Appendix D

Geotechnical Investigation Report

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GEOTECHNICAL INVESTIGATION

Proposed 5-Story Data Center
1150 Walsh Avenue
Santa Clara, California

PREPARED FOR:

RAGINGWIRE DATA CENTERS, INC.
5470 KIETZKE LANE, SUITE 230
RENO, NEVADA 89511

RagingWire® DATA CENTERS

PREPARED BY:

GEOCON CONSULTANTS, INC.
6671 BRISA STREET
LIVERMORE, CALIFORNIA 94550

GEOCON

GEOCON PROJECT NO. E9008-04-03

NOVEMBER 2017

Project No. E9008-04-03
November 17, 2017

RagingWire Data Centers, Inc.
5470 Kietzke Lane, Suite 230
Reno, Nevada 89511

Attention: Mr. Carl Lubawy

Subject: PROPOSED 5-STORY DATA CENTER
1150 WALSH AVENUE
SANTA CLARA, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Dear Mr. Lubawy:

In accordance with your authorization, we have performed a geotechnical investigation for the subject project in Santa Clara, California. Our investigation was performed to observe the soil and geologic conditions that may impact site development for the project as presently planned. The accompanying report presents the results of our investigation and conclusions and recommendations pertaining to the geotechnical aspects of the proposed project. The findings of this study indicate the site is suitable for development as planned provided the recommendations of this report are implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Sincerely,

GEOCON CONSULTANTS, INC.

DRAFT

Shane Rodacker, PE, GE
Senior Engineer

(1/e-mail) Addressee
(1/e-mail) Paradigm Structural Engineers
Attention: Mr. Kurt Lindorfer
(1/e-mail) Gensler
Attention: Mr. Michael Downey

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for a data center proposed at 1150 Walsh Avenue in Santa Clara, California (see Vicinity Map, Figure 1). The purpose of this investigation was to evaluate the subsurface soil and geologic conditions in the area of planned development and provide conclusions and recommendations pertaining to the geotechnical aspects of project design and construction, based on the conditions encountered during our study.

The scope of this investigation included field exploration, laboratory testing, engineering analysis and the preparation of this report. Our field exploration was performed on September 1 and September 13, 2017 and included the advancement of four Cone Penetrometer Test (CPT) soundings to maximum depths of approximately 60 feet and drilling four exploratory borings to approximately 45 foot below existing grade at the site. The locations of the CPT soundings and soil borings are depicted on the Site Plan, Figure 2. A detailed discussion of our field investigation and boring logs and CPT profiles are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent geotechnical parameters. Appendix B presents the laboratory test results in tabular format and graphical format. Appendix C presents selected output from our liquefaction analysis.

The opinions expressed herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE CONDITIONS AND PROJECT DESCRIPTION

The site is an approximately 3 ⅓-acre parcel (Santa Clara County APN 224-585-003) on the southern side of Walsh Avenue in Santa Clara. The parcel is occupied by various single-story office and warehouse buildings with associated paved areas used for parking, driveways and storage. All existing improvements will be razed to accommodate the new data center. Site topographic information (BKF Engineers, July 2017) indicates the ground surface at the site is generally flat with existing grades on the order of 45 feet MSL.

We understand the proposed data center will be five stories in height and up to 87 feet above existing grade. We anticipate the facility will utilize structural steel framing with cast-in-place concrete floor slabs. No significant subterranean levels are proposed but a base isolation system is planned to mitigate the effects of seismic shaking. The base isolation system will be contained in single level extending approximately 5 feet below existing grade. Information from the project structural engineer (Paradigm Structural Engineers) indicates factored dead + live column loads of approximately 2,600 kips are anticipated. Column spacing will be on the order of 33 to 35 feet throughout the data center.

With the exception of cuts for the base isolation level, we anticipate fills and cuts will be on the order of two feet or less to attain design subgrade elevation for the development. A dedicated electrical substation and ancillary site improvements such as new pavements, underground utilities and landscaping are also proposed. The proposed site configuration is depicted on Figure 2.

3. GEOLOGIC SETTING

Santa Clara is located within the Coast Ranges Geomorphic Province of California, which is characterized by a series of northwest trending mountains and valleys along the north and central coast of California. Topography is controlled by the predominant geological structural trends within the Coast Range that generally consist of northwest trending synclines, anticlines and faulted blocks. The dominant structure is a result of both active northwest trending strike-slip faulting, associated with the San Andreas Fault system, and east-west compression within the province.

The San Andreas Fault (SAF) is a major right-lateral strike-slip fault that extends from the Gulf of California in Mexico to Cape Mendocino in northern California. The SAF forms a portion of the boundary between two tectonic plates on the surface of the earth. To the west of the SAF is the Pacific Plate, which moves north relative to the North American Plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is concentrated on the SAF but also distributed, to a lesser extent, across several other faults including the Hayward and Calaveras faults, among others. Together, these faults are referred to as the SAF system.

Basement rock west of the SAF is generally granitic, while to the east it consists of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (205 to 65 million years old). Overlying the basement rocks are Cretaceous (about 140 to 65 million years old) marine, as well as Tertiary (about 65 to 1.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have typically been extensively folded and faulted largely because of movement along the SAF system, which has been ongoing for about the last 25 million years, and regional compression during the last about 4 million years. The inland valleys, as well as the structural depression within which San Francisco Bay is located, are filled with unconsolidated to semi-consolidated deposits of Quaternary age (about the last 1.6 million years). Continental deposits (alluvium) consist of unconsolidated to semi-consolidated sand, silt, clay and gravel, while the bay deposits typically consist of soft organic-rich silt and clay (bay mud) or sand.

Available geologic mapping by the United States Geological Survey (USGS) indicates the site is underlain by Holocene-age alluvial deposits. Some references describe this alluvium as flood basin deposits while others delineate the alluvial materials as clays of intra-fan areas.

4. GEOLOGIC HAZARDS

Geologists and seismologists recognize the San Francisco Bay Area as one of the most seismically-active regions in the United States. The significant earthquakes that occur in the Bay Area are associated with crustal movements along well-defined active fault zones that generally trend in a northwesterly direction.

The site and greater Bay Area are seismically dominated by the presence of the active San Andreas Fault System. In the theory of plate tectonics, the San Andreas Fault System is a transform fault that forms the boundary between the northward moving Pacific Plate (west of the fault) and the southward moving North American Plate (east of the fault). Locally, the movement is distributed across a complex system of strike-slip, right lateral parallel and subparallel faults, which include the San Andreas, Hayward and Calaveras faults, among others.

The table below presents approximate distances to active faults in the site vicinity based on web-based mapping by the USGS and CGS. Site latitude is 37.3692° N, 121.9533° W.

TABLE 4.1
REGIONAL FAULT SUMMARY

Fault Name	Approximate Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
Silver Creek	2 ¼	7.3
Hayward – S Extension	6 ½	6.7
Monte Vista Shannon	7 ¼	6.4
Hayward	8 ¼	7.3
Calaveras	9 ½	6.9
San Andreas	10 ¾	8.0
Sargent	16 ¼	7.0
Pleasanton	17 ¾	6.6
Los Positas	18 ¼	6.4
Zayante-Vergeles (Upper)	18 ½	7.0
Zayante-Vergeles (Lower)	21 ¼	7.0
San Gregorio	24 ¼	7.4
Greenville	24 ½	6.9

Faults tabulated above and many others in the Bay Area are sources of potential ground motion. However, earthquakes that might occur on other faults within the northern California area are also potential generators of significant ground motion and could cause ground shaking at the site.

4.1 Surface Fault Rupture

The site is not within a currently established State of California Earthquake Fault Zone or Santa Clara County Geologic Hazard Zone for surface fault rupture hazards. No active or potentially-active faults are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. By CGS definition, an active fault is one with surface displacement within the last 11,000 years. A potentially-active fault has demonstrated evidence of surface displacement with the past 1.6 million years. Faults that have not moved in the last 1.6 million years are typically considered inactive.

4.2 Ground Shaking

We used the USGS web-based *Unified Hazard Tool* to estimate the peak ground acceleration (PGA) and mean magnitude associated with a 2,475-year return period that corresponds to an event with 2 percent chance of exceedance in 50 years. The USGS estimated PGA is 1.22g and the mean magnitude is 7.1 for borderline Seismic Site Class D (V_S30 = 259 m/sec) base on a recent 2014 model within the application.

While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site. We understand a performance based structural design based on non-linear dynamic procedure (time history analyses) will be used for the data center.

4.3 Liquefaction

The site is located within State of California and County of Santa Clara Seismic Hazard Zones for liquefaction. Interactive web-based mapping by USGS and CGS indicates the site soils possesses a "high" susceptibility to liquefaction. Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore pressure buildup under the cyclic shear stresses associated with intense earthquakes. Primary factors that trigger liquefaction are: moderate to strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly graded sands and silty sands), and saturated soil conditions (shallow groundwater). Due to the increasing overburden pressure with depth, liquefaction of granular soils is generally limited to the upper 50 feet of a soil profile.

We assessed the potential for liquefaction using the computer software program *CLiq* (Version 1.7, Geologismiki) and the in-situ soil parameters measured in the CPT soundings. The software applied the methodology of Boulanger and Idriss (2014) to the CPT data to evaluate liquefaction potential and estimate resultant settlements. Our analysis also considered the potential for cyclic softening in clayey soils. Our evaluation incorporated an earthquake moment magnitude (M_w) of 7.1 and a groundwater depth of 8 feet. The groundwater depth in our analysis was based on historic high groundwater levels mapped by CGS. We used a ground motion/Peak Ground Acceleration (PGA) of 0.50 g for our analysis based on the USGS *US Seismic Design Maps* application.

Our liquefaction analysis identified potentially liquefiable layers at each CPT location. In general, these layers are less than 3 feet in thickness and located more than 15 feet below existing grade at the site. Consequences of liquefaction can include ground surface settlement, ground loss (sand boils) and lateral slope displacements (lateral spreading). For liquefaction-induced sand boils or fissures to occur, pore water pressure induced within liquefied strata must exert enough force to break through overlying, non-liquefiable layers. Based on methodology recommended by Youd and Garris (1995), which modified and advanced original research by Ishihara (1985), a capping layer of non-liquefiable soil can prevent the occurrence of sand boils and fissures. In our opinion, the presence of the non-liquefiable layer that mantles the site (which was observed to be at least 15 feet thick in our soil borings) and the depth to significant liquefiable layers, the potential for ground loss due to sand boils or fissures in a seismic event is considered low.

The likely consequence of potential liquefaction at the site is settlement. Our analysis indicates that total ground surface settlements on the order of 1 ½ inch or less may result from liquefaction and/or cyclic softening after a seismic event. Based on the soil conditions encountered in our CPTs, the liquefaction-induced settlements will generally occur in soil layers between approximately 15 feet and 45 feet below existing grade. Selected output from our liquefaction analysis is presented in Appendix C.

4.4 Landslides

There are no known landslides near the site, nor is the site in the path of any known or potential landslides. We do not consider the potential for a landslide to be a significant hazard to this project.

4.5 Tsunamis and Seiches

The site is not located within a coastal area. Therefore, tsunamis (seismic sea waves) are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

5. SOIL AND GROUNDWATER CONDITIONS

5.1 Artificial Fill

Each of our soil borings encountered undocumented fill materials. The fills were present below the existing pavement sections and extended to depths of approximately 1 ½ to 3 feet below existing grade. The source of the fill materials is unknown. As observed in our soil borings, the fill materials consisted of medium stiff fat clays with minor amounts of fine sand.

5.2 Alluvium

Geologic references map Holocene-age alluvial deposits at the site. As encountered in our borings, the deposits were observed as medium stiff to stiff lean and fat clays with variable amounts of sand. Sands with comparatively minor amounts of clay were encountered at some sample intervals below depths of approximately 18 feet. We encountered alluvium to the maximum depth explored – approximately 60 feet below existing grade.

5.3 Groundwater

Groundwater was encountered at depths of approximately 12 to 13 in our soil borings. Historic high groundwater levels in the immediate site vicinity are on the order of 8 or 9 feet below existing grade based on CGS mapping. Actual groundwater levels will fluctuate seasonally and with variations in rainfall, temperature and other factors and may be higher or lower than observed during our study.

5.4 Soil Corrosion Screening

Soil samples obtained during our field exploration were subjected to laboratory testing for minimum resistivity, pH, and chloride and water-soluble sulfate. The laboratory test results and published screening levels are presented in Appendix B. Soil corrosivity should be considered in the design of buried metal pipes, underground structures, etc.

Water-soluble sulfate test results on selected samples of site soils indicate an S0 exposure classification for sulfate attack on normal portland cement concrete (PCC) as defined in Chapter 318, Table 19.3.1.1 of the ACI *Building Code Requirements for Structural Concrete*. ACI does not set forth requirements for S0 sulfate exposure classification. In addition, neither of the two soil samples tested would be classified as corrosive to buried metal improvements based on Caltrans criteria.

Geocon does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No overriding geotechnical constraints were encountered during our investigation that would preclude the project as presently proposed. Primary geotechnical considerations are the potential for liquefaction-related settlements, the compressibility of the clayey alluvial materials that underlie the site and the expansive nature of near-surface soils. The compressible materials and anticipated foundation loads will likely warrant ground improvement or deep foundation systems to support the data center.

6.1.2 This report identifies foundation types and ground improvement techniques that are considered feasible for the proposed structure and the site soil and geologic conditions. General discussion on each foundation system is presented herein. Other foundation types may also be feasible and the selected foundation type may depend on the acceptability of vibration and noise from foundation construction, the environmental characteristics of the soils that underlie the site or other factors. The disposal of spoils generated by drilled shafts or similar foundation types may not be cost effective. The project team should review the information provided herein and other non-geotechnical factors when selecting foundation type for the project. The design of specialty foundation types should be reviewed by Geocon.

6.1.3 The proposed project redevelops a site with past episodes of grading and construction. As such, unknown underground improvements and areas of undocumented fill materials (not discussed herein) may be present. If encountered, supplemental recommendations will be provided during site development.

6.1.4 As discussed in Section 4.3, the site is susceptible to liquefaction. Our analysis indicates that, if liquefaction and/or cyclic softening were to occur, total ground surface settlements on the order of 1 ½ inch with corresponding differential settlements of up to 1 inch over 40 feet horizontally may result. Ground improvement or a deep foundation system for the project may mitigate the potential effects of liquefaction-induced settlement at the building location. Potential seismically-induced settlements should be considered in the design of exterior improvements and/or improvements not supported by deep foundations.

6.1.5 Where shallow foundation systems are used for ancillary structures such as screen walls in conjunction with the recommended remedial grading, post-construction settlement due to dead + live loads would be on the order of 1 inch or less with differential settlements of approximately ¾ inch across a horizontal distance of 50 feet.

6.1.6 Any changes in the design, location or elevation of the proposed improvements, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

6.1.7 All references to relative compaction and optimum moisture content in this report are based on ASTM D 1557 (latest edition).

6.2 Seismic Design Criteria

6.2.1 The data center will be a seismically isolated structure and seismic design will be per Chapter 17 of ASCE 7-10/16. Ground motion time histories and project seismic design criteria will be provided by others.

6.2.2 Conformance to seismic design criteria does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid structural damage, since such design may be economically prohibitive.

6.3 Soil and Excavation Characteristics

6.3.1 The onsite soils can be excavated with moderate effort using conventional excavation or drilling equipment. We do not anticipate excavations in the native alluvium at the site will generate oversize material (greater than 6 inches in nominal dimension). However, unknown or unanticipated constituents may exist, especially within areas of artificial fill. The artificial fills at the site are undocumented and may contain constituents not reported herein. Below-grade improvements associated with prior site development may also be present.

6.3.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Occupational Safety and Health Administration (OSHA) rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

6.3.3 Some of the alluvial soils encountered at the site are “expansive” as defined by 2016 CBC. The recommendations of this report assume proposed foundation systems will derive support in engineered fills or competent alluvial soils.

6.4 Materials for Fill

6.4.1 Soils generated from cut operations or foundation excavations at the site are suitable for use as engineered fill in structural areas provided they do not contain deleterious matter, organic material, or cementations larger than 6 inches in maximum dimension. Excavated soils may be wet and require drying prior to use and engineered fill.

6.4.2 Import fill material should be primarily granular with a “low” expansion potential (Expansion Index less than 50), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock larger than 6 inches in greatest dimension.

6.4.3 Environmental characteristics and corrosion potential of import soil materials may also be considered. Proposed import materials should be sampled, tested, and approved by Geocon prior to its transportation to the site.

6.5 Grading

6.5.1 All clearing operations and earthwork (including over-excavation, scarification, and recompaction) should be observed and all fills tested for recommended compaction and moisture content by representatives of Geocon.

6.5.2 Structural areas should be considered as areas extending a minimum of 5 feet horizontally from a foundation or beyond the outside dimensions of buildings, including footings and overhangs carrying structural loads, and where not restricted by property boundaries.

6.5.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

6.5.4 After complete demolition and removal of existing structures, site preparation should commence with the removal of all existing improvements from the area to be developed/graded. All active or inactive utilities within the construction area should be protected, relocated, or abandoned. Any pipelines to be abandoned that are greater than 2 inches and less than 18 inches in diameter should be removed or filled with sand-cement slurry. Utilities larger than 18 inches in diameter should be removed. Excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.5.5 Site soils at the proposed substation, areas to receive surface improvements (parking and driveway areas, exterior flatwork areas, etc.) and footing areas for site walls should be over-excavated to a depth of 2 feet below existing grade or to a depth sufficient to remove all artificial fill materials, whichever is deeper. The resultant bottom should then be scarified to a depth of approximately 1 foot, moisture conditioned to at least 2% above optimum moisture and recompacted to at least 88% relative compaction. In general, over-excavated materials may be used for new engineered fill provided they do not contain deleterious matter, organic material, or cementations larger than 6 inches in maximum dimension. Over-excavations and the exposed bottom surfaces and bottom processing should be observed by our representatives. Supplemental recommendations may be provided based on site conditions during grading.

6.5.6 All structural fill and backfill should be placed in layers no thicker than will allow for adequate bonding and compaction (typically 8 to 12 inches). Fill soils should be placed, moisture conditioned to at least 2% above optimum moisture content, and compacted to at least 88% relative compaction (at least 90% and near optimum moisture where fill materials are predominantly sandy). Fill areas with in-place density tests showing moisture contents less than optimum moisture content will require additional moisture conditioning prior to placing additional fill.

6.6 Mat Foundations

6.6.1 A reinforced mat foundation may be used beneath the base isolation system provided column loads are transferred to deep foundation elements or properly supported by a ground improvement system. A mat foundation consists of a thick, rigid concrete mat that allows the entire footprint to carry applied building loads. As such, the mat can tolerate significantly greater differential movements such as those associated with expansive soils or seismically-induced settlement. A mat foundation system will allow the structure to settle with the ground and move as a single unit. Mat thickness and reinforcement should be designed by the project structural engineer.

6.6.2 Consideration should be given to providing six inches of aggregate base (Class 2 Aggregate Base conforming to the latest Caltrans standard specifications or similar) beneath the mat foundation to provide a durable working platform. The aggregate base layer should be compacted to at least 90% relative compaction at near optimum moisture content.

6.6.3 Where the mat foundation is not supported by deep foundation elements or a ground improvement system, we recommended that a modulus of subgrade reaction of 80 pounds per cubic inch be utilized for the design. The modulus of subgrade reaction is based on the square-foot plate load method, and should be adjusted as needed to account for mat size. The modulus should be reduced in accordance with the following equation:

$$K_{(BxB)} = K\left(\frac{1}{B}\right)$$

Where: $K_{(BxB)}$ = subgrade modulus scaled for square mat size

K = unit subgrade modulus

B = foundation width in feet (assumes square mat shape)

If applicable, the reduced modulus of subgrade reaction value calculated above should be adjusted to accounted for the rectangularity of the mat foundation per the following:

$$K_R = \frac{K_{(BxB)}\left(1 + \frac{B}{2L}\right)}{1.5}$$

Where: K_R = reduced subgrade modulus

$K_{(BxB)}$ = subgrade modulus scaled for square mat size

B = mat foundation width in feet

L = mat foundation length in feet

6.6.4 The allowable coefficient of friction to resist sliding is 0.35 for mat foundation concrete atop aggregate materials. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%.

6.6.5 The allowable passive earth pressure for the sides of mat foundations and slabs poured against competent undisturbed alluvial soils may be computed as an equivalent fluid having a density of 200 pounds per cubic foot (pcf) with a maximum earth pressure of 1,500 pounds per square foot. Where not protected by pavement, the upper 1 foot of soils should be ignored when calculating passive resistance.

6.7 Ground Improvement – Rammed Aggregate Piers

6.7.1 The mat foundation for the data center may be supported by Rammed Aggregate Piers (RAPs). RAPs such as Geopier® Foundation Systems are designed and installed by specialty ground improvement contractors. The RAP system is based on soil improvement that consists of installing densified, aggregate columns within drilled shafts. Drilling depths depend on specific site soil conditions but are generally on the order of 10 to 25 feet. Shaft diameters are commonly 30 inches. The system increases density and lateral stress in the surrounding soil and claims improvement in bearing capacity and settlement potential; thus, allowing the use of conventional shallow foundations over the RAP elements. RAPs typically allow the use of increased allowable bearing pressures for foundation design and result in estimated post-construction total and differential settlements of less than 1 inch and ¾ inch, respectively. It should be noted that RAP foundations constructed to the general depths above may not mitigate the seismically-induced settlements to acceptable levels; liquefaction-induced settlements of approximately 1 inch are estimated in site soils below depths of 25 feet at our CPT4 location.

6.7.2 RAP elements are constructed by drilling shafts that are subsequently backfilled with Class 2 aggregate base (AB) in approximate 1-foot lifts. An excavator equipped with a special ramming attachment is used to compact each lift of aggregate. Drill spoils are commonly reused as fill material or exported for offsite disposal. Soil and groundwater conditions at the site may require the use of temporary casing during RAP construction.

6.7.3 If the RAP system is selected for structural support, the RAP specialty contractor would provide a complete design-build submittal with design recommendations, engineered plans and specifications. Geocon will need to perform a geotechnical review of the RAP design.

6.7.4 Geocon should monitor RAP construction. Our Quality Assurance (QA) services will supplement the contractor internal Quality Control (QC) program. Together the QA/QC program will monitor drill depths, shaft length, average lift thicknesses, installation procedures, aggregate quality, and densification of lifts. The allowable vertical capacities should be verified by full-scale modulus and uplift load tests performed on RAP elements. The contractor QC program should document each RAP element installed, which will be reviewed by Geocon.

6.8 Deep Foundations

6.8.1 Deep foundations may be used to support the data center. A variety of foundation types are available and may be feasible for this project. General recommendations for Augercast Pressure-Grout Displacement Piles (APGD) are provided herein as a suggestion only.

6.8.2 APGD piles are installed using a specialized plugged auger that laterally displaces the soil as it is advanced into ground. Once the desired depth is reached, the plug is removed and high-strength grout is pumped under pressure as the auger is withdrawn. After the auger is removed, the required steel reinforcement is then wet-set into the pile to complete the installation. This pile type produces few spoils, approximately 20 percent or less of the theoretical hole volume. Installation difficulties associated with APGD piles can include early refusal in soils that are not displaceable, such as very dense gravels and cemented soils.

6.8.3 APGD piles are typically designed and installed by specialty geotechnical contractors because constructability, installation production, performance and capacity will vary depending on the contractor's equipment, experience, skill, materials and installation procedures. We strongly recommend performing a comprehensive pile installation and load testing program to evaluate constructability as well as capacity. The program should include environmental sampling and analysis of any soil cuttings generated during test pile installation to evaluate offsite disposal options.

6.8.4 The specialty foundation contractor should prepare a complete design-build submittal with design details, calculations, estimated capacities, installation procedures, proposed load testing procedures, acceptance criteria and quality control procedures. Geocon should perform a geotechnical review of the design-build submittal.

6.9 Shallow Foundation Recommendations

6.9.1 Ancillary site structures such as short retaining walls, screen walls, or trash enclosures may utilize conventional foundations consisting of continuous strip footings founded in competent native alluvial materials or properly compacted fill. The following recommendations are based on the assumption that the soils within 5 feet of finish grade will consist of low to moderately expansive materials (Expansion Index less than 90). Over-excavations may be required if soft or loose soils are encountered in footing excavations.

6.9.2 It is recommended that conventional continuous footings have a minimum embedment depth of 18 inches below lowest adjacent pad grade. The footings should be at least 12 inches wide.

6.9.3 Footings proportioned as recommended may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf). The allowable bearing pressure is for dead + live loads may be increased by up to one-third for transient loads due to wind or seismic forces.

6.9.4 The allowable passive pressure used to resist lateral movement of the footings may be assumed to be equal to a fluid weighing 250 pounds per cubic foot (pcf). The allowable coefficient of friction to resist sliding is 0.25 for concrete against soil. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%.

