	1.373	91.867	1.4984	0.0004	5	0.6879	0.8359	2.0281	99.64956
23	103.9	1.35	0.77	1.3	103.909	1.2992	6	1.99068	123.7928	1.43493	0	1.4349	71.414	1.3174	0.0005	5	0.7063	0.8064	2.0695	78.09804
24	89.1	0.93	0.85	1.05	89.1104	1.0437	6	1.97985	120.6912	1.49528	0	1.4953	58.595	1.0615	0.0007	5	0.7111	0.782	2.0745	64.75151
25	30.8	0.31	0.94	1.01	30.8115	1.0061	5	2.34908	110.0624	1.55031	0	1.5503	18.874	1.0594	0.0023	5	0.8781	0.7151	2.5052	19.77413
26	62.8	0.72	0.94	1.14	62.8115	1.1463	5	2.12478	117.9655	1.60929	0	1.6093	38.031	1.1764	0.0011	5	0.7858	0.7193	2.2564	41.60508
27	66.1	1.13	0.94	1.72	66.1115	1.7092	5	2.2167	121.3883	1.66999	0	1.67	38.588	1.7535	0.0011	5	0.8282	0.6853	2.3587	41.73571
28	75.1	1	0.69	1.34	75.1085	1.3314	5	2.10424	120.8052	1.73039	0	1.7304	42.406	1.3628	0.0007	5	0.7894	0.6782	2.2507	47.0349
29	110.5	1.42	0.69	1.28	110.508	1.285	6	1.96768	124.3128	1.79254	0	1.7925	60.649	1.3062	0.0005	5	0.7369	0.6781	2.1052	69.67346
30	117.3	1.51	-1.96	1.29	117.276	1.2876	6	1.94931	124.9074	1.855	0	1.855	62.222	1.3083	-0.001	5	0.7355	0.6617	2.0938	72.18323
31	105.2	1.36	-3.33	1.3	105.159	1.2933	6	1.9855	123.8759	1.91694	0	1.9169	53.858	1.3173	-0.002	5	0.7583	0.6372	2.1459	62.17744
32	124.9	1.63	-5.12	1.3	124.837	1.3057	6	1.93375	125.6194	1.97975	0	1.9798	62.057	1.3267	-0.003	5	0.7415	0.6284	2.0941	72.96616
33	119.4	1.62	-1.04	1.36	119.387	1.3569	6	1.95932	125.4655	2.04248	0	2.0425	57.452	1.3806	-6E-04	5	0.7589	0.6071	2.132	67.32266
34	120.9	1.34	-2.3	1.11	120.872	1.1086	6	1.89578	124.1072	2.10453	0.0312	2.0733	57.283	1.1283	-0.002	5	0.7376	0.6089	2.072	68.34273
35	137.1	1.43	0.16	1.04	137.102	1.043	6	1.83743	124.8901	2.16698	0.0624	2.1046	64.115	1.0598	-4E-04	6	0.7155	0.6114	2.0102	77.96764
36	83.1	0.88	-4.82	1.06	83.041	1.0597	6	2.00795	120.1148	2.22703	0.0936	2.1334	37.88	1.0889	-0.005	5	0.7968	0.5719	2.2199	43.68258
37	69.2	0.43	-5.78	0.62	69.1293	0.622	6	1.94072	114.4276	2.28425	0.1248	2.1595	30.955	0.6433	-0.008	5	0.779	0.5737	2.17	36.23979
38	25.1	0.64	-2.25	2.54	25.0725	2.5526	4	2.65283	114.8638	2.34168	0.156	2.1857	10.4	2.8156	-0.014	3	1	0.4841	2.9672	10.39986
39	43.5	1.58	3.86	3.63	43.5473	3.6282	4	2.57108	122.8227	2.40309	0.1872	2.2159	18.568	3.8402	0.0022	3	1	0.4775	2.8463	18.56776
40	94.4	1.38	0.08	1.46	94.401	1.4619	5	2.05597	123.7195	2.46495	0.2184	2.2466	40.923	1.501	-0.002	5	0.8245	0.5375	2.2786	46.70509
41	23.5	0.5	12.37	2.11	23.6514	2.114	4	2.62386	112.9152	2.52141	0.2496	2.2718	9.301	2.3663	0.0303	3	1	0.4658	2.9662	9.30096
42	53.3	1.36	12.52	2.54	53.4532	2.5443	5	2.40066	122.2255	2.58252	0.2808	2.3017	22.101	2.6734	0.0122	4	0.9816	0.4663	2.6841	22.41958
43	32.6	1.71	10.57	5.23	32.7294	5.2247	3	2.77035	122.7048	2.64387	0.312	2.3319	12.902	5.6838	0.0149	3	1	0.4538	3.0766	12.90185
44	190.4	3.52	4.24	1.85	190.452	1.8482	6	1.91991	132.2828	2.71002	0.3432	2.3668	79.323	1.8749	-2E-04	5	0.7653	0.5401	2.1083	95.82264
45	171.6	3.7	0.62	2.15	171.608	2.1561	5	2.00036	132.3936	2.77621	0.3744	2.4018	70.293	2.1915	-0.002	5	0.8025	0.518	2.2016	82.64948
46	152.5	3.58	0.7	2.35	152.509	2.3474	5	2.06139	131.8646	2.84214	0.4056	2.4365	61.426	2.392	-0.002	5	0.8328	0.4993	2.2768	70.62141
47	268.4	2.81	7.46	1.05	268.491	1.0466	6	1.63479	131.472	2.90788	0.4368	2.4711	107.48	1.0581	0.0004	6	0.6561	0.5732	1.8084	143.8789
48	329.7	4.49	7.31	1.36	329.789	1.3615	6	1.66928	135.4028	2.97558	0.468	2.5076	130.33	1.3739	0.0002	6	0.6669	0.5625	1.8323	173.7324
49	218.1	3.54	4.42	1.62	218.154	1.6227	6	1.83885	132.6555	3.04191	0.4992	2.5427	84.6	1.6457	-8E-04	5	0.747	0.5195	2.0384	105.6078
50	216.2	3.05	5.15	1.41	216.263	1.4103	6	1.79432	131.5441	3.10768	0.5304	2.5773	82.705	1.4309	-8E-04	6	0.7334	0.5205	1.9983	104.8583

	CPT-3	In situ data										Basic output data								
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	54.6	0.44	0.06	0.8	54.6007	0.8059	5	2.08726	114.0204	0.05701	0	0.057	956.74	0.8067	8E-05	6	0.5544	2	1.8416	103.0967
2	36.6	0.49	0.06	1.34	36.6007	1.3388	5	2.35426	113.8323	0.11393	0	0.1139	320.27	1.343	0.0001	5	0.6617	2	2.1163	68.96613
3	35.7	0.59	0.14	1.66	35.7017	1.6526	5	2.41641	115.1306	0.17149	0	0.1715	207.18	1.6606	0.0003	5	0.6905	2	2.1848	67.15803
4	45.3	0.89	0.2	1.96	45.3025	1.9646	5	2.38112	118.7195	0.23085	0	0.2309	195.24	1.9746	0.0003	5	0.684	2	2.1603	85.19281
5	37.1	0.68	0.06	1.85	37.1007	1.8329	5	2.4302	116.2631	0.28898	0	0.289	127.38	1.8472	0.0001	5	0.7035	2	2.2042	69.58033
6	31.3	1.3	0.41	4.16	31.305	4.1527	4	2.71572	120.5904	0.34928	0	0.3493	88.628	4.1995	0.001	4	0.8226	2	2.5093	58.51149
7	51.2	1.03	0.24	2.02	51.2029	2.0116	5	2.34697	120.087	0.40932	0	0.4093	124.09	2.0278	0.0003	5	0.6869	1.9201	2.1443	92.17314
8	33.3	1.35	0.44	4.05	33.3054	4.0534	4	2.68882	121.0177	0.46983	0	0.4698	69.888	4.1114	0.001	4	0.8222	1.9494	2.4928	60.49535
9	42	1.01	0.25	2.4	42.0031	2.4046	5	2.4627	119.4605	0.52956	0	0.5296	78.317	2.4353	0.0004	5	0.7525	1.6834	2.3033	65.98383
10	27.3	1.01	0.25	3.72	27.3031	3.6992	4	2.72654	118.4099	0.58877	0	0.5888	45.373	3.7808	0.0007	4	0.8602	1.6558	2.5786	41.80387
11	79.7	1.29	0.37	1.61	79.7045	1.6185	5	2.14013	122.8133	0.65017	0	0.6502	121.59	1.6318	0.0003	5	0.6592	1.3786	2.0435	102.9968
12	124.8	1.64	0.44	1.31	124.805	1.3141	6	1.93575	125.6635	0.713	0	0.713	174.04	1.3216	0.0003	6	0.595	1.2647	1.8669	148.3243
13	104.6	1.49	0.53	1.42	104.606	1.4244	6	2.01554	124.531	0.77527	0	0.7753	133.93	1.435	0.0004	6	0.6328	1.2175	1.9584	119.4735
14	113.3	1.8	0.71	1.59	113.309	1.5886	5	2.02327	126.1089	0.83832	0	0.8383	134.16	1.6004	0.0005	6	0.645	1.162	1.9819	123.5175
15	112.9	2.08	0.63	1.84	112.908	1.8422	5	2.07003	127.1582	0.9019	0	0.9019	124.19	1.8571	0.0004	5	0.671	1.1131	2.0428	117.8313
16	109	2	0.72	1.83	109.009	1.8347	5	2.07943	126.7855	0.9653	0	0.9653	111.93	1.8511	0.0005	5	0.6828	1.0647	2.0659	108.7152
17	104.9	2.17	0.82	2.07	104.91	2.0684	5	2.12839	127.2889	1.02894	0	1.0289	100.96	2.0889	0.0006	5	0.7096	1.02	2.1285	100.143
18	87.9	1.92	0.57	2.19	87.907	2.1841	5	2.19896	125.9621	1.09192	0	1.0919	79.507	2.2116	0.0005	5	0.7451	0.9768	2.2138	80.14684
19	65.6	1.61	0.63	2.46	65.6077	2.454	5	2.32507	123.96	1.1539	0	1.1539	55.857	2.4979	0.0007	5	0.8029	0.9328	2.3579	56.81968
20	43	1.54	0.46	3.59	43.0056	3.5809	4	2.57107	122.6046	1.2152	0	1.2152	34.39	3.6851	0.0008	4	0.9089	0.8818	2.6285	34.82607
21	90.8	1.7	0.85	1.87	90.8104	1.872	5	2.14185	125.1509	1.27778	0	1.2778	70.069	1.8988	0.0007	5	0.7468	0.8686	2.1948	73.49768
22	132.5	2.39	1.11	1.81	132.514	1.8036	5	2.01547	128.5652	1.34206	0	1.3421	97.739	1.822	0.0006	5	0.703	0.8461	2.0718	104.889
23	188.8	4.17	1.76	2.21	188.822	2.2084	6	1.98269	133.5017	1.40881	0	1.4088	133.03	2.225	0.0007	5	0.6947	0.8197	2.0415	145.1803
24	191.2	3.16	1.38	1.65	191.217	1.6526	6	1.88141	131.5031	1.47456	0	1.4746	128.68	1.6654	0.0005	6	0.6623	0.8027	1.9491	143.9381
25	172.2	2.63	1.21	1.53	172.215	1.5272	6	1.88537	129.9045	1.53952	0	1.5395	110.86	1.5409	0.0005	6	0.6717	0.7774	1.9656	125.3884
26	120.1	1.55	1	1.29	120.112	1.2905	6	1.94241	125.157	1.60209	0	1.6021	73.972	1.3079	0.0006	5	0.7049	0.7465	2.045	83.60532
27	213.5	2.74	1.49	1.28	213.518	1.2833	6	1.76688	130.7287	1.66746	0	1.6675	127.05	1.2934	0.0005	6	0.6364	0.7487	1.857	149.9005
28	191.7	3.31	1.67	1.72	191.72	1.7265	6	1.89524	131.8489	1.73338	0	1.7334	109.6	1.7422	0.0006	6	0.6936	0.7101	1.9992	127.4974
29	218.6	3.53	1.76	1.62	218.622	1.6147	6	1.83659	132.64	1.7997	0	1.7997	120.48	1.6281	0.0006	6	0.6756	0.6985	1.9434	143.1334
30	143.4	1.47	0.91	1.02	143.411	1.025	6	1.81791	125.2017	1.8623	0	1.8623	76.007	1.0385	0.0005	6	0.682	0.6801	1.9524	90.97711
31	159.6	1.46	-0.06	0.91	159.599	0.9148	6	1.75028	125.4126	1.92501	0	1.925	81.908	0.926	-3E-05	6	0.6604	0.6735	1.8879	100.3666
32	134.3	1.52	-0.13	1.13	134.298	1.1318	6	1.86808	125.2863	1.98765	0	1.9877	66.566	1.1488	-7E-05	6	0.7156	0.6369	2.0249	79.64079
33	21.5	0.85	3.89	3.96	21.5476	3.9448	3	2.82284	116.5706	2.04594	0	2.0459	9.5319	4.3586	0.0144	3	1	0.5172	3.1083	9.53189
34	26.8	1.1	8.28	4.1	26.9014	4.089	4	2.76008	118.9984	2.10544	0.0312	2.0742	11.954	4.4362	0.0228	3	1	0.5101	3.0348	11.95422
35	28	1.05	11.55	3.73	28.1414	3.7312	4	2.7191	118.7679	2.16482	0.0624	2.1024	12.356	4.0421	0.0296	3	1	0.5033	2.9987	12.35553
36	59.6	1.05	13.83	1.76	59.7693	1.7568	5	2.25767	120.6051	2.22512	0.0936	2.1315	26.997	1.8247	0.0157	5	0.9025	0.5315	2.496	28.90486
37	216.4	3.15	2.24	1.45	216.427	1.4555	6	1.80457	131.782	2.29102	0.1248	2.1662	98.853	1.471	0.0002	6	0.6985	0.6063	1.9578	122.6924
38	135.4	2.67	2.07	1.97	135.425	1.9716	5	2.03764	129.4288	2.35573	0.156	2.1997	60.494	2.0065	-5E-05	5	0.8026	0.5558	2.2272	69.89481
39	216.4	2.97	2.18	1.37	216.427	1.3723	6	1.78506	131.3515	2.42141	0.1872	2.2342	95.786	1.3878	-1E-04	6	0.6974	0.5938	1.9466	120.091
40	144.4	2.32	1.81	1.61	144.422	1.6064	6	1.9532	128.5576	2.48568	0.2184	2.2673	62.602	1.6345	-6E-04	5	0.7755	0.5538	2.1475	74.28299
41	156.7	2.28	1.67	1.46	156.72	1.4548	6	1.89753	128.6297	2.55	0.2496	2.3004	67.019	1.4789	-8E-04	5	0.7553	0.5562	2.0903	81.0442
42	66	2.3	0.86	3.48	66.0105	3.4843	4	2.43113	126.5847	2.61329	0.2808	2.3325	27.18	3.6279	-0.003	4	0.9904	0.4571	2.7015	27.38637
43	69.2	1.55	5.19	2.23	69.2635	2.2378	5	2.28039	123.8143	2.6752	0.312	2.3632	28.177	2.3277	0.0009	4	0.9342	0.4721	2.5509	29.70711
44	203.3	3.19	4.59	1.57	203.356	1.5687	6	1.84688	131.7224	2.74106	0.3432	2.3979	83.664	1.5901	-6E-05	5	0.7384	0.5466	2.0337	103.6317
45	276.1	3.88	4.87	1.4	276.16	1.405	6	1.7263	133.9016	2.80801	0.3744	2.4336	112.32	1.4194	-9E-05	6	0.6871	0.5643	1.8945	145.77
46	234.6	3.04	4.21	1.29	234.652	1.2955	6	1.74328	131.7191	2.87387	0.4056	2.4683	93.903	1.3116	-4E-04	6	0.7012	0.5521	1.9273	120.9438
47	193.9	3.28	2.85	1.69	193.935	1.6913	6	1.88517	131.8103	2.93978	0.4368	2.503	76.307	1.7173	-0.001	5	0.7645	0.5178	2.0892	93.46103
48	280.9	3.4	3.31	1.21	280.941	1.2102	6	1.67075	132.9771	3.00626	0.468	2.5383	109.5	1.2233	-8E-04	6	0.6746	0.5542	1.8487	145.5622
49	258.2	2.83	3.29	1.1	258.24	1.0959	6	1.66097	131.429	3.07198	0.4992	2.5728	99.18	1.1091	-0.001	6	0.6765	0.5483	1.8491	132.213
50	323.1	3.09	3.97	0.96	323.149	0.9562	6	1.553	132.619	3.13829	0.5304	2.6079	122.71	0.9656	-8E-04	6	0.6315	0.5657	1.7266	171.099

	CPT-4	In situ data										Basic output data									
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn
1	129.1	1.32	0.12	1.02	129.101	1.0225	6	1.85104	124.1578	0.06208	0	0.0621	2078.6	1.0229	7E-05	6	0.4772	2	1.6386	243.9059
2	68.2	0.65	0.03	0.95	68.2004	0.9531	6	2.04841	117.4179	0.12079	0	0.1208	563.63	0.9548	3E-05	6	0.5469	2	1.814	128.6817
3	46.9	0.83	-0.16	1.77	46.898	1.7698	5	2.34083	118.2932	0.17993	0	0.1799	259.64	1.7766	-3E-04	5	0.6651	2	2.1172	88.30499
4	27	0.42	-0.16	1.56	26.998	1.5557	5	2.50006	111.9622	0.23592	0	0.2359	113.44	1.5694	-4E-04	5	0.7235	2	2.2634	50.58487
5	31.8	0.36	-0.07	1.14	31.7991	1.1321	5	2.36459	111.2335	0.29153	0	0.2915	108.08	1.1426	-2E-04	5	0.6726	2	2.1228	59.55462
6	37.5	0.51	-0.07	1.35	37.4991	1.36	5	2.34956	114.1842	0.34862	0	0.3486	106.56	1.3728	-1E-04	5	0.6728	2	2.1164	70.22066
7	38.7	0.47	-0.1	1.21	38.6988	1.2145	5	2.3103	113.6634	0.40546	0	0.4055	94.445	1.2274	-2E-04	5	0.6676	1.8972	2.0931	68.65983
8	45.7	0.45	-0.07	0.98	45.6991	0.9847	5	2.19953	113.7507	0.46233	0	0.4623	97.845	0.9948	-1E-04	6	0.6403	1.6992	2.0177	72.64417
9	44.3	0.44	-0.07	1	44.2991	0.9933	5	2.21288	113.5104	0.51909	0	0.5191	84.341	1.005	-1E-04	6	0.6566	1.5962	2.0535	66.04354
10	58.7	0.5	-0.06	0.86	58.6993	0.8518	5	2.07413	115.1322	0.57665	0	0.5767	100.79	0.8603	-7E-05	6	0.6187	1.4558	1.9465	79.96562
11	64.8	0.55	0.03	0.85	64.8004	0.8488	6	2.03765	116.0708	0.63469	0	0.6347	101.1	0.8572	3E-05	6	0.6161	1.3701	1.9322	83.08386
12	101.3	0.78	0.12	0.77	101.301	0.77	6	1.85504	119.7169	0.69455	0	0.6946	144.85	0.7753	9E-05	6	0.5602	1.266	1.7781	120.3698
13	59.5	1.22	0.08	2.06	59.501	2.0504	5	2.30325	121.6921	0.75539	0	0.7554	77.768	2.0768	0.0001	5	0.736	1.2815	2.2318	71.14811
14	57.9	1.59	0.83	2.74	57.9102	2.7456	5	2.39795	123.5641	0.81717	0	0.8172	69.866	2.7849	0.0011	5	0.7819	1.2239	2.3441	66.03828
15	81.3	1.81	1.06	2.23	81.313	2.226	5	2.22875	125.3402	0.87984	0	0.8798	91.417	2.2503	0.001	5	0.7278	1.1437	2.1944	86.9392
16	111.3	2.47	0.69	2.22	111.308	2.2191	5	2.13298	128.3808	0.94404	0	0.944	116.91	2.238	0.0005	5	0.7004	1.0832	2.1148	112.9791
17	124.7	2.34	0.6	1.87	124.707	1.8764	5	2.04602	128.2624	1.00817	0	1.0082	122.7	1.8917	0.0004	5	0.6754	1.0332	2.0413	120.7866
18	114.7	2.01	0.22	1.75	114.703	1.7524	5	2.04971	126.9462	1.07164	0	1.0716	106.03	1.7689	0.0001	5	0.6848	0.9913	2.0581	106.4604
19	94.2	1.85	0.22	1.96	94.2027	1.9639	5	2.14507	125.859	1.13457	0	1.1346	82.03	1.9878	0.0002	5	0.7297	0.9504	2.1682	83.59099
20	70.2	1.66	-0.2	2.37	70.1976	2.3648	5	2.29276	124.3487	1.19674	0	1.1967	57.657	2.4058	-2E-04	5	0.7959	0.9067	2.3341	59.12448
21	41.4	1.48	-2.04	3.58	41.375	3.577	4	2.58292	122.2195	1.25785	0	1.2579	31.893	3.6892	-0.004	4	0.9201	0.8529	2.6525	32.33717
22	110.3	1.59	-0.97	1.44	110.288	1.4417	6	2.0024	125.1353	1.32042	0	1.3204	82.525	1.4592	-6E-04	5	0.6971	0.857	2.0589	88.25163
23	82.2	0.68	-0.52	0.83	82.1936	0.8273	6	1.94672	118.2032	1.37952	0	1.3795	58.581	0.8414	-5E-04	6	0.6854	0.8338	2.0215	63.67938
24	97.3	1.5	0.22	1.54	97.3027	1.5416	5	2.06195	124.4035	1.44172	0	1.4417	66.491	1.5648	0.0002	5	0.7355	0.7965	2.1444	72.15975
25	96.5	1.46	0.2	1.51	96.5025	1.5129	5	2.05902	124.1855	1.50382	0	1.5038	63.172	1.5369	0.0002	5	0.7414	0.7706	2.153	69.18371
26	85.6	1.71	0.06	1.99	85.6007	1.9977	5	2.1799	125.0497	1.56634	0	1.5663	53.65	2.0349	5E-05	5	0.7972	0.7315	2.2905	58.09337
27	71.8	1.41	-0.95	1.96	71.7884	1.9641	5	2.23031	123.209	1.62795	0	1.628	43.098	2.0097	-1E-03	5	0.8267	0.7004	2.3603	46.43846
28	76.9	1.6	-1.96	2.09	76.876	2.0813	5	2.22587	124.301	1.6901	0	1.6901	44.486	2.1281	-0.002	5	0.8313	0.6775	2.3646	48.14273
29	107.2	1.74	-2.49	1.62	107.17	1.6236	5	2.04717	125.725	1.75296	0	1.753	60.136	1.6506	-0.002	5	0.7635	0.6801	2.1802	67.76088
30	80	1.43	-3.55	1.79	79.9566	1.7885	5	2.16833	123.5749	1.81475	0	1.8148	43.059	1.83	-0.003	5	0.8226	0.6416	2.3275	47.38378
31	81.2	1.68	-3.47	2.07	81.1575	2.0701	5	2.20728	124.7902	1.87714	0	1.8771	42.235	2.1191	-0.003	5	0.8444	0.6163	2.3771	46.17393
32	54.2	1.26	-4.1	2.33	54.1498	2.3269	5	2.37039	121.6983	1.93799	0	1.938	26.941	2.4133	-0.006	4	0.9243	0.5716	2.5783	28.20356
33	91.8	1.25	-2.05	1.36	91.7749	1.362	5	2.04457	122.9268	1.99945	0	1.9995	44.9	1.3924	-0.002	5	0.7931	0.6037	2.227	51.21983
34	21	1.04	-2.06	4.98	20.9748	4.9583	3	2.8965	117.981	2.05844	0.0312	2.0272	9.3311	5.4979	-0.009	3	1	0.5219	3.1769	9.33106
35	25.7	1.22	-0.29	4.72	25.6965	4.7477	3	2.81824	119.6442	2.11827	0.0624	2.0559	11.469	5.1743	-0.004	3	1	0.5147	3.0903	11.46873
36	110.6	1.33	1.97	1.2	110.624	1.2023	6	1.94794	123.8363	2.18018	0.0936	2.0866	51.972	1.2264	0.0004	5	0.7613	0.5964	2.1326	61.11886
37	113	2.03	1.97	1.8	113.024	1.7961	5	2.06183	126.9827	2.24368	0.1248	2.1189	52.283	1.8325	0.0002	5	0.808	0.5706	2.2513	59.74075
38	57.7	2.16	1.15	3.75	57.7141	3.7426	4	2.49392	125.7976	2.30658	0.156	2.1506	25.764	3.8984	-0.001	4	0.9963	0.4933	2.7412	25.83106
39	128.6	1.85	1.03	1.44	128.613	1.4384	6	1.95384	126.6184	2.36988	0.1872	2.1827	57.838	1.4654	-9E-04	5	0.7701	0.5726	2.1438	68.31532
40	90.7	2.4	2.54	2.65	90.7311	2.6452	5	2.2493	127.6719	2.43372	0.2184	2.2153	39.858	2.7181	-4E-04	4	0.8972	0.5153	2.4717	43.00375
41	154	2.56	2.86	1.66	154.035	1.662	6	1.9451	129.4351	2.49844	0.2496	2.2488	67.384	1.6894	-3E-04	5	0.7689	0.5601	2.1325	80.20913
42	213.7	2.1	3.66	0.98	213.745	0.9825	6	1.6811	128.7848	2.56283	0.2808	2.282	92.541	0.9944	-8E-05	6	0.6624	0.601	1.8486	119.9544
43	276.6	2.38	3.34	0.86	276.641	0.8603	6	1.56245	130.3297	2.628	0.312	2.316	118.31	0.8686	-3E-04	6	0.6135	0.6184	1.7159	160.148
44	253.2	2.43	3.46	0.96	253.242	0.9596	6	1.62334	130.2663	2.69313	0.3432	2.3499	106.62	0.9699	-4E-04	6	0.6422	0.599	1.7871	141.8441
45	374.9	3.39	4.64	0.9	374.957	0.9041	6	1.49362	133.6596	2.75996	0.3744	2.3856	156.02	0.9108	-1E-04	6	0.5855	0.6213	1.6337	218.5402
46	378.1	3.9	5.81	1.03	378.171	1.0313	6	1.53682	134.7059	2.82731	0.4056	2.4217	154.99	1.0391	3E-05	6	0.6049	0.606	1.6803	214.966
47	344.1	2.84	5.42	0.83	344.166	0.8252	6	1.48629	132.1554	2.89339	0.4368	2.4566	138.92	0.8322	-1E-04	6	0.591	0.6079	1.6392	196.0591
48	324.3	2.97	6.51	0.91	324.38	0.9156	6	1.53742	132.3384	2.95956	0.468	2.4916	129	0.924	0	6	0.6153	0.5904	1.6986	179.3482
49	316	2.92	8.38	0.92	316.103	0.9238	6	1.54757	132.1512	3.02563	0.4992	2.5264	123.92	0.9327	0.0003	6	0.623	0.5815	1.7144	172.0525
50	205.7	1.47	6.84	0.71	205.784	0.7143	6	1.59634	126.0824	3.08867	0.5304	2.5583	79.231	0.7252	-2E-04	6	0.6572	0.5598	1.8	107.2349

