PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed Apartment Building

1600 West Commonwealth Avenue
Fullerton, CA

for

Meta Housing Corporation
11150 West Olympic Boulevard
Los Angeles, CA 90064

Project 22-02182

August 23, 2022

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

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INTRODUCTION

This report presents the results of a Preliminary Geotechnical Engineering Investigation on a portion of the subject property. The purpose of this investigation has been to ascertain the subsurface conditions pertaining to the proposed project. The work performed for the project included reconnaissance mapping, description of earth materials, obtaining representative samples of earth materials, laboratory testing, engineering analyses, and preparation of this report. Results of the project include findings, conclusions, and appropriate recommendations.

SCOPE

The scope of this investigation included the following:

- Review of preliminary plans by the client.
- Review of four borings. Explorations were backfilled with the excavated materials but not compacted.
- Preparation of the enclosed Plot Map, (see Appendix I).
- Sampling of representative earth materials, laboratory testing, and engineering analyses (see Appendix II).
- Review of referenced materials (see Appendix V).
- Presentation of findings, conclusions, and recommendations for the proposed project.

Hahn & Associates, Inc. prepared the topographic base map utilized in this investigation. Preliminary building plans were prepared by studioeleven and incorporated onto the base map for this investigation.

The scope of this investigation is limited to the project area explored as depicted on the Plot Map. This report has not been prepared for use by other parties or for purposes other than the proposed project. GeoConcepts, 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.

PROPOSED DEVELOPMENT

It is our understanding that the site will be developed with a three story at grade apartment building. Anticipated foundations will range from 4 to 5 kips per lineal foot and 100-200 kips for column foundations. The proposed development is depicted on the enclosed Plot Map.

Grading will consist of conventional cut and fill methods. Final plans have not been prepared and await the conclusions and recommendations of this investigation. These plans should be reviewed by GeoConcepts, Inc. to ensure that our recommendations have been followed.

SITE DESCRIPTION

Location and Description

Access to the property is via Commonwealth Avenue from Basque Avenue (see Location Map in Appendix I). The site is developed with a parking area and is otherwise vacant and generally unimproved.

The pad has a light growth of vegetation consisting of grasses, lawn areas, shrubs and trees.

Adjacent sites are developed with a gas station and parking area to the east, bounded by Commonwealth Avenue to the north, and a rail line to the south and west. Adjacent structures to the east are greater than 20 feet from the property line.

Drainage

Surface water at the site consists of direct precipitation onto the property. Much of this water drains as sheet flow down descending slopes to low-lying areas, offsite, and/or to the street. No area drains and/or subdrain outlet pipes were observed on the property.

Groundwater

The subsurface exploration encountered groundwater at a depth of 42 feet. 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 about 20 feet below the surface. Seasonal fluctuations of groundwater levels may occur by varying amounts of rainfall, irrigation and recharge.

FIELD EXPLORATION

The scope of the field exploration was developed based on the preliminary plans of the proposed development available at the time of the exploration and was limited to the area of the proposed development. The locations of the explorations are depicted on the Plot Map.

The field exploration of the site was conducted on July 15, 2022. The geotechnical conditions were mapped by a representative of this office (refer to Exploration Logs). Subsurface exploration was performed by drill rig into the underlying earth materials. Explorations were excavated to a maximum depth of 50 feet. All explorations were backfilled and tamped upon completion of down-hole observation. However, some settlement within exploration areas should be anticipated.

Detailed descriptions of the earth materials encountered during the field exploration are provided in the Boring Logs in Appendix I.

Undisturbed and bulk samples representative of the earth materials were obtained and transported to our laboratory. Undisturbed Modified California (MC) samples and Standard Penetration Test (SPT) samples were obtained within the explorations through the use of a thin-walled steel sampler with successive blows of a 140 pound drop hammer dropped thirty inches

(30"). MC samples were retained in brass rings of two and one-half inches (2½") in diameter and one inch (1") in height. The samples were transported in moisture tight containers. The results of the laboratory testing and a summary of the test procedures are included within Appendix II.

SUMMARY OF FINDINGS

Stratigraphy

The site is underlain by Quaternary (Q) earth materials and artificial fill. The earth materials encountered on the subject property are briefly described below. Approximate depths and more detailed descriptions are given in the enclosed Exploration Logs (see Appendix I).

Artificial Fill (Af)

Artificial fill was encountered on the subject site. Fill was encountered in all of the borings ranging from (0.25) to (0.33) feet in thickness. Contact between the fill and the underlying soil was exposed within the exploratory boring. Fill generally consists of sand with abundant rock fragments.

Quaternary Alluvium (Qal)

Alluvial deposits occupy the site. Alluvium is weathered bedrock material and sediments that have been eroded from natural slopes and deposited in generally flat lying areas. Alluvium primarily consists of medium to dark brown, moderately firm to stiff, silty sand to sandy silt. These deposits were encountered within all the exploratory borings.

Excavation Characteristics

Subsurface exploration was performed through the use of hollow-stem drill rig excavating into generally fill and alluvium. Due to the nature of hollow stem drilling, observation of the caving potential of the soil is not possible. Excavation difficulty is considered normal within the earth materials encountered and should not be limited to consideration of rippability of the earth material. Cohesionless sandy material, although easy to remove, may be subject to sloughing and caving. Therefore difficulty may be encountered maintaining an open excavation. Fine grained materials such as clays and silts may increase in density with depth due to overburden pressure. Thus, difficulty excavating into the material may increase with depth.

Landslides

Landslides are a mass wasting phenomenon in mountainous and hillside areas which include a wide range of movements. In Southern California common slope movements include shallow surficial slumps and flows, deep-seated rotational and translational bedrock failures, and rock falls. Landslides occur when the stability of the slopes change to an unstable condition resulting from a number of factors. Common natural factors include the physical and/or chemical weathering of earth materials, unfavorable geologic structure relative to the slope geometry, erosion at the toe of a slope, and precipitation. These factors may be further aggravated by human activities such as excavations, removal of lateral support at the toe of a slope, surcharge at the top of a slope, clearing of vegetation, alteration of drainage, and the addition of water from irrigation and leaking pipes.

The subject site is relatively flat with very little topography which precludes the potential for landslides and/or other hazards typically associated with hillside properties.

Seismic Hazards

Seismic Effects

During an earthquake there are several primary geologic hazards such as ground rupture, ground shaking, landslides, and liquefaction that can adversely affect property, structures, and improvements. On hillside properties, the potential exists for landsliding from ground shaking which may adversely affect property, structures, and improvements. Properties near and along the coastline may potentially be affected by inundation due to tsunamis generated from a seismic event. The State of California has prepared maps that detail areas which may require assessment for ground rupture, landsliding and/or liquefaction. Strong ground shaking is the primary hazard that causes damage from earthquakes and these areas have been zoned with a high level of seismic shaking hazard. The historical earthquake record in Southern California is less than 200 years; therefore, potential damage from a seismic event is not limited areas that have experienced damage in the past. Based on the above discussion, earthquake insurance with building code upgrades is suggested.

Although all of Southern California is within a seismically active region, some areas have a higher potential for seismic damage than others. The current scientific technology does not provide for accurate prediction of the time, location, or magnitude of an earthquake event.

It should be understood that the following discussion is an evaluation of risk and degree of potential damage to a structure if a fault were to rupture on or near the site and does not imply that a fault may or may not be present beneath the site. An assessment of damage to the structure is based on the Modified Mercalli Intensity Scale which is correlated to observed damage from seismic events. Intensity/damage associated with an earthquake is not directly correlated to magnitude. For a given magnitude of an earthquake, the intensity/damage to a structure may vary depending on the subsurface earth materials, type of fault rupture, hypocenter depth, and local building practices in effect during the construction of a structure.

An evaluation of the seismic effects on a property is designed to provide the client with rational and believable seismic data that could affect the property during the lifetime of the proposed improvements. The minimum design acceleration for a project is listed in the Building Code. It is recommended that the structural design of the proposed project be based on current design and acceleration practices of similar projects in the area. The project structural designer should review and verify all of the seismic design parameters prior to utilizing the information for the design.

Ground Rupture

Ground rupture is the result of movement from a Holocene-active fault. A fault is a fracture in the crust of the earth along which rocks on one side have moved relative to those on the other side. No known Holocene-active fault is mapped on the subject site.

Ground Shaking

Ground shaking caused by an earthquake is likely to occur at the site during the lifetime of the development due to the proximity of several Holocene-active and Pre-Holocene faults.