6.9.5 Minimum reinforcement for continuous footings should consist of four No. 5 steel reinforcing bars; two placed near the top of the footing and two near the bottom.

6.9.6 The foundation dimensions and minimum reinforcement recommendations presented herein are based upon soil conditions only and are not intended to be used in lieu of those required for structural purposes.

6.9.7 Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area beneath the footing and within a 1:1 plane extending out and down from the bottom edge of the footing.

6.9.8 The foundation subgrade should be sprinkled as necessary to maintain a moist condition without significant shrinkage cracks as would be expected in any concrete placement. Our representative should observe all footing excavations prior to placing reinforcing steel.

6.10 Temporary Excavations

6.10.1 The native alluvium can be considered a Type B soil in accordance with OSHA guidelines. If free water, sandy or cohesionless soils or undocumented fills are encountered the materials should be downgraded to Type C. The contractor should have a “competent person” as defined by OSHA evaluate all excavations. All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring.

6.10.2 It is the contractor’s responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements which may be damaged by earth movements.

6.11 Underground Utilities

6.11.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than six inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding eight inches and should be compacted to at least 90% relative compaction at least 2% above optimum moisture content (near optimum where backfill materials are predominantly sands and gravels).

6.11.2 Bedding and pipe zone backfill typically extends from the bottom of the trench excavations to a minimum of 6 inches above the crown of the pipe. Pipe bedding material should consist of crushed aggregate, clean sand or similar open-graded material. Proposed bedding and pipe zone materials should be reviewed by Geocon prior to construction; open-graded materials such as ¾ inch drain rock may require wrapping with filter fabric to mitigate the potential for piping. Pipe bedding and backfill should also conform to the requirements of the governing utility agency.

6.12 Exterior Concrete Slabs

6.12.1 Exterior concrete slabs-on-grade subject to vehicle loading are considered pavements should be designed in accordance with the recommendations in Section 6.14 of this report.

6.12.2 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. We recommend that at least 6 inches of Class 2 Aggregate Base (AB) compacted to at least 95% relative compaction be used below exterior concrete slabs. Prior to placing AB, the subgrade should be moisture conditioned to at least 2% over optimum and properly compacted to at least 88% relative compaction.

6.12.3 In lieu of specific recommendations from the structural or civil engineer, we recommend that crack control joints be spaced at intervals not greater than 8 feet for 4-inch-thick slabs. Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. Construction joints should be designed by the project structural engineer.

6.12.4 The recommendations of this report are intended to reduce the potential for cracking of slabs due to soil movement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to soil movement. This is common for project areas that contain expansive soils since designing to eliminate potential soil movement is cost prohibitive. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.13 Moisture Protection Considerations

6.13.1 A vapor barrier is not required beneath mat foundations for geotechnical purposes. Further, the migration of moisture through concrete slabs or moisture otherwise released from slabs is not a geotechnical issue. However, for the convenience of the owner, we are providing the following general suggestions for consideration by the owner, architect, structural engineer, and contractor.

The suggested procedures may reduce the potential for moisture-related floor covering failures on concrete slabs-on-grade, but moisture problems may still occur even if the procedures are followed. If more detailed recommendations are desired, we recommend consulting a specialist in this field. If a vapor barrier is used beneath mat slab foundations, friction between the mat slab and underlying substrate when evaluating lateral loading resistance.

6.13.2 A vapor barrier meeting ASTM E 1745-09 Class C requirements may be placed directly below the slab, without a sand cushion. To reduce the potential for punctures, a higher quality vapor barrier (15 mil, Class A or B) should be used. The vapor barrier, if used, should extend to the edges of the slab, and should be sealed at all seams and penetrations.

6.13.3 The concrete water/cement ratio should be as low as possible. The water/cement ratio should not exceed 0.45 for concrete placed directly on the vapor barrier. Midrange plasticizers could be used to facilitate concrete placement and workability.

6.13.4 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

6.14 Pavement Recommendations

6.14.1 The upper 12 inches of pavement subgrade should be scarified, moisture conditioned to at least 2% over optimum and compacted to at least 92% relative compaction. Prior to placing aggregate base, the finished subgrade should be proof-rolled with a laden water truck (or similar equipment with high contact pressure) to verify stability.

6.14.2 Sidewalk, curb, gutter, and driveway encroachments should be designed and constructed in accordance with City of Santa Clara requirements, as applicable.

6.14.3 We recommend the following asphalt concrete (AC) pavement sections for design to establish subgrade elevations in pavement areas. The project civil engineer should determine the appropriate Traffic Index (TI) based on anticipated traffic conditions. The flexible pavement sections below are based on estimated design TIs and an R-Value of 5 for the subgrade soils. We can provide additional sections based on other TIs if necessary.

TABLE 6.14

FLEXIBLE PAVEMENT SECTION RECOMMENDATIONS

Location	Estimated Traffic Index (TI)	AC Thickness (inches)	Class 2 AB Thickness (inches)
Parking Stalls	4.5	3	8
Driveways	6.0	3½	12½
Heavy-Duty	7.0	4	15½

Note: The recommended flexible pavement sections are based on the following assumptions:

1. AB: Class 2 AB with a minimum R-Value of 78 and meeting the requirements of Section 26 of the latest Caltrans Standard Specifications.
2. AB is compacted to 95% or higher relative compaction at or near optimum moisture content. Prior to placing AB, the subgrade should be proof-rolled with a loaded water truck to verify stability.

3. AC: Asphalt concrete conforming to local agency standards or Section 39 of the latest Caltrans Standard Specifications.

6.14.4 The AC sections in Table 6.14 are final, minimum thicknesses. If staged-pavements are used, the construction bottom AC lift should be at least 2 inches thick. Following construction, the finish top AC lift should be at least 1½ inches thick.

6.14.5 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, we recommend the concrete be a minimum of 6 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. In addition, doweling, reinforcing steel or other load-transfer mechanism should be provided at joints if desired to reduce the potential for vertical offset. The concrete should have a minimum 28-day compressive strength of 3,500 psi. We should evaluate pavements to support heavy truck traffic on a case-by-case basis; supplemental recommendations may be provided.

6.14.6 We recommend that at least 6 inches of Class 2 Aggregate Base be used below rigid concrete pavements. The aggregate base should be compacted to at least 95% relative compaction near optimum moisture content.

6.14.7 In general, we recommend that concrete pavements be designed, constructed and maintained in accordance with industry standards such as those provided by the American Concrete Pavement Association.

6.14.8 Crack control joints should be spaced at intervals not greater than 12 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. Construction joints should be designed by the project structural engineer.

6.14.9 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 6 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving. Alternatives such as plastic moisture cut-offs or modified drop-inlets may also be considered in lieu of deepened curbs.

6.15 Retaining Wall Design

6.15.1 Lateral earth pressures may be used in the design of retaining walls and buried structures. Lateral earth pressures against these facilities may be assumed to be equal to the pressure exerted by an equivalent fluid. The unit weight of the equivalent fluid depends on the design conditions. Table 6.15 summarizes the weights of the equivalent fluid based on the different design conditions.

TABLE 6.15
RECOMMENDED LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density
Active	50 pcf
At-Rest	70 pcf

6.15.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.001H (where H is the height of the wall). Walls restrained from movement such as basement walls should be designed using the at-rest case. The above soil pressures assume level backfill under drained conditions within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall and no surcharges within that same area. Where restrained basement walls will be undrained, an at-rest equivalent fluid density of 100 pounds per cubic foot (pcf) should be used for retaining wall design.

6.15.3 Unless project-specific loading information is provided by the structural engineer, where vehicle loads are expected atop the wall backfill, an additional uniform surcharge pressure equivalent to 2 feet of backfill soil should be used for design. Where the vehicle loading will be limited to passenger cars, the additional uniform surcharge equivalent may be reduced to 1 foot of backfill soil.

6.15.4 Retaining walls greater than 2 feet tall (retained height) should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. Positive drainage for retaining walls should consist of a vertical layer of permeable material positioned between the retaining wall and the soil backfill. The permeable material may be composed of a composite drainage geosynthetic or a natural permeable material such as crushed gravel at least 12 inches thick and capped with at least 12 inches of native soil. A geosynthetic filter fabric should be placed between the gravel and the soil backfill. Provisions for removal of collected water should be provided for either system by installing a perforated drainage pipe along the bottom of the permeable material which leads to suitable drainage facilities.

6.15.5 We recommend that all retaining wall designs be reviewed by Geocon to confirm the incorporation of the recommendations provided herein. In particular, potential surcharges from adjacent structures and other improvements should be reviewed by Geocon.

6.16 Surface Drainage

6.16.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change to important engineering properties. Proper drainage should be maintained at all times.

6.16.2 All site drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundations or retaining walls. Drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structures should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers not permitted onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed or properly drained to prevent moisture intrusion into the materials providing foundation support. Landscape irrigation within five feet of the building perimeter footings should be kept to a minimum to just support vegetative life.

6.16.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond. Final soil grade should slope a minimum of 2% away from structures.

6.16.4 We recommend implemented measures to reduce infiltrating surface water near buildings and slabs-on-grade. Such measures may include:

- Selecting drought-tolerant plants that require little or no irrigation, especially within 5 feet of buildings, slabs-on-grade, or pavements.
- Using drip irrigation or low-output sprinklers.
- Using automatic timers for irrigation systems.
- Appropriately spaced area drains.
- Hard-piping roof downspouts to appropriate collection facilities.

7. FURTHER GEOTECHNICAL SERVICES

7.1 Plan and Specification Review

7.1.1 We should review project plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

7.2 Testing and Observation Services

7.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase and provide compaction testing and observation services and foundation observations throughout the project. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for others interpretation of our recommendations, and therefore the future performance of the project.

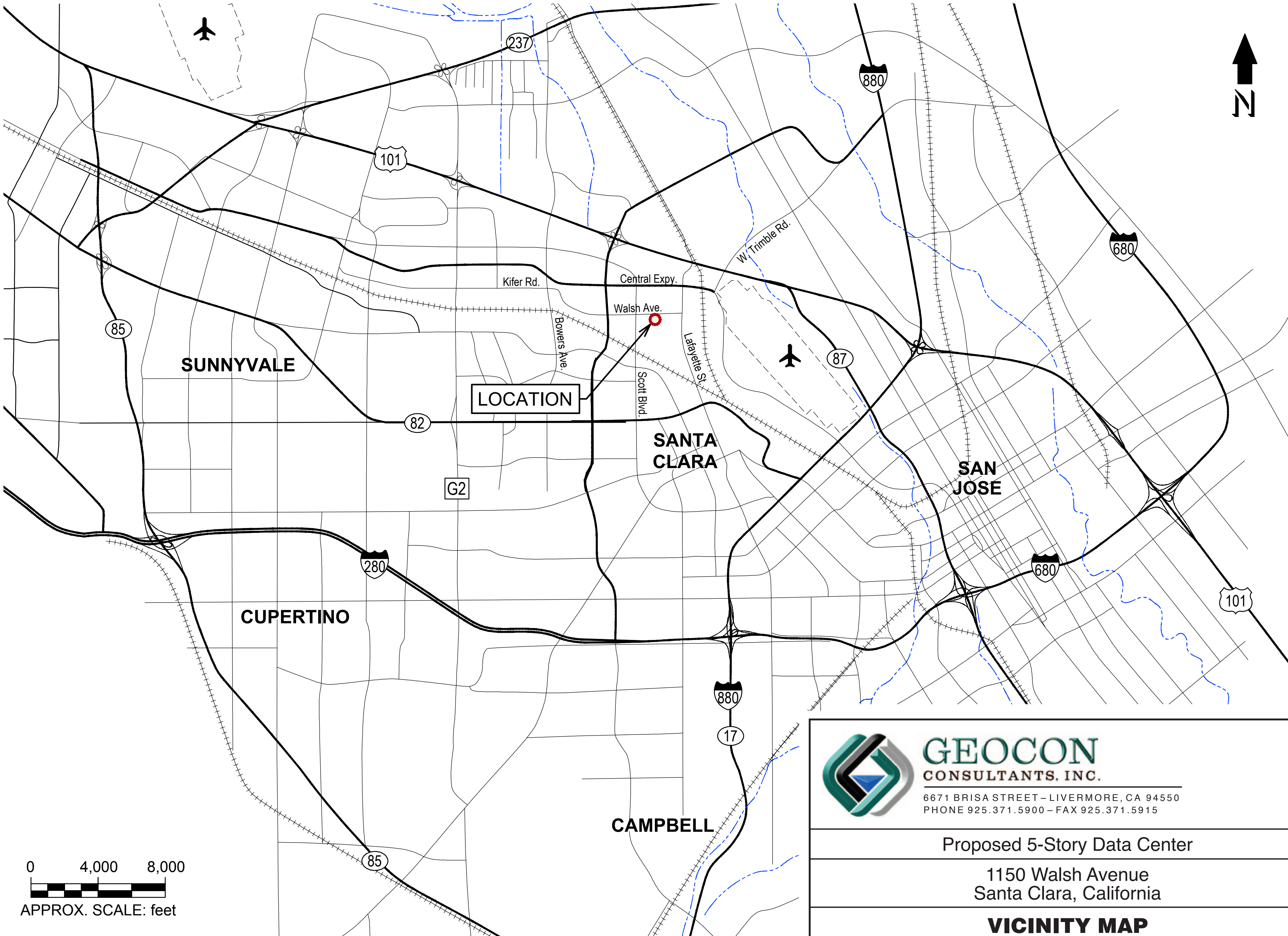
LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Consultants, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the geotechnical scope of services provided by Geocon Consultants, Inc.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in the site area at this time. No warranty is provided, express or implied.



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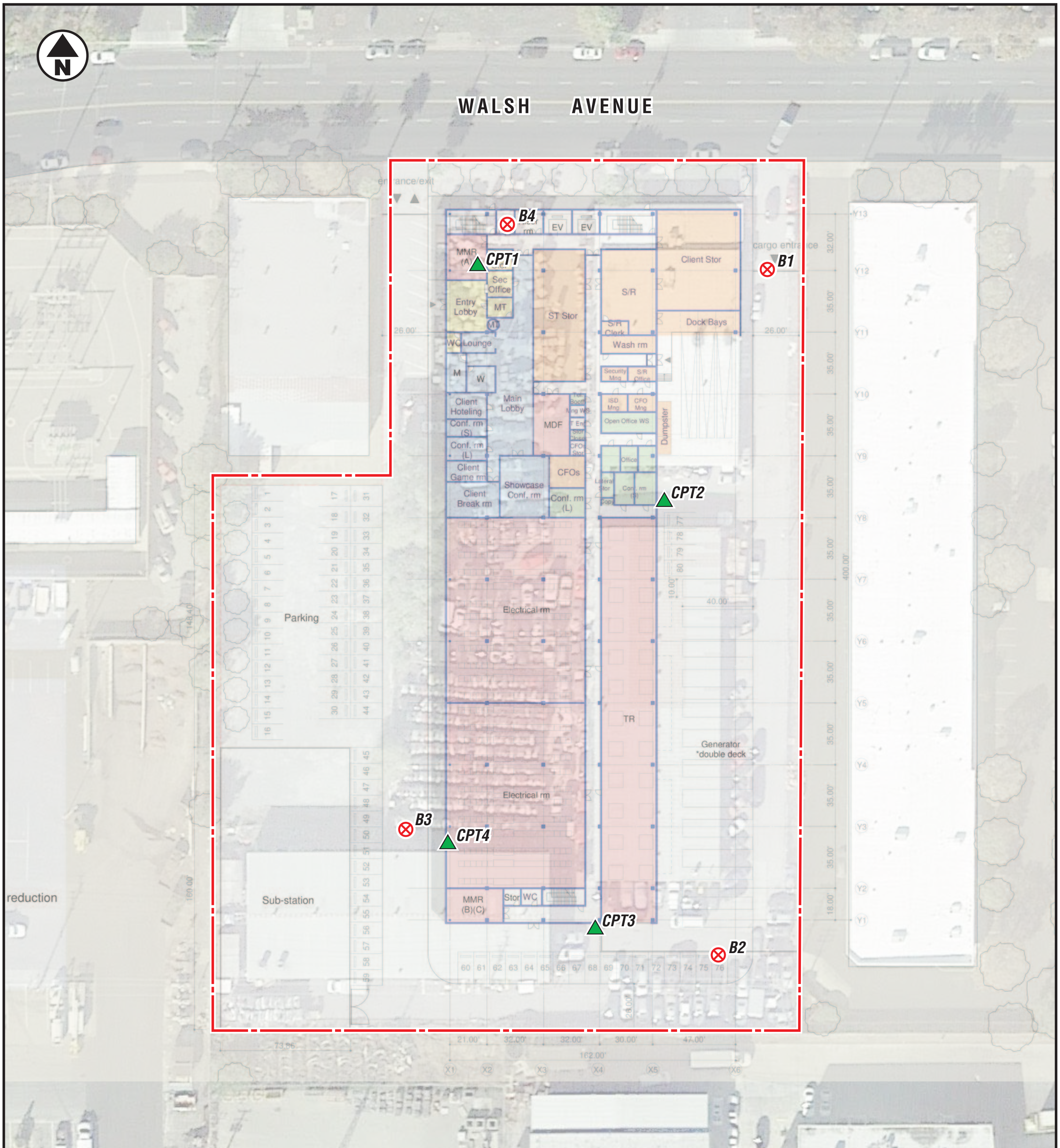
Proposed 5-Story Data Center

1150 Walsh Avenue
Santa Clara, California

VICINITY MAP

E9008-04-03	November 2017	Figure 1



LEGEND:

B4 ⊗ Approximate Boring Location

CPT4 ▲ Approximate CPT Location

0 80
Scale in Feet

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SITE PLAN

E9008-04-03	November 2017	Figure 2

APPENDIX A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation included a site visit, subsurface exploration, and soil sampling. The locations of our exploratory borings and CPTs are shown on the Site Plan, Figure 2. Soil boring logs and CPT profiles for our exploration are presented as figures following the text in this appendix. The borings and CPTs were located by pacing from existing reference points. Therefore, the exploration locations shown on Figure 2 are approximate.

Our field exploration included the advancement of four CPT soundings to maximum depths of approximately 60 feet below the existing ground surface utilizing a truck-mounted CPT rig with a down-pressure capacity of approximately 20 tons. The CPTs were performed on September 1, 2017 by Middle Earth Geo Testing of Fremont, California using an integrated electronic cone system. The cone has a tip area of 10 square centimeters, a friction sleeve area of 150 square centimeters, and a ratio of friction sleeve area to tip end area equal to 0.85. The cone bearing (Q_c) and sleeve friction (F_s) were measured and recorded during tests at approximately 2-inch depth intervals. The CPT data consisting of cone bearing, sleeve friction, friction ratio and equivalent standard penetration blow counts (N) versus penetration depth below the existing ground surface for each location has been recorded and is presented in this appendix.

Our borings were performed on September 13, 2017 using a truck-mounted Mobile B-53 drill rig equipped with 8-inch hollow-stem augers. Sampling in the borings was accomplished using a down-hole wire-line 140-pound hammer with a 30-inch drop. Samples were obtained with a 3-inch outside-diameter (OD), split spoon (California Modified) sampler, and a 2-inch OD, Standard Penetration Test (SPT) sampler. The number of blows required to drive the sampler the last 12 inches (or fraction thereof) of the 18-inch sampling interval were recorded on the boring logs. The blow counts shown on the boring logs should not be interpreted as standard SPT "N" values; corrections have not been applied.

Subsurface conditions encountered in the exploratory boring were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488). This system uses the Unified Soil Classification System (USCS) for soil designations. The log depicts soil and geologic conditions encountered and depths at which samples were obtained. The log also includes our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, drill rig penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing.

Upon completion, our hollow-stem auger borings were backfilled with lean concrete grout. Our CPTs were backfilled in accordance with Santa Clara Valley Water District permit requirements.

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO.4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC	CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO.4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL	ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OR CLAYS OF MEDIUM TO HIGH PLASTICITY
	HIGHLY ORGANIC SOILS		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

BORING/TRENCH LOG LEGEND

- No Recovery
- Shelby Tube Sample
- Bulk Sample
- SPT Sample
- Modified California Sample
- Groundwater Level (At Completion)
- Groundwater Level (Seepage)

PENETRATION RESISTANCE

SAND AND GRAVEL			SILT AND CLAY			
RELATIVE DENSITY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	CONSISTENCY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	COMPRESSIVE STRENGTH (tsf)
VERY LOOSE	0 - 4	0 - 6	VERY SOFT	0 - 2	0 - 3	0 - 0.25
LOOSE	5 - 10	7 - 16	SOFT	3 - 4	4 - 6	0.25 - 0.50
MEDIUM DENSE	11 - 30	17 - 48	MEDIUM STIFF	5 - 8	7 - 13	0.50 - 1.0
DENSE	31 - 50	49 - 79	STIFF	9 - 15	14 - 24	1.0 - 2.0
VERY DENSE	OVER 50	OVER 79	VERY STIFF	16 - 30	25 - 48	2.0 - 4.0
			HARD	OVER 30	OVER 48	OVER 4.0

*NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE LAST 12 INCHES OF AN 18-INCH DRIVE

MOISTURE DESCRIPTIONS

FIELD TEST	APPROX. DEGREE OF SATURATION, S (%)	DESCRIPTION
NO INDICATION OF MOISTURE; DRY TO THE TOUCH	$S<25$	DRY
SLIGHT INDICATION OF MOISTURE	$25\leq S<50$	DAMP
INDICATION OF MOISTURE; NO VISIBLE WATER	$50\leq S<75$	MOIST
MINOR VISIBLE FREE WATER	$75\leq S<100$	WET
VISIBLE FREE WATER	100	SATURATED

QUANTITY DESCRIPTIONS

APPROX. ESTIMATED PERCENT	DESCRIPTION
<5%	TRACE
5 - 10%	FEW
11 - 25%	LITTLE
26 - 50%	SOME
>50%	MOSTLY

GRAVEL/COBBLE/BOULDER DESCRIPTIONS

CRITERIA	DESCRIPTION
PASS THROUGH A 3-INCH SIEVE AND BE RETAINED ON A NO. 4 SIEVE (#4 TO 3")	GRAVEL
PASS A 12-INCH SQUARE OPENING AND BE RETAINED ON A 3-INCH SIEVE (3"-12")	COBBLE
WILL NOT PASS A 12-INCH SQUARE OPENING (>12")	BOULDER

BEDDING SPACING DESCRIPTIONS

THICKNESS/SPACING	DESCRIPTOR
GREATER THAN 10 FEET	MASSIVE
3 TO 10 FEET	VERY THICKLY BEDDED
1 TO 3 FEET	THICKLY BEDDED
3 5/8-INCH TO 1 FOOT	MODERATELY BEDDED
1 1/4-INCH TO 3 5/8-INCH	THINLY BEDDED
3/8-INCH TO 1 1/4-INCH	VERY THINLY BEDDED
LESS THAN 3/8-INCH	LAMINATED

STRUCTURE DESCRIPTIONS

CRITERIA	DESCRIPTION
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS AT LEAST 1/4-INCH THICK	STRATIFIED
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS LESS THAN 1/4-INCH THICK	LAMINATED
BREAKS ALONG DEFINITE PLANES OF FRACTURE WITH LITTLE RESISTANCE TO FRACTURING	FISSURED
FRACTURE PLANES APPEAR POLISHED OR GLOSSY, SOMETIMES STRIATED	SLICKENSIDED
COHESIVE SOIL THAT CAN BE BROKEN DOWN INTO SMALLER ANGULAR LUMPS WHICH RESIST FURTHER BREAKDOWN	BLOCKY
INCLUSION OF SMALL POCKETS OF DIFFERENT SOIL, SUCH AS SMALL LENSES OF SAND SCATTERED THROUGH A MASS OF CLAY	LENSED
SAME COLOR AND MATERIAL THROUGHOUT	HOMOGENOUS

CEMENTATION/INDURATION DESCRIPTIONS

FIELD TEST	DESCRIPTION
CRUMBLES OR BREAKS WITH HANDLING OR LITTLE FINGER PRESSURE	WEAKLY CEMENTED/INDURATED
CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE	MODERATELY CEMENTED/INDURATED
WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE	STRONGLY CEMENTED/INDURATED

IGNEOUS/METAMORPHIC ROCK STRENGTH DESCRIPTIONS

FIELD TEST	DESCRIPTION
MATERIAL CRUMBLES WITH BARE HAND	WEAK
MATERIAL CRUMBLES UNDER BLOWS FROM GEOLOGY HAMMER	MODERATELY WEAK
1/8-INCH INDENTATIONS WITH SHARP END FROM GEOLOGY HAMMER	MODERATELY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **ONE** BLOW FROM GEOLOGY HAMMER	STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **COUPLE** BLOWS FROM GEOLOGY HAMMER	VERY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **MANY** BLOWS FROM GEOLOGY HAMMER	EXTREMELY STRONG

