Appendix C

Geotechnical Engineering Study

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April 9, 2021 File No.: 304309-001

Mr. John Suppes
Clarum Homes
P.O. Box 60970
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PROJECT: PROPOSED SINGLE FAMILY RESIDENCE and ADU
575 LOS TRANCOS ROAD
PALO ALTO, CALIFORNIA

SUBJECT: Geotechnical Engineering Study

REF.: Revised Proposal to Perform a Geotechnical Engineering Study and Liquefaction Analysis, Proposed Single Family Residence and ADU, 575 Los Trancos Road, Palo Alto, California, by Earth Systems Pacific, dated November 20, 2020, revised December 4, 2020.

Soil Investigation, Proposed Single-Family Residence, Los Trancos Property (APN 182-46-003), Palo Alto, California, by Harding Lawson Associates, dated January 26, 1990.

Dear Mr. Suppes:

In accordance with your authorization of the above referenced proposal, this geotechnical engineering study has been prepared by Earth Systems Pacific (Earth Systems) for use in the development of plans and specifications for the proposed single family residence and accessory dwelling unit (ADU) in Palo Alto, California. Preliminary geotechnical recommendations for site preparation and grading; foundations; slabs-on-grade; exterior flatwork; swimming pool; utility trench backfill; site drainage and finish improvements; and observation and testing are presented herein.

We appreciate the opportunity to have provided services for this project and look forward to working with you again in the future. Please do not hesitate to contact this office if there are any questions concerning this report.

Sincerely,
Earth Systems Pacific

Bill E Zehrbach

Phillip Penrose
Staff Engineer

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Doc. No.: 2104-004.SER/kt

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1.0 INTRODUCTION

Site Setting

The subject property is an irregular shaped, 5.47-acre parcel located at 575 Los Trancos Road in Palo Alto, California (APN 182-46-012). The site has a latitude of 37.3666°N and a longitude of 122.2012°W. The general location of the site is shown on the Site Location Map (Figure 1).

Site Description

The subject property is located on the west side of Los Trancos Road, about a half mile south of the intersection of Los Trancos Road and Alpine Road. The property is bounded by Los Trancos Road to the east, Los Trancos Creek and Valley Oak Street to the west, an existing residence to the north and undeveloped land to the south.

The property is currently undeveloped. The center of the parcel is covered with grasses and the property borders are covered by trees and dense brush. Los Trancos Creek runs along the western edge of the property. An existing gravel road starts at the northeastern corner of the property off Los Trancos Road and grants access to the property and the neighboring property to the north. The center of the lot, where the proposed developments lie, is mostly flat. The lot slopes towards the creek on the west side and slopes upwards towards Los Trancos Road on the east side.

Planned Development

We understand that you plan to construct a new residence in approximately the center of the parcel. The proposed ADU is expected to be constructed on the southern portion of the parcel and the swimming pool is proposed on the southwestern portion of the parcel. See Figure 2, Site Plan. Based on the preliminary plans by *LNAI Architecture* (dated February 10, 2021), it is our understanding that the new residence will be a two-story building with a partial second story.

Scope of Services

The scope of work for the geotechnical engineering study included a general site reconnaissance, evaluation of the subsurface soil and groundwater conditions from a geotechnical engineering standpoint by drilling borings and laboratory testing of selected samples, engineering analysis of the collected data, and preparation of this report. The analysis and subsequent recommendations were based on our understanding of the proposed development at the subject site.

The report and recommendations are intended to comply with the considerations of Section 1803 of the California Building Code (CBC), 2019 Edition, and common geotechnical engineering practice in this area at this time under similar conditions. The tests were performed in general conformance with the standards noted, as modified by common geotechnical practice in this area at this time under similar conditions.

Preliminary geotechnical recommendations for site preparation and grading, foundations, slabs-on-grade, exterior flatwork, swimming pool, utility trench backfill, site drainage and finish improvements, and geotechnical observation and testing are presented to guide the development of project plans and specifications. It is our intent that this report be used by the client to form the geotechnical basis of the design of the project as described herein, and in the preparation of plans and specifications.

Detailed evaluation of the site geology and potential geologic hazards, and analyses of the soil for mold or other microbial content, asbestos, percolation rates, corrosion potential, radioisotopes, hydrocarbons, or other chemical properties are beyond the scope of this report. This report also does not address issues in the domain of contractors such as, but not limited to, site safety, loss of volume due to stripping of the site, shrinkage of soils during compaction, excavatability, shoring, temporary slope angles, and construction means and methods. Ancillary features such as swimming pools, temporary access roads, fences, light poles, and non-structural fills are not within our scope and are also not addressed.

To verify that pertinent issues have been addressed and to aid in conformance with the intent of this report, it is requested that final grading and foundation plans be submitted to this office for review. In the event that there are any changes in the nature, design, or locations of improvements, or if any assumptions used in the preparation of this report prove to be incorrect, the conclusions and recommendations contained herein should not be considered valid unless the changes are reviewed, and the conclusions of this report are verified or modified in writing by the Geotechnical Engineer. The criteria presented in this report are considered preliminary until such time as they are verified or modified in writing by the Geotechnical Engineer in the field during construction.

2.0 GEOLOGIC SETTING

According to the Geologic Map of the Palo Alto 30’ x 60’ Quadrangle, California (Brabb et. al, 2000), the site is mapped as being underlain by Pleistocene older alluvial fan deposits (Qpoaf). The site is located in a liquefaction hazards zone as delineated by the State of California and the County of Santa Clara.

The entire San Francisco Bay Area is considered to be an active seismic region due to the presence of several active faults. Three northwest-trending major earthquake faults that are responsible for the majority of the movement on the San Andreas fault system extend through the Bay Area. They include the San Andreas fault, the Hayward fault and the Calaveras fault, which are respectively located approximately 0.4 miles to the southwest, 19.3 miles to the northeast and 22.4 miles to the northeast. The Monte Vista-Shannon fault is located approximately 1.4 miles northeast of the site. Using information from recent earthquakes, improved mapping of active faults, and a new model for estimating earthquake probabilities, the 2014 Working Group on California Earthquake Probabilities updated the 30 year earthquake forecast for California. They concluded that there is a 72 percent probability (or likelihood) of at least one earthquake of magnitude 6.7 greater striking somewhere in the San Francisco Bay region before 2043. A summary of the significant faults in the near vicinity of the site are listed below.

Major Active Faults

Fault	Distance from Site (miles)	Probability of $M_w \geq 6.7$ within 30 Years[1]
San Andreas	0.4 (SW)	6%
Monte-Vista Shannon	1.9 (NE)	1%
Hayward	19.3 (NE)	21%
Calaveras	22.4 (NE)	7%

[1] Working Group on California Earthquake Probabilities, 2015

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Previous Geotechnical Studies

Harding Lawson Associates prepared a Soil Investigation for the subject lot dated January 26, 1990. Their investigation included the drilling of 5 exploratory borings on the lot at the approximate locations indicated on Figure 2, Site Plan. The logs of these borings are presented in Appendix A.

Subsurface Exploration (Current)

The subsurface exploration for this study consisted of drilling two exploratory borings at the site on February 23, 2021. The approximate locations of the test borings are shown on (Figure 2).

The borings were advanced to depths of 34 feet below ground surface (bgs). The drilling process consisted of using a truck-mounted drilling rig equipped with 8-inch diameter hollow stem augers. Once reaching the desired depth, a standard Mod-Cal or SPT sampler, connected to steel rods was lowered into the hole. The samplers were driven into undisturbed ground with a 140-pound, safety hammer falling about 30 inches per drop. The samplers were driven up to 18 inches and the hammer blows required to drive every six inches of the samplers were recorded and are presented on the boring logs. The number of blows required to drive the final 12 inches of the sampler into the undisturbed ground were used as Penetration Resistance and this was used to interpret soil consistency/density. The borings were then backfilled with lean cement grout. The boring logs show soil description including: color, major and minor components, USCS classification, changes in soil conditions with depth, moisture content, consistency/density, plasticity, sampler type, and sampling depths and laboratory test results. Copies of the logs of boring drilled for this investigation are presented in Appendix B.

Soils encountered in the borings were logged in general accordance with the Unified Soil Classification System. An Earth Systems engineer prepared the logs and retained samples for laboratory testing.

Subsurface Profile

The borings drilled at the site revealed the presence of loose to very dense sand with variable percentages of clay and gravel. This is consistent with the geological mapping by Brabb et al.(2000). In Boring B-1, the upper 5 feet consisted of medium dense well graded sand with gravel. Below the well-graded sand, a clayey sand layer with variable percentages of gravel was encountered and extended to the bottom of the boring at 34 feet bgs. Some cobbles were encountered in the boring at 7 feet bgs. In Boring B-2, loose clayey sand with gravel was encountered at the surface and extended to 17 feet bgs. The sand became denser at approximately 7 feet bgs. At 17 feet bgs, a medium dense, well graded sand with clay and gravel layer was encountered. The clay content increased at 23 feet and decreased again at 28 feet bgs to well graded sand with clay and gravel, which extended to the bottom of the boring at 34 feet bgs.

Groundwater was encountered at 17 to 18 feet bgs in the borings drilled at the site to the maximum depth of exploration of 34 feet bgs.

Laboratory Testing

Five liner samples were tested to measure moisture content and dry density (ASTM D 2216-17 and D 2937-17), and four samples were tested to determine the percentage of material passing the minus #200 sieve (ASTM D 1140-17). Copies of the laboratory test results are included in Appendix C.

4.0 DATA ANALYSIS

Subsurface Soil Classification

Based on the subsurface data collected as a part of our subsurface exploration and our review of the published geologic literature, the site is assigned to Site Class C (very dense soil and soft rock) as defined by Table 20.3-1 of the ASCE 7-16.

Seismic Design Parameters

The following seismic design parameters represent the general procedure as outlined in Section 1613 of the CBC and in ASCE 7. The values determined below are based on the 2009 National Earthquake Hazard Reduction Program (NEHRP) maps and were obtained using the United States Geological Survey's Design Maps Web Application.

Summary of Seismic Parameters - CBC 2019
(Site Coordinates 37.3859°N, 122.1399°W)

Parameter	Design Value
Site Class	C
Mapped Short Term Spectral Response Parameter, (S_s)	2.549
Mapped 1-second Spectral Response Parameter, (S_1)	1.008
Site Coefficient, (F_a)	1.2
Site Coefficient, (F_v)	1.4
Site Modified Short Term Response Parameter, (S_{Ms})	3.059
Site Modified 1-second Response Parameter, (S_{M1})	1.411
Design Short Term Response Parameter, (S_{Ds})	2.04
Design 1-second Response Parameter, (S_{D1})	0.94
Seismic Design Category	E

Static Settlement

Based on our understanding of the proposed development and because the building loads are anticipated to be fairly light, anticipated static settlements are on the order of 1 inch with a differential settlement of ½ inch.

Liquefaction

Soil liquefaction is a phenomenon where saturated granular soils undergo a substantial loss of strength due to increased pore water pressure resulting from cyclic stress applications induced by earthquakes or other vibrations. In this process, the soil acquires mobility sufficient to permit both vertical and horizontal movements, which may result in significant deformations. Soils most susceptible to liquefaction are loose, uniformly graded, fine-grained sands. In addition, recent literature indicates that fine grained soils may also be susceptible to liquefaction or cyclic strain softening. Examples of highly susceptible fine-grained soil include “non-plastic silts and clayey silts of low plasticity (PI<12) at high water content to liquid limit ratios (w_c/LL>0.85).” Examples of soils moderately susceptible to liquefaction include “clayey silts and silty clays of moderate plasticity (12<PI<18) at natural water content and Liquid Limits ratios (w_c/LL) greater than 0.80.” (Bray and Sancio, 2006). It is generally acknowledged that liquefaction will not affect surface improvements if these deposits are located at a depth greater than 50 feet below the ground surface. In the deeper deposits, the greater overburden pressure is sufficient to prevent liquefaction effects from occurring.

Analysis Parameters

The referenced 1990 report by Harding Lawson Associates, gave a historic groundwater level of 8 feet bgs from an unknown reference, thus we used this value in our analysis. It should be noted that this value is likely conservative. According to United States Geological Survey’s (USGS) Unified Hazard Tool, the predominant earthquake contributor is the San Andreas fault with mean magnitude using deaggregation of 7.8. The liquefaction analysis was performed utilizing the peak ground acceleration of 1.16g (PGAm) based on the Office of Statewide Health Planning and Development Seismic Design Maps Web Application. Any sand-like deposit (Soil Behavior Type Index, I_c < 2.6) below the groundwater table was assumed to be potentially liquefiable. The liquefaction analysis was based on the methodologies suggested by Idriss and Boulanger (2008 and 2014). The loose sand layers above the water table are subject to dry sand settlement. A two-thirds reduction in the PGA was used for the dry sand settlement, thus a separate analysis is presented in Appendix D.

Analysis Results

The calculated seismically induced settlement (liquefaction and dry sand settlement) was calculated to be approximately 1 to 1.7 inches. The liquefaction and dry sand analysis results are included in Appendix D.

Discussion

In general, there is a high potential of granular deposits to liquefy during a seismic event. Seismically induced settlements are expected to be on the order of 1.7 inches total or less and approximately 1 inch of differential settlement during a design level seismic event.

The creek at the rear of the property is approximately 80 feet from the building and is approximately 10 feet high. Estimates of lateral displacement are approximately 10 inches at the site. The zone of soil susceptible to liquefaction and lateral displacement are present at depths from 19 to 23 feet at Boring B-1and appear to be at an elevation below the channel. The zone of soil susceptible to liquefaction at Boring B-2 is 8.5 to 13.5 feet bgs, indicating that the potentially liquefiable soils across the site are discontinuous. This is consistent with the analysis results of Harding Lawson Associates. As such, the potential for lateral displacement is considered low.

5.0 CONCLUSIONS

Site Suitability

The subject site is suitable for the proposed residential improvements from a geotechnical engineering standpoint, provided the recommendations included in this report are followed. The primary geotechnical concerns at the site are loose soils in the upper 5 feet and the settlement due to sesmic shaking.

Soil Expansion Potential

The near surface soils were sandy in nature and thus not deemed expansive. Thus, no measures other than moisture conditioning the pad are deemed yesterday.

Foundations

Due to the settlement from sesmic shaking, the proposed loads of the residence and ADU may be supported on a either a mat slab foundation or a post-tensioned slab foundation. Details of the foundation recommendations are included in the following sections of the report.

Site Preparation and Grading

Due to the loose soil in the upper 5 feet, a program of over-excavation is deemed necessary. The upper 2½ feet of existing ground in the building areas should be over-excavated and recompacted. Cuts and fills to create the pad for the residence are expected to be minimal. Additional grading work is anticipated to include backfill work related to placement of new utility lines and construction of the driveway, patios, and pool decking. Grading operations are discussed in detail in the *Recommendations* section of this report.

Groundwater

Groundwater was encountered at approximately 17 to 18 feet bgs during our subsurface exploration. Harding Lawson Associates reported an historic high groundwater level of 8 feet bgs. Variations in rainfall, temperature, and other factors may affect water levels, and therefore groundwater levels should not be considered constant. Groundwater is not expected to have an adverse effect on the construction or performance of the proposed residence and related structures.

Seismicity

The San Francisco Bay area is recognized by geologists and seismologists as one of the most seismically active regions in the United States. The significant earthquakes in this area are generally associated with crustal movement along well-defined, active fault zones which regionally trend in a northwesterly direction. Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when and where an earthquake will occur. Nevertheless, on the basis of current technology, it is reasonable to assume that the proposed development will be subjected to at least one moderate to severe earthquake during its lifetime. During such an earthquake, the danger from fault offset on the site is low, but strong shaking of the site is likely to occur and, therefore, the project should be designed in accordance with the seismic design provisions of the latest California Building Code. It should be understood that the California Building Code seismic design parameters are not intended to prevent structural damage during an earthquake, but to reduce damage and minimize loss of life.

6.0 RECOMMENDATIONS

Site Preparation and Grading

General Site Preparation

1. The site should be prepared for grading by removing existing trees to be removed and their root systems, vegetation, debris, and other potentially deleterious materials from areas to receive improvements. Existing utility lines that will not be serving the proposed residence should be either removed or abandoned. The appropriate method of utility abandonment will depend upon the type and depth of the utility. Recommendations for abandonment can be made as necessary.

2. Due to the loose surficial soil, a program of over-excavation and backfilling is deemed necessary. The upper loose soil within the area of the proposed improvements should be (over-excavated to 2½ feet bgs. The lateral extent of the over-excavation should extend at least 5 feet beyond the perimeter of the proposed residence, ADU, driveway and pool decking as determined in the field by the Geotechnical Engineer during grading operations. The exposed ground should be reviewed by the Geotechnical Engineer to determine the need for additional excavation work.

3. Ruts or depressions resulting from the removal of tree root systems should be properly cleaned out down to undisturbed native soil. The bottoms of the resulting depressions should be scarified and cross-scarified at least 8 inches in depth, moisture conditioned

and recompacted. The depressions should then be backfilled with approved, compacted, moisture conditioned structural fill, as recommended in other sections of this report.

4. Site clearing, and backfilling operations, should be conducted under the field observation of the Geotechnical Engineer. The Geotechnical Engineer should be notified at least 48 hours prior to commencement of grading operations.

Compaction Recommendations

1. In general, the underlying native soil in the areas proposed to receive additional fill, exterior flatwork or new structures should be scarified at least 8 inches, moisture conditioned and recompacted to the recommended relative compaction presented below, unless noted otherwise.

2. Recompacted native soils and fill soils should be compacted to a minimum relative compaction of 90 percent of maximum dry density at a moisture content at least 2 percentage points above optimum.

3. In areas to be paved, the upper 8 inches of subgrade soil should be compacted to a minimum 92 percent of maximum dry density at a moisture content at least 2 percentage points above optimum. The aggregate base courses should be compacted to a minimum 95 percent of maximum dry density at a moisture content that is slightly over optimum. The subgrade and base should be firm and unyielding when proof-rolled with heavy, rubber-tired equipment prior to paving. The pavement subgrade soils should be frequently moistened as necessary prior to placement of the aggregate base to maintain the soil moisture content near optimum.

Fill Recommendations

1. Structural fill is defined herein as a native or import fill material which, when properly compacted, will support foundations, pavements, and other fills. The on-site native soils that are free of debris, organics and other deleterious material, may be used as structural fill.

2. Import fill is not anticipated at the site. Should import fill be required, the soil should meet the following criteria:

   a. Be coarse grained and have a plasticity index of less than 12 and/or an expansion index less than 20;

b. Be free of organics, debris or other deleterious material;
c. Have a maximum rock size of 3 inches; and
d. Contain sufficient clay binder to allow for stable foundation and utility trench excavations.

3. A sample of the of the soil proposed to be imported to the site should be submitted at least three days before being transported to the site for evaluation by the geotechnical engineer. During importation to the site the material should be further reviewed on an intermittent basis.