Generally, on a regional scale, quantitative predictions of ground motion values are linked to peak acceleration and repeatable acceleration, which are a response to earthquake magnitudes relative to the fault distance from the subject property. Southern California major earthquakes are generally the result of large-scale earth processes in which the Pacific plate slides northwestward relative to the North American plate at about 2 inches/year.

The potential for lurching, surface manifestations, landslides, and topographic related features from ground/seismic shaking can occur almost anywhere in Southern California. Proper maintenance of properties can mitigate some of the potential for these types of manifestations, but the potential cannot be completely eliminated. Many structures were built before earthquake codes were adopted; others were built according to codes formulated when less was known about the intensity of near-fault shaking. Therefore, the margin of safety is difficult to quantify.

A publicly available computer program provided by the United States Geological Survey (USGS) was utilized for the probabilistic prediction of peak horizontal ground acceleration from digitized design maps of Maximum Considered Earthquake (MCE) ground response. A summary of the seismic design parameters is provided in Appendix III. The project structural designer should verify all of the input parameters and review all of the resulting seismic design parameters prior to utilizing the information for the design.

Tsunamis & Seiches

Properties located along the coastline have a potential for inundation from a tsunami. Tsunamis are ocean waves produced by sudden water displacement resulting generally from offshore earthquakes, large submarine landslides or submarine volcanic eruptions. Once generated, a tsunami can travel thousands of miles at high speeds up to 400 miles per hour. However, the topography of the sea floor and Channel Islands may minimize the risk of a large tsunami generated from a distant offshore earthquake impacting the Southern California coast.

The 1964 Alaskan Earthquake produced sea waves of less than four feet in the Los Angeles Harbor. The 1960 Chilean Earthquake produced sea waves of about five feet at Redondo Beach. Little data exists to evaluate the potential for a local tsunami generated off the coast of Southern California. Historically, two documented tsunamis have been generated off the coast of Southern California. The 1812 Santa Barbara Earthquake was reported to generate (10) to (12) foot high sea waves at Gaviota. The 1927 Point Arguello Ms 7.3 Earthquake produced run-up heights of (5) feet at Port San Luis.

The lower threshold for tsunami development is considered to be about a magnitude of M6.5. Offshore faults and the Santa Monica faults appear capable of producing a magnitude of M6.5 earthquake and conceivably producing a sea wave. In their 2003 study, Evaluation of Tsunami Risk to Southern California Coastal Cities, Legg et al modeled tsunami propagation and run-up from a potential M7 to M7.4 magnitude earthquake on the offshore Catalina fault near Santa Catalina Island. The report concluded that run-up heights along the coast of Southern California could be on the order of 2 to 4 meters. Their stated recurrence times are on the order of a few hundred years for a large earthquake on offshore faults.

Seiches are waves with low-energy within reservoirs, lakes, and bays that are generally produced by strong earthquake shaking. The proposed project is not located near a reservoir, lake, or bay; therefore, the potential for damage to the site from a seiche is considered nil.

Earthquake Induced Landslides

The State of California has prepared Seismic Hazard Zone Reports to regionally map areas of potential increased risk of permanent ground displacement based on historic occurrence of landslide movement, local topographic expression, and geological and geotechnical subsurface conditions. The maps may not identify all areas that have potential for earthquake-induced landsliding, strong ground shaking, or other earthquake-related geologic hazards. The subject site is not located within an earthquake-induced landslide hazard zone on the State of California Seismic Hazard Map.

The subject site is relatively flat with very little topography which precludes the potential for landslides and/or other hazards typically associated with hillside properties.

Liquefaction

The State of California has prepared Seismic Hazard Zone Reports to regionally map areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacement. The maps may not identify all areas that have potential for liquefaction, strong ground shaking, and other earthquake and geologic hazards. The subject site is located within a liquefaction hazard zone on the State of California Seismic Hazard Zone Map.

Liquefaction is a process by which sediments below the water table temporarily lose strength and behave as a viscous liquid rather than a solid. The types of sediments most susceptible are clay-free deposits of sand and silts; occasionally gravel liquefies. Liquefaction can occur when seismic waves, primarily shear waves, pass through saturated granular layers distorting the granular structure, and causing loosely packed groups of particles to collapse. These collapses increase the pore-water pressure between grains if drainage cannot occur. If the pore-water pressure rises to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid.

In the liquefied condition, soil may deform with little shear resistance; deformations large enough to cause damage to buildings and other structures are called ground failures. The ease with which a soil can be liquefied depends primarily on the looseness of the material, the depth, thickness and areal extent of the liquefied layer, the ground slope and the distribution of loads applied by buildings and other structures.

Liquefaction induced ground deformations (detailed below) will have an effect on the proposed and existing development that can result in significant structural damage, collapse or partial collapse of a structure, especially if there is significant differential settlement or lateral spreading between adjacent structural elements. Even without collapse, significant settlement or lateral spreading could result in significant structural damage including, but not limited to, blocked doors and windows that could trap occupants.

To quantify the potential for liquefaction at the subject site two borings were 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 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 1.3. 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
7.3 (2% probability of exceedance in 50 years)	0.734 (PGA_m)	42 feet	20 feet

Based on Bray and Sancio's 2006 publication regarding liquefaction potential of fine grained soils, layers that with a saturated water content to Liquid Limit ratio less than 80% and/or a Plasticity Index higher than 18 are not susceptible to liquefaction. The table below presents which layers have been excluded from the liquefaction analysis based on the above guidelines.

<u>Boring 1</u>

Depth of Layer (ft)	Fines Content (%)	Saturated W_c (%)	Liquid Limit	W_c/LL	Plasticity Index
10.0 – 15.0	66	20.7	38	0.55	16
20 – 25.0	70	18.6	25	0.74	3

<u>Boring 2</u>

Depth of Layer (ft)	Fines Content (%)	Saturated W_c (%)	Liquid Limit	W_c/LL	Plasticity Index
7.5 – 15.0	66	25.3	35	0.72	10
20.0 – 27.5	76	22.3	29	0.77	4
35.0 – 45.0	61	18.1	24	0.75	6

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:

Total Settlement (in)	Differential Settlement (in)
3.19	1.60

Surface Manifestations

The determination of whether surface manifestation of liquefaction (such as sand boils, ground fissures etc.) will occur during earthquake shaking at a level-ground site can be made using the method outlined by Ishihara (1985). It is emphasized that settlement may occur, even with the absence of surface manifestation. Youd and Garris (1994 and 1995) evaluated the Ishihara method and concluded that the method is not appropriate for level ground sites subject to lateral spreading and/or ground oscillation.

Based upon the depth to groundwater, surface manifestations of liquefaction should not pose any significant hazard to the proposed development provided the recommendations contained within this report are followed and maintained.

Lateral Spreads

Whereas the potential for flow slides may exist at a building site, the degradation in undrained shear resistance arising from liquefaction may lead to limited lateral spreads (of the order of feet or less) induced by earthquake inertial loading. Such spreads can occur on gently sloping ground or where nearby drainage or stream channels can lead to static shear stress biases on essentially horizontal ground (Youd, 1995). At larger cyclic shear strains, the effects of dilation may significantly increase post liquefaction undrained shear resistance. However, incremental permanent deformations will still accumulate during portions of the earthquake load cycles when low residual resistance is available. Such low resistance will continue even while large permanent shear deformations accumulate through a ratcheting effect. Such effects have recently been demonstrated in centrifuge tests to study liquefaction induced lateral spreads, as described by Balakrishnan et al. (1998). Once earthquake loading has ceased, the effects of dilation under static loading can mitigate the potential for a flow slide.

It is clear from past earthquakes that damage to structures can be severe, if permanent ground displacements on the order of several feet occur. However, during the Northridge earthquake significant damage to building structures (floor slab and wall cracks) occurred with less than one (1) foot of lateral spread. The complexities of post-liquefaction behavior of soils noted above, coupled with the additional complexities of potential pore water pressure redistribution effects and the nature of earthquake loading on the sliding mass, lead to difficulties in providing specific guidelines for lateral spread evaluations.

Based upon the depth to groundwater, liquefaction lateral spreads should not pose any significant hazard to the proposed development.