4747 Daisy Ave
Long Beach, CA

CPT Shear Wave Measurements

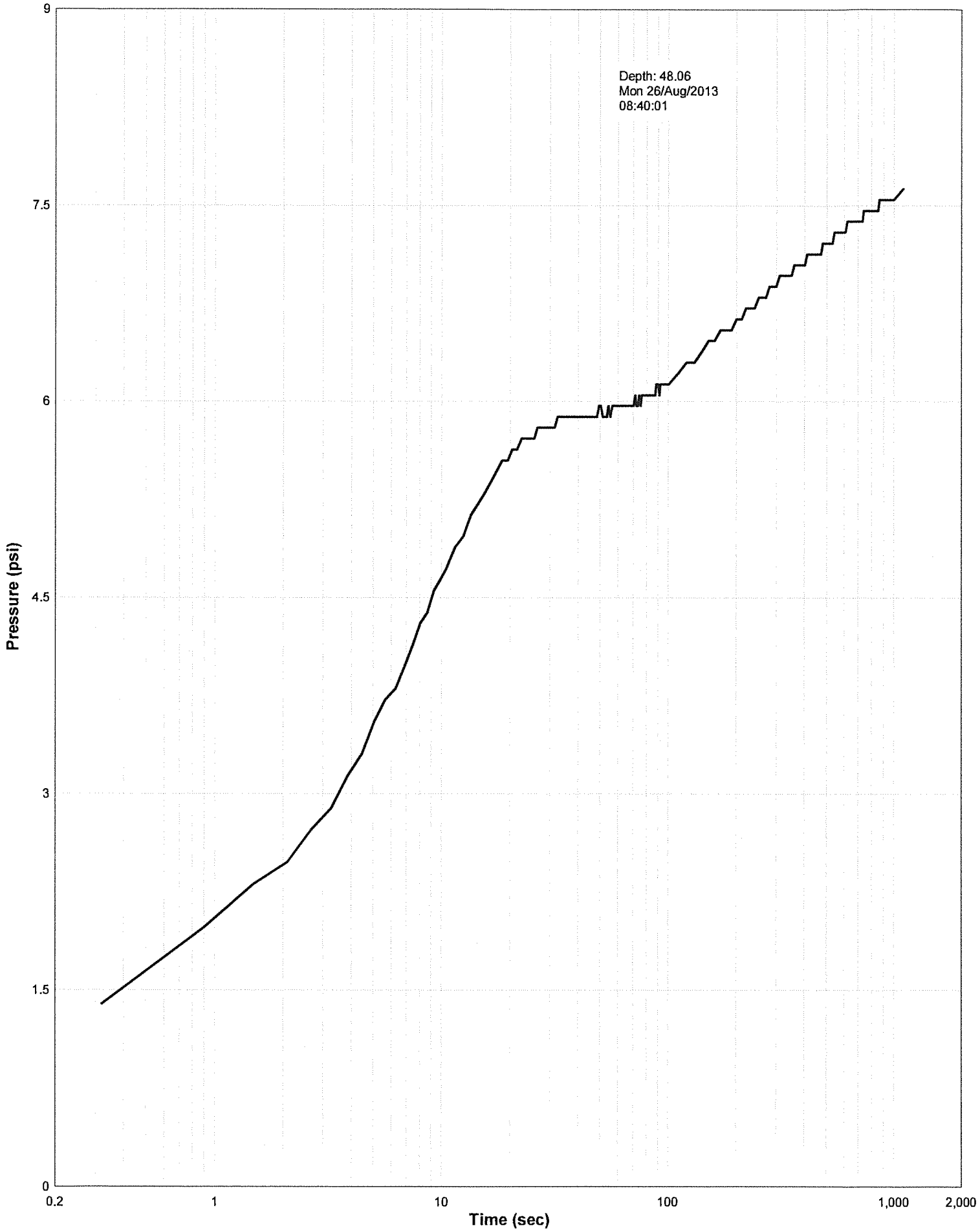
CPT-3	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
	10.10	9.10	10.38	12.65	820.80	
	19.98	18.98	19.63	22.98	854.11	894.91
	30.08	29.08	29.51	32.60	905.11	1026.94
	39.99	38.99	39.31	43.46	904.49	902.63
	49.48	48.48	48.74	51.79	941.05	1131.80

Shear Wave Source Offset = 5 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

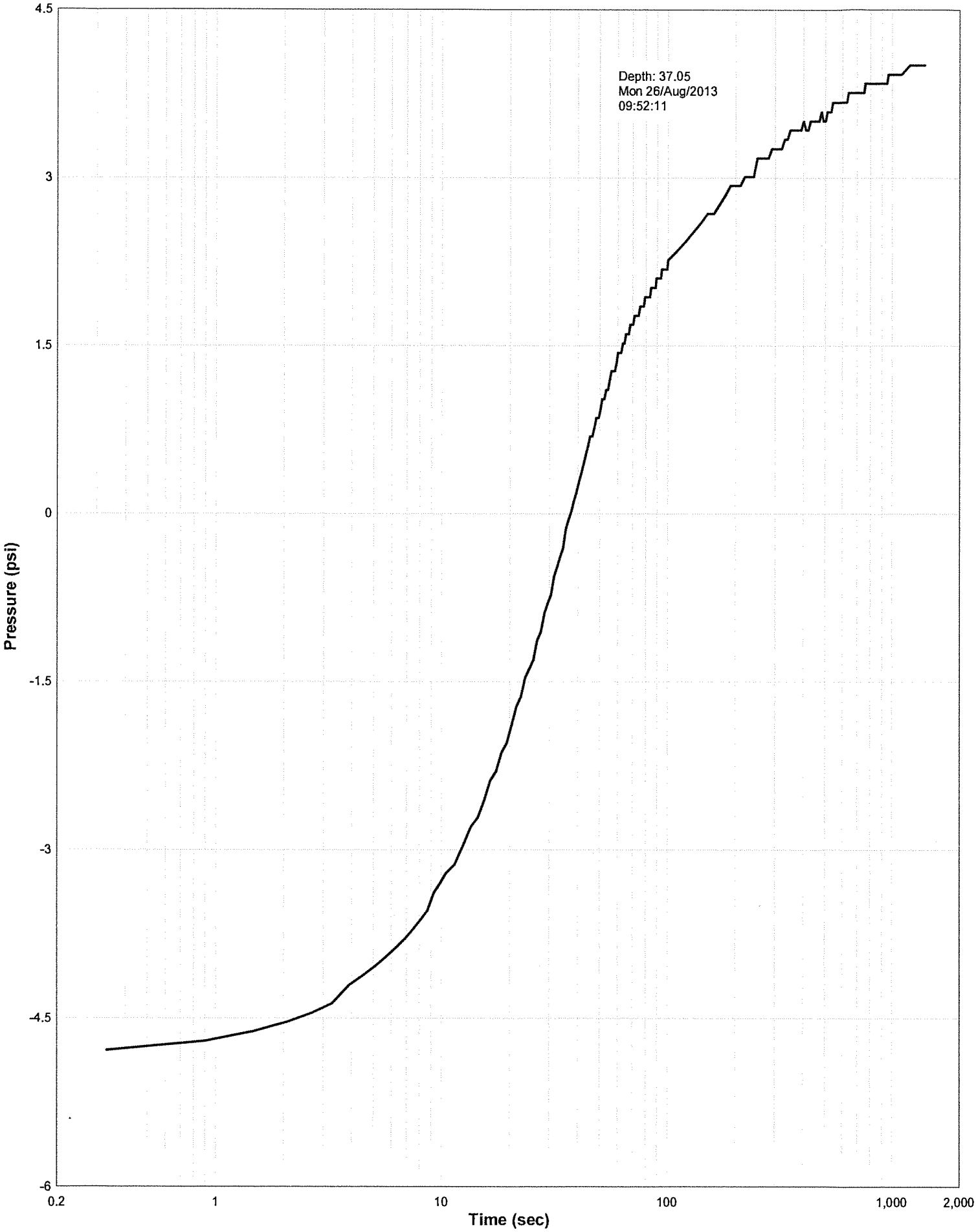
Petra Geotechnical, Inc. CPT-1

Kehoe Testing



Petra Geotechnical, Inc. CPT-2

Kehoe Testing



Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \le 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Peak drained friction angle, φ (°) ::

$$\varphi = 17.60 + 11 \cdot \log(Q_{tn})$$

(applicable only to SBT_n: 5, 6, 7 and 8)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$

$\alpha = 14$ for $Q_{tn} > 14$

$\alpha = Q_{tn}$ for $Q_{tn} \le 14$

$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \le 2.20$

$M_{CPT} = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$

:: Small strain shear Modulus, Go (MPa) ::

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

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot +7 \cdot \log(F_r))}\right]^{-1.25}$$ or user defined

$OCR = k_{OCR} \cdot Q_{tn}$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = 0.1 \cdot \left(\frac{q_t - \sigma_v}{\sigma'_{vo}}\right)$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: Effective Stress Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 4th Edition, July 2010
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)

APPENDIX B

LABORATORY TEST PROCEDURES

LABORATORY TEST DATA

CONSOLIDATION TEST RESULTS

PETRA

LABORATORY TEST PROCEDURES

Soil Classification

Soils encountered within the exploratory borings were classified and described utilizing the visual-manual procedures of the Unified Soil Classification System, and in general accordance with Test Method ASTM D 2488. The assigned group symbols are presented on the exploration logs, Appendix A.

In Situ Moisture and Density

Moisture content and dry density of the in place soils were determined in representative strata in accordance with test method ASTM D 2216. Test data are presented in the exploration logs, Appendix A.

Laboratory Maximum Dry Density/Optimum Moisture

The maximum dry density and optimum moisture content of the near-surface soil materials were determined for a selected sample in accordance with Method A of ASTM D 1557. The results of this test are presented on Plate B-1.

Expansion Potential

A preliminary expansion index test was performed on a selected sample in accordance with Test Method ASTM 4829. The results of this test are presented on Plate B-1.

Soluble Sulfates and Chlorides

Chemical analyses were performed on a selected sample of near-surface soils to determine preliminary soluble sulfate and chloride contents in accordance with California Test Method Nos. 417 and 422, respectively. Test results are presented on Plate B-1.

pH and Minimum Resistivity

pH and minimum resistivity tests were performed on a selected sample of near-surface site soils to provide a preliminary evaluation of their corrosive potential to concrete and metallic construction materials. These tests were performed in accordance with California Test Method Nos. 532 and 643, respectively. The results of these tests are included in Plate B-1.

Consolidation

Settlement predictions under anticipated loads were made on the basis of one-dimensional consolidation tests. These tests were performed in general accordance with Test Method ASTM D 2435. Axial loads were applied in several increments to a laterally restrained 1-inch-high sample. Loads were applied in a geometric progression by doubling the previous load, and the resulting deformations were recorded at selected time intervals. The test samples were inundated at the approximate in-situ overburden pressure in order to evaluate the effect of a sudden increase in moisture content (hydroconsolidation potential). Results of these tests are graphically presented on Plates B-2 through B-4.

PLATE B-1 - ADDITIONAL LABORATORY DATA SUMMARY*														
Boring Number	Sample Depth (ft)	Soil Description	Max. Dry Density[1] (pcf)	Optimum Moisture[1] (%)	Expansion Index[2]	CBC Soil Classification[3]	Atterberg Limits[4]			Sulfate Content[5] (%)	Chloride Content[6] (ppm)	pH[7]	Minimum Resistivity[7] (Ohm-cm)	Percent Passing No. 200 Sieve[8]
							LL	PL	PI					
B-1	3 - 6	Sand (SP)	102	8.5	-	-	-	-	-	-	-	-	-	-
B-2	0 - 5	Clayey Silt with Sand (ML/SP)	112	13.5	-	-	-	-	-	0.03	83	7.2	2,500	-
B-4	0 - 5	Silty Sand/Sand (SM/SP)	114	14.5	Less than 20	-	-	-	-	-	-	-	-	-

*Note: Test Procedures:

[1] Per ASTM Test Method D 1557
[2] Per ASTM Test Method D 4829
[3] Per 2013 California Building Code Section 1803.5.3
[4] Per ASTM Test Method D 4318
[5] Per Caltrans Test Method 417
[6] Per Caltrans Test Method 422
[7] Per Caltrans Test Method 643
[8] Per ASTM Test Method D 1140

Laboratory data pertaining to in-place soil moisture content and dry density are provided on the exploration logs included in Appendix A of this report.

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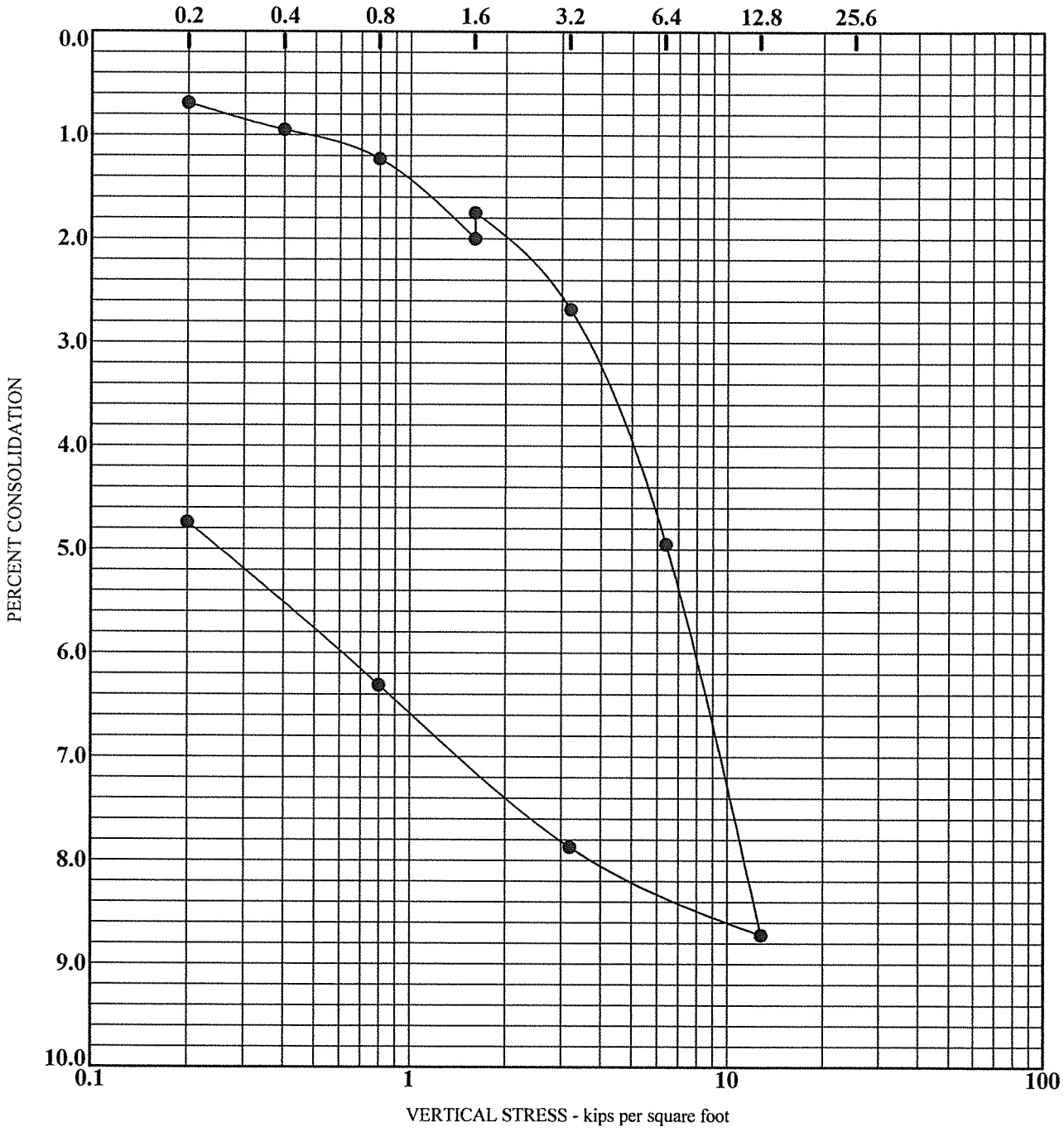
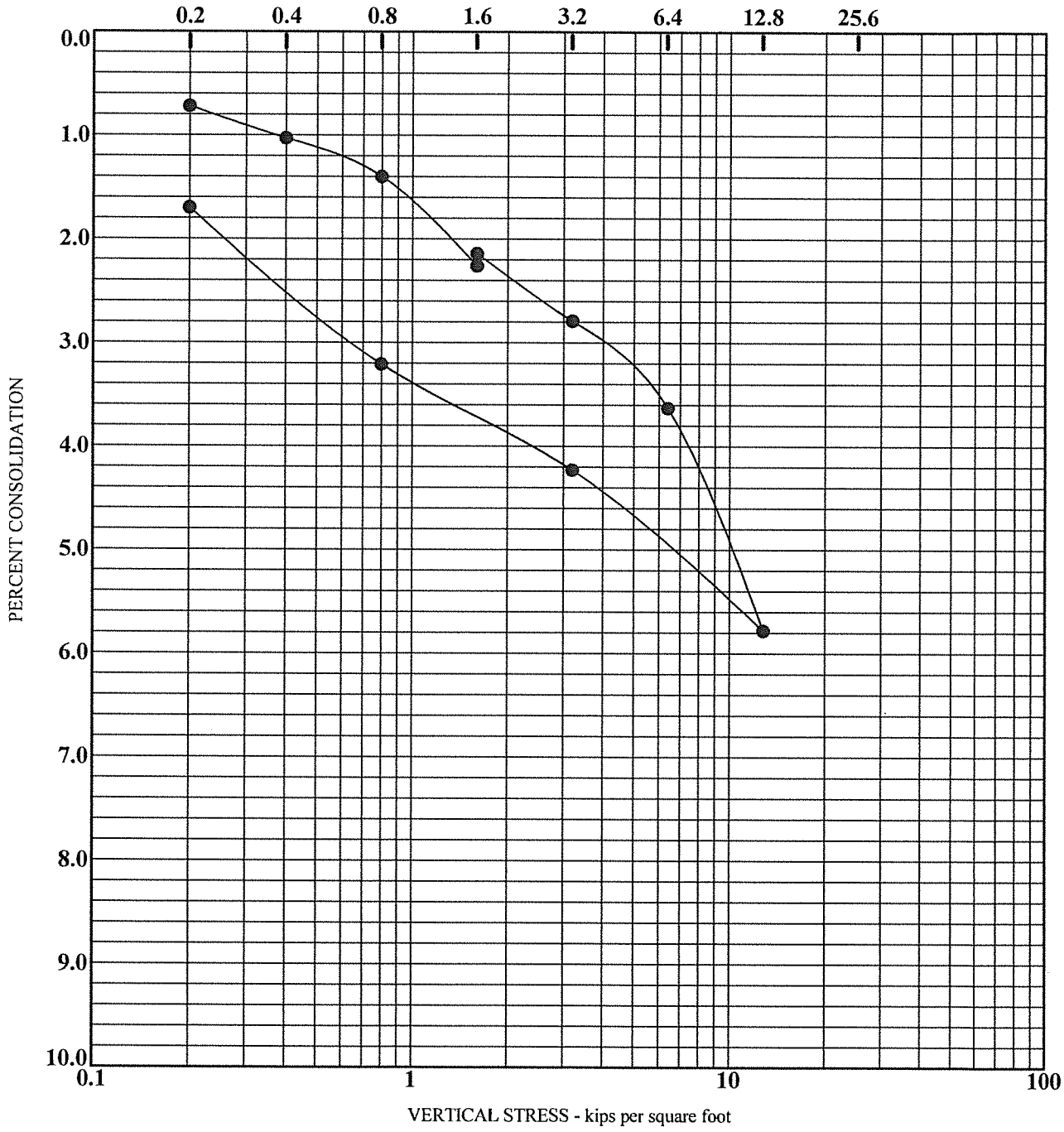
SAMPLE LOCATION	MATERIAL DESCRIPTION	INITIAL			INUNDATED
		DENSITY (pcf)	MOISTURE (%)	SATURATION (%)	LOAD (ksf)
● B-3 @ 6.0	Sandy Silt (ML)				