Foundations

Mat Slab Foundation

1. The proposed residence and ADU may be supported by a concrete mat foundation bearing on the native soil. The mat slab should be designed using a maximum localized allowable bearing pressure of 2,000 psf for dead plus live load. This value may be increased by one-third when transient loads such as wind or seismicity are included. The mat slab should be sufficiently thick to uniformly spread the concentrated loads imposed by any building columns. The mat should be designed using a modulus of subgrade reaction value of 125 psi per inch. The slab should be designed for an edge cantilever distance of 6 feet and an interior span condition of 10 feet.

2. The mat slab should be thickened at the edges to penetrate a minimum of 6 inches into the prepared subgrade for a minimum width of 2 feet. The mat slab should be placed on top of a vapor retarder and capillary break layer extending to the thickened edge along the perimeter.

3. Resistance to lateral loads should be calculated based on a passive equivalent fluid pressure of 300 pcf and a friction factor of 0.3.

Post-Tensioned Slab Foundation

1. The post-tensioned slabs should be designed in accordance with the provisions of the current edition of the California Building Code and the recommendations of the Post-Tensioning Institute. Values for Edge Moisture Variation Distance and Estimated Differential Swell were calculated in accordance with the third edition of *Design of Post-Tensioned Slabs-on-Ground* by the Post-Tensioning Institute (2008).

Edge Moisture Variation Distance (e_m)	
Center Lift Condition	9.0 feet
Edge Lift Condition	5.0 feet
Estimated Differential Swell (y_m)	
Center Lift Condition	0.5 inches
Edge Lift Condition	0.8 inches
Allowable Bearing Capacity (dead load)	1,500 psf
Allowable Bearing Capacity (dead + live loads)	2,000 psf
Allowable Bearing Capacity (DL+LL+ wind or seismic)	2,500 psf
Subgrade Friction Factor (slab against subgrade)	0.3
Total settlement (static)	< 1 inch
Differential settlement (static)	< 0.5 inches

2. To further protect moisture-sensitive floor coverings, the perimeters of the post-tensioned slabs should be deepened to penetrate a minimum of 6 inches into the subgrade soil. Also, the concrete could be proportioned to reduce its porosity (and its corresponding potential for transmitting moisture) by limiting the w/c ratio to 0.48 or less.

3. Post-tensioned slabs should be constructed and maintained in accordance with the publication *Construction and Maintenance Manual for Post-Tensioned Slab-on-Ground Foundations* by the Post-Tensioning Institute. Particular attention should be paid to the "Property Owner Maintenance" and "Landscaping" sections of the Manual.

Interior Slab-on-Grade Construction

4. The building pad should be periodically moisture conditioned as necessary to maintain the soil moisture content at a minimum of 2 percent above optimum until the placement of concrete or vapor retarding membranes. The moisture content of the soil should be verified by the Geotechnical Engineer prior to placement of the concrete or vapor retarding membranes.

5. In areas where moisture transmitted from the subgrade would be undesirable, a vapor retarder underlain by a capillary break consisting of 4 inches of crushed rock should be utilized beneath the floor slab. The vapor retarder should comply with ASTM Standard Specification E 1745-17 and the latest recommendations of ACI Committee 302. The vapor retarder should be installed in accordance with ASTM Standard Practice E 1643-18a. Care should be taken to properly lap and seal the vapor retarder, particularly around utilities, and to protect it from damage during construction. A sand layer above the vapor retarder is optional.

6. If sand, gravel or other permeable material is to be placed over the vapor retarder, the material over the vapor retarder should be only lightly moistened and not saturated prior to casting the slab. Excess water above the vapor retarder would increase the potential for moisture damage to floor coverings. Recent studies, including those by ACI Committee 302, have concluded that excess water above the vapor retarder would increase the potential for moisture damage to floor coverings and could increase the potential for mold growth or other microbial contamination. These studies also concluded that it is preferable to eliminate the sand layer and place the slab in direct contact with the vapor retarder, particularly during wet weather construction. However, placing the concrete directly on the vapor retarder would require special attention to using the proper vapor retarder, concrete mix design, and finishing and curing techniques.

7. When concrete slabs are in direct contact with vapor retarders, the concrete water to cement (w/c) ratio must be correctly specified to control bleed water and plastic shrinkage and cracking. The concrete w/c ratio for this type of application is typically in the range of 0.45 to 0.50. The concrete should be properly cured to reduce slab curling and plastic shrinkage cracking. Concrete materials, placement, and curing methods should be specified by the architect/engineer.

Exterior Flatwork

1. Exterior flatwork should have a minimum thickness of 4 full inches and should be reinforced as directed by the architect/engineer. Patio slabs and walkways should be underlain by a minimum 4 inches of compacted aggregate base over properly compacted subgrade soil.

2. Assuming that movement (i.e., 1/4-inch or more) of exterior flatwork beyond the structure is acceptable, the flatwork should be designed to be independent of the building foundations. The flatwork should not be doweled to foundations, and a separator should be placed between the two.

3. To reduce shrinkage cracks in concrete, the concrete aggregates should be of appropriate size and proportion, the water/cement ratio should be low, the concrete should be properly placed and finished, contraction joints should be installed, and the concrete should be properly cured. Concrete materials, placement and curing specifications should be at the direction of the designer; ACI 302.1R-04 and ACI 302.2R-04 are suggested as resources for the designer in preparing such specifications.

Swimming Pool

1. The swimming pool design should be based on a minimum soil equivalent fluid pressure of 45 pcf. To reduce the potential for future expansion, the soil exposed in the pool excavation should be kept in a moist condition prior to placement of the gunite.

2. The pool may be designed with a pressure relief valve. The necessity of the valve should be under the discretion of the pool designer.

3. The pool excavation should be observed by a representative from Earth Systems. If soft soils or other unanticipated conditions are observed in the excavation, compaction of the soil or other remedial measures may be recommended. Recommendations for remedial grading or other measures (if deemed necessary) should be provided by the Geotechnical Engineer based on the conditions observed at the time of construction.

4. Any portions of the pool shell that will be above ground should be designed to support the water in the pool without soil support in accordance with Section 1808.7.3 of the California Building Code.

5. If portions of the pool walls will be within a horizontal distance of 7 feet from the top of an adjacent slope, those portions of the wall should be capable of supporting the water in the pool without soil support per section 1808.7.3 of the California Building Code.

Utility Trench Backfills

1. A select, noncorrosive, granular, easily compacted material should be used as bedding and shading immediately around utility pipes. The site soils may be used for trench backfill above the select material.

2. Trench backfill in the upper 8 inches of subgrade beneath pavement areas should be compacted to a minimum of 92 percent of maximum dry density at a moisture content at least 2 percentage points above optimum moisture content and the aggregate base courses should be compacted to a minimum 95 percent of maximum dry density at a moisture content at least 2 percentage points over optimum. Trench backfill in other areas should be compacted to a minimum of 90 percent of maximum dry density at a moisture content at least 2 percentage points above optimum moisture content. Jetting of utility trench backfill should not be allowed.

3. Where utility trenches extend under perimeter foundations, the trenches should be backfilled entirely with approved fill soil compacted to a minimum of 90 percent of maximum dry density at a moisture content at least 2 percentage points above optimum moisture content. The zone of approved fill soil should extend a minimum distance of 2 feet on both sides of the foundation. If utility pipes pass through sleeves cast into the perimeter foundations, the annulus between the pipes and sleeves should be completely sealed.

4. Parallel trenches excavated in the area under foundations defined by a plane radiating at a 45-degree angle downward from the bottom edge of the footing should be avoided, if possible. Trench backfill within this zone, if necessary, should consist of Controlled Density Fill (Flowable Fill).

Management of Site Drainage and Finish Improvements

1. Unpaved ground surfaces should be finish graded to direct surface runoff away from site improvements at a minimum 5 percent grade for a minimum distance of 10 feet. If this is not practical due to the terrain or other site features, swales with improved surfaces should be provided to divert drainage away from improvements. The landscaping should be planned and installed to maintain proper surface drainage conditions.

2. Runoff from driveways, roof gutters, downspouts, planter drains and other improvements should discharge in a non-erosive manner away from foundations, pavements, and other improvements. The downspouts may discharge onto splash blocks that direct the flow away from the foundation.

3. Stabilization of surface soils, particularly those disturbed during construction, by vegetation or other means during and following construction is essential to protect the site from erosion damage. Care should be taken to establish and maintain vegetation.

4. Open areas adjacent to exterior flatwork should be irrigated or otherwise maintained so that constant moisture conditions are created throughout the year. Irrigation systems should be controlled to the minimum levels that will sustain the vegetation without saturating the soil.

5. Bio-retention swales constructed within 10 feet or less from the building foundation should be lined with a 20-mil pond liner.

Geotechnical Observation and Testing

1. It must be recognized that the recommendations contained in this report are based on a limited number of borings and rely on continuity of the subsurface conditions encountered.

2. It is assumed that the Geotechnical Engineer will be retained to provide consultation during the design phase, to interpret this report during construction, and to provide construction monitoring in the form of testing and observation.

3. Unless otherwise stated, the terms "compacted" and "recompacted" refer to soils placed in level lifts not exceeding 8 inches in loose thickness and compacted to a minimum of 90 percent of maximum dry density. The standard tests used to define maximum dry density and field density should be ASTM D 1557-12 and ASTM D 6938-17, respectively, or other methods acceptable to the geotechnical engineer and jurisdiction.

4. “Moisture conditioning” refers to adjusting the soil moisture to at least 3 percentage points above optimum moisture content prior to application of compactive effort. If the soils are overly moist so that they become unstable, or if the recommended compaction cannot be readily achieved, drying the soil to optimum moisture content or just above may be necessary. Placement of gravel layers or geotextiles may also be necessary to help stabilize unstable soils. The Geotechnical Engineer should be contacted for recommendations for mitigating unstable soils.

5. At a minimum, the following should be provided by the Geotechnical Engineer:
   - Review of final grading and foundation plans,
   - Professional observation during site preparation, grading, and foundation excavation,
   - Oversight of soil compaction testing during grading,
   - Oversight of soil special inspection during grading.

6. Special inspection of grading should be provided as per Section 1705.6 and Table 1705.6 of the CBC; the soils special inspector should be under the direction of the Geotechnical Engineer. In our opinion, the following operations should be subject to *continuous* soils special inspection:
   - Scarification and recompaction,
   - Fill placement and compaction,
   - Over-excavation to the recommended depth.

7. In our opinion, the following operations may be subject to *periodic* soils special inspection, subject to approval by the Building Official:

    - Site preparation,
    - Compaction of utility trench backfill,
    - Retaining wall backfill,
    - Pool excavation,
    - Removal of existing development features,
    - Compaction of subgrade and aggregate base,
    - Observation of foundation and basement excavations,
    - Building pad moisture conditioning.

8. It will be necessary to develop a program of quality control prior to beginning grading. It is the responsibility of the owner, contractor, or project manager to determine any additional inspection items required by the architect/engineer or the governing jurisdiction.

9. The locations and frequencies of compaction tests should be as per the recommendations of the Geotechnical Engineer at the time of construction. The recommended test locations and frequencies may be subject to modification by the geotechnical engineer based upon soil and moisture conditions encountered, the size and type of equipment used by the contractor, the general trend of the compaction test results, and other factors.

10. A preconstruction conference among a representative of the owner, the Geotechnical Engineer, soils special inspector, the architect/engineer, and contractors is recommended to discuss planned construction procedures and quality control requirements. Earth Systems should be notified at least 48 hours prior to beginning grading operations.

7.0 CLOSURE

This report is valid for conditions as they exist at this time for the type of project described herein. Our intent was to perform the investigation in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in the locality of this project at this time under similar conditions. No representation, warranty, or guarantee is either expressed or implied. This report is intended for the exclusive use by the client as discussed in the Scope of Services section. Application beyond the stated intent is strictly at the user's risk.

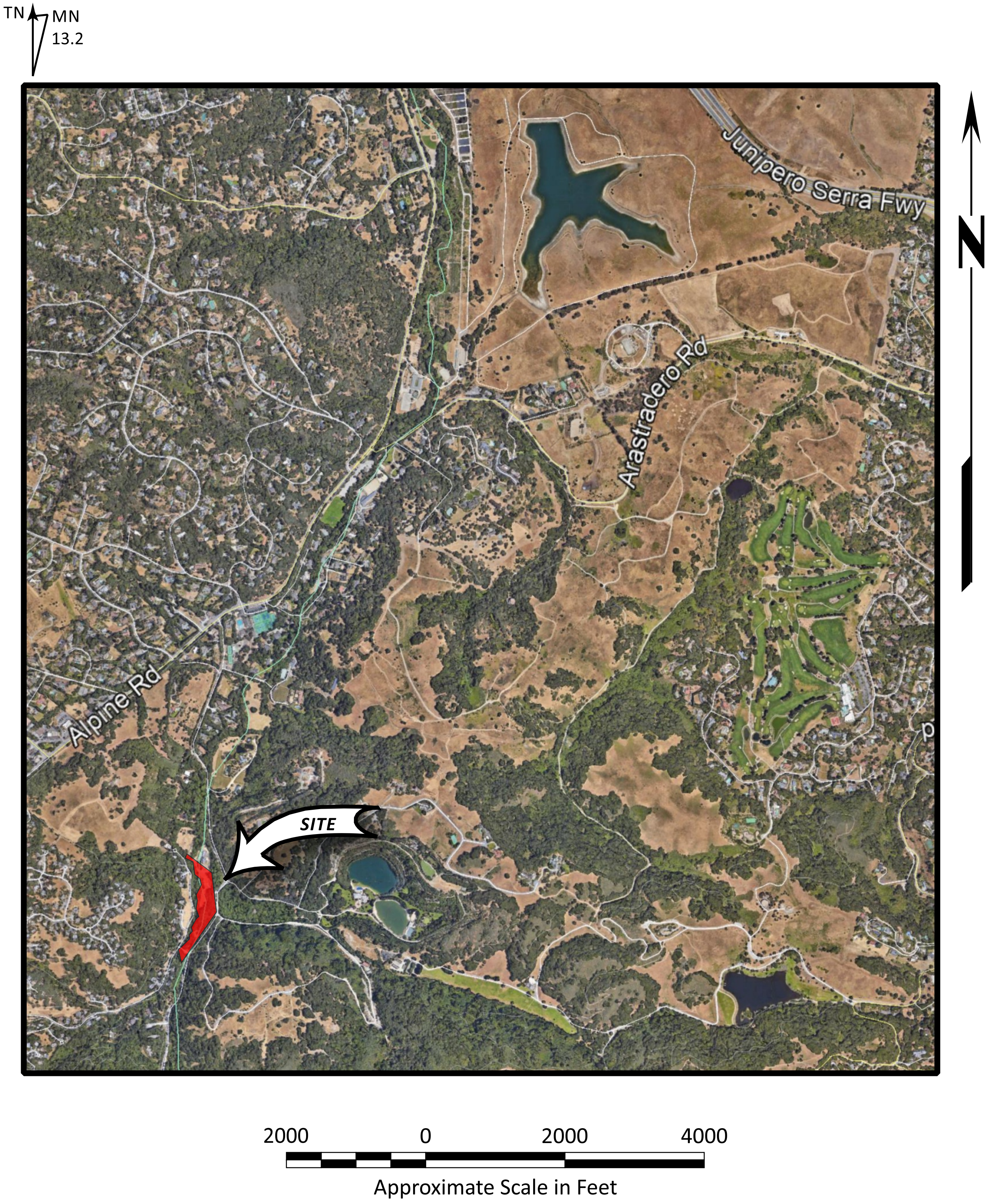
If changes with respect to the project type or location become necessary, if items not addressed in this report are incorporated into plans, or if any of the assumptions stated in this report are not correct, Earth Systems should be notified for modifications to this report. Any items not specifically addressed in this report should comply with the CBC and the requirements of the governing jurisdiction.

The preliminary recommendations of this report are based upon the geotechnical conditions encountered during the investigation and may be augmented by additional requirements of the architect/engineer, or by additional recommendations provided by Earth Systems based on conditions exposed at the time of construction.

This document, the data, conclusions, and recommendations contained herein are the property of Earth Systems. This report should be used in its entirety, with no individual sections reproduced or used out of context. Copies may be made only by Earth Systems, the client, and his authorized agents for use exclusively on the subject project. Any other use is subject to federal copyright laws and the written approval of Earth Systems.