IGNEOUS/METAMORPHIC ROCK WEATHERING DESCRIPTIONS

DEGREE OF DECOMPOSITION	FIELD RECOGNITION	ENGINEERING PROPERTIES
SOIL	DISCOLORED, CHANGED TO SOIL, FABRIC DESTROYED	EASY TO DIG
COMPLETELY WEATHERED	DISCOLORED, CHANGED TO SOIL, FABRIC MAINLY PRESERVED	EXCAVATED BY HAND OR RIPPING (Saprolite)
HIGHLY WEATHERED	DISCOLORED, HIGHLY FRACTURED, FABRIC ALTERED AROUND FRACTURES	EXCAVATED BY HAND OR RIPPING, WITH SLIGHT DIFFICULTY
MODERATELY WEATHERED	DISCOLORED, FRACTURES, INTACT ROCK-NOTICEABLY WEAKER THAN FRESH ROCK	EXCAVATED WITH DIFFICULTY WITHOUT EXPLOSIVES
SLIGHTLY WEATHERED	MAY BE DISCOLORED, SOME FRACTURES, INTACT ROCK-NOT NOTICEABLY WEAKER THAN FRESH ROCK	REQUIRES EXPLOSIVES FOR EXCAVATION, WITH PERMEABLE JOINTS AND FRACTURES
FRESH	NO DISCOLORATION, OR LOSS OF STRENGTH	REQUIRES EXPLOSIVES

IGNEOUS/METAMORPHIC ROCK JOINT/FRACTURE DESCRIPTIONS

FIELD TEST	DESCRIPTION
NO OBSERVED FRACTURES	UNFRACTURED/UNJOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1 TO 3 FOOT INTERVALS	SLIGHTLY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 4-INCH TO 1 FOOT INTERVALS	MODERATELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1-INCH TO 4-INCH INTERVALS WITH SCATTERED FRAGMENTED INTERVALS	INTENSELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT LESS THAN 1-INCH INTERVALS; MOSTLY RECOVERED AS CHIPS AND FRAGMENTS	VERY INTENSELY FRACTURED/JOINTED

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KEY TO LOGS

PROJECT NO. **E9008-04-03** PROJECT NAME: **RagingWire Santa Clara**

BORING B1

ELEV. (MSL.) ____ DATE COMPLETED **9/13/2017**
ENG./GEO. **JBM** DRILLER **EGI**
EQUIPMENT **Mobile B53 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					Approximately 3½ inches AC			
1	B1-1-5			CH	Approximately 6 inches AB			
2					FILL			
3	B1-2.5-3 B1-3				Medium stiff, moist, black, CLAY with few (f) sands -pp=2-2½	11	89.1	31.4
4	B1-4-4.5 B1-4.5			CH	ALLUVIUM Stiff, moist, brownish-gray streaked white, CLAY with few (f) sands	14	89.5	29.6
5					-pp=2¾-3½ -brownish-gray			
6								
7								
8								
9	B1-9-9.5 B1-9.5				-medium stiff, gray, more sand (f) -pp=1¾-2½	9	102.1	23.9
10								
11								
12			▼					
13								
14	B1-14-14.5 B1-14.5				-gray mottled brown, moist to wet, sand (f-m) -pp=1½-2¼	11		
15								
16								
17								
18								
19	B1-19-19.5 B1-19.5				-blue-gray, more sand (f) -pp=1	9		
20								
21								
22								
23								
24	B1-24-25				-stiff, wet -no recovery with Modified California sampler, SPT sampler used to recover sample	17		
25				SW-SC	Medium dense, wet, brown to gray, (f-c) SAND with little (f) gravel and trace clays			
26								
27								
28								
29	B1-29-30					12		40.7

Figure A2, Log of Boring B1, page 1 of 2

GEOCON BORING LOG E9008-04-03 BORING LOGS.GPJ 10/31/17

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

BORING B1

ELEV. (MSL.) ______ DATE COMPLETED 9/13/2017

ENG./GEO. JBM DRILLER EGI

EQUIPMENT Mobile B53 w/ 8-inch HSA HAMMER TYPE Downhole-Wireline

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30				CL	Stiff, moist, dark gray, CLAY with few (f) sands			
31								
32								
33								
34	B1-34-34.5 B1-34.5				-pp=2-2½	16		
35								
36								
37								
38								
39	B1-39-39.5 B1-39.5				-hard, more sand	55		20.6
40				SP	Dense, moist, dark gray, (f) SAND			
41								
42								
43								
44	B1-44-45			CL	Very stiff, moist, dark gray, (f) Sandy CLAY	18		
45								

END OF BORING AT APPROXIMATELY 45 FEET
GROUNDWATER INITIALLY ENCOUNTERED AT APPROXIMATELY 12 FEET
BACKFILLED WITH GROUT AND CAPPED WITH CONCRETE

Figure A2, Log of Boring B1, page 2 of 2

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING B2

ELEV. (MSL.) ________ DATE COMPLETED **9/13/2017**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B53 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				Approximately 2 inches AC			
1			CH	Approximately 4 inches AB			
2			CH	FILL Medium stiff, moist, black, CLAY with few (f) sands			
	B2-2.5-3				12		
3	B2-3			ALLUVIUM Medium stiff, moist, gray mottled brown, CLAY with few (f) sands		88.8	30.0
4	B2-4-4.5			-pp=3½-4	17		
	B2-4.5			-stiff, black -pp=3½		95.5	25.5
5							
6							
7							
8							
9	B2-9-9.5			-medium stiff, gray	8		
	B2-9.5			-pp=1¾-2			
10							
11							
12							
13		▼					
14	B2-14-14.5			-brown	9		
	B2-14.5			-pp=1½-1¾			
15							
16							
17							
18							
19	B2-19-19.5			-stiff, more sand	14		
	B2-19.5						
20							

END OF BORING AT APPROXIMATELY 20 FEET
GROUNDWATER INITIALLY ENCOUNTERED AT APPROXIMATELY 13 FEET
BACKFILLED WITH GROUT AND CAPPED WITH CONCRETE

GEOCON BORING LOG E9008-04-03 BORING LOGS.GPJ 10/31/17

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B3

ELEV. (MSL.) ______ DATE COMPLETED **9/13/2017**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B53 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CH	FILL Medium stiff, damp, black, CLAY with few (f) sands			
2	B3-2.5-3 B3-3			CH	ALLUVIUM Medium stiff, damp to moist, light gray, CLAY with silt and few (f) sands -pp=4-4½	13		
4	B3-4-4.5 B3-4.5				-stiff, brownish-gray, less silt -pp=4-4½	15	97.9	25.1
9	B3-9-9.5 B3-9.5				-medium stiff, moist, greenish-gray -pp=2¾-3	12	106.3	21.3
13			▼					
14	B3-14-14.5 B3-14.5				-brown mottled gray -pp=2	9		
19	B3-19-20			SW	Medium dense, wet, brown, (f-c) SAND with few (f) rounded gravels -sand (f-m), less gravels	15		
24	B3-24-24.5 B3-24.5-25			SW-SC	Medium dense, wet, (f-c) SAND with few clays	20		17.2
25				CL	Stiff, moist, gray, CLAY with (f) sand			
29	B3-29-29.5 B3-29.5				-medium stiff -pp=2	13		

Figure A4, Log of Boring B3, page 1 of 2

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING B3

ELEV. (MSL.) ____________ DATE COMPLETED **9/13/2017**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B53 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30								
31								
32								
33								
34	B3-34-34.5 B3-34.5			SC	Medium dense, moist to wet, (f-m) SAND with clay	22		42.9
35								
36								
37								
38								
39	B3-39-40			SW	Very dense, moist to wet, brown, (f-c) SAND with few (f) rounded gravels and trace clay	73		
40								
41								
42								
43								
44	B3-44-44.5					50/5"		

END OF BORING AT APPROXIMATELY 44½ FEET
GROUNDWATER INITIALLY ENCOUNTERED AT APPROXIMATELY 13 FEET
BACKFILLED WITH GROUT AND CAPPED WITH CONCRETE

Figure A4, Log of Boring B3, page 2 of 2

GEOCON BORING LOG E9008-04-03 BORING LOGS.GPJ 10/31/17

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. **E9008-04-03** PROJECT NAME: **RagingWire Santa Clara**

BORING B4

ELEV. (MSL.)		DATE COMPLETED	**9/13/2017**
ENG./GEO.	**JBM**	DRILLER	**EGI**
EQUIPMENT	**Mobile B53 w/ 8-inch HSA**	HAMMER TYPE	**Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					Approximately 2 inches AC			
1				CH	Approximately 5 inches AB			
2				CH	FILL Medium stiff, moist, black, CLAY wiht few (f) sands			
3	B4-2.5-3 B4-3				ALLUVIUM Stiff, moist, gray mottled brown, CLAY with few (f) sands	15	95.4	26.4
4	B4-4-4.5 B4-4.5				-pp=2½-3¼ -medium stiff, brown -pp=2½-3¼	13	101.4	23.2
5								
6								
7								
8								
9	B4-9-9.5 B4-9.5				-gray mottled brown -pp=2½	11		
10								
11								
12								
13			▼					
14	B4-14-14.5 B4-14.5				-tan to light brown	12		
15								
16								
17								
18								
19	B4-19-19.5 B4-19.5				-stiff, brown -pp=1½-2¼ -dark gray	16		
20								

END OF BORING AT APPROXIMATELY 20 FEET
GROUNDWATER INITIALLY ENCOUNTERED AT APPROXIMATELY 13 FEET
BACKFILLED WITH GROUT AND CAPPED WITH CONCRETE

Figure A5, Log of Boring B4, page 1 of 1

GEOCON BORING LOG E9008-04-03 BORING LOGS.GPJ 10/31/17

GEOCON

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

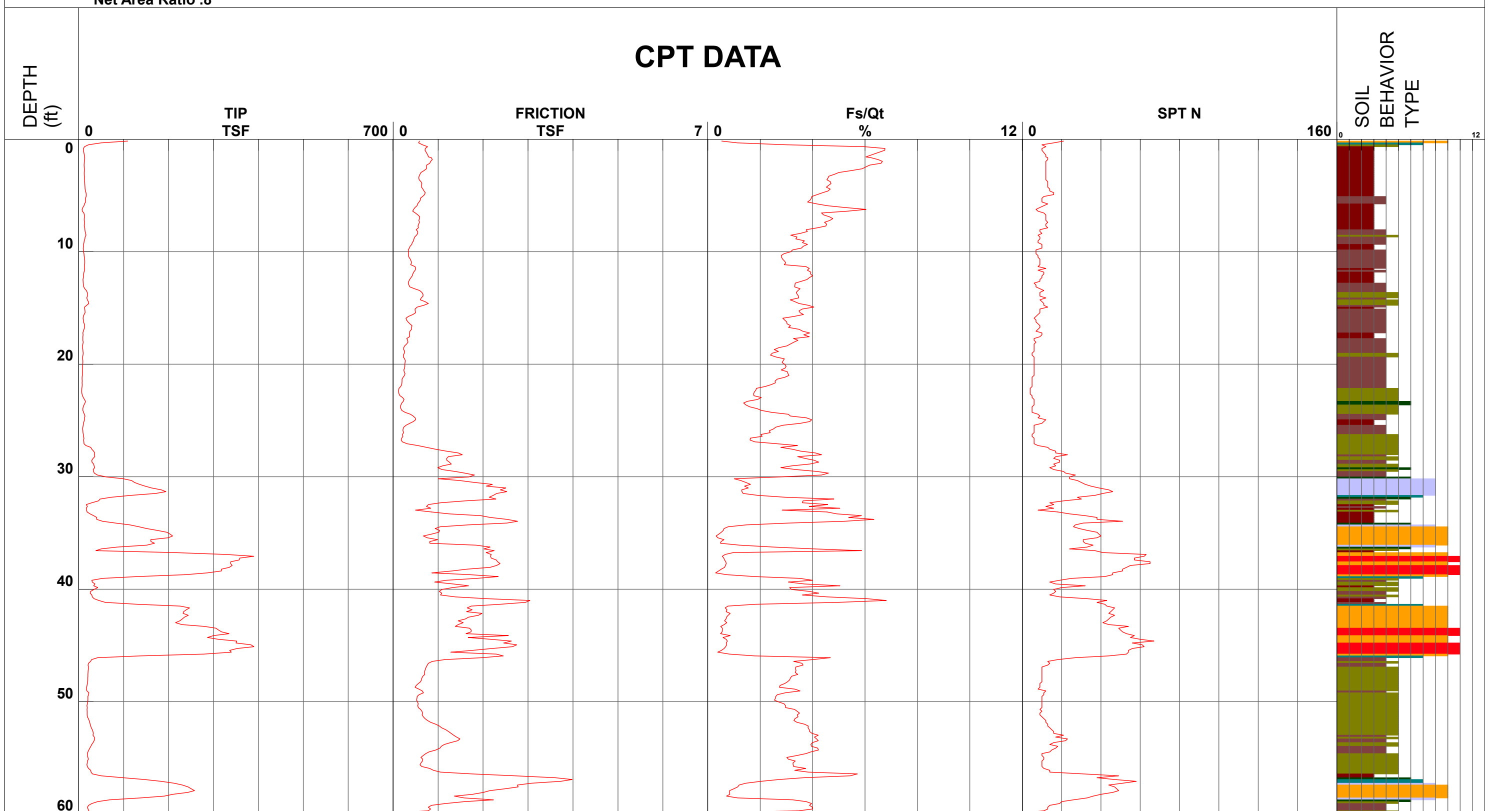
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Geocon Inc

Project	Raging Wire Santa Clara	Operator	KK RB	Filename	SDF(068).cpt
Job Number	E9008-04-03	Cone Number	DDG1333	GPS	
Hole Number	CPT-01	Date and Time	9/1/2017 10:33:57 AM	Maximum Depth	60.04 ft
EST GW Depth During Test		2.20 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

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CONE PENETROMETER TEST CPT1

Project: RagingWire Santa Clara
Project No. E9008-04-03
Date: November 2017

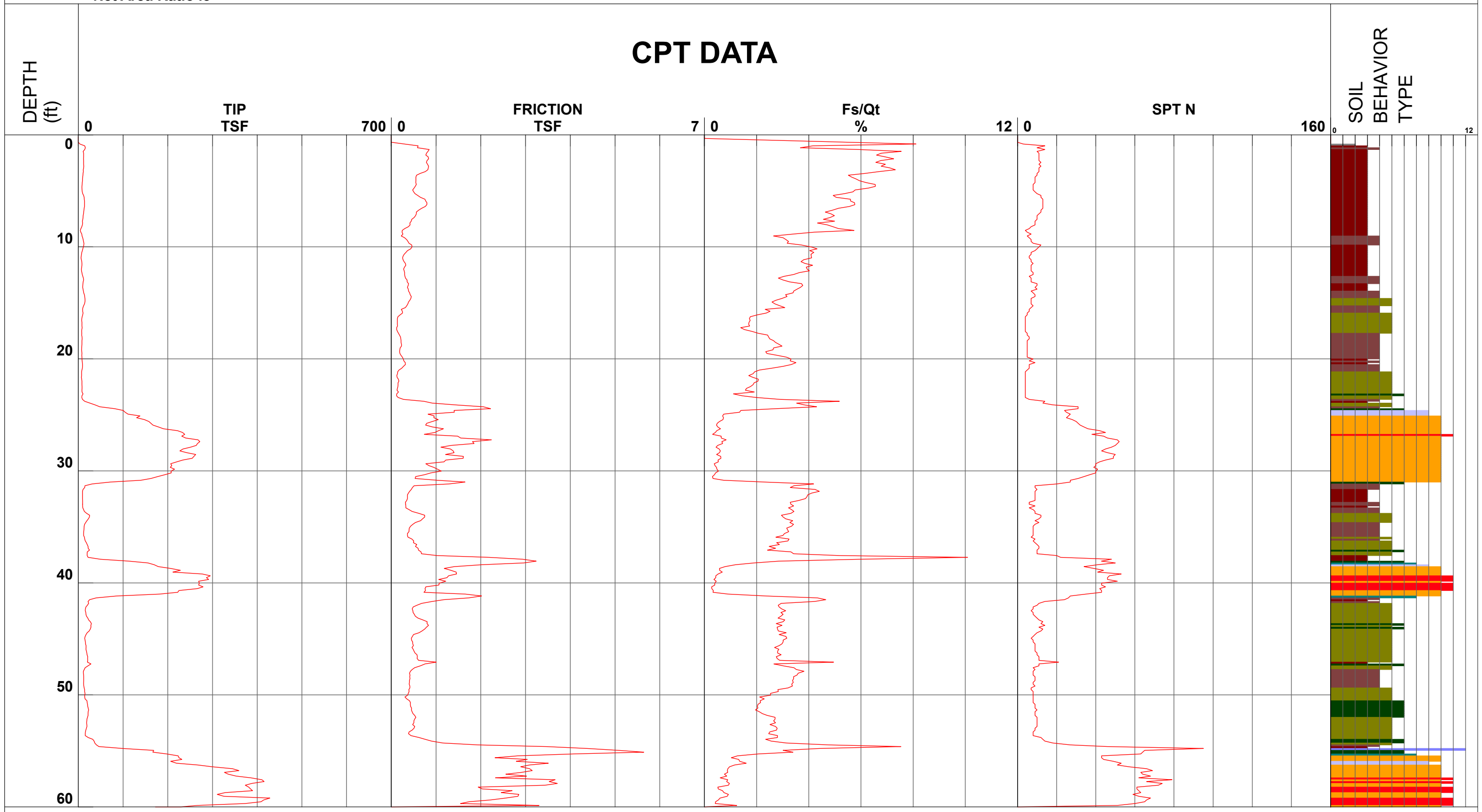
FIGURE A6

Geocon Inc

Project	Raging Wire Santa Clara	Operator	KK RB	Filename	SDF(067).cpt
Job Number	E9008-04-03	Cone Number	DDG1333	GPS	
Hole Number	CPT-02	Date and Time	9/1/2017 9:44:32 AM	Maximum Depth	60.20 ft
EST GW Depth During Test	7.20 ft				

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

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CONE PENETROMETER TEST CPT2

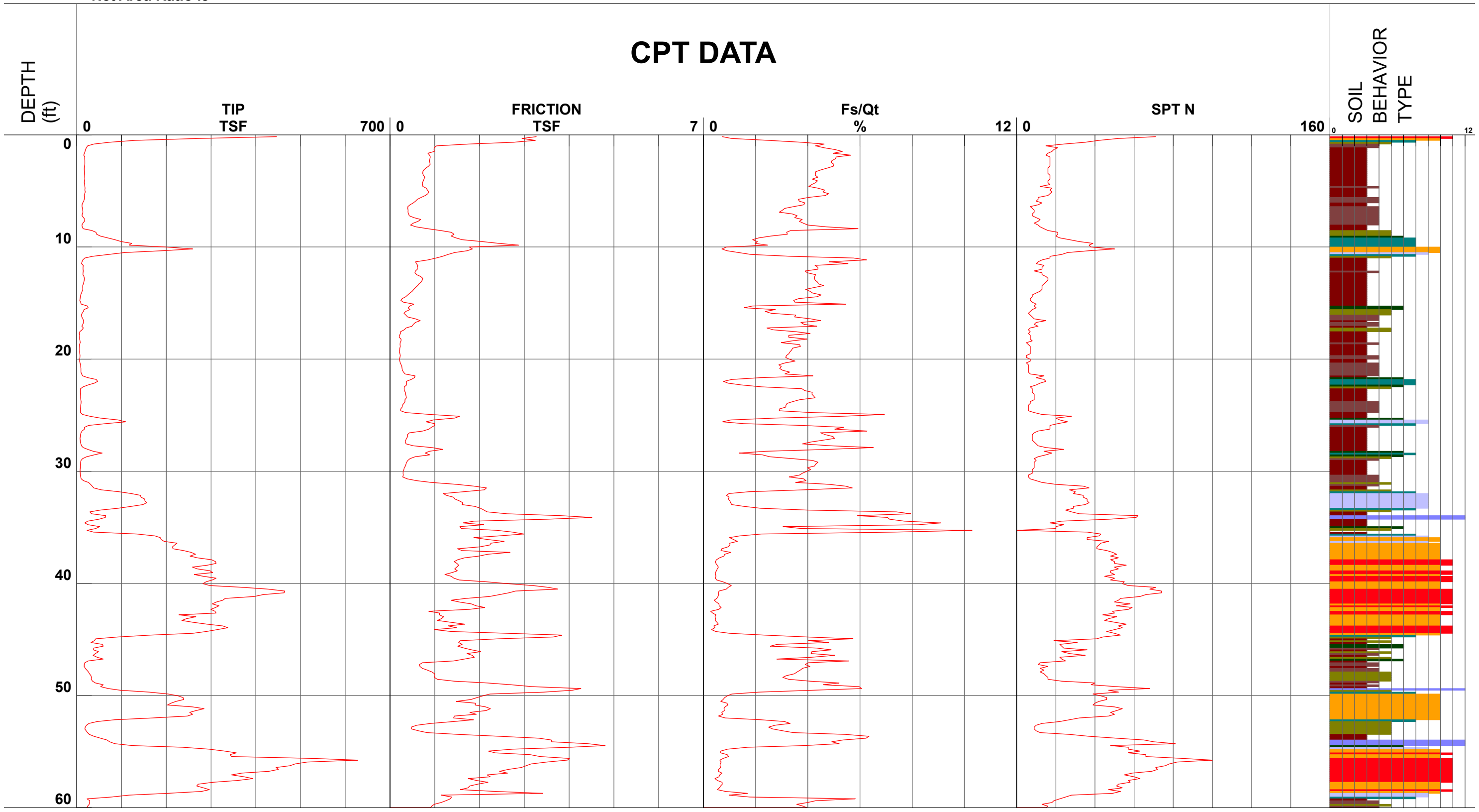
Project: RagingWire Santa Clara
Project No. E9008-04-03
Date: November 2017

FIGURE A7

Project	Raging Wire Santa Clara	Operator	KK RB	Filename	SDF(066).cpt
Job Number	E9008-04-03	Cone Number	DDG1333	GPS	
Hole Number	CPT-03	Date and Time	9/1/2017 8:38:19 AM	Maximum Depth	60.37 ft
EST GW Depth During Test		7.20 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

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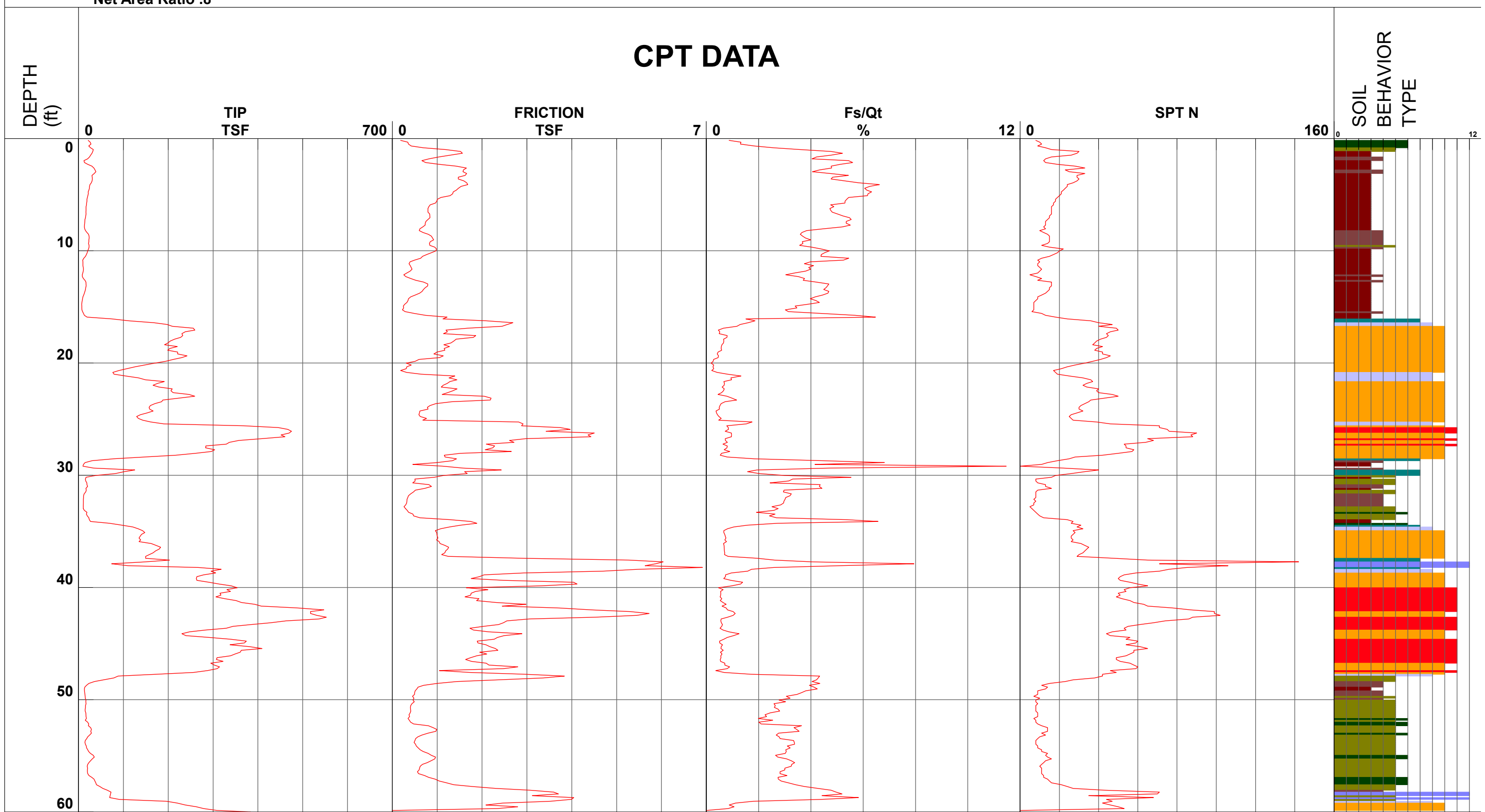
CONE PENETROMETER TEST CPT3

Project: RagingWire Santa Clara
Project No. E9008-04-03
Date: November 2017

FIGURE A8

Project	Raging Wire Santa Clara	Operator	KK RB	Filename	SDF(065).cpt
Job Number	E9008-04-03	Cone Number	DDG1333	GPS	
Hole Number	CPT-04	Date and Time	9/1/2017 7:52:03 AM	Maximum Depth	60.04 ft
EST GW Depth During Test		9.30 ft			

Net Area Ratio .8



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

Cone Size 10cm squared

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

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CONE PENETROMETER TEST CPT4

Project: RagingWire Santa Clara
Project No. E9008-04-03
Date: November 2017

FIGURE A9

APPENDIX B

APPENDIX B
LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for in-situ dry density and moisture content, grain size distribution, consolidation, Atterberg Limits, expansion potential, unconfined compressive strength and screening-level corrosion parameters. The results of our testing are summarized in tabular format below and the following figures. In-situ dry density and/or moisture content test results are included on the boring logs in Appendix A.