J.N. 13-443	CONSOLIDATION TEST RESULTS	March, 2014
PETRA GEOTECHNICAL, INC.		PLATE B-2

SAMPLE LOCATION	MATERIAL DESCRIPTION	INITIAL			INUNDATED
		DENSITY (pcf)	MOISTURE (%)	SATURATION (%)	LOAD (ksf)
● B-4 @ 5.0	Sandy Silt (ML)				



J.N. 13-443	CONSOLIDATION TEST RESULTS	March, 2014
PETRA GEOTECHNICAL, INC.		PLATE B-3

CONSOLIDATION - STRAIN 13-443.GPJ PETRA.GDT 3/25/14

APPENDIX C

GROUND MOTION ANALYSIS

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USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.84194°N, 118.20085°W)

Site Class D - "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_s) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1[1] $S_S = 1.654$ g

From Figure 22-2[2] $S_1 = 0.613$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.654 g, F_a = 1.000

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.613 g, F_v = 1.500

Equation (11.4-1): $S_{MS} = F_a S_S = 1.000 \times 1.654 = 1.654$ g

Equation (11.4-2): $S_{M1} = F_v S_1 = 1.500 \times 0.613 = 0.920$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

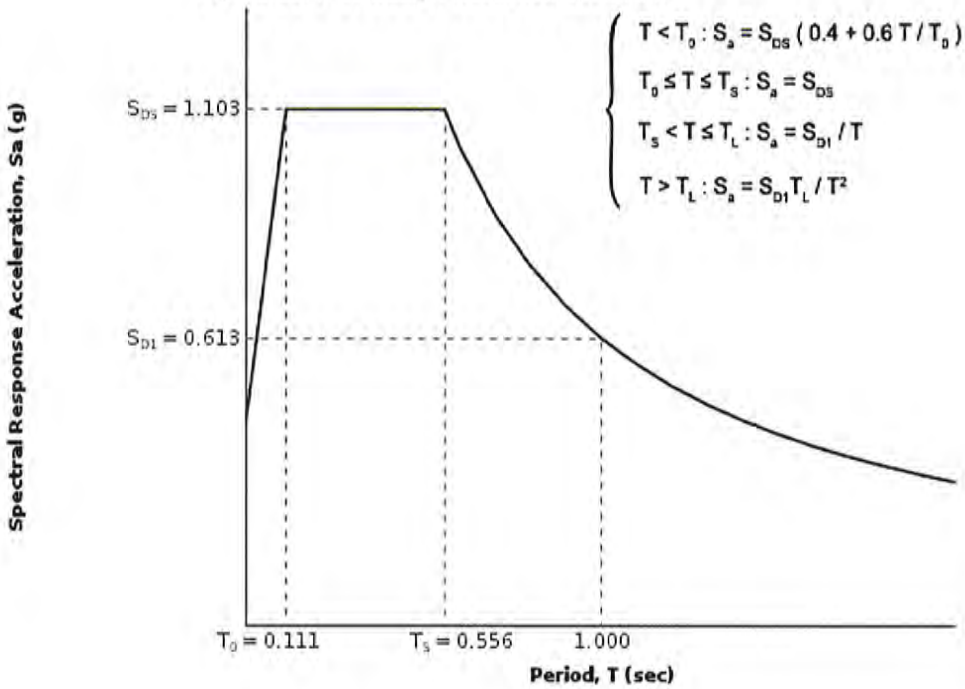
Equation (11.4-3): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.654 = 1.103$ g

Equation (11.4-4): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.920 = 0.613$ g

Section 11.4.5 — Design Response Spectrum

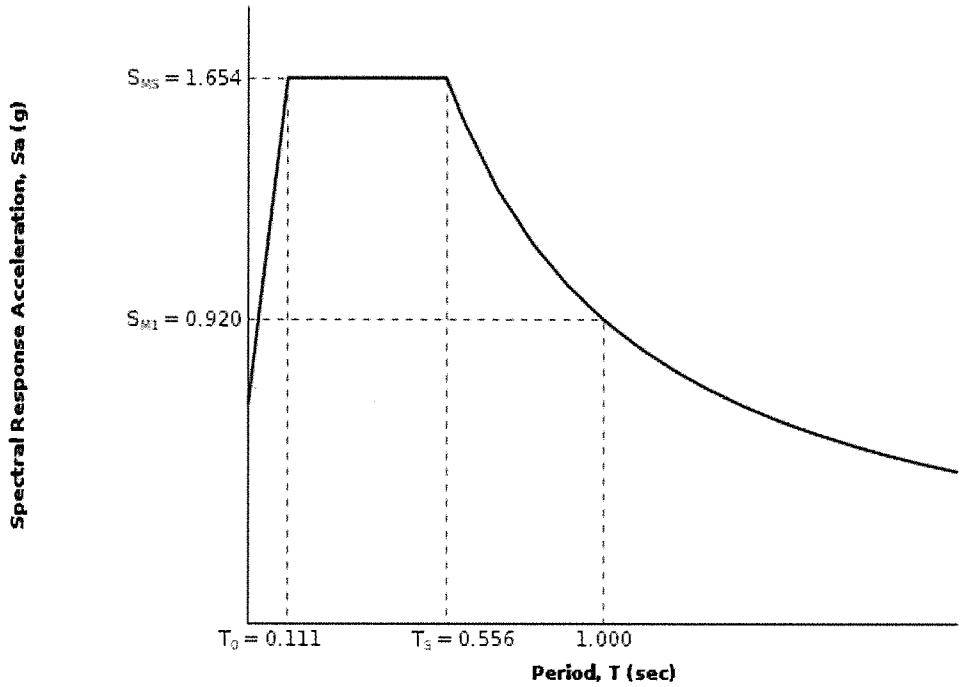
From Figure 22-12[3] $T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7[4] PGA = 0.627

Equation (11.8-1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.627 = 0.627$ g

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.627 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 - Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17[5] C_{RS} = 0.966

From Figure 22-18[6] C_{R1} = 0.984

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.103 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.613 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

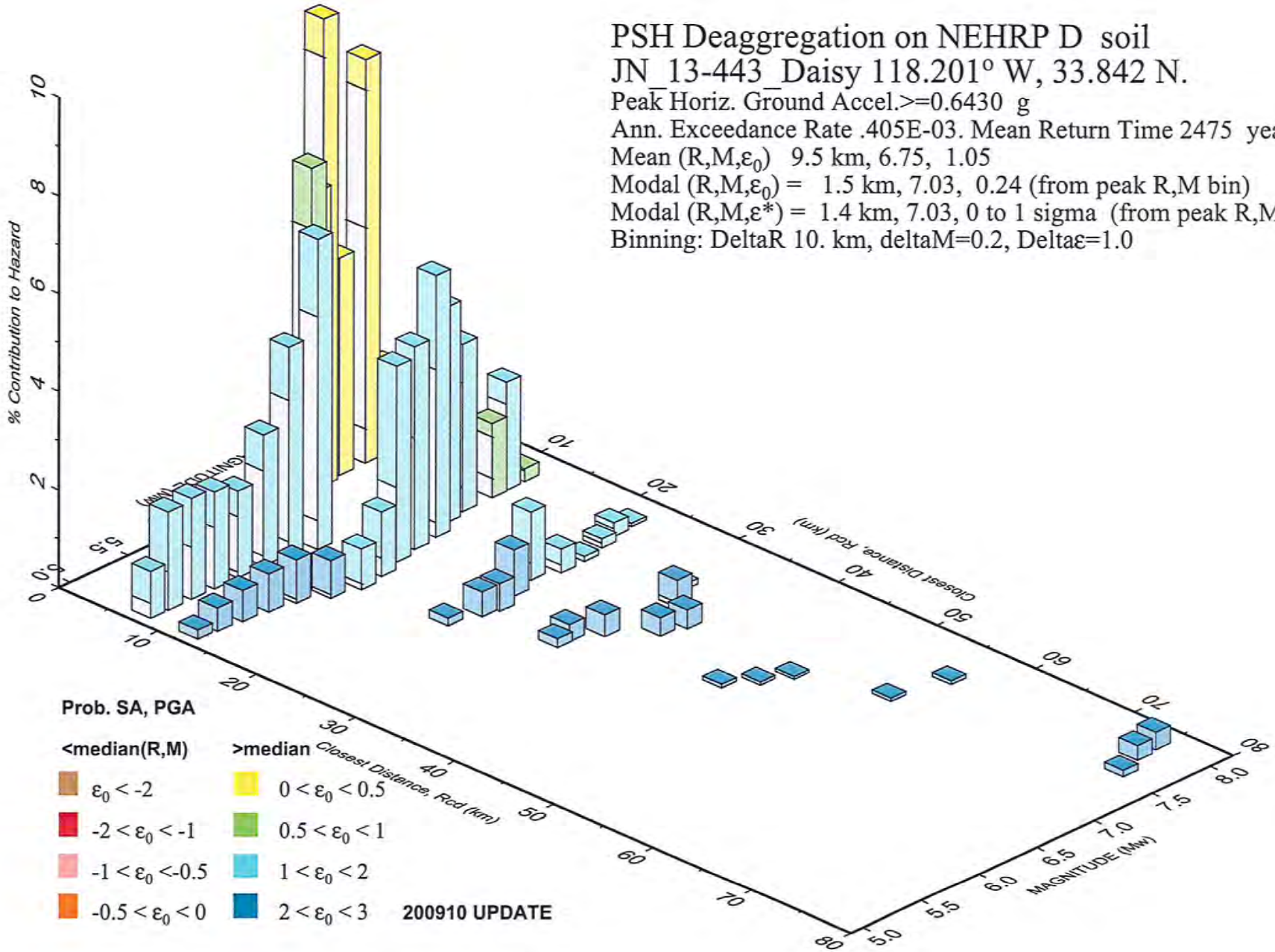
Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

PSH Deaggregation on NEHRP D soil
JN_13-443_Daisy 118.201° W, 33.842 N.

Peak Horiz. Ground Accel.>=0.6430 g
Ann. Exceedance Rate .405E-03. Mean Return Time 2475 years
Mean (R,M,ε_0) 9.5 km, 6.75, 1.05
Modal (R,M,ε_0) = 1.5 km, 7.03, 0.24 (from peak R,M bin)
Modal (R,M,ε*) = 1.4 km, 7.03, 0 to 1 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0



GMT 2014 Mar 18 20:58:41 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 287. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

*** Deaggregation of Seismic Hazard at One Period of Spectral Accel. ***
*** Data from U.S.G.S. National Seismic Hazards Mapping Project, 2008 version ***
PSHA Deaggregation. %contributions. site: JN_13-443_Daisy long: 118.201 W., lat: 33.842 N.
Vs30(m/s)= 287.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.6430 g. Weight * Computed_Rate_Ex 0.405E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00173
#This deaggregation corresponds to Mean Hazard w/all GMPEs

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
7.3	5.05	0.955	0.656	0.300	0.000	0.000	0.000	0.000
12.1	5.05	0.188	0.188	0.000	0.000	0.000	0.000	0.000
7.3	5.20	2.012	1.231	0.781	0.000	0.000	0.000	0.000
12.3	5.20	0.481	0.481	0.000	0.000	0.000	0.000	0.000
7.3	5.40	2.072	1.003	1.069	0.000	0.000	0.000	0.000
12.5	5.40	0.638	0.638	0.000	0.000	0.000	0.000	0.000
7.3	5.60	1.982	0.736	1.206	0.040	0.000	0.000	0.000
12.8	5.60	0.783	0.769	0.013	0.000	0.000	0.000	0.000
7.3	5.80	1.775	0.526	1.132	0.117	0.000	0.000	0.000
13.2	5.80	0.884	0.801	0.083	0.000	0.000	0.000	0.000
7.3	6.02	2.674	0.732	1.735	0.206	0.000	0.000	0.000
14.5	5.99	0.774	0.698	0.077	0.000	0.000	0.000	0.000
7.6	6.21	4.276	1.089	2.880	0.307	0.000	0.000	0.000
15.1	6.20	0.831	0.713	0.118	0.000	0.000	0.000	0.000
23.7	6.22	0.160	0.160	0.000	0.000	0.000	0.000	0.000
8.4	6.40	6.336	1.589	4.159	0.588	0.000	0.000	0.000
14.7	6.41	1.323	0.945	0.378	0.000	0.000	0.000	0.000
24.4	6.45	0.505	0.505	0.000	0.000	0.000	0.000	0.000
31.8	6.47	0.178	0.178	0.000	0.000	0.000	0.000	0.000
5.5	6.59	7.312	1.241	3.931	2.120	0.019	0.000	0.000
13.9	6.61	3.999	2.461	1.537	0.000	0.000	0.000	0.000
24.8	6.57	0.564	0.563	0.000	0.000	0.000	0.000	0.000
31.7	6.57	0.351	0.351	0.000	0.000	0.000	0.000	0.000
3.5	6.80	6.427	0.693	2.892	2.769	0.073	0.000	0.000
13.4	6.80	4.138	1.867	2.271	0.000	0.000	0.000	0.000
23.8	6.77	0.944	0.868	0.077	0.000	0.000	0.000	0.000
33.0	6.77	0.440	0.440	0.000	0.000	0.000	0.000	0.000
44.9	6.77	0.081	0.081	0.000	0.000	0.000	0.000	0.000
1.5	7.03	9.497	0.775	3.480	4.663	0.579	0.000	0.000
13.1	7.01	5.337	1.924	3.114	0.299	0.000	0.000	0.000
22.9	7.00	1.389	1.047	0.342	0.000	0.000	0.000	0.000
35.9	7.00	0.370	0.368	0.002	0.000	0.000	0.000	0.000
46.6	6.96	0.063	0.063	0.000	0.000	0.000	0.000	0.000
1.2	7.20	4.432	0.343	1.557	2.172	0.360	0.000	0.000
12.1	7.19	4.396	1.187	2.797	0.412	0.000	0.000	0.000
23.6	7.19	0.489	0.331	0.159	0.000	0.000	0.000	0.000
36.5	7.19	0.381	0.355	0.025	0.000	0.000	0.000	0.000
47.7	7.15	0.051	0.051	0.000	0.000	0.000	0.000	0.000
1.2	7.42	8.210	0.615	2.794	4.108	0.693	0.000	0.000
11.6	7.37	3.412	0.799	2.108	0.505	0.000	0.000	0.000
23.6	7.40	0.106	0.059	0.047	0.000	0.000	0.000	0.000
33.6	7.35	0.557	0.457	0.100	0.000	0.000	0.000	0.000
55.2	7.35	0.059	0.059	0.000	0.000	0.000	0.000	0.000
1.1	7.61	1.768	0.137	0.645	0.842	0.144	0.000	0.000
11.6	7.62	1.503	0.268	0.846	0.389	0.000	0.000	0.000
23.0	7.60	0.186	0.085	0.100	0.000	0.000	0.000	0.000
32.0	7.54	0.110	0.085	0.025	0.000	0.000	0.000	0.000
75.7	7.60	0.130	0.130	0.000	0.000	0.000	0.000	0.000
1.1	7.71	0.577	0.042	0.195	0.290	0.050	0.000	0.000
11.5	7.75	2.192	0.449	1.413	0.329	0.000	0.000	0.000
22.3	7.76	0.276	0.116	0.159	0.000	0.000	0.000	0.000
56.4	7.76	0.077	0.076	0.000	0.000	0.000	0.000	0.000
74.6	7.82	0.297	0.297	0.000	0.000	0.000	0.000	0.000
11.5	7.91	0.265	0.046	0.172	0.046	0.000	0.000	0.000
22.1	7.93	0.061	0.021	0.038	0.002	0.000	0.000	0.000

74.4	7.99	0.349	0.349	0.000	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 100.0
Mean src-site R= 9.5 km; M= 6.75; eps0= 1.05. Mean calculated for all sources.
Modal src-site R= 1.5 km; M= 7.03; eps0= 0.24 from peak (R,M) bin
MODE R*= 1.4km; M*= 7.03; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 4.663

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	40.58	8.8	7.17	0.85
California B-faults GR	28.27	7.4	6.83	0.90
California A-faults	3.72	36.4	7.33	2.00
CA Compr. crustal gridded	27.43	9.1	5.95	1.38

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Palos Verdes Char	5.85	11.5	7.26	1.19	-147.5
Newport-Inglewood, alt 1 Char	4.62	1.6	7.14	0.21	-137.3
Newport-Inglewood, alt 2 Char	5.05	0.7	7.14	0.13	-119.8
Puente Hills (Santa Fe Springs)	4.32	9.7	6.51	1.35	38.6
Palos Verdes Connected Char	3.90	11.5	7.71	0.98	-147.6
Newport Inglewood Connected alt	3.97	1.6	7.50	0.17	-137.3
Newport Inglewood Connected alt	4.37	0.7	7.50	0.08	-119.8
Palos Verdes GR	4.54	12.3	6.93	1.44	-148.9
Newport-Inglewood, alt 1 GR	5.45	2.5	6.83	0.39	-140.9
Newport-Inglewood, alt 2 GR	5.87	1.9	6.83	0.32	-122.8
Newport Inglewood Connected alt2	2.08	2.4	6.99	0.33	-129.6

#*********End of deaggregation corresponding to Mean Hazard w/all GMPEs *********#

PSHA Deaggregation. %contributions. site: JN_13-443_Daisy long: 118.201 W., lat: 33.842 N.
Vs30(m/s)= 287.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.6430 g. Weight * Computed_Rate_Ex 0.161E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00240
#This deaggregation corresponds to Boore-Atkinson 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
7.1	5.05	0.078	0.078	0.000	0.000	0.000	0.000	0.000
7.2	5.20	0.202	0.202	0.000	0.000	0.000	0.000	0.000
7.2	5.40	0.253	0.253	0.001	0.000	0.000	0.000	0.000
12.0	5.42	0.026	0.026	0.000	0.000	0.000	0.000	0.000
7.2	5.60	0.291	0.264	0.027	0.000	0.000	0.000	0.000
12.6	5.61	0.067	0.067	0.000	0.000	0.000	0.000	0.000
7.3	5.80	0.312	0.257	0.056	0.000	0.000	0.000	0.000
13.4	5.81	0.123	0.123	0.000	0.000	0.000	0.000	0.000
7.1	6.02	0.584	0.431	0.153	0.000	0.000	0.000	0.000
14.9	5.99	0.156	0.156	0.000	0.000	0.000	0.000	0.000
7.3	6.21	0.956	0.648	0.309	0.000	0.000	0.000	0.000
15.2	6.20	0.207	0.207	0.000	0.000	0.000	0.000	0.000
24.1	6.22	0.077	0.077	0.000	0.000	0.000	0.000	0.000
8.2	6.40	1.468	0.826	0.642	0.000	0.000	0.000	0.000
14.9	6.41	0.365	0.355	0.011	0.000	0.000	0.000	0.000
24.6	6.45	0.274	0.274	0.000	0.000	0.000	0.000	0.000
31.7	6.47	0.175	0.175	0.000	0.000	0.000	0.000	0.000
4.7	6.60	2.723	0.517	1.333	0.871	0.002	0.000	0.000
13.7	6.60	1.369	0.923	0.446	0.000	0.000	0.000	0.000
24.8	6.57	0.356	0.356	0.000	0.000	0.000	0.000	0.000
31.8	6.57	0.348	0.348	0.000	0.000	0.000	0.000	0.000
43.3	6.59	0.025	0.025	0.000	0.000	0.000	0.000	0.000
3.0	6.80	2.739	0.300	1.160	1.258	0.022	0.000	0.000
13.3	6.79	1.677	0.848	0.829	0.000	0.000	0.000	0.000
23.6	6.77	0.711	0.636	0.075	0.000	0.000	0.000	0.000
33.0	6.77	0.432	0.432	0.000	0.000	0.000	0.000	0.000
44.8	6.77	0.078	0.078	0.000	0.000	0.000	0.000	0.000
1.4	7.03	4.315	0.297	1.578	2.215	0.224	0.000	0.000

