GEOLOTECH INC
GEOLOGICAL & GEOTECHNICAL ENGINEERING

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed Multi-Story Building with One Level of Underground Parking

Tract: 19363, Lot: 8
6616 N. Reseda Blvd.
Reseda, CA

for

Reseda Senior Housing JV
c/o Land Use Developers
7136 Haskell Ave., Suite 320
Van Nuys, CA 91406

Project 20008
March 24, 2020

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INTRODUCTION

This report presents the results of a Preliminary Geotechnical Engineering investigation on the property located at 6616 N. Reseda Blvd, Reseda, California. The purpose of this study is to evaluate the subsurface conditions of the subject site and to provide geotechnical engineering recommendations with respect to the proposed development. The report includes a description of the encountered earth materials and the local geology, presentation of reconnaissance mapping, engineering analyses, descriptions of the obtained soil samples, and the results of laboratory testing.

PROPOSED DEVELOPMENT

Final building plans are not yet available; however, it is understood that the subject property will be developed with a seven-story building over one level of underground parking. The proposed development is shown on the enclosed Geotechnical Map and Cross Section.

The proposed grading at the site consists of excavation for the new basement and foundation excavation. The anticipated design loads will be 6 to 8 kips per lineal foot for continuous footings and 500 to 600 kips for pad footings. Final civil and structural plans have not yet been prepared for the proposed development. The final plans should be reviewed by this office to ensure that our recommendations have been followed.

SCOPE OF WORK

The following have been performed as part of this investigation and presented in the report;

- Research at the City of Los Angeles for available geotechnical reports or letters for the subject site and adjacent properties (see References).
- Review of regional geologic and seismic hazard maps.
- Excavation and detailed logging of two borings.
- Sampling of representative earth materials and laboratory testing (see Appendix B).
- Preparation of the enclosed Geotechnical Map and one Cross Section, (see Appendix A). De Land Services prepared the topographic base map utilized in this investigation.
- Analyzing the obtained geotechnical data.
- Preparation of this report and presentation of findings, conclusions, and recommendations for the proposed project.

The scope of this investigation is limited to the project area explored as depicted on the enclosed Geotechnical Map. This report has been prepared for the exclusive use of Reseda Senior Housing JV, and may not be used by other parties or for purposes other than the proposed project.

SITE CONDITIONS

Location

The property is located in the City of Los Angeles, Los Angeles County, and it may be accessed from Reseda Boulevard. Kittridge Street to the south is the nearest major intersecting street (see Vicinity Map in Appendix A).

The existing development on the property consists of a one-story commercial building with storages and a carport. The lot size of the property is approximately 0.4 acre. The adjacent properties to the north and east have been developed with one-story commercial buildings. The adjacent structure to the north is approximately along the property line. The adjacent structure to the east is about 7 feet from the property line.

Topography

The subject site is situated in a generally flat area within northern portion of Los Angeles County. Details of the topography are depicted on the subject sites Geotechnical Map in Appendix A.

Surface Water

Surface water at the site consists of direct precipitation. The surface water drains as sheet flow down descending slopes to low-lying areas, area drains, and/or offsite. Area drains were observed in the yard areas. Two subdrain outlet pipes are located in the curb along Kittridge Street and Reseda Boulevard.

PREVIOUS WORK

According to the available data obtained from the City of Los Angeles, the subject property was developed in 1946. No geology and/or geotechnical reports were found on file at the City of Los Angeles, Department of Building and Safety, covering the grading and/or construction of the site.

GEOLOGY

Regional Geology

The subject site is located within the south-central portion of the Transverse Ranges geologic province which is a narrow long east-west trending province composed of several east-west trending mountain Ranges such as the Santa Monica and San Gabriel Mountains. The

Transverse Ranges are geologically characterized by east-west trending folds and faults indicative of active tectonism in the region caused by the northwesterly movement of the Pacific Plate with relation to the Atlantic Plate along the San Andreas Fault.

Local Geology

The subject site is underlain by Quaternary (Q) alluvial deposits that have originated from the Santa Monica Mountains. The site is also located along the northern flank of the Los Angeles River.

Groundwater

Groundwater seeps or springs were not observed on the site at the time of field investigation. However, groundwater was encountered in the excavated boring at a depth of 29 feet below the surface. The depth to groundwater, when encountered in the explorations, is only valid for the date of exploration. Based on the Seismic Hazard Zone Report by the California Geological Survey (formerly Division of Mines and Geology), the depth to historical high groundwater level is 5 to 10 feet below the surface. The groundwater elevation may fluctuate seasonally due to varying amounts of rainfall, irrigation, and the rate of groundwater recharge. However, these fluctuations are often gradual.

SUBSURFACE INVESTIGATION

Field Exploration

The subject site was explored by this firm on March 5, 2020. The surface mapping of the site was supplemented with subsurface exploration by drilling two borings utilizing a truck mounted hollow-stem flight auger drill rig. The subsurface explorations were excavated to a maximum depth of 61.5 feet and logged in detail by the undersigned geologist (see Exploration Logs in Appendix A). The locations of these excavations are shown on the Geotechnical Map and Cross Section.

The subsurface exploration program was developed based on the preliminary plans of the proposed development available at the time of the investigation, and explorations were generally limited to the proposed project area. The subsurface investigation was limited by existing structures, hardscape, and underground utilities on the property.

All explorations were backfilled with the excavated materials upon completion of the field investigation and patched with asphalt. However, the backfill was not compacted and some future settlement within the excavated areas should be anticipated.

Sampling

During the field exploration, undisturbed and bulk samples of the encountered earth materials were obtained and preserved in moisture-tight containers for later laboratory testing. Two and one-half (2½) inch diameter undisturbed Modified California (MC) samples and Standard Penetration Test (SPT) bulk samples were obtained from the explorations at various depths. The undisturbed and bulk samples from the drilled borings were attained through the use of a steel sampler with successive blows of a 140-pound drop hammer dropped 30 inches. The results of the laboratory testing program are attached to this report for reference (Appendix B).

Earth Material

The encountered earth materials on the site are briefly described below. Detailed descriptions of the explorations and approximate depths of the earth materials are given in the enclosed Exploration Logs (see Appendix A).

Artificial Fill (Af)

Artificial fill was encountered on the site in both borings. The fill thickness varied from 5 feet in boring (B-2) to 7 feet in boring (B-1). Fill materials were composed of very dark yellowish-brown silty clay with abundant asphalt debris. The contact between the fill and the underlying alluvium was exposed within the explorations. The approximate limit of the encountered artificial fill is shown on the attached Geotechnical Map and Cross Section. The existing artificial fill should be considered uncertified and not suitable for support of future structural foundations or engineered fills.

Quaternary Alluvium (Qal)

The subject site is located in an area of thick alluvial deposits that have been accumulating since Pleistocene time. These deposits were encountered within both exploratory borings to the depth of the exploration. The alluvium generally consists of very dark to dark yellowish-brown clayey silt to silty clay to clayey sand.

Excavation Characteristics

The earth materials encountered during the field investigation consist of soft to firm artificial fill and alluvium. Excavation through the earth materials was achieved by a full-size hollow-stem drill rig. Even though rippability of the encountered soils is considered normal, excavation difficulty may increase with depth due to increase in soil density.

GEOLOGIC HAZARDS

Seismic Hazards

The subject site is not within State-designated “Earthquake Zones of Required Investigation,” also known as “Alquist-Priolo Earthquake Fault Zones.” Therefore, no active faults are known to traverse the subject property. However, Southern California is a seismically active region with numerous faults capable of producing seismic waves and ground shaking on the subject site.

An active fault, as determined by the California Geological Survey (CGS), is a fault that has exhibited surface displacement within Holocene time (during the last 11,000 years). The active faults capable of causing ground shaking at the site are listed in the table below. The approximate locations of these faults are also shown on the USGS Fault Map in Appendix A.

Active fault/ Fault zone name	Location/Vicinity	Magnitude of latest recorded earthquake (M)	Date
Northridge	*San Fernando Valley*	*6.7*	*1994*
Hollywood	*West Hollywood*	*-*	*-*
Newport-Inglewood	*Newport Torrance Inglewood*	*6.3*	*1933*
San Andreas (southern segment)	*Southern California*	*7.9*	*1857*
Raymond	*Arcadia South Pasadena*	*4.9?*	*1988*
Malibu Coast	*Malibu*	*5.0*	*1989*
San Fernando	*Northeastern San Fernando Valley*	*6.5*	*1971*
San Gabriel (active segment)	*Newhall*	*-*	*-*
Sierra Madre	*Southern San Gabriel Mts.*	*5.8*	*1991*
Santa Monica	*Santa Monica*	*-*	*-*
Santa Susana	*Santa Susana Mts.*	*5.9?*	*1893*
Whittier-Elsinore	*Southern California*	*5.9*	*1987*

Properties and structures may be affected by different types of geologic hazards triggered by seismic events. These hazards include ground rupture, ground shaking, liquefaction, and landslides or rock falls in hillside areas. Properties along coast lines or on low-lying coastal areas are also in danger of inundation due to tsunamis caused by earthquakes or submarine landslides. However, strong ground shaking is often the most devastating seismic hazard affecting properties in Southern California. The areas that may be potentially affected by landslides and/or liquefaction are shown on State of California Seismic Hazards Zones Maps (see Appendix A).

The potential seismic hazards that may affect the site are briefly discussed below. It should be noted that the following is only an evaluation of risk and degree of potential structural damage due to possible future fault rupture on or near the property and does not indicate that an active fault may or may not be present under or near the subject site.

It is recommended that the proposed project be designed in accordance with the current seismic design parameters based on 2019 California Building Code (CBC) and ASCE 7-16 guidelines. The project structural engineer should verify the seismic design parameters prior to applying the data for the structural design. The recommendations presented in this report and the present building codes are intended to minimize structural damage and loss of life as a result of a seismic event and do not guarantee total earthquake damage prevention. Damage to patios, sidewalks, steps, and hardscape should be anticipated during moderate to strong seismic events as these are not generally covered by the current Building Code.

Ground Rupture

Ground rupture is caused by surficial displacement along a fault trace. Ruptures normally take place along previously existing fault traces. A fault is a discontinuity in the earth crust along which earth materials on one side have moved relative to those on the opposing side. No active faults are known to traverse the subject property.

Ground Shaking

The subject site will most likely experience ground shaking caused by seismic events during the lifespan of the proposed development. However, ground shaking is controlled by several factors including proximity to the ruptured fault, type of fault movement, depth of hypocenter, and the local and regional geology.

The seismic design parameters presented in this report were determined by the U.S. Seismic Design Maps program available on the USGS website. The calculated Maximum Considered Earthquake (MCE) ground motion is based on the site proximity to active and potentially active faults, the Site Class, and the Occupancy Category. A summary of the seismic design parameters is provided in Appendix C.

Liquefaction

Liquefaction is a seismic hazard that results in loss of strength in sandy soils below the water table by causing the soil to behave as a liquid rather than a solid for a short period of time. If liquefaction takes place in the bearing zone of a structure it may cause it to partially or totally settle, overturn, or collapse. When total collapse of the structure does not take place as a result of liquefaction, significant structural damage due to settlement or lateral spreading may be anticipated. The liquefaction phenomenon is normally limited to the upper 50 feet of non-cohesive soils that have not been densified, such as younger alluvial deposits. Dense soils with high plasticity and soils located above the groundwater table are generally not susceptible to liquefaction.

The Seismic Hazard Zone maps provided by the State of California depict the areas prone to liquefaction based on historical occurrences, groundwater conditions, and the nature of earth materials underlying the mapped areas. Based on the Seismic Hazard Maps, the subject property is located within the liquefaction hazard zone. However, it should be noted that the Seismic Hazard Maps may not be showing all the potential liquefaction hazard areas and site-specific liquefaction analysis may be conducted to identify liquefaction potential beneath the subject property.

A detailed liquefaction analysis has been performed on the subject property and results discussed in the "LIQUEFACTION ANALYSIS" section of this report.

Seismically Induced Settlement

Seismically induced settlement occurs when non-cohesive soils densify as result of ground shaking. Liquefaction-induced settlement may result in partial or total collapse of a structure, especially if there is significant differential settlement between adjacent structural elements. Typically, seismically induced settlement is greatest in loose cohesionless and poorly-graded sands.

The soils encountered at the subject site consist of firm clayey silt to silty clay to clayey sand. Based upon the liquefaction analysis, liquefaction induced settlement is estimated to be 2.4 inches and differential settlement of 1.6 inches.

Landslides

A landslide is a gravitationally induced mass movement of rock, debris, or earth down a slope.

Common types of landslides include rotational slumps, translational rockslides, rock or debris falls, debris flows, and lateral spreads. Slides are more common in hillside areas, however even areas with near flat topography may experience certain types of ground movement such as lateral spreads. Landslides are generally caused by several natural or man-made factors such as physical and/or chemical weathering of bedrock material, placement of inadequate or steep fill slopes, or adverse geologic structure. Common triggers of slides are earthquakes, undercutting or removal of lateral support at the toe of a slope, rainstorms, water leakage, and/or rise in groundwater elevation. In addition, hillside areas that have been cleared of vegetation are generally prone to surficial slope failures.

Due to the generally flat topography of the property, the risks of landslides or other geologic hazards associated with hillside areas are nil.

Earthquake Induced Landslides

Based on the State of California Seismic Hazard Maps, the subject property is not located within an earthquake-induced landslide hazard zone. However, it should be noted that the Seismic Hazard Maps may not identify all areas that have high potential for earthquake-induced landslides, strong ground shaking, or other seismic-related hazards. Due to the generally flat topography of the property, the risks of landslides or other geologic hazards associated with hillside areas are nil.

Lateral Spreads

Lateral spread is a type of slide where a cohesive rock or soil mass goes through simultaneous extension and subsidence into softer underlying material. Lateral spreads may be induced by earthquakes and liquefaction or flow of the softer underlying strata. Spreads could occur on gently sloping ground or even on horizontal ground incised by stream or river channels. Damage to structures could be severe even when less than one foot of permanent ground displacement occurs, especially when induced by seismic events. However, due to the complex nature of earthquake loading on the sliding mass and the post-liquefaction soil behavior, it is difficult to provide specific evaluation guidelines for lateral spreads. Based upon the liquefaction analysis, liquefaction induced settlement is estimated to be 2.4 inch and differential settlement of 1.6 inch.

Flood Hazard

The subject property is located within an area with “1% Annual Chance Flood Hazard Contained in Chanel.” However, based on the elevation of the building pad area relative to the nearby Los Angeles River channel, risk of storm-induced flooding appears to be low.

Seismic-induced flooding types include tsunamis, seiches, and reservoir failures. Tsunamis are ocean waves produced by sudden water displacement generally caused by offshore earthquakes or large submarine landslides. Properties along coast lines or on low-lying coastal areas are in danger of inundation due to tsunamis. However, due to the inland location of the subject property, the risk of inundation of the site from a tsunami is extremely low. Seiches are low-energy waves within lakes and reservoirs that are generally produced by strong earthquake shaking. The subject site is not located near a lake or a reservoir, therefore the potential for damage to the site from a seiche or a possible reservoir failure is nil.

Subsidence

Subsidence is the sinking or collapse of the ground surface caused by factors such as compaction of subsurface materials, hydro-consolidation, solution, erosion, liquefaction, lateral spreads, and extraction of subsurface liquids, solids, or gases. In California the main causes of ground subsidence are hydrocompaction of alluvial deposits and withdrawal of groundwater, oil, and gas.

Expansive Soils Hazard

Expansive soils expand (heave) when moisture is introduced and contract as they dry. As the water infiltrates the soil during heavy rainfalls or as a result of excessive landscape watering, some clay rich soils expand. When the moist expanded soil dries as a result of evaporation, it shrinks and its volume decreases. The soils expansion and contraction and change of volume can cause hardscape, on-grade concrete slabs, and foundations to crack. This movement can also result in misalignment of doors and windows. Expansive soils were encountered on the subject site and design for foundations, slabs on grade, and retaining walls have been provided in this report to mitigate this soil condition. However, these designs do not guarantee or warrant that cracking will not occur.

Methane Hazard

Methane gas accumulation which may result in explosions is a geologic hazard associated with gas and oil fields or landfills. Based on the Navigate LA interactive website, the subject property is not located within a methane/methane buffer zone.

LIQUEFACTION ANALYSIS

To quantify the potential for liquefaction at the subject site one deep boring was drilled to test the soils and collect samples. Site liquefaction analysis of the soils underlying the subject site was performed using the computer program LiquefyPro by CivilTech Software. LiquefyPro is software that evaluates liquefaction potential and calculates the settlement of soil deposits due to seismic loads. The program is based on the most recent publications of the NCEER Workshop and SP117 Implementation. The program requires in-situ test data of the soils, laboratory soils data, and earthquake design input.

For the PGA corresponding to two-thirds of the PGA_m, seismic-induced liquefaction settlements shall be determined. The predominant earthquake magnitude may be obtained from the USGS Interactive Deaggregation web site: https://geohazards.usgs.gov/deaggint/2008/. A 10% probability of exceedance in 50 years (475-year return period) may be used (either modal or mean values may be used). Potential seismic-induced settlements shall be determined when the safety factor is less than 1.1.

For the PGA corresponding to the PGA_M, seismic induced liquefaction settlements shall be determined. The predominant earthquake magnitude may be obtained from the USGS Interactive Deaggregation web site: https://geohazards.usgs.gov/deaggint/2008/. A 2% probability of exceedance in 50 years (2475-year return period) shall be used (either modal or mean values may be used). Potential seismic-induced settlements shall be determined when the safety factor is less than 1.0. Deformations of any foundations shall be such that the foundations of the buildings or other structures do not lose their ability to carry gravity loads and that collapse of the building or other structures is prevented.

The following earthquake input parameters and groundwater conditions were adopted for the analysis.

Earthquake Magnitude	Peak Horizontal Ground Acceleration	Groundwater Level During Testing	Groundwater Level During Earthquake
6.53 (10% probability of exceedance in 50 years)	0.933 (2/3*PGA_m)	29 feet	10 feet
7.52 (2% probability of exceedance in 50 years)	0.625 (PGA_m)	29 feet	10 feet

The results of the liquefaction analysis indicate a potential for liquefaction with the design earthquake input parameters. The following are the results of our liquefaction analysis:

PGA	Total Settlement (in)	Differential Settlement (in)
2/3*PGA_m	2.4	1.6
PGA_m	4.79	3.2

The liquefaction potential at the subject site is considered moderate to high. Therefore, mat-type foundation is considered appropriate for the proposed development.

For the seismic settlements associated with a higher ground motion due to earthquakes, the project structural engineer shall verify that the recommended foundation system of the proposed building will not lose ability to carry gravity loads and that collapse of the building or other structures is prevented.

CONCLUSIONS

1. Based on the results of this investigation and a thorough review of the proposed development, as discussed, the project is suitable for the intended use providing the following recommendations are incorporated into the design and subsequent construction of the project. Also, the development must be performed in an acceptable manner conforming to building code requirements of the controlling governing agency.

2. Based on the State of California Seismic Hazard Maps, the subject site is located within a liquefaction hazard zone. Based upon the liquefaction analysis, liquefaction induced settlement is estimated to be 2.4 inch and differential settlement of 1.6 inch.

3. Based on the State of California Seismic Hazard Maps, the subject site is not located within an earthquake-induced landslide hazard zone.

4. The subject site is located within a liquefaction hazard zone; therefore, the subject site is NOT suitable for storm water infiltration.

5. The SITE CLASS based on California Building Code is D.

6. Based upon field observations, laboratory testing and analysis, the alluvium found in the explorations has sufficient strength to support the proposed structure.

RECOMMENDATIONS

<u>Specific</u>

1. The proposed multi-story building should be supported on foundations embedded into competent alluvium.
2. The property owner shall maintain the site as outlined in the Drainage and Maintenance Section.

Drainage and Maintenance

Maintenance of properties must be performed to minimize the chance of serious damage and/or instability to improvements. Most problems are associated with or triggered by water. Therefore, a comprehensive drainage system should be designed and incorporated into the final plans. In addition, pad areas should be maintained and planted in a way that will allow this drainage system to function as intended. The property owner shall be fully responsible for dampness or water accumulation caused by alteration in grading, irrigation or installation of improper drainage system, and failure to maintain drain systems. The following are specific drainage, maintenance, and landscaping recommendations. Reductions in these recommendations will reduce their effectiveness and may lead to damage and/or instability to the improvements. It is the responsibility of the property owner to ensure that improvements, structures and drainage devices are maintained in accordance with the following recommendations and the requirements of all applicable government agencies.

Drainage

Positive pad drainage should be incorporated into the final plans. The pad should slope away from the footings at a minimum five percent slope for a horizontal distance of ten feet. In areas where there is insufficient space for the recommended ten-foot horizontal distance concrete or other impermeable surface should be provided for a minimum of three feet adjacent the structure. Pad drainage should be at a minimum of two percent slope where water flow over lawn or other planted areas. Drainage swales should be provided with area drains about every fifteen feet. Area drains should be provided in the rear and side yards to collect drainage. All drainage from the pad should be directed so that water does not pond adjacent to the foundations or flow toward them. Roof gutters and downspouts are required for the proposed structures and should be connected into a buried area drain system. All drainage from the site should be collected and directed via non-erosive devices to a location approved by the building official. Area drains, subdrains, weep holes, roof gutters and downspouts should be inspected periodically to ensure that they are not clogged with debris or damaged. If they are clogged or damaged, they should be cleaned out or repaired.

Landscaping (Planting)

The property owner is advised not to develop planter areas between patios, sidewalk and structures. Planters placed immediately adjacent to the structures are not recommended. If planters are proposed immediately adjacent to structures, impervious above-grade or below-grade planter boxes with solid bottoms and drainage pipes away from the structure are suggested. All slopes should be maintained with a dense growth of plants, ground-covering vegetation, shrubs and trees that possess dense, deep root structures and require a minimum of irrigation. Plants surrounding the development should be of a variety that requires a minimum of watering. It is recommended that a landscape architect be consulted regarding planting adjacent to improvements. It will be the responsibility of the property owner to maintain the planting. Alterations of planting schemes should be reviewed by the landscape architect.

Irrigation

An adequate irrigation system is required to sustain landscaping. Over-watering resulting in runoff and/or ground saturation must be avoided. Irrigation systems must be adjusted to account for natural rainfall conditions. Any leaks or defective sprinklers must be repaired immediately. To mitigate erosion and saturation, automatic sprinkling systems must be adjusted for rainy seasons. A landscape architect should be consulted to determine the best times for landscape watering and the proper usage.

Pools/Plumbing

Leakage from a swimming pool or plumbing can produce a perched groundwater condition that may cause instability or damage to improvements. Therefore, all plumbing should be leak-free.