Seismically Induced Settlements

Seismic settlement occurs when cohesionless soils densify as result of ground shaking. Typically seismically induced settlement is greatest in loose cohesionless sands. Lee and Albaisa (1974) and Yoshimi (1975) studied the volumetric strains (or settlements) in saturated sands due to dissipation of excess pore pressures generated in saturated granular soils by the cyclic ground motions. The volumetric strain, in the absence of lateral flow or spreading, results in settlement. Liquefaction-induced settlement could result in collapse or partial collapse of a structure, especially if there is significant differential settlement between adjacent structural elements. Even without collapse, significant settlement could result in blocked doors and windows that could trap occupants.

Based upon the liquefaction analysis, liquefaction induced settlement is estimated to be 3.19 inch and differential settlement of 1.60 inch.

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 3.19 inch and differential settlement of 1.60 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 SITE CLASS based on California Building Code is D.

5. Based upon field observations, laboratory testing and analysis, the alluvium found in the exploratory borings should possess sufficient strength to support the compacted fill blanket for the proposed three story at grade apartment building.

RECOMMENDATIONS

Specific

1. To create a uniform building pad for the proposed three story at grade apartment building, the existing fill and soil should be removed to competent alluvium and replaced as compacted fill. In addition, the proposed removals should extend a minimum of four feet below the proposed foundations. Grading should be performed as outlined the Grading and Earthwork section below.
2. The proposed three story at grade apartment building should be supported on foundations embedded into compacted fill. Foundations should be designed as outlined the Foundations section below.
3. The soils chemistry results should be incorporated into the design of the proposed project.
4. 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 remedial grading and foundation excavations.

Remedial grading is recommended within the building areas in order to remove the existing fill and upper portion of the alluvial soils. Based on the conditions encountered in the explorations the recommended removals are anticipated to depths of about six feet from the existing grade. The over-excavation should extend a minimum of four feet beyond the building perimeters, and to an extent equal to the depth of fill below the new foundations. If the proposed structure incorporates exterior columns (such as for an overhang) the over-excavation should also encompass these areas.

Following the completion of the over-excavation, the subgrade soils should be evaluated by the project geotechnical engineer to verify their suitability to support the structural fill as well as to support the foundation loads of the proposed development. This evaluation may include probing and proof-rolling to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or dry, loose, porous or otherwise unsuitable materials are encountered at the base of the over-excavation.

Flatland Grading

1. Prior to commencement of work, a pre-grading meeting shall be held. Participants at this meeting will consist of the contractor, the owner or his representative, and the soils engineer. The purpose of the meeting is to avoid misunderstanding of the recommendations set forth in this report that might cause delays in the project.
2. Prior to placement of fill, all vegetation, rubbish, and other deleterious material should be disposed of offsite. The proposed structures should be staked out in the field by a surveyor. This staking should, as a minimum, include areas for overexcavation, toes of slopes, tops of cuts, setbacks, and easements. All staking shall be offset from the proposed grading area at least five feet (5'). Line and grade verification is not provided by GeoConcepts, Inc.
3. The natural ground, that is determined to be satisfactory for the support of the filled ground, shall then be scarified to a depth of at least six inches (6") and moistened as required. The scarified ground should be compacted to at least 90 percent of the maximum laboratory density (ASTM D 1557).
4. The fill soils shall consist of materials approved by the project Soils Engineer or his representative. These materials may be obtained from the excavation areas and any other approved sources, and by blending soils from one or more sources. The material used shall be free from organic vegetable matter and other deleterious substances, and shall not contain rocks greater than eight inches (8") in diameter nor of a quantity sufficient to make compaction difficult.
5. The approved fill material shall be placed in approximately level layers six inches (6") thick, and moistened as required. Each layer shall be thoroughly mixed to attain uniformity of moisture in each layer.

When the moisture content is less than the optimum moisture content, as specified by the Soils Engineer, water shall be added and thoroughly mixed in until the moisture content is a minimum of the optimum moisture content to (3) percent above the optimum moisture content.

When the moisture content of the fill is (3) percent or more above the optimum moisture content as specified by the Soils Engineer, the fill material shall be aerated by scarifying or shall be blended with additional materials and thoroughly mixed until the moisture content is within (3) percent above the optimum moisture content.

Each layer of fill material shall be compacted to a minimum of (90) percent of the maximum dry density as determined by ASTM D 1557, using approved compaction equipment. Where cohesionless soil having less than (15) percent finer than (0.005) millimeters is used for fill, the fill material shall be compacted to a minimum of (95) percent of the maximum dry density.

6. Review of the fill placement should be provided by the Soils Engineer or his representative during the progress of grading. In general, density tests (ASTM D 1556) and (*ASTM D 2922 & 3017*) will be made at intervals not exceeding two feet (2') of fill height or every 500 cubic yards of fill placed.

7. During the inclement part of the year, or during periods when rain is threatening, all fill that has been spread and awaits compaction shall be compacted before stopping work for the day or before stopping because of inclement weather. These fills, once compacted, shall have the surfaces sloped to drain to one area where water may be removed.

   Work may start again, after the rainy period, once the site has been reviewed by the Soils Engineer and he has given his authorization to resume. Loose materials not compacted prior to the rain shall be removed and aerated so that the moisture content of these fills will be within (3) percent of the optimum moisture content.

   Surface materials previously compacted before the rain, shall be scarified, brought to the proper moisture content, and re-compacted prior to placing additional fill, if deemed necessary by the Soils Engineer.

8. Review of geotechnical data available for the local vicinity of the site indicates that septic tanks, seepage pits, or leach fields may be encountered during site grading. If encountered, these should be drained of effluent or drilled out if they have been backfilled. The cleaned-out area should be inspected by the soils engineer and governing inspector prior to backfill. The pool may be filled with approved compacted fill, lean concrete, or gravel. Whichever backfill material is selected, at least five feet (5') of approved manmade fill, placed at 90 percent relative compaction should cap the pool.

Foundations

It is recommended that the proposed structure be founded into compacted fill.

Conventional Foundations

The minimum continuous footing size is (18) inches wide and (24) inches deep into the compacted fill, measured from the lowest adjacent grade. Continuous footings may be proportioned, using a bearing value of (2000) pounds per square foot. Column footings placed into the compacted fill may be proportioned, using a bearing value of (2500) pounds per square foot, and should be a minimum of (2) feet in width and (24) inches deep, below the lowest adjacent grade.

All continuous footings shall be reinforced with a minimum of (4) #(5) bars, two placed near the top and two near the bottom. Reinforcing recommendations are minimums and may be revised by the structural engineer.

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.

Lateral loads may be resisted by friction at the base of the foundations and by passive resistance within the compacted fill. A coefficient of friction of (0.4) may be used between the foundations and the compacted fill. The passive resistance may be assumed to act as a fluid with a density of (300) pounds per square foot, with a maximum earth pressure of (3000) pounds per square foot. When combining passive and friction for resistance of lateral loads, the passive component should be reduced by one-third.

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.

Mat Foundation Recommendations

The mat foundation may be proportioned using an average bearing value of (2,500) pounds per square foot, and the maximum allowable bearing capacity should not exceed (4,000) pounds per square foot. The mat foundation structural design should be done by the project structural engineer.

The coefficient of static vertical subgrade reaction is defined as:

Granular Soil:

$$K_b = K_{v1} * \left[\frac{m + 0.5}{1.5m}\right] * \left[\frac{B + 1}{2B}\right]^2$$

K_b: Coefficient of static vertical subgrade reaction

K_{v1}: Normalized subgrade reaction coefficient (namely, corresponding to a 1 foot square bearing plate), estimated at 125 pounds per cubic inch (pci) for engineered fill subgrade. It should be noted that this value applies to dry or moist materials, with groundwater at a depth of at least 1.5B below the base of the footing. If groundwater is at the base of the footing, use $K_{v1}/2$ to calculate settlements.

B: Width of the mat foundation measured in feet.

m: Ratio of length over width of a rectangular footing.

The mat foundation structural design should be done by the project structural engineer.

Lateral loads may be resisted by friction at the base of the conventional foundations and by passive resistance within the compacted fill. A coefficient of friction of (0.4) may be used between the foundations and the compacted fill. The passive resistance may be assumed to act as a fluid with a density of (300) pounds per cubic foot.

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). GeoConcepts, 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.

Settlement

Settlement of the proposed three story at grade apartment building will occur. Settlement of (1/8) to (1/4) inches between walls, within 20 feet or less, of each other, and under similar loading conditions, are considered normal. Total settlement on the order of (1/2) inches should be anticipated.