FIGURES

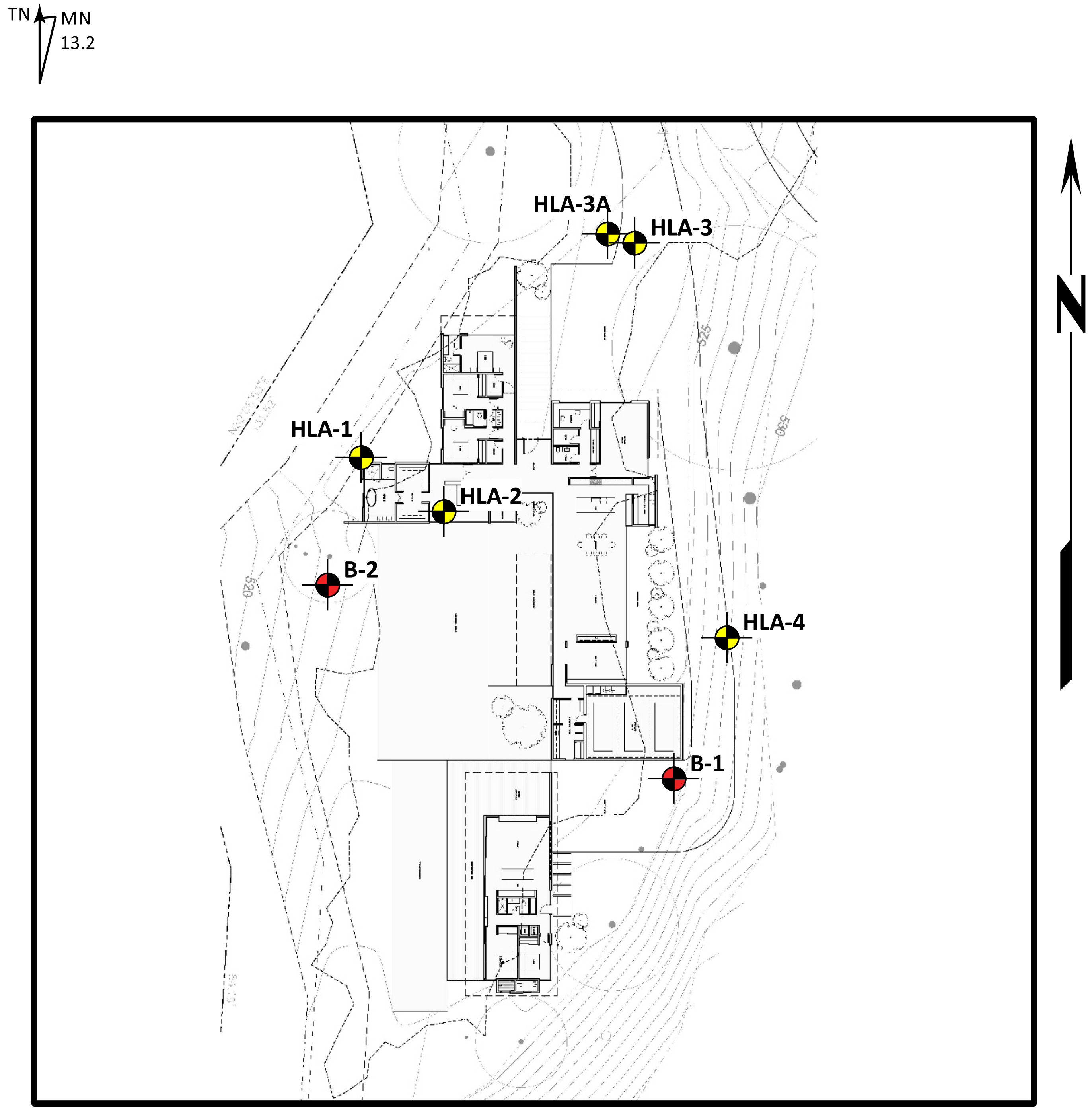
Figure 1



Base: Google Earth (2021)

Earth Systems Pacific	**Proposed Single Family Residence and ADU** **575 Los Trancos Road** **Palo Alto, California**	**Site Location Map** **304309-001**

Figure 2



● **B-2** Approximate Boring Location (ESP, 2021)

● **HLA-4** Approximate Boring Location (Harding Lawson Associates, 1989)

Base: Google Earth (2021)

Earth Systems Pacific

Proposed Single Family Residence and ADU
575 Los Trancos Road
Palo Alto, California

Site Plan

304309-001

APPENDIX A

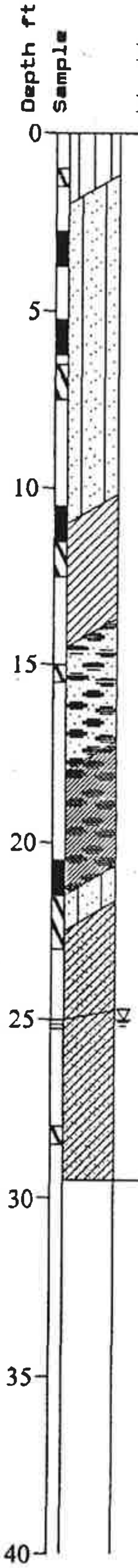
Boring Logs
Harding Lawson Associates
1990

DIETERICH-POST 130713

Equipment 8" Hollow Stem Auger

Elevation 520.0 ft Date 10/30/89

Laboratory Tests	Blows/ft	Moisture Cont.(%)	Dry Density (pcf)
	15*		
19.5 percent passing No. 200 sieve	10*		
	16		
PI=12 LL=34	8*	12.8	93
	16		
	56		
	27*	8.0	117
	16		
26.1 percent passing No. 200 sieve (see Plate 7)			
	30/3"*		
	50/5"		



DARK BROWN SANDY SILT WITH GRAVEL (ML)
medium stiff, moist, with roots

DARK BROWN SILTY SAND WITH GRAVEL (SM)
medium dense, dry, with roots, fine-grained sand
becoming brown in color and decreasing fine content

increasing fines content

DARK BROWN SANDY CLAY WITH GRAVEL (CL/ML)
stiff to very stiff, moist

RED AND BROWN POORLY GRADED GRAVEL WITH SAND (GP)
dense to very dense, dry, gravel in tip of sampler

ORANGE, RED, AND BROWN CLAYEY GRAVEL WITH SAND (GC)
medium dense, moist to wet, gravels rounded and subangular, approximately 15 percent fines

LIGHT BROWN SILTY SAND WITH GRAVEL (SM)
medium dense, wet, approximately 30 percent fines and 20 percent gravel

MOTTLED ORANGE AND GRAY-BROWN CLAYEY SAND WITH GRAVEL (SC/SM)
medium dense, wet

BROWN CLAYEY SAND WITH GRAVEL (SC/GC)
very dense, saturated, rounded and subrounded gravels
approxiamtely 15 percent fines

*Marked blow counts represent S&H blow counts converted to "pseudo, SPT N-values" by multiplying field values by 0.6; all other values are SPT N-values as defined in ASTM D1586-84 Test Method; see text for more detail.

**Elevations are approximate and referenced to Mean Sea Level.

Harding Lawson Associates
Engineering and
Environmental Services

Log of Boring B-1 (sheet 1 of 1)
Conroe Residence
Palo Alto, California

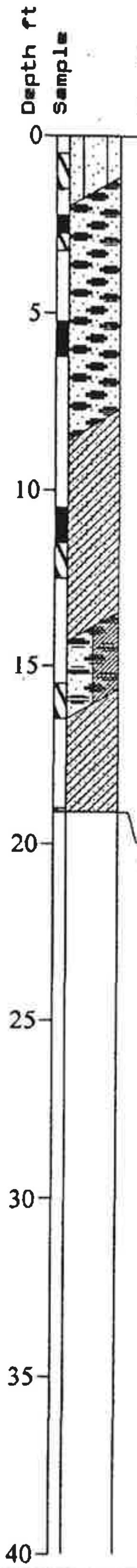
PLATE
2

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
B4740-G5	19640,001.04	WAH	1/90	

Equipment 8" Hollow Stem Auger

Elevation 520.0 ft Date 10/30/89

Laboratory Tests	Blows/ft	Moisture Cont. (%)	Dry Density (pcf)
17.5 percent passing No. 200 sieve			
2.2 percent passing No. 200 sieve (see Plate 8)	8*		
	12*		
37.6 percent passing No. 200 sieve (see Plate 9)	20*		
	30		
	46		
	45/1"		



DARK BROWN SILTY SAND WITH GRAVEL (SM)
loose, moist, with roots

BROWN GRAVEL WITH SAND (GP)
loose to medium dense, dry, with roots

DARK BROWN CLAYEY SAND (SC/SM)
medium dense, moist, 14 percent gravel, with roots

GRAY AND BROWN POORLY GRADED GRAVEL WITH SAND (GP-GC)
dense, moist, approximtely 10 percent fines and 30 percent sand

ORANGE-BROWN CLAYEY SAND WITH GRAVEL (SC)
dense, moist, approximately 15 percent fines and 20 percent rounded gravel

No ground water was encountered.

Harding Lawson Associates
Engineering and Environmental Services

Log of Boring B-2
Conroe Residence
Palo Alto, California

(sheet 1 of 1)

PLATE 3

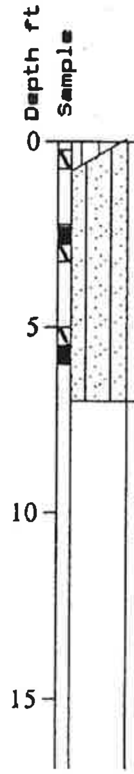
DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
B4740-G5	19640,001.04	WAA	1/90	

Boring No. B-3

Equipment 8" Hollow Stem Auger

Elevation 520.0 ft   Date 10/30/89

Laboratory Tests	Blows/ft	Moisture Content (%)	Dry Density (pcf)
	8*		
30.5 percent passing No. 200 sieve (see Plate 10)			
	15*		



DARK BROWN SILT (ML)
soft to medium stiff, moist, with roots

DARK BROWN SILTY SAND WITH GRAVEL (SM)
loose to medium dense, moist

increasing gravel and cobbles

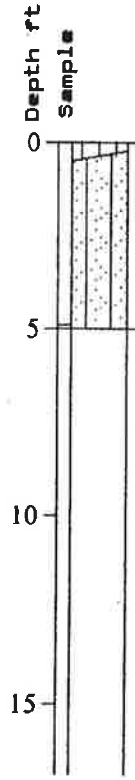
Refusal on cobbles at 7.0 feet.
No ground water was encountered.

Boring No. B-3A

Equipment 8" Hollow Stem Auger

Elevation 520.0 ft   Date 10/30/89

Laboratory Tests	Blows/ft	Moisture Content (%)	Dry Density (pcf)
	17/0"		



BROWN SILT (ML)
soft to medium stiff, moist, with roots

DARK BROWN SILTY SAND WITH GRAVEL (SM)
loose to medium dense, moist
increasing gravel and cobbles at 4 feet

Refusal on cobbles at 5.0 feet.
No ground water was encountered.

Harding Lawson Associates
Engineering and
Environmental Services

Logs of Borings B-3 and B-3A
Conroe Residence
Palo Alto, California

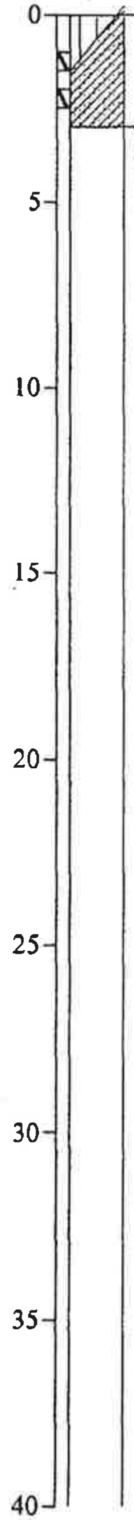
PLATE **4**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
B4740-G5	19640,001.04		1/90	

Equipment 8" Hollow Stem Auger

Elevation 529.0 ft Date 10/30/89

Laboratory Tests	Blows/ft	Moisture Cont.(%)	Dry Density (pcf)	Depth ft	Sample
	30/5"*				



BLACK SILT (ML)
soft to medium stiff, moist,
with roots

BROWN CLAYEY SAND WITH GRAVEL (SC/GC)
dry, with cobbles

Refusal on cobbles at 3.0 feet.
No ground water was encountered.

Harding Lawson Associates
Engineering and
Environmental Services

Log of Boring B-4
Conroe Residence
Palo Alto, California

(sheet 1 of 1)

PLATE
5

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
B4740-G5	19640,001.04	WAH	11/89	

APPENDIX B

Logs of Test Borings
Earth Systems Pacific
2021

Earth Systems Pacific

Boring No. 1

PAGE 1 OF 2

LOGGED BY: P. Penrose
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

JOB NO.: 304309-001
DATE: February 23, 2021

Proposed Residence
575 Los Trancos Road
Palo Alto, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA INTERVAL (feet)	SAMPLE NUMBER	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.	POCKET PEN (t.s.f)
0	SW	▨	Well graded SAND with GRAVEL; medium dense, dark gray brown, very moist, fine to coarse sand, fine to coarse gravel							
2		▨		1.0-2.5	1-1	■	110.6	4.9	8 9 16	
5		▨		3.5-5.0	1-2	■	113.4	7.3	6 9 9	
5	SC	▨	CLAYEY SAND with GRAVEL; medium dense, gray brown, very moist, fine to coarse sand, fine to coarse gravel							
7		▨	- cobbles, dense							
9		▨		7.5-9.0	1-3	●			24 21 22	
15		▨		13.5-15.0	1-4	●			16 40 17	
18	SC	▨	CLAYEY SAND; loose, brown, wet, mostly fine to medium sand, trace gravel ▼							
20		▨	[% passing #200 = 18%]	18.5-20.0	1-5	●			9 6 8	
23		▨	- very dense, less clay, more gravel							
24		▨		23.5-24.0	1-6	●			50/5"	
26		▨								

LEGEND: ■ 2.5" Mod Cal Sample ☐ 2.0" Cal Sample ● SPT ○ Bulk Sample ▼ Groundwater

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Boring No. 1

PAGE 2 OF 2

LOGGED BY: P. Penrose
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

JOB NO.: 304309-001
DATE: February 23, 2021

Proposed Residence
575 Los Trancos Road
Palo Alto, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA						
				INTERVAL (feet)	SAMPLE NUMBER	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.	POCKET PEN (t.s.f)
26	SC	▨	CLAYEY SAND with GRAVEL (same as above)							
-										
27										
-										
28										
-			- blue gray	28.5-29.0	1-7	●			50/4"	
29										
-										
30										
-										
31										
-										
32										
-										
33										
-										
34				33.5-34.0	1-8	●			50/5"	
-			Bottom of boring at 34' bgs No Groundwater encountered							
35										
-										
36										
-										
37										
-										
38										
-										
39										
-										
40										
-										
41										
-										
42										
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46										
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48										
-										
49										
-										
50										
-										
51										
-										
52										
-										

LEGEND: ■ 2.5" Mod Cal Sample ☐ 2.0" Cal Sample ● SPT ○ Bulk Sample ▼ Groundwater

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Earth Systems Pacific

Boring No. 2

PAGE 1 OF 2

LOGGED BY: P. Penrose
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

JOB NO.: 304309-001
DATE: February 23, 2021

Proposed Residence
575 Los Trancos Road
Palo Alto, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL (feet)	SAMPLE NUMBER	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.	POCKET PEN (t.s.f)
0	SC	▨	CLAYEY SAND with GRAVEL; loose, gray brown, moist, fine to coarse sand, fine to coarse gravel							
2				1.0-2.5	2-1	■	103.4	12.9	5 5 6	
5				3.5-5.0	2-2	■	102.1	11.1	5 6 6	
			- medium dense							
10			[% passing #200 = 21%]	8.5-10.0	2-3	●			6 9 12	
13			- very dense, gray, very moist							
15				13.5-15.0	2-4	■	115.7	18.4	5 10 41	
17	SW-SC	▨	▼ Well graded SAND with CLAY and GRAVEL; medium dense, gray brown, wet, fine to coarse sand, fine to coarse gravel							
20			[% passing #200 = 9%]	18.5-20.0	2-5	●			10 15 20	
23	SC	▨	CLAYEY SAND with GRAVEL; medium dense, gray brown, wet, fine to coarse sand, fine gravel							
25			[% passing #200 = 31%]	23.5-25.0	2-6	●			25 12 16	
26										

LEGEND: ■ 2.5" Mod Cal Sample ☐ 2.0" Cal Sample ● SPT ○ Bulk Sample ▼ Groundwater

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

Boring No. 2
PAGE 2 OF 2

LOGGED BY: P. Penrose
DRILL RIG: Mobile B-53
AUGER TYPE: 8" Hollow Stem

JOB NO.: 304309-001
DATE: February 23, 2021

Proposed Residence
575 Los Trancos Road
Palo Alto, California

DEPTH (feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA INTERVAL (feet)	SAMPLE NUMBER	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.	POCKET PEN (t.s.f)
26	SC	▨	CLAYEY SAND with GRAVEL (same as above)							
27		▨								
28	SW-SC	▨	Well graded SAND with CLAY and GRAVEL; dense, gray brown, wet, fine to coarse sand, fine to coarse gravel							
29		▨							9 11 30	
30		▨		28.5-29.0	2-7	●				
31		▨								
32		▨								
33		▨								
34		▨		33.5-34.0	2-8	●			50/6"	
35			Bottom of boring at 34' bgs Groundwater encountered at 17' bgs							
36										
37										
38										
39										
40										
41										
42										
43										
44										
45										
46										
47										
48										
49										
50										
51										
52										

LEGEND: ■ 2.5" Mod Cal Sample □ 2.0" Cal Sample ● SPT ○ Bulk Sample ▼ Groundwater

NOTE: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

APPENDIX C

Summary of Laboratory Test Results

575 Los Trancos Road

304309-001

BULK DENSITY TEST RESULTS

ASTM D 2937-17 (modified for ring liners)

March 4, 2021

BORING NO.	DEPTH feet	MOISTURE CONTENT, %	WET DENSITY, pcf	DRY DENSITY, pcf
1-1	2.0 - 2.5	4.9	116.0	110.6
1-2	4.5 - 5.0	7.3	121.7	113.4
2-1	2.0 - 2.5	12.9	116.7	103.4
2-2	4.5 - 5.0	11.1	113.4	102.1
2-4	14.5 - 15.0	18.4	137.0	115.7

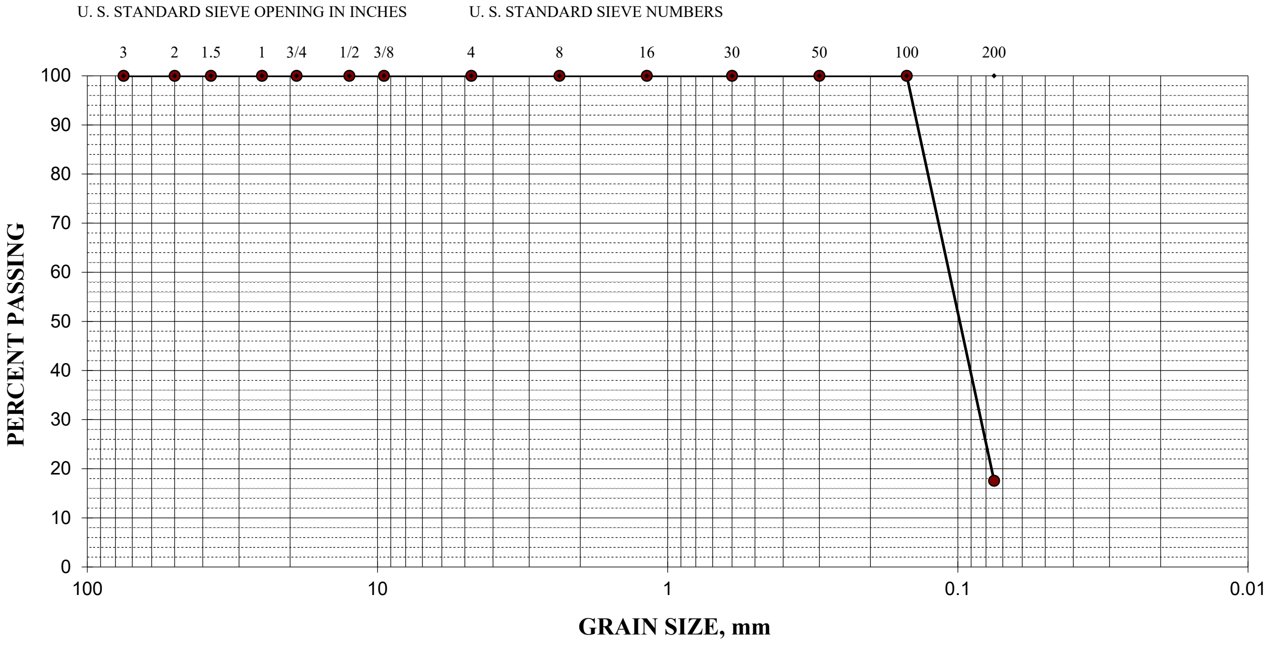
PARTICLE SIZE ANALYSIS

ASTM D 422-63/07; D 1140-17

Boring #1 @ 18.5 - 20.0'

March 4, 2021

Sieve size	% Retained	% Passing
3" (75-mm)	0	100
2" (50-mm)	0	100
1.5" (37.5-mm)	0	100
1" (25-mm)	0	100
3/4" (19-mm)	0	100
1/2" (12.5-mm)	0	100
3/8" (9.5-mm)	0	100
#4 (4.75-mm)	0	100
#8 (2.36-mm)	0	100
#16 (1.18-mm)	0	100
#30 (600-µm)	0	100
#50 (300-µm)	0	100
#100 (150-µm)	0	100
#200 (75-µm)	82	18