Grading and Earthwork

Proposed grading will consist of basement and foundation excavations.

Excavations

Excavations ranging in vertical height up to 12 feet will be required for the proposed basement. Conventional excavation equipment may be used to make these excavations. Excavations should expose fill and alluvium. These soils are suitable for vertical excavations up to 4 feet, cuts above 4 feet shall be trimmed back at 1:1 (H:V) slope or shored. This should be verified by the project geotechnical engineer during construction so that modifications can be made if variations in the soil occur.

All excavations should be stabilized within 30 days of initial excavation. If this time is exceeded, the project geotechnical engineer must be notified, and modifications, such as shoring or slope trimming may be required. Water should not be allowed to pond on top of the excavation, nor to flow toward it. All excavations should be protected from inclement weather. This is required to keep the surface of the open excavation from becoming saturated during rainfall. Saturation of the excavation may result in a relaxation of the soils which may result in failures. Excavations should be kept moist, not saturated, to reduce the potential for raveling and sloughing during construction. No vehicular surcharge should be allowed within three feet of the top of cut.

Temporary Shoring

The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that a review of the final shoring plans and specifications be made by this office prior to bidding or negotiating with a shoring contractor be made.

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tie-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than two diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation, may be assumed to be 220 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.2 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using the attached skin-friction graph. The minimum depth of embedment for shoring piles is five feet below the bottom of the footing excavation, or seven feet below the bottom of excavated plane, whichever is deeper.

Casing may be required should caving be experienced in the saturated earth materials. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than five feet.

Groundwater was encountered during exploration at a depth of 29 feet below grade. Therefore, it is anticipated that the proposed piles in excess of 29 feet in depth will encounter water. Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than ten inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed, and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to ensure that the tip of the tremie tube is never raised above the surface of the concrete.

A special concrete mix should be used for concrete to be placed below water. The design shall provide for concrete with a strength of 1,000 psi over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. The slump shall be commensurate to any research report for the admixture, provided that it shall also be the minimum for a reasonable consistency for placing when water is present.

Lagging

Due to the low cohesion nature of the soil and fill materials, it is anticipated that lagging will be required for the soil and fill. To develop the full lateral support, provisions should be implemented to assure firm contact between the lagging and the undisturbed earth materials. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the lagging to the earth materials. It is recommended that the lagging and slurry backfill be installed the same day as excavation.

If the clear spacing between soldier piles does not exceed four feet, lagging between soldier piles could possibly be omitted within the bedrock. It is recommended that the exposed earth materials be observed by the soils engineer to verify the cohesive nature of the soils and the area where lagging may be omitted.

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the earth materials, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but may be limited to a maximum of 400 pounds per square foot.

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of cantilevered shoring system. A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. The design of trapezoidal distribution of pressure is shown in a diagram in the “Retaining Wall” section of this report. Equivalent fluid pressures for the design of cantilevered and restrained shoring are presented in the following table:

Height of Shoring (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
12 feet	45 pcf	30H psf

*Where H is the height of the shoring in feet.

Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressures should be applied where the shoring will be surcharged by adjacent traffic or structures.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of one-half inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent streets and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

Monitoring

Because of the depth of the excavation, some mean of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of this office. Many building officials require that shoring installation should be performed during the continuous observations of the geotechnical engineer. The observations are made so that modifications of the recommendations can be made if variations in the earth material or groundwater conditions occur. Also, the observations will allow for a report to be prepared on the installation of shoring for the use of the local building official.

Excavations Maintenance – Erosion Control

The following recommendations should be considered a part of the excavation/erosion control plan for the subject site and are intended to supplement, but not supersede nor limit the erosion control plans produced by the Project Civil Engineer and/or Qualified SWPPP Developer. These recommendations should be implemented during periods required by the Building Code (typically between the months of October and April) or at any time of the year prior to a predicted rain event. Consideration should also be given to potential local sources of water/runoff such as existing drainage pipes or irrigation systems that remain in operation during construction activities.

Open Excavations:

All open excavations shall be protected from inclement weather, including areas above and at the toe of the excavation. This is required to keep the excavations from becoming saturated. Saturation of the excavation may result in a relaxation of the soils which may result in failures. Water/runoff should be diverted away from the excavation and not be allowed to flow over the excavation in a concentrated manner.

Hillside Excavations:

All hillside excavations shall be protected during inclement weather and should extend beyond the edges of the excavations in all directions. Plastic sheeting along with stakes, ropes and sandbags may be used to provide protection of the excavations. Water/runoff should be diverted away from the excavation and not be allowed to flow over the excavation.

The project Civil Engineer should provide a plan depicting the required limits of erosion control. Slopes around an open excavation should be trimmed to slope away from the open excavation so that water/runoff will not drain into the excavation. Any trees or planters that might cause failures around an open excavation shall be anchored safely. After the inclement weather has ceased, the excavations shall be reviewed by the project geotechnical engineer and geologist for safety prior to recommencement of work.

Open Trenches/Foundation Excavations:

No water should be allowed to pond adjacent to or flow into open trenches. All open trenches shall be covered with plastic sheeting that is anchored with sandbags. Areas around the trenches should be sloped away from the trenches to prevent water runoff from flowing into or ponding adjacent to the trenches.

After the inclement weather has ceased, the excavations shall be reviewed by the project geotechnical engineer and geologist for safety prior to recommencement of work. Foundation excavations that remain open during inclement weather shall be reviewed by the project geotechnical engineer and geologist prior to the placement of steel and concrete to ensure that proper embedment and contact with the bearing material have been maintained.

Open Pile/Caisson Excavations:

All pile/caisson excavations should be reviewed and poured prior to the onset of inclement weather. It is not recommended that any pile/caisson excavations remain open through any inclement weather. However, if it is necessary to leave pile/caisson excavations open during inclement weather, all water and runoff shall be diverted away from and prevented from entering the pile/caisson excavations. Pile/caisson excavations that remain open during inclement weather shall be reviewed by the project geotechnical engineer and geologist prior to the placement of steel and concrete to ensure that proper embedment has been maintained. The base of all end-bearing caissons shall be re-cleaned to ensure contact with the proper bearing material. All stockpiled cuttings from the pile borings shall be removed.

Grading In Progress:

During the inclement time of the year, or during periods prior to the onset of rain, all fill that has been spread and is awaiting compaction shall be compacted before stopping work for the day or before stopping work because of inclement weather. These fills, once compacted, shall have the surface sloped to drain to one area where water may be removed.

Additionally, it is suggested that all stock-piled fill materials be covered with plastic sheeting. This action will reduce the potential for the moisture content of the fill from becoming too high for compaction. If the fill stockpile is not covered during inclement weather, then aerating the fill to reduce the moisture content would be required. This action is generally very time consuming and may result in construction delays.

Work may recommence, after the rain event, once the site has been reviewed by the project geotechnical engineer.

Foundations

The proposed structure may be supported on mat foundation system embedded into the alluvium. The foundation/mat should be designed with a minimum foundation/mat thickness of (12) inches unless superseded by the project structural engineer. Rigid and flexible mat foundation design values are presented below:

Conventional rigid method:

The mat foundation may be proportioned using an average bearing value of (1500) pounds per square foot, and the maximum allowable bearing capacity should not exceed (3000) pounds per square foot. The mat foundation structural design should be done by the project structural engineer.

Approximate flexible method:

An estimated coefficient of subgrade reaction, K_1, of (60) pounds per cubic inch (pci) may be used in the design of the mat. The maximum allowable bearing capacity should not exceed (3000) pounds per square foot. This value is a unit value for use with a one-foot square footing.

For sandy soil, the coefficient of subgrade reaction should be reduced in accordance with the following equation when used with larger square mat foundation, B(ft) x B(ft):

$$K\ (BxB) = K_1 \left(\frac{B+1}{2B}\right)^2$$

K_1: The coefficient of subgrade reaction of foundation for 1 (ft) X 1 (ft)
K (BxB): The coefficient of subgrade reaction of foundation for B (ft) X B (ft)
B: Width of mat foundation (ft)

The coefficient of subgrade reaction should be reduced in accordance with the following equation when used with larger rectangular mat foundation, B(ft) x L(ft):

$$K\ (BxL) = K\ (BxB) * \left(\frac{1 + 0.5 * \frac{B}{L}}{1.5}\right)$$

K_1: The coefficient of subgrade reaction of foundation for 1 (ft) X 1 (ft)
K (BxL): The coefficient of subgrade reaction of foundation for B (ft) X B (ft)
B: Width of mat foundation (ft)
L: Length of mat foundation (ft)

The bearing values given above are net bearing values; the weight of concrete below grade may be neglected. These bearing values may be increased by one-third (1/3) for temporary loads, such as, wind and seismic forces.

All footing excavation depths will be measured from the lowest adjacent grade of recommended bearing material. Footing depths will not be measured from any proposed elevations or grades. Any foundation excavations that are not the recommended depth <u>into</u> the recommended bearing materials will not be acceptable to this office.

Lateral loads may be resisted by friction at the base of the conventional foundations and by passive resistance within the alluvium. A coefficient of friction of (0.2) may be used between the foundations and the alluvium. The passive resistance may be assumed to act as a fluid with a density of (220) pounds per cubic foot, with a maximum earth pressure of (2200) pounds per square foot. When combining passive and friction for resistance of lateral loads, the passive component should be reduced by one-third. All piles shall be considered fixed 5 feet into alluvium. Piles may be considered isolated if the distance between piles is greater that (3.0) time the pile diameter. For isolated poles, the allowable passive earth pressure may be doubled.

It is recommended that a vapor retarder/waterproofing be placed below the concrete slab on grade. Vapor/moisture transmission through slabs does occur and can impact various components of the structure. Vapor retarder/waterproofing design and inspection of installation is not the responsibility of the geotechnical engineer (most often the responsibility of the architect). Our firm does not practice in the field of water and moisture vapor transmission evaluation/mitigation. Therefore, we recommend that a qualified person/firm be engaged/consulted to evaluate the general and specific water and moisture vapor transmission paths and any impact on the proposed development. This person/firm should provide recommendations for mitigation of potential adverse impact of water and moisture vapor transmission on various components of the structure as deemed necessary. The actual waterproofing design shall be provided by the architect, structural engineer or contractor with experience in waterproofing

In order to promote good building practices and alert the rest of the design/construction team of the appropriate standards and expert recommendations pertaining to vapor barriers/retarders, engineers (especially those aware of the issues surrounding below-slab moisture protection and its effects on the success of their projects) should consider recommending and citing specific performance characteristics. The following paragraph includes criteria from the latest standards and expert recommendations and should be considered for use in your firm's own recommendations: *Vapor barrier shall consist of a minimum 15 mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditions (ASTM E 1745 Section 7.1 and Sub-Paragraph 7.1.1-7.1.5): less than 0.01 perms [grains/(ft^2-hr-inHg)] and comply with the ASTM E 1745 Class A requirements. Install vapor barrier according to ASTM E1643, including proper perimeter seal. Basis of design: Stego Wrap Vapor Barrier 15 mil and Stego Crete Claw Tape (perimeter seal tape). Approved Alternatives: Vaporguard by Reef Industries, Sundance 15 mil Vapor Barrier by Sundance Inc.*

Settlement

Settlement of the proposed mat foundation will occur. Based on the anticipated loading condition, settlement on the order of (2) inches under the heavily-loaded center of the proposed mat foundation should be anticipated, and settlement on the order of (1) inch under the edge of the proposed foundation should be anticipated.

Slabs on Grade

Slabs on grade should be reinforced with minimum #4 reinforcing bars, placed at 12 inches on center each way and supported on alluvium. Provisions for cracks should be incorporated into the design and construction of the foundation system, slabs, and proposed floor coverings. Concrete slabs should have sufficient control joints spaced at a maximum of approximately eight feet. These recommendations are considered minimums unless superseded by the project structural engineer. Prior to placing the vapor retarder/waterproofing the moisture content of the subgrade should be raised to 120 percent of the optimum moisture content to a depth of 18 inches.

It is recommended that a vapor retarder/waterproofing be placed below the concrete slab on grade. Vapor/moisture transmission through slabs does occur and can impact various components of the structure.

Vapor retarder/waterproofing design and inspection of installation is not the responsibility of the geotechnical engineer (most often the responsibility of the architect). Geolotech, Inc. does not practice in the field of water and moisture vapor transmission evaluation/mitigation. Therefore, we recommend that a qualified person/firm be engaged/consulted to evaluate the general and specific water and moisture vapor transmission paths and any impact on the proposed development. This person/firm should provide recommendations for mitigation of potential adverse impact of water and moisture vapor transmission on various components of the structure as deemed necessary. The actual waterproofing design shall be provided by the architect, structural engineer or contractor with experience in waterproofing

In order to promote good building practices and alert the rest of the design/construction team of some of the appropriate standards and expert recommendations pertaining to vapor barriers/retarders, the waterproofing designer should consider recommending and citing specific performance characteristics. The following paragraph includes some of the standards and expert recommendations and should be considered for use waterproofing designer own recommendations:

Vapor barrier shall consist of a minimum 15 mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditions (ASTM E 1745 Section 7.1 and Sub-Paragraph 7.1.1-7.1.5): less than 0.01 perms [grains/(ft^2-hr-inHg)] and comply with the ASTM E 1745 Class A requirements. Install vapor barrier according to ASTM E1643, including proper perimeter seal. Basis of design: Stego Wrap Vapor Barrier 15 *mil and Stego Crete Claw Tape (perimeter seal tape). Approved Alternatives: Vaporguard by Reef Industries, Sundance 15 mil Vapor Barrier by Sundance Inc.*

Decking

Exterior decking slabs on grade should be reinforced with minimum #4 reinforcing bars, placed at 12 inches on center each way and supported on alluvium. Provisions for cracks should be incorporated into the design and construction of the decking. Concrete slabs should have sufficient control joints spaced at a maximum of approximately 8 feet. Decking planned adjacent to lawns, planters or adjacent to descending slopes should be provided with a 12-inch thickened edge. The deck reinforcement should be bent down into the edge. These recommendations are considered minimums unless superseded by the project structural engineer. Prior to placing the concrete, the subgrade should be raised to 120 percent of the optimum moisture content to a depth of 18 inches.

Expansive Soils

Expansive soils were encountered on the subject property. Expansive soils can be a problem, as variation in moisture content will cause a volume change in the soil.

To reduce the effect of expansive soils, foundation systems are usually deepened and/or provided with additional reinforcement design by the structural engineer. Planning of yard improvements should take into consideration maintaining uniform moisture conditions around structures. Soils should be kept moist, but water should not be allowed to pond. These designs are intended to reduce deflection and cracking; however, they will not eliminate deflection and cracking and do not guarantee that cracking will not occur.

Retaining Walls

Cantilever retaining walls should be designed to resist an active earth pressure such as that exerted by compacted backfill. Retaining walls up to 12 feet in height may be designed per the following table. The ‘active’ pressure assumes that the wall will be allowed to deflect 0.01H to 0.02H. Basement walls and other walls where horizontal movement is restricted at the top or not allowed to deflect shall be designed for at-rest pressure.

Surface Slope of Retained Material Horizontal to Vertical	Active Equivalent Fluid Weight p.c.f.	At-Rest Pressure Fluid Weight p.c.f.
Level	75	95

The entire wall should be designed for full hydrostatic pressure based on a water level at the ground surface. In addition, floors would need to be designed for hydrostatic uplift and waterproofed.

In addition to lateral earth pressure, these retaining walls should be designed to resist the surcharge imposed by the proposed structures, footings, any adjacent buildings, or by adjacent traffic surcharge, per the attached figures 11 and 12 obtained from the Naval Facilities Engineering Command, Design Manual 7.02 (Foundation and Earth Structures, pages 74 and 75).

Lateral Earth Pressure Due to Earth Motion

Retaining walls should be designed to resist an active earth pressure due to earth motion, if required by the building official, distributed as a triangle pressure. Retaining walls up to 12 feet in height may be designed per the following table. The seismic equivalent fluid pressure is in addition to static earth pressures.

The seismic loading is based on a horizontal acceleration coefficient of ½ of 2/3 PGA_M = 0.31.

Surface Slope of Retained Material Horizontal to Vertical	Seismically Induced Earth Pressure - Equivalent Fluid Weight p.c.f.
Level	21

REVIEWS

Plan Review and Plan Notes

The final grading, building, and/or structural plans shall be reviewed and approved by the consultants to ensure that all recommendations are incorporated into the design or shown as notes on the plan.

The final plans should reflect the following:

1. The Geotechnical Engineering Investigation by Geolotech, Inc. is a part of the plans.

2. Plans must be reviewed and signed by Geolotech, Inc.

3. The project geotechnical engineer and/or geologist must review all grading.

4. The project geotechnical engineer and/or geologist shall review all foundations.

Construction Review

During construction, Geolotech, Inc. should review and verify all geotechnical and geological work in progress. This office should be notified at least two working days in advance of any field reviews so that staff personnel may be made available. Foundation reviews should be performed prior to the placement of forms and steel. The following site reviews are recommended or required. Should the observations reveal any unforeseen hazards; the project geotechnical engineer will recommend mitigation methods.

- Pre-construction meeting ……………………………………...……Recommended
- Temporary excavations ………………………………………………….…Required
- Shoring pile and lagging placement ………………………………………Required
- Keyway excavations and benching ……………………………………….Required
- Bottom excavations for removals for footings, slabs, and decking…..... Required
- Compaction of primary and secondary fill ………………………………..Required
- Foundation excavation review for main structures ……………………...Required
- Foundation excavation review for retaining wall ………………………...Required
- Slab subgrade moisture barrier membrane ……………...…….Recommended
- Excavation review for pool and spa …………………………………..…..Required
- Subdrain and rock placement behind retaining walls ……………..…… Required
- Compaction of retaining wall backfill …………………………………..……. Required
- Compaction of utility trench backfill ………………………………. Recommended

The property owner should take an active role in project safety by assigning responsibility and authority to individuals qualified in appropriate construction safety principles and practices. Generally, site safety should be assigned to the general contractor or construction manager that is in control of the site and has the required expertise, which includes but not limited to construction means, methods and safety precautions. When excavations exist on a site, the area should be fenced, and warning signs posted. All pile excavations must be properly covered and secured. Soil generated from excavations and cuts should not be spilled over descending slopes or piled against fences.

LIMITATIONS

General

This report is intended to be used only in its entirety. No portion or section of the report, by itself, is designed to completely represent any aspect of the project described herein. Geolotech, Inc. should be contacted if additional information or clarification is needed regarding this report.

This report was prepared based on the preliminary development plan or concept. In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by us and the conclusions and recommendations are modified or reaffirmed after such review.

Subsurface conditions were interpreted based on our field explorations and professional experience. However, between exploratory excavations, subsurface earth materials may vary in type, strength and many other properties from those interpreted. The findings, conclusions and recommendations presented herein are for the soil conditions encountered in the specific locations. Earth materials and conditions immediately adjacent to, or beneath those observed, may have different characteristics, such as earth type, physical properties and strength. Other soil conditions due to non-uniformity of the soil conditions or manmade alterations may be revealed during construction. If subsurface conditions differ from those encountered in the described exploration, this office should be advised immediately so that further recommendations may be made if required. If it is desired to minimize the possibility of such changes, additional explorations and testing can/should be performed. Geolotech, Inc. should be consulted to determine if additional work is required when our work is used by others or if the scope of the project has changed. If the project is delayed for more than one year, this office should be contacted to verify the current site conditions and to prepare an update report.

Findings, conclusions and recommendations presented herein are based on experience and background. Therefore, findings, conclusions and recommendations are professional opinions and are not meant to indicate a control of nature.

Fluctuations in groundwater level may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be hazardous to health and property and saturation of earth materials can cause subsidence or slippage of the site.

Expansive soils were encountered on the subject property. Design for foundations, slabs on grade, and retaining walls have been provided to mitigate this soil condition. These designs do not guarantee or warrant that cracking will not occur.

This preliminary report provides information regarding the findings on the subject property. It is not designed to provide a guarantee that the site will be free of hazards in the future, such as but not limited to, landslides, slippage, liquefaction, expansive soils, differential settlement, debris flows, seepage, concentrated drainage or flooding. It may not be possible to eliminate all hazards, but homeowners must maintain their property and improve deficiencies to minimize these hazards.

This report is issued and made for the sole use and benefit of the client, is not transferable, and is as of the exploration date. Any liability in connection herewith shall not exceed the fee for the exploration. No warranty, expressed or implied, is made for intended in connection with the above exploration or by the furnishing of this report or by any other oral or written statement.

This report may not be copied. If you wish to purchase additional copies, you may order them from this office.

CONSTRUCTION NOTICE

Construction can be challenging. Geolotech, Inc. has provided this report to advise you of the general site conditions, geotechnical feasibility of the proposed project, and overall site stability. It must be understood that the professional opinions provided herein are based upon subsurface data, laboratory testing, analyses, and interpretation thereof. Recommendations contained herein are based upon surface reconnaissance and minimum subsurface explorations deemed suitable by your consultants.

Although quantities for foundation concrete and steel may be estimated based on the findings provided in this report, provision should be made for possible changes in quantities during construction. If it is desired to minimize the possibility of such changes, additional exploration and testing should be considered. However, you must be aware that depths and magnitudes will most likely vary between explorations given in the report.

We appreciate the opportunity of serving you on this project. If you have any questions concerning this report, please contact the undersigned.

Respectfully submitted,
GEOLOTECH, Inc.