Expansive Soils

Expansive soils were not encountered on the subject property. Expansive soils can be a problem, as variation in moisture content will cause a volume change in the soil. Expansive soils heave when moisture is introduced and contract as they dry. During inclement weather and/or excessive landscape watering, moisture infiltrates the soil and causes the soil to heave (expansion). When drying occurs the soils will shrink (contraction).

Repeated cycles of expansion and contraction of soils can cause pavement, concrete slabs on grade and foundations to crack. This movement can also result in misalignment of doors and windows. 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, but will not eliminate deflection and cracking and do not guarantee or warrant that cracking will not occur.

Excavations

Excavations ranging in vertical height up to six feet will be required for the remedial grading. Conventional excavation equipment may be used to make these excavations. Excavations should expose alluvium. These soils are suitable for vertical excavations up to five feet. This should be verified by the project geotechnical engineer during construction so that modifications can be made if variations in the soil occur.

Excavations located along the property line may be made by the slot-cutting method to six feet high. This method employs the use of the earth as a buttress and allows the excavation to proceed in phases. The initial excavation is made at a slope of 1:1 (h:v). Slots are cut, using the ABC method, in which all slots are of the same width. The initial slot "A" is cut eight feet in width, leaving the "B" and "C" slots to buttress the excavation. The "A" slot is backfilled; the same procedure is used for the "B" slots; then the "C" slots.

All excavations should be stabilized within 10 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 (3') of the top of cut.

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.

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.

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.

Slabs on Grade

Slabs on grade should be reinforced with minimum #4 reinforcing bars, placed at (16) inches on center each way and supported on compacted fill. 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 8 feet.

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). GeoConcepts, 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 (16) inches on center each way and supported on compacted fill. 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.

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 Preliminary Geotechnical Engineering Investigation by GeoConcepts, Inc. is a part of the plans.

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

3. The project geotechnical engineer must review all grading.

4. The project geotechnical engineer shall review all foundations.

Construction Review

Reviews will be required to verify all geotechnical work. It is required that all footing excavations, seepage pits, and grading be reviewed by this office. This office should be notified at least **two working days** in advance of any field reviews so that staff personnel may be made available.

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.

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. If any reader requires additional information or has questions regarding this report, GeoConcepts, Inc. should be contacted.

Subsurface conditions were interpreted on the basis of our field explorations and past experience. Although, 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.

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.

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

CONSTRUCTION NOTICE

Construction can be challenging. GeoConcepts, 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,
GEOCONCEPTS, INC.