13.0	7.01	2.401	0.855	1.471	0.075	0.000	0.000	0.000
22.8	6.99	1.110	0.796	0.313	0.000	0.000	0.000	0.000
36.0	7.00	0.357	0.355	0.002	0.000	0.000	0.000	0.000
46.4	6.96	0.056	0.056	0.000	0.000	0.000	0.000	0.000
56.2	6.99	0.033	0.033	0.000	0.000	0.000	0.000	0.000
1.2	7.20	2.017	0.123	0.717	1.035	0.143	0.000	0.000
11.8	7.19	2.094	0.547	1.359	0.189	0.000	0.000	0.000
23.5	7.19	0.382	0.248	0.133	0.000	0.000	0.000	0.000
36.6	7.19	0.326	0.301	0.025	0.000	0.000	0.000	0.000
47.6	7.15	0.049	0.049	0.000	0.000	0.000	0.000	0.000
55.5	7.19	0.041	0.041	0.000	0.000	0.000	0.000	0.000
1.2	7.42	3.714	0.224	1.312	1.914	0.265	0.000	0.000
11.6	7.36	1.781	0.409	1.143	0.230	0.000	0.000	0.000
23.4	7.40	0.078	0.041	0.038	0.000	0.000	0.000	0.000
33.9	7.35	0.436	0.340	0.096	0.000	0.000	0.000	0.000
55.2	7.35	0.059	0.059	0.000	0.000	0.000	0.000	0.000
74.4	7.37	0.046	0.046	0.000	0.000	0.000	0.000	0.000
1.1	7.61	0.746	0.044	0.259	0.388	0.055	0.000	0.000
11.7	7.59	0.491	0.099	0.315	0.078	0.000	0.000	0.000
23.1	7.61	0.147	0.059	0.088	0.000	0.000	0.000	0.000
32.3	7.54	0.078	0.057	0.021	0.000	0.000	0.000	0.000
56.1	7.57	0.035	0.035	0.000	0.000	0.000	0.000	0.000
75.7	7.60	0.130	0.130	0.000	0.000	0.000	0.000	0.000
1.1	7.71	0.256	0.015	0.089	0.134	0.019	0.000	0.000
11.5	7.74	1.331	0.235	0.850	0.246	0.000	0.000	0.000
22.4	7.76	0.176	0.059	0.117	0.000	0.000	0.000	0.000
56.4	7.76	0.072	0.071	0.000	0.000	0.000	0.000	0.000
74.6	7.82	0.297	0.297	0.000	0.000	0.000	0.000	0.000
11.5	7.91	0.108	0.021	0.078	0.009	0.000	0.000	0.000
22.1	7.93	0.038	0.011	0.025	0.002	0.000	0.000	0.000
74.4	7.99	0.329	0.329	0.000	0.000	0.000	0.000	0.000
74.2	8.20	0.039	0.039	0.000	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 39.8
Mean src-site R= 11.7 km; M= 6.96; eps0= 1.09. Mean calculated for all sources.
Modal src-site R= 1.4 km; M= 7.03; eps0= 0.22 from peak (R,M) bin
MODE R*= 1.3km; M*= 7.03; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 2.215

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	18.40	10.3	7.21	0.91
California B-faults GR	12.86	7.8	6.86	0.91
California A-faults	3.14	38.6	7.34	1.99
CA Compr. crustal gridded	5.38	10.0	6.14	1.61

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Palos Verdes Char	3.04	11.5	7.25	1.17	-147.5
Newport-Inglewood, alt 1 Char	2.15	1.6	7.14	0.19	-137.3
Newport-Inglewood, alt 2 Char	2.28	0.7	7.14	0.14	-119.8
Puente Hills (Santa Fe Springs)	1.00	9.7	6.53	1.56	38.6
Palos Verdes Connected Char	1.90	11.5	7.71	1.00	-147.6
Newport Inglewood Connected alt	1.76	1.6	7.50	0.20	-137.3
Newport Inglewood Connected alt	1.95	0.7	7.50	0.11	-119.8
Palos Verdes GR	2.48	12.5	6.93	1.44	-148.9
Newport-Inglewood, alt 1 GR	2.61	2.5	6.82	0.37	-140.9
Newport-Inglewood, alt 2 GR	2.76	2.0	6.82	0.31	-122.8
Newport Inglewood Connected alt2	0.98	2.7	6.98	0.36	-129.6

#*********End of deaggregation corresponding to Boore-Atkinson 2008 *********#

PSHA Deaggregation. %contributions. site: JN_13-443_Daisy long: 118.201 W., lat: 33.842 N.
Vs30(m/s)= 287.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.6430 g. Weight * Computed_Rate_Ex 0.463E-04

#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00000
#This deaggregation corresponds to Campbell-Bozorgnia 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
7.2	5.05	0.145	0.145	0.000	0.000	0.000	0.000	0.000
7.3	5.20	0.374	0.364	0.010	0.000	0.000	0.000	0.000
12.0	5.21	0.028	0.028	0.000	0.000	0.000	0.000	0.000
7.3	5.40	0.481	0.401	0.081	0.000	0.000	0.000	0.000
12.1	5.41	0.086	0.086	0.000	0.000	0.000	0.000	0.000
7.3	5.60	0.492	0.357	0.135	0.000	0.000	0.000	0.000
12.3	5.60	0.135	0.135	0.000	0.000	0.000	0.000	0.000
7.3	5.80	0.419	0.287	0.131	0.000	0.000	0.000	0.000
12.7	5.80	0.148	0.148	0.000	0.000	0.000	0.000	0.000
7.3	6.02	0.560	0.434	0.125	0.000	0.000	0.000	0.000
14.0	5.99	0.121	0.121	0.000	0.000	0.000	0.000	0.000
7.7	6.21	0.975	0.642	0.333	0.000	0.000	0.000	0.000
15.0	6.20	0.134	0.134	0.000	0.000	0.000	0.000	0.000
23.2	6.23	0.012	0.012	0.000	0.000	0.000	0.000	0.000
8.5	6.40	1.692	0.815	0.877	0.000	0.000	0.000	0.000
14.6	6.41	0.248	0.238	0.010	0.000	0.000	0.000	0.000
24.4	6.46	0.066	0.066	0.000	0.000	0.000	0.000	0.000
8.0	6.59	1.078	0.403	0.675	0.000	0.000	0.000	0.000
14.1	6.60	0.607	0.478	0.129	0.000	0.000	0.000	0.000
24.8	6.56	0.077	0.077	0.000	0.000	0.000	0.000	0.000
6.1	6.79	0.523	0.249	0.274	0.000	0.000	0.000	0.000
13.8	6.79	0.548	0.387	0.161	0.000	0.000	0.000	0.000
25.1	6.75	0.055	0.055	0.000	0.000	0.000	0.000	0.000
2.1	7.03	0.541	0.238	0.292	0.012	0.000	0.000	0.000
13.9	6.97	0.590	0.348	0.243	0.000	0.000	0.000	0.000
26.8	6.97	0.017	0.017	0.000	0.000	0.000	0.000	0.000
36.6	7.07	0.007	0.007	0.000	0.000	0.000	0.000	0.000
1.2	7.20	0.245	0.098	0.139	0.008	0.000	0.000	0.000
13.9	7.14	0.276	0.138	0.138	0.000	0.000	0.000	0.000
29.1	7.14	0.015	0.015	0.000	0.000	0.000	0.000	0.000
34.8	7.20	0.008	0.008	0.000	0.000	0.000	0.000	0.000
1.1	7.42	0.450	0.180	0.257	0.013	0.000	0.000	0.000
12.1	7.35	0.061	0.054	0.007	0.000	0.000	0.000	0.000
31.7	7.35	0.024	0.024	0.000	0.000	0.000	0.000	0.000
1.1	7.61	0.090	0.036	0.052	0.003	0.000	0.000	0.000
11.5	7.58	0.011	0.011	0.000	0.000	0.000	0.000	0.000
30.7	7.53	0.006	0.006	0.000	0.000	0.000	0.000	0.000
1.1	7.71	0.031	0.012	0.018	0.001	0.000	0.000	0.000
11.5	7.75	0.042	0.042	0.000	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 11.4
Mean src-site R= 9.0 km; M= 6.41; eps0= 1.55. Mean calculated for all sources.
Modal src-site R= 8.5 km; M= 6.40; eps0= 1.31 from peak (R,M) bin
MODE R*= 8.8km; M*= 6.40; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 0.877

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	3.74	9.8	6.89	1.54
CA Compr. crustal gridded	5.64	8.3	5.99	1.52

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Palos Verdes Char	0.09	11.5	7.25	2.15	-147.5
Newport-Inglewood, alt 1 Char	0.26	1.6	7.15	1.33	-137.3
Newport-Inglewood, alt 2 Char	0.27	0.7	7.15	1.32	-119.8
Puente Hills (Santa Fe Springs)	1.28	9.7	6.50	1.42	38.6
Palos Verdes Connected Char	0.06	11.5	7.71	2.11	-147.6
Newport Inglewood Connected alt	0.22	1.6	7.50	1.31	-137.3
Newport Inglewood Connected alt	0.23	0.7	7.50	1.29	-119.8
Palos Verdes GR	0.00	0.0	0.00	0.00	-148.9
Newport-Inglewood, alt 1 GR	0.18	1.9	6.89	1.64	-140.9

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Newport-Inglewood, alt 2 GR          0.19     1.2   6.89    1.62    -122.8
Newport Inglewood Connected alt2     0.07     1.1   7.07    1.51    -129.6
#*********End of deaggregation corresponding to Campbell-Bozorgnia 2008  *********#

PSHA Deaggregation. %contributions. site: JN_13-443_Daisy long: 118.201 W., lat: 33.842 N.
 Vs30(m/s)= 287.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08  See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475  yrs. Exceedance PGA =0.6430   g. Weight * Computed_Rate_Ex 0.197E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00279
#This deaggregation corresponds to Chiou-Youngs 2008
DIST(KM) MAG(MW) ALL_EPS EPSILON>2  1<EPS<2 0<EPS<1 -1<EPS<0 -2<EPS<-1 EPS<-2
    7.3     5.05    0.732    0.596    0.135    0.000    0.000    0.000    0.000
   12.1     5.05    0.184    0.184    0.000    0.000    0.000    0.000    0.000
    7.3     5.20    1.436    1.096    0.340    0.000    0.000    0.000    0.000
   12.3     5.20    0.447    0.447    0.000    0.000    0.000    0.000    0.000
    7.3     5.40    1.337    0.913    0.424    0.000    0.000    0.000    0.000
   12.6     5.40    0.526    0.526    0.000    0.000    0.000    0.000    0.000
    7.3     5.60    1.199    0.710    0.488    0.000    0.000    0.000    0.000
   13.0     5.60    0.581    0.581    0.000    0.000    0.000    0.000    0.000
    7.4     5.80    1.045    0.515    0.530    0.000    0.000    0.000    0.000
   13.3     5.80    0.613    0.611    0.002    0.000    0.000    0.000    0.000
    7.4     6.02    1.530    0.713    0.817    0.000    0.000    0.000    0.000
   14.5     5.99    0.498    0.489    0.008    0.000    0.000    0.000    0.000
    7.7     6.21    2.320    0.901    1.419    0.000    0.000    0.000    0.000
   15.1     6.20    0.490    0.478    0.012    0.000    0.000    0.000    0.000
   23.3     6.21    0.069    0.069    0.000    0.000    0.000    0.000    0.000
    8.3     6.40    3.199    0.931    2.229    0.039    0.000    0.000    0.000
   14.6     6.41    0.710    0.625    0.085    0.000    0.000    0.000    0.000
   24.1     6.44    0.166    0.166    0.000    0.000    0.000    0.000    0.000
    5.3     6.59    3.486    0.506    1.935    1.029    0.016    0.000    0.000
   13.9     6.61    1.913    1.073    0.840    0.000    0.000    0.000    0.000
   24.7     6.57    0.134    0.134    0.000    0.000    0.000    0.000    0.000
    3.6     6.80    3.196    0.329    1.498    1.319    0.051    0.000    0.000
   13.3     6.81    1.863    0.782    1.081    0.000    0.000    0.000    0.000
   23.9     6.77    0.163    0.163    0.000    0.000    0.000    0.000    0.000
    1.5     7.03    4.634    0.298    1.627    2.355    0.354    0.000    0.000
   13.0     7.01    2.418    0.742    1.452    0.224    0.000    0.000    0.000
   22.8     7.01    0.264    0.237    0.027    0.000    0.000    0.000    0.000
    1.2     7.20    2.171    0.123    0.701    1.129    0.218    0.000    0.000
   12.1     7.19    2.097    0.570    1.305    0.223    0.000    0.000    0.000
   23.8     7.21    0.104    0.077    0.027    0.000    0.000    0.000    0.000
   36.0     7.18    0.039    0.039    0.000    0.000    0.000    0.000    0.000
    1.2     7.42    4.113    0.221    1.282    2.183    0.428    0.000    0.000
   11.6     7.37    1.586    0.345    0.966    0.275    0.000    0.000    0.000
   32.4     7.36    0.100    0.096    0.003    0.000    0.000    0.000    0.000
    1.1     7.61    0.864    0.047    0.277    0.450    0.089    0.000    0.000
   11.6     7.60    0.638    0.128    0.351    0.158    0.000    0.000    0.000
   22.4     7.61    0.053    0.033    0.020    0.000    0.000    0.000    0.000
   31.4     7.53    0.026    0.021    0.005    0.000    0.000    0.000    0.000
    1.1     7.71    0.290    0.015    0.088    0.156    0.031    0.000    0.000
   11.5     7.75    1.182    0.202    0.744    0.236    0.000    0.000    0.000
   22.2     7.76    0.084    0.050    0.035    0.000    0.000    0.000    0.000
   11.5     7.91    0.155    0.024    0.095    0.037    0.000    0.000    0.000

Summary statistics for above PSHA PGA  deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%):   48.8
 Mean src-site R=    7.9 km; M= 6.65; eps0=   0.90. Mean calculated for all sources.
Modal src-site R=    1.5 km; M= 7.03; eps0=   0.12 from peak (R,M) bin
 MODE R*=   1.3km; M*= 7.03; EPS.INTERVAL: 0 to 1 sigma  % CONTRIB.=  2.355

Principal sources (faults, subduction, random seismicity having > 3% contribution)
Source Category:                 % contr.  R(km)    M   epsilon0 (mean values).
California B-faults Char           18.44     7.1   7.20    0.64
California B-faults GR             13.36     6.7   6.83    0.77
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CA Compr. crustal gridded          16.42     9.1   5.87    1.26
Individual fault hazard details if its contribution to mean hazard > 2%:
Fault ID                         % contr.   Rcd(km)  M   epsilon0 Site-to-src azimuth(d)
Palos Verdes Char                  2.72    11.5   7.26    1.17    -147.5
Newport-Inglewood, alt 1 Char      2.21     1.6   7.14    0.10    -137.3
Newport-Inglewood, alt 2 Char      2.50     0.7   7.14   -0.01    -119.8
Puente Hills (Santa Fe Springs)    2.04     9.7   6.51    1.20      38.6
Palos Verdes Connected Char        1.94    11.5   7.71    0.93    -147.6
Newport Inglewood Connected alt    1.99     1.6   7.50    0.03    -137.3
Newport Inglewood Connected alt    2.18     0.7   7.50   -0.07    -119.8
Palos Verdes GR                    2.05    12.0   6.93    1.44    -148.9
Newport-Inglewood, alt 1 GR        2.66     2.6   6.82    0.34    -140.9
Newport-Inglewood, alt 2 GR        2.92     1.9   6.83    0.24    -122.8
Newport Inglewood Connected alt2   1.03     2.2   6.98    0.22    -129.6
#**********End of deaggregation corresponding to Chiou-Youngs 2008        *********#

******************** Southern California ****************************************
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APPENDIX D

LIQUEFACTION ANALYSIS

PETRA

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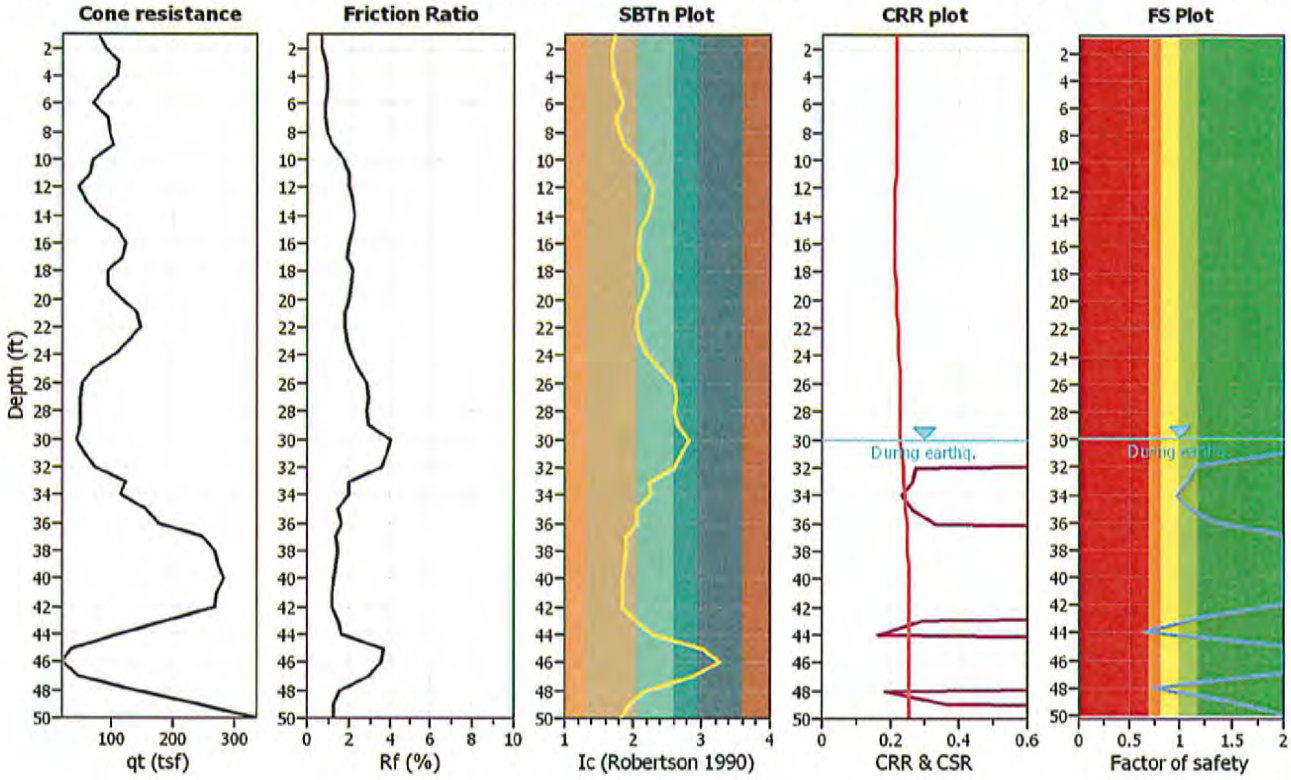
LIQUEFACTION ANALYSIS REPORT

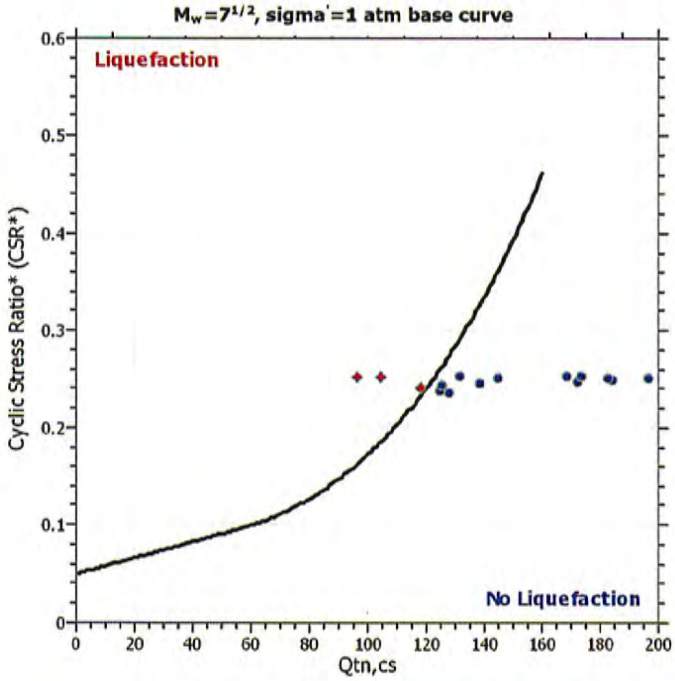
Project title : Integral Communities **Location : 4747 Daisy Avenue, Long Beach, CA**

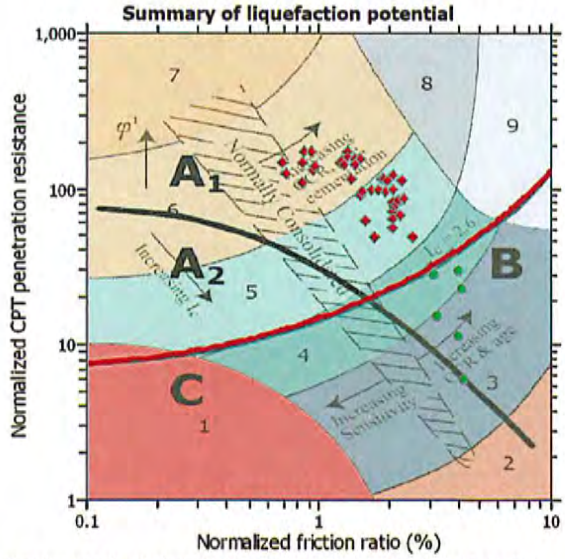
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	44.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	30.00 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:	6.75	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