AMIR MIRZADEH
Principal Engineer
RCE 58119

REGISTERED PROFESSIONAL ENGINEER
AMIR MIRZADEH
No. C058119
CIVIL
STATE OF CALIFORNIA

CERTIFIED ENGINEERING GEOLOGIST
PEDRAM RAHIMIKIAN
No. 2683
STATE OF CALIFORNIA

PEDRAM RAHIMIKIAN
Principal Geologist
CEG 2683

Distribution: (3) Addressee

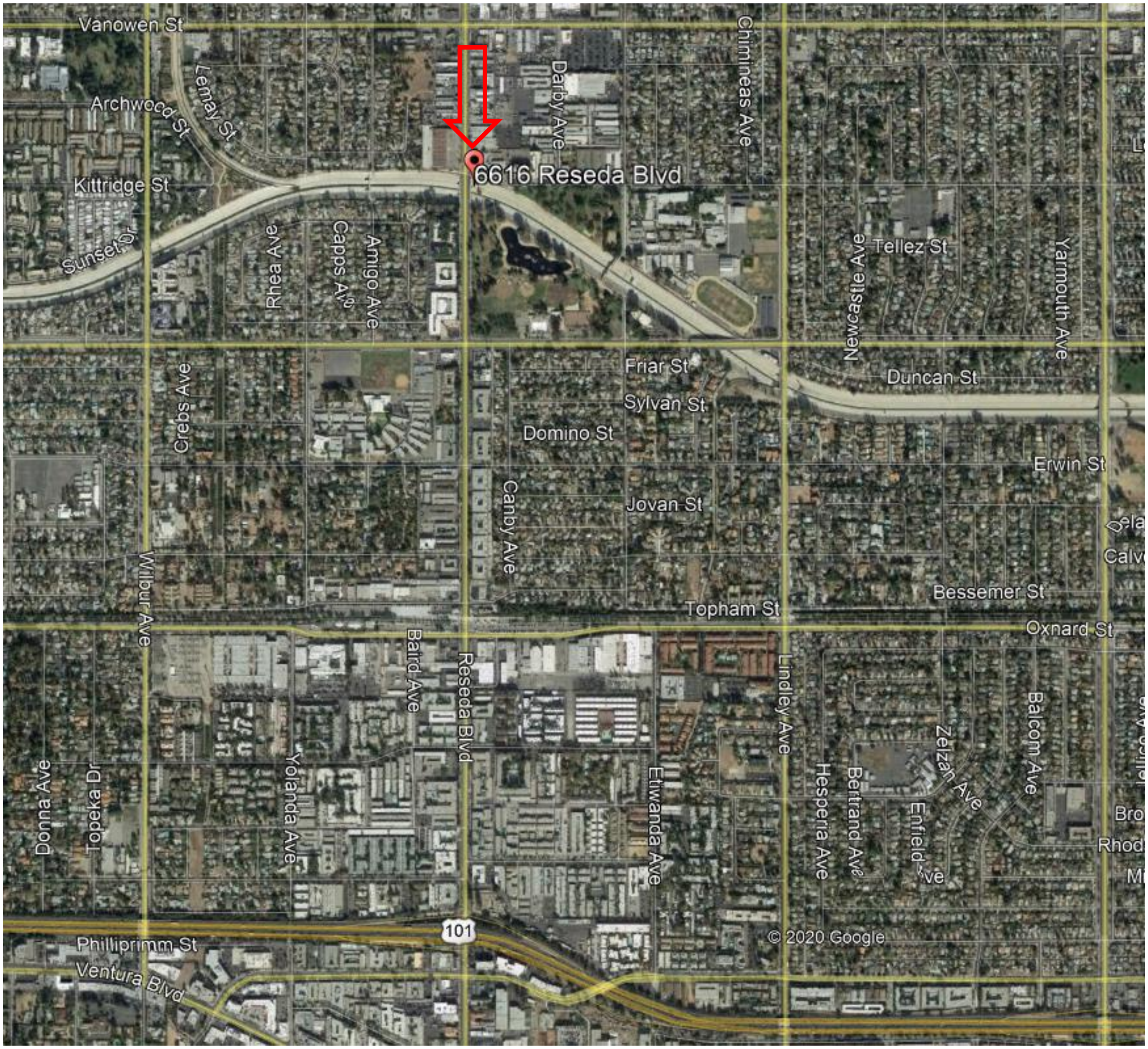
APPENDIX A

SITE INFORMATION

VICINITY MAP
REGIONAL GEOLOGIC MAP
REGIONAL GROUNDWATER MAP
USGS FAULT MAP
SEISMIC HAZARD MAP

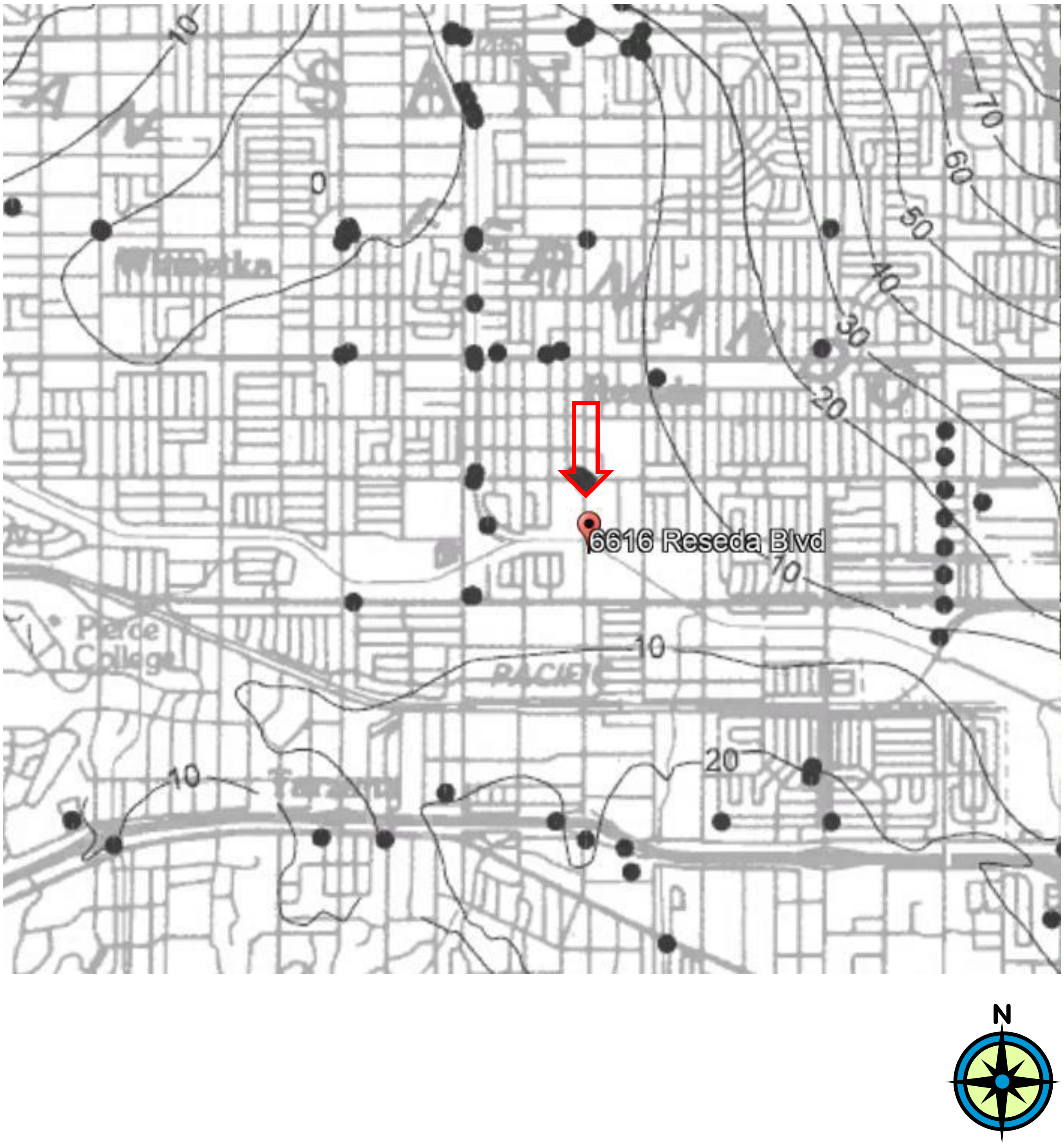
GEOTECHNICAL MAP
CROSS-SECTION

FIELD EXPLORATION
BORING LOGS B-1 & B-2



VICINITY MAP

Google Earth	Scale 1" = ~1300'



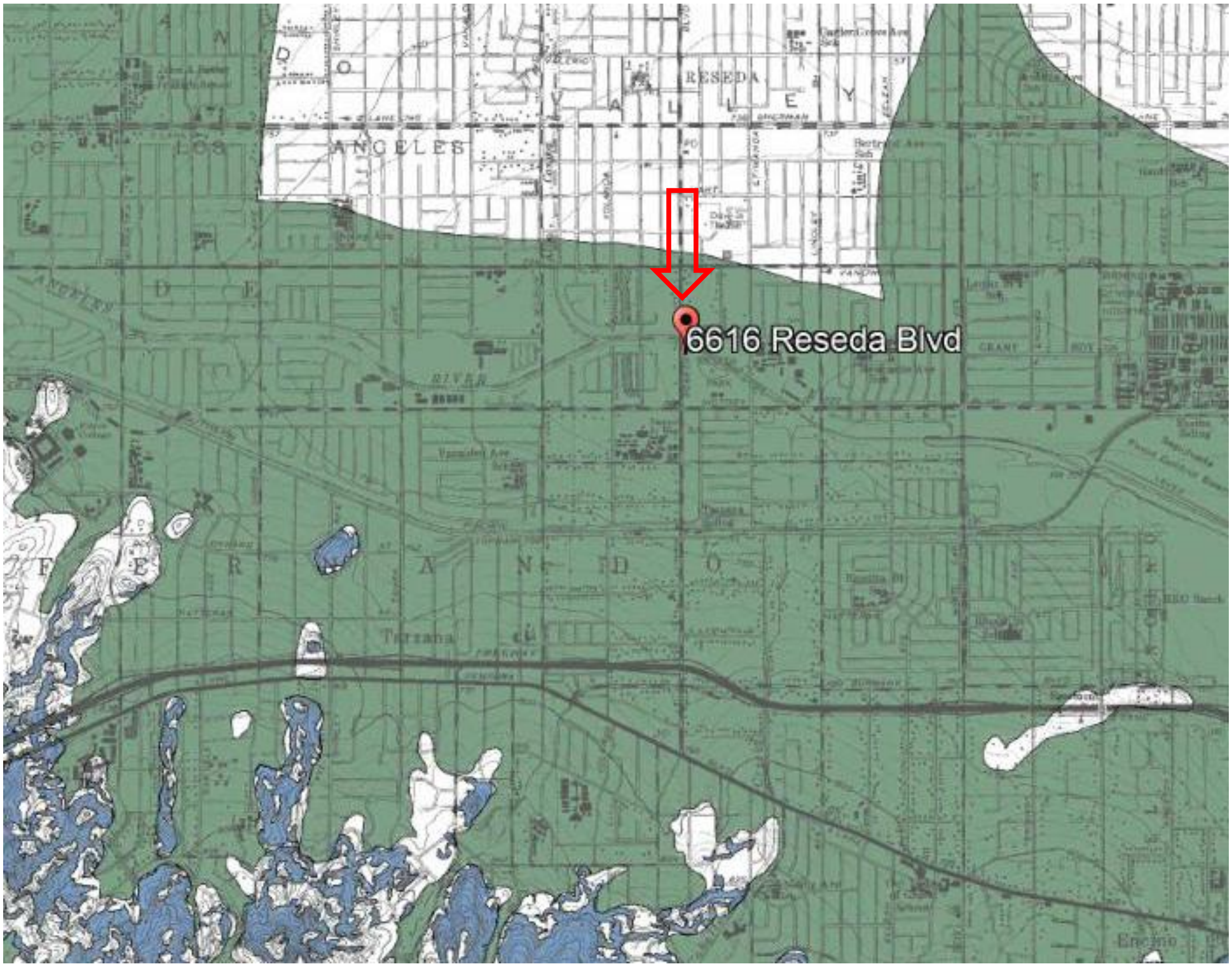
REGIONAL GROUNDWATER MAP

California State Seismic Hazard Report 007	Scale 1" = ~3300'

1 Alamo thrust
2 Arrowhead fault
3 Bailey fault
4 Big Mountain fault
5 Big Pine fault
6 Blake Ranch fault
7 Cabrillo fault
8 Chatsworth fault
9 Chino fault
10 Clamshell-Sawpit fault
11 Clearwater fault
12 Cleghorn fault
13 Crafton Hills fault zone
14 Cucamonga fault zone
15 Dry Creek
16 Eagle Rock fault
17 El Modeno
18 Frazier Mountain thrust
19 Garlock fault zone
20 Grass Valley fault
21 Helendale fault
22 Hollywood fault
23 Holser fault
24 Lion Canyon fault
25 Llano fault
26 Los Alamitos fault
27 Malibu Coast fault
28 Mint Canyon fault
29 Mirage Valley fault zone
30 Mission Hills fault
31 Newport Inglewood fault zone
32 North Frontal fault zone
33 Northridge Hills fault
34 Oak Ridge fault
35 Palos Verdes fault zone
36 Pelona fault
37 Peralta Hills fault
38 Pine Mountain fault
39 Raymond fault
40 Red Hill (Etiwanda Ave) fault
41 Redondo Canyon fault
42 San Andreas Fault
43 San Antonio fault
44 San Cayetano fault
45 San Fernando fault zone
46 San Gabriel fault zone
47 San Jacinto fault
48 San Jose fault
49 Santa Cruz-Santa Catalina Ridge f.z.
50 Santa Monica fault
51 Santa Ynez fault
52 Santa Susana fault zone
53 Sierra Madre fault zone
54 Simi fault
55 Soledad Canyon fault
56 Stoddard Canyon fault
57 Tunnel Ridge fault
58 Verdugo fault
59 Waterman Canyon fault
60 Whittier fault

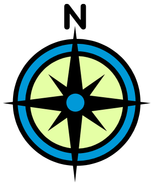
USGS FAULT MAP

Lisa Wald - USGS: active faults (red) and potentially active faults (green)	Scale As Shown



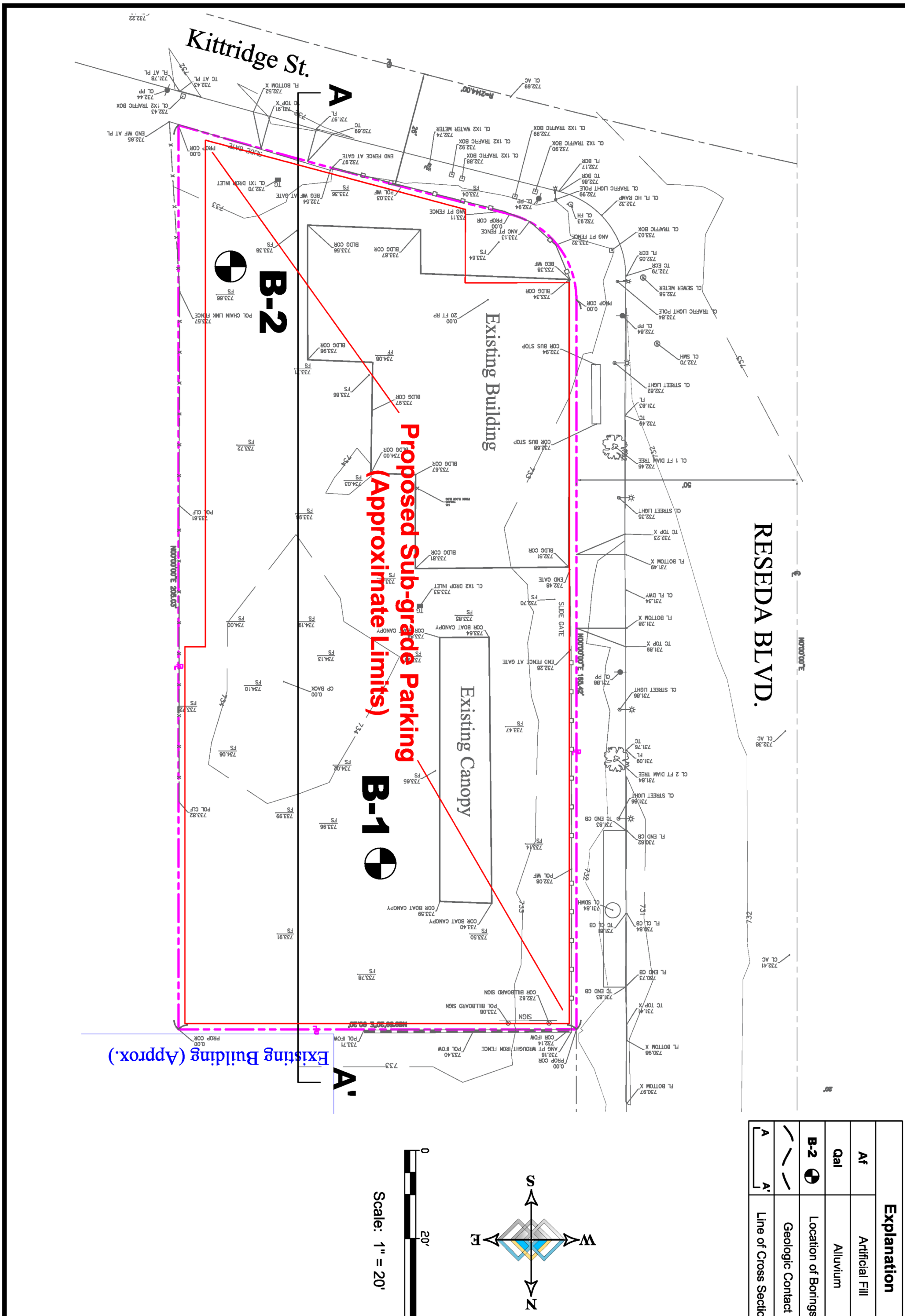
BLUE = LANDSLIDE HAZARD ZONES

GREEN = LIQUEFACTION HAZARD ZONES



SEISMIC HAZARD MAP

California State Seismic Hazard Map - Canoga Quadrangle	Scale 1" = ~3300'



GEOLOTECH INC
GEOLOGICAL & GEOTECHNICAL ENGINEERING
(818) 383-5081 | info@geolotech.com | www.geolotech.com

Description: **Geotechnical Map**	Project Address: 6616 Reseda Blvd. Reseda, California	Date: March 2020	
Survey Provided By: De Land Services		Job No. 20008-1	

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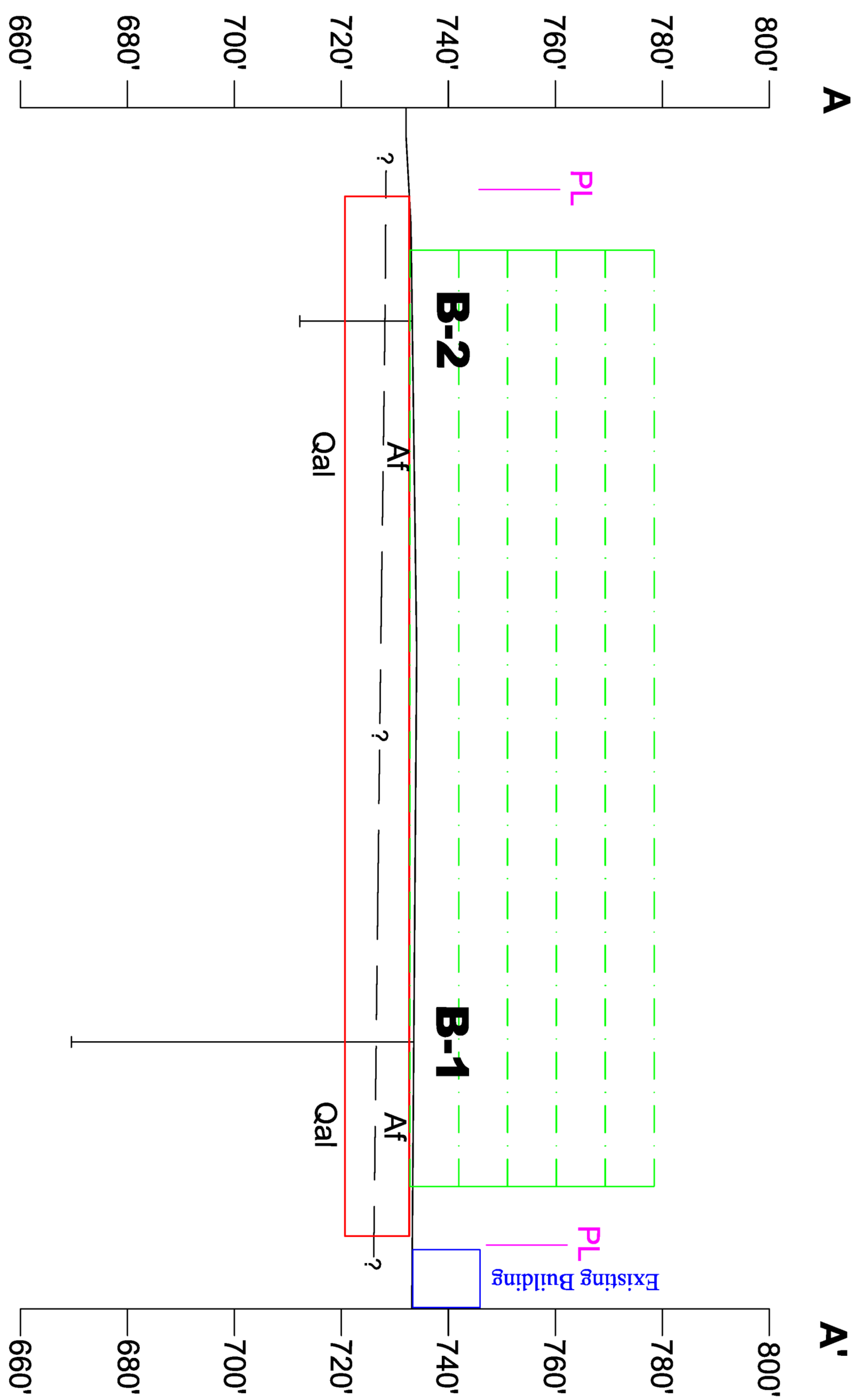

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Description:	Project Address:	Date:	Job No.
Geotechnical Cross Section A-A'	6616 Reseda Blvd. Reseda, California	March 2020	20008-1

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GEOLOTECH, Inc. Project #: 20008 Address: 6616 Reseda Blvd						Drilling Date: March 5, 2020 Drilling Method: Hollow-Stem Boring Diameter: 8”	**Boring B-1**
Water Content (%)	Dry Density (pcf)	Blow Counts per 12”	Sample	Depth (ft.)	Graphic Log	**DESCRIPTIONS**	
				0		0 – 7.5’ **ARTIFICIAL FILL (Af)**, silty clay (CL), very dark yellowish brown, moist, soft to firm, abundant asphalt debris	
				2.5			
29	72	10	MC	**5**			
				7.5		7.5’ – 64.0’ **ALLUVIUM (Qal)**	
		10	SPT	**10**		Clayey silt (ML), dark yellowish brown, moist	
		11	SPT	12.5			
23	102	20	MC	**15**			
		12	SPT	17.5			
		12	SPT	**20**		Silty clay (CL), very dark yellowish brown, moist, hydrocarbon odor	
		14	SPT	22.5			
21	105	18	MC	**25**			
		12	SPT	27.5			
		20	SPT	**30**		Silty clay (CH), light yellowish brown, very moist to wet	
		14	SPT	32.5			
23	102	18	MC	**35**			
		16	SPT	37.5			
		22	SPT	**40**			
		27	SPT	42.5		Grayish mottling	
		47	SPT	**45**		Sand (SW), orange brown, fine grained, wet	
15	119	45	MC	47.5		Sand (SW), yellowish brown to medium gray, fine to coarse	
		76	SPT	**50**		Medium to coarse grained	
		41	SPT	52.5		Clayey sand (SC), yellowish brown, fine to medium grained	
23	103	60	MC	**55**			
		44	SPT	57.5			
		46	SPT	**60**			
		50	SPT	62.5			
140 lb. Auto Hammer				Logger: PRK		TOTAL DEPTH: 64 FEET GROUNDWATER AT: 29 FEET	

GEOLOTECH, Inc. Project #: 20008 Address: 6616 Reseda Blvd						Drilling Date: March 5, 2020 Drilling Method: Hollow-Stem Boring Diameter: 8”	Boring B-2
Water Content (%)	Dry Density (pcf)	Blow Counts per 12”	Sample	Depth (ft.)	Graphic Log	DESCRIPTIONS	
				0		0 – 5.0’ **ARTIFICIAL FILL (Af)**, silty clay (CL), very dark yellowish brown, moist, soft to firm, abundant asphalt debris	
				2.5			
30	92	6	MC	5		5.0’ – 21.0’ **ALLUVIUM (Qal)**	
				7.5		Clayey silt (ML), dark yellowish brown, moist	
23	102	16	MC	10			
23	101	18	MC	12.5			
				15			
				17.5			
15	111	19	MC	20		Silty clay to clayey sand (CL-SC), very dark yellowish brown, moist, fine grained	
140 lb. Auto Hammer				Logger: PRK		TOTAL DEPTH: 21 FEET GROUNDWATER NOT ENCOUNTERED	

APPENDIX B

LABORATORY TESTING

Laboratory testing was performed on samples obtained as outlined in the Field Exploration section of this report. All samples were sent to the laboratory for examination, testing in general conformance to specified test methods, and classification using the Unified Soil Classification System and group symbol. All of the testing performed complies with current ASTM standards.