TABLE B-I
SUMMARY OF LABORATORY ATTERBERG LIMITS TEST RESULTS
ASTM D 4318

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index
B2-2.5-3	60	31	29
B4-2.5-3	60	27	33

TABLE B-II
SUMMARY OF LABORATORY GRAIN SIZE ANALYSIS – NO. 200 WASH
ASTM D1140

Boring No.	Sample Depth (feet)	Fraction Passing No. 200 Sieve (%)
B1	39-39.5	56
B3	24-24.5	9

TABLE B-III
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content		Dry Density* (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B1-1-5	14.9	32.0	92.4	53
B3-0.5-5	15.3	31.6	92.3	47

*Before saturation.

APPENDIX B
LABORATORY TESTING (cont.)

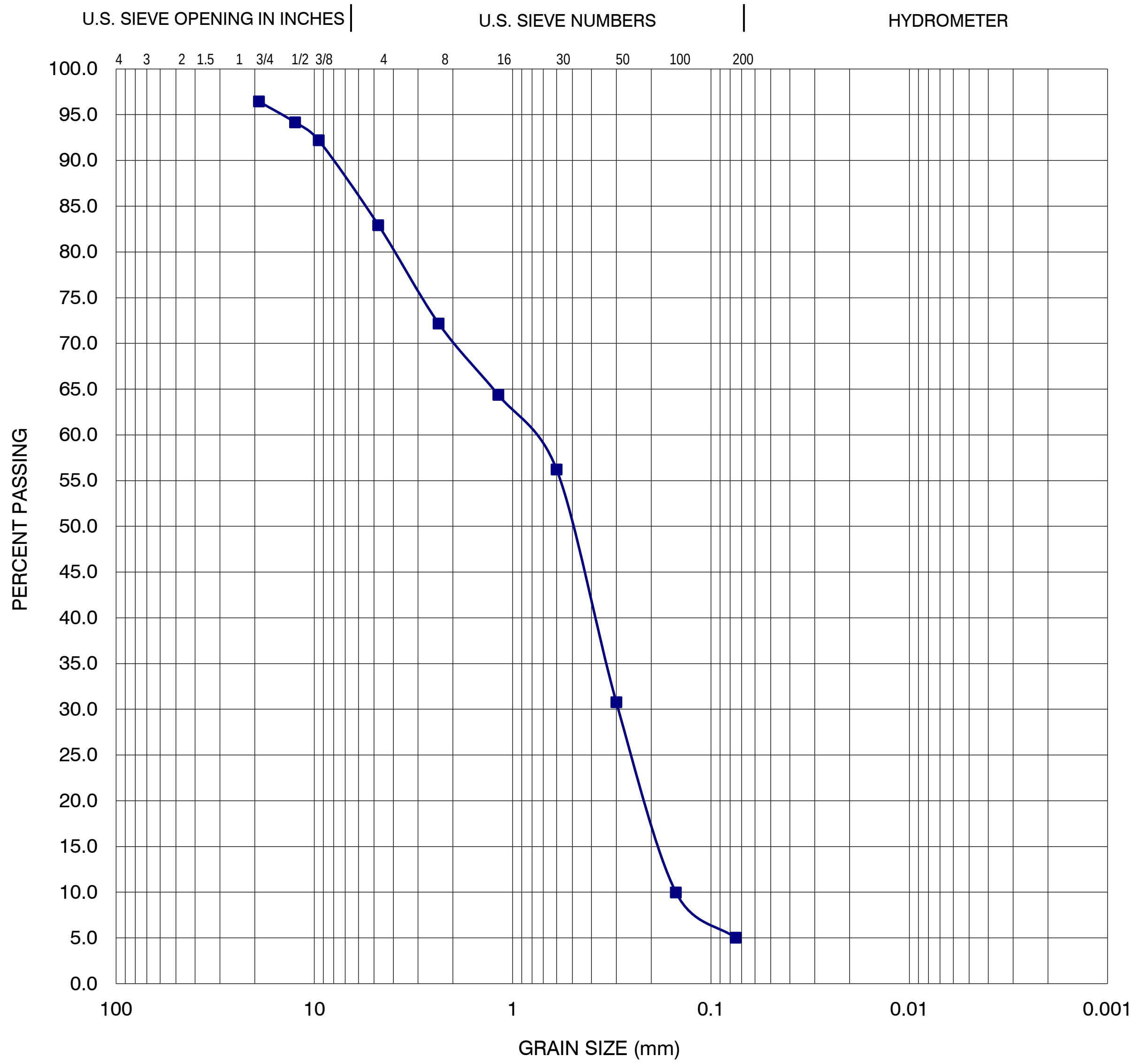
TABLE B-IV
SUMMARY OF SOIL CORROSION PARAMETERS
(CTM 643, CTM 417, CTM 422)

Boring No. (sample depth in feet)	Soil Type (USCS Classification)	Resistivity (ohm-cm)	pH	Chloride (ppm)	Sulfate (ppm)
B1 (1-5)	Fat CLAY with few sands (CH)	1,100	7.7	58	160
B3 (0-5)	Fat CLAY with silt and few sands (CH)	1,300	7.7	52	<10

*Caltrans considers a site corrosive to foundation elements if one or more of the following conditions exist for the representative soil samples at the site:

- o The pH is equal to or less than 5.5.
- o Chloride concentration is equal to or greater than 500 parts per million (ppm) or 0.05%.
- o Sulfate concentration is equal to or greater than 2,000 ppm (0.2%)

**According to the American Concrete Institute 318 Chapter 19, Type II cement may be used where sulfate levels are below 2,000 ppm (0.2%)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring: B1

Depth To Sample: 29' -30'

Sieve Date: 9/27/17

Tested and Computed by: AC

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100	100	96.4	94.2	92.2	82.9	72.2	64.4	56.2	30.8	10.0	5.0

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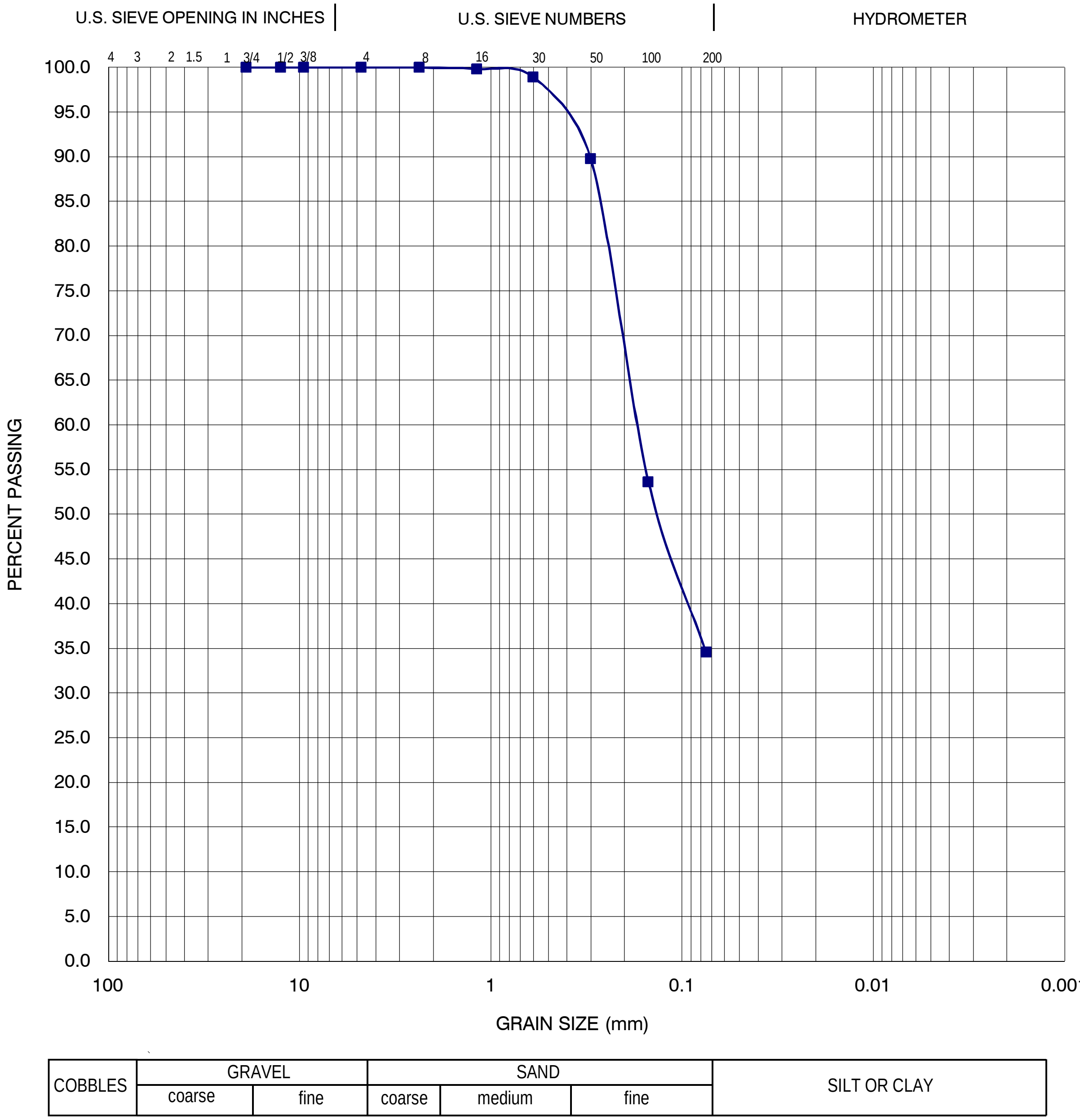
Particle Size Analysis - ASTM D422

Project: RagingWire Santa Clara

Location: Santa Clara, CA

Project No.: E9008-04-03

FIGURE B1



Boring: B3

Depth To Sample: 34'-34.5'

Sieve Date: 9/27/17

Tested and Computed by: AC

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100	100	100	100	100	100	100.0	99.8	98.9	89.7	53.6	34.6

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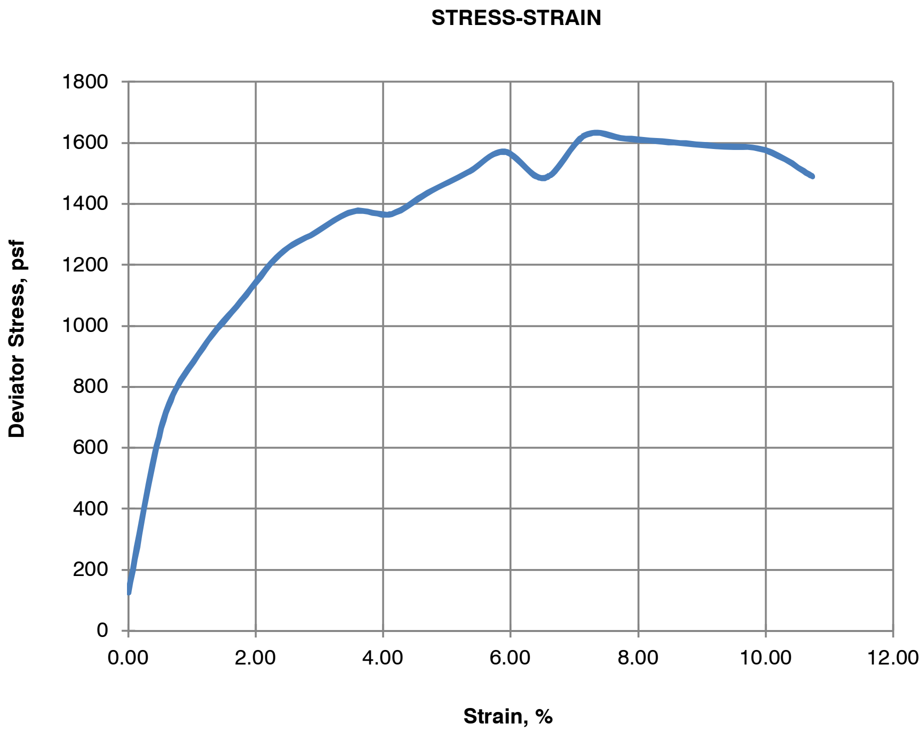
Particle Size Analysis - ASTM D422

Project: RagingWire Santa Clara

Location: Santa Clara, CA

Project No.: E9008-04-03

FIGURE B2



Failure Photo

Sample Description	
Boring Number	B1
Sample Depth (feet)	9.5'
Material Description	Gray CLAY with little (f) sand
Initial Conditions at Start of Test	
Height (inch) average of 3	6.00
Diameter (inch) average of 3	2.41
Moisture Content (%)	23.9
Dry Density (pcf)	102.1
Estimated Specific Gravity	2.7
Saturation (%)	99.3
Shear Test Conditions	
Strain Rate (%/min)	1.1927
Major Principal Stress at Failure (psf)	1630
Strain at Failure (%)	7.2
Test Results	
Unconfined Compressive Strength (tons/ft^2)	0.8
Unconfined Compressive Strength (lbs/ft^2)	1625
Shear Strength (tons/ft^2)	0.4
Shear Strength (lbs/ft^2)	813

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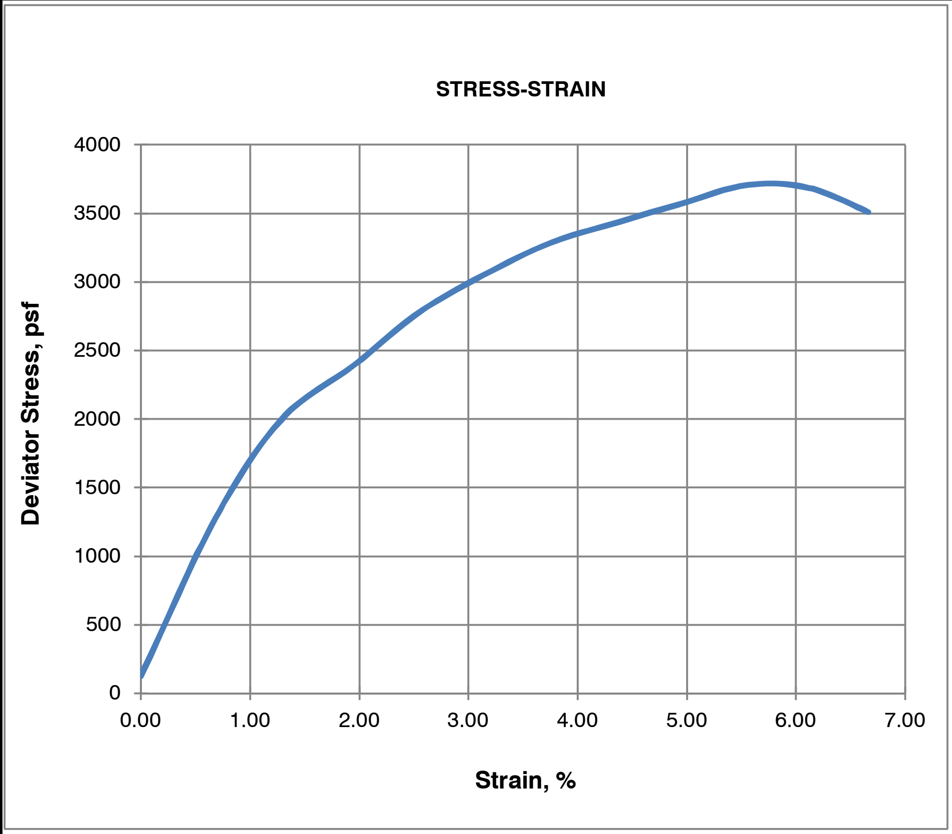
Unconfined Compressive Strength (ASTM D2166)

Project: RagingWire Santa Clara

Location: Santa Clara, California

Project No.: E9008-04-03

FIGURE B3



Failure Photo

Sample Description	
Boring Number	B2
Sample Depth (feet)	3'
Material Description	Gray speckled brown, CLAY with few (f) sands
Initial Conditions at Start of Test	
Height (inch) average of 3	5.18
Diameter (inch) average of 3	2.40
Moisture Content (%)	30.0
Dry Density (pcf)	88.8
Estimated Specific Gravity	2.7
Saturation (%)	90.2
Shear Test Conditions	
Strain Rate (%/min)	1.1104
Major Principal Stress at Failure (psf)	3710
Strain at Failure (%)	5.6
Test Results	
Unconfined Compressive Strength (tons/ft^2)	1.9
Unconfined Compressive Strength (lbs/ft^2)	3708
Shear Strength (tons/ft^2)	0.9
Shear Strength (lbs/ft^2)	1854

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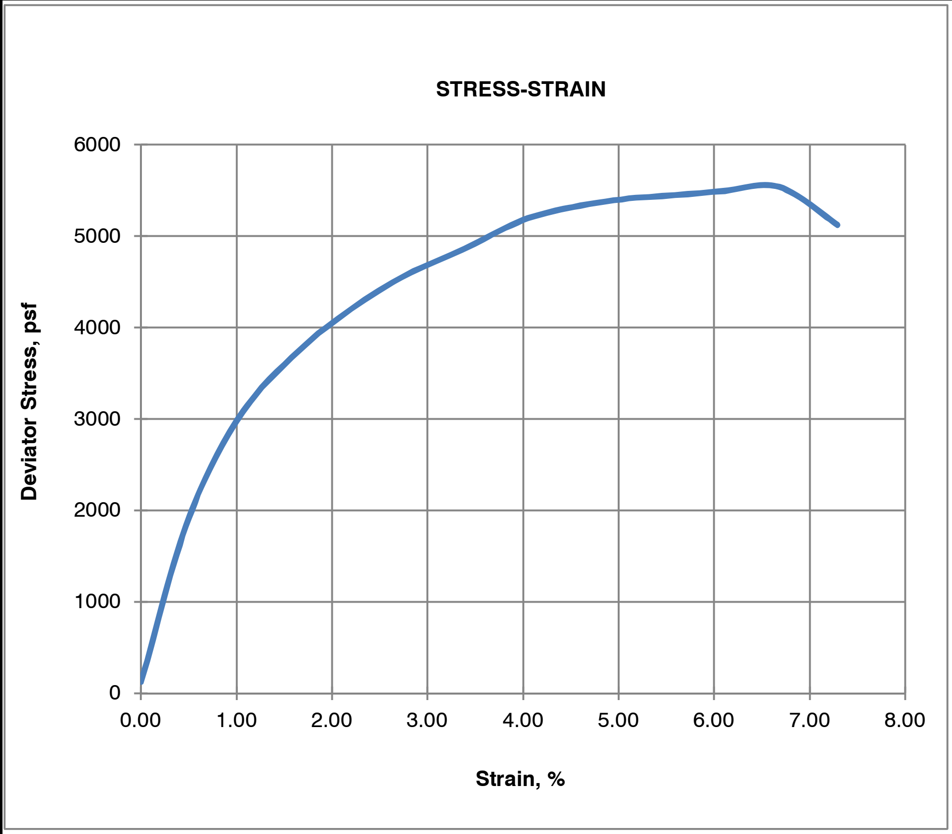
Unconfined Compressive Strength (ASTM D2166)

Project: RagingWire Santa Clara

Location: Santa Clara, California

Project No.: E9008-04-03

FIGURE B4



Failure Photo

Sample Description	
Boring Number	B3
Sample Depth (feet)	4.5'
Material Description	Gray-brown CLAY with few (f) sands
Initial Conditions at Start of Test	
Height (inch) average of 3	5.76
Diameter (inch) average of 3	2.41
Moisture Content (%)	25.1
Dry Density (pcf)	97.9
Estimated Specific Gravity	2.7
Saturation (%)	93.9
Shear Test Conditions	
Strain Rate (%/min)	1.0416
Major Principal Stress at Failure (psf)	5540
Strain at Failure (%)	6.7
Test Results	
Unconfined Compressive Strength (tons/ft^2)	2.8
Unconfined Compressive Strength (lbs/ft^2)	5539
Shear Strength (tons/ft^2)	1.4
Shear Strength (lbs/ft^2)	2769

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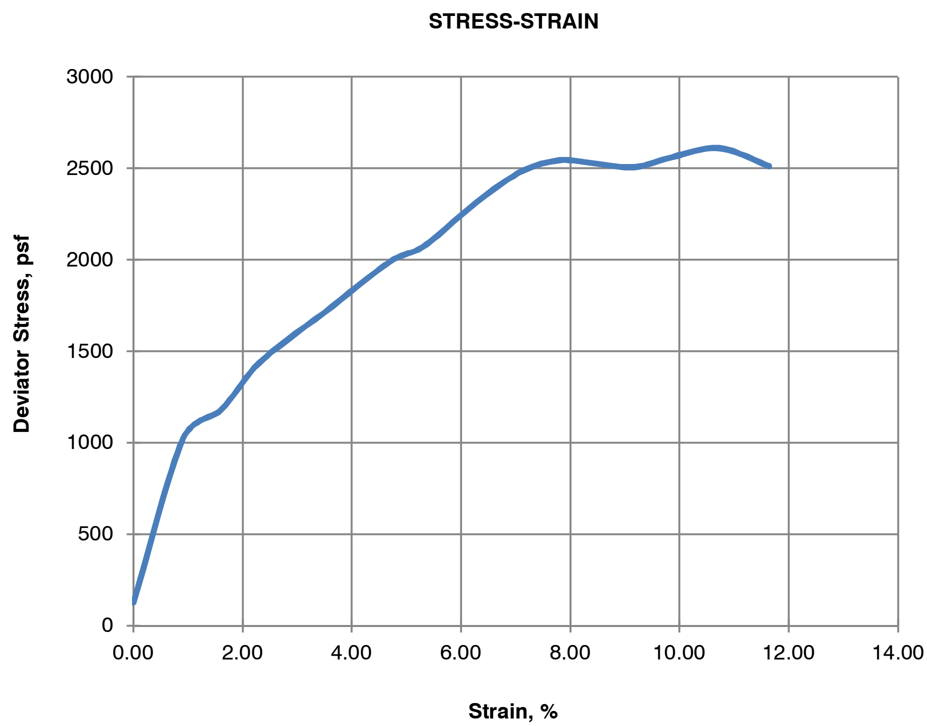
Unconfined Compressive Strength (ASTM D2166)