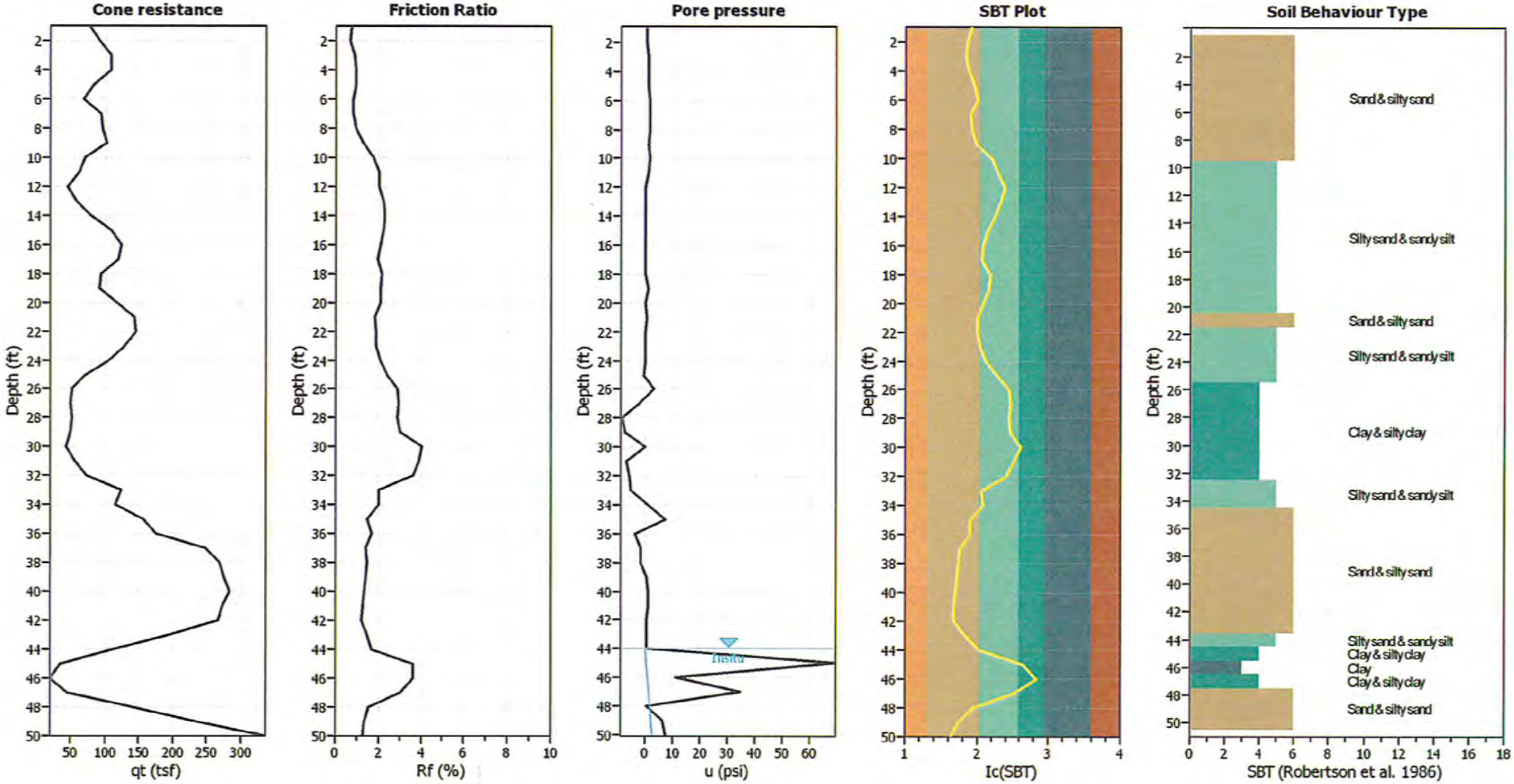






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



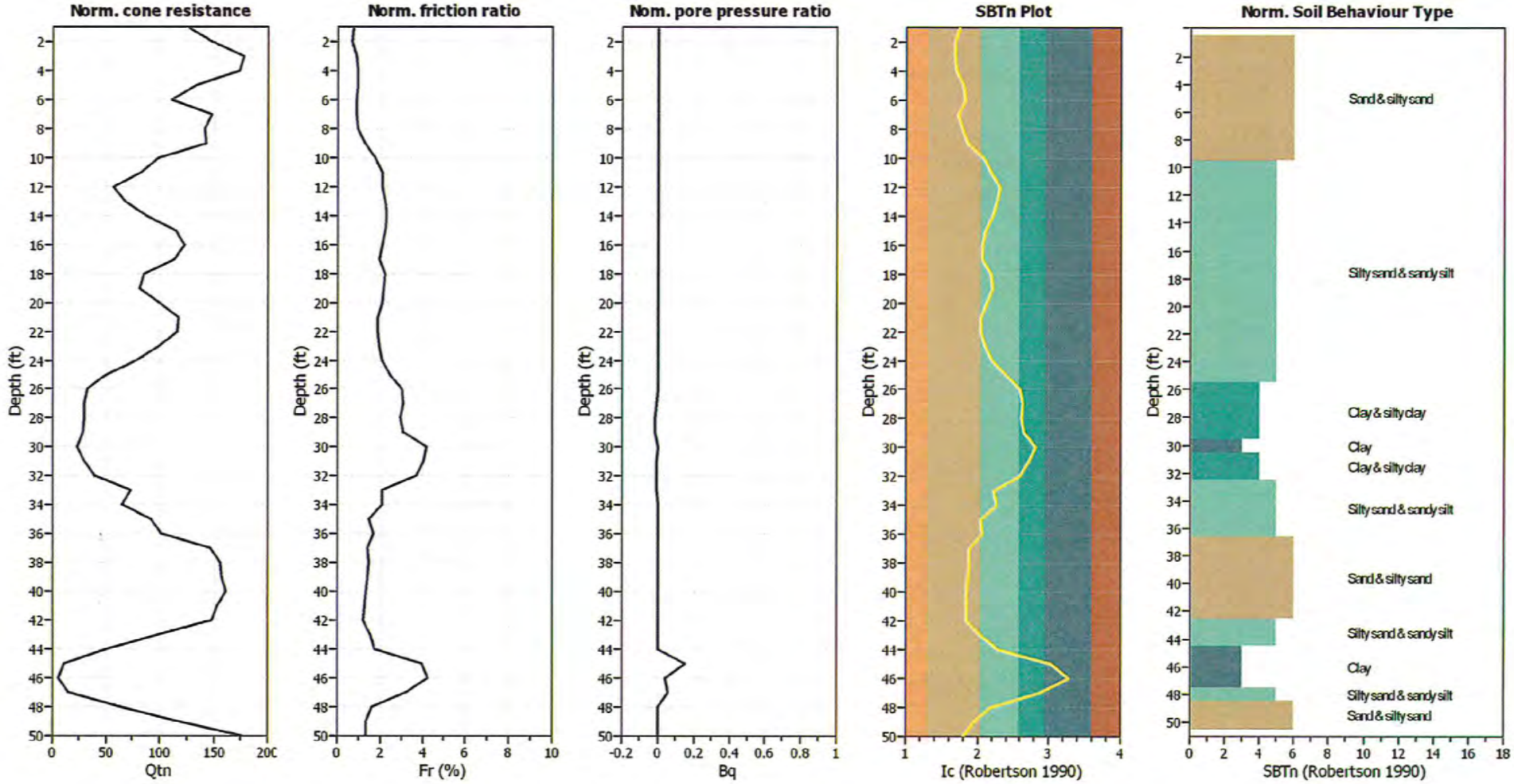
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



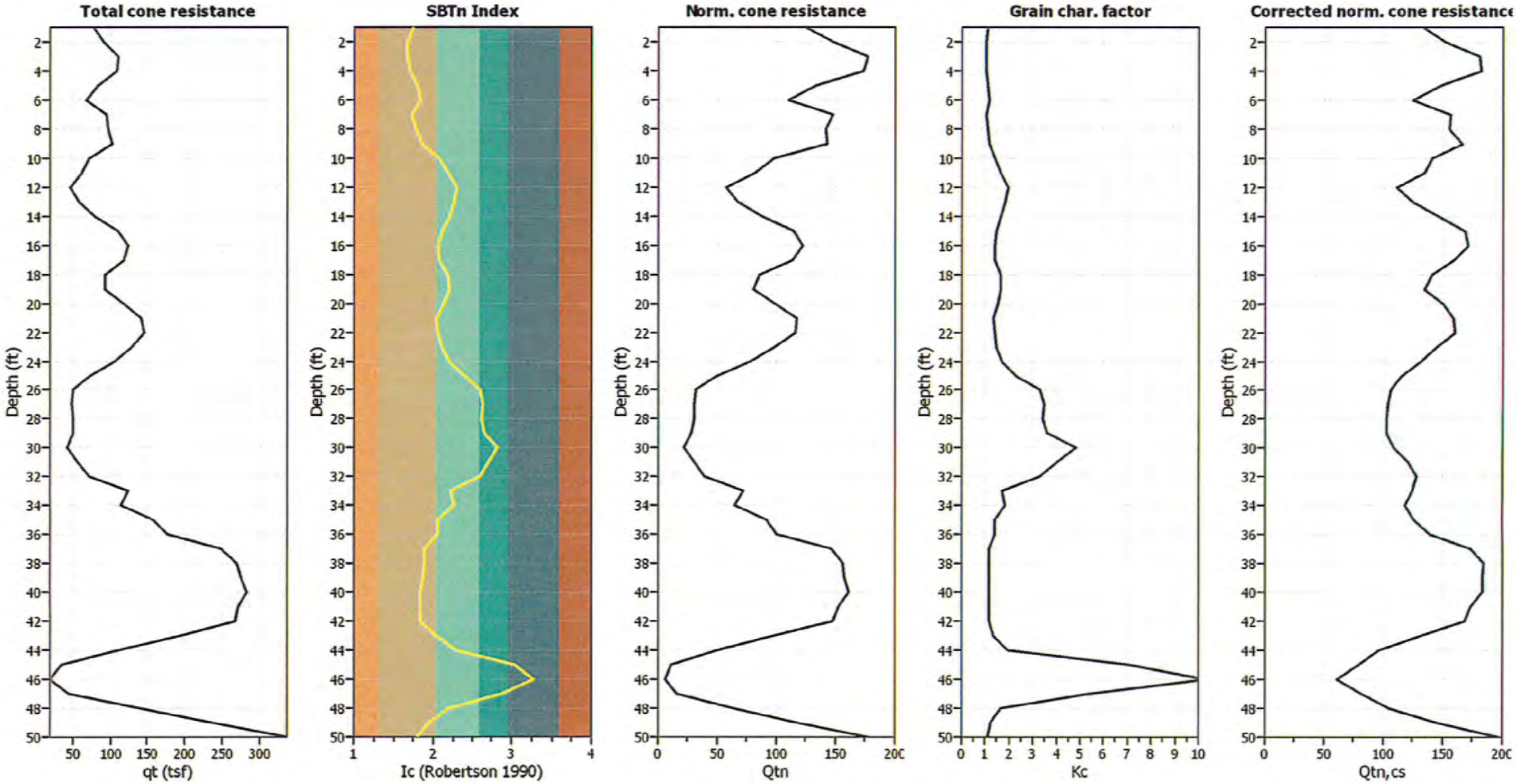
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

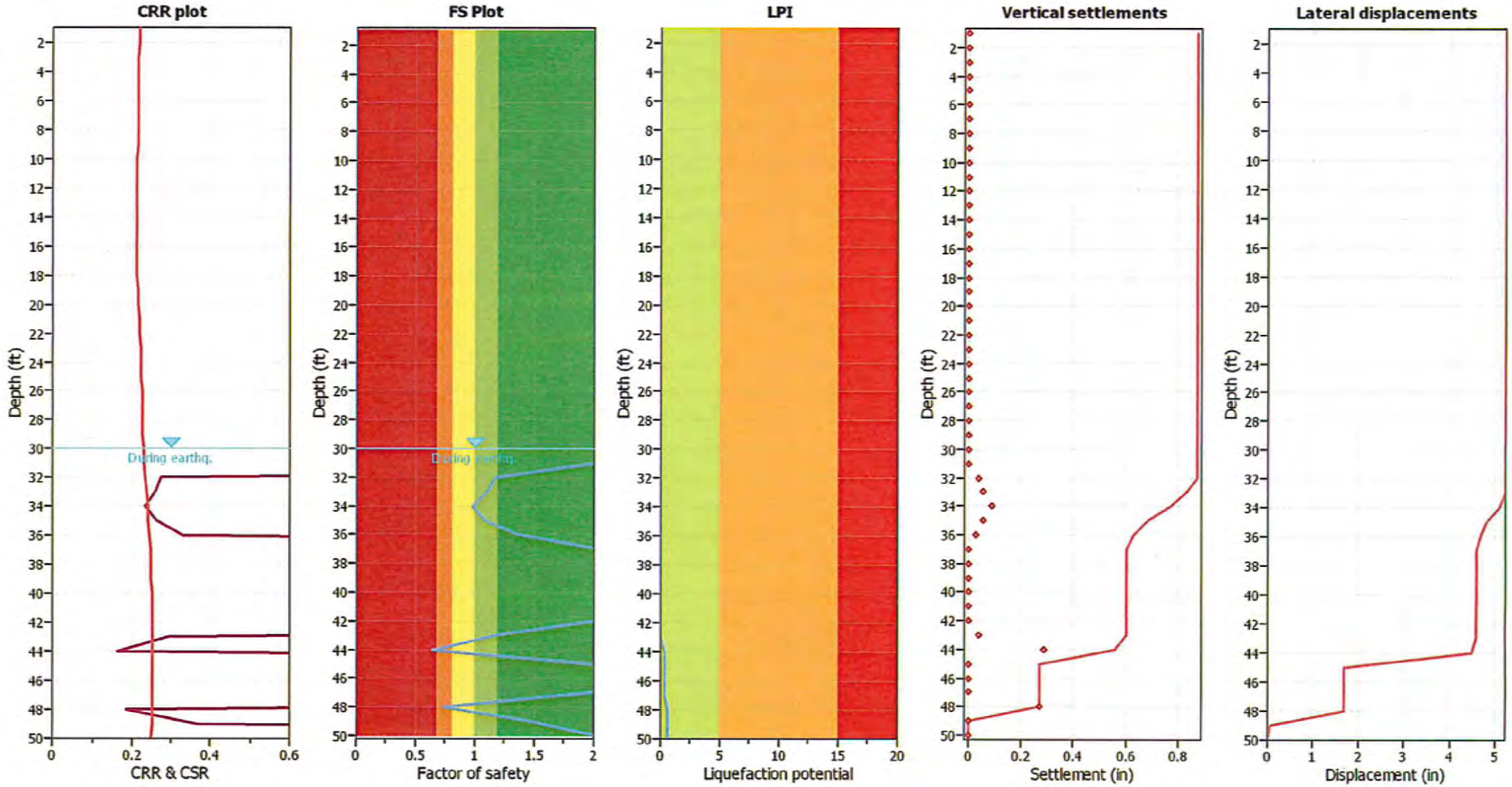
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	44.00 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

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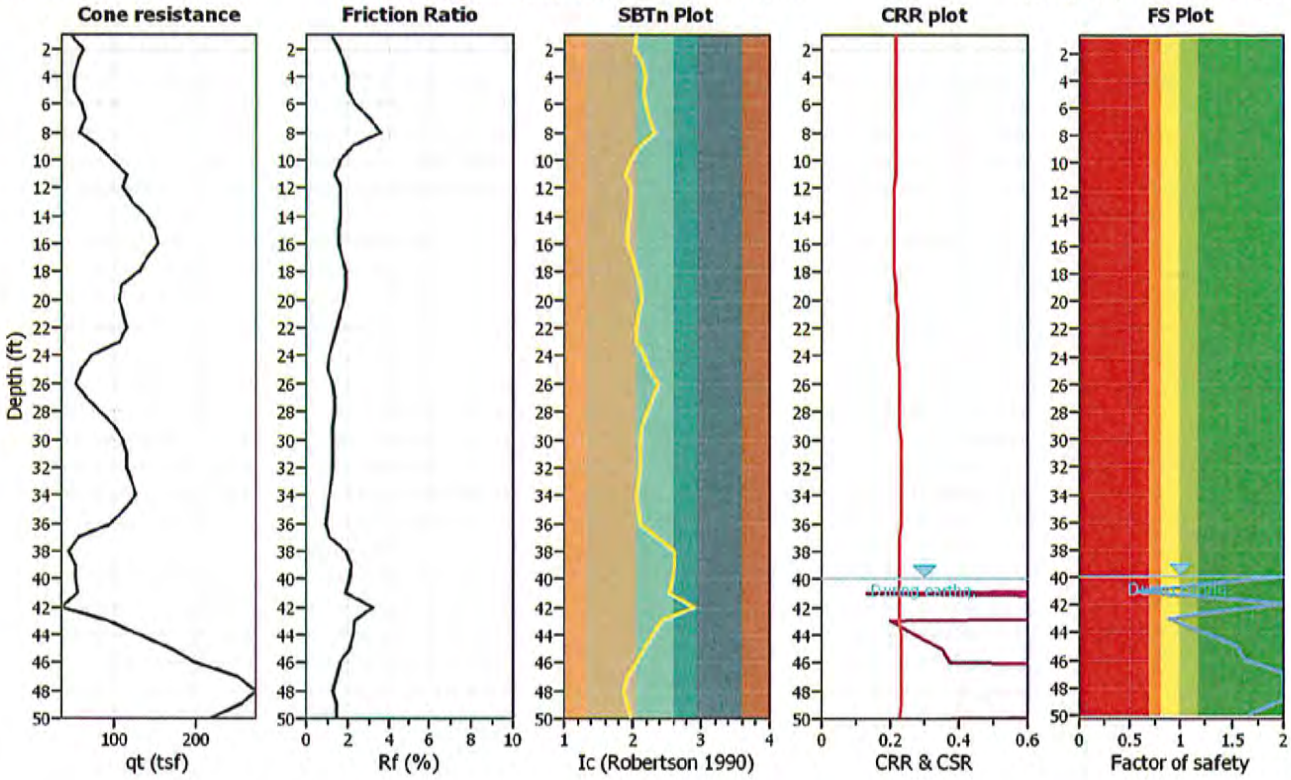
LIQUEFACTION ANALYSIS REPORT

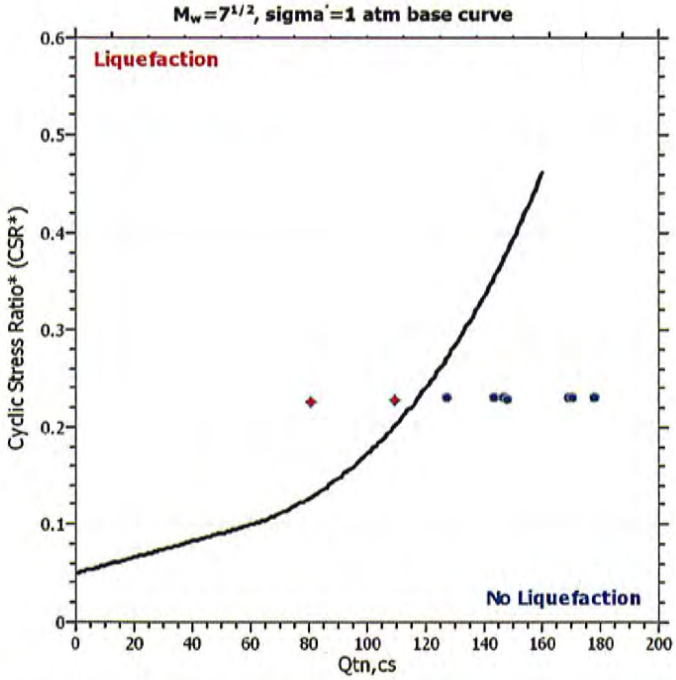
Project title : Integral Communities **Location : 4747 Daisy Avenue, Long Beach, CA**

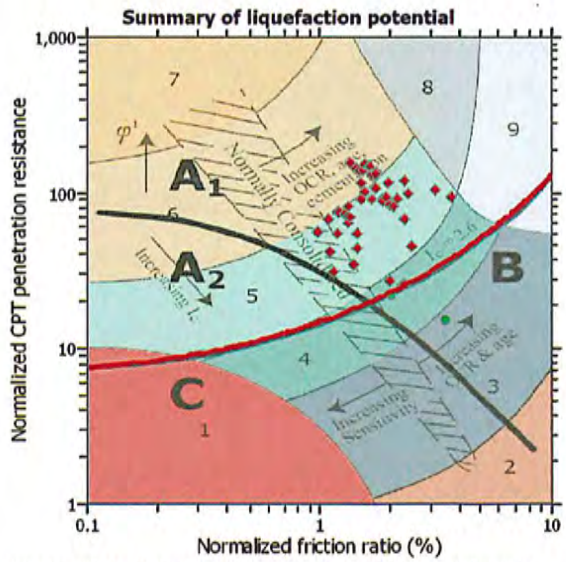
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	32.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	40.00 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:	6.75	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