The physical and chemical properties of the soils were tested at Creative Geotechnical, Inc., a City of Los Angeles approved testing laboratory. In accordance with Section 91.7008.5 of the 2017 Los Angeles Building Code, I, the undersigned engineer, have reviewed, concur with, and accept **all** the laboratory testing data and results provided by Creative Geotechnical, Inc., in the letter dated March 16, 2020.

CREATIVE GEOTECHNICAL, INC.
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

March 16, 2020

Project 20008

Mr. Pedram Rahimikian
Geolotech, Inc.
6311 Van Nuys Blvd., # 223
Van Nuys, California 91401

Subject: **RESULTS OF LABORATORY TESTING**
Project No. 20008
6616 N. Reseda Blvd
Reseda, California

Mr. Rahimikian:

Pursuant to your request, this is a letter to certify that Creative Geotechnical, Inc. has performed laboratory soil tests for the subject project under the supervision of the undersigned engineer. Services performed by this facility were conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality under similar conditions. No other warranties are expressed nor implied. Interpretation of the laboratory test results and applications of the results on the design and construction of the project are beyond the scope of our services.

The laboratory testing program as requested by your firm consists of:

- 10 Moisture Content & Density-ASTM D2216 & D2937
- 5 Atterberg Limits-ASTM D4318
- 6 Particle-Size Analysis Fine-Percent Passing #200 Sieve-ASTM D1140
- 2 Direct Shear-ASTM D3080
- 3 Consolidation Test-ASTM D2435
- 1 Corrosion Suite (CT-417, 422, 643)
- 1 Expansion Index Test-ASTM D4829

Moisture and Density Tests

The dry unit weight and moisture content of the undisturbed samples were determined. The results are tabulated in the Laboratory Recapitulation - Table 1.

Shear Tests

Direct single-shear tests were performed with a direct shear machine. The desired normal load is applied to the specimen and allowed to come to equilibrium. The rate of deflection on the sample is approximately 0.005 inches per minute. The samples are tested at higher and/or lower normal loads in

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order to determine the angle of internal friction and the cohesion. The results are plotted on the Shear Test Diagrams and the results tabulated in the Laboratory Recapitulation - Table 1. Although the soil was described to include gravels they were not included within the samples tested, therefore, the results provide a conservative estimate of the shear strength of the soil.

Consolidation

Consolidation tests were performed on samples, within the brass ring, to predict the soils behavior under a specific load. Porous stones are placed in contact with top and bottom of the samples to permit to allow the addition or release of water. Loads are applied in several increments and the results are recorded at selected time intervals. Samples are tested at field and increased moisture content. The results are plotted on the Consolidation Test Curve and the load at which the water is added as noted on the drawing.

Grain Size Analysis

Sieve
A group of sieves is assembled with a solid collecting pan at the bottom. The sample is placed in top sieve. The assembly is placed in the sieve shaker. Upon completion of the sieving operation the weight of the material retained on each is determined.

Hydrometer
The sample is thoroughly mixed with sodium hexametaphosphate solution. The mixed solution is transferred to the sedimentation cylinder. The hydrometer reading is recorded at specified time intervals.

Atterbergs Limits

Liquid Limit
A sample at a specified moisture content is placed in the liquid limit device. The cup drops required to close a groove in the sample is recorded. Three samples at varying moisture contents are tested.

Plastic Limit
A sample at a specified moisture content is rolled on a glass plate. The moisture content is varied until the sample crumbles at a diameter of 1/8".

pH (CTM 643)

A sample of dry soil and distilled water are placed in a flask and allowed to stand for approximately an hour to stabilize. The pH is measured using a pH meter that has been compensated for temperature. The results are tabulated in the Laboratory Recapitulation - Table 2.

Minimum Resistivity (CTM 643)

The electrical resistivity of each soil specimen is conducted in a two-stage process using the soil box method. The first stage measures the resistivity of the soil in its as-received condition and the second stage records the value after saturation with distilled water. The results are tabulated in the Laboratory Recapitulation - Table 2.

March 16, 2020
Project 20008

Page 3

Chloride Content (CTM 422)

A sample of dry soil is mixed with distilled water and allowed to stand overnight. The top aliquot of the sample is mixed with chloride indicator and titrated over silver nitrate solution. The chloride content is determined by the difference of the volumes required to complete titration. The results are tabulated in the Laboratory Recapitulation - Table 2.

Sulfate Content (CTM 417)

A sample of dry soil is mixed with distilled water and allowed to stand overnight. The top aliquot is mixed with distilled water and a conditioning agent. The solution is then placed in a photometer and the value recorded. The process is repeated with the addition of barium chloride. The sulfate content is determined by the difference of the photometer readings. The results are tabulated in the Laboratory Recapitulation - Table 2.

Expansion Index Tests

The sample is compacted into an expansion mold with a degree of saturation between 40-60%. A vertical confining pressure of 144 psf is applied to the sample. The sample is inundated with distilled water. The deformation is recorded after 24 hours. The test results are shown in the Laboratory Recapitulation - Table 2.

Should you have any questions regarding these laboratory test results, please do not hesitate to contact the undersigned at your convenience.

Respectfully submitted,
Creative Geotechnical, Inc.

Raymond Haddad
President
RMH/20008 - 1
Attachments: Laboratory Test Results

PROJECT NO.: 20008
PROJECT ADDRESS: 6616 Reseda

LABORATORY RECAPITULATION 1

Explorations	Depth (ft)	Material	Dry Density (p.c.f.)	Moisture Content (%)
B-1	5.0	Af	72	29
B-1	15.0	Qal	102	23
B-1	25.0	Qal	105	21
B-1	35.0	Qal	102	23
B-1	47.5	Qal	119	15
B-1	55.0	Qal	103	23
B-2	5.0	Qal	92	30
B-2	10.0	Qal	102	23
B-2	12.5	Qal	101	23
B-2	20.0	Qal	111	15

LABORATORY RECAPITULATION 2

Explorations	Depth (ft)	pH	As-Is Soil Resistivity (ohm-cm)	Minimum Soil Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)
B-2	5'-10'	7.80	1,000	700	63	40

LABORATORY RECAPITULATION 3

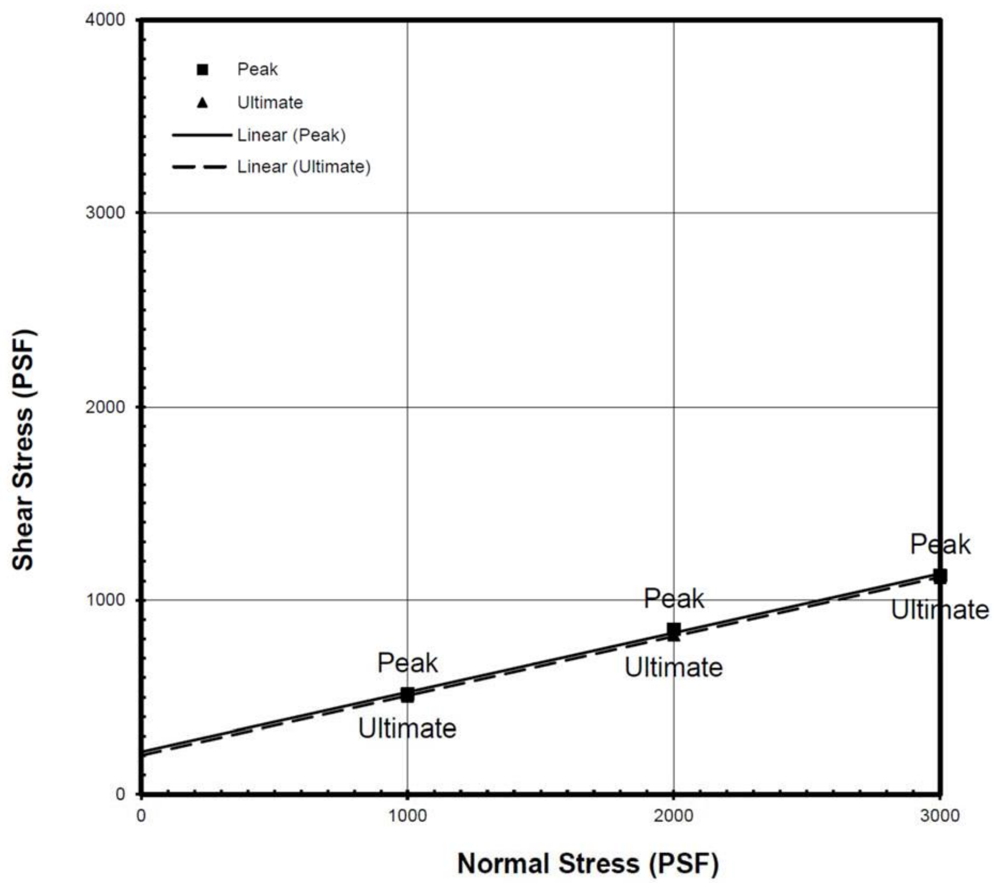
Exploration	Depth (ft)	Expansion Index
B-2	5'	92

Direct Shear Test Diagram (D-3080)

PLATE: S-1
P.N. 20008

Sample Description	Sample Identification	Test Type	Sample Test State	Number of Passes
Af	B-1 @ 5'	Ultimate/Peak	Saturated	1

			Ultimate	Peak
Soil Dry Density (PCF)	72			
Soil Moisture Content (%)	49	**Phi (Degrees)**	17.0	17.0
Soil Saturation (%)	100.1	**Cohesion (PSF)**	201	219

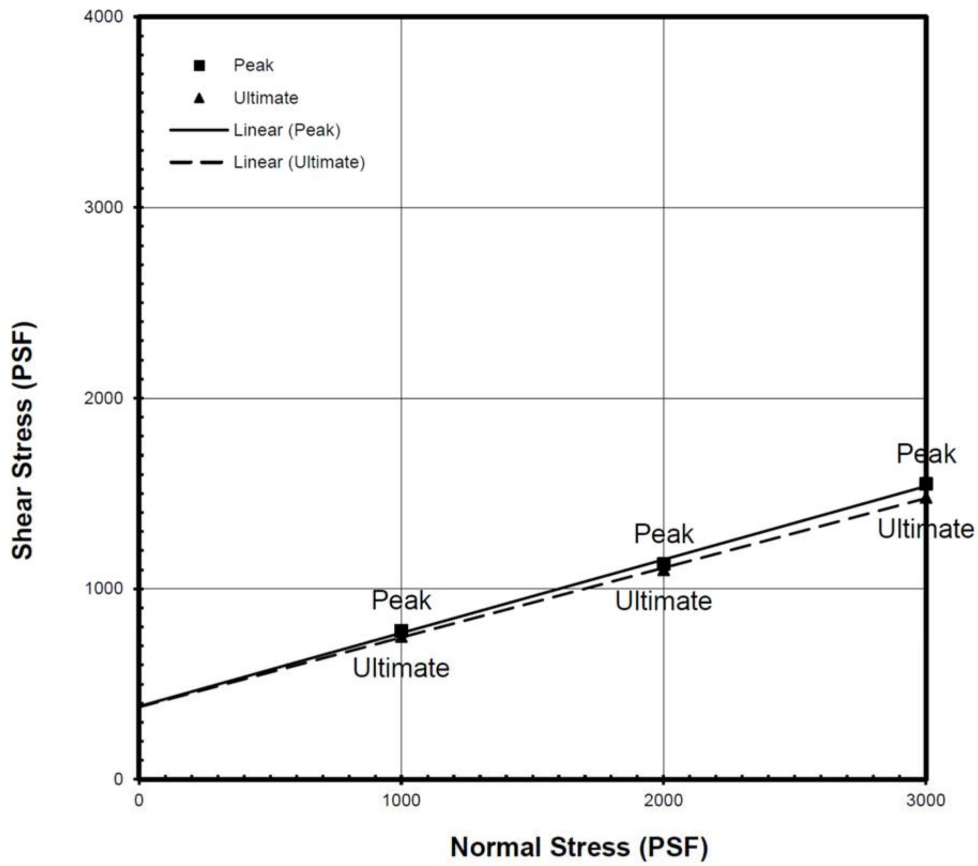


Direct Shear Test Diagram (D-3080)

PLATE: S-2
P.N. 20008

Sample Description	Sample Identification	Test Type	Sample Test State	Number of Passes
Qal	B-2 @ 12.5'	Ultimate/Peak	Saturated	1

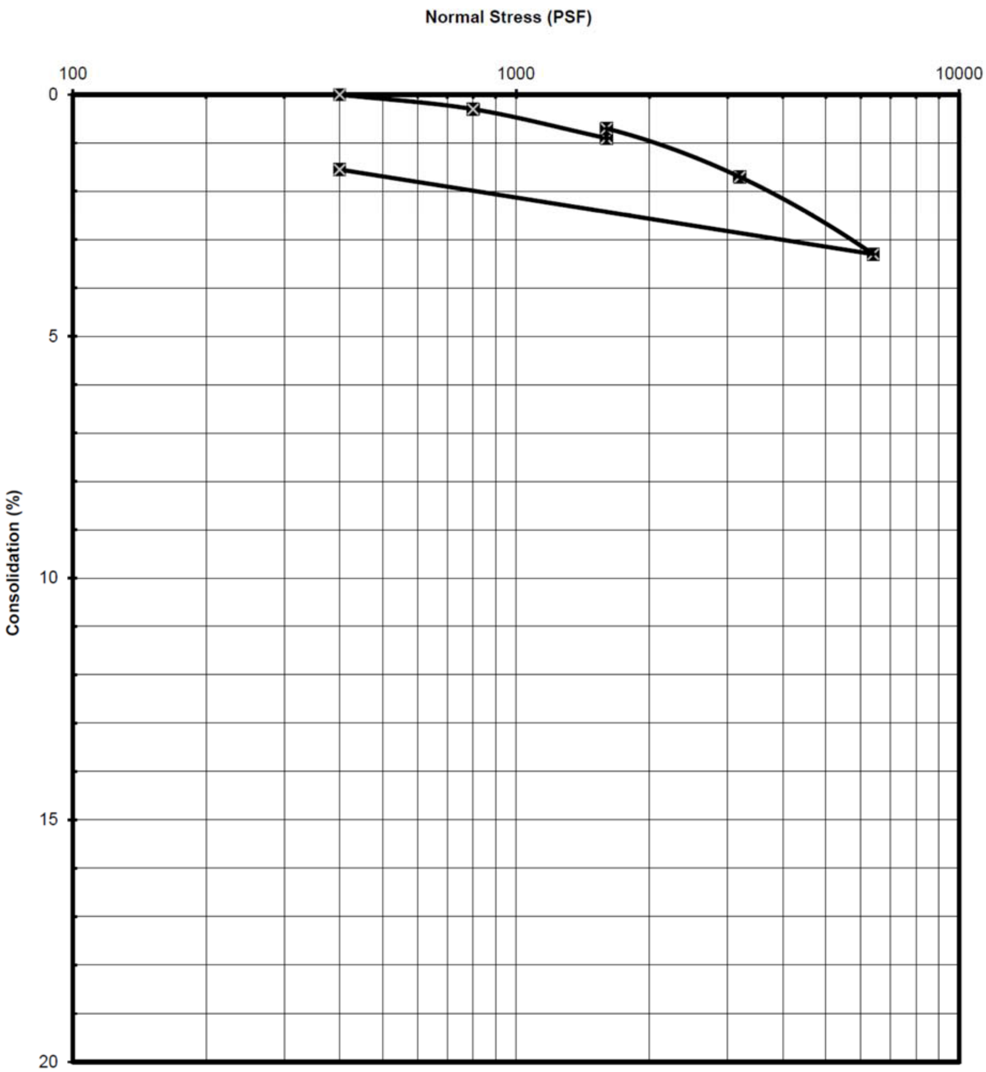
			Ultimate	Peak
Soil Dry Density (PCF)	101			
Soil Moisture Content (%)	24	Phi (Degrees)	20.1	21.1
Soil Saturation (%)	99.8	Cohesion (PSF)	380	383



Consolidation Pressure Curve (D-2435)

Sample Identification	Sample Description
B-1 @ 15'	Qal

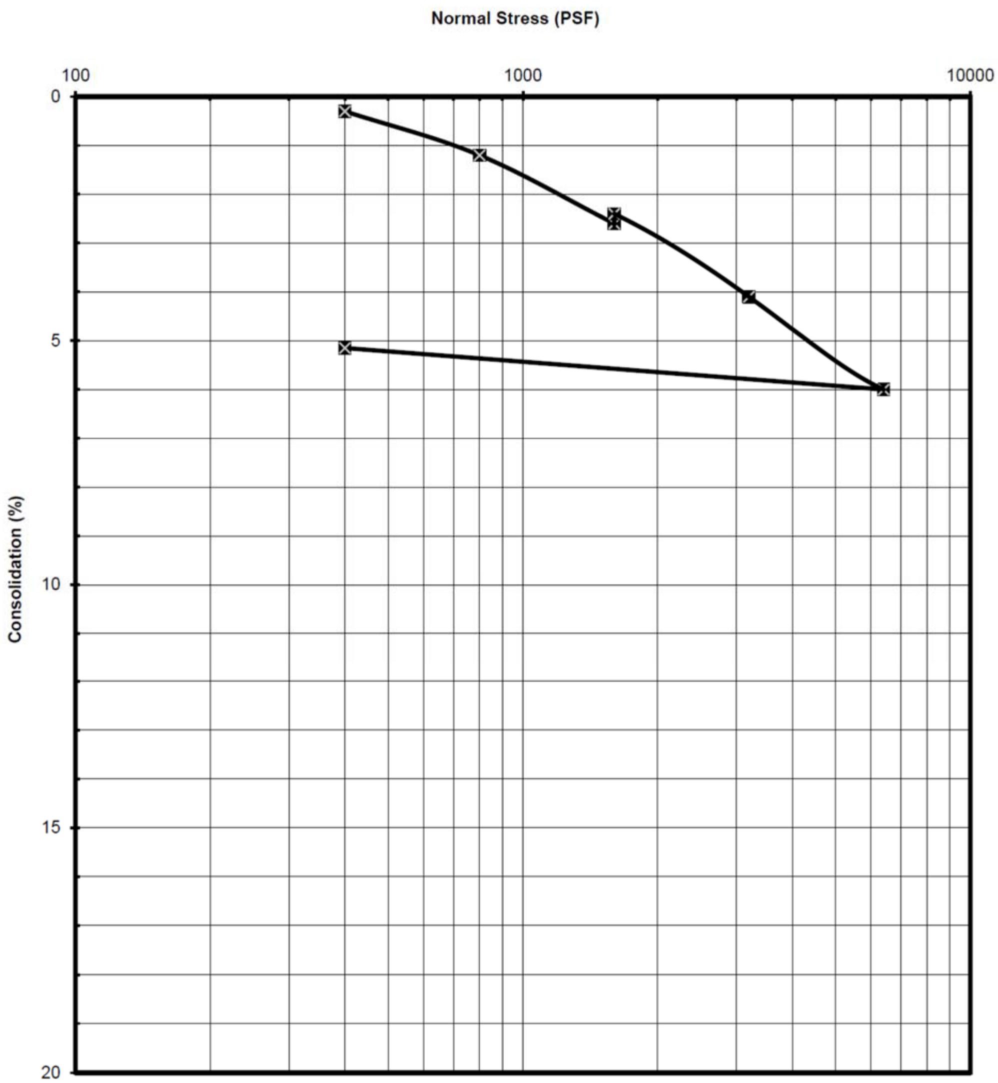
PLATE: C-1
P.N. 20008



Consolidation Pressure Curve (D-2435)

Sample Identification	Sample Description
B-1 @ 25'	Qal

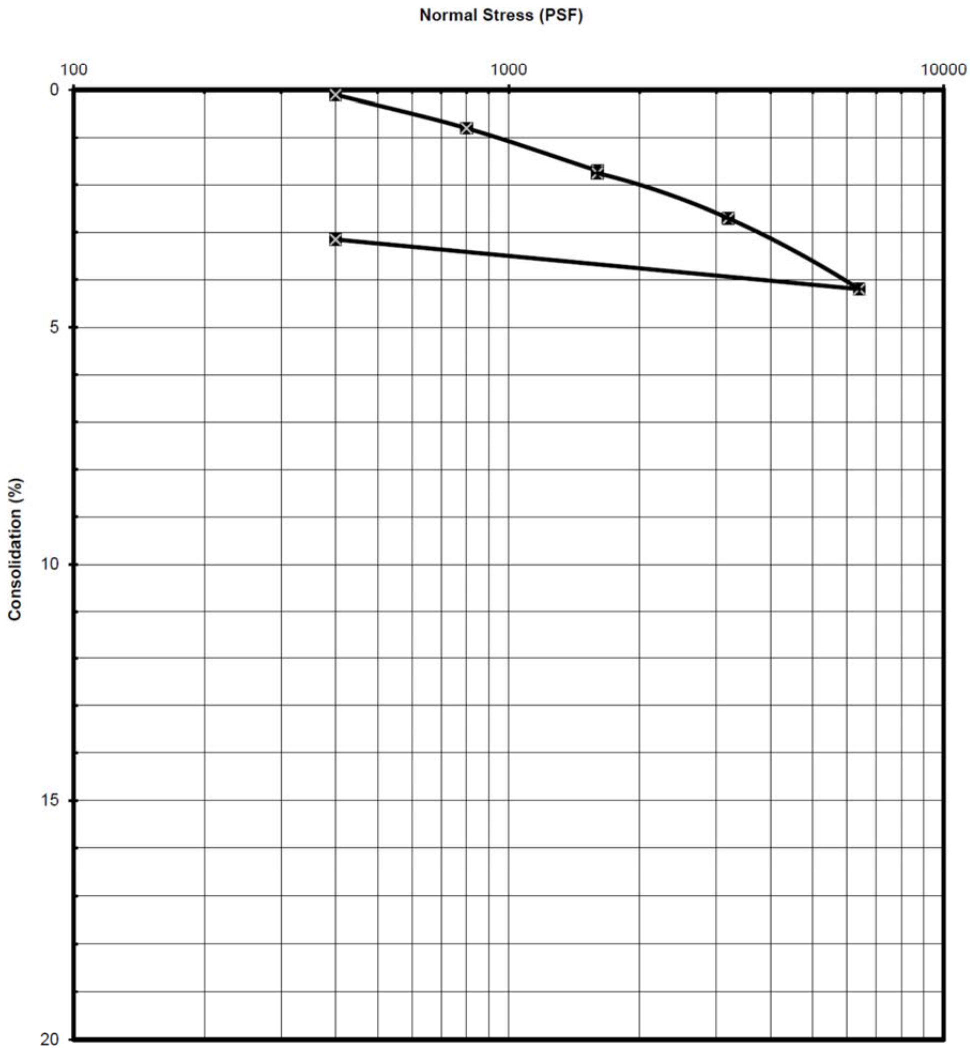
PLATE: C-2
P.N. 20008



Consolidation Pressure Curve (D-2435)

Sample Identification	Sample Description
B-1 @ 47.5'	Qal

PLATE: C-3
P.N. 20008



Grain Size Analysis and Atterberg Limits (ASTM D422 & ASTM D4318)

PLATE:	S-1
P.N.	20008

Exploratrions	Sample Depth (ft)	Soil Description	% of Fines (-200)	LL (%)	PL (%)	PI (%)
B-1	12.5	Qal	84	40	26	14
B-1	22.5	Qal	78	36	22	14
B-1	30	Qal	87	54.5	23	31.5
B-1	40	Qal	87	59	30.3	28.7
B-1	45	Qal	5	-	-	-
B-1	52.5	Qal	49	30.5	16	14.5

APPENDIX C

ANALYSES

Bearing Capacity

Lateral Design

Liquefaction

Seismic Evaluation

BEARING CAPACITY ANALYSIS

CALCULATE THE ULTIMATE AND ALLOWABLE BEARING CAPACITIES OF THE BEARING MATERIAL LISTED BELOW USING HANSEN'S METHOD. (REFERENCE: J. BOWLES, *FOUNDATION ANALYSIS AND DESIGN*, 1988, p. 188-194).