575 Los Trancos Road 304309-001

PARTICLE SIZE ANALYSIS

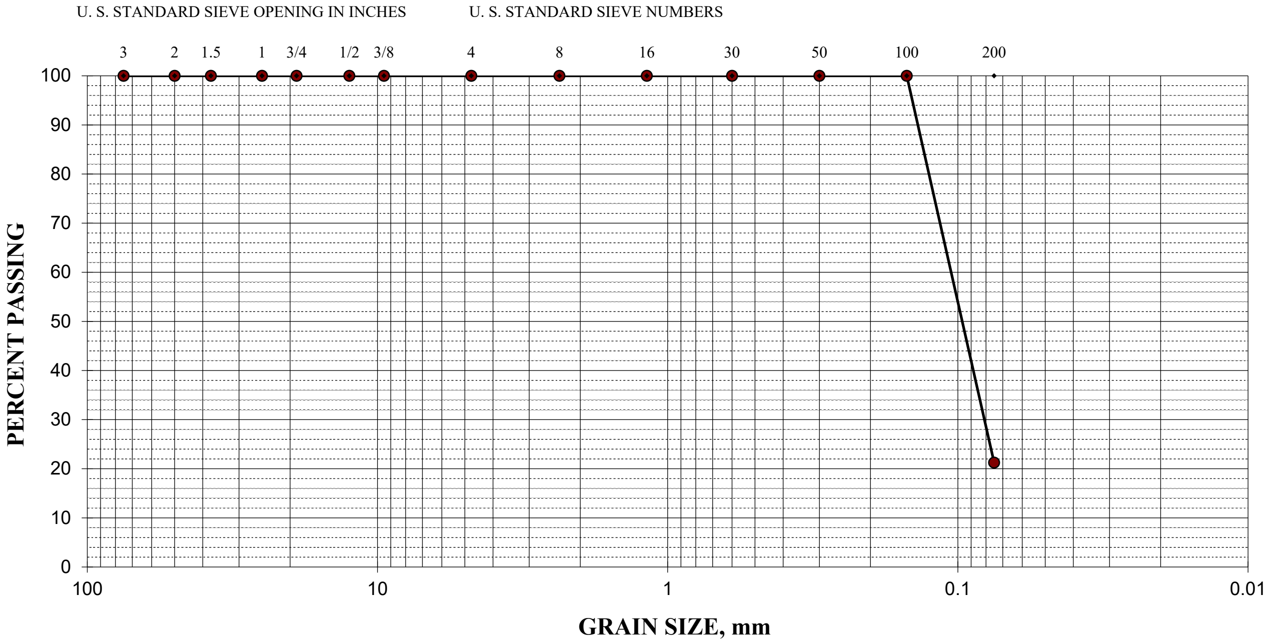
ASTM D 422-63/07; D 1140-14

Boring #2 @ 8.5 - 10.0'

March 4, 2021

Dark Brown Well Graded Sand with Clay and Gravel (SW-SC)

Sieve size	% Retained	% Passing
3" (75-mm)	0	100
2" (50-mm)	0	100
1.5" (37.5-mm)	0	100
1" (25-mm)	0	100
3/4" (19-mm)	0	100
1/2" (12.5-mm)	0	100
3/8" (9.5-mm)	0	100
#4 (4.75-mm)	0	100
#8 (2.36-mm)	0	100
#16 (1.18-mm)	0	100
#30 (600-µm)	0	100
#50 (300-µm)	0	100
#100 (150-µm)	0	100
#200 (75-µm)	79	21



575 Los Trancos Road 304309-001

PARTICLE SIZE ANALYSIS

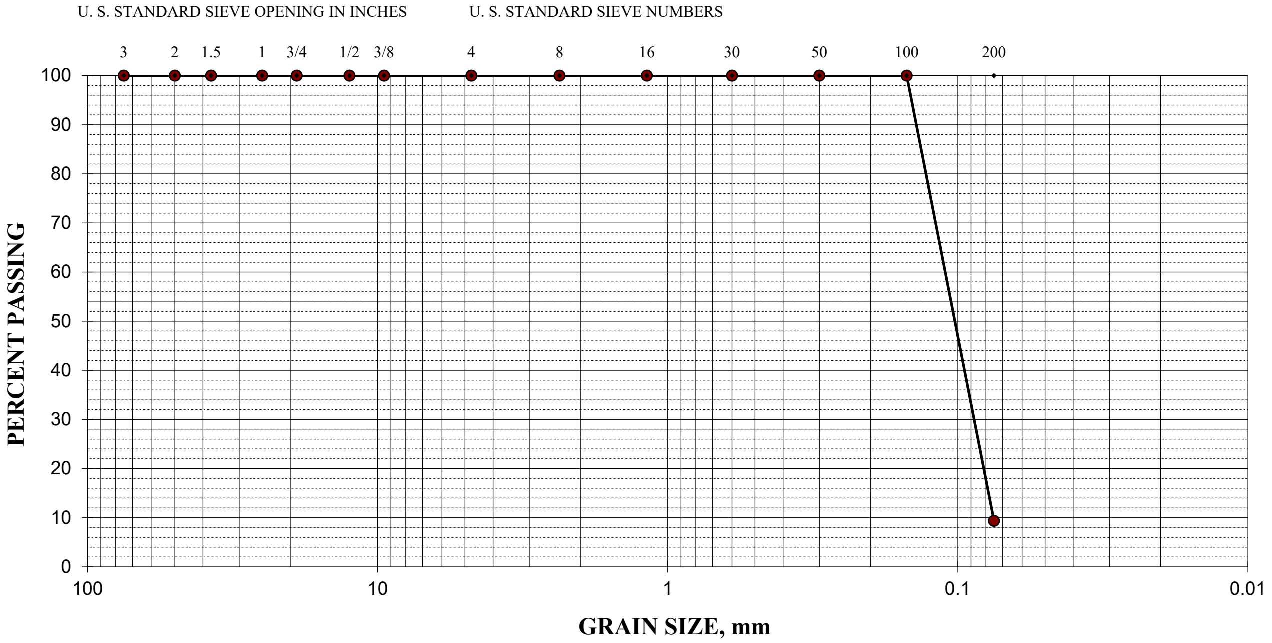
ASTM D 422-63/07; D 1140-17

Boring #2 @ 18.5 - 20.0'

March 4, 2021

Dark Yellowish Brown Clayey Sand with Gravel (SC)

Sieve size	% Retained	% Passing
3" (75-mm)	0	100
2" (50-mm)	0	100
1.5" (37.5-mm)	0	100
1" (25-mm)	0	100
3/4" (19-mm)	0	100
1/2" (12.5-mm)	0	100
3/8" (9.5-mm)	0	100
#4 (4.75-mm)	0	100
#8 (2.36-mm)	0	100
#16 (1.18-mm)	0	100
#30 (600-µm)	0	100
#50 (300-µm)	0	100
#100 (150-µm)	0	100
#200 (75-µm)	91	9



575 Los Trancos Road 304309-001

PARTICLE SIZE ANALYSIS

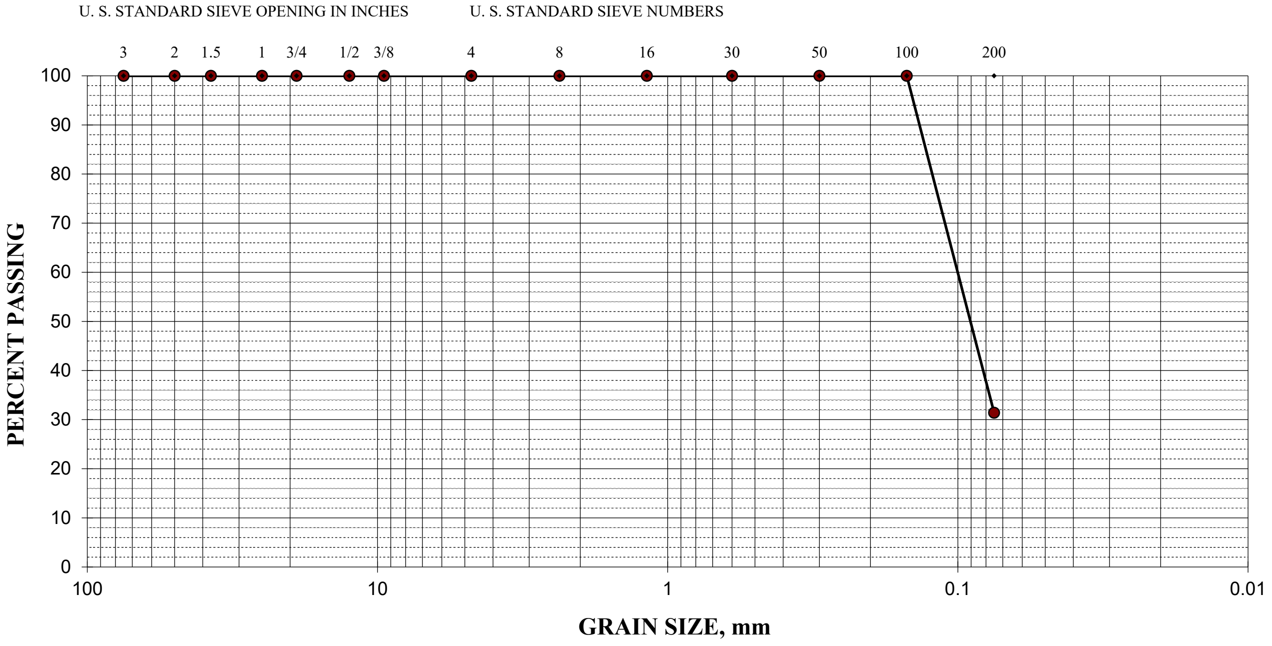
ASTM D 422-63/07; D 1140-17

Boring #2 @ 23.5 - 25.0'

March 4, 2021

Dark Yellowish Brown Clayey Sand with Gravel (SC)

Sieve size	% Retained	% Passing
3" (75-mm)	0	100
2" (50-mm)	0	100
1.5" (37.5-mm)	0	100
1" (25-mm)	0	100
3/4" (19-mm)	0	100
1/2" (12.5-mm)	0	100
3/8" (9.5-mm)	0	100
#4 (4.75-mm)	0	100
#8 (2.36-mm)	0	100
#16 (1.18-mm)	0	100
#30 (600-µm)	0	100
#50 (300-µm)	0	100
#100 (150-µm)	0	100
#200 (75-µm)	69	31



APPENDIX D

Liquefaction Analysis
Dry Sand Settlement

SPT BASED LIQUEFACTION ANALYSIS REPORT

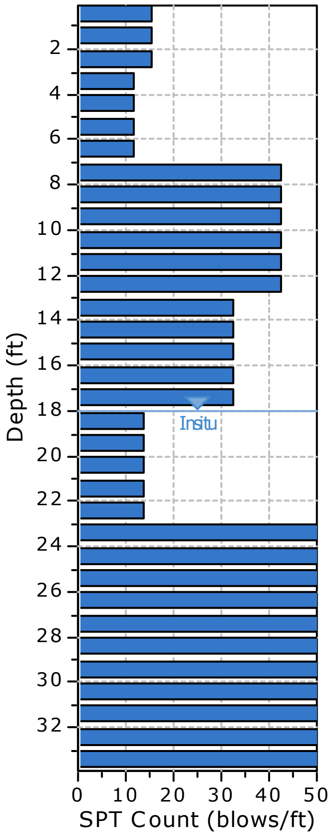
Project title : 575 Los Trancos Road Residence **SPT Name: B-1**

Location : Palo Alto, California

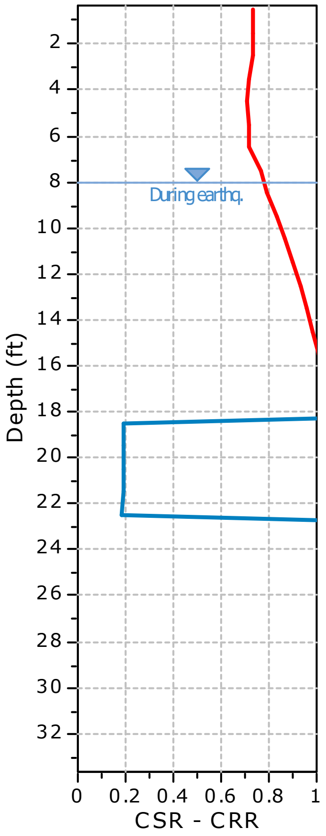
:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	18.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	8.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.80
Borehole diameter:	200mm	Peak ground acceleration:	1.16 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		