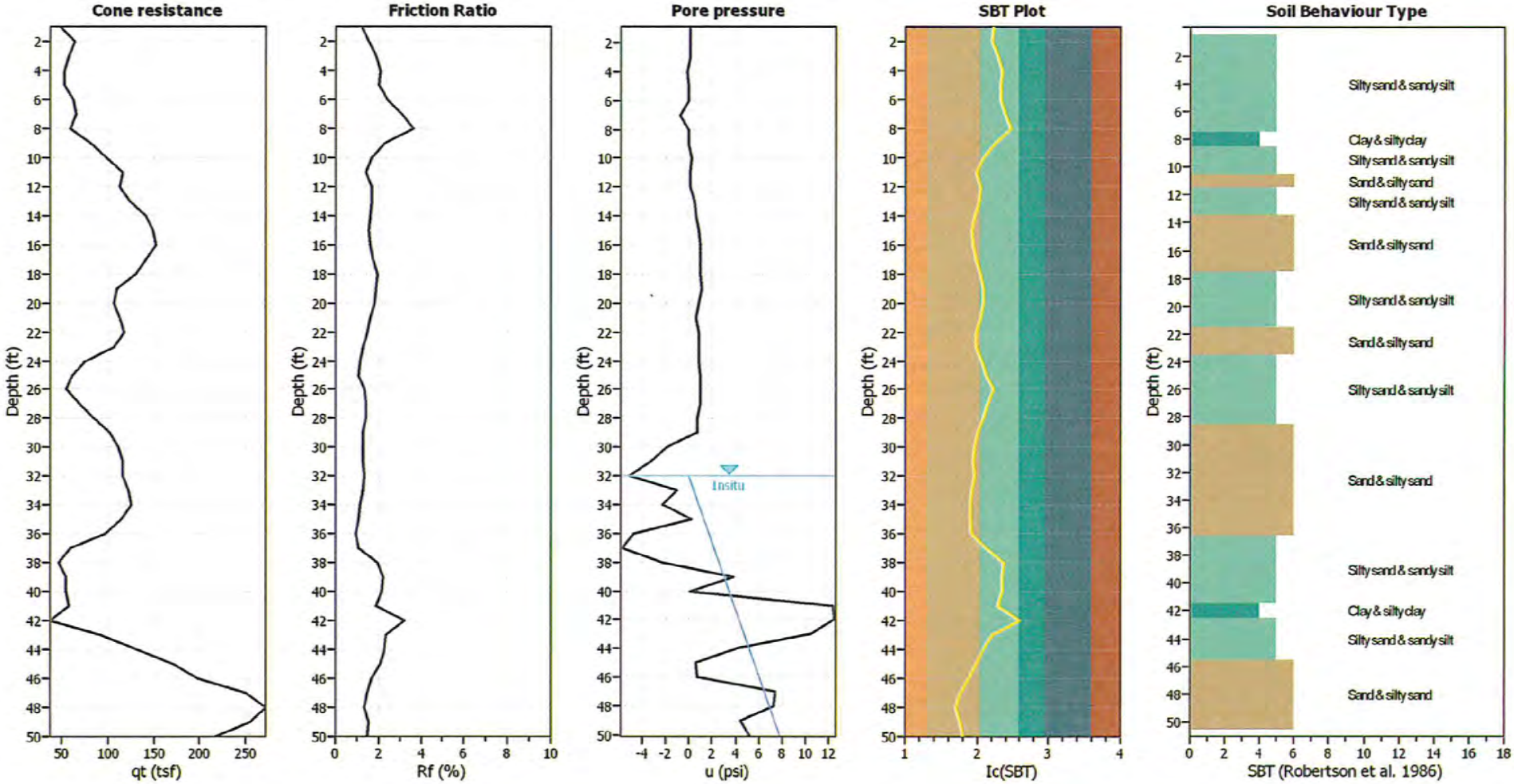






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



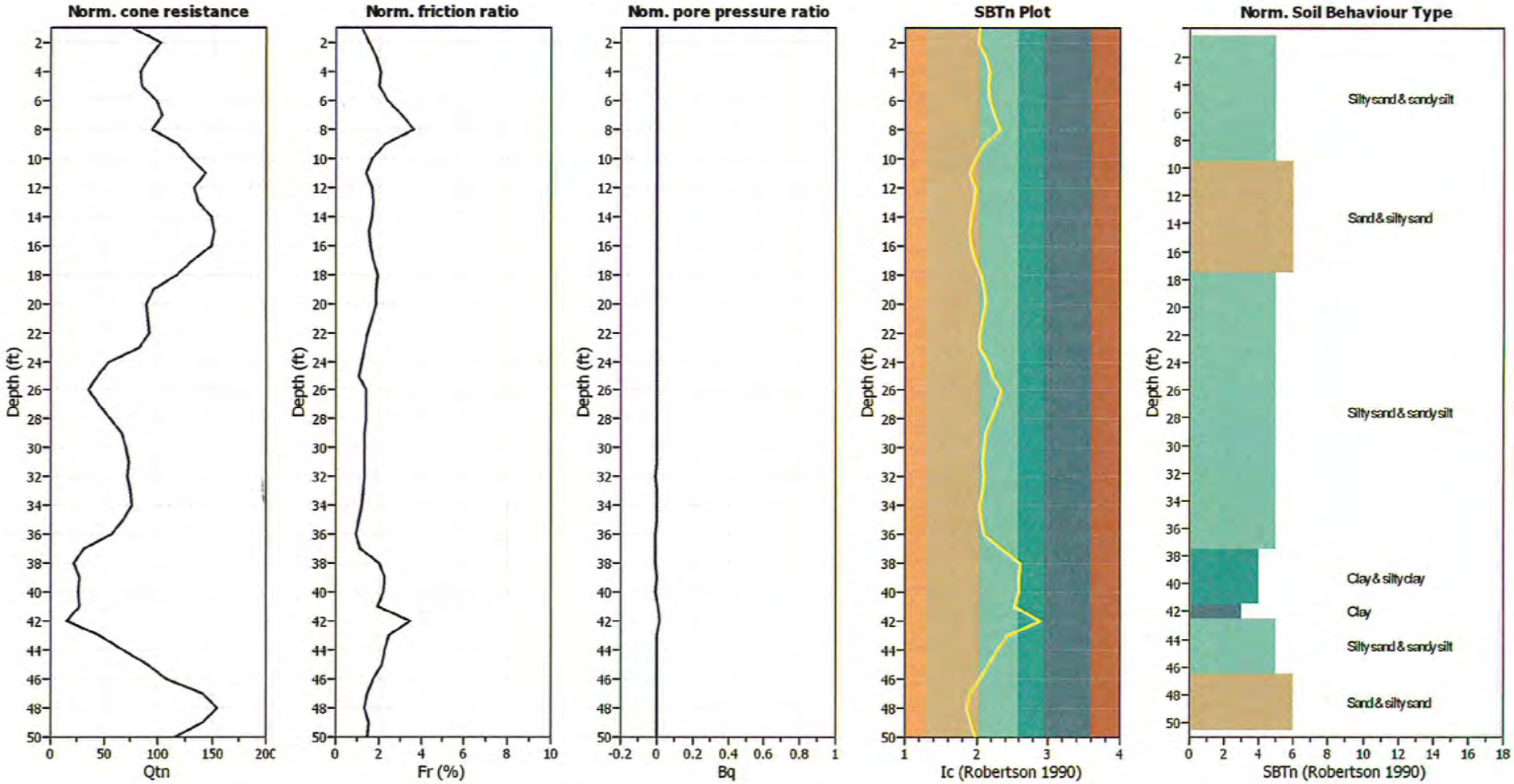
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	40.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	32.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



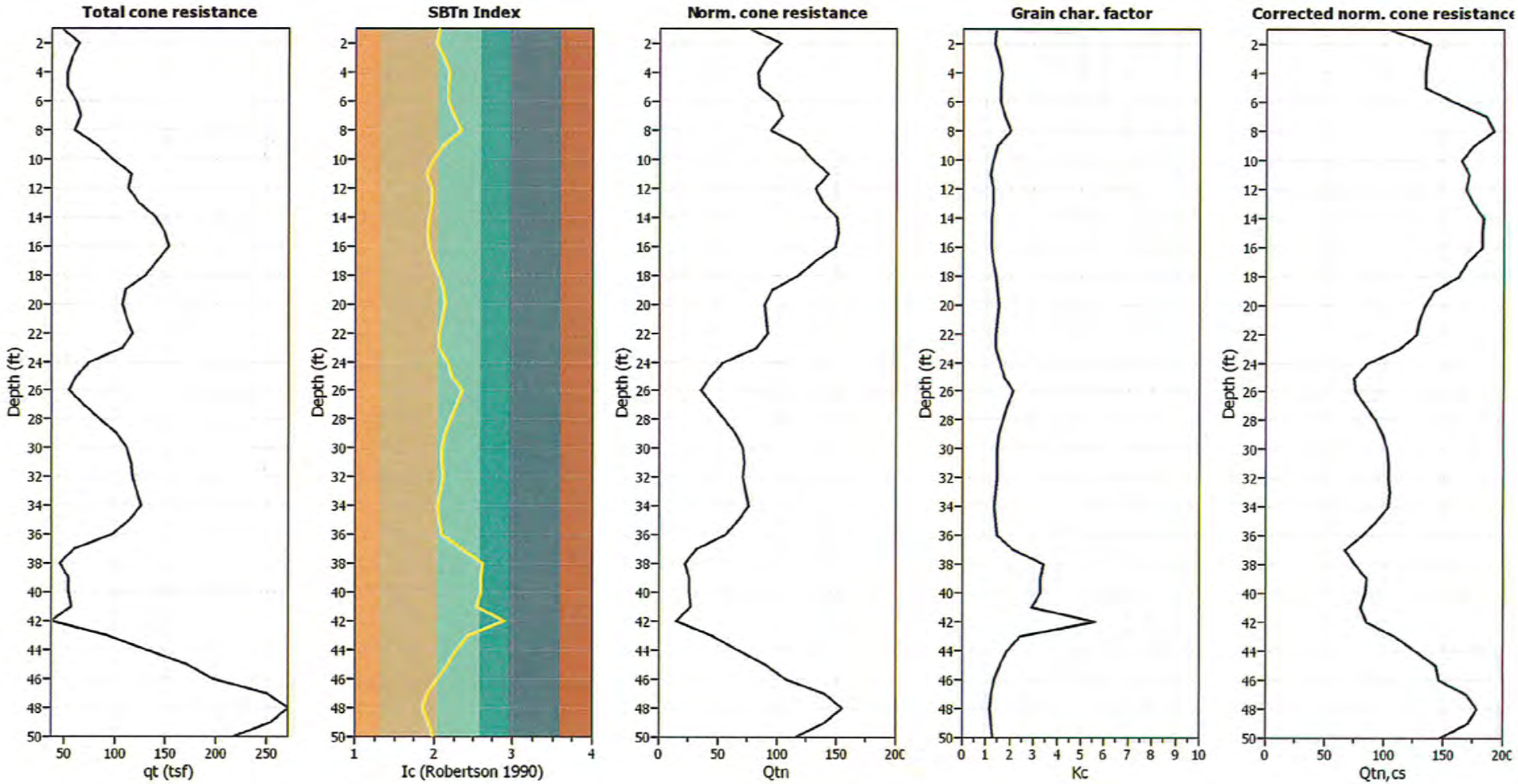
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	40.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	32.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

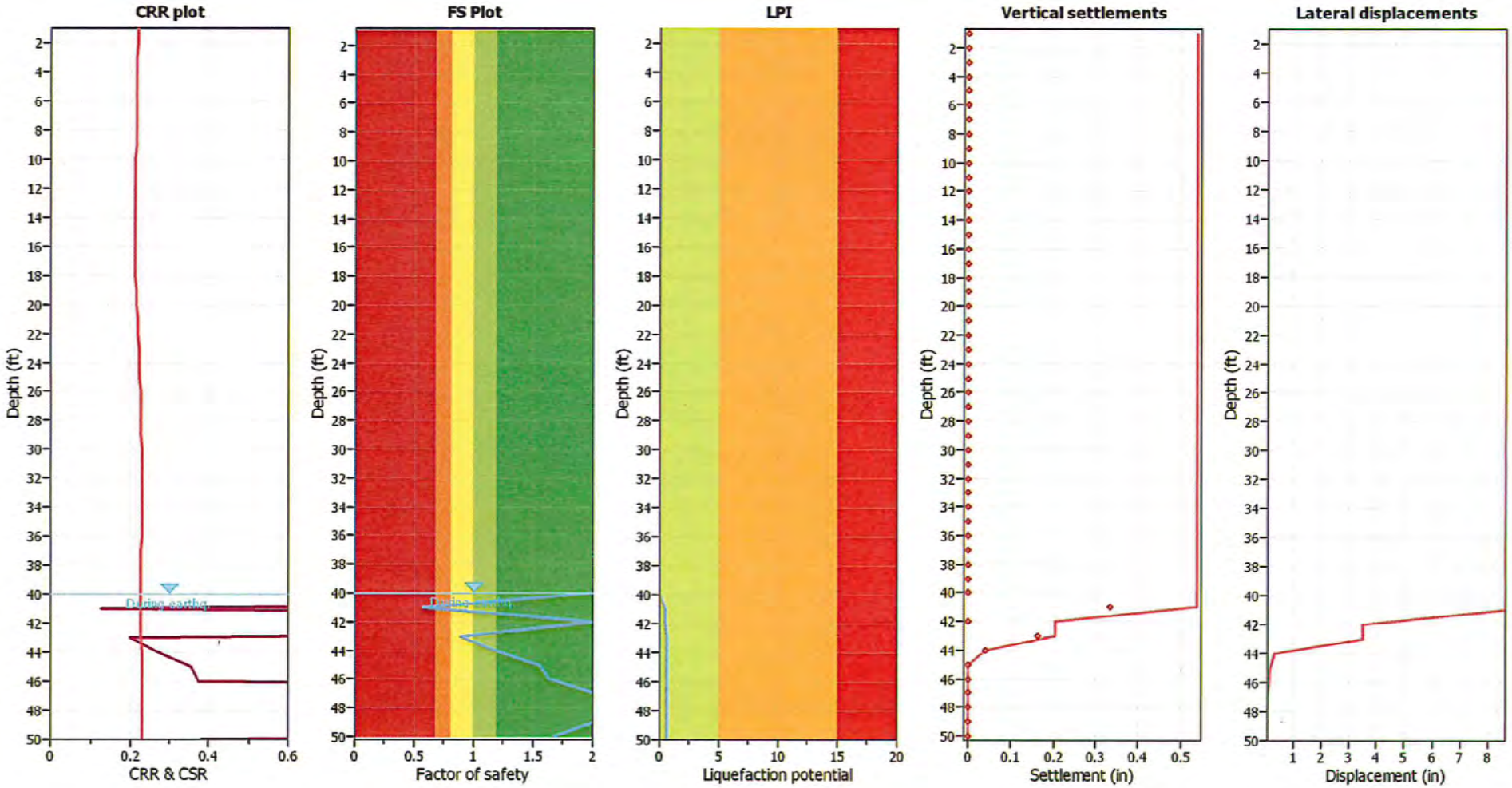
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	40.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	32.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	40.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	32.00 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

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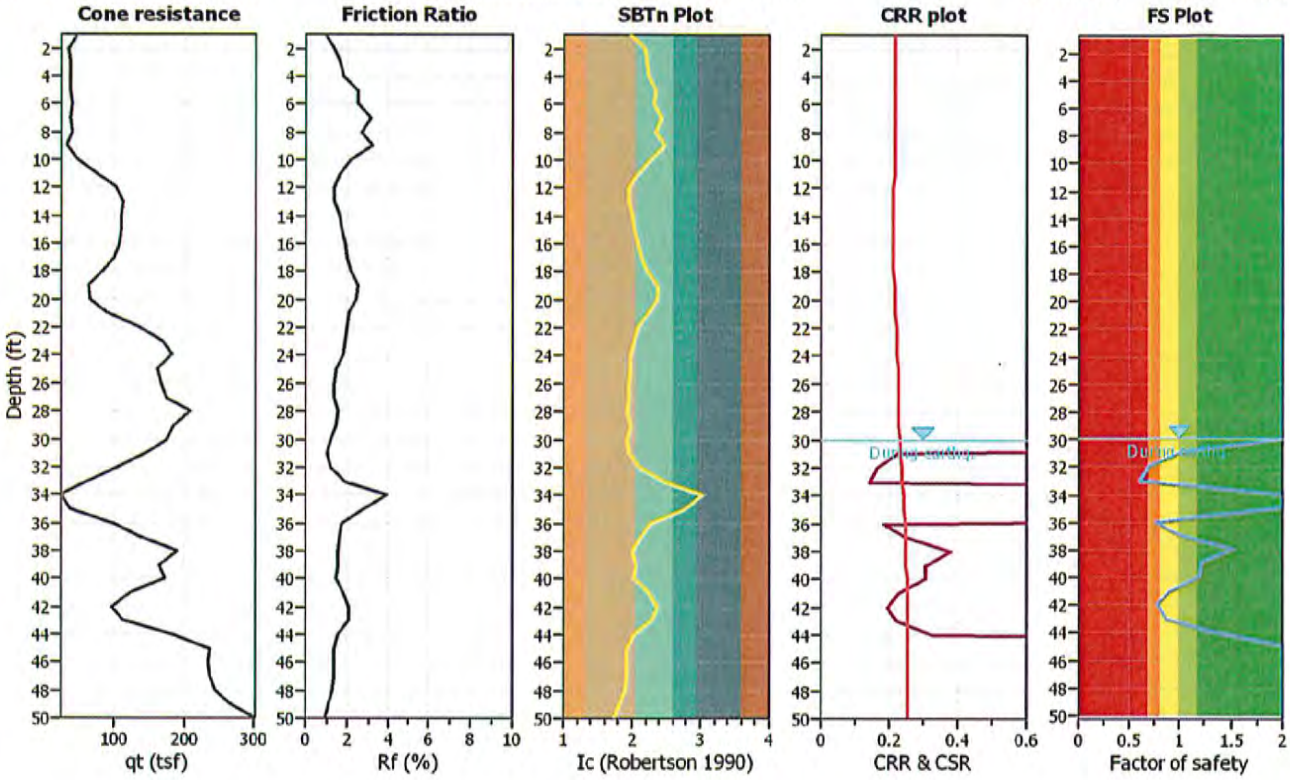
LIQUEFACTION ANALYSIS REPORT

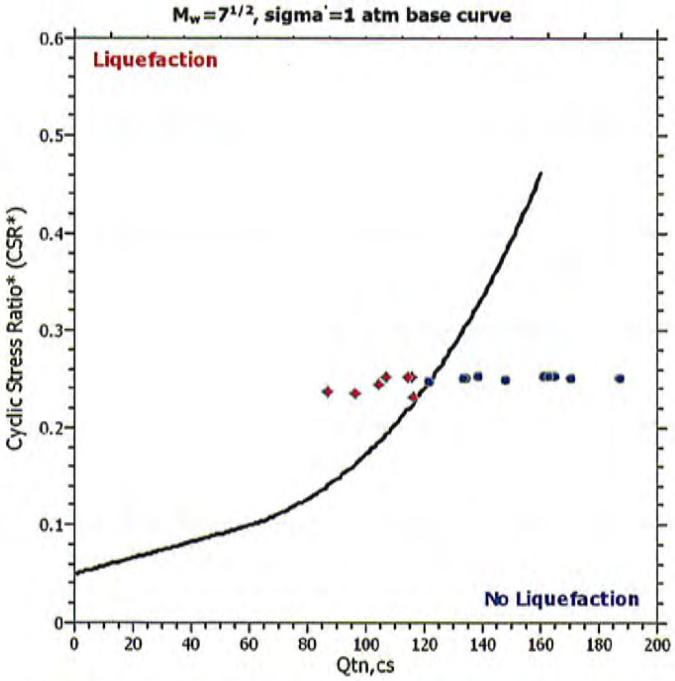
Project title : Integral Communities **Location : 4747 Daisy Avenue, Long Beach, CA**

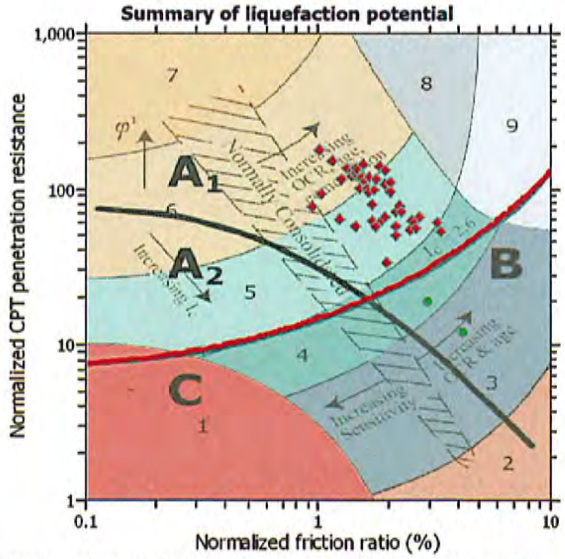
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	30.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	30.00 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:	6.75	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