CALCULATION PARAMETERS

EARTH MATERIAL:	Qal	EMBEDMENT DEPTH:	2 feet
SHEAR DIAGRAM:	0	PAD LENGTH:	180 feet
COHESION:	380 psf	PAD WIDTH:	90 feet
PHI ANGLE:	20.1 degrees	SLOPE ANGLE:	0 degrees
DENSITY:	68 pcf	PAD INCLINATION:	0 degrees
SAFETY FACTOR:	5		
FOOTING TYPE:	P Pad		

CALCULATED RESULTS

HANSEN'S SHAPE, DEPTH, AND INCLINATION FACTORS

Nq =	6.46	Dq =	1.01	Sy =	0.80
Nc =	14.93	Gc =	1.00	Dy =	1.00
Ny =	3.00	Bc =	1.00	Iy =	1.00
Sc =	1.22	Iq =	1.00	Gy =	1.00
Sq =	1.18	Ic =	1.00	Gq =	1.00
Dc =	1.01	Bq =	1.00	By =	1.00

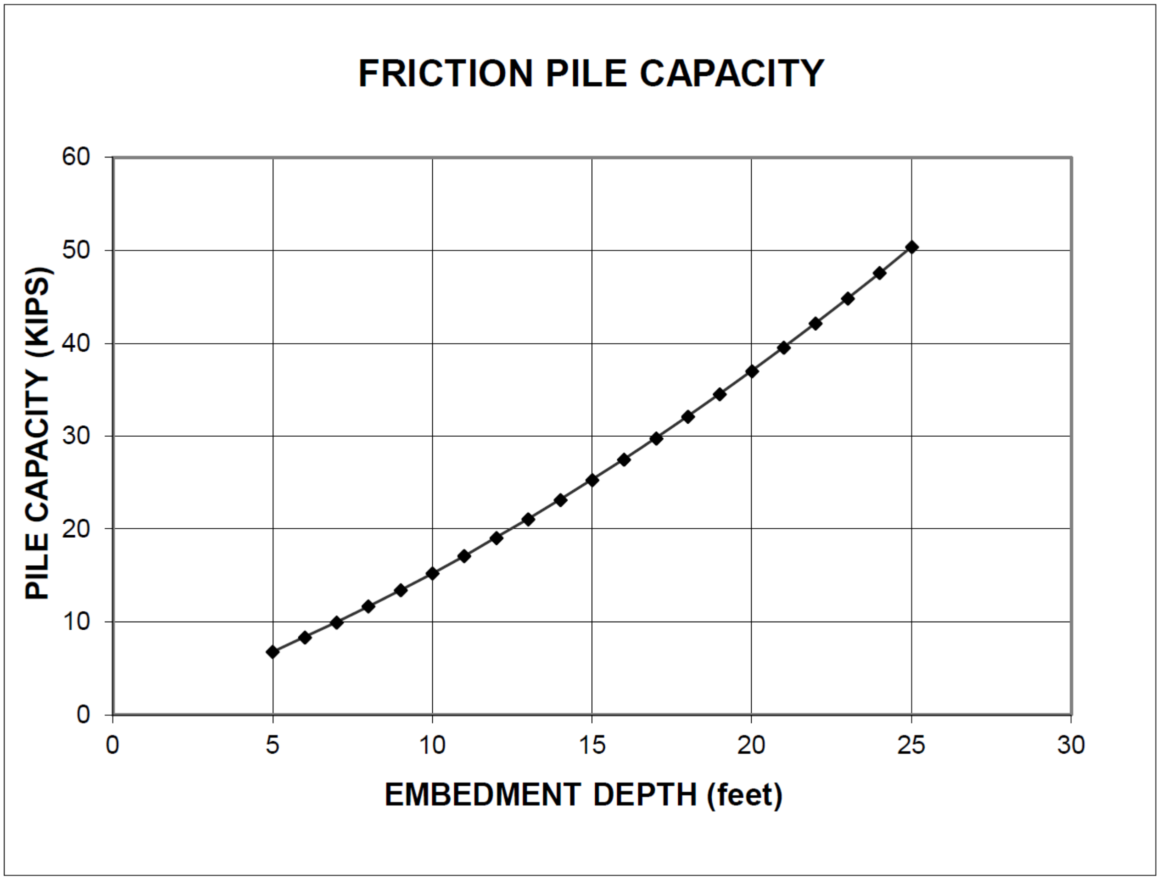
CALCULATED ULTIMATE BEARING CAPACITY (Qult)	15,350.8 pounds
ALLOWABLE BEARING CAPACITY (Qa = Qult / fs)	3,070.2 pounds
PERCENT INCREASE FOR EMBEDMENT DEPTH	3.6%

PILE CAPACITY

CALCULATE ALLOWABLE SKIN FRICTION RESISTANCE FOR DRILLED, CAST IN PLACE CONCRETE PILES. SKIN FRICTION IS TABULATED AS A FUNCTION OF EMBEDMENT DEPTH. (REFERENCES: NAVFAC DM-7.2, PAGES 193-193 AND J.E. BOWLES, "*FOUNDATION ANALYSIS AND DESIGN*," 1988.)

CALCULATION PARAMETERS

EARTH MATERIAL:	Qal	PILE DIAMETER:	2 feet
SHEAR DIAGRAM:	0	INITIAL PILE DEPTH:	5 feet
COHESION:	380 psf	FINAL PILE DEPTH:	25 feet
PHI ANGLE:	20.1 degrees	EXTERNAL SURCHARGE:	0 pounds
DENSITY:	68 pcf	ADHESION VALUE:	1.00
SAFETY FACTOR:	2	PILE/SOIL FRICTION:	23.7 degrees
COMPRESSION/TENSION:	C	LATERAL COEFF. (Ko)	0.70
PILE TYPE:	COMPRESSION PILE	NO GROUNDWATER	0.00 feet



CONCLUSIONS:

THE CALCULATED CAPACITY OF 24 INCH DIAMETER PILES, AS A FUNCTION OF EMBEDMENT, ARE SHOWN IN THE GRAPH.

PASSIVE EARTH PRESSURE

USE RANKINE'S METHOD TO CALCULATE THE PASSIVE EARTH PRESSURE. USE THE PROCEDURE IN NAVFAC DM-7, 1982, (p 7.2-21, Figure 2).

CALCULATION PARAMETERS

EARTH MATERIAL:	Qal	SAFETY FACTOR (fs):	1.5
SHEAR DIAGRAM:	0	INITIAL SEARCH DEPTH:	1
COHESION:	380 psf	FINAL SEARCH DEPTH:	21
PHI ANGLE:	20.1 degrees	LIMIT PASSIVE (Y OR N):	Y
DENSITY:	68 pcf	MAXIMUM PASSIVE:	100,000.0 pounds
		Cd (C/fs):	253.3 psf
		PhiD = atan(tan(phi)/fs) =	13.7 degrees

FOOTING DEPTH (feet)	TOTAL PASSIVE FORCE Pp (pounds)	PASSIVE EARTH PRESSURE AT DEPTH - SigmaP (psf)	INCREASE IN PASSIVE EARTH PRESSURE WITH EMBEDMENT DEPTH (psf/f)
1	700.3	755.4	755.4
2	1,510.8	865.6	110.2
3	2,431.5	975.9	110.2
4	3,462.5	1,086.1	110.2
5	4,603.8	1,196.4	110.2
6	5,855.3	1,306.6	110.2
7	7,217.0	1,416.9	110.2
8	8,689.0	1,527.1	110.2
9	10,271.2	1,637.4	110.2
10	11,963.7	1,747.6	110.2
11	13,766.4	1,857.9	110.2
12	15,679.4	1,968.1	110.2
13	17,702.6	2,078.3	110.2
14	19,836.1	2,188.6	110.2
15	22,079.8	2,298.8	110.2
16	24,433.8	2,409.1	110.2
17	26,898.0	2,519.3	110.2
18	29,472.5	2,629.6	110.2
19	32,157.2	2,739.8	110.2
20	34,952.1	2,850.1	110.2
21	37,857.3	2,960.3	110.2

RETAINING WALL

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. THE MONONOBE-OKABE METHOD USED TO CALCULATE SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Af	WALL HEIGHT	12 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	201 psf	SURCHARGE:	0 pounds
PHI ANGLE:	17 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	130 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	134.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 11.5 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	50 degrees
AREA OF TRIAL FAILURE WEDGE	57.9 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	7522.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	12 feet
DEPTH OF TENSION CRACK	2.5 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	8.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**3891.7 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**54.1 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**55.0 pcf**

RETAINING WALL

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. THE MONONOBE-OKABE METHOD USED TO CALCULATE SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Af	WALL HEIGHT	12 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	201 psf	SURCHARGE:	0 pounds
PHI ANGLE:	17 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	68 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	134.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 11.5 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	50 degrees
AREA OF TRIAL FAILURE WEDGE	50.5 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	3437.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	9 feet
DEPTH OF TENSION CRACK	4.8 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	6.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**1166.6 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**16.2 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**16.2 pcf**

SHORING PILE

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. THE MONONOBE-OKABE METHOD USED TO CALCULATE SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Af	RETAINED LENGTH	12 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	201 psf	SURCHARGE:	0 pounds
PHI ANGLE:	17 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	130 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	160.8 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 13.7 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	52 degrees
AREA OF TRIAL FAILURE WEDGE	52.6 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	6843.4 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	11 feet
DEPTH OF TENSION CRACK	3.0 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	7.0 feet
CALCULATED THRUST ON PILE	**3134.5 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**43.5 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**45.0 pcf**

TEMPORARY EXCAVATION HEIGHT

CALCULATE THE HEIGHT TO WHICH TEMPORARY EXCAVATIONS ARE STABLE (NEGATIVE THRUST).
THE EXCAVATION HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW.
ASSUME THE EARTH MATERIAL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

CALCULATION PARAMETERS

EARTH MATERIAL:	Af	WALL HEIGHT:	4 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	201 psf	SURCHARGE:	0 pounds
PHI ANGLE:	17 degrees	SURCHARGE TYPE:	U Uniform
DENSITY:	130 pcf	INITIAL FAILURE ANGLE:	20 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	3 feet
CD (C/FS):	160.8 psf	FINAL TENSION CRACK:	10 feet

PHID = ATAN(TAN(PHI)/FS) = 13.7 degrees

CALCULATED RESULTS

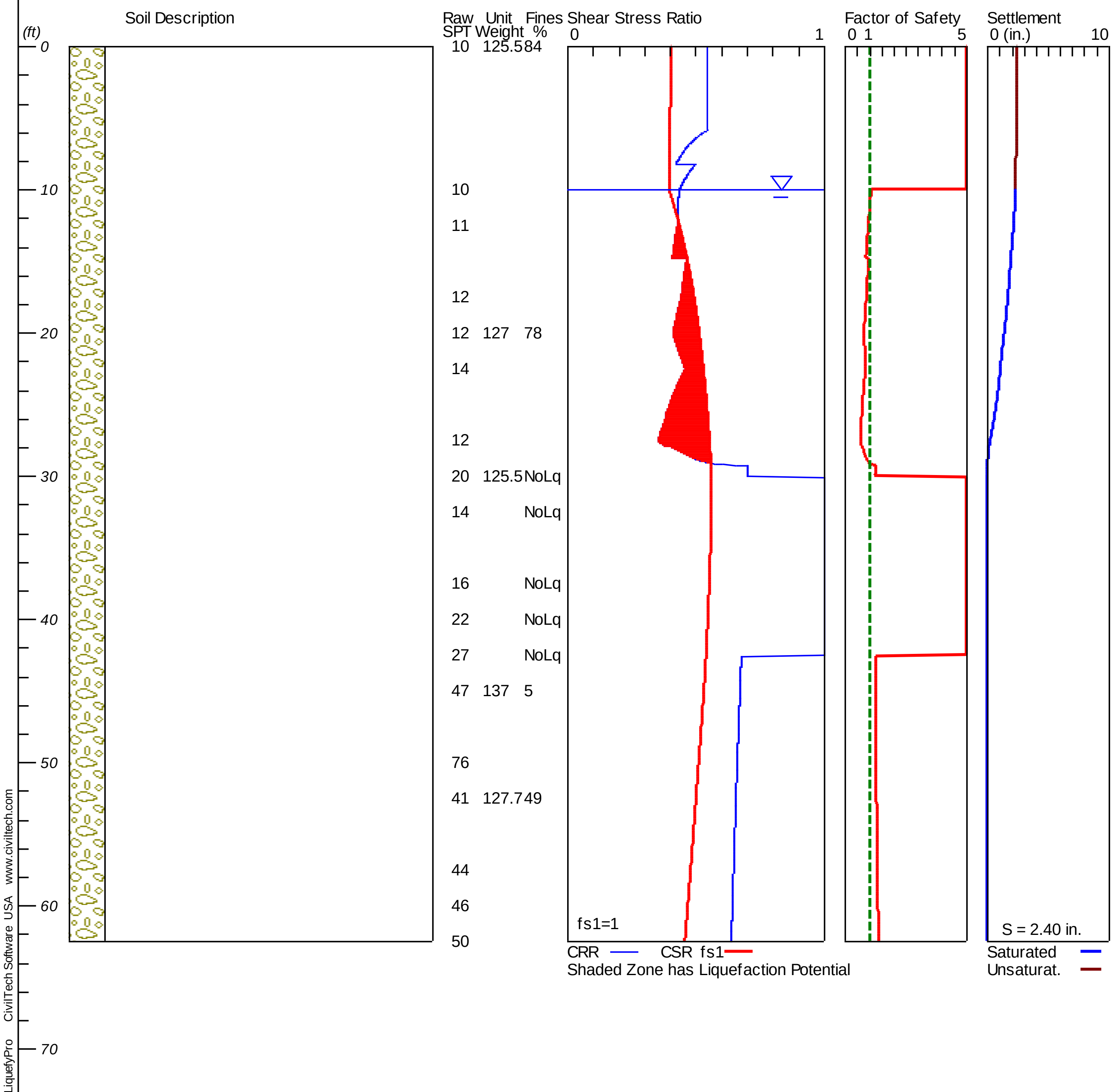
CRITICAL FAILURE ANGLE	39 degrees
AREA OF TRIAL FAILURE WEDGE	8.4 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	1086.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	408 trials
LENGTH OF FAILURE PLANE	3.9 feet
DEPTH OF TENSION CRACK	1.6 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	3.0 feet
CALCULATED HORIZONTAL THRUST	**-154.2 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-19.3 pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**4.0 feet**

LIQUEFACTION ANALYSIS

6616 Reseda Blvd.

Hole No.=B-1 Water Depth=10 ft

Magnitude=6.53
Acceleration=0.625g



CivilTech Corporation

20008

Plate A-1

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*******************************************************************************************************
                                    LIQUEFACTION ANALYSIS CALCULATION DETAILS
                                          Copyright by CivilTech Software
                                               www.civiltech.com

*******************************************************************************************************
       Font: Courier New, Regular, Size 8 is recommended for this report.
       Licensed to Raymond M. Haddad, Creative Geotechnical, Inc.  3/16/2020      12:14:21 PM

       Input File Name: C:\Liquefy5\20008-1 B1.liq
       Title:  6616 Reseda Blvd.
       Subtitle:  20008

 Input Data:
       Surface Elev.=
       Hole No.=B-1
       Depth of Hole=62.50 ft
       Water Table during Earthquake= 10.00 ft
       Water Table during In-Situ Testing= 29.00 ft
       Max. Acceleration=0.63 g
       Earthquake Magnitude=6.53
       No-Liquefiable Soils:   CL, OL are Non-Liq. Soil
       1. SPT or BPT Calculation.
       2. Settlement Analysis Method: Ishihara / Yoshimine
       3. Fines Correction for Liquefaction: Stark/Olson et al.*
       4. Fine Correction for Settlement: During Liquefaction*
       5. Settlement Calculation in: All zones*
       6. Hammer Energy Ratio,                                   Ce = 1.25
       7. Borehole Diameter,                                         Cb= 1.15
       8. Sampling Method,                                          Cs= 1.2
       9. User request factor of safety (apply to CSR) ,   User= 1.3
          Plot one CSR curve (fs1=1)
       10. Average two input data between two Depths: Yes*
       * Recommended Options

       In-Situ Test Data:
       Depth   SPT    Gamma   Fines
       ft             pcf     %
       __________________________________
       0.00    10.00  125.50  84.00
       10.00   10.00  125.50  84.00
       12.50   11.00  125.50  84.00
       17.50   12.00  125.50  84.00
       20.00   12.00  127.00  78.00
       22.50   14.00  127.00  78.00
       27.50   12.00  127.00  78.00
       30.00   20.00  125.50  NoLiq
       32.50   14.00  125.50  NoLiq
       37.50   16.00  125.50  NoLiq
       40.00   22.00  125.50  NoLiq
       42.50   27.00  125.50  NoLiq
       45.00   47.00  137.00  5.00
       50.00   76.00  137.00  5.00
       52.50   41.00  127.70  49.00
       57.50   44.00  127.70  49.00
       60.00   46.00  127.70  49.00
       62.50   50.00  127.70  49.00
       __________________________________


 Output Results:
       Calculation segment, dz=0.050 ft
       User defined Print Interval, dp=2.00 ft