Project: RagingWire Santa Clara

Location: Santa Clara, California

Project No.: E9008-04-03

FIGURE B5



Failure Photo

Sample Description	
Boring Number	B3
Sample Depth (feet)	9.5'
Material Description	Yellow-gray CLAY with little (f) sand
Initial Conditions at Start of Test	
Height (inch) average of 3	5.50
Diameter (inch) average of 3	2.40
Moisture Content (%)	21.3
Dry Density (pcf)	106.3
Estimated Specific Gravity	2.7
Saturation (%)	98.5
Shear Test Conditions	
Strain Rate (%/min)	1.2935
Major Principal Stress at Failure (psf)	2610
Strain at Failure (%)	10.7
Test Results	
Unconfined Compressive Strength (tons/ft^2)	1.3
Unconfined Compressive Strength (lbs/ft^2)	2611
Shear Strength (tons/ft^2)	0.7
Shear Strength (lbs/ft^2)	1305

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Unconfined Compressive Strength (ASTM D2166)

Project: RagingWire Santa Clara

Location: Santa Clara, California

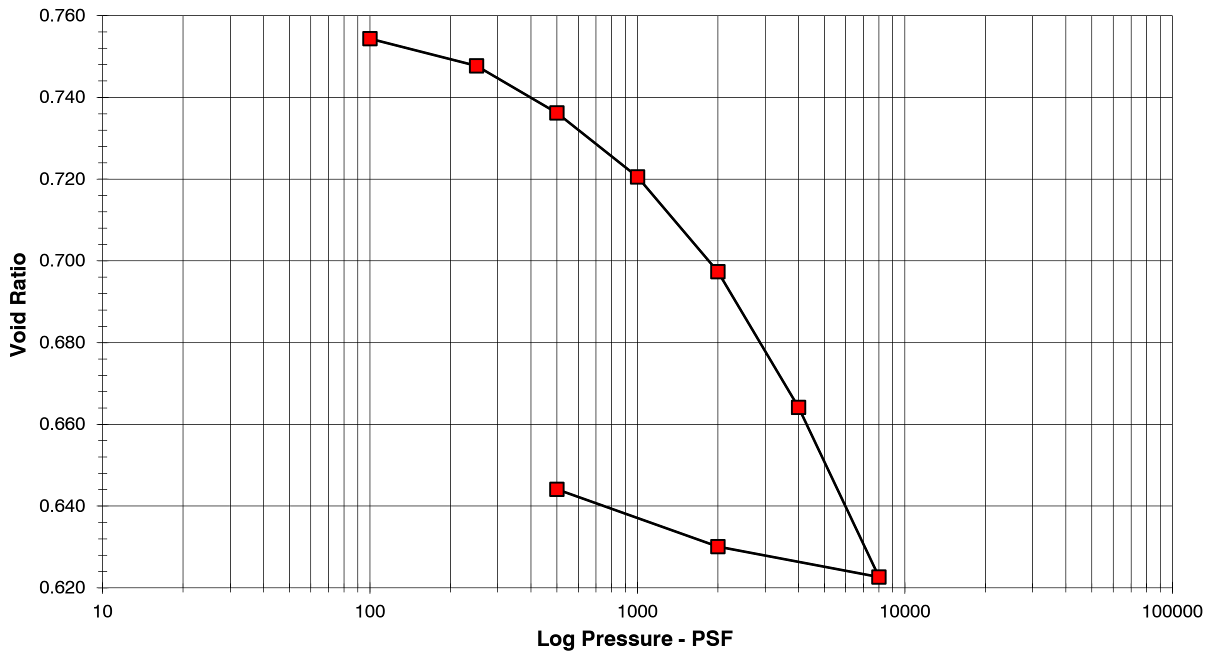
Project No.: E9008-04-03

FIGURE B6

CONSOLIDATION TEST - ASTM D2435

Project Name: RagingWire Santa Clara

Project Number: E9008-04-03 Sample Number: B2-9.5



Axial Load (psf)	Void Ratio	Axial Strain (%)	m_V, coef of vol Compres (in^2/lb)	c_C, Comp Index	50% Consolidation: t_{50}, Time to Consol (min)	50% Consolidation: C_V, Coeff of Consol (ft^2/yr)	90% Consolidation: t_{90}, Time to Consol (min)	90% Consolidation: C_V, Coeff of Consol (ft^2/yr)
initial	0.756	0.00						
100	0.754	0.08						
250	0.748	0.46	0.0036	0.017	3.36	29.97	14.38	30.14
500	0.736	1.11	0.0038	0.038	0.50	197.82	2.16	198.97
1000	0.721	2.01	0.0026	0.052	0.44	220.47	1.90	221.74
2000	0.697	3.33	0.0019	0.077	0.35	273.54	1.50	275.11
4000	0.664	5.22	0.0014	0.110	0.41	224.57	1.77	225.87
8000	0.623	7.58	0.0009	0.138	0.63	139.88	1.31	291.22
2000	0.630	7.16						
500	0.644	6.36						

G_s = 2.75 (assumed)	CONDITIONS AT START OF TEST	CONDITIONS AT END OF TEST
HEIGHT (in.)	0.7500	0.6967
MOISTURE CONTENT (%)	26.0	22.7
DRY DENSITY (pcf):	97.8	105.3
SATURATION (%)	94.6	99.3

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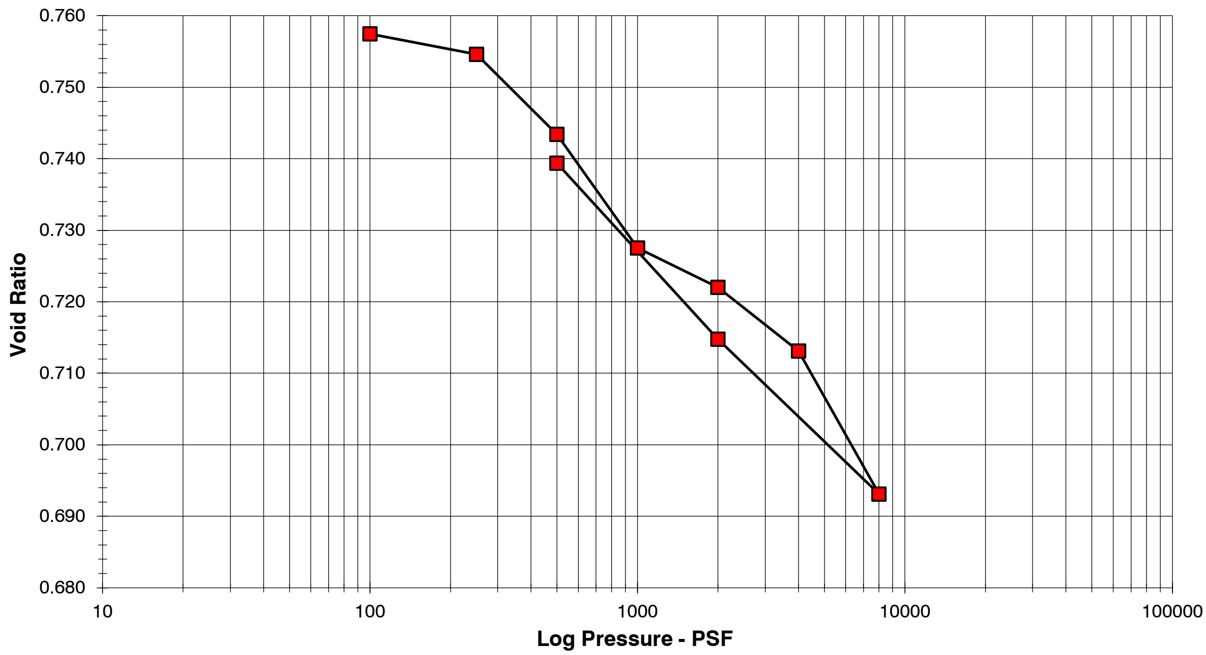
FIGURE B7

CONSOLIDATION TEST - ASTM D2435

Project Name: RagingWire Santa Clara

Project Number: E9008-04-03 Sample Number: B4-4.5



Axial Load (psf)	Void Ratio	Axial Strain (%)	m_V, coef of vol Compres (in^2/lb)	c_C, Comp Index	50% Consolidation: t_{50}, Time to Consol (min)	50% Consolidation: C_V, Coeff of Consol (ft^2/yr)	90% Consolidation: t_{90}, Time to Consol (min)	90% Consolidation: C_V, Coeff of Consol (ft^2/yr)
initial	0.759	0.00						
100	0.757	0.08						
250	0.755	0.24	0.0016	0.007	0.03	2998.71	0.14	3016.03
500	0.743	0.88	0.0037	0.037	0.44	224.97	1.90	226.27
1000	0.728	1.78	0.0026	0.053	0.12	838.75	0.50	843.59
2000	0.722	2.09	0.0005	0.018	0.35	277.65	1.50	279.25
4000	0.713	2.60	0.0004	0.030	1.78	54.24	7.62	54.55
8000	0.693	3.74	0.0004	0.066	0.50	190.73	1.03	397.10
2000	0.715	2.51						
500	0.739	1.11						

G_s = 2.75 (assumed)

	CONDITIONS AT START OF TEST	CONDITIONS AT END OF TEST
HEIGHT (in.)	0.7500	0.7337
MOISTURE CONTENT (%)	25.2	26.0
DRY DENSITY (pcf):	97.6	99.8
SATURATION (%)	91.4	99.2

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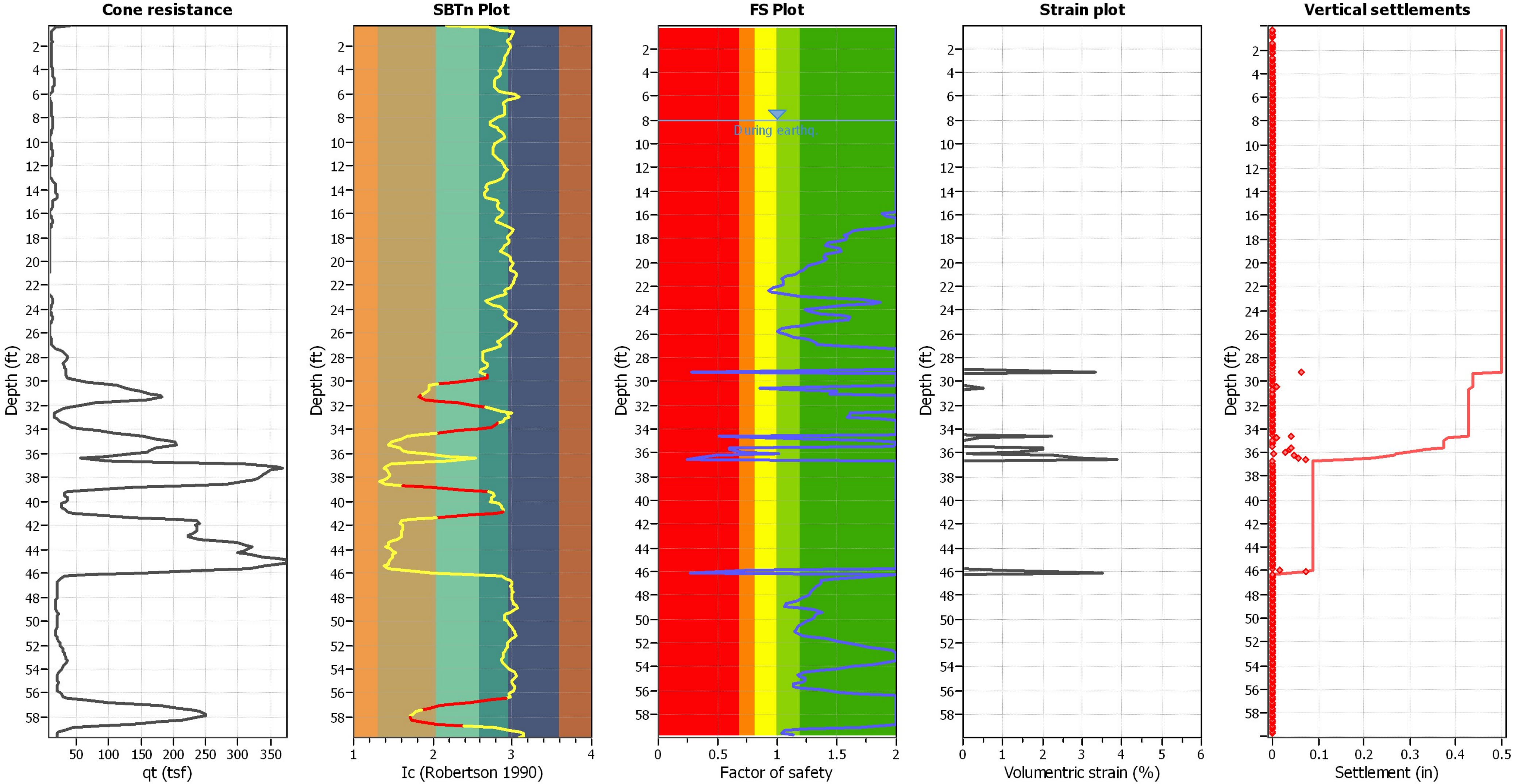
FIGURE B8

APPENDIX C

APPENDIX C
SELECTED OUTPUT – LIQUEFACTION ANALYSIS

T A B L E   O F   C O N T E N T S

Estimation of post-earthquake settlements



Abbreviations

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

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
8.04	20.42	2.00	0.00	1.00	0.00	8.20	21.48	2.00	0.00	1.00	0.00
8.37	22.79	2.00	0.00	1.00	0.00	8.53	23.93	2.00	0.00	1.00	0.00
8.69	20.78	2.00	0.00	1.00	0.00	8.86	20.10	2.00	0.00	1.00	0.00
9.02	18.06	2.00	0.00	1.00	0.00	9.19	17.28	2.00	0.00	1.00	0.00
9.35	15.95	2.00	0.00	1.00	0.00	9.51	15.03	2.00	0.00	1.00	0.00
9.68	14.69	2.00	0.00	1.00	0.00	9.84	14.92	2.00	0.00	1.00	0.00
10.01	15.56	2.00	0.00	1.00	0.00	10.17	16.61	2.00	0.00	1.00	0.00
10.34	17.23	2.00	0.00	1.00	0.00	10.50	17.70	2.00	0.00	1.00	0.00
10.66	18.71	2.00	0.00	1.00	0.00	10.83	19.04	2.00	0.00	1.00	0.00
10.99	18.97	2.00	0.00	1.00	0.00	11.16	18.48	2.00	0.00	1.00	0.00
11.32	18.00	2.00	0.00	1.00	0.00	11.48	17.93	2.00	0.00	1.00	0.00
11.65	17.71	2.00	0.00	1.00	0.00	11.81	15.93	2.00	0.00	1.00	0.00
11.98	15.20	2.00	0.00	1.00	0.00	12.14	14.49	2.00	0.00	1.00	0.00
12.30	14.03	2.00	0.00	1.00	0.00	12.47	13.19	2.00	0.00	1.00	0.00
12.63	13.14	2.00	0.00	1.00	0.00	12.80	13.62	2.00	0.00	1.00	0.00
12.96	14.09	2.00	0.00	1.00	0.00	13.12	14.95	2.00	0.00	1.00	0.00
13.29	17.85	2.00	0.00	1.00	0.00	13.45	22.35	2.00	0.00	1.00	0.00
13.62	24.51	2.00	0.00	1.00	0.00	13.78	25.16	2.00	0.00	1.00	0.00
13.94	24.70	2.00	0.00	1.00	0.00	14.11	23.14	2.00	0.00	1.00	0.00
14.27	24.53	2.00	0.00	1.00	0.00	14.44	27.12	2.00	0.00	1.00	0.00
14.60	28.21	2.00	0.00	1.00	0.00	14.76	22.42	2.00	0.00	1.00	0.00
14.93	16.81	2.00	0.00	1.00	0.00	15.09	17.12	2.00	0.00	1.00	0.00
15.26	17.67	2.00	0.00	1.00	0.00	15.42	17.86	2.00	0.00	1.00	0.00
15.58	15.11	2.00	0.00	1.00	0.00	15.75	12.97	2.00	0.00	1.00	0.00
15.91	12.44	1.88	0.00	1.00	0.00	16.08	12.65	1.91	0.00	1.00	0.00
16.24	13.58	2.00	0.00	1.00	0.00	16.40	15.24	2.00	0.00	1.00	0.00
16.57	16.27	2.00	0.00	1.00	0.00	16.73	16.22	2.00	0.00	1.00	0.00
16.90	14.36	2.00	0.00	1.00	0.00	17.06	12.51	1.86	0.00	1.00	0.00
17.23	11.51	1.71	0.00	1.00	0.00	17.39	11.83	1.63	0.00	1.00	0.00
17.55	11.44	1.60	0.00	1.00	0.00	17.72	11.16	1.56	0.00	1.00	0.00
17.88	11.25	1.59	0.00	1.00	0.00	18.05	12.05	1.58	0.00	1.00	0.00
18.21	11.30	1.51	0.00	1.00	0.00	18.37	10.09	1.42	0.00	1.00	0.00
18.54	10.30	1.41	0.00	1.00	0.00	18.70	10.86	1.46	0.00	1.00	0.00
18.87	11.30	1.53	0.00	1.00	0.00	19.03	11.85	1.54	0.00	1.00	0.00
19.19	11.24	1.48	0.00	1.00	0.00	19.36	10.27	1.41	0.00	1.00	0.00
19.52	10.24	1.39	0.00	1.00	0.00	19.69	11.02	1.41	0.00	1.00	0.00
19.85	10.87	1.40	0.00	1.00	0.00	20.01	10.38	1.34	0.00	1.00	0.00
20.18	10.01	1.29	0.00	1.00	0.00	20.34	9.98	1.26	0.00	1.00	0.00
20.51	9.84	1.23	0.00	1.00	0.00	20.67	9.24	1.22	0.00	1.00	0.00
20.83	10.01	1.19	0.00	1.00	0.00	21.00	9.31	1.14	0.00	1.00	0.00
21.16	8.60	1.07	0.00	1.00	0.00	21.33	8.58	1.04	0.00	1.00	0.00
21.49	8.67	1.04	0.00	1.00	0.00	21.65	8.54	1.05	0.00	1.00	0.00
21.82	8.85	1.04	0.00	1.00	0.00	21.98	8.60	1.01	0.00	1.00	0.00
22.15	7.91	0.96	0.00	1.00	0.00	22.31	7.56	0.93	0.00	1.00	0.00
22.47	7.77	0.95	0.00	1.00	0.00	22.64	8.31	1.04	0.00	1.00	0.00
22.80	9.62	1.19	0.00	1.00	0.00	22.97	11.47	1.43	0.00	1.00	0.00
23.13	14.08	1.71	0.00	1.00	0.00	23.30	16.45	1.86	0.00	1.00	0.00
23.46	15.43	1.80	0.00	1.00	0.00	23.62	12.89	1.57	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
23.79	11.00	1.35	0.00	1.00	0.00	23.95	10.10	1.23	0.00	1.00	0.00
24.12	10.19	1.25	0.00	1.00	0.00	24.28	11.78	1.37	0.00	1.00	0.00
24.44	13.58	1.55	0.00	1.00	0.00	24.61	15.14	1.61	0.00	1.00	0.00
24.77	14.04	1.60	0.00	1.00	0.00	24.94	13.57	1.47	0.00	1.00	0.00
25.10	12.47	1.34	0.00	1.00	0.00	25.26	11.17	1.22	0.00	1.00	0.00
25.43	10.71	1.11	0.00	1.00	0.00	25.59	9.73	1.03	0.00	1.00	0.00
25.76	9.39	1.00	0.00	1.00	0.00	25.92	9.79	1.02	0.00	1.00	0.00
26.08	10.30	1.08	0.00	1.00	0.00	26.25	10.70	1.13	0.00	1.00	0.00
26.41	10.99	1.23	0.00	1.00	0.00	26.58	12.22	1.28	0.00	1.00	0.00
26.74	11.46	1.33	0.00	1.00	0.00	26.90	11.96	1.33	0.00	1.00	0.00
27.07	12.24	1.54	0.00	1.00	0.00	27.23	16.55	2.00	0.00	1.00	0.00
27.40	27.45	2.00	0.00	1.00	0.00	27.56	30.12	2.00	0.00	1.00	0.00
27.72	34.08	2.00	0.00	1.00	0.00	27.89	35.99	2.00	0.00	1.00	0.00
28.05	36.00	2.00	0.00	1.00	0.00	28.22	36.21	2.00	0.00	1.00	0.00
28.38	32.42	2.00	0.00	1.00	0.00	28.54	29.74	2.00	0.00	1.00	0.00
28.71	29.75	2.00	0.00	1.00	0.00	28.87	33.36	2.00	0.00	1.00	0.00
29.04	34.37	2.00	0.00	1.00	0.00	29.20	96.70	0.28	3.32	1.00	0.06
29.36	32.23	2.00	0.00	1.00	0.00	29.53	32.92	2.00	0.00	1.00	0.00
29.69	34.41	2.00	0.00	1.00	0.00	29.86	103.15	2.00	0.00	1.00	0.00
30.02	120.59	2.00	0.00	1.00	0.00	30.19	149.17	2.00	0.00	1.00	0.00
30.35	150.27	2.00	0.00	1.00	0.00	30.51	159.91	0.85	0.50	1.00	0.01
30.68	168.11	1.10	0.05	1.00	0.00	30.84	176.54	1.50	0.00	1.00	0.00
31.01	175.42	1.44	0.00	1.00	0.00	31.17	186.04	2.00	0.00	1.00	0.00
31.33	191.09	2.00	0.00	1.00	0.00	31.50	189.24	2.00	0.00	1.00	0.00
31.66	165.39	2.00	0.00	1.00	0.00	31.83	132.40	2.00	0.00	1.00	0.00
31.99	107.97	2.00	0.00	1.00	0.00	32.15	42.20	2.00	0.00	1.00	0.00
32.32	27.86	2.00	0.00	1.00	0.00	32.48	16.23	2.00	0.00	1.00	0.00
32.65	18.08	1.60	0.00	1.00	0.00	32.81	15.68	1.61	0.00	1.00	0.00
32.97	16.50	1.59	0.00	1.00	0.00	33.14	17.59	1.87	0.00	1.00	0.00
33.30	23.65	2.00	0.00	1.00	0.00	33.47	31.09	2.00	0.00	1.00	0.00
33.63	37.57	2.00	0.00	1.00	0.00	33.79	37.20	2.00	0.00	1.00	0.00
33.96	114.12	2.00	0.00	1.00	0.00	34.12	141.71	2.00	0.00	1.00	0.00
34.29	157.85	2.00	0.00	1.00	0.00	34.45	129.63	2.00	0.00	1.00	0.00
34.61	139.88	0.51	2.22	1.00	0.04	34.78	161.35	0.87	0.41	1.00	0.01
34.94	181.54	1.81	0.00	1.00	0.00	35.11	188.11	2.00	0.00	1.00	0.00
35.27	192.91	2.00	0.00	1.00	0.00	35.43	183.61	1.98	0.00	1.00	0.00
35.60	146.81	0.59	2.01	1.00	0.04	35.76	147.02	0.60	2.00	1.00	0.04
35.93	154.21	0.71	1.34	1.00	0.03	36.09	166.27	1.01	0.11	1.00	0.00
36.26	137.51	0.49	2.28	1.00	0.05	36.42	108.01	0.32	2.97	1.00	0.06
36.58	82.87	0.25	3.87	1.00	0.07	36.75	193.68	2.00	0.00	1.00	0.00
36.91	254.00	2.00	0.00	1.00	0.00	37.08	254.00	2.00	0.00	1.00	0.00
37.24	254.00	2.00	0.00	1.00	0.00	37.40	254.00	2.00	0.00	1.00	0.00
37.57	254.00	2.00	0.00	1.00	0.00	37.73	254.00	2.00	0.00	1.00	0.00
37.90	254.00	2.00	0.00	1.00	0.00	38.06	254.00	2.00	0.00	1.00	0.00
38.22	254.00	2.00	0.00	1.00	0.00	38.39	254.00	2.00	0.00	1.00	0.00
38.55	254.00	2.00	0.00	1.00	0.00	38.72	213.91	2.00	0.00	1.00	0.00
38.88	154.31	2.00	0.00	1.00	0.00	39.04	105.45	2.00	0.00	1.00	0.00
39.21	25.31	2.00	0.00	1.00	0.00	39.37	25.88	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
39.54	31.24	2.00	0.00	1.00	0.00	39.70	28.47	2.00	0.00	1.00	0.00
39.86	36.45	2.00	0.00	1.00	0.00	40.03	28.96	2.00	0.00	1.00	0.00
40.19	22.43	2.00	0.00	1.00	0.00	40.36	21.62	2.00	0.00	1.00	0.00
40.52	24.76	2.00	0.00	1.00	0.00	40.68	25.66	2.00	0.00	1.00	0.00
40.85	29.05	2.00	0.00	1.00	0.00	41.01	37.98	2.00	0.00	1.00	0.00
41.18	113.44	2.00	0.00	1.00	0.00	41.34	165.02	2.00	0.00	1.00	0.00
41.50	197.79	2.00	0.00	1.00	0.00	41.67	217.18	2.00	0.00	1.00	0.00
41.83	210.06	2.00	0.00	1.00	0.00	42.00	204.06	2.00	0.00	1.00	0.00
42.16	202.75	2.00	0.00	1.00	0.00	42.32	213.33	2.00	0.00	1.00	0.00
42.49	204.90	2.00	0.00	1.00	0.00	42.65	200.24	2.00	0.00	1.00	0.00
42.82	194.93	2.00	0.00	1.00	0.00	42.98	185.73	2.00	0.00	1.00	0.00
43.15	198.77	2.00	0.00	1.00	0.00	43.31	245.24	2.00	0.00	1.00	0.00
43.47	254.00	2.00	0.00	1.00	0.00	43.64	254.00	2.00	0.00	1.00	0.00
43.80	254.00	2.00	0.00	1.00	0.00	43.97	254.00	2.00	0.00	1.00	0.00
44.13	254.00	2.00	0.00	1.00	0.00	44.29	250.50	2.00	0.00	1.00	0.00
44.46	254.00	2.00	0.00	1.00	0.00	44.62	254.00	2.00	0.00	1.00	0.00
44.79	254.00	2.00	0.00	1.00	0.00	44.95	254.00	2.00	0.00	1.00	0.00
45.11	254.00	2.00	0.00	1.00	0.00	45.28	254.00	2.00	0.00	1.00	0.00
45.44	254.00	2.00	0.00	1.00	0.00	45.61	254.00	2.00	0.00	1.00	0.00
45.77	240.23	2.00	0.00	1.00	0.00	45.93	157.40	0.77	0.89	1.00	0.02
46.10	91.31	0.27	3.52	1.00	0.07	46.26	22.59	2.00	0.00	1.00	0.00
46.43	20.79	1.68	0.00	1.00	0.00	46.59	16.92	1.49	0.00	1.00	0.00
46.75	16.26	1.37	0.00	1.00	0.00	46.92	17.02	1.36	0.00	1.00	0.00
47.08	16.60	1.36	0.00	1.00	0.00	47.25	16.18	1.32	0.00	1.00	0.00
47.41	16.07	1.30	0.00	1.00	0.00	47.57	15.81	1.29	0.00	1.00	0.00
47.74	15.94	1.28	0.00	1.00	0.00	47.90	15.60	1.26	0.00	1.00	0.00
48.07	15.42	1.24	0.00	1.00	0.00	48.23	15.32	1.21	0.00	1.00	0.00
48.39	14.59	1.17	0.00	1.00	0.00	48.56	14.18	1.12	0.00	1.00	0.00
48.72	13.54	1.07	0.00	1.00	0.00	48.89	13.05	1.06	0.00	1.00	0.00
49.05	13.96	1.19	0.00	1.00	0.00	49.22	17.35	1.31	0.00	1.00	0.00
49.38	16.47	1.38	0.00	1.00	0.00	49.54	15.59	1.33	0.00	1.00	0.00
49.71	15.41	1.31	0.00	1.00	0.00	49.87	15.39	1.31	0.00	1.00	0.00
50.04	15.06	1.29	0.00	1.00	0.00	50.20	14.35	1.24	0.00	1.00	0.00
50.36	13.71	1.21	0.00	1.00	0.00	50.53	14.07	1.18	0.00	1.00	0.00
50.69	13.75	1.17	0.00	1.00	0.00	50.86	13.95	1.15	0.00	1.00	0.00
51.02	13.86	1.15	0.00	1.00	0.00	51.18	13.99	1.15	0.00	1.00	0.00
51.35	14.42	1.20	0.00	1.00	0.00	51.51	15.85	1.30	0.00	1.00	0.00
51.68	17.35	1.41	0.00	1.00	0.00	51.84	18.23	1.51	0.00	1.00	0.00
52.00	19.05	1.59	0.00	1.00	0.00	52.17	19.93	1.66	0.00	1.00	0.00
52.33	20.97	1.77	0.00	1.00	0.00	52.50	22.47	1.87	0.00	1.00	0.00
52.66	23.51	1.96	0.00	1.00	0.00	52.82	24.09	2.00	0.00	1.00	0.00
52.99	23.66	2.00	0.00	1.00	0.00	53.15	25.78	2.00	0.00	1.00	0.00
53.32	26.59	2.00	0.00	1.00	0.00	53.48	25.24	2.00	0.00	1.00	0.00
53.64	23.60	1.94	0.00	1.00	0.00	53.81	21.20	1.79	0.00	1.00	0.00
53.97	19.81	1.63	0.00	1.00	0.00	54.14	18.05	1.49	0.00	1.00	0.00
54.30	16.31	1.34	0.00	1.00	0.00	54.46	14.65	1.23	0.00	1.00	0.00
54.63	13.96	1.17	0.00	1.00	0.00	54.79	14.09	1.18	0.00	1.00	0.00
54.96	14.66	1.23	0.00	1.00	0.00	55.12	15.45	1.23	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