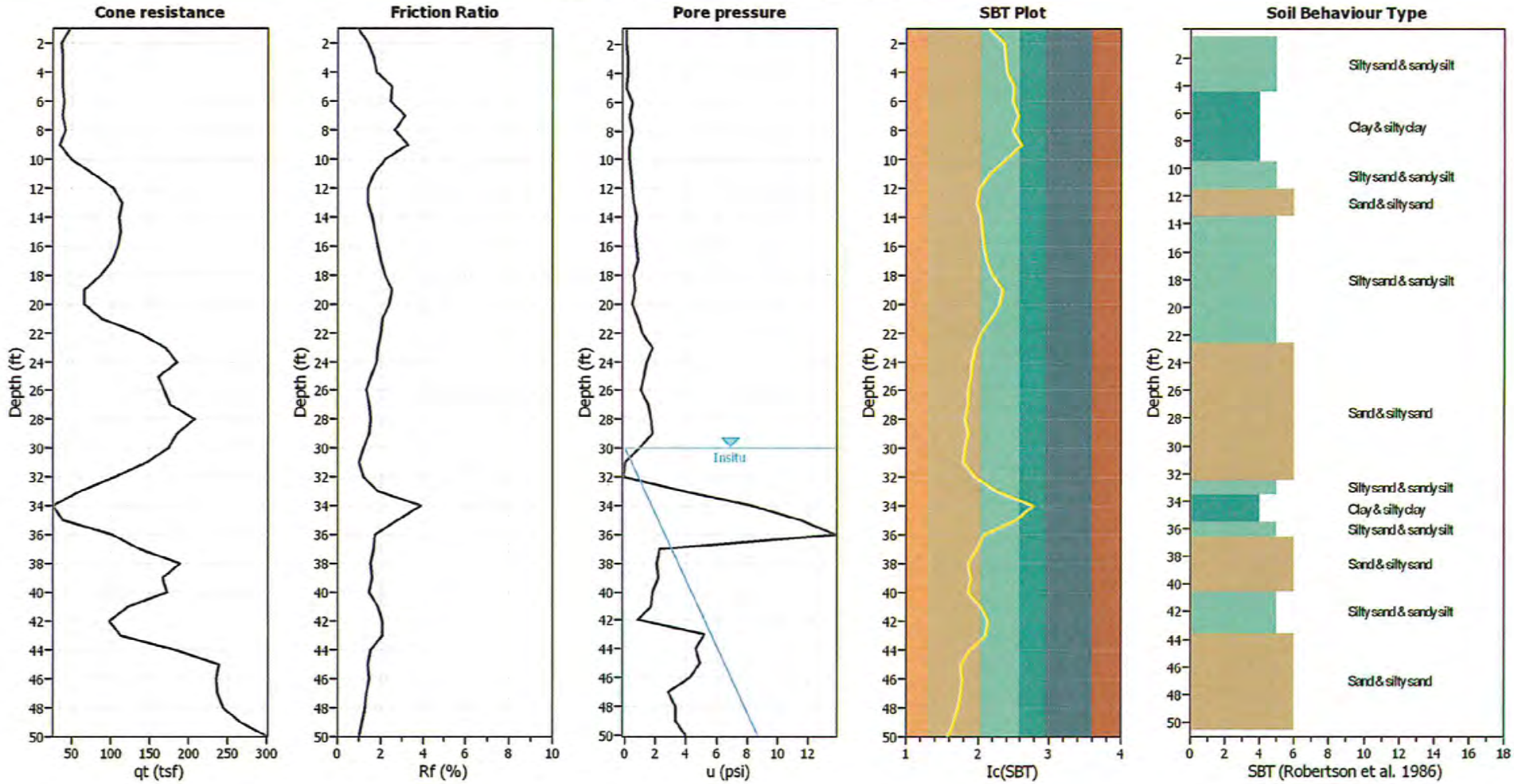






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



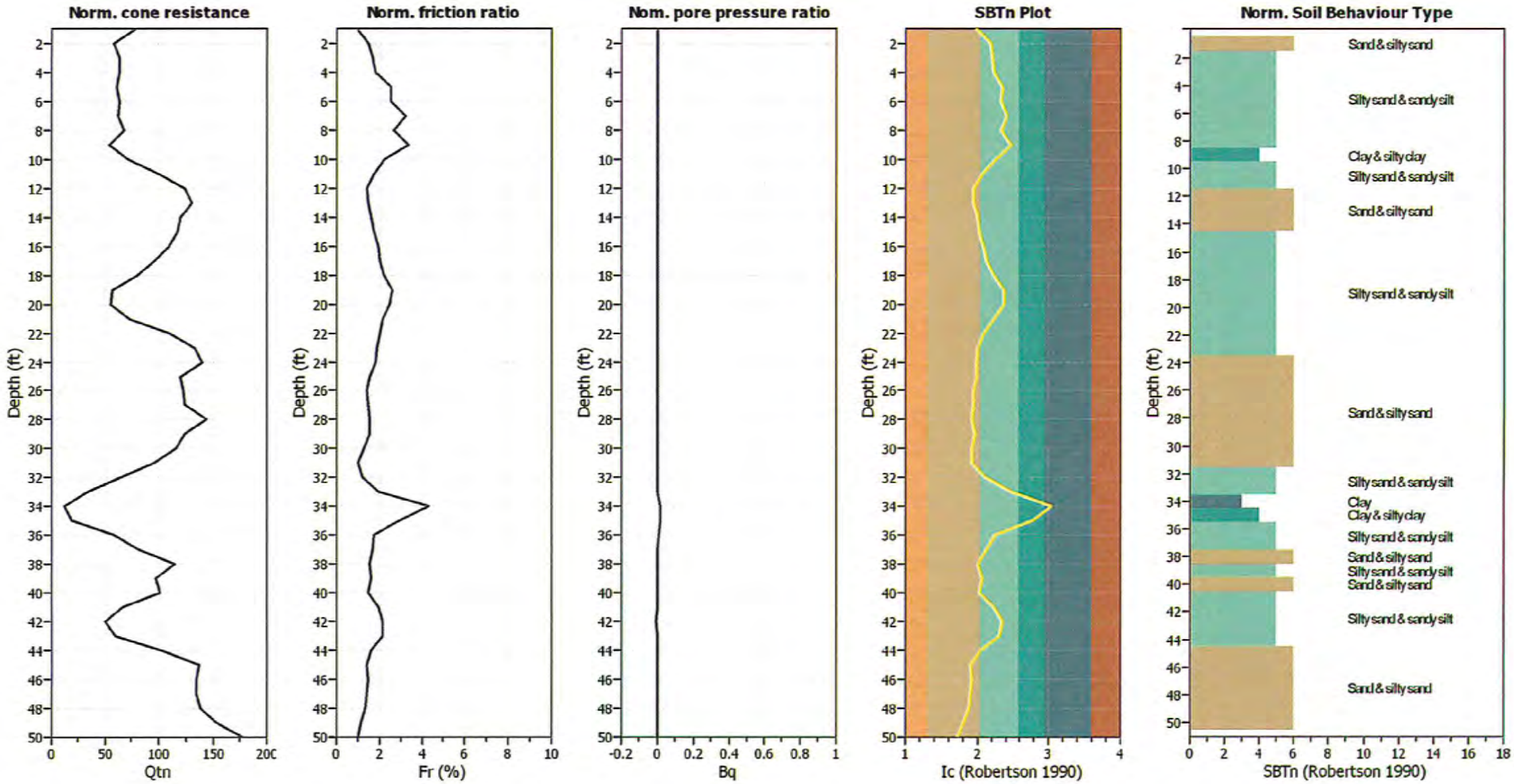
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



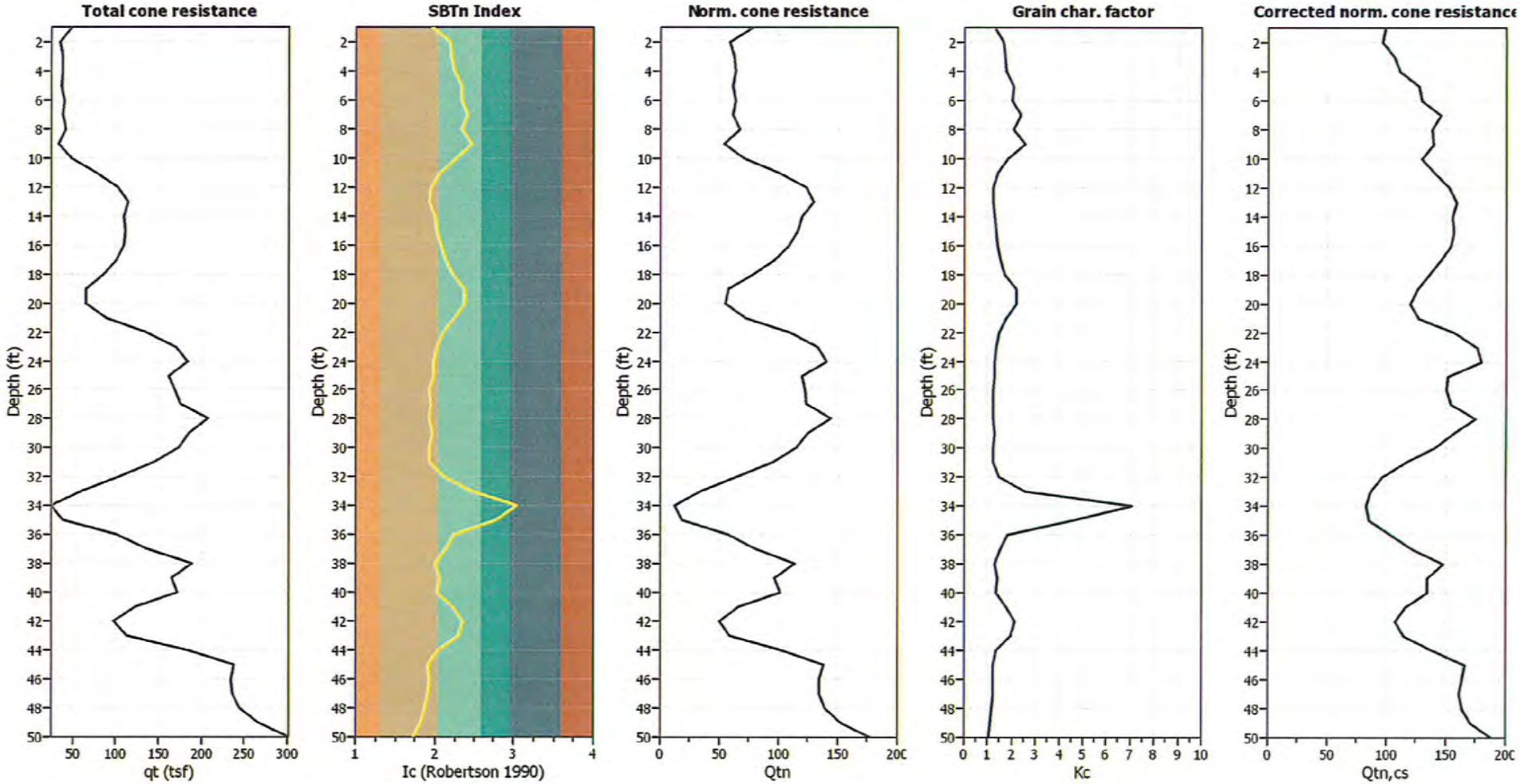
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

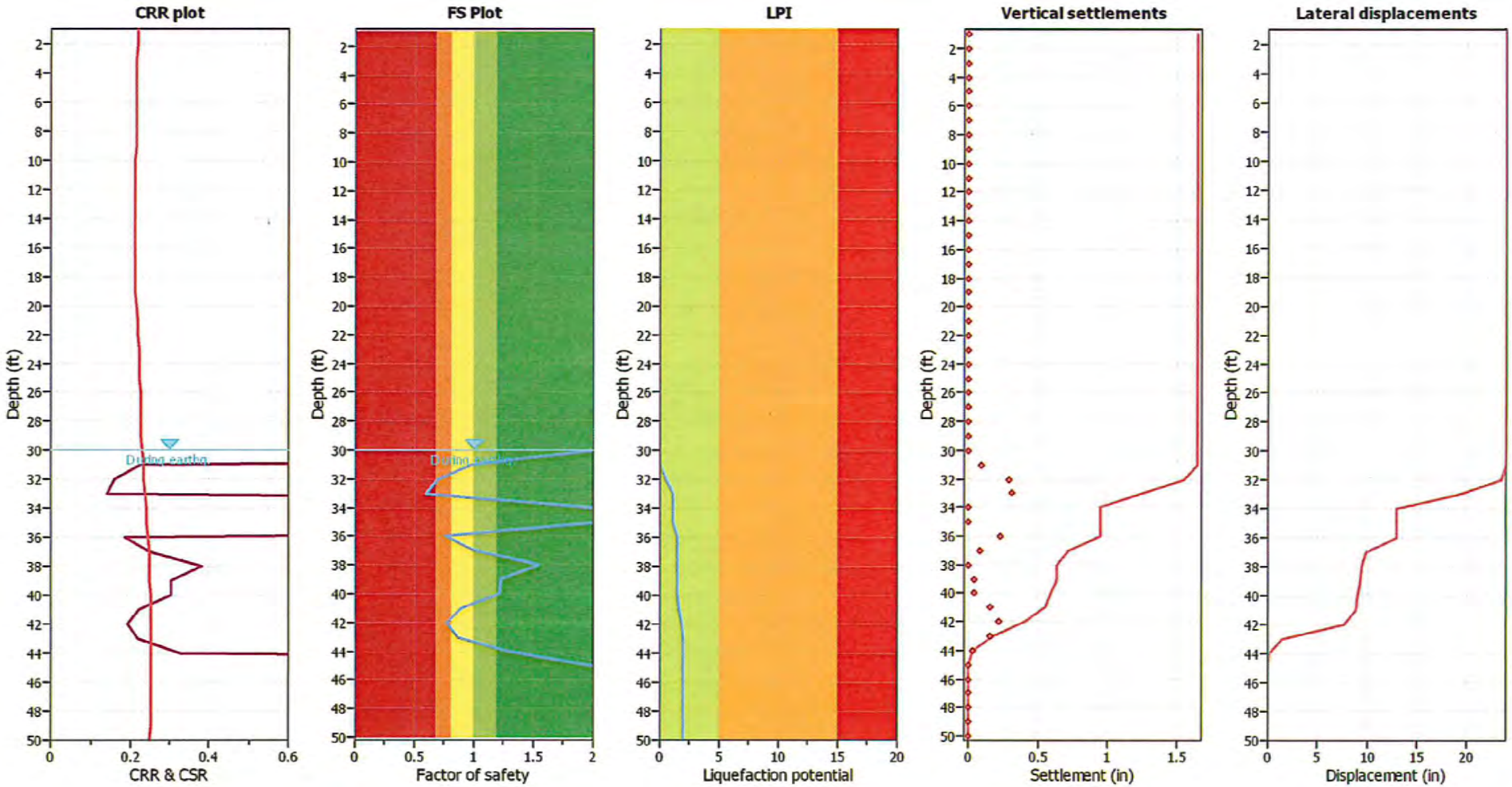
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	30.00 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

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http://www.geologismiki.gr

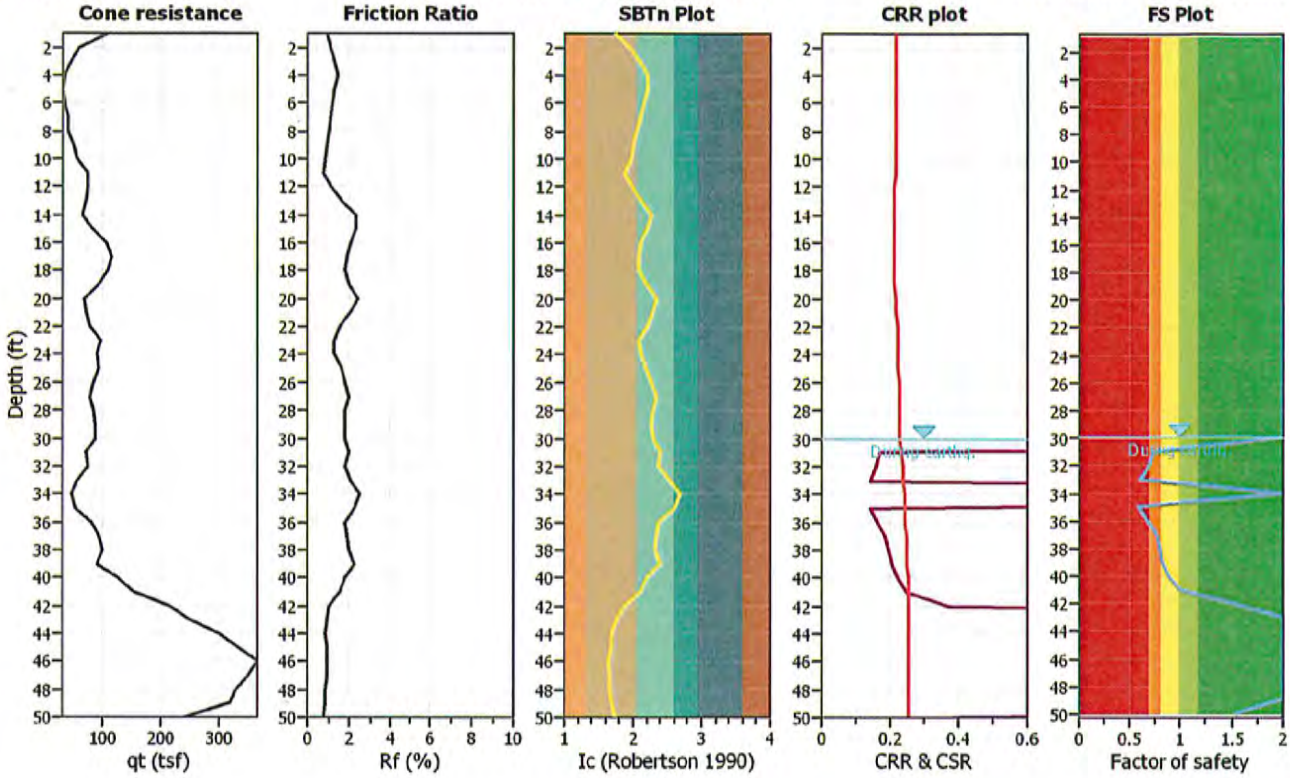
LIQUEFACTION ANALYSIS REPORT

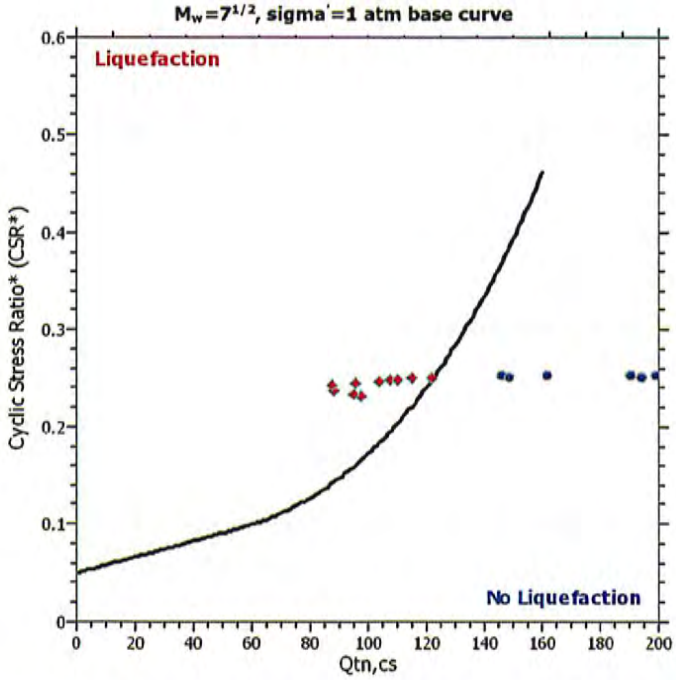
Project title : Integral Communities **Location : 4747 Daisy Avenue, Long Beach, CA**

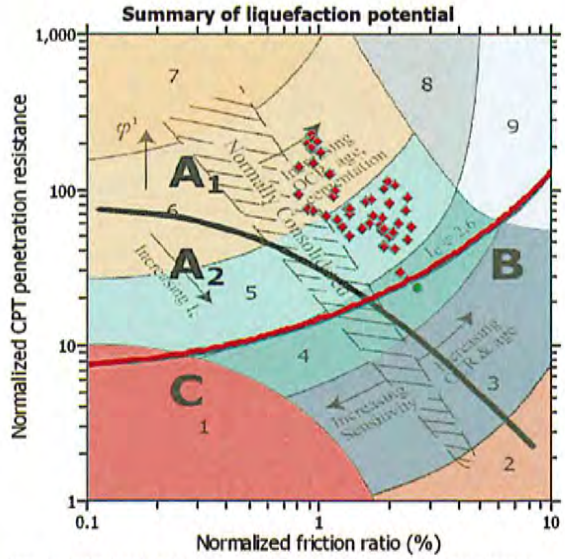
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	34.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	30.00 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:	6.75	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.44	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