       Peak Ground Acceleration (PGA), a_max = 0.63g

       CSR Calculation:
       Depth   gamma   sigma   gamma'  sigma'  rd     mZ      a(z)    CSR     x fs1   =CSRfs
       ft      pcf     atm     pcf     atm            g       g
       ____________________________________________________________________________________
```

0.00	125.50	0.000	125.50	0.000	1.00	0.000	0.625	0.41	1.00	0.41
2.00	125.50	0.119	125.50	0.119	1.00	0.000	0.625	0.40	1.00	0.40
4.00	125.50	0.237	125.50	0.237	0.99	0.000	0.625	0.40	1.00	0.40
6.00	125.50	0.356	125.50	0.356	0.99	0.000	0.625	0.40	1.00	0.40
8.00	125.50	0.474	125.50	0.474	0.98	0.000	0.625	0.40	1.00	0.40
10.00	125.50	0.593	63.10	0.593	0.98	0.000	0.625	0.40	1.00	0.40
12.00	125.50	0.712	63.10	0.653	0.97	0.000	0.625	0.43	1.00	0.43
14.00	125.50	0.830	63.10	0.712	0.97	0.000	0.625	0.46	1.00	0.46
16.00	125.50	0.949	63.10	0.772	0.96	0.000	0.625	0.48	1.00	0.48
18.00	125.80	1.068	63.40	0.832	0.96	0.000	0.625	0.50	1.00	0.50
20.00	127.00	1.187	64.60	0.892	0.95	0.000	0.625	0.52	1.00	0.52
22.00	127.00	1.307	64.60	0.953	0.95	0.000	0.625	0.53	1.00	0.53
24.00	127.00	1.427	64.60	1.014	0.94	0.000	0.625	0.54	1.00	0.54
26.00	127.00	1.547	64.60	1.075	0.94	0.000	0.625	0.55	1.00	0.55
28.00	126.70	1.667	64.30	1.136	0.93	0.000	0.625	0.56	1.00	0.56
30.00	125.50	1.786	63.10	1.196	0.93	0.000	0.625	0.56	1.00	0.56
32.00	125.50	1.905	63.10	1.256	0.91	0.000	0.625	0.56	1.00	0.56
34.00	125.50	2.023	63.10	1.316	0.90	0.000	0.625	0.56	1.00	0.56
36.00	125.50	2.142	63.10	1.375	0.88	0.000	0.625	0.56	1.00	0.56
38.00	125.50	2.261	63.10	1.435	0.86	0.000	0.625	0.55	1.00	0.55
40.00	125.50	2.379	63.10	1.495	0.85	0.000	0.625	0.55	1.00	0.55
42.00	125.50	2.498	63.10	1.554	0.83	0.000	0.625	0.54	1.00	0.54
44.00	132.40	2.619	70.00	1.616	0.82	0.000	0.625	0.54	1.00	0.54
46.00	137.00	2.747	74.60	1.686	0.80	0.000	0.625	0.53	1.00	0.53
48.00	137.00	2.877	74.60	1.756	0.78	0.000	0.625	0.52	1.00	0.52
50.00	137.00	3.006	74.60	1.827	0.77	0.000	0.625	0.51	1.00	0.51
52.00	129.56	3.132	67.16	1.894	0.75	0.000	0.625	0.50	1.00	0.50
54.00	127.70	3.253	65.30	1.956	0.73	0.000	0.625	0.50	1.00	0.50
56.00	127.70	3.374	65.30	2.017	0.72	0.000	0.625	0.49	1.00	0.49
58.00	127.70	3.494	65.30	2.079	0.70	0.000	0.625	0.48	1.00	0.48
60.00	127.70	3.615	65.30	2.141	0.69	0.000	0.625	0.47	1.00	0.47
62.00	127.70	3.736	65.30	2.203	0.67	0.000	0.625	0.46	1.00	0.46

CSR is based on water table at 10.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	10.00	1.73	0.75	0.000	1.70	21.99	84.00	7.20	29.19	0.38
2.00	10.00	1.73	0.75	0.119	1.70	21.99	84.00	7.20	29.19	0.38
4.00	10.00	1.73	0.75	0.237	1.70	21.99	84.00	7.20	29.19	0.38
6.00	10.00	1.73	0.75	0.356	1.68	21.69	84.00	7.20	28.89	0.37
8.00	10.00	1.73	0.75	0.474	1.45	18.78	84.00	7.20	25.98	0.30
10.00	10.00	1.73	0.85	0.593	1.30	19.04	84.00	7.20	26.24	0.30
12.00	10.80	1.73	0.85	0.712	1.19	18.77	84.00	7.20	25.97	0.30
14.00	11.30	1.73	0.85	0.830	1.10	18.18	84.00	7.20	25.38	0.29
16.00	11.70	1.73	0.95	0.949	1.03	19.68	84.00	7.20	26.88	0.32
18.00	12.00	1.73	0.95	1.068	0.97	19.03	82.80	7.20	26.23	0.30
20.00	12.00	1.73	0.95	1.187	0.92	18.05	78.00	7.20	25.25	0.29
22.00	13.60	1.73	0.95	1.307	0.87	19.49	78.00	7.20	26.69	0.31
24.00	13.40	1.73	0.95	1.427	0.84	18.38	78.00	7.20	25.58	0.29
26.00	12.60	1.73	0.95	1.547	0.80	16.60	78.00	7.20	23.80	0.26
28.00	13.60	1.73	1.00	1.667	0.77	18.17	82.60	7.20	25.37	0.29
30.00	20.00	1.73	1.00	1.758	0.75	26.02	NoLiq	7.20	33.22	0.50
32.00	15.20	1.73	1.00	1.818	0.74	19.45	NoLiq	7.20	26.65	0.31
34.00	14.60	1.73	1.00	1.877	0.73	18.38	NoLiq	7.20	25.58	0.29
36.00	15.40	1.73	1.00	1.937	0.72	19.09	NoLiq	7.20	26.29	0.31
38.00	17.20	1.73	1.00	1.997	0.71	21.00	NoLiq	7.20	28.20	0.35
40.00	22.00	1.73	1.00	2.056	0.70	26.46	NoLiq	7.20	33.66	0.50
42.00	26.00	1.73	1.00	2.116	0.69	30.83	NoLiq	7.20	38.03	0.50
44.00	39.00	1.73	1.00	2.178	0.68	45.58	43.41	7.20	52.78	0.50
46.00	52.80	1.73	1.00	2.247	0.67	60.75	5.00	0.00	60.75	0.50
48.00	64.40	1.73	1.00	2.318	0.66	72.96	5.00	0.00	72.96	0.50
50.00	76.00	1.73	1.00	2.388	0.65	84.83	5.00	0.00	84.83	0.50
52.00	48.01	1.73	1.00	2.455	0.64	52.85	40.19	7.20	60.05	0.50
54.00	41.90	1.73	1.00	2.517	0.63	45.55	49.00	7.20	52.75	0.50
56.00	43.10	1.73	1.00	2.579	0.62	46.29	49.00	7.20	53.49	0.50
58.00	44.40	1.73	1.00	2.641	0.62	47.13	49.00	7.20	54.33	0.50
60.00	46.00	1.73	1.00	2.703	0.61	48.27	49.00	7.20	55.47	0.50
62.00	49.20	1.73	1.00	2.764	0.60	51.05	49.00	7.20	58.25	0.50

CRR is based on water table at 29.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.53:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	0.38	1.00	0.38	1.43	0.55	0.41	5.00
2.00	0.08	0.38	1.00	0.38	1.43	0.55	0.40	5.00
4.00	0.15	0.38	1.00	0.38	1.43	0.55	0.40	5.00
6.00	0.23	0.37	1.00	0.37	1.43	0.53	0.40	5.00
8.00	0.31	0.30	1.00	0.30	1.43	0.43	0.40	5.00
10.00	0.39	0.30	1.00	0.30	1.43	0.43	0.40	1.09
12.00	0.46	0.30	1.00	0.30	1.43	0.43	0.43	0.99 *
14.00	0.54	0.29	1.00	0.29	1.43	0.41	0.46	0.90 *
16.00	0.62	0.32	1.00	0.32	1.43	0.45	0.48	0.94 *
18.00	0.69	0.30	1.00	0.30	1.43	0.43	0.50	0.87 *
20.00	0.77	0.29	1.00	0.29	1.43	0.41	0.52	0.79 *
22.00	0.85	0.31	1.00	0.31	1.43	0.45	0.53	0.84 *
24.00	0.93	0.29	1.00	0.29	1.43	0.42	0.54	0.77 *
26.00	1.01	0.26	1.01	0.27	1.43	0.38	0.55	0.69 *
28.00	1.08	0.29	0.99	0.29	1.43	0.41	0.56	0.73 *
30.00	1.14	0.50	0.98	0.49	1.43	0.70	0.56	1.24
32.00	1.18	0.31	0.98	0.31	1.43	2.00	0.56	5.00 ^
34.00	1.22	0.29	0.97	0.28	1.43	2.00	0.56	5.00 ^
36.00	1.26	0.31	0.97	0.29	1.43	2.00	0.56	5.00 ^
38.00	1.30	0.35	0.96	0.33	1.43	2.00	0.55	5.00 ^
40.00	1.34	0.50	0.96	0.48	1.43	2.00	0.55	5.00 ^
42.00	1.38	0.50	0.95	0.47	1.43	2.00	0.54	5.00 ^
44.00	1.42	0.50	0.94	0.47	1.43	0.67	0.54	1.25
46.00	1.46	0.50	0.94	0.47	1.43	0.67	0.53	1.26
48.00	1.51	0.50	0.93	0.47	1.43	0.66	0.52	1.27
50.00	1.55	0.50	0.93	0.46	1.43	0.66	0.51	1.29
52.00	1.60	0.50	0.92	0.46	1.43	0.66	0.50	1.30
54.00	1.64	0.50	0.92	0.46	1.43	0.65	0.50	1.31
56.00	1.68	0.50	0.91	0.45	1.43	0.65	0.49	1.33
58.00	1.72	0.50	0.90	0.45	1.43	0.64	0.48	1.35
60.00	1.76	0.50	0.90	0.45	1.43	0.64	0.47	1.36
62.00	1.80	0.50	0.90	0.45	1.43	0.64	0.46	1.38

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
^ No-liquefiable Soils or above Water Table.
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	29.19	84.00	0.00	29.19
2.00	-	-	-	29.19	84.00	0.00	29.19
4.00	-	-	-	29.19	84.00	0.00	29.19
6.00	-	-	-	28.89	84.00	0.00	28.89
8.00	-	-	-	25.98	84.00	0.00	25.98
10.00	-	-	-	26.24	84.00	0.00	26.24
12.00	-	-	-	25.97	84.00	0.00	25.97
14.00	-	-	-	25.38	84.00	0.00	25.38
16.00	-	-	-	26.88	84.00	0.00	26.88
18.00	-	-	-	26.23	82.80	0.00	26.23
20.00	-	-	-	25.25	78.00	0.00	25.25
22.00	-	-	-	26.69	78.00	0.00	26.69
24.00	-	-	-	25.58	78.00	0.00	25.58
26.00	-	-	-	23.80	78.00	0.00	23.80
28.00	-	-	-	25.37	82.60	0.00	25.37
30.00	-	-	-	33.22	NoLiq	0.00	33.22
32.00	-	-	-	26.65	NoLiq	0.00	26.65
34.00	-	-	-	25.58	NoLiq	0.00	25.58
36.00	-	-	-	26.29	NoLiq	0.00	26.29
38.00	-	-	-	28.20	NoLiq	0.00	28.20
40.00	-	-	-	33.66	NoLiq	0.00	33.66

42.00	-	-	-	38.03	NoLiq	0.00	38.03
44.00	-	-	-	52.78	43.41	0.00	52.78
46.00	-	-	-	60.75	5.00	0.00	60.75
48.00	-	-	-	72.96	5.00	0.00	72.96
50.00	-	-	-	84.83	5.00	0.00	84.83
52.00	-	-	-	60.05	40.19	0.00	60.05
54.00	-	-	-	52.75	49.00	0.00	52.75
56.00	-	-	-	53.49	49.00	0.00	53.49
58.00	-	-	-	54.33	49.00	0.00	54.33
60.00	-	-	-	55.47	49.00	0.00	55.47
62.00	-	-	-	58.25	49.00	0.00	58.25

(N1)60s has been fines corrected in liquefaction analysis, therefore d(N1)60=0.
Fines=NoLiq means the soils are not liquefiable.

Settlement of Saturated Sands:
Settlement Analysis Method: Ishihara / Yoshimine

Depth	CSRsf	/ MSF*	=CSRm	F.S.	Fines	(N1)60s	Dr	ec	dsz	dsp	S
ft					%		%	%	in.	in.	in.
62.45	0.46	1.00	0.46	1.39	49.00	58.86	100.00	0.000	0.0E0	0.000	0.000
62.00	0.46	1.00	0.46	1.38	49.00	58.25	100.00	0.000	0.0E0	0.000	0.000
60.00	0.47	1.00	0.47	1.36	49.00	55.47	100.00	0.000	0.0E0	0.000	0.000
58.00	0.48	1.00	0.48	1.35	49.00	54.33	100.00	0.000	0.0E0	0.000	0.000
56.00	0.49	1.00	0.49	1.33	49.00	53.49	100.00	0.000	0.0E0	0.000	0.000
54.00	0.50	1.00	0.50	1.31	49.00	52.75	100.00	0.000	0.0E0	0.000	0.000
52.00	0.50	1.00	0.50	1.30	40.19	60.05	100.00	0.000	0.0E0	0.000	0.000
50.00	0.51	1.00	0.51	1.29	5.00	84.83	100.00	0.000	0.0E0	0.000	0.000
48.00	0.52	1.00	0.52	1.27	5.00	72.96	100.00	0.000	0.0E0	0.000	0.000
46.00	0.53	1.00	0.53	1.26	5.00	60.75	100.00	0.000	0.0E0	0.000	0.000
44.00	0.54	1.00	0.54	1.25	43.41	52.78	100.00	0.000	0.0E0	0.000	0.000
42.00	0.54	1.00	0.54	5.00	NoLiq	38.03	100.00	0.000	0.0E0	0.000	0.000
40.00	0.55	1.00	0.55	5.00	NoLiq	33.66	99.00	0.000	0.0E0	0.000	0.000
38.00	0.55	1.00	0.55	5.00	NoLiq	28.20	86.12	0.000	0.0E0	0.000	0.000
36.00	0.56	1.00	0.56	5.00	NoLiq	26.29	82.22	0.000	0.0E0	0.000	0.000
34.00	0.56	1.00	0.56	5.00	NoLiq	25.58	80.84	0.000	0.0E0	0.000	0.000
32.00	0.56	1.00	0.56	5.00	NoLiq	26.65	82.94	0.000	0.0E0	0.000	0.000
30.00	0.56	1.00	0.56	1.24	NoLiq	33.22	97.83	0.052	0.0E0	0.000	0.000
28.00	0.56	1.00	0.56	0.73	82.60	25.37	80.43	1.321	7.9E-3	0.140	0.140
26.00	0.55	1.00	0.55	0.69	78.00	23.80	77.46	1.630	9.8E-3	0.417	0.557
24.00	0.54	1.00	0.54	0.77	78.00	25.58	80.84	1.202	7.2E-3	0.340	0.897
22.00	0.53	1.00	0.53	0.84	78.00	26.69	83.04	0.944	5.7E-3	0.242	1.140
20.00	0.52	1.00	0.52	0.79	78.00	25.25	80.20	1.175	7.0E-3	0.255	1.394
18.00	0.50	1.00	0.50	0.87	82.80	26.23	82.12	0.920	5.5E-3	0.251	1.645
16.00	0.48	1.00	0.48	0.94	84.00	26.88	83.41	0.722	4.3E-3	0.195	1.840
14.00	0.46	1.00	0.46	0.90	84.00	25.38	80.46	0.890	5.3E-3	0.185	2.024
12.00	0.43	1.00	0.43	0.99	84.00	25.97	81.60	0.649	3.9E-3	0.183	2.207
10.00	0.40	1.00	0.40	1.09	84.00	26.24	82.13	0.460	2.8E-3	0.134	2.341

Settlement of Saturated Sands=2.341 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth	sigma'	sigC'	(N1)60s	CSRsf	Gmax	g*Ge/Gm	g_eff	ec7.5	Cec	ec	dsz	dsp	S
ft	atm	atm			atm			%		%	in.	in.	in.
9.95	0.59	0.38	26.29	0.40	822.63	2.8E-4	0.0746	0.0513	0.78	0.0401	4.81E-4	0.000	0.000
8.00	0.47	0.31	25.98	0.40	734.77	2.6E-4	0.0886	0.0619	0.78	0.0484	5.81E-4	0.021	0.021
6.00	0.36	0.23	28.89	0.40	659.20	2.2E-4	0.0446	0.0269	0.78	0.0210	2.52E-4	0.015	0.036
4.00	0.24	0.15	29.19	0.40	540.13	1.8E-4	0.0331	0.0196	0.78	0.0154	1.84E-4	0.009	0.045

```
2.00   0.12   0.08   29.19   0.40   381.94  1.3E-4  0.0232  0.0138  0.78   0.0108  1.29E-4 0.007
0.052
0.00   0.00   0.00   29.19   0.41   3.51    1.2E-6  0.0010  0.0006  0.78   0.0005  5.66E-6 0.003
0.055
```

Settlement of Unsaturated Sands=0.055 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=2.396 in.
Differential Settlement=1.198 to 1.581 in.

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

```
1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)
SPT            Field data from Standard Penetration Test (SPT)
BPT            Field data from Becker Penetration Test (BPT)
qc             Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs             Friction from CPT testing [atm (tsf)]
Rf             Ratio of fs/qc (%)
gamma          Total unit weight of soil
gamma'         Effective unit weight of soil
Fines          Fines content [%]
D50            Mean grain size
Dr             Relative Density
sigma          Total vertical stress [atm]
sigma'         Effective vertical stress [atm]
sigC'          Effective confining pressure [atm]
rd             Acceleration reduction coefficient by Seed
a_max.         Peak Ground Acceleration (PGA) in ground surface
mZ             Linear acceleration reduction coefficient X depth
a_min.         Minimum acceleration under linear reduction, mZ
CRRv           CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
  CRR7.5              Cyclic resistance ratio (M=7.5)
  Ksig         Overburden stress correction factor for CRR7.5
CRRm           After magnitude scaling correction CRRm=CRRv * MSF
  MSF          Magnitude scaling factor from M=7.5 to user input M
CSR            Cyclic stress ratio induced by earthquake
CSRfs          CSRfs=CSR*fs1 (Default fs1=1)
  fs1          First CSR curve in graphic defined in #9 of Advanced page
  fs2          2nd CSR curve in graphic defined in #9 of Advanced page
F.S.           Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs           Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr             Rod Length Corrections
Cn             Overburden Pressure Correction
(N1)60         SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60        Fines correction of SPT
(N1)60f        (N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq             Overburden stress correction factor
qc1            CPT after Overburden stress correction
dqc1           Fines correction of CPT
qc1f           CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n           CPT after normalization in Robertson's method
Kc             Fine correction factor in Robertson's Method
qc1f           CPT after Fines correction in Robertson's Method
Ic             Soil type index in Suzuki's and Robertson's Methods
(N1)60s        (N1)60 after settlement fines corrections
CSRm           After magnitude scaling correction for Settlement calculation  CSRm=CSRsf / MSF*
  CSRfs               Cyclic stress ratio induced by earthquake with user inputed fs
  MSF*                Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec             Volumetric strain for saturated sands
dz             Calculation segment, dz=0.050 ft
dsz            Settlement in each segment, dz
dp             User defined print interval
dsp            Settlement in each print interval, dp
Gmax           Shear Modulus at low strain
g_eff          gamma_eff, Effective shear Strain
```

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g*Ge/Gm        gamma_eff * G_eff/G_max,      Strain-modulus ratio
ec7.5          Volumetric Strain for magnitude=7.5
Cec            Magnitude correction factor for any magnitude
ec             Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq          No-Liquefy Soils
```

References:

1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.
SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.
2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.
3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK, Earthquake Engineering Research Center, Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

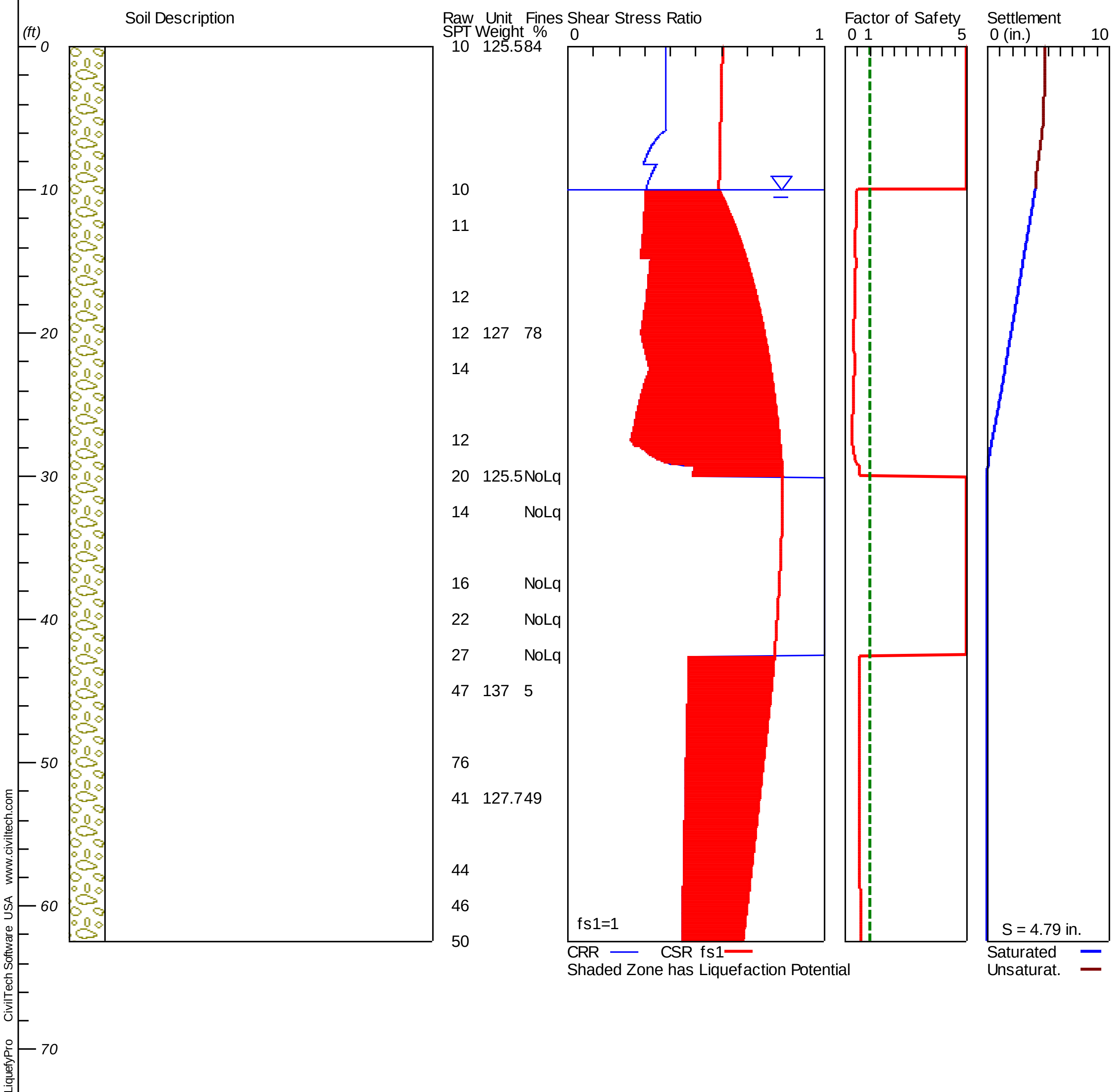
Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

LIQUEFACTION ANALYSIS

6616 Reseda Blvd.

Hole No.=B-1   Water Depth=10 ft

Magnitude=7.52
Acceleration=0.933g



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****************************************************************************************************
                                    LIQUEFACTION ANALYSIS CALCULATION DETAILS
                                          Copyright by CivilTech Software
                                               www.civiltech.com

****************************************************************************************************
       Font: Courier New, Regular, Size 8 is recommended for this report.
       Licensed to Raymond M. Haddad, Creative Geotechnical, Inc.  3/16/2020      12:15:49 PM

       Input File Name: C:\Liquefy5\20008-1 B1.liq
       Title:  6616 Reseda Blvd.
       Subtitle:  20008

 Input Data:
       Surface Elev.=
       Hole No.=B-1
       Depth of Hole=62.50 ft
       Water Table during Earthquake= 10.00 ft
       Water Table during In-Situ Testing= 29.00 ft
       Max. Acceleration=0.93 g
       Earthquake Magnitude=7.52
       No-Liquefiable Soils:   CL, OL are Non-Liq. Soil
       1. SPT or BPT Calculation.
       2. Settlement Analysis Method: Ishihara / Yoshimine
       3. Fines Correction for Liquefaction: Stark/Olson et al.*
       4. Fine Correction for Settlement: During Liquefaction*
       5. Settlement Calculation in: All zones*
       6. Hammer Energy Ratio,                                  Ce = 1.25
       7. Borehole Diameter,                                        Cb= 1.15
       8. Sampling Method,                                         Cs= 1.2
       9. User request factor of safety (apply to CSR) ,   User= 1.3
          Plot one CSR curve (fs1=1)
       10. Average two input data between two Depths: Yes*
       * Recommended Options