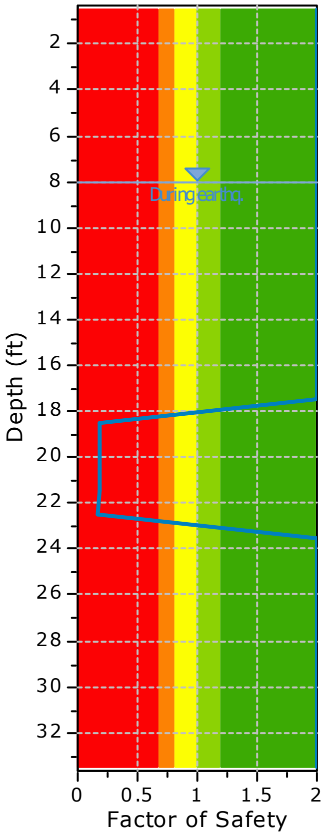
Raw SPT Data



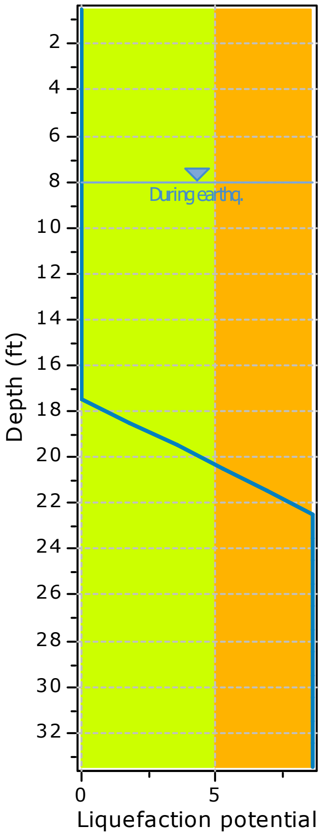
CSR - CRR Plot



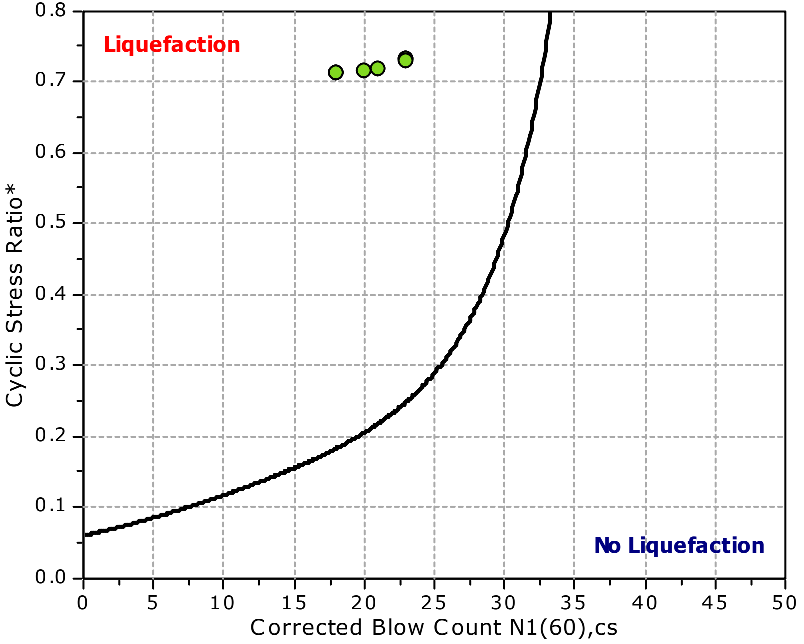
FS Plot



LPI



CRR 7.50 clean sand curve



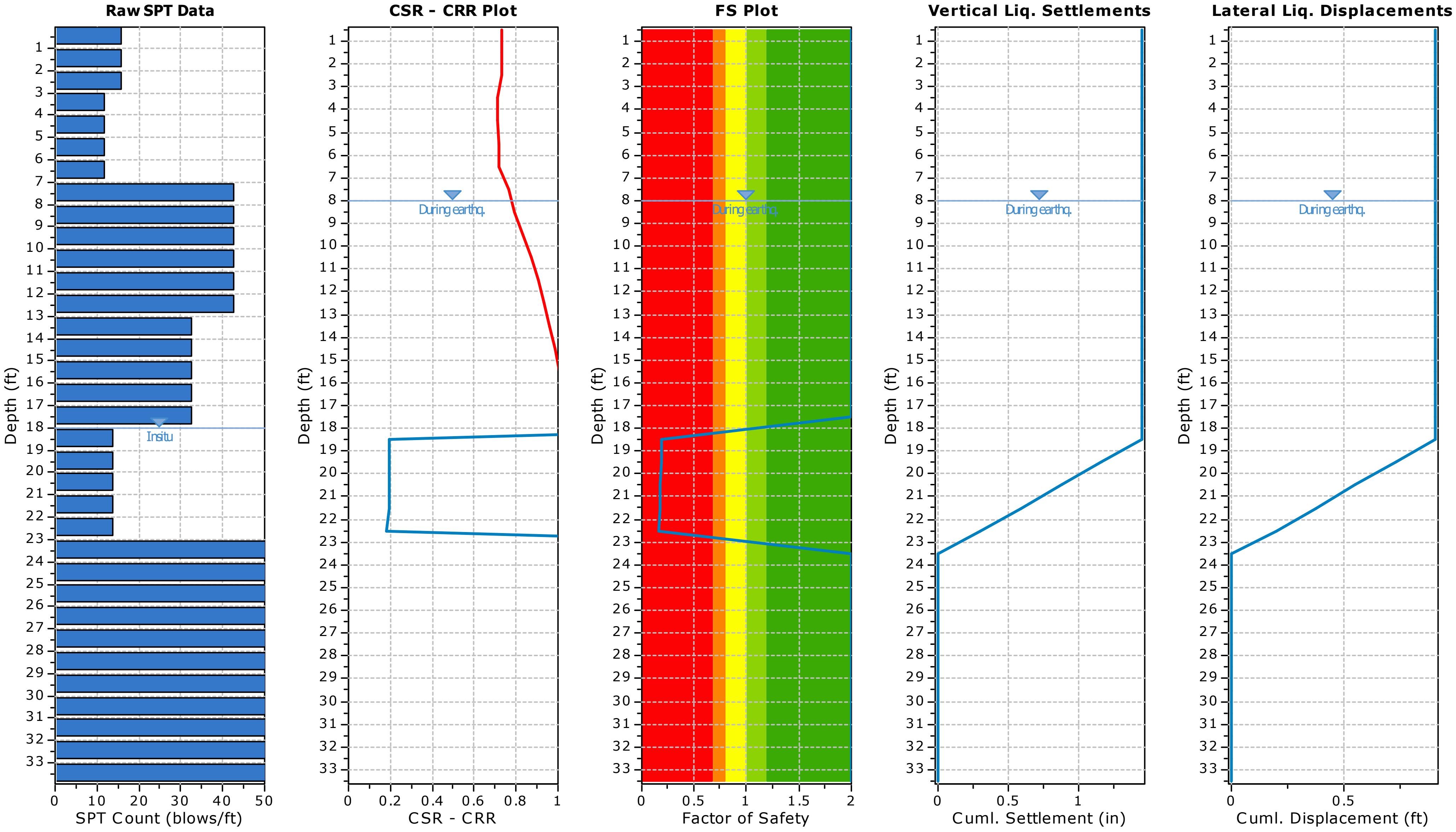
F.S. color scheme

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

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
0.50	16	5.00	116.00	1.00	Yes
1.50	16	5.00	116.00	1.00	Yes
2.50	16	5.00	116.00	1.00	Yes
3.50	12	5.00	122.00	1.00	Yes
4.50	12	5.00	122.00	1.00	Yes
5.50	12	18.00	122.00	1.00	Yes
6.50	12	18.00	122.00	1.00	Yes
7.50	43	18.00	120.00	1.00	Yes
8.50	43	18.00	120.00	1.00	Yes
9.50	43	18.00	120.00	1.00	Yes
10.50	43	18.00	120.00	1.00	Yes
11.50	43	18.00	120.00	1.00	Yes
12.50	43	18.00	120.00	1.00	Yes
13.50	33	18.00	120.00	1.00	Yes
14.50	33	18.00	120.00	1.00	Yes
15.50	33	18.00	120.00	1.00	Yes
16.50	33	18.00	120.00	1.00	Yes
17.50	33	18.00	120.00	1.00	Yes
18.50	14	18.00	120.00	1.00	Yes
19.50	14	18.00	120.00	1.00	Yes
20.50	14	18.00	120.00	1.00	Yes
21.50	14	18.00	120.00	1.00	Yes
22.50	14	18.00	120.00	1.00	Yes
23.50	100	18.00	120.00	1.00	Yes
24.50	100	18.00	120.00	1.00	Yes
25.50	100	18.00	120.00	1.00	Yes
26.50	100	18.00	120.00	1.00	Yes
27.50	100	18.00	120.00	1.00	Yes
28.50	100	18.00	120.00	1.00	Yes
29.50	100	18.00	120.00	1.00	Yes
30.50	100	18.00	120.00	1.00	Yes
31.50	100	18.00	120.00	1.00	Yes
32.50	100	18.00	120.00	1.00	Yes
33.50	100	18.00	120.00	1.00	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
0.50	16	116.00	0.03	0.00	0.03	0.41	1.70	1.00	1.15	0.75	1.00	23	5.00	0.00	23	4.000
1.50	16	116.00	0.09	0.00	0.09	0.41	1.70	1.00	1.15	0.75	1.00	23	5.00	0.00	23	4.000
2.50	16	116.00	0.15	0.00	0.15	0.41	1.70	1.00	1.15	0.75	1.00	23	5.00	0.00	23	4.000

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
3.50	12	122.00	0.21	0.00	0.21	0.46	1.70	1.00	1.15	0.75	1.00	18	5.00	0.00	18	4.000
4.50	12	122.00	0.27	0.00	0.27	0.46	1.70	1.00	1.15	0.75	1.00	18	5.00	0.00	18	4.000
5.50	12	122.00	0.33	0.00	0.33	0.41	1.62	1.00	1.15	0.75	1.00	17	18.00	4.09	21	4.000
6.50	12	122.00	0.39	0.00	0.39	0.42	1.53	1.00	1.15	0.75	1.00	16	18.00	4.09	20	4.000
7.50	43	120.00	0.45	0.00	0.45	0.26	1.25	1.00	1.15	0.80	1.00	50	18.00	4.09	54	4.000
8.50	43	120.00	0.51	0.00	0.51	0.26	1.21	1.00	1.15	0.80	1.00	48	18.00	4.09	52	4.000
9.50	43	120.00	0.57	0.00	0.57	0.26	1.18	1.00	1.15	0.80	1.00	47	18.00	4.09	51	4.000
10.50	43	120.00	0.63	0.00	0.63	0.26	1.15	1.00	1.15	0.85	1.00	48	18.00	4.09	52	4.000
11.50	43	120.00	0.69	0.00	0.69	0.26	1.12	1.00	1.15	0.85	1.00	47	18.00	4.09	51	4.000
12.50	43	120.00	0.75	0.00	0.75	0.26	1.10	1.00	1.15	0.85	1.00	46	18.00	4.09	50	4.000
13.50	33	120.00	0.81	0.00	0.81	0.30	1.08	1.00	1.15	0.85	1.00	35	18.00	4.09	39	4.000
14.50	33	120.00	0.87	0.00	0.87	0.31	1.06	1.00	1.15	0.85	1.00	34	18.00	4.09	38	4.000
15.50	33	120.00	0.93	0.00	0.93	0.31	1.04	1.00	1.15	0.85	1.00	34	18.00	4.09	38	4.000
16.50	33	120.00	0.99	0.00	0.99	0.29	1.02	1.00	1.15	0.95	1.00	37	18.00	4.09	41	4.000
17.50	33	120.00	1.05	0.00	1.05	0.30	1.00	1.00	1.15	0.95	1.00	36	18.00	4.09	40	4.000
18.50	14	120.00	1.11	0.02	1.09	0.45	0.99	1.00	1.15	0.95	1.00	15	18.00	4.09	19	0.194
19.50	14	120.00	1.17	0.05	1.12	0.45	0.97	1.00	1.15	0.95	1.00	15	18.00	4.09	19	0.194
20.50	14	120.00	1.23	0.08	1.15	0.45	0.96	1.00	1.15	0.95	1.00	15	18.00	4.09	19	0.194
21.50	14	120.00	1.29	0.11	1.18	0.45	0.95	1.00	1.15	0.95	1.00	15	18.00	4.09	19	0.194
22.50	14	120.00	1.35	0.14	1.21	0.46	0.94	1.00	1.15	0.95	1.00	14	18.00	4.09	18	0.184
23.50	100	120.00	1.41	0.17	1.24	0.26	0.96	1.00	1.15	0.95	1.00	105	18.00	4.09	109	4.000
24.50	100	120.00	1.47	0.20	1.27	0.26	0.95	1.00	1.15	0.95	1.00	104	18.00	4.09	108	4.000
25.50	100	120.00	1.53	0.23	1.30	0.26	0.95	1.00	1.15	0.95	1.00	104	18.00	4.09	108	4.000
26.50	100	120.00	1.59	0.27	1.32	0.26	0.94	1.00	1.15	0.95	1.00	103	18.00	4.09	107	4.000
27.50	100	120.00	1.65	0.30	1.35	0.26	0.94	1.00	1.15	0.95	1.00	102	18.00	4.09	106	4.000
28.50	100	120.00	1.71	0.33	1.38	0.26	0.93	1.00	1.15	0.95	1.00	102	18.00	4.09	106	4.000
29.50	100	120.00	1.77	0.36	1.41	0.26	0.93	1.00	1.15	0.95	1.00	101	18.00	4.09	105	4.000
30.50	100	120.00	1.83	0.39	1.44	0.26	0.92	1.00	1.15	1.00	1.00	106	18.00	4.09	110	4.000
31.50	100	120.00	1.89	0.42	1.47	0.26	0.92	1.00	1.15	1.00	1.00	106	18.00	4.09	110	4.000
32.50	100	120.00	1.95	0.45	1.50	0.26	0.91	1.00	1.15	1.00	1.00	105	18.00	4.09	109	4.000
33.50	100	120.00	2.01	0.48	1.53	0.26	0.91	1.00	1.15	1.00	1.00	104	18.00	4.09	108	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
0.50	116.00	0.03	0.00	0.03	1.01	1.00	0.758	1.62	23	0.94	0.806	1.10	0.733	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
1.50	116.00	0.09	0.00	0.09	1.00	1.00	0.757	1.62	23	0.94	0.805	1.10	0.731	2.000	🟢
2.50	116.00	0.15	0.00	0.15	1.00	1.00	0.755	1.62	23	0.94	0.803	1.10	0.730	2.000	🟢
3.50	122.00	0.21	0.00	0.21	1.00	1.00	0.754	1.42	18	0.96	0.785	1.10	0.714	2.000	🟢
4.50	122.00	0.27	0.00	0.27	1.00	1.00	0.752	1.42	18	0.96	0.784	1.10	0.712	2.000	🟢
5.50	122.00	0.33	0.00	0.33	1.00	1.00	0.751	1.53	21	0.95	0.791	1.10	0.719	2.000	🟢
6.50	122.00	0.39	0.00	0.39	0.99	1.00	0.749	1.49	20	0.95	0.787	1.10	0.715	2.000	🟢
7.50	120.00	0.45	0.00	0.45	0.99	1.00	0.748	2.20	54	0.89	0.845	1.10	0.768	2.000	🟢
8.50	120.00	0.51	0.02	0.49	0.99	1.00	0.770	2.20	52	0.89	0.870	1.10	0.791	2.000	🟢
9.50	120.00	0.57	0.05	0.52	0.99	1.00	0.811	2.20	51	0.89	0.916	1.10	0.833	2.000	🟢
10.50	120.00	0.63	0.08	0.55	0.98	1.00	0.848	2.20	52	0.89	0.958	1.10	0.871	2.000	🟢
11.50	120.00	0.69	0.11	0.58	0.98	1.00	0.880	2.20	51	0.89	0.995	1.10	0.904	2.000	🟢
12.50	120.00	0.75	0.14	0.61	0.98	1.00	0.909	2.20	50	0.89	1.027	1.10	0.934	2.000	🟢
13.50	120.00	0.81	0.17	0.64	0.98	1.00	0.935	2.20	39	0.89	1.057	1.10	0.961	2.000	🟢
14.50	120.00	0.87	0.20	0.67	0.97	1.00	0.959	2.20	38	0.89	1.083	1.10	0.985	2.000	🟢
15.50	120.00	0.93	0.23	0.70	0.97	1.00	0.980	2.20	38	0.89	1.107	1.10	1.006	2.000	🟢
16.50	120.00	0.99	0.27	0.72	0.97	1.00	0.999	2.20	41	0.89	1.128	1.10	1.026	2.000	🟢
17.50	120.00	1.05	0.30	0.75	0.97	1.00	1.016	2.20	40	0.89	1.148	1.10	1.043	2.000	🟢
18.50	120.00	1.11	0.33	0.78	0.96	1.00	1.031	1.45	19	0.96	1.078	1.04	1.038	0.187	🔴
19.50	120.00	1.17	0.36	0.81	0.96	1.00	1.045	1.45	19	0.96	1.093	1.03	1.056	0.184	🔴
20.50	120.00	1.23	0.39	0.84	0.96	1.00	1.058	1.45	19	0.96	1.106	1.03	1.074	0.181	🔴
21.50	120.00	1.29	0.42	0.87	0.95	1.00	1.069	1.45	19	0.96	1.118	1.03	1.090	0.178	🔴
22.50	120.00	1.35	0.45	0.90	0.95	1.00	1.079	1.42	18	0.96	1.124	1.02	1.102	0.167	🔴
23.50	120.00	1.41	0.48	0.93	0.95	1.00	1.089	2.20	109	0.89	1.230	1.04	1.183	2.000	🟢
24.50	120.00	1.47	0.51	0.95	0.95	1.00	1.097	2.20	108	0.89	1.240	1.03	1.203	2.000	🟢
25.50	120.00	1.53	0.55	0.98	0.94	1.00	1.105	2.20	108	0.89	1.248	1.02	1.222	2.000	🟢
26.50	120.00	1.59	0.58	1.01	0.94	1.00	1.111	2.20	107	0.89	1.256	1.01	1.239	2.000	🟢
27.50	120.00	1.65	0.61	1.04	0.94	1.00	1.118	2.20	106	0.89	1.263	1.00	1.256	2.000	🟢
28.50	120.00	1.71	0.64	1.07	0.93	1.00	1.123	2.20	106	0.89	1.269	1.00	1.273	2.000	🟢
29.50	120.00	1.77	0.67	1.10	0.93	1.00	1.128	2.20	105	0.89	1.274	0.99	1.288	2.000	🟢
30.50	120.00	1.83	0.70	1.13	0.93	1.00	1.132	2.20	110	0.89	1.279	0.98	1.303	2.000	🟢
31.50	120.00	1.89	0.73	1.16	0.92	1.00	1.136	2.20	110	0.89	1.283	0.97	1.317	2.000	🟢
32.50	120.00	1.95	0.76	1.18	0.92	1.00	1.139	2.20	109	0.89	1.287	0.97	1.331	2.000	🟢
33.50	120.00	2.01	0.80	1.21	0.91	1.00	1.141	2.20	108	0.89	1.290	0.96	1.344	2.000	🟢

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
0.50	2.000	0.00	9.92	1.00	0.00
1.50	2.000	0.00	9.77	1.00	0.00
2.50	2.000	0.00	9.62	1.00	0.00
3.50	2.000	0.00	9.47	1.00	0.00
4.50	2.000	0.00	9.31	1.00	0.00
5.50	2.000	0.00	9.16	1.00	0.00
6.50	2.000	0.00	9.01	1.00	0.00
7.50	2.000	0.00	8.86	1.00	0.00
8.50	2.000	0.00	8.70	1.00	0.00
9.50	2.000	0.00	8.55	1.00	0.00
10.50	2.000	0.00	8.40	1.00	0.00
11.50	2.000	0.00	8.25	1.00	0.00
12.50	2.000	0.00	8.10	1.00	0.00
13.50	2.000	0.00	7.94	1.00	0.00
14.50	2.000	0.00	7.79	1.00	0.00
15.50	2.000	0.00	7.64	1.00	0.00
16.50	2.000	0.00	7.49	1.00	0.00
17.50	2.000	0.00	7.33	1.00	0.00
18.50	0.187	0.81	7.18	1.00	1.78
19.50	0.184	0.82	7.03	1.00	1.75
20.50	0.181	0.82	6.88	1.00	1.72
21.50	0.178	0.82	6.72	1.00	1.68
22.50	0.167	0.83	6.57	1.00	1.67
23.50	2.000	0.00	6.42	1.00	0.00
24.50	2.000	0.00	6.27	1.00	0.00
25.50	2.000	0.00	6.11	1.00	0.00
26.50	2.000	0.00	5.96	1.00	0.00
27.50	2.000	0.00	5.81	1.00	0.00
28.50	2.000	0.00	5.66	1.00	0.00
29.50	2.000	0.00	5.50	1.00	0.00
30.50	2.000	0.00	5.35	1.00	0.00
31.50	2.000	0.00	5.20	1.00	0.00
32.50	2.000	0.00	5.05	1.00	0.00
33.50	2.000	0.00	4.89	1.00	0.00

Overall potential I_L : 8.60

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
0.50	23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
1.50	23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
2.50	23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
3.50	18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
4.50	18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
5.50	17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
6.50	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
7.50	50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
8.50	52	0.01	-1.75	2.000	0.00	0.00	1.00	0.000	0.00
9.50	51	0.02	-1.67	2.000	0.00	0.00	1.00	0.000	0.00
10.50	52	0.01	-1.75	2.000	0.00	0.00	1.00	0.000	0.00
11.50	51	0.02	-1.67	2.000	0.00	0.00	1.00	0.000	0.00
12.50	50	0.04	-1.59	2.000	0.00	0.00	1.00	0.000	0.00
13.50	39	1.07	-0.73	2.000	0.00	0.00	1.00	0.000	0.00
14.50	38	1.30	-0.65	2.000	0.00	0.00	1.00	0.000	0.00
15.50	38	1.30	-0.65	2.000	0.00	0.00	1.00	0.000	0.00
16.50	41	0.70	-0.88	2.000	0.00	0.00	1.00	0.000	0.00
17.50	40	0.87	-0.80	2.000	0.00	0.00	1.00	0.000	0.00
18.50	19	17.78	0.57	0.187	17.78	2.40	1.00	0.288	0.18
19.50	19	17.78	0.57	0.184	17.78	2.40	1.00	0.288	0.18
20.50	19	17.78	0.57	0.181	17.78	2.40	1.00	0.288	0.18
21.50	19	17.78	0.57	0.178	17.78	2.40	1.00	0.288	0.18
22.50	18	19.85	0.62	0.167	19.85	2.51	1.00	0.301	0.20
23.50	109	0.00	-6.93	2.000	0.00	0.00	1.00	0.000	0.00
24.50	108	0.00	-6.84	2.000	0.00	0.00	1.00	0.000	0.00
25.50	108	0.00	-6.84	2.000	0.00	0.00	1.00	0.000	0.00
26.50	107	0.00	-6.74	2.000	0.00	0.00	1.00	0.000	0.00
27.50	106	0.00	-6.64	2.000	0.00	0.00	1.00	0.000	0.00
28.50	106	0.00	-6.64	2.000	0.00	0.00	1.00	0.000	0.00
29.50	105	0.00	-6.55	2.000	0.00	0.00	1.00	0.000	0.00
30.50	110	0.00	-7.03	2.000	0.00	0.00	1.00	0.000	0.00
31.50	110	0.00	-7.03	2.000	0.00	0.00	1.00	0.000	0.00
32.50	109	0.00	-6.93	2.000	0.00	0.00	1.00	0.000	0.00
33.50	108	0.00	-6.84	2.000	0.00	0.00	1.00	0.000	0.00

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
							Cumulative settlements:	**1.454**	**0.91**

Abbreviations

γ_{lim}:	Limiting shear strain (%)
F_α/N:	Maximun shear strain factor
γ_{max}:	Maximum shear strain (%)
e_v::	Post liquefaction volumetric strain (%)
S_{v-1D}:	Estimated vertical settlement (in)
LDI:	Estimated lateral displacement (ft)

SPT BASED LIQUEFACTION ANALYSIS REPORT

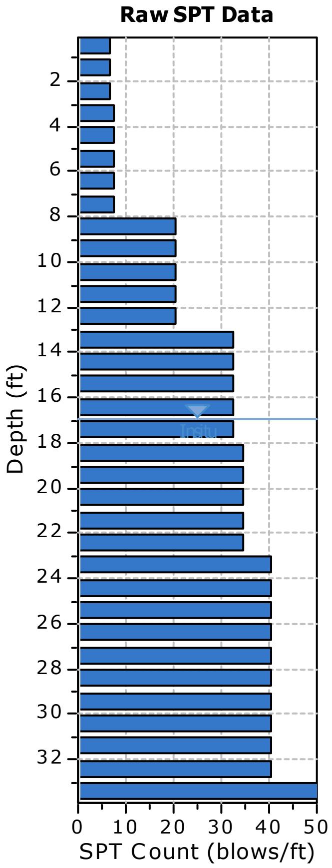
Project title : 575 Los Trancos Road Residence **SPT Name: B-2**