Raffi Dermendjian
Project Engineer
PE C. 88261
RD/HU: 22-02182-1

Distribution: (1) Addressee

APPENDIX I

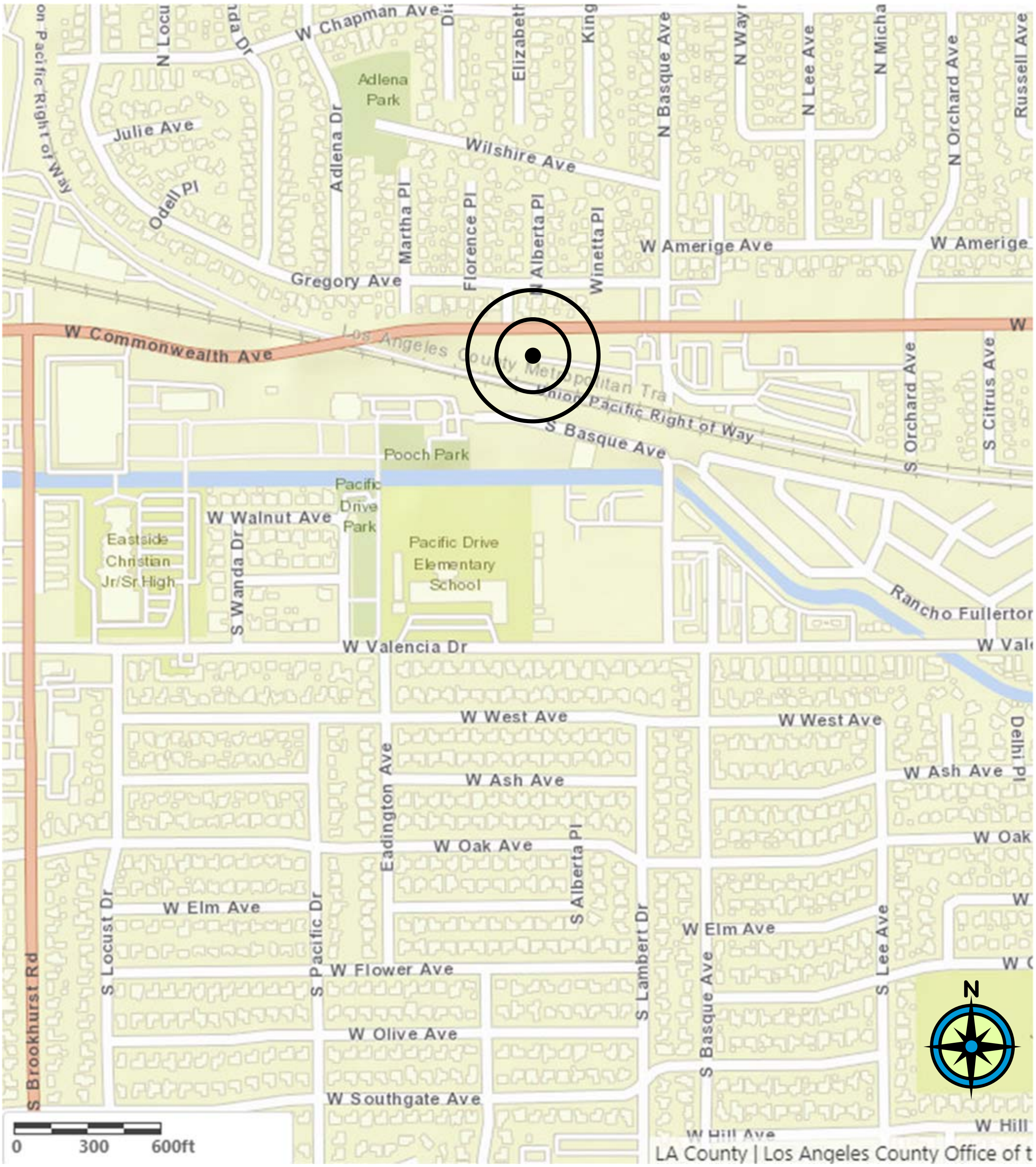
SITE INFORMATION

Location Map
Groundwater Map
USGS Fault Map
Earthquake Zone Map

Plot Map

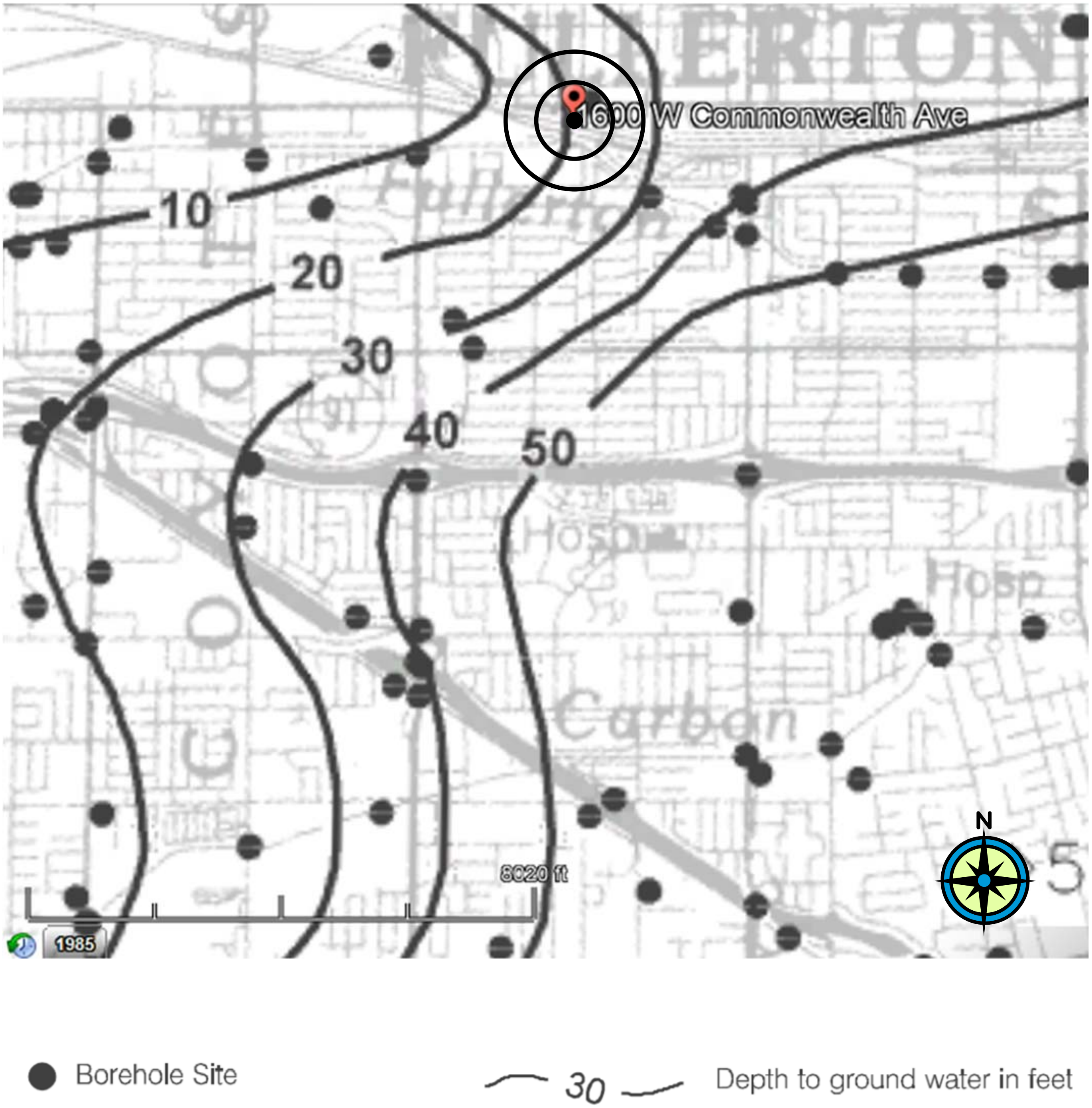
Field Exploration
Borings 1 through 4

LOCATION MAP



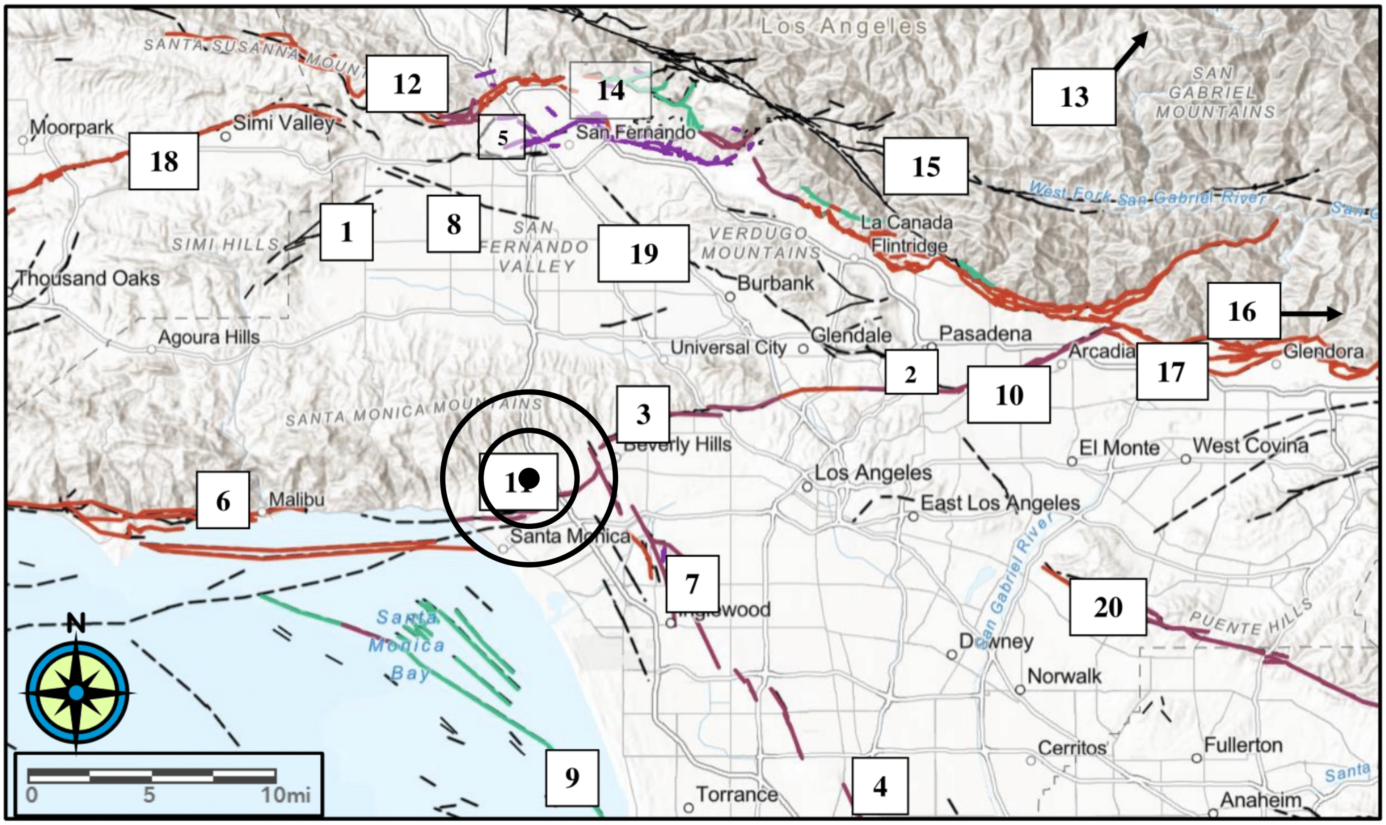
Reference:	Los Angeles County of Public Works, GIS-Net3	Scale: As Shown

GROUNDWATER MAP



Reference:	State of California Seismic Hazard Report, Fullerton Quadrangle	Scale: As Shown

USGS FAULT MAP



Historic High Magnitude Quaternary Fault Activity

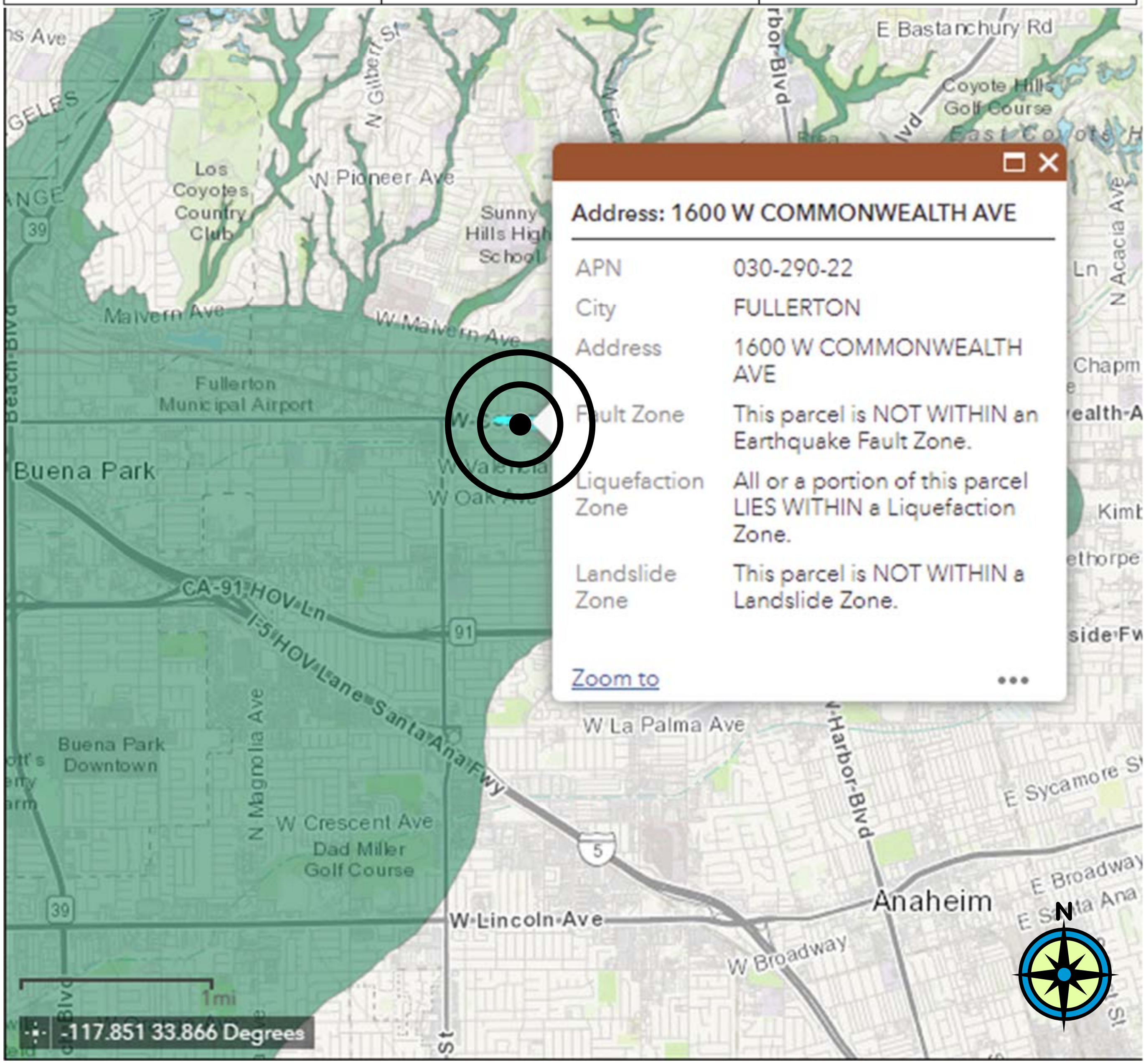
- Approximate Location
- > 1.6 million years
- > 750,000 years
- > 130,000 years
- > 15,000 years
- > 150 years
- Class B*
- Unknown