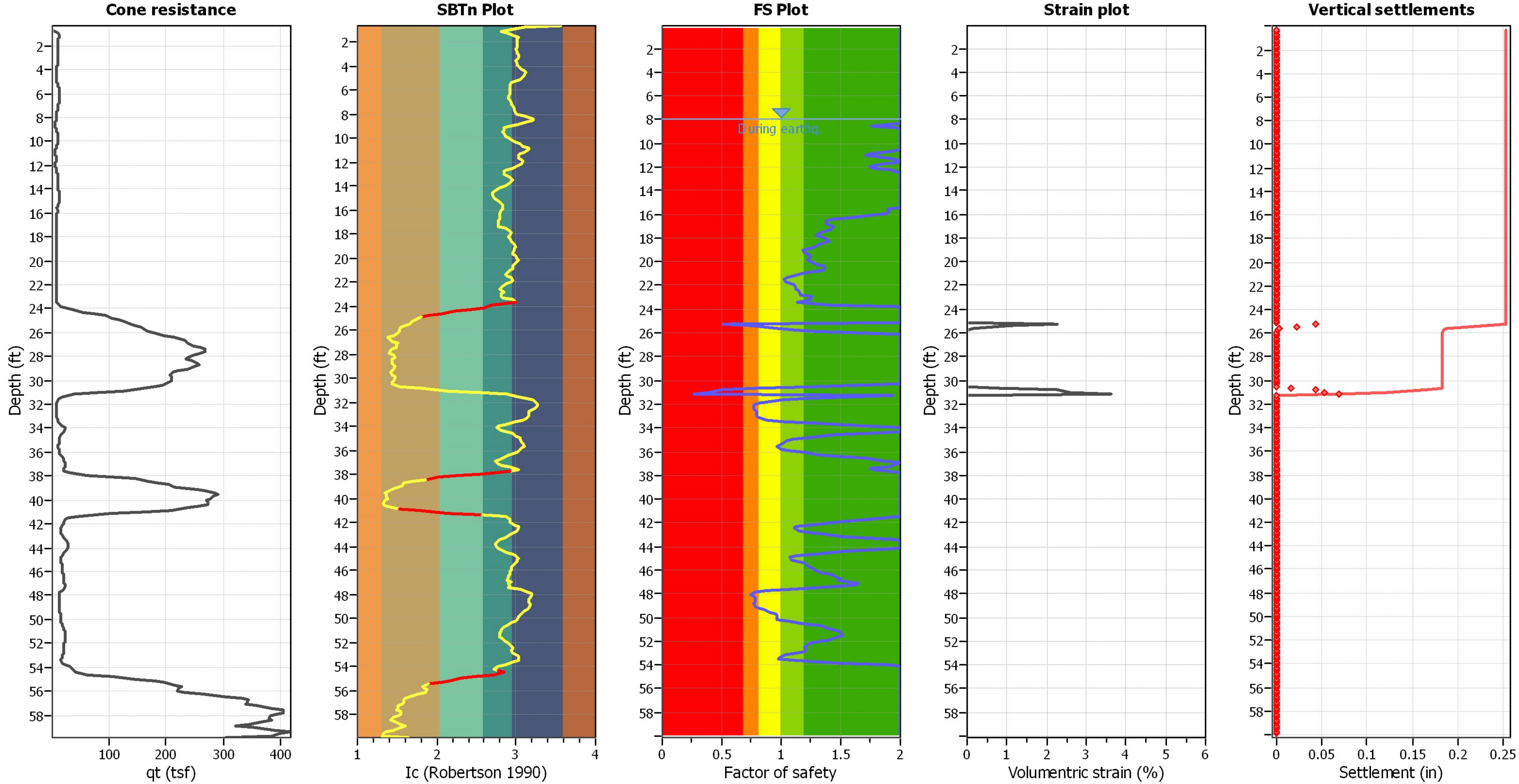
Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
55.28	13.96	1.20	0.00	1.00	0.00	55.45	13.57	1.13	0.00	1.00	0.00
55.61	13.04	1.13	0.00	1.00	0.00	55.78	13.82	1.19	0.00	1.00	0.00
55.94	15.49	1.36	0.00	1.00	0.00	56.11	19.30	1.52	0.00	1.00	0.00
56.27	19.72	1.70	0.00	1.00	0.00	56.43	22.06	2.00	0.00	1.00	0.00
56.60	36.54	2.00	0.00	1.00	0.00	56.76	145.32	2.00	0.00	1.00	0.00
56.93	187.76	2.00	0.00	1.00	0.00	57.09	210.95	2.00	0.00	1.00	0.00
57.25	212.07	2.00	0.00	1.00	0.00	57.42	196.83	2.00	0.00	1.00	0.00
57.58	193.52	2.00	0.00	1.00	0.00	57.75	198.11	2.00	0.00	1.00	0.00
57.91	204.63	2.00	0.00	1.00	0.00	58.07	190.59	2.00	0.00	1.00	0.00
58.24	160.45	2.00	0.00	1.00	0.00	58.40	151.71	2.00	0.00	1.00	0.00
58.57	158.71	2.00	0.00	1.00	0.00	58.73	110.91	2.00	0.00	1.00	0.00
58.89	30.02	2.00	0.00	1.00	0.00	59.06	20.46	1.70	0.00	1.00	0.00
59.22	15.18	1.23	0.00	1.00	0.00	59.39	13.39	1.06	0.00	1.00	0.00
59.55	14.36	1.04	0.00	1.00	0.00	59.71	15.48	1.13	0.00	1.00	0.00

Total estimated settlement: 0.50

Abbreviations

$Q_{tn,cs}$: Equivalent clean sand normalized cone resistance
FS: Factor of safety against liquefaction
e_v (%): Post-liquefaction volumentric strain
DF: e_v depth weighting factor
Settlement: Calculated settlement

Estimation of post-earthquake settlements



Abbreviations

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

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
8.04	13.78	2.00	0.00	1.00	0.00	8.20	11.94	2.00	0.00	1.00	0.00
8.37	9.03	2.00	0.00	1.00	0.00	8.53	6.57	1.75	0.00	1.00	0.00
8.69	8.81	1.96	0.00	1.00	0.00	8.86	11.74	2.00	0.00	1.00	0.00
9.02	12.72	2.00	0.00	1.00	0.00	9.19	14.84	2.00	0.00	1.00	0.00
9.35	16.21	2.00	0.00	1.00	0.00	9.51	16.99	2.00	0.00	1.00	0.00
9.68	18.17	2.00	0.00	1.00	0.00	9.84	17.81	2.00	0.00	1.00	0.00
10.01	16.46	2.00	0.00	1.00	0.00	10.17	14.71	2.00	0.00	1.00	0.00
10.34	13.11	2.00	0.00	1.00	0.00	10.50	11.37	2.00	0.00	1.00	0.00
10.66	10.20	1.95	0.00	1.00	0.00	10.83	9.31	1.74	0.00	1.00	0.00
10.99	8.85	1.71	0.00	1.00	0.00	11.16	9.94	1.81	0.00	1.00	0.00
11.32	11.01	1.97	0.00	1.00	0.00	11.48	11.52	2.00	0.00	1.00	0.00
11.65	11.06	1.98	0.00	1.00	0.00	11.81	10.61	1.84	0.00	1.00	0.00
11.98	9.74	1.75	0.00	1.00	0.00	12.14	9.98	1.79	0.00	1.00	0.00
12.30	11.44	1.91	0.00	1.00	0.00	12.47	11.94	2.00	0.00	1.00	0.00
12.63	13.64	2.00	0.00	1.00	0.00	12.80	15.32	2.00	0.00	1.00	0.00
12.96	15.92	2.00	0.00	1.00	0.00	13.12	14.67	2.00	0.00	1.00	0.00
13.29	13.95	2.00	0.00	1.00	0.00	13.45	13.51	2.00	0.00	1.00	0.00
13.62	13.19	2.00	0.00	1.00	0.00	13.78	14.32	2.00	0.00	1.00	0.00
13.94	14.92	2.00	0.00	1.00	0.00	14.11	15.90	2.00	0.00	1.00	0.00
14.27	17.89	2.00	0.00	1.00	0.00	14.44	18.60	2.00	0.00	1.00	0.00
14.60	19.17	2.00	0.00	1.00	0.00	14.76	19.60	2.00	0.00	1.00	0.00
14.93	19.15	2.00	0.00	1.00	0.00	15.09	17.59	2.00	0.00	1.00	0.00
15.26	15.39	2.00	0.00	1.00	0.00	15.42	13.20	2.00	0.00	1.00	0.00
15.58	12.40	1.89	0.00	1.00	0.00	15.75	12.49	1.90	0.00	1.00	0.00
15.91	13.58	1.89	0.00	1.00	0.00	16.08	12.41	1.79	0.00	1.00	0.00
16.24	10.76	1.57	0.00	1.00	0.00	16.40	9.85	1.42	0.00	1.00	0.00
16.57	9.57	1.37	0.00	1.00	0.00	16.73	10.05	1.38	0.00	1.00	0.00
16.90	10.02	1.40	0.00	1.00	0.00	17.06	10.12	1.43	0.00	1.00	0.00
17.23	10.71	1.42	0.00	1.00	0.00	17.39	9.82	1.38	0.00	1.00	0.00
17.55	9.41	1.32	0.00	1.00	0.00	17.72	9.63	1.30	0.00	1.00	0.00
17.88	9.35	1.33	0.00	1.00	0.00	18.05	10.18	1.37	0.00	1.00	0.00
18.21	10.52	1.40	0.00	1.00	0.00	18.37	10.12	1.37	0.00	1.00	0.00
18.54	9.85	1.32	0.00	1.00	0.00	18.70	9.70	1.28	0.00	1.00	0.00
18.87	9.31	1.24	0.00	1.00	0.00	19.03	9.16	1.19	0.00	1.00	0.00
19.19	8.78	1.19	0.00	1.00	0.00	19.36	9.47	1.21	0.00	1.00	0.00
19.52	9.68	1.24	0.00	1.00	0.00	19.69	9.42	1.24	0.00	1.00	0.00
19.85	9.51	1.22	0.00	1.00	0.00	20.01	9.60	1.25	0.00	1.00	0.00
20.18	10.16	1.29	0.00	1.00	0.00	20.34	10.60	1.35	0.00	1.00	0.00
20.51	11.15	1.37	0.00	1.00	0.00	20.67	10.77	1.35	0.00	1.00	0.00
20.83	10.51	1.27	0.00	1.00	0.00	21.00	9.55	1.18	0.00	1.00	0.00
21.16	8.72	1.09	0.00	1.00	0.00	21.33	8.58	1.05	0.00	1.00	0.00
21.49	8.56	1.02	0.00	1.00	0.00	21.65	8.08	1.03	0.00	1.00	0.00
21.82	8.86	1.07	0.00	1.00	0.00	21.98	9.53	1.11	0.00	1.00	0.00
22.15	9.05	1.12	0.00	1.00	0.00	22.31	9.26	1.12	0.00	1.00	0.00
22.47	9.58	1.15	0.00	1.00	0.00	22.64	9.68	1.15	0.00	1.00	0.00
22.80	9.31	1.16	0.00	1.00	0.00	22.97	9.63	1.26	0.00	1.00	0.00
23.13	12.20	1.25	0.00	1.00	0.00	23.30	9.24	1.20	0.00	1.00	0.00
23.46	8.76	1.13	0.00	1.00	0.00	23.62	10.53	1.42	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
23.79	15.83	2.00	0.00	1.00	0.00	23.95	28.53	2.00	0.00	1.00	0.00
24.12	103.18	2.00	0.00	1.00	0.00	24.28	113.92	2.00	0.00	1.00	0.00
24.44	149.51	2.00	0.00	1.00	0.00	24.61	158.98	2.00	0.00	1.00	0.00
24.77	135.57	2.00	0.00	1.00	0.00	24.94	120.57	2.00	0.00	1.00	0.00
25.10	144.64	2.00	0.00	1.00	0.00	25.26	137.08	0.51	2.28	1.00	0.04
25.43	154.00	0.74	1.11	1.00	0.02	25.59	163.46	0.98	0.16	1.00	0.00
25.76	167.30	1.11	0.04	1.00	0.00	25.92	171.44	1.28	0.01	1.00	0.00
26.08	183.21	2.00	0.00	1.00	0.00	26.25	193.44	2.00	0.00	1.00	0.00
26.41	223.09	2.00	0.00	1.00	0.00	26.58	233.54	2.00	0.00	1.00	0.00
26.74	237.92	2.00	0.00	1.00	0.00	26.90	230.91	2.00	0.00	1.00	0.00
27.07	239.23	2.00	0.00	1.00	0.00	27.23	254.00	2.00	0.00	1.00	0.00
27.40	254.00	2.00	0.00	1.00	0.00	27.56	254.00	2.00	0.00	1.00	0.00
27.72	254.00	2.00	0.00	1.00	0.00	27.89	247.17	2.00	0.00	1.00	0.00
28.05	231.01	2.00	0.00	1.00	0.00	28.22	224.08	2.00	0.00	1.00	0.00
28.38	238.32	2.00	0.00	1.00	0.00	28.54	254.00	2.00	0.00	1.00	0.00
28.71	252.59	2.00	0.00	1.00	0.00	28.87	248.75	2.00	0.00	1.00	0.00
29.04	228.23	2.00	0.00	1.00	0.00	29.20	215.37	2.00	0.00	1.00	0.00
29.36	202.02	2.00	0.00	1.00	0.00	29.53	204.25	2.00	0.00	1.00	0.00
29.69	200.40	2.00	0.00	1.00	0.00	29.86	209.46	2.00	0.00	1.00	0.00
30.02	201.44	2.00	0.00	1.00	0.00	30.19	201.39	2.00	0.00	1.00	0.00
30.35	187.97	2.00	0.00	1.00	0.00	30.51	172.26	1.28	0.01	1.00	0.00
30.68	157.37	0.79	0.80	1.00	0.02	30.84	137.53	0.50	2.27	1.00	0.04
31.01	122.78	0.39	2.59	1.00	0.05	31.17	88.70	0.26	3.62	1.00	0.07
31.33	14.36	1.94	0.00	1.00	0.00	31.50	13.85	1.21	0.00	1.00	0.00
31.66	10.25	1.01	0.00	1.00	0.00	31.83	8.97	0.84	0.00	1.00	0.00
31.99	8.95	0.78	0.00	1.00	0.00	32.15	8.74	0.77	0.00	1.00	0.00
32.32	8.72	0.77	0.00	1.00	0.00	32.48	8.89	0.78	0.00	1.00	0.00
32.65	9.07	0.79	0.00	1.00	0.00	32.81	9.14	0.80	0.00	1.00	0.00
32.97	9.12	0.79	0.00	1.00	0.00	33.14	9.01	0.81	0.00	1.00	0.00
33.30	9.56	0.88	0.00	1.00	0.00	33.47	11.24	1.01	0.00	1.00	0.00
33.63	12.72	1.29	0.00	1.00	0.00	33.79	17.40	1.72	0.00	1.00	0.00
33.96	23.65	2.00	0.00	1.00	0.00	34.12	23.23	2.00	0.00	1.00	0.00
34.29	20.93	2.00	0.00	1.00	0.00	34.45	17.99	1.73	0.00	1.00	0.00
34.61	15.35	1.44	0.00	1.00	0.00	34.78	12.99	1.23	0.00	1.00	0.00
34.94	11.94	1.10	0.00	1.00	0.00	35.11	11.73	1.04	0.00	1.00	0.00
35.27	11.52	1.02	0.00	1.00	0.00	35.43	11.23	0.99	0.00	1.00	0.00
35.60	10.83	0.96	0.00	1.00	0.00	35.76	10.90	1.00	0.00	1.00	0.00
35.93	12.35	1.08	0.00	1.00	0.00	36.09	12.96	1.20	0.00	1.00	0.00
36.26	14.22	1.33	0.00	1.00	0.00	36.42	16.19	1.51	0.00	1.00	0.00
36.58	17.89	1.71	0.00	1.00	0.00	36.75	19.68	1.85	0.00	1.00	0.00
36.91	19.91	2.00	0.00	1.00	0.00	37.08	22.59	1.97	0.00	1.00	0.00
37.24	18.02	1.90	0.00	1.00	0.00	37.40	17.71	1.74	0.00	1.00	0.00
37.57	17.67	1.78	0.00	1.00	0.00	37.73	19.43	2.00	0.00	1.00	0.00
37.90	107.69	2.00	0.00	1.00	0.00	38.06	172.86	2.00	0.00	1.00	0.00
38.22	197.36	2.00	0.00	1.00	0.00	38.39	170.26	2.00	0.00	1.00	0.00
38.55	161.26	2.00	0.00	1.00	0.00	38.72	186.34	2.00	0.00	1.00	0.00
38.88	206.86	2.00	0.00	1.00	0.00	39.04	191.10	2.00	0.00	1.00	0.00
39.21	252.40	2.00	0.00	1.00	0.00	39.37	254.00	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
39.54	254.00	2.00	0.00	1.00	0.00	39.70	254.00	2.00	0.00	1.00	0.00
39.86	243.36	2.00	0.00	1.00	0.00	40.03	244.85	2.00	0.00	1.00	0.00
40.19	242.97	2.00	0.00	1.00	0.00	40.36	252.08	2.00	0.00	1.00	0.00
40.52	239.66	2.00	0.00	1.00	0.00	40.68	200.63	2.00	0.00	1.00	0.00
40.85	197.91	2.00	0.00	1.00	0.00	41.01	160.29	2.00	0.00	1.00	0.00
41.18	129.49	2.00	0.00	1.00	0.00	41.34	94.60	2.00	0.00	1.00	0.00
41.50	21.51	2.00	0.00	1.00	0.00	41.67	18.29	1.74	0.00	1.00	0.00
41.83	19.54	1.60	0.00	1.00	0.00	42.00	17.03	1.47	0.00	1.00	0.00
42.16	14.36	1.26	0.00	1.00	0.00	42.32	13.15	1.12	0.00	1.00	0.00
42.49	12.87	1.11	0.00	1.00	0.00	42.65	13.86	1.14	0.00	1.00	0.00
42.82	14.18	1.22	0.00	1.00	0.00	42.98	15.24	1.35	0.00	1.00	0.00
43.15	17.57	1.51	0.00	1.00	0.00	43.31	18.97	1.75	0.00	1.00	0.00
43.47	22.15	1.97	0.00	1.00	0.00	43.64	24.13	2.00	0.00	1.00	0.00
43.80	23.16	2.00	0.00	1.00	0.00	43.97	23.37	2.00	0.00	1.00	0.00
44.13	21.90	1.99	0.00	1.00	0.00	44.29	19.94	1.76	0.00	1.00	0.00
44.46	16.23	1.52	0.00	1.00	0.00	44.62	14.14	1.29	0.00	1.00	0.00
44.79	12.30	1.13	0.00	1.00	0.00	44.95	11.54	1.07	0.00	1.00	0.00
45.11	12.09	1.09	0.00	1.00	0.00	45.28	12.81	1.14	0.00	1.00	0.00
45.44	13.03	1.19	0.00	1.00	0.00	45.61	13.91	1.22	0.00	1.00	0.00
45.77	13.89	1.25	0.00	1.00	0.00	45.93	13.78	1.27	0.00	1.00	0.00
46.10	14.49	1.30	0.00	1.00	0.00	46.26	15.20	1.36	0.00	1.00	0.00
46.43	15.67	1.42	0.00	1.00	0.00	46.59	16.95	1.47	0.00	1.00	0.00
46.75	16.59	1.50	0.00	1.00	0.00	46.92	16.89	1.48	0.00	1.00	0.00
47.08	16.21	1.65	0.00	1.00	0.00	47.25	22.71	1.63	0.00	1.00	0.00
47.41	17.21	1.49	0.00	1.00	0.00	47.57	12.02	1.11	0.00	1.00	0.00
47.74	10.40	0.87	0.00	1.00	0.00	47.90	8.80	0.78	0.00	1.00	0.00
48.07	8.86	0.75	0.00	1.00	0.00	48.23	9.16	0.76	0.00	1.00	0.00
48.39	9.47	0.78	0.00	1.00	0.00	48.56	9.53	0.78	0.00	1.00	0.00
48.72	9.36	0.78	0.00	1.00	0.00	48.89	9.26	0.77	0.00	1.00	0.00
49.05	9.33	0.78	0.00	1.00	0.00	49.22	9.55	0.81	0.00	1.00	0.00
49.38	10.01	0.86	0.00	1.00	0.00	49.54	10.78	0.91	0.00	1.00	0.00
49.71	10.84	0.95	0.00	1.00	0.00	49.87	11.53	0.96	0.00	1.00	0.00
50.04	11.12	0.96	0.00	1.00	0.00	50.20	11.02	0.96	0.00	1.00	0.00
50.36	11.55	1.07	0.00	1.00	0.00	50.53	14.67	1.20	0.00	1.00	0.00
50.69	15.35	1.33	0.00	1.00	0.00	50.86	15.95	1.39	0.00	1.00	0.00
51.02	16.16	1.44	0.00	1.00	0.00	51.18	16.92	1.49	0.00	1.00	0.00
51.35	17.68	1.52	0.00	1.00	0.00	51.51	16.95	1.51	0.00	1.00	0.00
51.68	16.61	1.47	0.00	1.00	0.00	51.84	16.59	1.40	0.00	1.00	0.00
52.00	15.48	1.32	0.00	1.00	0.00	52.17	14.68	1.25	0.00	1.00	0.00
52.33	14.43	1.20	0.00	1.00	0.00	52.50	14.02	1.20	0.00	1.00	0.00
52.66	14.38	1.21	0.00	1.00	0.00	52.82	14.59	1.19	0.00	1.00	0.00
52.99	13.65	1.12	0.00	1.00	0.00	53.15	12.63	1.03	0.00	1.00	0.00
53.32	11.85	0.98	0.00	1.00	0.00	53.48	11.60	0.97	0.00	1.00	0.00
53.64	11.74	1.13	0.00	1.00	0.00	53.81	16.60	1.50	0.00	1.00	0.00
53.97	24.40	1.92	0.00	1.00	0.00	54.14	26.62	2.00	0.00	1.00	0.00
54.30	27.77	2.00	0.00	1.00	0.00	54.46	30.83	2.00	0.00	1.00	0.00
54.63	35.02	2.00	0.00	1.00	0.00	54.79	150.10	2.00	0.00	1.00	0.00
54.96	221.82	2.00	0.00	1.00	0.00	55.12	209.23	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