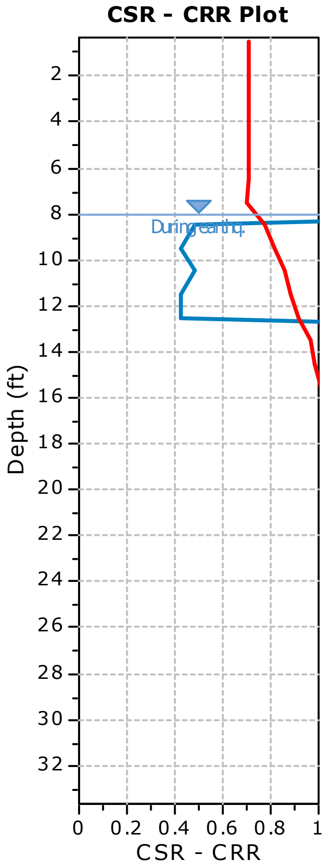
Location : Palo Alto, California

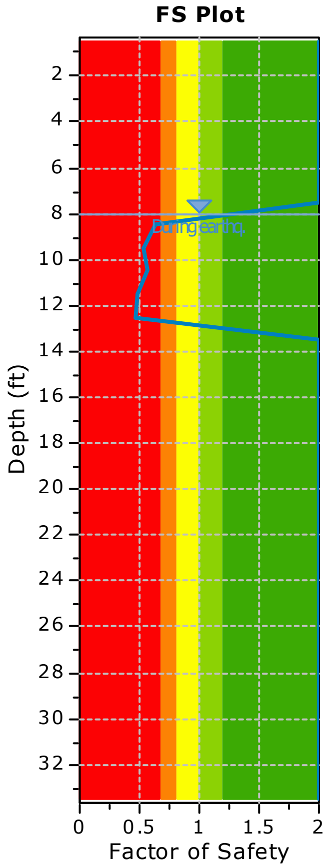
:: Input parameters and analysis properties ::

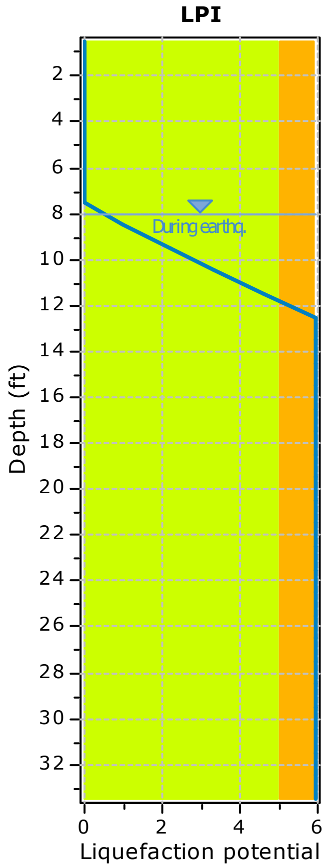
Analysis method: Boulanger & Idriss, 2014
Fines correction method: Boulanger & Idriss, 2014
Sampling method: Standard Sampler
Borehole diameter: 200mm
Rod length: 3.30 ft
Hammer energy ratio: 1.00

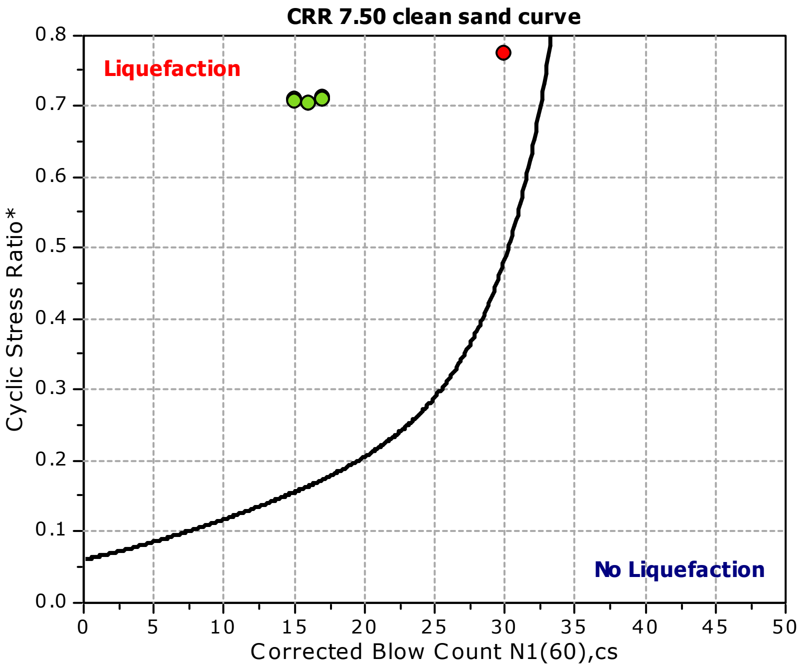
G.W.T. (in-situ): 17.00 ft
G.W.T. (earthq.): 8.00 ft
Earthquake magnitude M_w: 7.80
Peak ground acceleration: 1.16 g
Eq. external load: 0.00 tsf











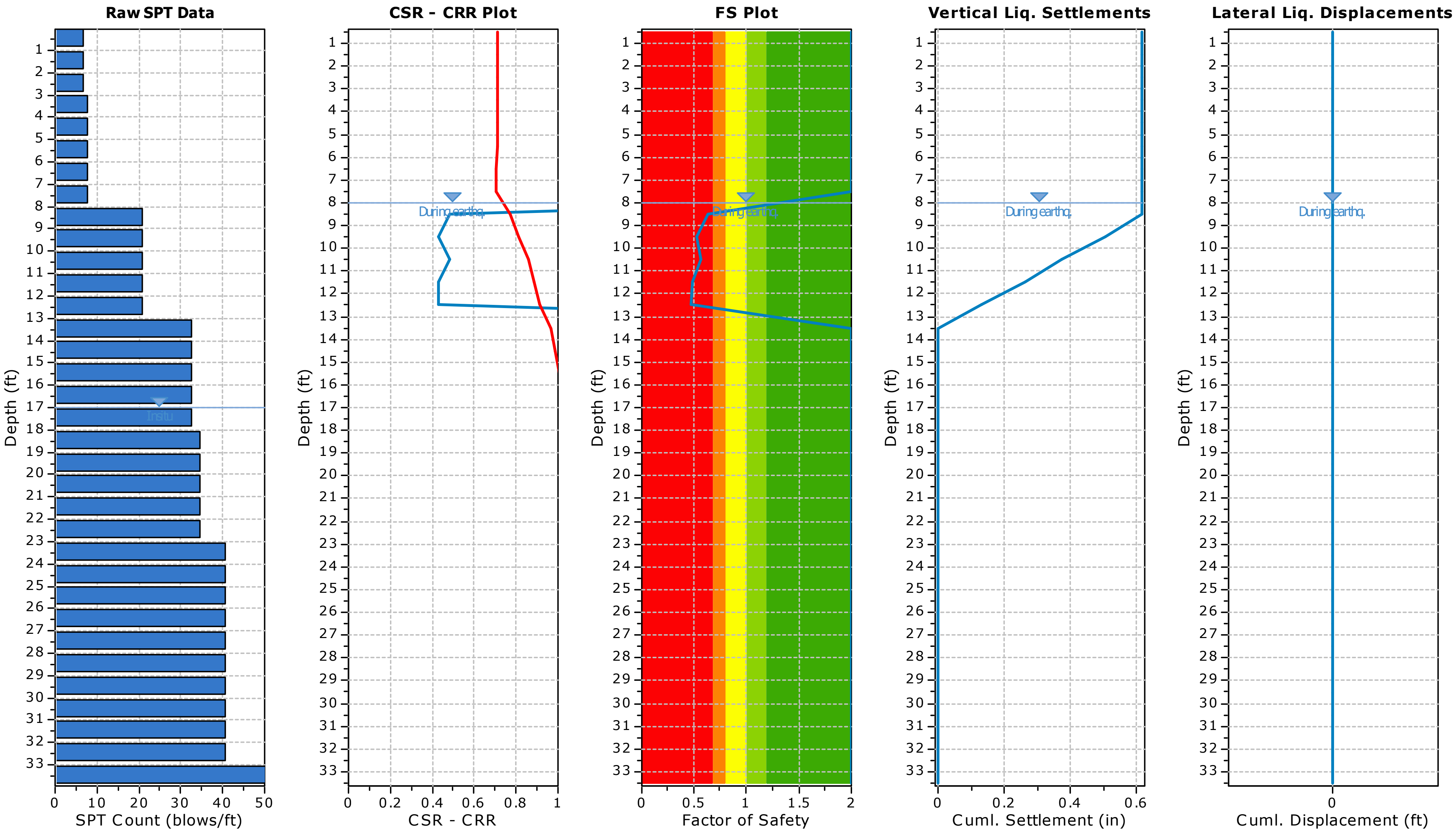
F.S. color scheme

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

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
0.50	7	21.00	117.00	1.00	Yes
1.50	7	21.00	117.00	1.00	Yes
2.50	7	21.00	117.00	1.00	Yes
3.50	8	21.00	113.00	1.00	Yes
4.50	8	21.00	113.00	1.00	Yes
5.50	8	21.00	113.00	1.00	Yes
6.50	8	21.00	113.00	1.00	Yes
7.50	8	21.00	113.00	1.00	Yes
8.50	21	21.00	120.00	1.00	Yes
9.50	21	21.00	120.00	1.00	Yes
10.50	21	21.00	120.00	1.00	Yes
11.50	21	21.00	120.00	1.00	Yes
12.50	21	21.00	120.00	1.00	Yes
13.50	33	21.00	137.00	1.00	Yes
14.50	33	21.00	137.00	1.00	Yes
15.50	33	21.00	137.00	1.00	Yes
16.50	33	21.00	137.00	1.00	Yes
17.50	33	9.00	120.00	1.00	Yes
18.50	35	9.00	120.00	1.00	Yes
19.50	35	9.00	120.00	1.00	Yes
20.50	35	9.00	120.00	1.00	Yes
21.50	35	9.00	120.00	1.00	Yes
22.50	35	9.00	120.00	1.00	Yes
23.50	41	31.00	120.00	1.00	Yes
24.50	41	31.00	120.00	1.00	Yes
25.50	41	31.00	120.00	1.00	Yes
26.50	41	31.00	120.00	1.00	Yes
27.50	41	31.00	120.00	1.00	Yes
28.50	41	9.00	120.00	1.00	Yes
29.50	41	9.00	120.00	1.00	Yes
30.50	41	9.00	120.00	1.00	Yes
31.50	41	9.00	120.00	1.00	Yes
32.50	41	9.00	120.00	1.00	Yes
33.50	100	9.00	120.00	1.00	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
0.50	7	117.00	0.03	0.00	0.03	0.46	1.70	1.00	1.15	0.75	1.00	10	21.00	4.63	15	4.000
1.50	7	117.00	0.09	0.00	0.09	0.46	1.70	1.00	1.15	0.75	1.00	10	21.00	4.63	15	4.000
2.50	7	117.00	0.15	0.00	0.15	0.46	1.70	1.00	1.15	0.75	1.00	10	21.00	4.63	15	4.000

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
3.50	8	113.00	0.20	0.00	0.20	0.44	1.70	1.00	1.15	0.75	1.00	12	21.00	4.63	17	4.000
4.50	8	113.00	0.26	0.00	0.26	0.44	1.70	1.00	1.15	0.75	1.00	12	21.00	4.63	17	4.000
5.50	8	113.00	0.32	0.00	0.32	0.44	1.70	1.00	1.15	0.75	1.00	12	21.00	4.63	17	4.000
6.50	8	113.00	0.37	0.00	0.37	0.45	1.61	1.00	1.15	0.75	1.00	11	21.00	4.63	16	4.000
7.50	8	113.00	0.43	0.00	0.43	0.46	1.51	1.00	1.15	0.80	1.00	11	21.00	4.63	16	4.000
8.50	21	120.00	0.49	0.00	0.49	0.35	1.31	1.00	1.15	0.80	1.00	25	21.00	4.63	30	0.485
9.50	21	120.00	0.55	0.00	0.55	0.36	1.27	1.00	1.15	0.80	1.00	24	21.00	4.63	29	0.429
10.50	21	120.00	0.61	0.00	0.61	0.36	1.22	1.00	1.15	0.85	1.00	25	21.00	4.63	30	0.485
11.50	21	120.00	0.67	0.00	0.67	0.36	1.18	1.00	1.15	0.85	1.00	24	21.00	4.63	29	0.429
12.50	21	120.00	0.73	0.00	0.73	0.37	1.15	1.00	1.15	0.85	1.00	24	21.00	4.63	29	0.429
13.50	33	137.00	0.80	0.00	0.80	0.30	1.09	1.00	1.15	0.85	1.00	35	21.00	4.63	40	4.000
14.50	33	137.00	0.87	0.00	0.87	0.30	1.06	1.00	1.15	0.85	1.00	34	21.00	4.63	39	4.000
15.50	33	137.00	0.93	0.00	0.93	0.31	1.04	1.00	1.15	0.85	1.00	34	21.00	4.63	39	4.000
16.50	33	137.00	1.00	0.00	1.00	0.29	1.02	1.00	1.15	0.95	1.00	37	21.00	4.63	42	4.000
17.50	33	120.00	1.06	0.02	1.05	0.32	1.00	1.00	1.15	0.95	1.00	36	9.00	0.72	37	4.000
18.50	35	120.00	1.12	0.05	1.08	0.31	0.99	1.00	1.15	0.95	1.00	38	9.00	0.72	39	4.000
19.50	35	120.00	1.18	0.08	1.10	0.31	0.99	1.00	1.15	0.95	1.00	38	9.00	0.72	39	4.000
20.50	35	120.00	1.24	0.11	1.13	0.31	0.98	1.00	1.15	0.95	1.00	37	9.00	0.72	38	4.000
21.50	35	120.00	1.30	0.14	1.16	0.31	0.97	1.00	1.15	0.95	1.00	37	9.00	0.72	38	4.000
22.50	35	120.00	1.36	0.17	1.19	0.31	0.96	1.00	1.15	0.95	1.00	37	9.00	0.72	38	4.000
23.50	41	120.00	1.42	0.20	1.22	0.26	0.96	1.00	1.15	0.95	1.00	43	31.00	5.40	48	4.000
24.50	41	120.00	1.48	0.23	1.25	0.26	0.96	1.00	1.15	0.95	1.00	43	31.00	5.40	48	4.000
25.50	41	120.00	1.54	0.27	1.28	0.26	0.95	1.00	1.15	0.95	1.00	43	31.00	5.40	48	4.000
26.50	41	120.00	1.60	0.30	1.31	0.26	0.95	1.00	1.15	0.95	1.00	42	31.00	5.40	47	4.000
27.50	41	120.00	1.66	0.33	1.34	0.26	0.94	1.00	1.15	0.95	1.00	42	31.00	5.40	47	4.000
28.50	41	120.00	1.72	0.36	1.36	0.28	0.93	1.00	1.15	0.95	1.00	42	9.00	0.72	43	4.000
29.50	41	120.00	1.78	0.39	1.39	0.29	0.92	1.00	1.15	0.95	1.00	41	9.00	0.72	42	4.000
30.50	41	120.00	1.84	0.42	1.42	0.27	0.92	1.00	1.15	1.00	1.00	43	9.00	0.72	44	4.000
31.50	41	120.00	1.90	0.45	1.45	0.28	0.92	1.00	1.15	1.00	1.00	43	9.00	0.72	44	4.000
32.50	41	120.00	1.96	0.48	1.48	0.28	0.91	1.00	1.15	1.00	1.00	43	9.00	0.72	44	4.000
33.50	100	120.00	2.02	0.51	1.51	0.26	0.91	1.00	1.15	1.00	1.00	105	9.00	0.72	106	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
0.50	117.00	0.03	0.00	0.03	1.01	1.00	0.758	1.32	15	0.97	0.782	1.10	0.711	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
1.50	117.00	0.09	0.00	0.09	1.00	1.00	0.757	1.32	15	0.97	0.780	1.10	0.709	2.000	🟢
2.50	117.00	0.15	0.00	0.15	1.00	1.00	0.755	1.32	15	0.97	0.779	1.10	0.708	2.000	🟢
3.50	113.00	0.20	0.00	0.20	1.00	1.00	0.754	1.38	17	0.96	0.782	1.10	0.711	2.000	🟢
4.50	113.00	0.26	0.00	0.26	1.00	1.00	0.752	1.38	17	0.96	0.781	1.10	0.710	2.000	🟢
5.50	113.00	0.32	0.00	0.32	1.00	1.00	0.751	1.38	17	0.96	0.779	1.10	0.709	2.000	🟢
6.50	113.00	0.37	0.00	0.37	0.99	1.00	0.749	1.35	16	0.97	0.775	1.10	0.705	2.000	🟢
7.50	113.00	0.43	0.00	0.43	0.99	1.00	0.748	1.35	16	0.97	0.774	1.10	0.703	2.000	🟢
8.50	120.00	0.49	0.02	0.47	0.99	1.00	0.771	2.00	30	0.90	0.852	1.10	0.775	0.626	🔴
9.50	120.00	0.55	0.05	0.50	0.99	1.00	0.814	1.94	29	0.91	0.894	1.10	0.813	0.528	🔴
10.50	120.00	0.61	0.08	0.53	0.98	1.00	0.852	2.00	30	0.90	0.942	1.10	0.856	0.566	🔴
11.50	120.00	0.67	0.11	0.56	0.98	1.00	0.885	1.94	29	0.91	0.973	1.10	0.884	0.485	🔴
12.50	120.00	0.73	0.14	0.59	0.98	1.00	0.915	1.94	29	0.91	1.005	1.10	0.914	0.469	🔴
13.50	137.00	0.80	0.17	0.63	0.98	1.00	0.939	2.20	40	0.89	1.061	1.10	0.965	2.000	🟢
14.50	137.00	0.87	0.20	0.66	0.97	1.00	0.960	2.20	39	0.89	1.084	1.10	0.986	2.000	🟢
15.50	137.00	0.93	0.23	0.70	0.97	1.00	0.978	2.20	39	0.89	1.105	1.10	1.004	2.000	🟢
16.50	137.00	1.00	0.27	0.74	0.97	1.00	0.994	2.20	42	0.89	1.123	1.10	1.021	2.000	🟢
17.50	120.00	1.06	0.30	0.77	0.97	1.00	1.011	2.20	37	0.89	1.142	1.10	1.043	2.000	🟢
18.50	120.00	1.12	0.33	0.80	0.96	1.00	1.026	2.20	39	0.89	1.159	1.08	1.069	2.000	🟢
19.50	120.00	1.18	0.36	0.82	0.96	1.00	1.040	2.20	39	0.89	1.175	1.07	1.094	2.000	🟢
20.50	120.00	1.24	0.39	0.85	0.96	1.00	1.052	2.20	38	0.89	1.189	1.06	1.118	2.000	🟢
21.50	120.00	1.30	0.42	0.88	0.95	1.00	1.064	2.20	38	0.89	1.202	1.05	1.140	2.000	🟢
22.50	120.00	1.36	0.45	0.91	0.95	1.00	1.074	2.20	38	0.89	1.213	1.04	1.162	2.000	🟢
23.50	120.00	1.42	0.48	0.94	0.95	1.00	1.083	2.20	48	0.89	1.224	1.04	1.182	2.000	🟢
24.50	120.00	1.48	0.51	0.97	0.95	1.00	1.092	2.20	48	0.89	1.233	1.03	1.202	2.000	🟢
25.50	120.00	1.54	0.55	1.00	0.94	1.00	1.099	2.20	48	0.89	1.242	1.02	1.220	2.000	🟢
26.50	120.00	1.60	0.58	1.03	0.94	1.00	1.106	2.20	47	0.89	1.250	1.01	1.238	2.000	🟢
27.50	120.00	1.66	0.61	1.05	0.94	1.00	1.112	2.20	47	0.89	1.257	1.00	1.255	2.000	🟢
28.50	120.00	1.72	0.64	1.08	0.93	1.00	1.118	2.20	43	0.89	1.263	0.99	1.271	2.000	🟢
29.50	120.00	1.78	0.67	1.11	0.93	1.00	1.122	2.20	42	0.89	1.268	0.99	1.287	2.000	🟢
30.50	120.00	1.84	0.70	1.14	0.93	1.00	1.127	2.20	44	0.89	1.273	0.98	1.302	2.000	🟢
31.50	120.00	1.90	0.73	1.17	0.92	1.00	1.130	2.20	44	0.89	1.277	0.97	1.316	2.000	🟢
32.50	120.00	1.96	0.76	1.20	0.92	1.00	1.134	2.20	44	0.89	1.281	0.96	1.330	2.000	🟢
33.50	120.00	2.02	0.80	1.23	0.91	1.00	1.136	2.20	106	0.89	1.284	0.96	1.343	2.000	🟢