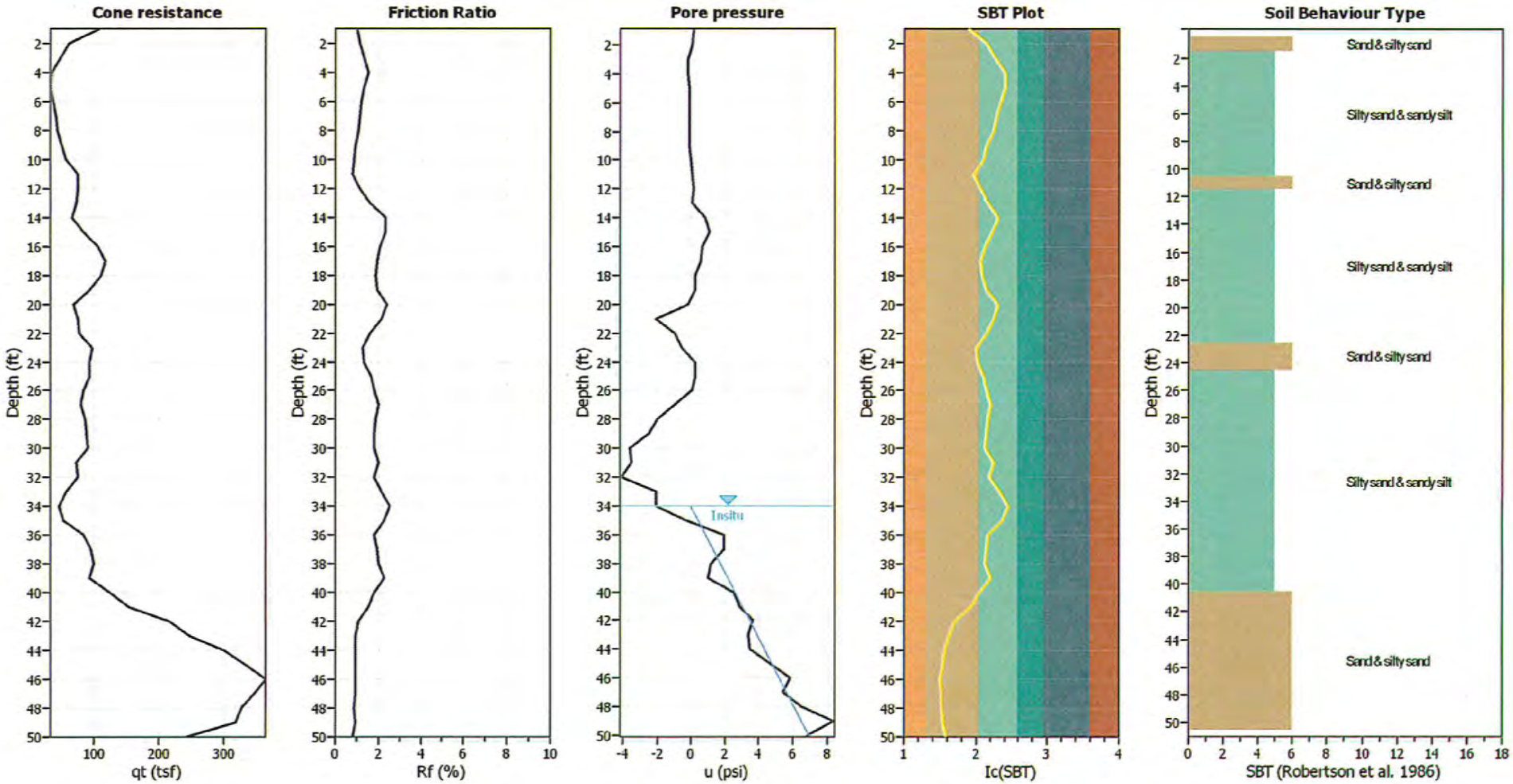






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



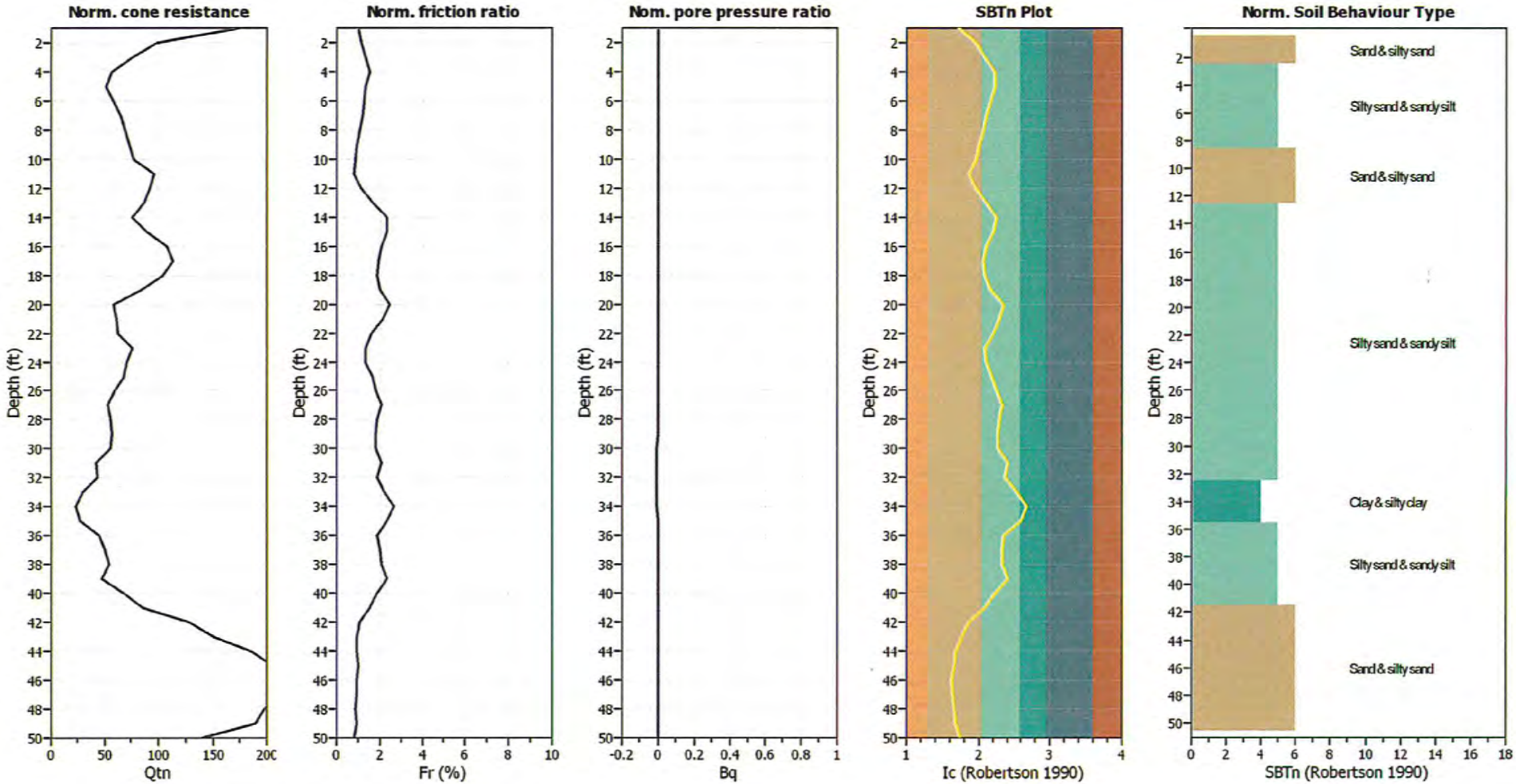
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	34.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	34.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
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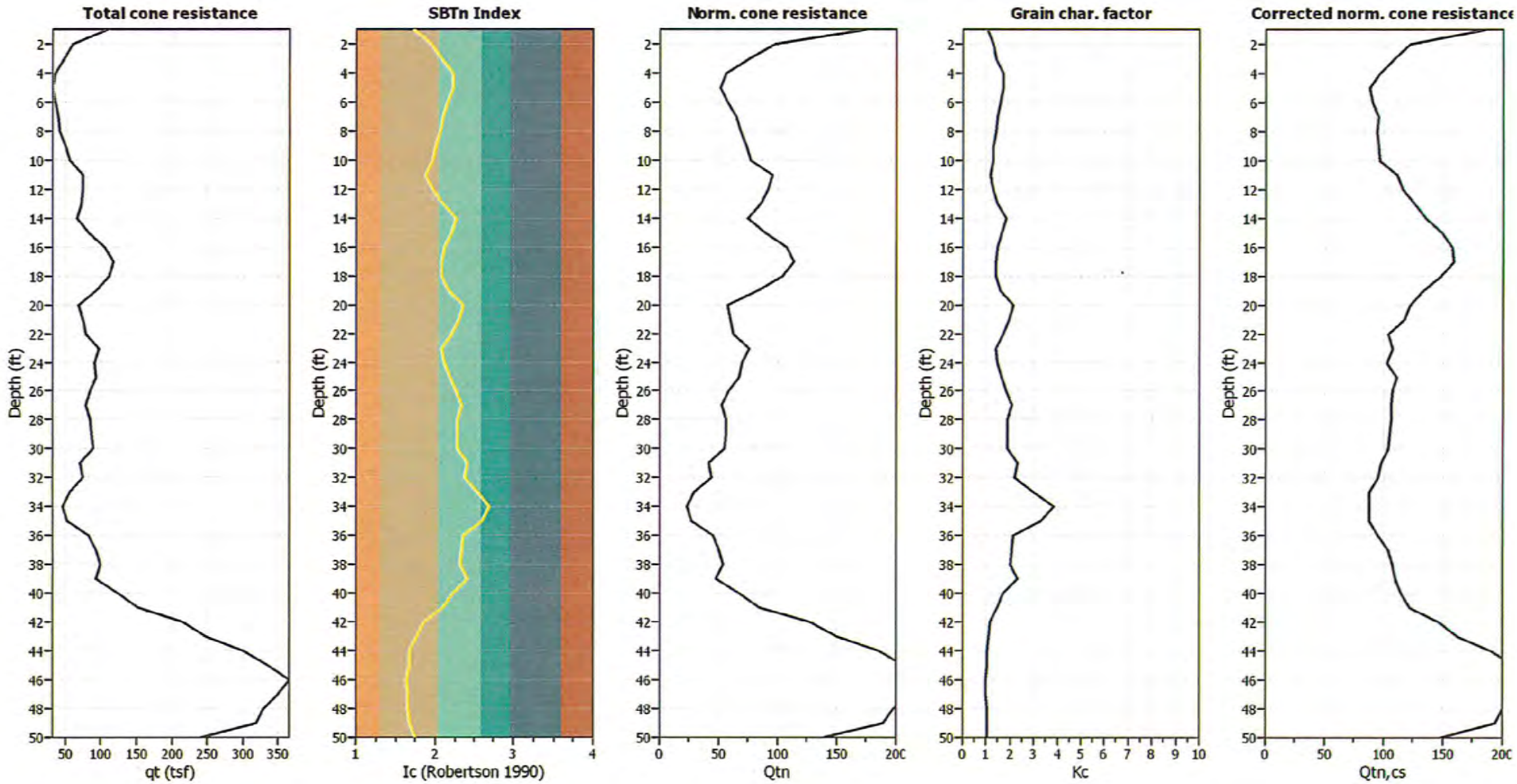
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

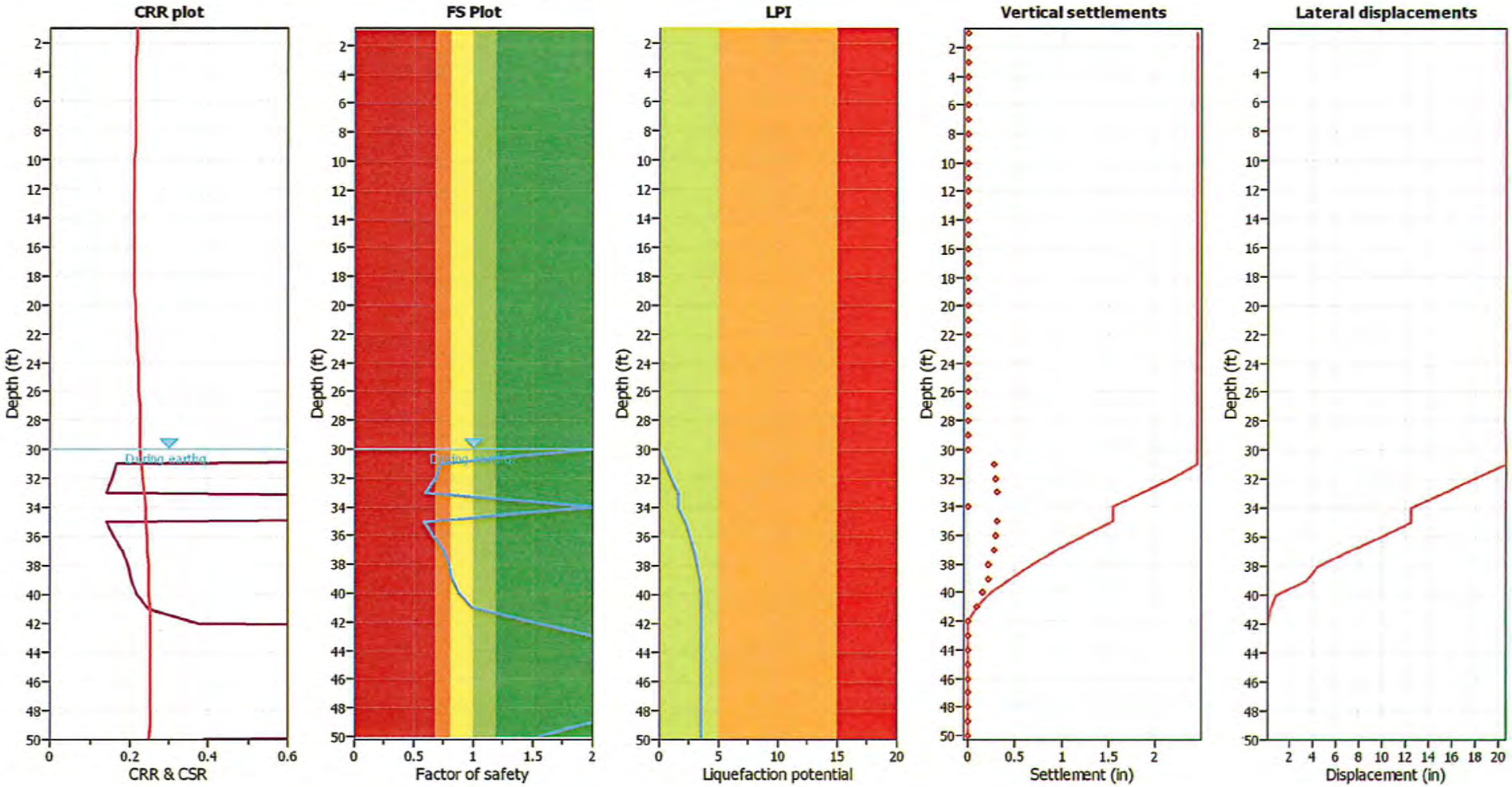
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	34.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	30.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.75	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.44	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	34.00 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

APPENDIX E

SOIL PERCOLATION TEST DATA

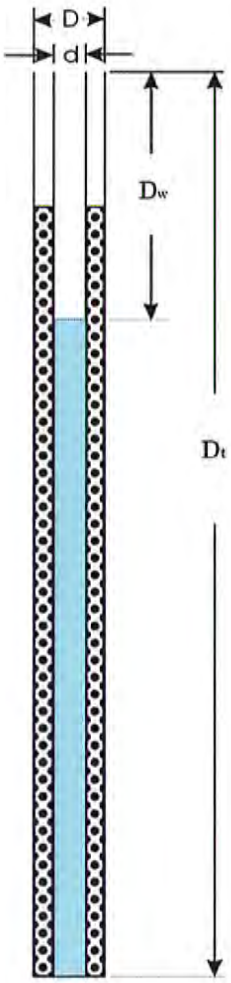
PETRA

PERCOLATION TEST SUMMARY

Job No. 13-443
Project Name: TriPointe Homes - Daisy Avenue, Long Beach
Date: March 4, 2014

Test Number: B-2A

Depth to Bottom,ft (D_t): 3
Diameter of Hole, in (D): 8
Diameter of Pipe, in (d): 3
Agg. Correction (% Voids): 39



Time Interval (min)	Depth to Water Surface Dw (ft)		Change in Head (in)	Perc Rate min/in	Perc Rate gal/day/ft^2
	1st Reading	2nd Reading			
10	1.15	2.43	15.36	0.65	79.44
10	1.10	2.41	15.72	0.64	79.28
10	1.11	2.46	16.20	0.62	83.48
10	1.12	2.53	16.92	0.59	89.79
10	1.17	2.60	17.16	0.58	95.33
10	1.12	2.57	17.40	0.57	93.73
10	1.15	2.61	17.52	0.57	96.95

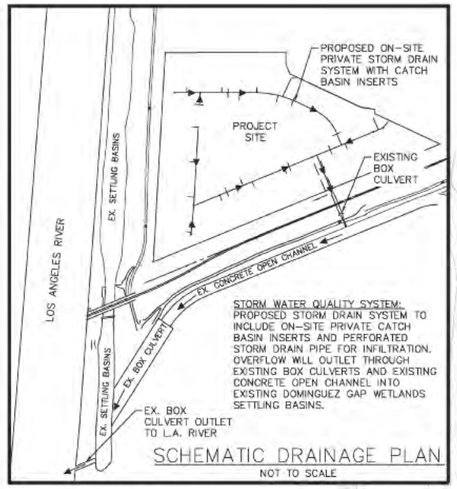
Perc. Rate: 0.57 Minutes/Inch
96.95 gal/day/ft^2

Infiltration Rate: 13.62 Inches/Hour
(Porchet Method)

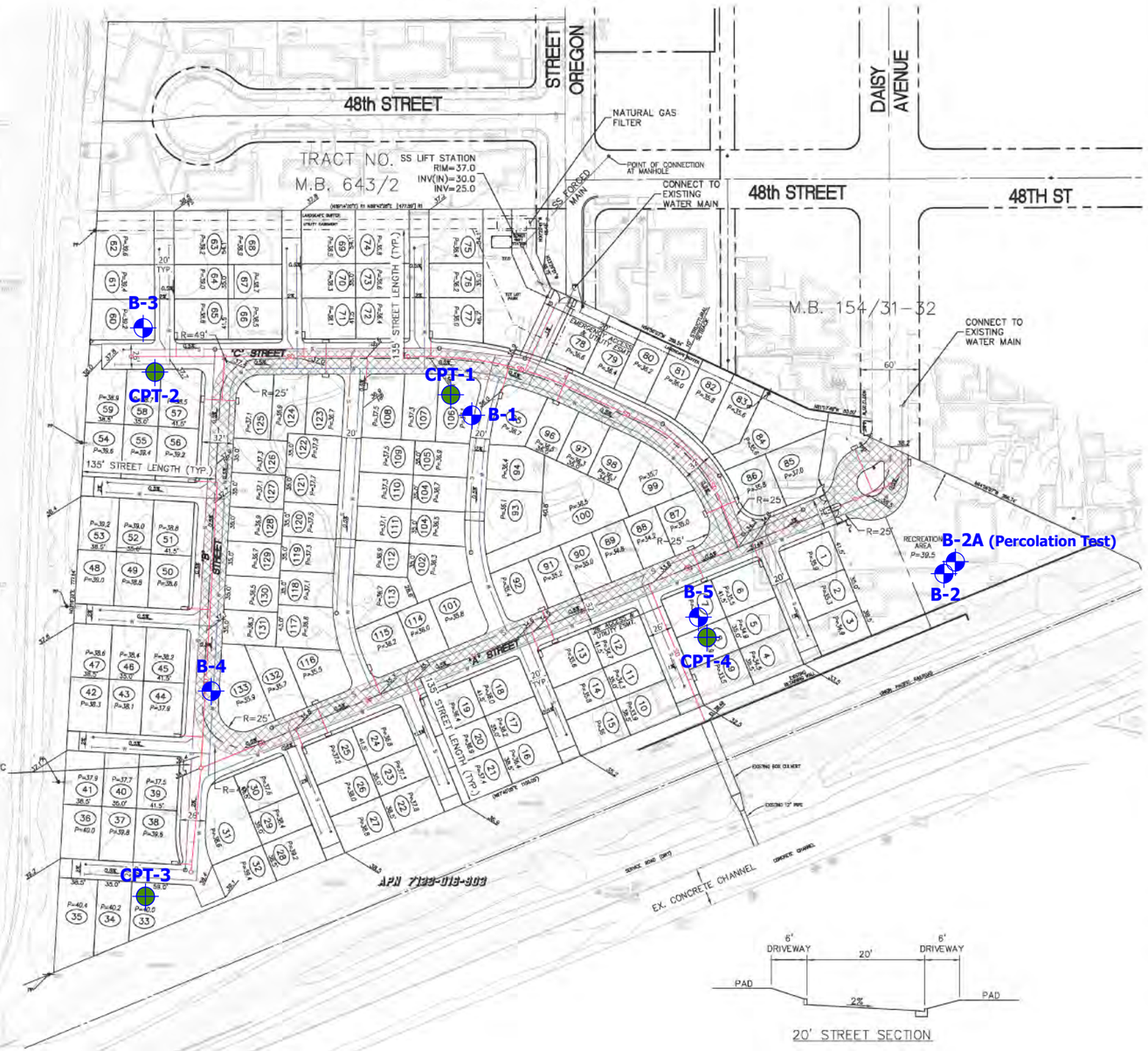
PETRA GEOTECHNICAL, INC.

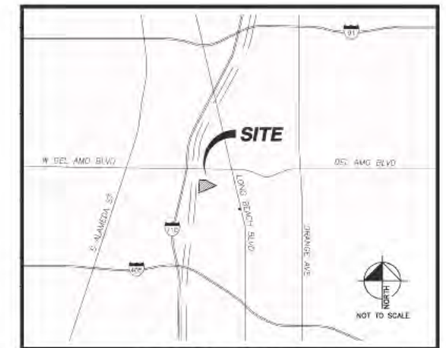
VESTING TENTATIVE TRACT NO. 72608

IN THE CITY OF LONG BEACH, COUNTY OF LOS ANGELES
STATE OF CALIFORNIA
DECEMBER 12, 2013



SCHEMATIC DRAINAGE PLAN
NOT TO SCALE





VICINITY MAP

OWNERS:

THE LONG BEACH PROJECT, LLC
888 SAN CLEMENTE, SUITE 100
NEWPORT BEACH, CA 92660
PHONE: (949) 720-3612

ENGINEER:

KIMLEY-HORN AND ASSOCIATES
6800 OWENSMOUTH AVE., SUITE 410
CANOGA PARK, CA 91303
PHONE: (747) 900-8381
EMAIL: JON.WAKENHUT@KIMLEY-HORN.COM

APN:

7133-016-005

ZONING:

1. EXISTING LAND USE: INACTIVE BOY SCOUT SITE
2. PROPOSED LAND USE: RESIDENTIAL SINGLE FAMILY DWELLINGS
3. EXISTING ZONING: (I) INSTITUTIONAL
4. PROPOSED ZONING: TBD
5. WATER SERVICE PROVIDED BY: LONG BEACH WATER DEPARTMENT
6. SANITARY SEWER SERVICE PROVIDED BY: LONG BEACH WATER DEPARTMENT
7. ELECTRIC SERVICE PROVIDED BY: LONG BEACH GAS & ELECTRIC DEPARTMENT
8. GAS SERVICE PROVIDED BY: LONG BEACH GAS & ELECTRIC DEPARTMENT
9. TELEPHONE SERVICE PROVIDED BY: AT&T
10. CABLE SERVICE PROVIDED BY: TIME WARNER CABLE
11. ALL PROPOSED UTILITIES TO BE UNDERGROUND.
12. DRAINAGE FACILITIES TO BE DESIGNED IN ACCORDANCE WITH THE MASTER PLAN OF DRAINAGE, CITY OF LONG BEACH.
13. ALL GRADING TO CONFORM TO THE CITY OF LONG BEACH GRADING AND EXCAVATION CODE.
14. ALL SLOPES SHALL BE 2:1 UNLESS OTHERWISE NOTED.
15. THIS PROPERTY IS WITHIN THE DOMINGUEZ GAP SPREADING GROUNDS OF COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS SPREADING GROUND FACILITIES.

PROPOSED DEVELOPMENT:

PROPOSED SINGLE FAMILY RESIDENTIAL COMPLEX WITH 133 DWELLING UNITS.

AREA:

EXISTING AREA = 460,093 S.F. (10.56 ACRES)

LEGEND:

- BLOCK WALL
- CHAIN-LINK FENCE
- WROUGHT IRON FENCE
- WOOD FENCE
- UNDERGROUND GAS
- W — UNDERGROUND WATER
- SD — STORM DRAIN
- S — UNDERGROUND SEWER
- BUILDING PERIMETER
- WATER VALVE
- LIGHT POLE
- POWER POLE
- GUY WIRE
- UNIT NUMBER
- SEWER MANHOLE
- PROPERTY LINE
- BOUNDARY
- RIGHT OF WAY
- EASEMENT
- CENTERLINE
- CURB AND GUTTER
- 24' FIRELANE

ABBREVIATIONS:

BFP	BACKFLOW PREVENTOR
CLF	CHAIN LINK FENCE
EPB	ELECTRIC PULL-BOX
ET	ELECTRIC TRANSFORMER
FDC	FIRE DEPARTMENT CONNECTION
FH	FIRE HYDRANT
GPB	GAS PULL-BOX
GVT	GAS VAULT
MB	MAIL BOX
TPB	TELEPHONE PULL-BOX
UTCN	TRAILOR/MODULAR UTILITY CONNECTIONS
WM	WATER METER
WV	WATER VALVE

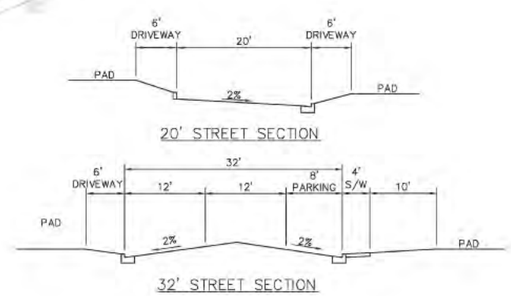
NORTH

GRAPHIC SCALE IN FEET
0 30 60 120

EXPLANATION

B-5 — Approximate Location of Exploratory Boring

CPT-4 — Approximate Location of Cone Penetrometer Sounding



Kimley-Horn and Associates, Inc.

PETRA GEOTECHNICAL, INC.
3190 AIRPORT LOOP DRIVE, SUITE J-1
COSTA MESA, CALIFORNIA 92626
PHONE: (714) 549-9821
COSTA MESA MURRIETA PALM DESERT SANTA CLARITA

EXPLORATION MAP

Vesting Tentative Tract No. 72608
4747 Daisy Ave., Long Beach, CA

SCALE:
see plan

DWG BY	SW
J.N.	13-443
DATE	March 2014

Plate 1