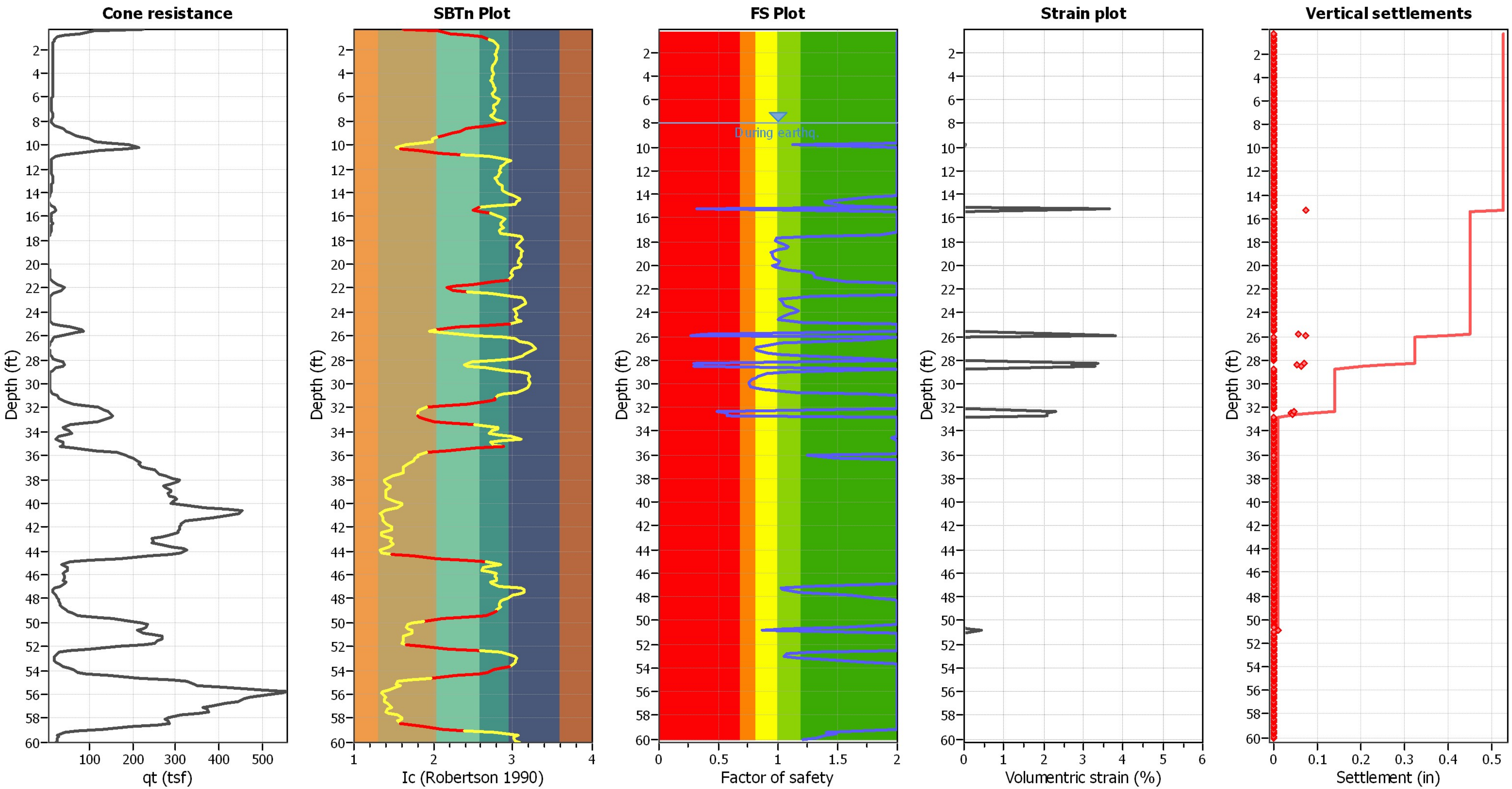
Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
55.28	228.11	2.00	0.00	1.00	0.00	55.45	210.71	2.00	0.00	1.00	0.00
55.61	186.81	2.00	0.00	1.00	0.00	55.78	197.26	2.00	0.00	1.00	0.00
55.94	191.30	2.00	0.00	1.00	0.00	56.11	192.74	2.00	0.00	1.00	0.00
56.27	219.87	2.00	0.00	1.00	0.00	56.43	250.07	2.00	0.00	1.00	0.00
56.60	254.00	2.00	0.00	1.00	0.00	56.76	254.00	2.00	0.00	1.00	0.00
56.93	254.00	2.00	0.00	1.00	0.00	57.09	254.00	2.00	0.00	1.00	0.00
57.25	254.00	2.00	0.00	1.00	0.00	57.42	254.00	2.00	0.00	1.00	0.00
57.58	254.00	2.00	0.00	1.00	0.00	57.75	254.00	2.00	0.00	1.00	0.00
57.91	254.00	2.00	0.00	1.00	0.00	58.07	254.00	2.00	0.00	1.00	0.00
58.24	254.00	2.00	0.00	1.00	0.00	58.40	254.00	2.00	0.00	1.00	0.00
58.57	254.00	2.00	0.00	1.00	0.00	58.73	254.00	2.00	0.00	1.00	0.00
58.89	254.00	2.00	0.00	1.00	0.00	59.06	254.00	2.00	0.00	1.00	0.00
59.22	254.00	2.00	0.00	1.00	0.00	59.39	254.00	2.00	0.00	1.00	0.00
59.55	254.00	2.00	0.00	1.00	0.00	59.71	254.00	2.00	0.00	1.00	0.00
59.88	212.95	2.00	0.00	1.00	0.00						

Total estimated settlement: 0.25

Abbreviations

$Q_{tn,cs}$:	Equivalent clean sand normalized cone resistance
FS:	Factor of safety against liquefaction
e_v (%):	Post-liquefaction volumentric strain
DF:	e_v depth weighting factor
Settlement:	Calculated settlement

Estimation of post-earthquake settlements



Abbreviations

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

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
8.04	17.19	2.00	0.00	1.00	0.00	8.20	18.24	2.00	0.00	1.00	0.00
8.37	20.87	2.00	0.00	1.00	0.00	8.53	109.69	2.00	0.00	1.00	0.00
8.69	121.36	2.00	0.00	1.00	0.00	8.86	121.26	2.00	0.00	1.00	0.00
9.02	130.15	2.00	0.00	1.00	0.00	9.19	145.87	2.00	0.00	1.00	0.00
9.35	159.64	2.00	0.00	1.00	0.00	9.51	169.37	2.00	0.00	1.00	0.00
9.68	198.07	2.00	0.00	1.00	0.00	9.84	155.12	1.12	0.04	1.00	0.00
10.01	231.49	2.00	0.00	1.00	0.00	10.17	254.00	2.00	0.00	1.00	0.00
10.34	221.35	2.00	0.00	1.00	0.00	10.50	136.41	2.00	0.00	1.00	0.00
10.66	141.44	2.00	0.00	1.00	0.00	10.83	104.27	2.00	0.00	1.00	0.00
10.99	23.96	2.00	0.00	1.00	0.00	11.16	18.29	2.00	0.00	1.00	0.00
11.32	16.01	2.00	0.00	1.00	0.00	11.48	14.27	2.00	0.00	1.00	0.00
11.65	19.23	2.00	0.00	1.00	0.00	11.81	18.41	2.00	0.00	1.00	0.00
11.98	18.34	2.00	0.00	1.00	0.00	12.14	19.16	2.00	0.00	1.00	0.00
12.30	18.33	2.00	0.00	1.00	0.00	12.47	18.01	2.00	0.00	1.00	0.00
12.63	20.70	2.00	0.00	1.00	0.00	12.80	21.98	2.00	0.00	1.00	0.00
12.96	21.53	2.00	0.00	1.00	0.00	13.12	20.22	2.00	0.00	1.00	0.00
13.29	18.78	2.00	0.00	1.00	0.00	13.45	17.11	2.00	0.00	1.00	0.00
13.62	17.05	2.00	0.00	1.00	0.00	13.78	17.23	2.00	0.00	1.00	0.00
13.94	15.33	2.00	0.00	1.00	0.00	14.11	13.42	2.00	0.00	1.00	0.00
14.27	11.15	1.77	0.00	1.00	0.00	14.44	10.25	1.53	0.00	1.00	0.00
14.60	9.35	1.39	0.00	1.00	0.00	14.76	8.82	1.41	0.00	1.00	0.00
14.93	10.64	1.56	0.00	1.00	0.00	15.09	12.10	2.00	0.00	1.00	0.00
15.26	87.43	0.32	3.68	1.00	0.08	15.42	89.15	2.00	0.00	1.00	0.00
15.58	76.15	2.00	0.00	1.00	0.00	15.75	18.03	2.00	0.00	1.00	0.00
15.91	14.64	2.00	0.00	1.00	0.00	16.08	12.20	2.00	0.00	1.00	0.00
16.24	13.70	2.00	0.00	1.00	0.00	16.40	16.61	2.00	0.00	1.00	0.00
16.57	17.95	2.00	0.00	1.00	0.00	16.73	18.13	2.00	0.00	1.00	0.00
16.90	16.09	2.00	0.00	1.00	0.00	17.06	13.37	2.00	0.00	1.00	0.00
17.23	17.62	2.00	0.00	1.00	0.00	17.39	13.98	1.86	0.00	1.00	0.00
17.55	7.87	1.30	0.00	1.00	0.00	17.72	6.79	0.99	0.00	1.00	0.00
17.88	7.71	0.97	0.00	1.00	0.00	18.05	7.45	0.99	0.00	1.00	0.00
18.21	7.31	1.03	0.00	1.00	0.00	18.37	8.46	1.09	0.00	1.00	0.00
18.54	8.90	1.07	0.00	1.00	0.00	18.70	7.14	1.00	0.00	1.00	0.00
18.87	7.24	0.94	0.00	1.00	0.00	19.03	7.45	0.95	0.00	1.00	0.00
19.19	7.43	0.95	0.00	1.00	0.00	19.36	7.29	0.96	0.00	1.00	0.00
19.52	7.73	0.99	0.00	1.00	0.00	19.69	8.28	1.02	0.00	1.00	0.00
19.85	8.03	1.00	0.00	1.00	0.00	20.01	7.33	0.95	0.00	1.00	0.00
20.18	7.09	0.97	0.00	1.00	0.00	20.34	8.31	1.09	0.00	1.00	0.00
20.51	9.75	1.21	0.00	1.00	0.00	20.67	9.94	1.29	0.00	1.00	0.00
20.83	10.36	1.30	0.00	1.00	0.00	21.00	10.22	1.31	0.00	1.00	0.00
21.16	10.20	1.38	0.00	1.00	0.00	21.33	12.04	1.58	0.00	1.00	0.00
21.49	14.87	2.00	0.00	1.00	0.00	21.65	84.13	2.00	0.00	1.00	0.00
21.82	102.48	2.00	0.00	1.00	0.00	21.98	102.25	2.00	0.00	1.00	0.00
22.15	96.06	2.00	0.00	1.00	0.00	22.31	83.70	2.00	0.00	1.00	0.00
22.47	15.59	2.00	0.00	1.00	0.00	22.64	9.18	1.29	0.00	1.00	0.00
22.80	8.94	1.02	0.00	1.00	0.00	22.97	8.92	1.03	0.00	1.00	0.00
23.13	9.11	1.04	0.00	1.00	0.00	23.30	9.08	1.05	0.00	1.00	0.00
23.46	9.17	1.07	0.00	1.00	0.00	23.62	9.68	1.12	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
23.79	10.50	1.17	0.00	1.00	0.00	23.95	10.27	1.15	0.00	1.00	0.00
24.12	9.29	1.10	0.00	1.00	0.00	24.28	9.16	1.04	0.00	1.00	0.00
24.44	8.72	1.01	0.00	1.00	0.00	24.61	8.70	1.00	0.00	1.00	0.00
24.77	8.88	1.21	0.00	1.00	0.00	24.94	13.55	2.00	0.00	1.00	0.00
25.10	30.21	2.00	0.00	1.00	0.00	25.26	113.27	2.00	0.00	1.00	0.00
25.43	141.20	2.00	0.00	1.00	0.00	25.59	148.11	2.00	0.00	1.00	0.00
25.76	118.12	0.38	2.70	1.00	0.06	25.92	84.59	0.26	3.80	1.00	0.07
26.08	17.66	2.00	0.00	1.00	0.00	26.25	17.71	1.72	0.00	1.00	0.00
26.41	12.09	1.39	0.00	1.00	0.00	26.58	9.82	1.02	0.00	1.00	0.00
26.74	8.48	0.89	0.00	1.00	0.00	26.90	8.15	0.81	0.00	1.00	0.00
27.07	7.42	0.81	0.00	1.00	0.00	27.23	7.81	0.82	0.00	1.00	0.00
27.40	8.29	0.91	0.00	1.00	0.00	27.56	9.48	1.02	0.00	1.00	0.00
27.72	10.66	1.25	0.00	1.00	0.00	27.89	14.32	1.82	0.00	1.00	0.00
28.05	24.49	2.00	0.00	1.00	0.00	28.22	95.80	0.29	3.36	1.00	0.07
28.38	118.07	0.37	2.70	1.00	0.05	28.54	97.55	0.29	3.30	1.00	0.06
28.71	21.81	2.00	0.00	1.00	0.00	28.87	12.92	1.54	0.00	1.00	0.00
29.04	9.46	1.06	0.00	1.00	0.00	29.20	8.26	0.90	0.00	1.00	0.00
29.36	8.04	0.83	0.00	1.00	0.00	29.53	7.93	0.81	0.00	1.00	0.00
29.69	7.81	0.78	0.00	1.00	0.00	29.86	7.31	0.76	0.00	1.00	0.00
30.02	7.30	0.75	0.00	1.00	0.00	30.19	7.47	0.77	0.00	1.00	0.00
30.35	7.75	0.81	0.00	1.00	0.00	30.51	8.41	0.89	0.00	1.00	0.00
30.68	9.64	1.14	0.00	1.00	0.00	30.84	14.51	1.75	0.00	1.00	0.00
31.01	25.10	2.00	0.00	1.00	0.00	31.17	29.42	2.00	0.00	1.00	0.00
31.33	32.87	2.00	0.00	1.00	0.00	31.50	35.92	2.00	0.00	1.00	0.00
31.66	114.30	2.00	0.00	1.00	0.00	31.83	147.84	2.00	0.00	1.00	0.00
31.99	143.88	2.00	0.00	1.00	0.00	32.15	139.96	2.00	0.00	1.00	0.00
32.32	136.41	0.49	2.30	1.00	0.05	32.48	144.36	0.57	2.09	1.00	0.04
32.65	144.60	0.57	2.08	1.00	0.04	32.81	150.85	2.00	0.00	1.00	0.00
32.97	159.17	2.00	0.00	1.00	0.00	33.14	160.70	2.00	0.00	1.00	0.00
33.30	149.05	2.00	0.00	1.00	0.00	33.47	112.19	2.00	0.00	1.00	0.00
33.63	27.05	2.00	0.00	1.00	0.00	33.79	30.21	2.00	0.00	1.00	0.00
33.96	59.53	2.00	0.00	1.00	0.00	34.12	58.75	2.00	0.00	1.00	0.00
34.29	44.78	2.00	0.00	1.00	0.00	34.45	22.15	2.00	0.00	1.00	0.00
34.61	16.18	1.95	0.00	1.00	0.00	34.78	22.95	2.00	0.00	1.00	0.00
34.94	46.42	2.00	0.00	1.00	0.00	35.11	38.37	2.00	0.00	1.00	0.00
35.27	20.80	2.00	0.00	1.00	0.00	35.43	93.11	2.00	0.00	1.00	0.00
35.60	190.87	2.00	0.00	1.00	0.00	35.76	191.34	2.00	0.00	1.00	0.00
35.93	172.89	2.00	0.00	1.00	0.00	36.09	171.93	1.24	0.01	1.00	0.00
36.26	176.63	1.48	0.00	1.00	0.00	36.42	202.98	2.00	0.00	1.00	0.00
36.58	197.70	2.00	0.00	1.00	0.00	36.75	192.71	2.00	0.00	1.00	0.00
36.91	193.54	2.00	0.00	1.00	0.00	37.08	215.46	2.00	0.00	1.00	0.00
37.24	231.17	2.00	0.00	1.00	0.00	37.40	240.21	2.00	0.00	1.00	0.00
37.57	228.18	2.00	0.00	1.00	0.00	37.73	243.38	2.00	0.00	1.00	0.00
37.90	254.00	2.00	0.00	1.00	0.00	38.06	254.00	2.00	0.00	1.00	0.00
38.22	254.00	2.00	0.00	1.00	0.00	38.39	254.00	2.00	0.00	1.00	0.00
38.55	231.42	2.00	0.00	1.00	0.00	38.72	241.89	2.00	0.00	1.00	0.00
38.88	254.00	2.00	0.00	1.00	0.00	39.04	254.00	2.00	0.00	1.00	0.00
39.21	233.93	2.00	0.00	1.00	0.00	39.37	253.08	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
39.54	254.00	2.00	0.00	1.00	0.00	39.70	254.00	2.00	0.00	1.00	0.00
39.86	254.00	2.00	0.00	1.00	0.00	40.03	251.03	2.00	0.00	1.00	0.00
40.19	254.00	2.00	0.00	1.00	0.00	40.36	254.00	2.00	0.00	1.00	0.00
40.52	254.00	2.00	0.00	1.00	0.00	40.68	254.00	2.00	0.00	1.00	0.00
40.85	254.00	2.00	0.00	1.00	0.00	41.01	254.00	2.00	0.00	1.00	0.00
41.18	254.00	2.00	0.00	1.00	0.00	41.34	254.00	2.00	0.00	1.00	0.00
41.50	254.00	2.00	0.00	1.00	0.00	41.67	254.00	2.00	0.00	1.00	0.00
41.83	254.00	2.00	0.00	1.00	0.00	42.00	254.00	2.00	0.00	1.00	0.00
42.16	254.00	2.00	0.00	1.00	0.00	42.32	254.00	2.00	0.00	1.00	0.00
42.49	254.00	2.00	0.00	1.00	0.00	42.65	254.00	2.00	0.00	1.00	0.00
42.82	195.61	2.00	0.00	1.00	0.00	42.98	230.85	2.00	0.00	1.00	0.00
43.15	210.93	2.00	0.00	1.00	0.00	43.31	200.79	2.00	0.00	1.00	0.00
43.47	223.27	2.00	0.00	1.00	0.00	43.64	253.23	2.00	0.00	1.00	0.00
43.80	254.00	2.00	0.00	1.00	0.00	43.97	254.00	2.00	0.00	1.00	0.00
44.13	254.00	2.00	0.00	1.00	0.00	44.29	250.63	2.00	0.00	1.00	0.00
44.46	222.02	2.00	0.00	1.00	0.00	44.62	196.60	2.00	0.00	1.00	0.00
44.79	142.72	2.00	0.00	1.00	0.00	44.95	33.82	2.00	0.00	1.00	0.00
45.11	30.93	2.00	0.00	1.00	0.00	45.28	25.07	2.00	0.00	1.00	0.00
45.44	45.71	2.00	0.00	1.00	0.00	45.61	47.29	2.00	0.00	1.00	0.00
45.77	29.86	2.00	0.00	1.00	0.00	45.93	29.27	2.00	0.00	1.00	0.00
46.10	38.60	2.00	0.00	1.00	0.00	46.26	32.73	2.00	0.00	1.00	0.00
46.43	28.46	2.00	0.00	1.00	0.00	46.59	35.60	2.00	0.00	1.00	0.00
46.75	46.91	2.00	0.00	1.00	0.00	46.92	19.65	2.00	0.00	1.00	0.00
47.08	13.70	1.24	0.00	1.00	0.00	47.25	12.83	1.02	0.00	1.00	0.00
47.41	12.42	1.04	0.00	1.00	0.00	47.57	14.41	1.17	0.00	1.00	0.00
47.74	17.02	1.39	0.00	1.00	0.00	47.90	19.80	1.61	0.00	1.00	0.00
48.07	21.79	1.82	0.00	1.00	0.00	48.23	24.03	1.97	0.00	1.00	0.00
48.39	24.85	2.00	0.00	1.00	0.00	48.56	24.57	2.00	0.00	1.00	0.00
48.72	26.74	2.00	0.00	1.00	0.00	48.89	31.99	2.00	0.00	1.00	0.00
49.05	45.81	2.00	0.00	1.00	0.00	49.22	41.16	2.00	0.00	1.00	0.00
49.38	54.96	2.00	0.00	1.00	0.00	49.54	146.01	2.00	0.00	1.00	0.00
49.71	199.29	2.00	0.00	1.00	0.00	49.87	190.96	2.00	0.00	1.00	0.00
50.04	183.14	2.00	0.00	1.00	0.00	50.20	193.63	2.00	0.00	1.00	0.00
50.36	194.50	2.00	0.00	1.00	0.00	50.53	178.32	1.56	0.00	1.00	0.00
50.69	168.14	1.08	0.06	1.00	0.00	50.86	160.98	0.86	0.45	1.00	0.01
51.02	211.70	2.00	0.00	1.00	0.00	51.18	235.85	2.00	0.00	1.00	0.00
51.35	218.40	2.00	0.00	1.00	0.00	51.51	205.09	2.00	0.00	1.00	0.00
51.68	211.27	2.00	0.00	1.00	0.00	51.84	199.89	2.00	0.00	1.00	0.00
52.00	143.96	2.00	0.00	1.00	0.00	52.17	121.06	2.00	0.00	1.00	0.00
52.33	33.85	2.00	0.00	1.00	0.00	52.50	19.85	2.00	0.00	1.00	0.00
52.66	15.68	1.28	0.00	1.00	0.00	52.82	13.60	1.07	0.00	1.00	0.00
52.99	12.92	1.05	0.00	1.00	0.00	53.15	15.03	1.16	0.00	1.00	0.00
53.32	17.21	1.37	0.00	1.00	0.00	53.48	20.15	1.75	0.00	1.00	0.00
53.64	28.10	2.00	0.00	1.00	0.00	53.81	39.43	2.00	0.00	1.00	0.00
53.97	50.39	2.00	0.00	1.00	0.00	54.14	55.16	2.00	0.00	1.00	0.00
54.30	133.21	2.00	0.00	1.00	0.00	54.46	165.44	2.00	0.00	1.00	0.00
54.63	246.22	2.00	0.00	1.00	0.00	54.79	241.44	2.00	0.00	1.00	0.00
54.96	254.00	2.00	0.00	1.00	0.00	55.12	254.00	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