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
0.50	2.000	0.00	9.92	1.00	0.00
1.50	2.000	0.00	9.77	1.00	0.00
2.50	2.000	0.00	9.62	1.00	0.00
3.50	2.000	0.00	9.47	1.00	0.00
4.50	2.000	0.00	9.31	1.00	0.00
5.50	2.000	0.00	9.16	1.00	0.00
6.50	2.000	0.00	9.01	1.00	0.00
7.50	2.000	0.00	8.86	1.00	0.00
8.50	0.626	0.37	8.70	1.00	0.99
9.50	0.528	0.47	8.55	1.00	1.23
10.50	0.566	0.43	8.40	1.00	1.11
11.50	0.485	0.51	8.25	1.00	1.29
12.50	0.469	0.53	8.10	1.00	1.31
13.50	2.000	0.00	7.94	1.00	0.00
14.50	2.000	0.00	7.79	1.00	0.00
15.50	2.000	0.00	7.64	1.00	0.00
16.50	2.000	0.00	7.49	1.00	0.00
17.50	2.000	0.00	7.33	1.00	0.00
18.50	2.000	0.00	7.18	1.00	0.00
19.50	2.000	0.00	7.03	1.00	0.00
20.50	2.000	0.00	6.88	1.00	0.00
21.50	2.000	0.00	6.72	1.00	0.00
22.50	2.000	0.00	6.57	1.00	0.00
23.50	2.000	0.00	6.42	1.00	0.00
24.50	2.000	0.00	6.27	1.00	0.00
25.50	2.000	0.00	6.11	1.00	0.00
26.50	2.000	0.00	5.96	1.00	0.00
27.50	2.000	0.00	5.81	1.00	0.00
28.50	2.000	0.00	5.66	1.00	0.00
29.50	2.000	0.00	5.50	1.00	0.00
30.50	2.000	0.00	5.35	1.00	0.00
31.50	2.000	0.00	5.20	1.00	0.00
32.50	2.000	0.00	5.05	1.00	0.00
33.50	2.000	0.00	4.89	1.00	0.00

Overall potential I_L : 5.94

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
0.50	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
1.50	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
2.50	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
3.50	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ϵ_{15}	N_c	ϵ_{Nc} (%)	Δh (ft)	ΔS (in)
4.50	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
5.50	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
6.50	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000
7.50	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.000

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ϵ_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ϵ_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
8.50	30	4.65	-0.09	0.626	4.65	0.92	1.00	0.111	0.00
9.50	29	5.33	-0.02	0.528	5.33	1.10	1.00	0.131	0.00
10.50	30	4.65	-0.09	0.566	4.65	0.92	1.00	0.111	0.00
11.50	29	5.33	-0.02	0.485	5.33	1.10	1.00	0.131	0.00
12.50	29	5.33	-0.02	0.469	5.33	1.10	1.00	0.131	0.00
13.50	40	0.87	-0.80	2.000	0.00	0.00	1.00	0.000	0.00
14.50	39	1.07	-0.73	2.000	0.00	0.00	1.00	0.000	0.00
15.50	39	1.07	-0.73	2.000	0.00	0.00	1.00	0.000	0.00
16.50	42	0.56	-0.96	2.000	0.00	0.00	1.00	0.000	0.00
17.50	37	1.56	-0.58	2.000	0.00	0.00	1.00	0.000	0.00
18.50	39	1.07	-0.73	2.000	0.00	0.00	1.00	0.000	0.00
19.50	39	1.07	-0.73	2.000	0.00	0.00	1.00	0.000	0.00
20.50	38	1.30	-0.65	2.000	0.00	0.00	1.00	0.000	0.00
21.50	38	1.30	-0.65	2.000	0.00	0.00	1.00	0.000	0.00
22.50	38	1.30	-0.65	2.000	0.00	0.00	1.00	0.000	0.00
23.50	48	0.09	-1.43	2.000	0.00	0.00	1.00	0.000	0.00
24.50	48	0.09	-1.43	2.000	0.00	0.00	1.00	0.000	0.00
25.50	48	0.09	-1.43	2.000	0.00	0.00	1.00	0.000	0.00
26.50	47	0.13	-1.35	2.000	0.00	0.00	1.00	0.000	0.00
27.50	47	0.13	-1.35	2.000	0.00	0.00	1.00	0.000	0.00
28.50	43	0.44	-1.03	2.000	0.00	0.00	1.00	0.000	0.00
29.50	42	0.56	-0.96	2.000	0.00	0.00	1.00	0.000	0.00
30.50	44	0.34	-1.11	2.000	0.00	0.00	1.00	0.000	0.00
31.50	44	0.34	-1.11	2.000	0.00	0.00	1.00	0.000	0.00
32.50	44	0.34	-1.11	2.000	0.00	0.00	1.00	0.000	0.00
33.50	106	0.00	-6.64	2.000	0.00	0.00	1.00	0.000	0.00

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)

Cumulative settlements: 0.616 0.00

Abbreviations

γ_{lim}:	Limiting shear strain (%)
F_α/N:	Maximun shear strain factor
γ_{max}:	Maximum shear strain (%)
e_v::	Post liquefaction volumetric strain (%)
S_{v-1D}:	Estimated vertical settlement (in)
LDI:	Estimated lateral displacement (ft)

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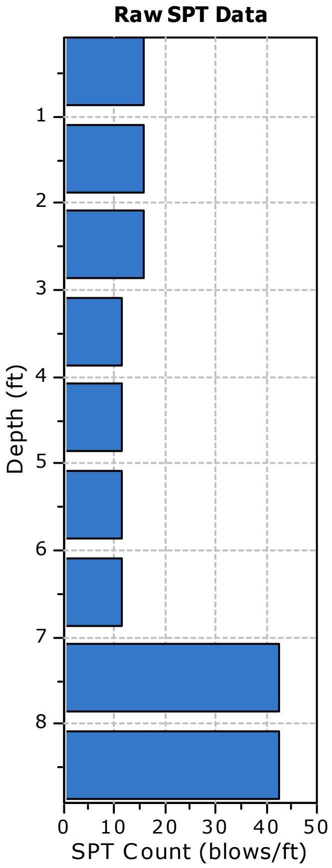
SPT BASED LIQUEFACTION ANALYSIS REPORT

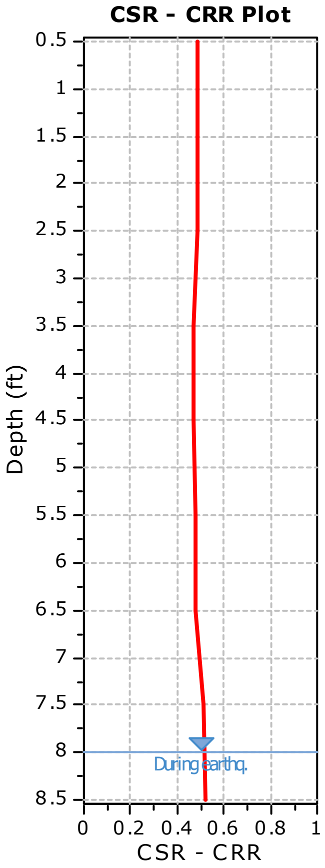
Project title : 575 Los Trancos Road Residence, Dry Sand **SPT Name: B-1**

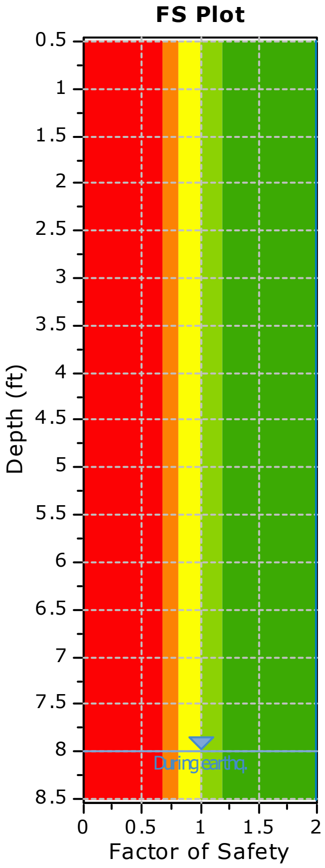
Location : Palo Alto, California

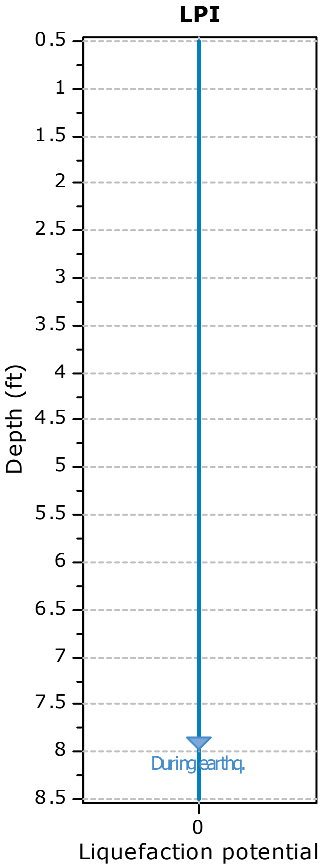
:: Input parameters and analysis properties ::

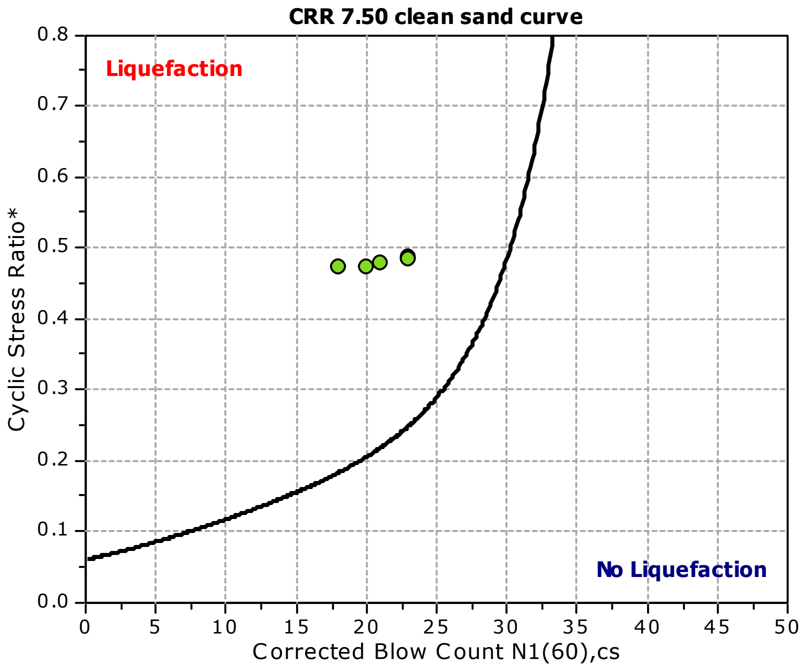
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	18.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	8.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.80
Borehole diameter:	200mm	Peak ground acceleration:	0.77 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		











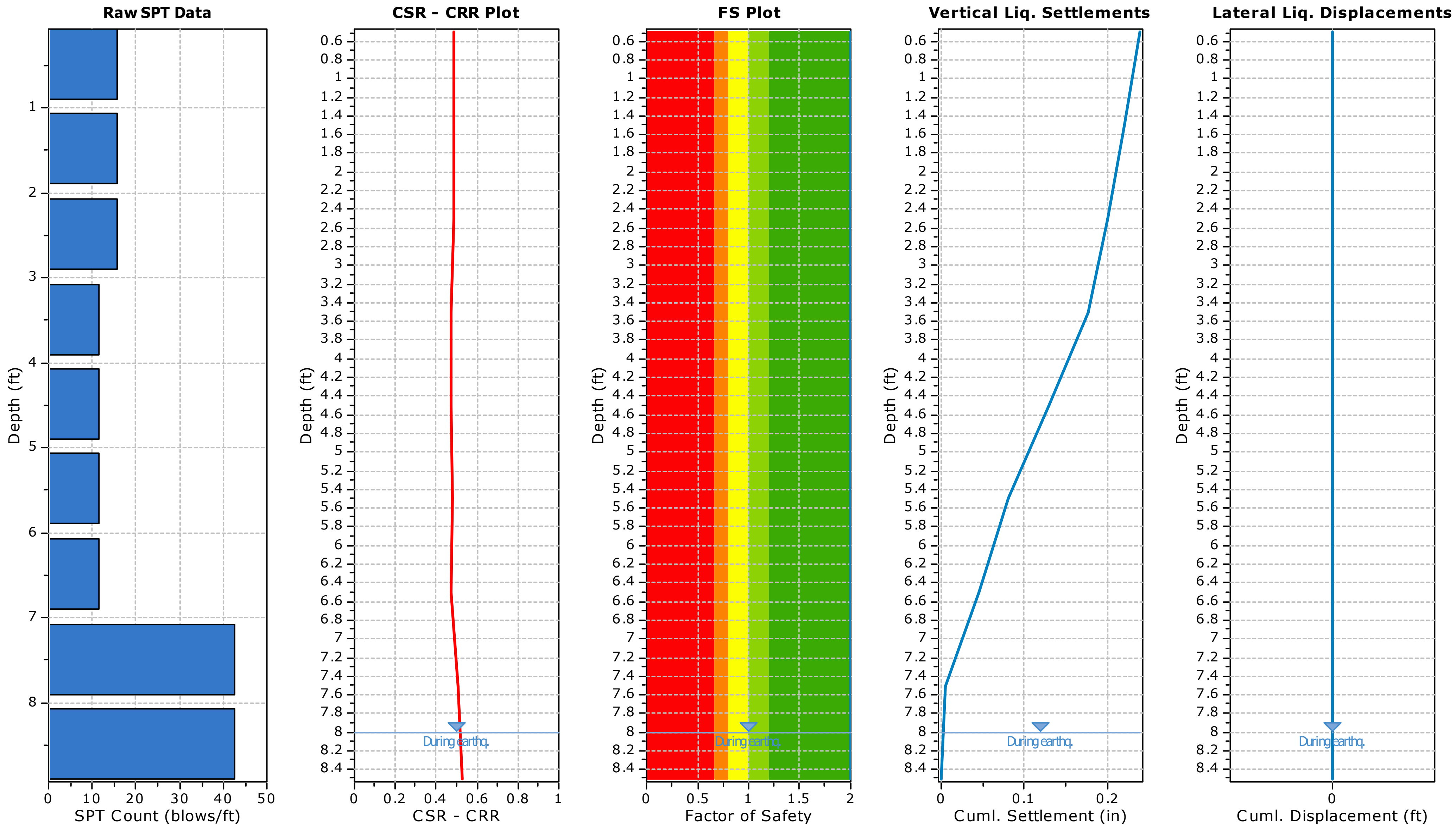
F.S. color scheme

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

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
0.50	16	5.00	116.00	1.00	Yes
1.50	16	5.00	116.00	1.00	Yes
2.50	16	5.00	116.00	1.00	Yes
3.50	12	5.00	122.00	1.00	Yes
4.50	12	5.00	122.00	1.00	Yes
5.50	12	18.00	122.00	1.00	Yes
6.50	12	18.00	122.00	1.00	Yes
7.50	43	18.00	120.00	1.00	No
8.50	43	18.00	120.00	1.00	No

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
0.50	16	116.00	0.03	0.00	0.03	0.41	1.70	1.00	1.15	0.75	1.00	23	5.00	0.00	23	4.000
1.50	16	116.00	0.09	0.00	0.09	0.41	1.70	1.00	1.15	0.75	1.00	23	5.00	0.00	23	4.000
2.50	16	116.00	0.15	0.00	0.15	0.41	1.70	1.00	1.15	0.75	1.00	23	5.00	0.00	23	4.000
3.50	12	122.00	0.21	0.00	0.21	0.46	1.70	1.00	1.15	0.75	1.00	18	5.00	0.00	18	4.000
4.50	12	122.00	0.27	0.00	0.27	0.46	1.70	1.00	1.15	0.75	1.00	18	5.00	0.00	18	4.000
5.50	12	122.00	0.33	0.00	0.33	0.41	1.62	1.00	1.15	0.75	1.00	17	18.00	4.09	21	4.000
6.50	12	122.00	0.39	0.00	0.39	0.42	1.53	1.00	1.15	0.75	1.00	16	18.00	4.09	20	4.000
7.50	43	120.00	0.45	0.00	0.45	0.26	1.25	1.00	1.15	0.80	1.00	50	18.00	4.09	54	4.000
8.50	43	120.00	0.51	0.00	0.51	0.26	1.21	1.00	1.15	0.80	1.00	48	18.00	4.09	52	4.000