1	Chatsworth fault	11	Santa Monica fault
2	Eagle Rock fault	12	Santa Susana fault
3	Hollywood fault	13	San Andreas fault
4	Los Alamitos fault	14	San Fernando fault zone
5	Mission Hills fault	15	San Gabriel fault zone
6	Malibu Coast fault	16	San Jacinto fault
7	Newport Inglewood fault zone	17	Sierra Madre fault zone
8	Northridge Hills fault	18	Simi fault
9	Palos Verdes fault zone	19	Verdugo fault
10	Raymond fault	20	Whittier fault

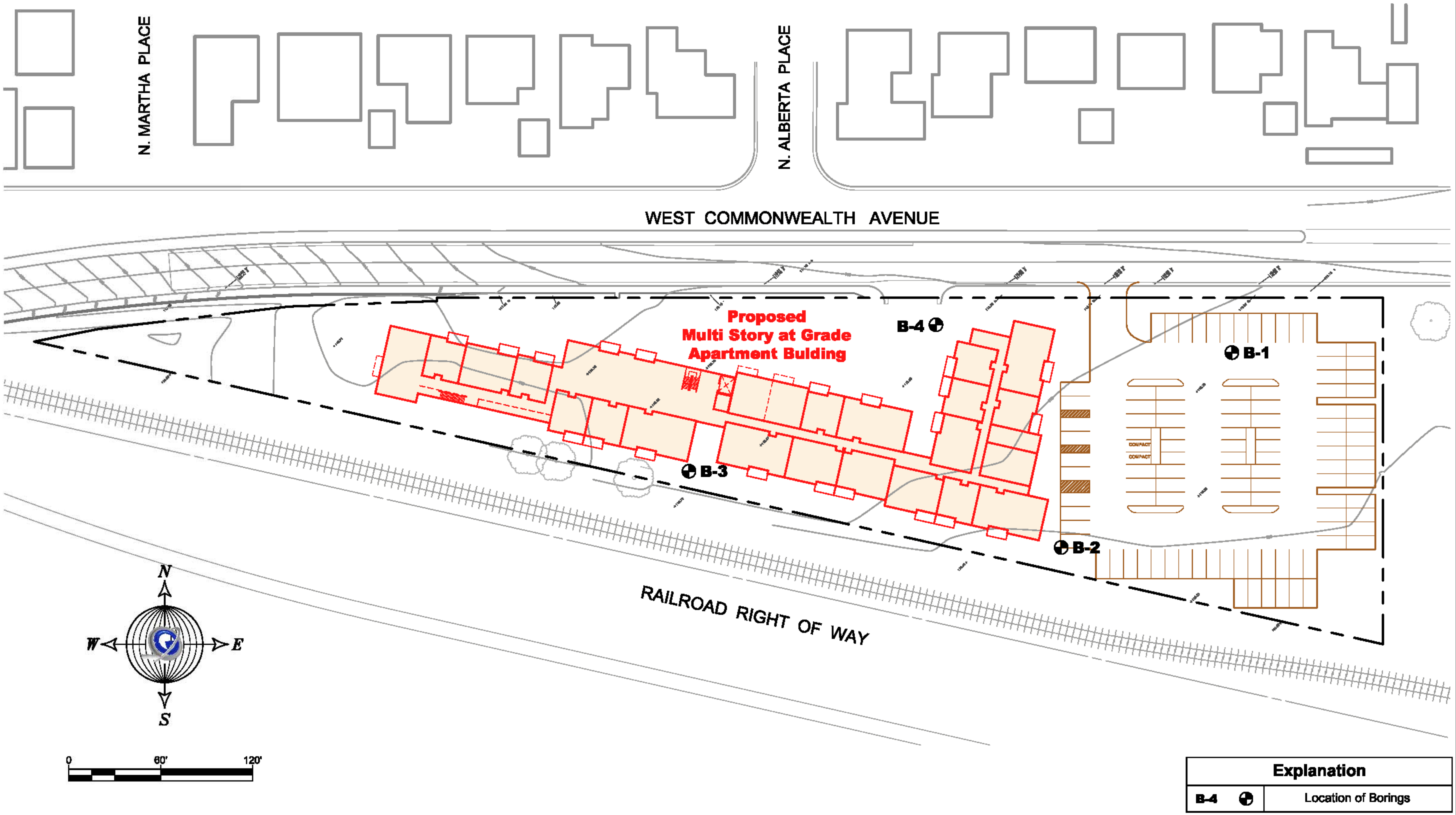
Reference:	https://www.arcgis.com/home/webmap/viewer.html Esri, USGS | Esri, HERE, Garmin, FAO, NOAA, USGS, EPA	Scale: As Shown

EARTHQUAKE ZONE MAP

Earthquake Induced Landslide Zones	Liquefaction Zones	Earthquake Fault Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resource Code Section 2693(c) would be required.	Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.	These features delineate areas where surface fault rupture previously has occurred, or where local topographic, geological, and geotechnical conditions indicate a potential for permanent ground displacements such that mitigation by avoidance as stated in Public Resources Code Section 2621.5 would be required.



Reference:	California Geological Survey, Seismic Hazard Map https://maps.conservation.ca.gov/cgs/DataViewer/index.html	Scale: As Shown



Explanation	
B-4	Location of Borings

GeoConcepts INC
Geology - Geotechnical Engineering
14428 Hamlin Street, Suite 200, Van Nuys, CA 91401
Ph (818) 994-8895 | Fax (818) 994-8599 | www.GeoConceptsInc.com

Description: **Plot Map**

Base Map Provided By: Hahn and Associates, Inc.