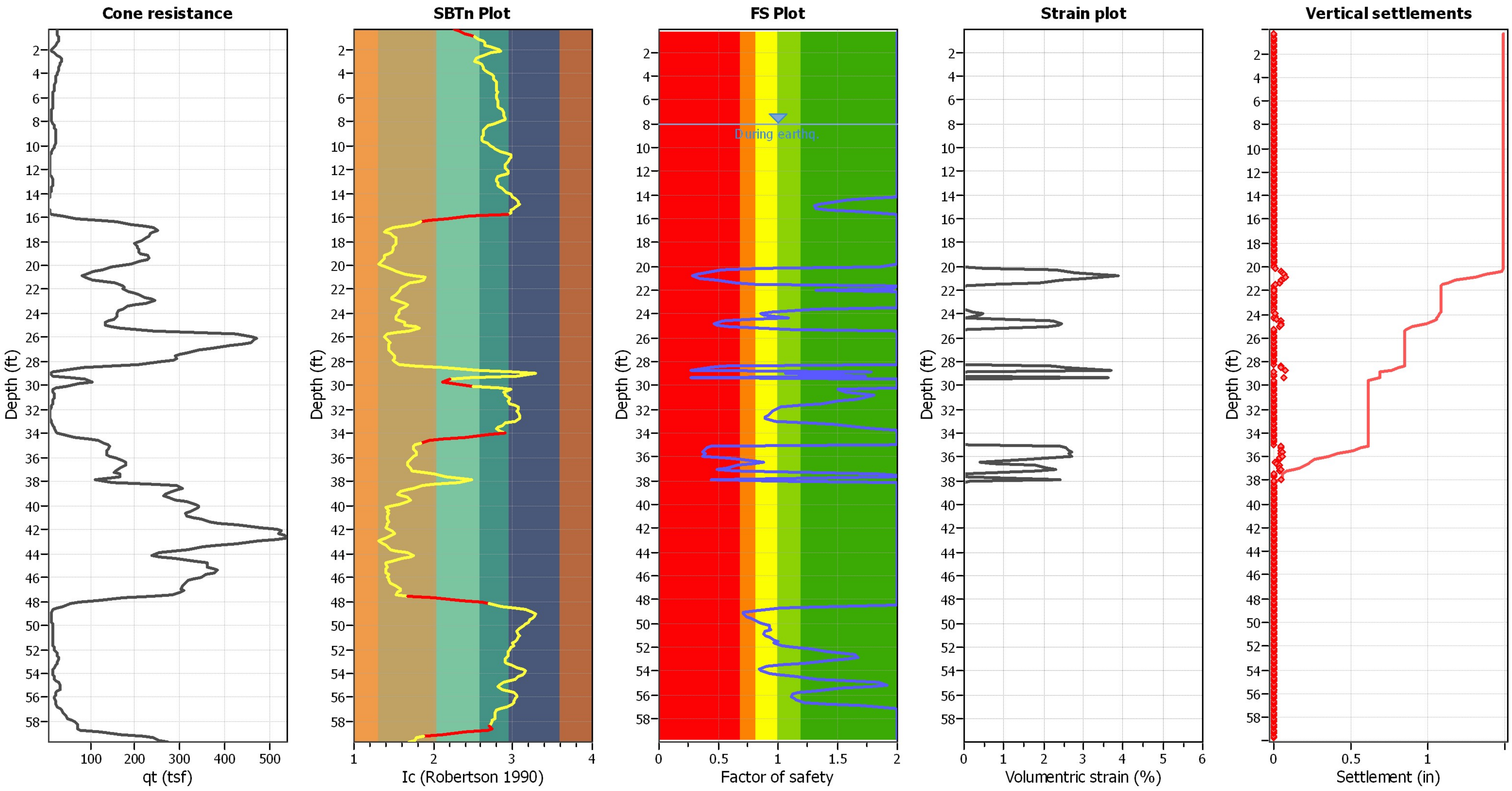
Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
55.28	254.00	2.00	0.00	1.00	0.00	55.45	254.00	2.00	0.00	1.00	0.00
55.61	254.00	2.00	0.00	1.00	0.00	55.78	254.00	2.00	0.00	1.00	0.00
55.94	254.00	2.00	0.00	1.00	0.00	56.11	254.00	2.00	0.00	1.00	0.00
56.27	254.00	2.00	0.00	1.00	0.00	56.43	254.00	2.00	0.00	1.00	0.00
56.60	254.00	2.00	0.00	1.00	0.00	56.76	254.00	2.00	0.00	1.00	0.00
56.93	254.00	2.00	0.00	1.00	0.00	57.09	254.00	2.00	0.00	1.00	0.00
57.25	254.00	2.00	0.00	1.00	0.00	57.42	254.00	2.00	0.00	1.00	0.00
57.58	254.00	2.00	0.00	1.00	0.00	57.75	245.38	2.00	0.00	1.00	0.00
57.91	213.66	2.00	0.00	1.00	0.00	58.07	210.46	2.00	0.00	1.00	0.00
58.24	224.29	2.00	0.00	1.00	0.00	58.40	236.51	2.00	0.00	1.00	0.00
58.57	219.68	2.00	0.00	1.00	0.00	58.73	177.23	2.00	0.00	1.00	0.00
58.89	140.99	2.00	0.00	1.00	0.00	59.06	117.85	2.00	0.00	1.00	0.00
59.22	15.57	2.00	0.00	1.00	0.00	59.39	19.65	1.40	0.00	1.00	0.00
59.55	19.27	1.49	0.00	1.00	0.00	59.71	18.61	1.40	0.00	1.00	0.00
59.88	16.28	1.32	0.00	1.00	0.00	60.04	15.22	1.20	0.00	1.00	0.00

Total estimated settlement: 0.53

Abbreviations

$Q_{tn,cs}$: Equivalent clean sand normalized cone resistance
FS: Factor of safety against liquefaction
e_v (%): Post-liquefaction volumentric strain
DF: e_v depth weighting factor
Settlement: Calculated settlement

Estimation of post-earthquake settlements



Abbreviations

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

:: **Post-earthquake settlement due to soil liquefaction** ::

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
8.04	20.14	2.00	0.00	1.00	0.00	8.20	22.70	2.00	0.00	1.00	0.00
8.37	26.83	2.00	0.00	1.00	0.00	8.53	31.52	2.00	0.00	1.00	0.00
8.69	33.10	2.00	0.00	1.00	0.00	8.86	32.95	2.00	0.00	1.00	0.00
9.02	32.17	2.00	0.00	1.00	0.00	9.19	31.90	2.00	0.00	1.00	0.00
9.35	30.87	2.00	0.00	1.00	0.00	9.51	32.29	2.00	0.00	1.00	0.00
9.68	32.13	2.00	0.00	1.00	0.00	9.84	30.44	2.00	0.00	1.00	0.00
10.01	28.23	2.00	0.00	1.00	0.00	10.17	26.18	2.00	0.00	1.00	0.00
10.34	23.73	2.00	0.00	1.00	0.00	10.50	21.03	2.00	0.00	1.00	0.00
10.66	15.95	2.00	0.00	1.00	0.00	10.83	12.95	2.00	0.00	1.00	0.00
10.99	13.57	2.00	0.00	1.00	0.00	11.16	13.39	2.00	0.00	1.00	0.00
11.32	13.21	2.00	0.00	1.00	0.00	11.48	14.47	2.00	0.00	1.00	0.00
11.65	14.81	2.00	0.00	1.00	0.00	11.81	14.11	2.00	0.00	1.00	0.00
11.98	12.88	2.00	0.00	1.00	0.00	12.14	11.27	2.00	0.00	1.00	0.00
12.30	11.36	2.00	0.00	1.00	0.00	12.47	14.43	2.00	0.00	1.00	0.00
12.63	18.45	2.00	0.00	1.00	0.00	12.80	21.38	2.00	0.00	1.00	0.00
12.96	21.79	2.00	0.00	1.00	0.00	13.12	21.83	2.00	0.00	1.00	0.00
13.29	20.64	2.00	0.00	1.00	0.00	13.45	19.70	2.00	0.00	1.00	0.00
13.62	18.14	2.00	0.00	1.00	0.00	13.78	16.71	2.00	0.00	1.00	0.00
13.94	14.55	2.00	0.00	1.00	0.00	14.11	12.51	2.00	0.00	1.00	0.00
14.27	11.47	1.77	0.00	1.00	0.00	14.44	10.56	1.58	0.00	1.00	0.00
14.60	9.53	1.43	0.00	1.00	0.00	14.76	9.00	1.34	0.00	1.00	0.00
14.93	9.10	1.31	0.00	1.00	0.00	15.09	9.07	1.32	0.00	1.00	0.00
15.26	9.41	1.43	0.00	1.00	0.00	15.42	11.22	1.68	0.00	1.00	0.00
15.58	13.73	2.00	0.00	1.00	0.00	15.75	15.96	2.00	0.00	1.00	0.00
15.91	78.07	2.00	0.00	1.00	0.00	16.08	144.92	2.00	0.00	1.00	0.00
16.24	162.84	2.00	0.00	1.00	0.00	16.40	195.25	2.00	0.00	1.00	0.00
16.57	213.63	2.00	0.00	1.00	0.00	16.73	224.06	2.00	0.00	1.00	0.00
16.90	254.00	2.00	0.00	1.00	0.00	17.06	254.00	2.00	0.00	1.00	0.00
17.23	250.21	2.00	0.00	1.00	0.00	17.39	242.70	2.00	0.00	1.00	0.00
17.55	244.17	2.00	0.00	1.00	0.00	17.72	239.14	2.00	0.00	1.00	0.00
17.88	225.93	2.00	0.00	1.00	0.00	18.05	216.18	2.00	0.00	1.00	0.00
18.21	210.67	2.00	0.00	1.00	0.00	18.37	203.11	2.00	0.00	1.00	0.00
18.54	230.16	2.00	0.00	1.00	0.00	18.70	213.57	2.00	0.00	1.00	0.00
18.87	208.53	2.00	0.00	1.00	0.00	19.03	229.51	2.00	0.00	1.00	0.00
19.19	229.03	2.00	0.00	1.00	0.00	19.36	248.69	2.00	0.00	1.00	0.00
19.52	237.17	2.00	0.00	1.00	0.00	19.69	218.77	2.00	0.00	1.00	0.00
19.85	201.90	2.00	0.00	1.00	0.00	20.01	178.30	1.85	0.00	1.00	0.00
20.18	155.54	0.85	0.51	1.00	0.01	20.34	134.21	0.53	2.32	1.00	0.04
20.51	115.27	0.39	2.77	1.00	0.06	20.67	97.38	0.32	3.30	1.00	0.06
20.83	83.11	0.28	3.86	1.00	0.07	21.00	106.04	0.35	3.02	1.00	0.06
21.16	128.73	0.47	2.45	1.00	0.05	21.33	144.84	0.64	1.92	1.00	0.04
21.49	154.55	0.81	0.69	1.00	0.01	21.65	195.15	2.00	0.00	1.00	0.00
21.82	180.41	1.97	0.00	1.00	0.00	21.98	170.17	1.31	0.00	1.00	0.00
22.15	182.87	2.00	0.00	1.00	0.00	22.31	210.81	2.00	0.00	1.00	0.00
22.47	207.39	2.00	0.00	1.00	0.00	22.64	211.87	2.00	0.00	1.00	0.00
22.80	245.02	2.00	0.00	1.00	0.00	22.97	254.00	2.00	0.00	1.00	0.00
23.13	222.72	2.00	0.00	1.00	0.00	23.30	188.53	2.00	0.00	1.00	0.00
23.46	185.30	2.00	0.00	1.00	0.00	23.62	170.83	1.32	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
23.79	160.66	0.94	0.23	1.00	0.00
24.12	158.49	0.88	0.40	1.00	0.01
24.44	152.60	0.75	1.11	1.00	0.02
24.77	129.21	0.46	2.45	1.00	0.05
25.10	142.24	0.58	2.11	1.00	0.04
25.43	186.29	2.00	0.00	1.00	0.00
25.76	254.00	2.00	0.00	1.00	0.00
26.08	254.00	2.00	0.00	1.00	0.00
26.41	254.00	2.00	0.00	1.00	0.00
26.74	254.00	2.00	0.00	1.00	0.00
27.07	254.00	2.00	0.00	1.00	0.00
27.40	254.00	2.00	0.00	1.00	0.00
27.72	254.00	2.00	0.00	1.00	0.00
28.05	247.01	2.00	0.00	1.00	0.00
28.38	138.96	0.53	2.23	1.00	0.04
28.71	86.80	0.27	3.70	1.00	0.08
29.04	10.28	1.07	0.00	1.00	0.00
29.36	88.60	0.27	3.63	1.00	0.07
29.69	152.33	2.00	0.00	1.00	0.00
30.02	94.46	2.00	0.00	1.00	0.00
30.35	13.93	1.50	0.00	1.00	0.00
30.68	16.98	1.69	0.00	1.00	0.00
31.01	18.37	1.71	0.00	1.00	0.00
31.33	15.66	1.51	0.00	1.00	0.00
31.66	11.26	1.17	0.00	1.00	0.00
31.99	10.13	0.97	0.00	1.00	0.00
32.32	9.74	0.93	0.00	1.00	0.00
32.65	9.25	0.89	0.00	1.00	0.00
32.97	9.49	0.97	0.00	1.00	0.00
33.30	16.65	1.46	0.00	1.00	0.00
33.63	17.30	1.80	0.00	1.00	0.00
33.96	21.13	2.00	0.00	1.00	0.00
34.29	125.55	2.00	0.00	1.00	0.00
34.61	143.23	2.00	0.00	1.00	0.00
34.94	125.93	2.00	0.00	1.00	0.00
35.27	123.58	0.40	2.57	1.00	0.05
35.60	117.99	0.37	2.70	1.00	0.06
35.93	117.39	0.36	2.72	1.00	0.06
36.26	151.81	0.68	1.62	1.00	0.03
36.58	157.66	0.80	0.75	1.00	0.01
36.91	144.58	0.58	2.08	1.00	0.04
37.24	149.86	0.65	1.82	1.00	0.03
37.57	254.00	2.00	0.00	1.00	0.00
37.90	130.23	0.44	2.43	1.00	0.05
38.22	254.00	2.00	0.00	1.00	0.00
38.55	254.00	2.00	0.00	1.00	0.00
38.88	246.08	2.00	0.00	1.00	0.00
39.21	230.07	2.00	0.00	1.00	0.00
23.95	157.46	0.86	0.49	1.00	0.01
24.28	165.51	1.09	0.05	1.00	0.00
24.61	137.81	0.54	2.25	1.00	0.05
24.94	132.50	0.48	2.37	1.00	0.05
25.26	166.40	1.11	0.04	1.00	0.00
25.59	254.00	2.00	0.00	1.00	0.00
25.92	254.00	2.00	0.00	1.00	0.00
26.25	254.00	2.00	0.00	1.00	0.00
26.58	254.00	2.00	0.00	1.00	0.00
26.90	254.00	2.00	0.00	1.00	0.00
27.23	254.00	2.00	0.00	1.00	0.00
27.56	254.00	2.00	0.00	1.00	0.00
27.89	254.00	2.00	0.00	1.00	0.00
28.22	205.37	2.00	0.00	1.00	0.00
28.54	126.34	0.43	2.51	1.00	0.05
28.87	13.59	1.78	0.00	1.00	0.00
29.20	9.14	1.73	0.00	1.00	0.00
29.53	187.71	2.00	0.00	1.00	0.00
29.86	138.70	2.00	0.00	1.00	0.00
30.19	15.52	2.00	0.00	1.00	0.00
30.51	14.73	1.57	0.00	1.00	0.00
30.84	17.04	1.80	0.00	1.00	0.00
31.17	14.87	1.64	0.00	1.00	0.00
31.50	14.45	1.37	0.00	1.00	0.00
31.83	10.15	1.02	0.00	1.00	0.00
32.15	9.75	0.96	0.00	1.00	0.00
32.48	9.63	0.91	0.00	1.00	0.00
32.81	9.15	0.89	0.00	1.00	0.00
33.14	11.33	1.25	0.00	1.00	0.00
33.47	15.46	1.68	0.00	1.00	0.00
33.79	20.20	1.99	0.00	1.00	0.00
34.12	23.38	2.00	0.00	1.00	0.00
34.45	143.05	2.00	0.00	1.00	0.00
34.78	124.60	2.00	0.00	1.00	0.00
35.11	130.20	0.44	2.43	1.00	0.05
35.43	120.74	0.38	2.64	1.00	0.05
35.76	121.55	0.39	2.62	1.00	0.05
36.09	142.06	0.55	2.16	1.00	0.04
36.42	160.86	0.87	0.41	1.00	0.01
36.75	149.95	0.65	1.81	1.00	0.04
37.08	136.67	0.49	2.29	1.00	0.05
37.40	181.50	1.82	0.00	1.00	0.00
37.73	210.90	2.00	0.00	1.00	0.00
38.06	165.34	1.00	0.13	1.00	0.00
38.39	254.00	2.00	0.00	1.00	0.00
38.72	254.00	2.00	0.00	1.00	0.00
39.04	231.60	2.00	0.00	1.00	0.00
39.37	231.01	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
39.54	254.00	2.00	0.00	1.00	0.00	39.70	254.00	2.00	0.00	1.00	0.00
39.86	254.00	2.00	0.00	1.00	0.00	40.03	254.00	2.00	0.00	1.00	0.00
40.19	254.00	2.00	0.00	1.00	0.00	40.36	254.00	2.00	0.00	1.00	0.00
40.52	254.00	2.00	0.00	1.00	0.00	40.68	254.00	2.00	0.00	1.00	0.00
40.85	254.00	2.00	0.00	1.00	0.00	41.01	254.00	2.00	0.00	1.00	0.00
41.18	254.00	2.00	0.00	1.00	0.00	41.34	254.00	2.00	0.00	1.00	0.00
41.50	254.00	2.00	0.00	1.00	0.00	41.67	254.00	2.00	0.00	1.00	0.00
41.83	254.00	2.00	0.00	1.00	0.00	42.00	254.00	2.00	0.00	1.00	0.00
42.16	254.00	2.00	0.00	1.00	0.00	42.32	254.00	2.00	0.00	1.00	0.00
42.49	254.00	2.00	0.00	1.00	0.00	42.65	254.00	2.00	0.00	1.00	0.00
42.82	254.00	2.00	0.00	1.00	0.00	42.98	254.00	2.00	0.00	1.00	0.00
43.15	254.00	2.00	0.00	1.00	0.00	43.31	254.00	2.00	0.00	1.00	0.00
43.47	254.00	2.00	0.00	1.00	0.00	43.64	254.00	2.00	0.00	1.00	0.00
43.80	241.19	2.00	0.00	1.00	0.00	43.97	207.21	2.00	0.00	1.00	0.00
44.13	191.47	2.00	0.00	1.00	0.00	44.29	199.35	2.00	0.00	1.00	0.00
44.46	248.11	2.00	0.00	1.00	0.00	44.62	254.00	2.00	0.00	1.00	0.00
44.79	254.00	2.00	0.00	1.00	0.00	44.95	254.00	2.00	0.00	1.00	0.00
45.11	254.00	2.00	0.00	1.00	0.00	45.28	254.00	2.00	0.00	1.00	0.00
45.44	254.00	2.00	0.00	1.00	0.00	45.61	254.00	2.00	0.00	1.00	0.00
45.77	254.00	2.00	0.00	1.00	0.00	45.93	254.00	2.00	0.00	1.00	0.00
46.10	254.00	2.00	0.00	1.00	0.00	46.26	254.00	2.00	0.00	1.00	0.00
46.43	254.00	2.00	0.00	1.00	0.00	46.59	254.00	2.00	0.00	1.00	0.00
46.75	247.58	2.00	0.00	1.00	0.00	46.92	254.00	2.00	0.00	1.00	0.00
47.08	254.00	2.00	0.00	1.00	0.00	47.25	254.00	2.00	0.00	1.00	0.00
47.41	240.05	2.00	0.00	1.00	0.00	47.57	209.74	2.00	0.00	1.00	0.00
47.74	193.03	2.00	0.00	1.00	0.00	47.90	133.21	2.00	0.00	1.00	0.00
48.07	60.73	2.00	0.00	1.00	0.00	48.23	42.19	2.00	0.00	1.00	0.00
48.39	26.29	2.00	0.00	1.00	0.00	48.56	16.65	1.51	0.00	1.00	0.00
48.72	12.76	1.03	0.00	1.00	0.00	48.89	9.96	0.80	0.00	1.00	0.00
49.05	9.22	0.71	0.00	1.00	0.00	49.22	9.57	0.70	0.00	1.00	0.00
49.38	9.77	0.73	0.00	1.00	0.00	49.54	10.12	0.76	0.00	1.00	0.00
49.71	10.47	0.79	0.00	1.00	0.00	49.87	10.67	0.84	0.00	1.00	0.00
50.04	11.68	0.87	0.00	1.00	0.00	50.20	11.44	0.93	0.00	1.00	0.00
50.36	12.01	0.93	0.00	1.00	0.00	50.53	11.05	0.93	0.00	1.00	0.00
50.69	10.96	0.89	0.00	1.00	0.00	50.86	10.37	0.88	0.00	1.00	0.00
51.02	10.36	0.89	0.00	1.00	0.00	51.18	11.42	0.93	0.00	1.00	0.00
51.35	11.62	0.97	0.00	1.00	0.00	51.51	11.68	0.99	0.00	1.00	0.00
51.68	12.38	0.96	0.00	1.00	0.00	51.84	10.43	1.03	0.00	1.00	0.00
52.00	14.23	1.13	0.00	1.00	0.00	52.17	16.02	1.27	0.00	1.00	0.00
52.33	15.49	1.42	0.00	1.00	0.00	52.50	19.18	1.54	0.00	1.00	0.00
52.66	20.33	1.64	0.00	1.00	0.00	52.82	19.27	1.67	0.00	1.00	0.00
52.99	20.34	1.58	0.00	1.00	0.00	53.15	17.40	1.44	0.00	1.00	0.00
53.32	14.21	1.22	0.00	1.00	0.00	53.48	12.76	1.02	0.00	1.00	0.00
53.64	10.41	0.90	0.00	1.00	0.00	53.81	9.83	0.84	0.00	1.00	0.00
53.97	10.52	0.86	0.00	1.00	0.00	54.14	11.85	0.93	0.00	1.00	0.00
54.30	12.40	1.03	0.00	1.00	0.00	54.46	14.15	1.15	0.00	1.00	0.00
54.63	15.55	1.33	0.00	1.00	0.00	54.79	18.38	1.59	0.00	1.00	0.00
54.96	22.83	1.84	0.00	1.00	0.00	55.12	24.62	1.92	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$q_{c1N,cs}$	FS	e_v (%)	DF	Settlement (in)
55.28	21.40	1.78	0.00	1.00	0.00	55.45	18.36	1.53	0.00	1.00	0.00
55.61	15.78	1.31	0.00	1.00	0.00	55.78	14.06	1.17	0.00	1.00	0.00
55.94	12.99	1.12	0.00	1.00	0.00	56.11	13.74	1.11	0.00	1.00	0.00
56.27	13.52	1.14	0.00	1.00	0.00	56.43	13.43	1.15	0.00	1.00	0.00
56.60	14.04	1.22	0.00	1.00	0.00	56.76	15.77	1.42	0.00	1.00	0.00
56.93	20.19	1.71	0.00	1.00	0.00	57.09	23.80	1.98	0.00	1.00	0.00
57.25	25.13	2.00	0.00	1.00	0.00	57.42	26.62	2.00	0.00	1.00	0.00
57.58	27.60	2.00	0.00	1.00	0.00	57.75	32.99	2.00	0.00	1.00	0.00
57.91	37.83	2.00	0.00	1.00	0.00	58.07	45.56	2.00	0.00	1.00	0.00
58.24	51.72	2.00	0.00	1.00	0.00	58.40	50.86	2.00	0.00	1.00	0.00
58.57	50.19	2.00	0.00	1.00	0.00	58.73	49.23	2.00	0.00	1.00	0.00
58.89	130.41	2.00	0.00	1.00	0.00	59.06	224.78	2.00	0.00	1.00	0.00
59.22	191.33	2.00	0.00	1.00	0.00	59.39	180.58	2.00	0.00	1.00	0.00
59.55	201.85	2.00	0.00	1.00	0.00	59.71	216.56	2.00	0.00	1.00	0.00

Total estimated settlement: 1.49

Abbreviations

$Q_{tn,cs}$: Equivalent clean sand normalized cone resistance
FS: Factor of safety against liquefaction
e_v (%): Post-liquefaction volumentric strain
DF: e_v depth weighting factor
Settlement: Calculated settlement

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