Abbreviations

σ_v:	Total stress during SPT test (tsf)
u_o:	Water pore pressure during SPT test (tsf)
σ'_{vo}:	Effective overburden pressure during SPT test (tsf)
m:	Stress exponent normalization factor
C_N:	Overburden corretion factor
C_E:	Energy correction factor
C_B:	Borehole diameter correction factor
C_R:	Rod length correction factor
C_S:	Liner correction factor
$N_{1(60)}$:	Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$	Equivalent clean sand adjustment
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
0.50	116.00	0.03	0.00	0.03	1.01	1.00	0.503	1.62	23	0.94	0.535	1.10	0.486	2.000	●
1.50	116.00	0.09	0.00	0.09	1.00	1.00	0.502	1.62	23	0.94	0.534	1.10	0.486	2.000	●
2.50	116.00	0.15	0.00	0.15	1.00	1.00	0.501	1.62	23	0.94	0.533	1.10	0.485	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
3.50	122.00	0.21	0.00	0.21	1.00	1.00	0.500	1.42	18	0.96	0.521	1.10	0.474	2.000	●
4.50	122.00	0.27	0.00	0.27	1.00	1.00	0.499	1.42	18	0.96	0.520	1.10	0.473	2.000	●
5.50	122.00	0.33	0.00	0.33	1.00	1.00	0.498	1.53	21	0.95	0.525	1.10	0.478	2.000	●
6.50	122.00	0.39	0.00	0.39	0.99	1.00	0.497	1.49	20	0.95	0.522	1.10	0.475	2.000	●
7.50	120.00	0.45	0.00	0.45	0.99	1.00	0.496	2.20	54	0.89	0.561	1.10	0.510	2.000	●
8.50	120.00	0.51	0.02	0.49	0.99	1.00	0.511	2.20	52	0.89	0.577	1.10	0.525	2.000	●

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
0.50	2.000	0.00	9.92	1.00	0.00
1.50	2.000	0.00	9.77	1.00	0.00
2.50	2.000	0.00	9.62	1.00	0.00
3.50	2.000	0.00	9.47	1.00	0.00
4.50	2.000	0.00	9.31	1.00	0.00
5.50	2.000	0.00	9.16	1.00	0.00
6.50	2.000	0.00	9.01	1.00	0.00
7.50	2.000	0.00	8.86	1.00	0.00
8.50	2.000	0.00	8.70	1.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
0.50	23	0.01	0.02	0.18	0.13	53547.74	0.00	0.00	18.12	0.08	1.00	0.018
1.50	23	0.04	0.06	0.31	0.13	27699.28	0.00	0.00	18.12	0.09	1.00	0.021
2.50	23	0.07	0.10	0.40	0.13	20387.27	0.00	0.00	18.12	0.10	1.00	0.023
3.50	18	0.10	0.14	0.44	0.13	16514.28	0.00	0.00	18.12	0.20	1.00	0.047
4.50	18	0.13	0.18	0.50	0.13	14134.26	0.00	0.00	18.12	0.21	1.00	0.049
5.50	17	0.16	0.22	0.58	0.14	12492.68	0.00	0.00	18.12	0.15	1.00	0.035
6.50	16	0.19	0.26	0.62	0.14	11277.43	0.00	0.00	18.12	0.17	1.00	0.041
7.50	50	0.22	0.30	0.93	0.14	10347.42	0.00	0.00	18.12	0.02	1.00	0.005

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)

Cumulative settlemetns: 0.239

Abbreviations

τ_{av}:	Average cyclic shear stress
p:	Average stress
G_{max}:	Maximum shear modulus (tsf)
α, b:	Shear strain formula variables
γ:	Average shear strain
ε_{15}:	Volumetric strain after 15 cycles
N_c:	Number of cycles
ε_{Nc}:	Volumetric strain for number of cycles N_c (%)
Δh:	Thickness of soil layer (in)
ΔS:	Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
8.50	52	0.00	0.00	2.000	0.00	0.00	1.00	0.000	0.00

Cumulative settlements: 0.000 0.00

Abbreviations

γ_{lim}:	Limiting shear strain (%)
F_α/N:	Maximun shear strain factor
γ_{max}:	Maximum shear strain (%)
e_v::	Post liquefaction volumetric strain (%)
S_{v-1D}:	Estimated vertical settlement (in)
LDI:	Estimated lateral displacement (ft)

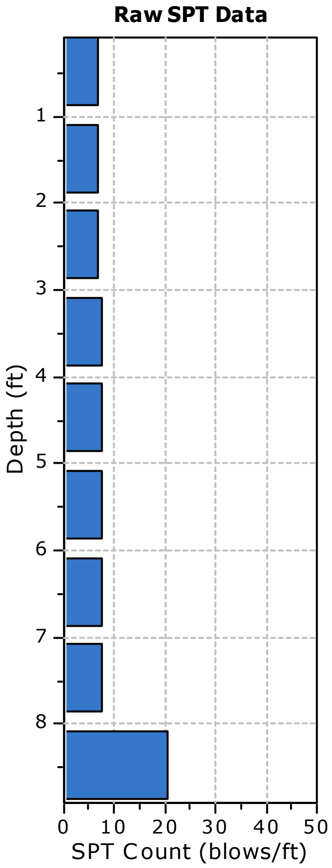
SPT BASED LIQUEFACTION ANALYSIS REPORT

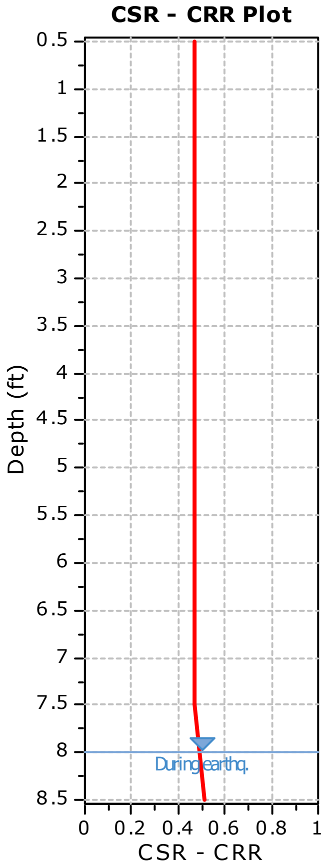
Project title : 575 Los Trancos Road Residence, Dry Sand **SPT Name: B-2**

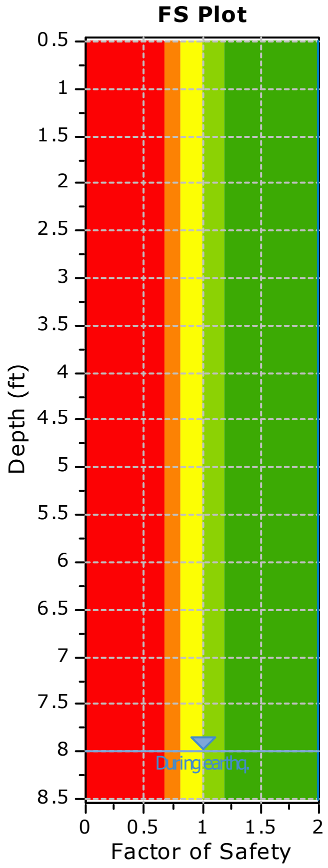
Location : Palo Alto, California

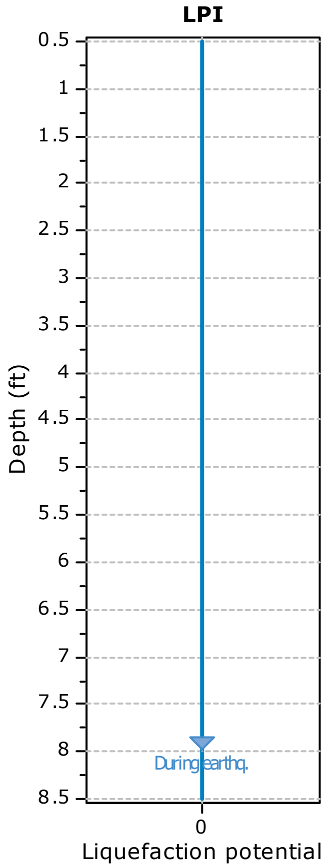
:: Input parameters and analysis properties ::

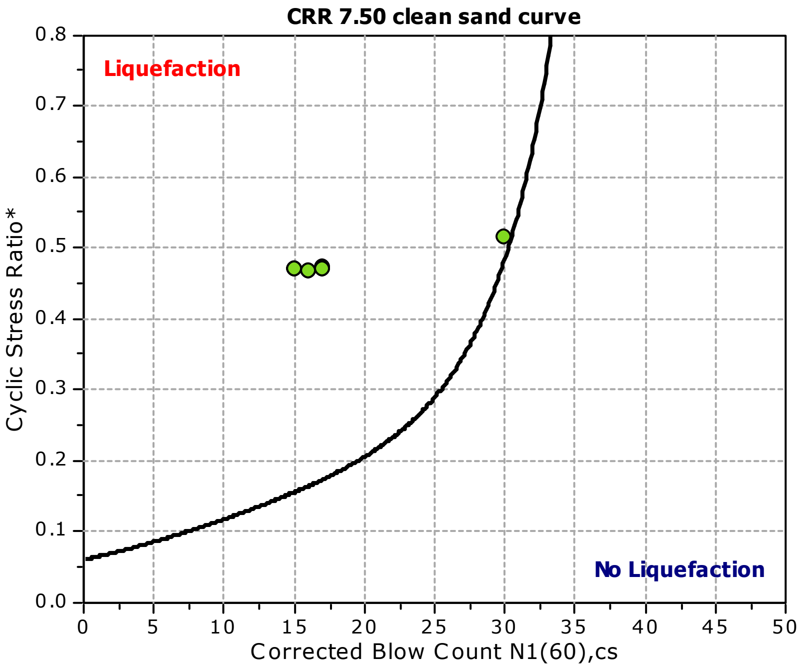
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	17.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	8.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.80
Borehole diameter:	200mm	Peak ground acceleration:	0.77 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		


Raw SPT Data


CSR - CRR Plot


FS Plot


LPI


CRR 7.50 clean sand curve

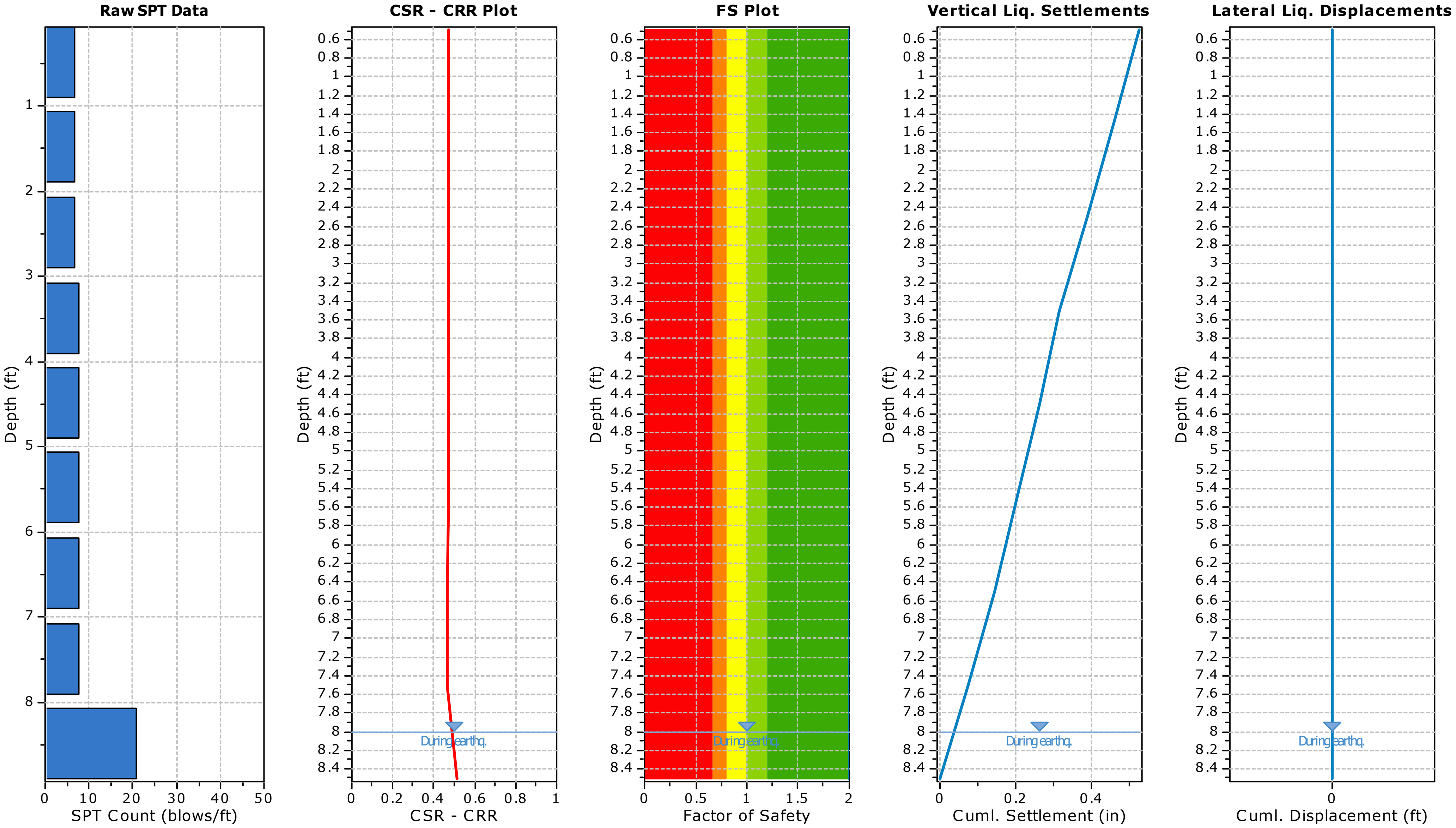
F.S. color scheme

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

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
0.50	7	21.00	117.00	1.00	Yes
1.50	7	21.00	117.00	1.00	Yes
2.50	7	21.00	117.00	1.00	Yes
3.50	8	21.00	113.00	1.00	Yes
4.50	8	21.00	113.00	1.00	Yes
5.50	8	21.00	113.00	1.00	Yes
6.50	8	21.00	113.00	1.00	Yes
7.50	8	21.00	113.00	1.00	No
8.50	21	21.00	120.00	1.00	No

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
0.50	7	117.00	0.03	0.00	0.03	0.46	1.70	1.00	1.15	0.75	1.00	10	21.00	4.63	15	4.000
1.50	7	117.00	0.09	0.00	0.09	0.46	1.70	1.00	1.15	0.75	1.00	10	21.00	4.63	15	4.000
2.50	7	117.00	0.15	0.00	0.15	0.46	1.70	1.00	1.15	0.75	1.00	10	21.00	4.63	15	4.000
3.50	8	113.00	0.20	0.00	0.20	0.44	1.70	1.00	1.15	0.75	1.00	12	21.00	4.63	17	4.000
4.50	8	113.00	0.26	0.00	0.26	0.44	1.70	1.00	1.15	0.75	1.00	12	21.00	4.63	17	4.000
5.50	8	113.00	0.32	0.00	0.32	0.44	1.70	1.00	1.15	0.75	1.00	12	21.00	4.63	17	4.000
6.50	8	113.00	0.37	0.00	0.37	0.45	1.61	1.00	1.15	0.75	1.00	11	21.00	4.63	16	4.000
7.50	8	113.00	0.43	0.00	0.43	0.46	1.51	1.00	1.15	0.80	1.00	11	21.00	4.63	16	4.000
8.50	21	120.00	0.49	0.00	0.49	0.35	1.31	1.00	1.15	0.80	1.00	25	21.00	4.63	30	4.000

Abbreviations

σ_v:	Total stress during SPT test (tsf)
u_o:	Water pore pressure during SPT test (tsf)
σ'_{vo}:	Effective overburden pressure during SPT test (tsf)
m:	Stress exponent normalization factor
C_N:	Overburden corretion factor
C_E:	Energy correction factor
C_B:	Borehole diameter correction factor
C_R:	Rod length correction factor
C_S:	Liner correction factor
$N_{1(60)}$:	Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$	Equivalent clean sand adjustment
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
0.50	117.00	0.03	0.00	0.03	1.01	1.00	0.503	1.32	15	0.97	0.519	1.10	0.472	2.000	●
1.50	117.00	0.09	0.00	0.09	1.00	1.00	0.502	1.32	15	0.97	0.518	1.10	0.471	2.000	●
2.50	117.00	0.15	0.00	0.15	1.00	1.00	0.501	1.32	15	0.97	0.517	1.10	0.470	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
3.50	113.00	0.20	0.00	0.20	1.00	1.00	0.500	1.38	17	0.96	0.519	1.10	0.472	2.000	●
4.50	113.00	0.26	0.00	0.26	1.00	1.00	0.499	1.38	17	0.96	0.518	1.10	0.471	2.000	●
5.50	113.00	0.32	0.00	0.32	1.00	1.00	0.498	1.38	17	0.96	0.517	1.10	0.470	2.000	●
6.50	113.00	0.37	0.00	0.37	0.99	1.00	0.497	1.35	16	0.97	0.515	1.10	0.468	2.000	●
7.50	113.00	0.43	0.00	0.43	0.99	1.00	0.496	1.35	16	0.97	0.513	1.10	0.467	2.000	●
8.50	120.00	0.49	0.02	0.47	0.99	1.00	0.512	2.00	30	0.90	0.566	1.10	0.514	2.000	●

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
0.50	2.000	0.00	9.92	1.00	0.00
1.50	2.000	0.00	9.77	1.00	0.00
2.50	2.000	0.00	9.62	1.00	0.00
3.50	2.000	0.00	9.47	1.00	0.00
4.50	2.000	0.00	9.31	1.00	0.00
5.50	2.000	0.00	9.16	1.00	0.00
6.50	2.000	0.00	9.01	1.00	0.00
7.50	2.000	0.00	8.86	1.00	0.00
8.50	2.000	0.00	8.70	1.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
0.50	10	0.01	0.02	0.15	0.13	53272.67	0.00	0.00	18.12	0.28	1.00	0.066
1.50	10	0.04	0.06	0.27	0.13	27556.98	0.00	0.00	18.12	0.29	1.00	0.070
2.50	10	0.07	0.10	0.35	0.13	20282.55	0.00	0.00	18.12	0.31	1.00	0.074
3.50	12	0.10	0.14	0.42	0.13	16672.60	0.00	0.00	18.12	0.23	1.00	0.055
4.50	12	0.13	0.17	0.48	0.13	14386.29	0.00	0.00	18.12	0.24	1.00	0.057
5.50	12	0.16	0.21	0.53	0.14	12781.27	0.00	0.00	18.12	0.25	1.00	0.059
6.50	11	0.19	0.25	0.56	0.14	11579.22	0.00	0.00	18.12	0.30	1.00	0.072
7.50	11	0.21	0.29	0.60	0.14	10637.94	0.00	0.00	18.12	0.31	1.00	0.074

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)

Cumulative settlemetns: 0.527

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	$F_α$	FS_{liq}	γ_{max} (%)	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
8.50	30	0.00	0.00	2.000	0.00	0.00	1.00	0.000	0.00

Cumulative settlements: 0.000 0.00

Abbreviations

γ_{lim}: Limiting shear strain (%)
$F_α$/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

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