Project Address: 1600 West Commonwealth Avenue, Fullerton, California

Date: Aug. 2022

Scale: 1" = 60'

Job No. 22-02182

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BORING: B-1

ADDRESS: 1600 W. Commonwealth Ave — PROJECT NO.: 22-02182

DATE LOGGED: July 15, 2022 — LOGGED BY: HU

ATTITUDES: b - bedding, j - joint, s - shear, f - fault

ATTITUDES	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT
	13	102	14
			7
	16	105	14
			11
	18	108	21
			23
	16	114	44
			9
	17	112	18
			21
	18	108	36
			28
	17	112	36
			28
	18	112	64
			27
	15	108	51
			27
@ 47.5' 50 blow for 6"	15	120	50
@ 50.0' 50 blows for 5"			50

DEPTH, FT	DESCRIPTION
0.0' - 0.33'	**ARTIFICIAL FILL; Af,** sand, light gray, dry to slightly moist, stone and gravels
0.33' - 50.0'	**ALLUVIUM; Qal,**
0.33' - 10.0'	sandy silt, medium brown, slightly moist, medium grained
10.0' - 15.0'	sandy silt with minor clay, medium olive brown, slightly moist, medium grained
15.0' - 20.0'	sand with minor silt, tan to medium brown, slightly moist, medium to coarse grained
20.0' - 25.0'	silty sand to sandy silt, medium olive brown to dark gray, slightly moist to moist, medium to fine grained, slightly cohesive
25.0' - 32.5'	sand with minor silt to silty sand, medium olive brown, slightly moist to moist, medium grained, slightly cohesive
32.5' - 47.5'	sandy silt to silty sand, medium olive brown to dark gray, slightly moist to moist, medium to fine grained, dense, minor oxidation
47.5' - 50.0'	sand, light gray to medium brown, very moist to wet, coarse to very coarse grained, very dense

Total Depth: 50.0 Feet
Groundwater: 45.0 Feet
8 Inch Hollow Stem Auger with Autohammer

BORING: B-2

ADDRESS: 1600 W. Commonwealth Ave PROJECT NO.: 22-02182

DATE LOGGED: July 15, 2022 LOGGED BY: HU

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	DEPTH, FT	DESCRIPTION
					0.0' - 0.33' **ARTIFICIAL FILL; Af,** sand, light gray, dry to slightly moist, stone and gravels
			14		0.33' - 50.0' **ALLUVIUM; Qal,**
	9	108	21	5	0.33' - 7.5' sandy silt, medium brown, slightly moist, medium to fine grained, decreasing cohesion with depth, increasing density with depth
			8		
	21	100	15	10	7.5' - 15.0' clayey silt with sand, light gray to yellowish brown, slightly moist, medium to coarse grained
			8		
	15	118	44	15	15.0' - 20.0' silty sand, light brown, slightly moist, fine do medium grained
			22		
	19	105	21	20	20.0' - 32.5' sandy silt, medium brown, slightly moist, medium to fine grained
			11		@22.5' fine to very fine grained, cohesive
	21	105	18	25	@25.0' minor clay binder, medium olive brown, medium to fine grained
			10		27.5' - 35.0' silty sand, medium brown to dark brown, slightly moist, fine to medium grained
	17	108	24	30	
			41		
@ 35.0' 50 blows for 6"	16	111	50	35	35.0' - 45.0' sandy silt, medium grayish brown to olive brown, slightly moist to moist, fine grained
			25	40	@40.0' - 45.0' moist to wet, medium to very coarse grained
	14	110	41	45	45.0' - 50.0 sand, medium gray, wet, medium to coarse grained
			34	50	Total Depth: 50.0 Feet Groundwater: 42.0 Feet 8 Inch Hollow Stem Auger with Autohammer
				55	

BORING: B-3

ADDRESS: 1600 W. Commonwealth Ave — PROJECT NO.: 22-02182

DATE LOGGED: July 15, 2022 — LOGGED BY: HU

ATTITUDES: b - bedding, j - joint, s - shear, f - fault

WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	DESCRIPTION
				0	0.0' - 0.25' **ASPHALT**
					0.25' - 0.33' **ARTIFICIAL FILL; Af,** sand, light gray, dry to slightly moist, stone and gravels
17	104	16	X	5	0.33' - 30.0' **ALLUVIUM; Qal,** 0.33' - 10.0' silty sand to sandy silt with minor clay, medium to dark brown, slightly moist, medium to fine grained
14	90	16	X	10	10.0' - 15.0' silty clay to clayey silt, medium brown, slightly moist, fine grained, cohesive
15	109	46	X	15	15.0' - 30.0' silty sand to sandy silt with clay, medium gray brown to dark olive brown, slightly moist, medium to very fine grained, increasing in fines with depth
22	101	24	X	20	
24	102	17	X	25	
21	104	25	X	30	Total Depth: 30.0 Feet No Groundwater 8 Inch Hollow Stem Auger with Autohammer
				35	
				40	
				45	
				50	
				55	

BORING: B-4

ADDRESS: 1600 W. Commonwealth Ave — PROJECT NO.: 22-02182

DATE LOGGED: July 18, 2022 — LOGGED BY: HU

ATTITUDES: b - bedding, j - joint, s - shear, f - fault

ATTITUDES	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
					0		0.0' - 0.33' **ARTIFICIAL FILL; Af,** sand, light gray, dry to slightly moist, stone and gravels
							0.33' - 30.0' **ALLUVIUM; Qal,**
							0.33' - 15.0' silty sand to sandy silt with clay, medium brown to dark olive brown, slightly moist, medium grained
	23	101	21	X	5		
	19	102	13	X	10		
	6	107	44	X	15		15.0' - 20.0' sand, light gray to tan brown, slightly moist, medium to coarse grained
	15	110	60	X	20		20.0' - 30.0' sandy silt to silty sand with clay, medium olive brown to tan and gray brown, slightly moist to moist, medium to fine grained
	4	117	35	X	25		
	12	104	15	X	30		Total Depth: 30.0 Feet No Groundwater 8 Inch Hollow Stem Auger with Autohammer
					35		
					40		
					45		
					50		
					55		

APPENDIX II

LABORATORY TESTING

Laboratory Procedures

Laboratory Recapitulation 1
Laboratory Recapitulation 2

Figures S.1 through S.3
Figures C.1 through C.11
Figures SV.1 through SV.4
Figure ATT. 1

LABORATORY PROCEDURES

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.

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 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.

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.

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.

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)

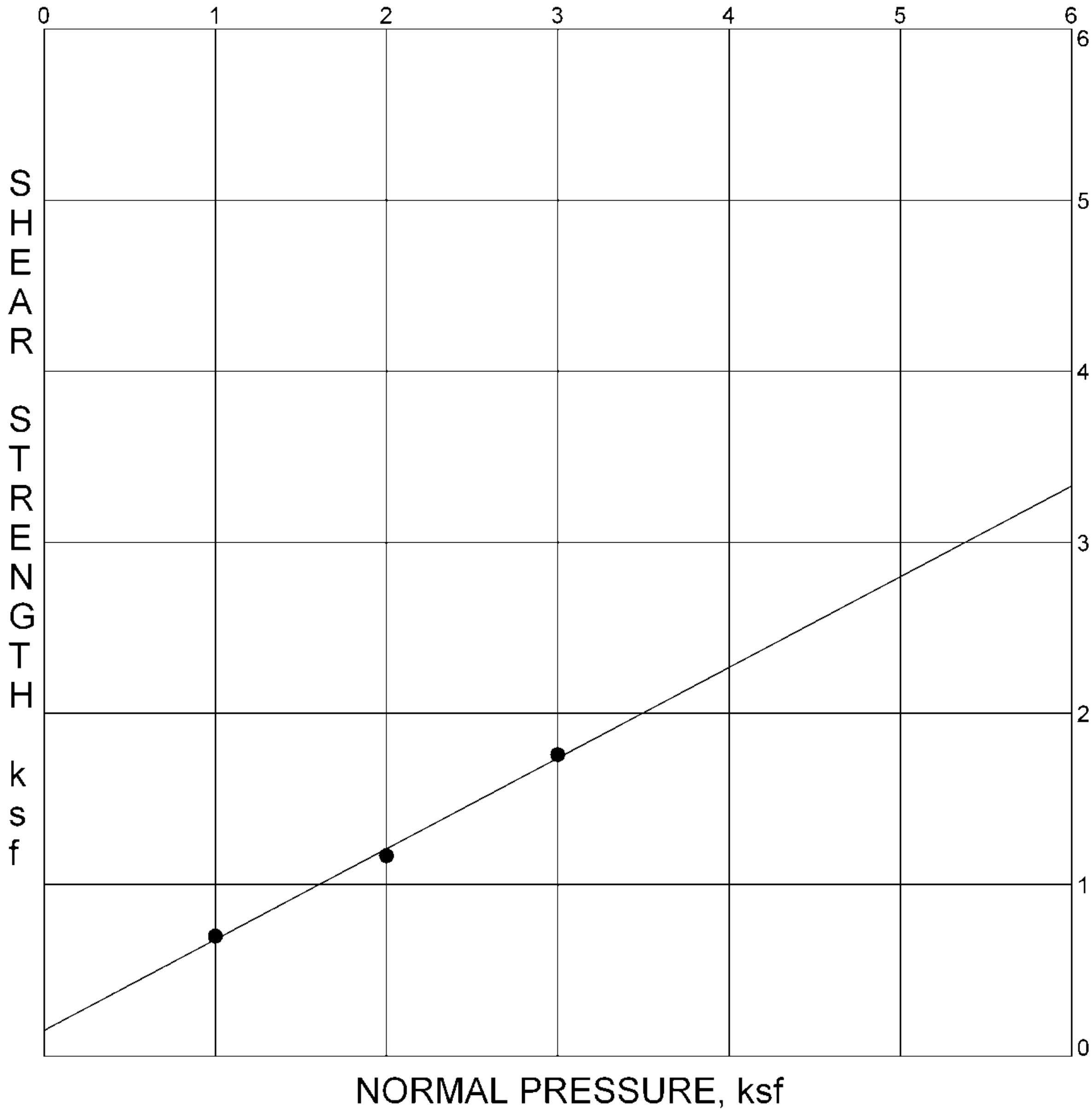
A sample of dry soil is mixed with distilled water and allowed to stand overnight. The top aliquot is mixed with distilled water and a conditioning agent. The solution is then placed in a photometer and the value recorded. The process is repeated with the addition of barium chloride. The sulfate content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