       In-Situ Test Data:
       Depth   SPT    Gamma   Fines
       ft             pcf     %
       __________________________________
       0.00    10.00  125.50  84.00
       10.00   10.00  125.50  84.00
       12.50   11.00  125.50  84.00
       17.50   12.00  125.50  84.00
       20.00   12.00  127.00  78.00
       22.50   14.00  127.00  78.00
       27.50   12.00  127.00  78.00
       30.00   20.00  125.50  NoLiq
       32.50   14.00  125.50  NoLiq
       37.50   16.00  125.50  NoLiq
       40.00   22.00  125.50  NoLiq
       42.50   27.00  125.50  NoLiq
       45.00   47.00  137.00  5.00
       50.00   76.00  137.00  5.00
       52.50   41.00  127.70  49.00
       57.50   44.00  127.70  49.00
       60.00   46.00  127.70  49.00
       62.50   50.00  127.70  49.00
       __________________________________


 Output Results:
       Calculation segment, dz=0.050 ft
       User defined Print Interval, dp=2.00 ft

       Peak Ground Acceleration (PGA), a_max = 0.93g

       CSR Calculation:
       Depth   gamma   sigma   gamma'  sigma'  rd     mZ      a(z)    CSR    x fs1   =CSRfs
       ft      pcf     atm     pcf     atm            g       g
       ____________________________________________________________________________________
```

0.00	125.50	0.000	125.50	0.000	1.00	0.000	0.933	0.61	1.00	0.61
2.00	125.50	0.119	125.50	0.119	1.00	0.000	0.933	0.60	1.00	0.60
4.00	125.50	0.237	125.50	0.237	0.99	0.000	0.933	0.60	1.00	0.60
6.00	125.50	0.356	125.50	0.356	0.99	0.000	0.933	0.60	1.00	0.60
8.00	125.50	0.474	125.50	0.474	0.98	0.000	0.933	0.60	1.00	0.60
10.00	125.50	0.593	63.10	0.593	0.98	0.000	0.933	0.59	1.00	0.59
12.00	125.50	0.712	63.10	0.653	0.97	0.000	0.933	0.64	1.00	0.64
14.00	125.50	0.830	63.10	0.712	0.97	0.000	0.933	0.68	1.00	0.68
16.00	125.50	0.949	63.10	0.772	0.96	0.000	0.933	0.72	1.00	0.72
18.00	125.80	1.068	63.40	0.832	0.96	0.000	0.933	0.75	1.00	0.75
20.00	127.00	1.187	64.60	0.892	0.95	0.000	0.933	0.77	1.00	0.77
22.00	127.00	1.307	64.60	0.953	0.95	0.000	0.933	0.79	1.00	0.79
24.00	127.00	1.427	64.60	1.014	0.94	0.000	0.933	0.81	1.00	0.81
26.00	127.00	1.547	64.60	1.075	0.94	0.000	0.933	0.82	1.00	0.82
28.00	126.70	1.667	64.30	1.136	0.93	0.000	0.933	0.83	1.00	0.83
30.00	125.50	1.786	63.10	1.196	0.93	0.000	0.933	0.84	1.00	0.84
32.00	125.50	1.905	63.10	1.256	0.91	0.000	0.933	0.84	1.00	0.84
34.00	125.50	2.023	63.10	1.316	0.90	0.000	0.933	0.84	1.00	0.84
36.00	125.50	2.142	63.10	1.375	0.88	0.000	0.933	0.83	1.00	0.83
38.00	125.50	2.261	63.10	1.435	0.86	0.000	0.933	0.83	1.00	0.83
40.00	125.50	2.379	63.10	1.495	0.85	0.000	0.933	0.82	1.00	0.82
42.00	125.50	2.498	63.10	1.554	0.83	0.000	0.933	0.81	1.00	0.81
44.00	132.40	2.619	70.00	1.616	0.82	0.000	0.933	0.80	1.00	0.80
46.00	137.00	2.747	74.60	1.686	0.80	0.000	0.933	0.79	1.00	0.79
48.00	137.00	2.877	74.60	1.756	0.78	0.000	0.933	0.78	1.00	0.78
50.00	137.00	3.006	74.60	1.827	0.77	0.000	0.933	0.77	1.00	0.77
52.00	129.56	3.132	67.16	1.894	0.75	0.000	0.933	0.75	1.00	0.75
54.00	127.70	3.253	65.30	1.956	0.73	0.000	0.933	0.74	1.00	0.74
56.00	127.70	3.374	65.30	2.017	0.72	0.000	0.933	0.73	1.00	0.73
58.00	127.70	3.494	65.30	2.079	0.70	0.000	0.933	0.72	1.00	0.72
60.00	127.70	3.615	65.30	2.141	0.69	0.000	0.933	0.70	1.00	0.70
62.00	127.70	3.736	65.30	2.203	0.67	0.000	0.933	0.69	1.00	0.69

CSR is based on water table at 10.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	10.00	1.73	0.75	0.000	1.70	21.99	84.00	7.20	29.19	0.38
2.00	10.00	1.73	0.75	0.119	1.70	21.99	84.00	7.20	29.19	0.38
4.00	10.00	1.73	0.75	0.237	1.70	21.99	84.00	7.20	29.19	0.38
6.00	10.00	1.73	0.75	0.356	1.68	21.69	84.00	7.20	28.89	0.37
8.00	10.00	1.73	0.75	0.474	1.45	18.78	84.00	7.20	25.98	0.30
10.00	10.00	1.73	0.85	0.593	1.30	19.04	84.00	7.20	26.24	0.30
12.00	10.80	1.73	0.85	0.712	1.19	18.77	84.00	7.20	25.97	0.30
14.00	11.30	1.73	0.85	0.830	1.10	18.18	84.00	7.20	25.38	0.29
16.00	11.70	1.73	0.95	0.949	1.03	19.68	84.00	7.20	26.88	0.32
18.00	12.00	1.73	0.95	1.068	0.97	19.03	82.80	7.20	26.23	0.30
20.00	12.00	1.73	0.95	1.187	0.92	18.05	78.00	7.20	25.25	0.29
22.00	13.60	1.73	0.95	1.307	0.87	19.49	78.00	7.20	26.69	0.31
24.00	13.40	1.73	0.95	1.427	0.84	18.38	78.00	7.20	25.58	0.29
26.00	12.60	1.73	0.95	1.547	0.80	16.60	78.00	7.20	23.80	0.26
28.00	13.60	1.73	1.00	1.667	0.77	18.17	82.60	7.20	25.37	0.29
30.00	20.00	1.73	1.00	1.758	0.75	26.02	NoLiq	7.20	33.22	0.50
32.00	15.20	1.73	1.00	1.818	0.74	19.45	NoLiq	7.20	26.65	0.31
34.00	14.60	1.73	1.00	1.877	0.73	18.38	NoLiq	7.20	25.58	0.29
36.00	15.40	1.73	1.00	1.937	0.72	19.09	NoLiq	7.20	26.29	0.31
38.00	17.20	1.73	1.00	1.997	0.71	21.00	NoLiq	7.20	28.20	0.35
40.00	22.00	1.73	1.00	2.056	0.70	26.46	NoLiq	7.20	33.66	0.50
42.00	26.00	1.73	1.00	2.116	0.69	30.83	NoLiq	7.20	38.03	0.50
44.00	39.00	1.73	1.00	2.178	0.68	45.58	43.41	7.20	52.78	0.50
46.00	52.80	1.73	1.00	2.247	0.67	60.75	5.00	0.00	60.75	0.50
48.00	64.40	1.73	1.00	2.318	0.66	72.96	5.00	0.00	72.96	0.50
50.00	76.00	1.73	1.00	2.388	0.65	84.83	5.00	0.00	84.83	0.50
52.00	48.01	1.73	1.00	2.455	0.64	52.85	40.19	7.20	60.05	0.50
54.00	41.90	1.73	1.00	2.517	0.63	45.55	49.00	7.20	52.75	0.50
56.00	43.10	1.73	1.00	2.579	0.62	46.29	49.00	7.20	53.49	0.50
58.00	44.40	1.73	1.00	2.641	0.62	47.13	49.00	7.20	54.33	0.50
60.00	46.00	1.73	1.00	2.703	0.61	48.27	49.00	7.20	55.47	0.50
62.00	49.20	1.73	1.00	2.764	0.60	51.05	49.00	7.20	58.25	0.50

CRR is based on water table at 29.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 7.52:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	0.38	1.00	0.38	0.99	0.38	0.61	5.00
2.00	0.08	0.38	1.00	0.38	0.99	0.38	0.60	5.00
4.00	0.15	0.38	1.00	0.38	0.99	0.38	0.60	5.00
6.00	0.23	0.37	1.00	0.37	0.99	0.37	0.60	5.00
8.00	0.31	0.30	1.00	0.30	0.99	0.30	0.60	5.00
10.00	0.39	0.30	1.00	0.30	0.99	0.30	0.59	0.51 *
12.00	0.46	0.30	1.00	0.30	0.99	0.30	0.64	0.46 *
14.00	0.54	0.29	1.00	0.29	0.99	0.29	0.68	0.42 *
16.00	0.62	0.32	1.00	0.32	0.99	0.31	0.72	0.44 *
18.00	0.69	0.30	1.00	0.30	0.99	0.30	0.75	0.40 *
20.00	0.77	0.29	1.00	0.29	0.99	0.28	0.77	0.37 *
22.00	0.85	0.31	1.00	0.31	0.99	0.31	0.79	0.39 *
24.00	0.93	0.29	1.00	0.29	0.99	0.29	0.81	0.36 *
26.00	1.01	0.26	1.01	0.27	0.99	0.26	0.82	0.32 *
28.00	1.08	0.29	0.99	0.29	0.99	0.28	0.83	0.34 *
30.00	1.14	0.50	0.98	0.49	0.99	0.49	0.84	0.58 *
32.00	1.18	0.31	0.98	0.31	0.99	2.00	0.84	5.00 ^
34.00	1.22	0.29	0.97	0.28	0.99	2.00	0.84	5.00 ^
36.00	1.26	0.31	0.97	0.29	0.99	2.00	0.83	5.00 ^
38.00	1.30	0.35	0.96	0.33	0.99	2.00	0.83	5.00 ^
40.00	1.34	0.50	0.96	0.48	0.99	2.00	0.82	5.00 ^
42.00	1.38	0.50	0.95	0.47	0.99	2.00	0.81	5.00 ^
44.00	1.42	0.50	0.94	0.47	0.99	0.47	0.80	0.58 *
46.00	1.46	0.50	0.94	0.47	0.99	0.47	0.79	0.59 *
48.00	1.51	0.50	0.93	0.47	0.99	0.46	0.78	0.59 *
50.00	1.55	0.50	0.93	0.46	0.99	0.46	0.77	0.60 *
52.00	1.60	0.50	0.92	0.46	0.99	0.46	0.75	0.61 *
54.00	1.64	0.50	0.92	0.46	0.99	0.45	0.74	0.61 *
56.00	1.68	0.50	0.91	0.45	0.99	0.45	0.73	0.62 *
58.00	1.72	0.50	0.90	0.45	0.99	0.45	0.72	0.63 *
60.00	1.76	0.50	0.90	0.45	0.99	0.45	0.70	0.64 *
62.00	1.80	0.50	0.90	0.45	0.99	0.44	0.69	0.65 *

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
^ No-liquefiable Soils or above Water Table.
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	29.19	84.00	0.00	29.19
2.00	-	-	-	29.19	84.00	0.00	29.19
4.00	-	-	-	29.19	84.00	0.00	29.19
6.00	-	-	-	28.89	84.00	0.00	28.89
8.00	-	-	-	25.98	84.00	0.00	25.98
10.00	-	-	-	26.24	84.00	0.00	26.24
12.00	-	-	-	25.97	84.00	0.00	25.97
14.00	-	-	-	25.38	84.00	0.00	25.38
16.00	-	-	-	26.88	84.00	0.00	26.88
18.00	-	-	-	26.23	82.80	0.00	26.23
20.00	-	-	-	25.25	78.00	0.00	25.25
22.00	-	-	-	26.69	78.00	0.00	26.69
24.00	-	-	-	25.58	78.00	0.00	25.58
26.00	-	-	-	23.80	78.00	0.00	23.80
28.00	-	-	-	25.37	82.60	0.00	25.37
30.00	-	-	-	33.22	NoLiq	0.00	33.22
32.00	-	-	-	26.65	NoLiq	0.00	26.65
34.00	-	-	-	25.58	NoLiq	0.00	25.58
36.00	-	-	-	26.29	NoLiq	0.00	26.29
38.00	-	-	-	28.20	NoLiq	0.00	28.20
40.00	-	-	-	33.66	NoLiq	0.00	33.66

42.00	-	-	-	38.03	NoLiq	0.00	38.03
44.00	-	-	-	52.78	43.41	0.00	52.78
46.00	-	-	-	60.75	5.00	0.00	60.75
48.00	-	-	-	72.96	5.00	0.00	72.96
50.00	-	-	-	84.83	5.00	0.00	84.83
52.00	-	-	-	60.05	40.19	0.00	60.05
54.00	-	-	-	52.75	49.00	0.00	52.75
56.00	-	-	-	53.49	49.00	0.00	53.49
58.00	-	-	-	54.33	49.00	0.00	54.33
60.00	-	-	-	55.47	49.00	0.00	55.47
62.00	-	-	-	58.25	49.00	0.00	58.25

(N1)60s has been fines corrected in liquefaction analysis, therefore d(N1)60=0.
Fines=NoLiq means the soils are not liquefiable.

Settlement of Saturated Sands:
Settlement Analysis Method: Ishihara / Yoshimine

Depth	CSRsf	/ MSF*	=CSRm	F.S.	Fines	(N1)60s	Dr	ec	dsz	dsp	S
ft					%		%	%	in.	in.	in.
62.45	0.69	1.00	0.69	0.65	49.00	58.86	100.00	0.000	0.0E0	0.000	0.000
62.00	0.69	1.00	0.69	0.65	49.00	58.25	100.00	0.000	0.0E0	0.000	0.000
60.00	0.70	1.00	0.70	0.64	49.00	55.47	100.00	0.000	0.0E0	0.000	0.000
58.00	0.72	1.00	0.72	0.63	49.00	54.33	100.00	0.000	0.0E0	0.000	0.000
56.00	0.73	1.00	0.73	0.62	49.00	53.49	100.00	0.000	0.0E0	0.000	0.000
54.00	0.74	1.00	0.74	0.61	49.00	52.75	100.00	0.000	0.0E0	0.000	0.000
52.00	0.75	1.00	0.75	0.61	40.19	60.05	100.00	0.000	0.0E0	0.000	0.000
50.00	0.77	1.00	0.77	0.60	5.00	84.83	100.00	0.000	0.0E0	0.000	0.000
48.00	0.78	1.00	0.78	0.59	5.00	72.96	100.00	0.000	0.0E0	0.000	0.000
46.00	0.79	1.00	0.79	0.59	5.00	60.75	100.00	0.000	0.0E0	0.000	0.000
44.00	0.80	1.00	0.80	0.58	43.41	52.78	100.00	0.000	0.0E0	0.000	0.000
42.00	0.81	1.00	0.81	5.00	NoLiq	38.03	100.00	0.000	0.0E0	0.000	0.000
40.00	0.82	1.00	0.82	5.00	NoLiq	33.66	99.00	0.000	0.0E0	0.000	0.000
38.00	0.83	1.00	0.83	5.00	NoLiq	28.20	86.12	0.000	0.0E0	0.000	0.000
36.00	0.83	1.00	0.83	5.00	NoLiq	26.29	82.22	0.000	0.0E0	0.000	0.000
34.00	0.84	1.00	0.84	5.00	NoLiq	25.58	80.84	0.000	0.0E0	0.000	0.000
32.00	0.84	1.00	0.84	5.00	NoLiq	26.65	82.94	0.000	0.0E0	0.000	0.000
30.00	0.84	1.00	0.84	0.58	NoLiq	33.22	97.83	0.249	0.0E0	0.000	0.000
28.00	0.83	1.00	0.83	0.34	82.60	25.37	80.43	1.728	1.0E-2	0.286	0.286
26.00	0.82	1.00	0.82	0.32	78.00	23.80	77.46	1.860	1.1E-2	0.456	0.743
24.00	0.81	1.00	0.81	0.36	78.00	25.58	80.84	1.710	1.0E-2	0.428	1.171
22.00	0.79	1.00	0.79	0.39	78.00	26.69	83.04	1.618	9.7E-3	0.393	1.564
20.00	0.77	1.00	0.77	0.37	78.00	25.25	80.20	1.738	1.0E-2	0.403	1.967
18.00	0.75	1.00	0.75	0.40	82.80	26.23	82.12	1.656	9.9E-3	0.407	2.374
16.00	0.72	1.00	0.72	0.44	84.00	26.88	83.41	1.598	9.6E-3	0.389	2.764
14.00	0.68	1.00	0.68	0.42	84.00	25.38	80.46	1.727	1.0E-2	0.394	3.158
12.00	0.64	1.00	0.64	0.46	84.00	25.97	81.60	1.676	1.0E-2	0.407	3.565
10.00	0.59	1.00	0.59	0.51	84.00	26.24	82.13	1.640	9.8E-3	0.399	3.964

Settlement of Saturated Sands=3.964 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth	sigma'	sigC'	(N1)60s	CSRsf	Gmax	g*Ge/Gm	g_eff	ec7.5	Cec	ec	dsz	dsp	S
ft	atm	atm			atm			%		%	in.	in.	in.
9.95	0.59	0.38	26.29	0.59	822.63	4.2E-4	0.3855	0.2650	1.06	0.2810	3.37E-3	0.003	0.003
8.00	0.47	0.31	25.98	0.60	734.77	3.8E-4	1.0000	0.6984	1.06	0.7405	8.89E-3	0.232	0.235
6.00	0.36	0.23	28.89	0.60	659.20	3.2E-4	1.0000	0.6021	1.06	0.6384	7.66E-3	0.331	0.566
4.00	0.24	0.15	29.19	0.60	540.13	2.6E-4	0.1041	0.0617	1.06	0.0655	7.86E-4	0.106	0.673

```
2.00   0.12   0.08   29.19   0.60   381.94  1.9E-4  0.0472  0.0280  1.06   0.0297  3.56E-4 0.150
0.823
0.00   0.00   0.00   29.19   0.61   3.51    1.7E-6  0.0010  0.0006  1.06   0.0006  7.67E-6 0.008
0.830
```

____

Settlement of Unsaturated Sands=0.830 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=4.795 in.
Differential Settlement=2.397 to 3.164 in.

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

____

```
1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)
SPT            Field data from Standard Penetration Test (SPT)
BPT            Field data from Becker Penetration Test (BPT)
qc             Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs             Friction from CPT testing [atm (tsf)]
Rf             Ratio of fs/qc (%)
gamma          Total unit weight of soil
gamma'         Effective unit weight of soil
Fines          Fines content [%]
D50            Mean grain size
Dr             Relative Density
sigma          Total vertical stress [atm]
sigma'         Effective vertical stress [atm]
sigC'          Effective confining pressure [atm]
rd             Acceleration reduction coefficient by Seed
a_max.         Peak Ground Acceleration (PGA) in ground surface
mZ             Linear acceleration reduction coefficient X depth
a_min.         Minimum acceleration under linear reduction, mZ
CRRv           CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
  CRR7.5              Cyclic resistance ratio (M=7.5)
  Ksig         Overburden stress correction factor for CRR7.5
CRRm           After magnitude scaling correction CRRm=CRRv * MSF
  MSF          Magnitude scaling factor from M=7.5 to user input M
CSR            Cyclic stress ratio induced by earthquake
CSRfs          CSRfs=CSR*fs1 (Default fs1=1)
  fs1          First CSR curve in graphic defined in #9 of Advanced page
  fs2          2nd CSR curve in graphic defined in #9 of Advanced page
F.S.           Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs           Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr             Rod Length Corrections
Cn             Overburden Pressure Correction
(N1)60         SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60        Fines correction of SPT
(N1)60f        (N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq             Overburden stress correction factor
qc1            CPT after Overburden stress correction
dqc1           Fines correction of CPT
qc1f           CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n           CPT after normalization in Robertson's method
Kc             Fine correction factor in Robertson's Method
qc1f           CPT after Fines correction in Robertson's Method
Ic             Soil type index in Suzuki's and Robertson's Methods
(N1)60s        (N1)60 after settlement fines corrections
CSRm           After magnitude scaling correction for Settlement calculation  CSRm=CSRsf / MSF*
  CSRfs               Cyclic stress ratio induced by earthquake with user inputed fs
  MSF*                Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec             Volumetric strain for saturated sands
dz             Calculation segment, dz=0.050 ft
dsz            Settlement in each segment, dz
dp             User defined print interval
dsp            Settlement in each print interval, dp
Gmax           Shear Modulus at low strain
g_eff          gamma_eff, Effective shear Strain
```

```
g*Ge/Gm        gamma_eff * G_eff/G_max,      Strain-modulus ratio
ec7.5          Volumetric Strain for magnitude=7.5
Cec            Magnitude correction factor for any magnitude
ec             Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq          No-Liquefy Soils
```

References:

1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.
SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.
2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.
3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK, Earthquake Engineering Research Center, Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (v4.1.

Latitude

Decimal degrees

34.191

Longitude

Decimal degrees, negative values for western longitudes

-118.536

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

Time Horizon

Return period in years

2475

Hazard Curve

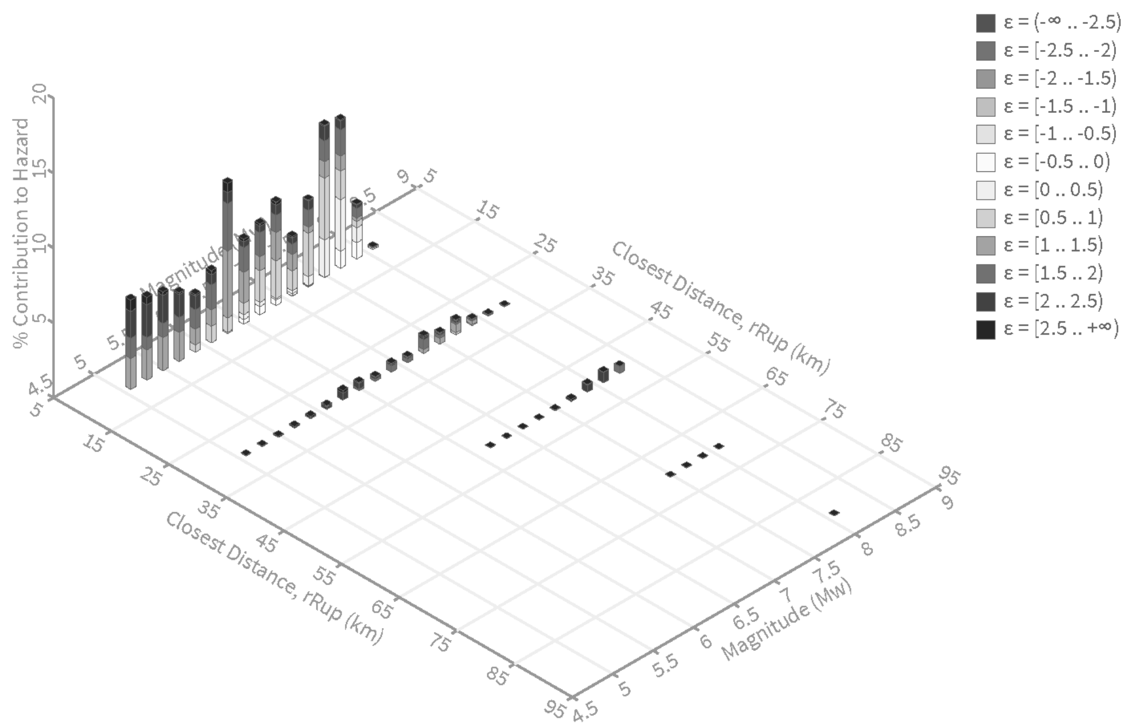
Please select “Edition”, “Location” & “Site Class” above to compute a hazard curve.

Compute Hazard Curve

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 1.1094839 g

Recovered targets

Return period: 2879.001 yrs
Exceedance rate: 0.0003473427 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.09 %

Mean (over all sources)

m: 6.65
r: 12.65 km
ε_0: 1.36 σ

Mode (largest m-r bin)

m: 7.52
r: 13.08 km
ε_0: 0.94 σ
Contribution: 10.14 %

Mode (largest m-r-ε_0 bin)

m: 6.31
r: 11.1 km
ε_0: 1.25 σ
Contribution: 4.48 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							30.93
Santa Susana East (connector) [0]		12.53	6.58	1.46	118.499°W	34.314°N	13.73	5.04
Santa Susana alt 2 [3]		13.23	7.38	0.95	118.545°W	34.309°N	356.30	4.61
Northridge Hills [0]		9.27	7.67	0.51	118.503°W	34.269°N	19.17	2.82
Compton [4]		13.48	7.58	1.05	118.608°W	34.022°N	199.43	2.54
Mission Hills 2011 [1]		10.42	7.18	0.86	118.546°W	34.283°N	354.98	2.54
Santa Monica alt 2 [2]		13.52	7.33	1.15	118.486°W	34.047°N	164.01	2.02
Sierra Madre (San Fernando) [2]		12.50	7.06	1.13	118.488°W	34.296°N	20.60	1.71
Hollywood [2]		15.60	7.02	1.41	118.422°W	34.084°N	138.65	1.69
Northridge [3]		16.19	7.38	1.26	118.500°W	34.334°N	11.83	1.26
UC33brAvg_FM31	System							27.05
Santa Susana East (connector) [0]		12.53	6.89	1.30	118.499°W	34.314°N	13.73	4.55
Mission Hills 2011 [1]		10.42	6.53	1.21	118.546°W	34.283°N	354.98	2.93
Northridge Hills [0]		9.27	7.67	0.51	118.503°W	34.269°N	19.17	2.78
Compton [4]		13.48	7.42	1.16	118.608°W	34.022°N	199.43	2.13
Sierra Madre (San Fernando) [2]		12.50	7.52	0.90	118.488°W	34.296°N	20.60	2.08
Santa Susana alt 1 [0]		13.40	7.55	0.91	118.544°W	34.310°N	356.92	1.95
Northridge [3]		16.19	7.30	1.33	118.500°W	34.334°N	11.83	1.83
Hollywood [2]		15.60	7.32	1.25	118.422°W	34.084°N	138.65	1.80
UC33brAvg_FM31 (opt)	Grid							21.12
PointSourceFinite: -118.536, 34.222		5.99	5.69	1.43	118.536°W	34.222°N	0.00	3.88
PointSourceFinite: -118.536, 34.222		5.99	5.69	1.43	118.536°W	34.222°N	0.00	3.88
PointSourceFinite: -118.536, 34.240		6.90	5.85	1.43	118.536°W	34.240°N	0.00	2.81
PointSourceFinite: -118.536, 34.240		6.90	5.85	1.43	118.536°W	34.240°N	0.00	2.81
PointSourceFinite: -118.536, 34.303		12.25	5.84	1.85	118.536°W	34.303°N	0.00	1.44
PointSourceFinite: -118.536, 34.303		12.25	5.84	1.85	118.536°W	34.303°N	0.00	1.44
UC33brAvg_FM32 (opt)	Grid							20.90
PointSourceFinite: -118.536, 34.222		5.99	5.69	1.43	118.536°W	34.222°N	0.00	3.81
PointSourceFinite: -118.536, 34.222		5.99	5.69	1.43	118.536°W	34.222°N	0.00	3.81
PointSourceFinite: -118.536, 34.240		6.90	5.85	1.43	118.536°W	34.240°N	0.00	2.81
PointSourceFinite: -118.536, 34.240		6.90	5.85	1.43	118.536°W	34.240°N	0.00	2.81
PointSourceFinite: -118.536, 34.303		12.30	5.83	1.86	118.536°W	34.303°N	0.00	1.38
PointSourceFinite: -118.536, 34.303		12.30	5.83	1.86	118.536°W	34.303°N	0.00	1.38

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (v4.1.

Latitude

Decimal degrees

34.191

Longitude

Decimal degrees, negative values for western longitudes

-118.536

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

Time Horizon

Return period in years

475

^ Hazard Curve

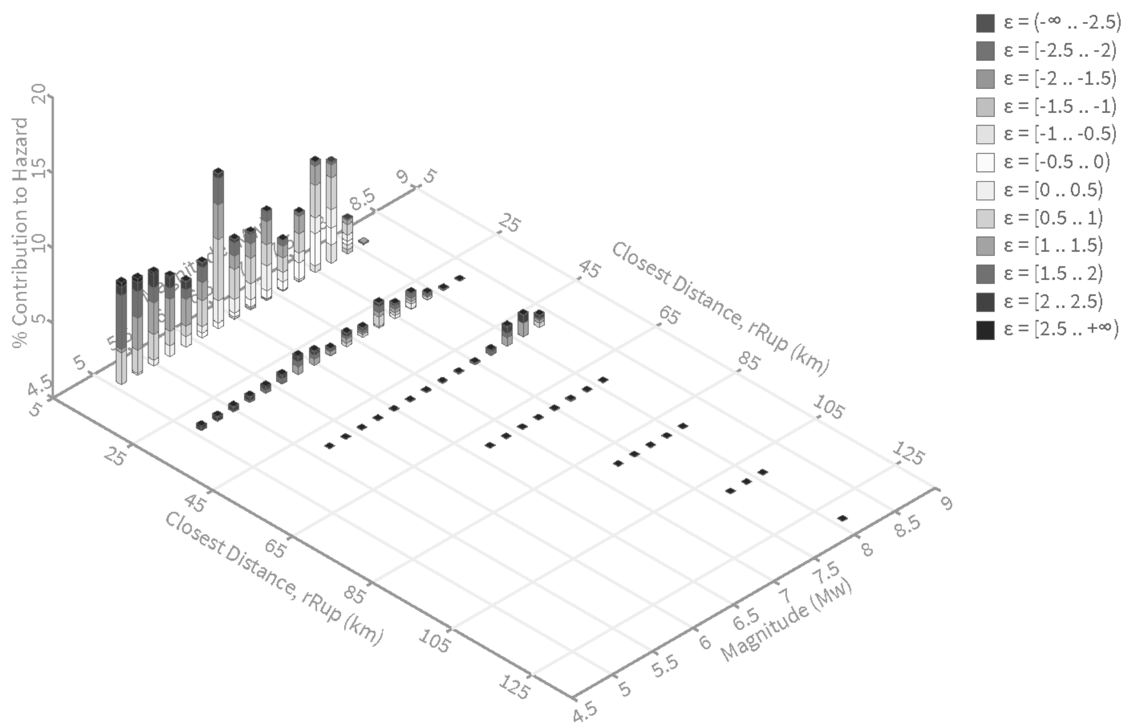
Please select “Edition”, “Location” & “Site Class” above to compute a hazard curve.

Compute Hazard Curve

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 475 yrs
Exceedance rate: 0.0021052632 yr^{-1}
PGA ground motion: 0.62519915 g

Recovered targets

Return period: 519.48407 yrs
Exceedance rate: 0.0019249869 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.22 %

Mean (over all sources)

m: 6.53
r: 14.7 km
ε_0: 1.01 σ

Mode (largest m-r bin)

m: 6.3
r: 11.65 km
ε_0: 0.97 σ
Contribution: 10.31 %

Mode (largest m-r-ε_0 bin)

m: 6.27
r: 11.67 km
ε_0: 0.73 σ
Contribution: 4.09 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

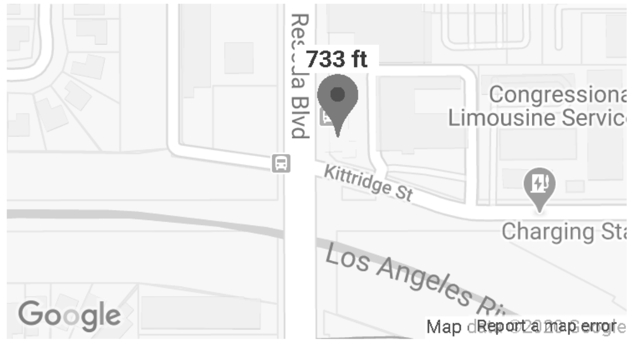
ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							28.38
Santa Susana East (connector) [0]		12.53	6.53	0.82	118.499°W	34.314°N	13.73	4.82
Santa Susana alt 2 [3]		13.23	7.35	0.54	118.545°W	34.309°N	356.30	3.43
Compton [4]		13.48	7.56	0.33	118.608°W	34.022°N	199.43	1.91
Mission Hills 2011 [1]		10.42	7.15	0.48	118.546°W	34.283°N	354.98	1.82
San Andreas (Mojave S) [3]		52.49	8.05	1.63	118.305°W	34.623°N	23.76	1.79
Hollywood [2]		15.60	6.98	0.92	118.422°W	34.084°N	138.65	1.72
Santa Monica alt 2 [2]		13.52	7.24	0.62	118.486°W	34.047°N	164.01	1.72
Northridge Hills [0]		9.27	7.65	0.17	118.503°W	34.269°N	19.17	1.66
Sierra Madre (San Fernando) [2]		12.50	7.04	0.57	118.488°W	34.296°N	20.60	1.31
Northridge [3]		16.19	7.27	0.62	118.500°W	34.334°N	11.83	1.08
UC33brAvg_FM31	System							25.13
Santa Susana East (connector) [0]		12.53	6.81	0.70	118.499°W	34.314°N	13.73	4.01
Mission Hills 2011 [1]		10.42	6.51	0.82	118.546°W	34.283°N	354.98	2.50
San Andreas (Mojave S) [3]		52.49	8.05	1.62	118.305°W	34.623°N	23.76	1.79
Compton [4]		13.48	7.40	0.40	118.608°W	34.022°N	199.43	1.72
Northridge [3]		16.19	7.20	0.65	118.500°W	34.334°N	11.83	1.66
Northridge Hills [0]		9.27	7.65	0.17	118.503°W	34.269°N	19.17	1.64
Hollywood [2]		15.60	7.28	0.75	118.422°W	34.084°N	138.65	1.64
Sierra Madre (San Fernando) [2]		12.50	7.50	0.37	118.488°W	34.296°N	20.60	1.43
Santa Susana alt 1 [0]		13.40	7.53	0.44	118.544°W	34.310°N	356.92	1.39
UC33brAvg_FM31 (opt)	Grid							23.34
PointSourceFinite: -118.536, 34.222		6.13	5.64	0.88	118.536°W	34.222°N	0.00	3.34
PointSourceFinite: -118.536, 34.222		6.13	5.64	0.88	118.536°W	34.222°N	0.00	3.34
PointSourceFinite: -118.536, 34.240		7.17	5.76	0.96	118.536°W	34.240°N	0.00	2.57
PointSourceFinite: -118.536, 34.240		7.17	5.76	0.96	118.536°W	34.240°N	0.00	2.57
PointSourceFinite: -118.536, 34.303		12.70	5.74	1.48	118.536°W	34.303°N	0.00	1.98
PointSourceFinite: -118.536, 34.303		12.70	5.74	1.48	118.536°W	34.303°N	0.00	1.98
UC33brAvg_FM32 (opt)	Grid							23.14
PointSourceFinite: -118.536, 34.222		6.13	5.64	0.88	118.536°W	34.222°N	0.00	3.29
PointSourceFinite: -118.536, 34.222		6.13	5.64	0.88	118.536°W	34.222°N	0.00	3.29
PointSourceFinite: -118.536, 34.240		7.18	5.76	0.96	118.536°W	34.240°N	0.00	2.57
PointSourceFinite: -118.536, 34.240		7.18	5.76	0.96	118.536°W	34.240°N	0.00	2.57
PointSourceFinite: -118.536, 34.303		12.75	5.73	1.50	118.536°W	34.303°N	0.00	1.91
PointSourceFinite: -118.536, 34.303		12.75	5.73	1.50	118.536°W	34.303°N	0.00	1.91

Hazards by Location

Search Information



Address: 6616 Reseda Blvd, Reseda, CA 91335, USA

Coordinates: 34.1906215, -118.5358137

Elevation: 733 ft

Timestamp: 2020-03-16T17:47:55.458Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D-default

Basic Parameters

Name	Value	Description
S_S	1.901	MCE_R ground motion (period=0.2s)
S_1	0.648	MCE_R ground motion (period=1.0s)
S_{MS}	2.281	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.521	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.926	Coefficient of risk (0.2s)
CR_1	0.911	Coefficient of risk (1.0s)
PGA	0.778	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.933	Site modified peak ground acceleration

T_L	8	Long-period transition period (s)
SsRT	1.928	Probabilistic risk-targeted ground motion (0.2s)
SsUH	2.082	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.901	Factored deterministic acceleration value (0.2s)
S1RT	0.682	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.748	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.648	Factored deterministic acceleration value (1.0s)
PGAd	0.778	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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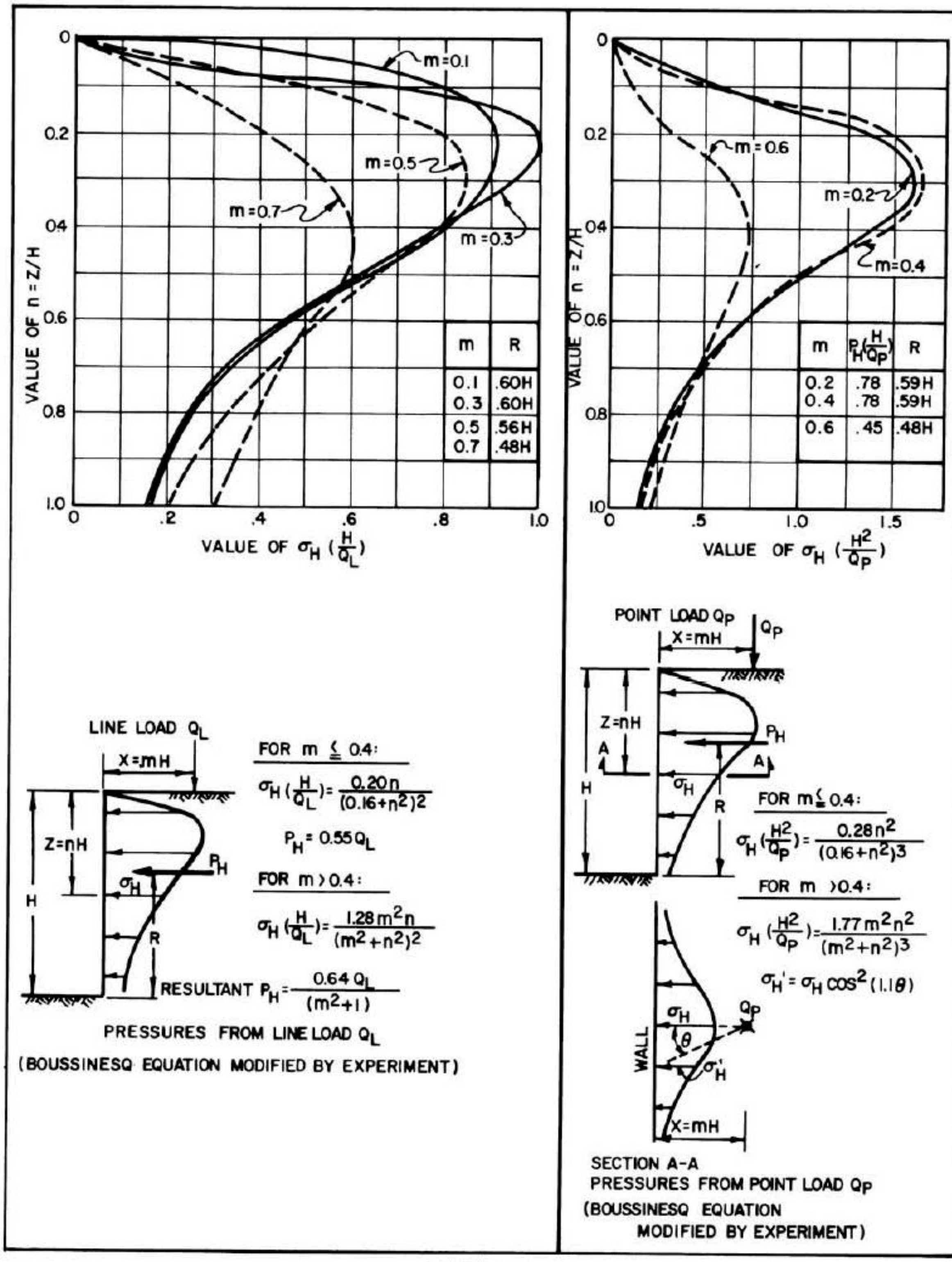

FIGURE 11
Horizontal Pressures on Rigid Wall from Surface Load

7.2-74

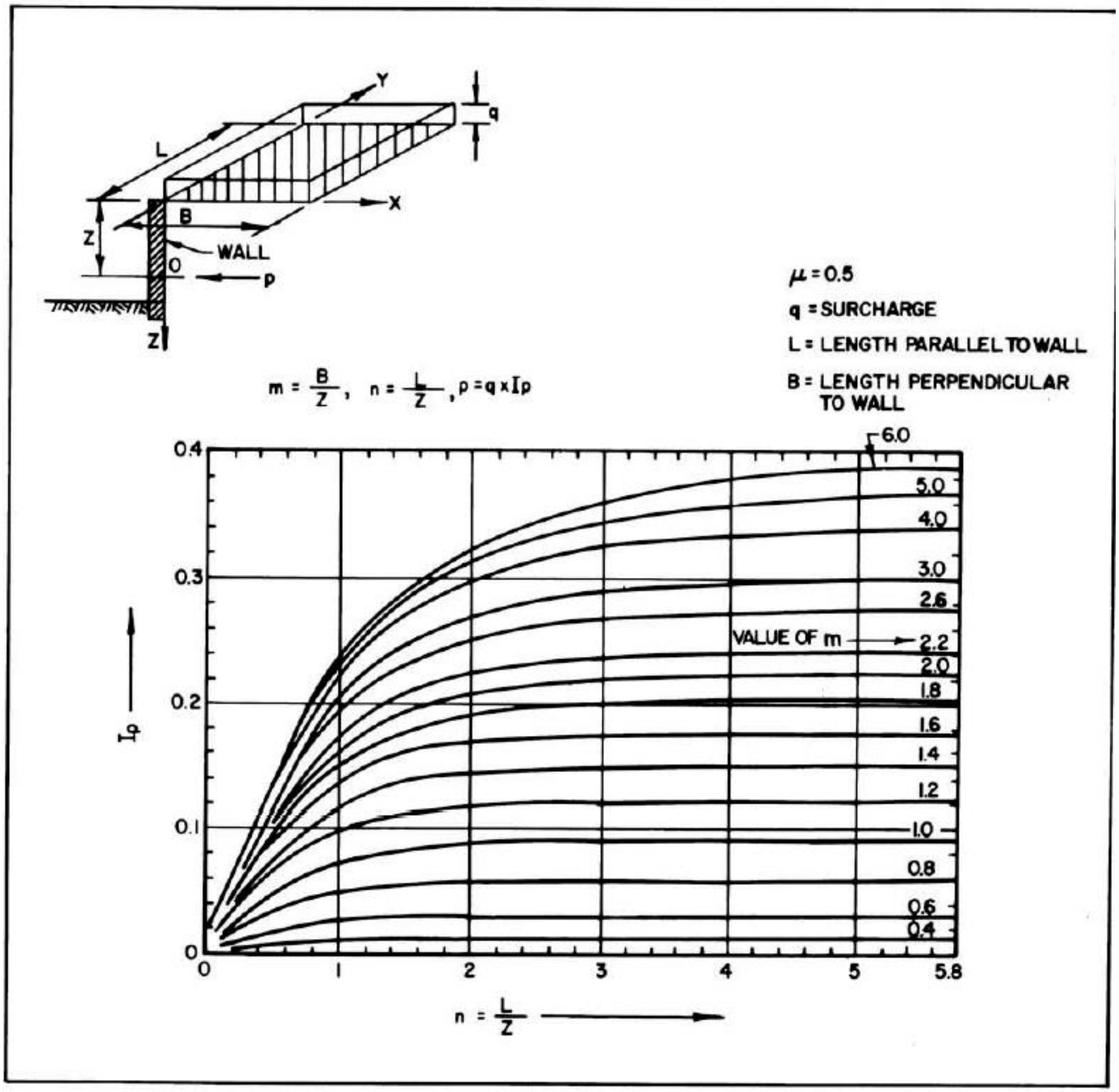

FIGURE 12
Lateral Pressure on an Unyielding Wall due to
Uniform Rectangular Surface Load

APPENDIX D

REFERENCES

1. Abramson, Lee W., et. al., Slope Stability and Stabilization Methods. Wiley & Sons, New York: 1996.
2. Bowen, Robert, Grouting in Engineering Practice - 2nd Edition. Applied Science Publishers, Ltd., New York: 1981.
3. Bowles, Joseph, E., Foundation Analysis and Design. McGraw-Hill, New York: 1988.
4. California Department of Conservation, Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada.
5. California Department of Conservation, Division of Mines and Geology, March 25, 1999, State of California Seismic Hazard Zones Map of the Canoga Park Quadrangle.
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