LABORATORY RECAPITULATION 1 PROJECT: 1600 W. Commonwealth Ave PROJECT NO.: 22-02182						
Exploration	Depth (ft)	Material	Dry Density In Situ (P.C.F.)	Moisture Content (%)	Cohesion (K.S.F.)	Friction Angle (degree)
B-1	1	Qal				
B-1	2.5	Qal	101.8	13.3	0.15	28
B-1	5	Qal				
B-1	7.5	Qal	104.6	16.4		
B-1	10	Qal				
B-1	12.5	Qal	107.9	18.4		
B-1	15	Qal				
B-1	17.5	Qal	113.8	15.9		
B-1	20	Qal				
B-1	22.5	Qal	112.4	16.9		
B-1	25	Qal				
B-1	27.5	Qal	108.4	18.2		
B-1	30	Qal				
B-1	32.5	Qal	111.8	16.9		
B-1	35	Qal				
B-1	37.5	Qal	112.2	18.1		
B-1	40	Qal				
B-1	42.5	Qal	108.1	15.2		
B-1	45	Qal				
B-1	47.5	Qal	120.1	14.6		
B-1	50	Qal				
B-2	2	Qal				
B-2	2.5	Qal				
B-2	5	Qal	107.6	9.5	0.15	29
B-2	7.5	Qal				
B-2	10	Qal	99.6	20.7		
B-2	12.5	Qal				
B-2	15	Qal	117.6	15.2		
B-2	17.5	Qal				
B-2	20	Qal	105.4	19.3		
B-2	22.5	Qal				
B-2	25	Qal	104.6	21.1		
B-2	27.5	Qal				
B-2	30	Qal	108.3	16.9		
B-2	32.5	Qal				
B-2	35	Qal	113.0	16.4		
B-2	40	Qal				
B-2	45	Qal	110.1	14.1		

B-2	50	Qal				
B-3	3	Qal				
B-3	5	Qal	104	16.9	0.15	28
B-3	10	Qal	90.2	14.1		
B-3	15	Qal	109.4	14.6		
B-3	20	Qal	101.1	22		
B-3	25	Qal	101.7	24.2		
B-3	30	Qal	104.3	21.3		
B-4	5	Qal	101.3	23.5		
B-4	10	Qal	102.2	18.9		
B-4	15	Qal	107.2	6.2		
B-4	20	Qal	110.1	15.1		
B-4	25	Qal	117.3	4.4		
B-4	30	Qal	104.1	12		

LABORATORY RECAPITULATION 2 PROJECT: 1600 W. Commonwealth Ave PROJECT NO.: 22-02182						
Exploration	**Depth (ft)**	**pH**	**As-Is Soil Resistivity (ohm-cm)**	**Minimum Soil Resistivity (ohm-cm)**	**Chloride (%)**	**Sulphate (%)**
B-1	1	7.49	820000	19000	0.004	0.024
B-3	3	7.42	32000	2900	0.005	0.001

PROJECT LOCATION: 1600 W. Commonwealth Ave	PROJECT NO.: 22-02182
SAMPLE LOCATION: B-1 @ 2.5	DESCRIPTION: Qal



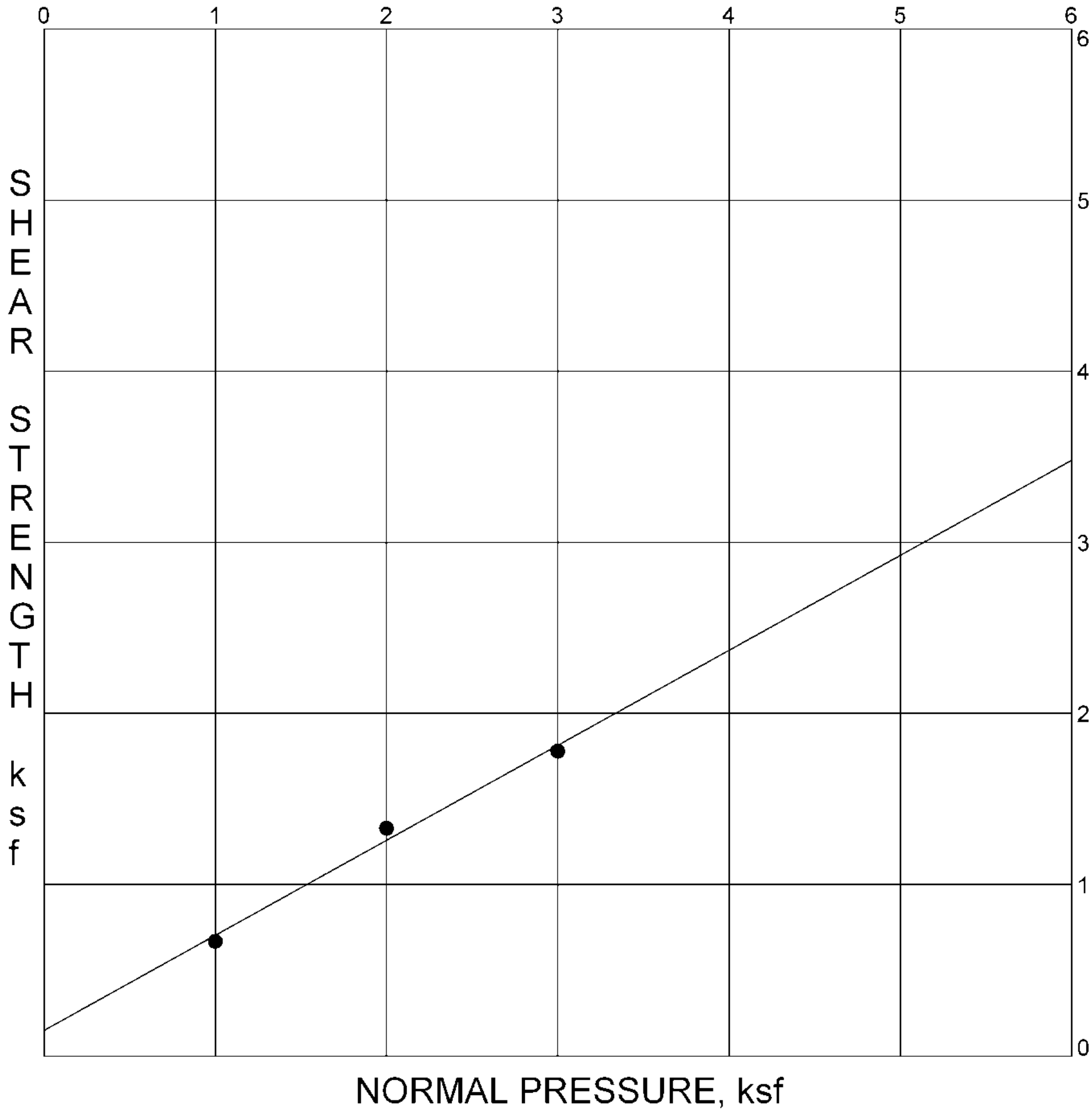
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 13.3	Dry Density: 101.8	Phi (deg): 28.0
Saturated: 24.2		Cohesion (ksf): 0.150

SHEAR TEST DIAGRAM

Figure S.1

PROJECT LOCATION: 1600 W. Commonwealth Ave	PROJECT NO.: 22-02182
SAMPLE LOCATION: B-2 @ 5.0	DESCRIPTION: Qal



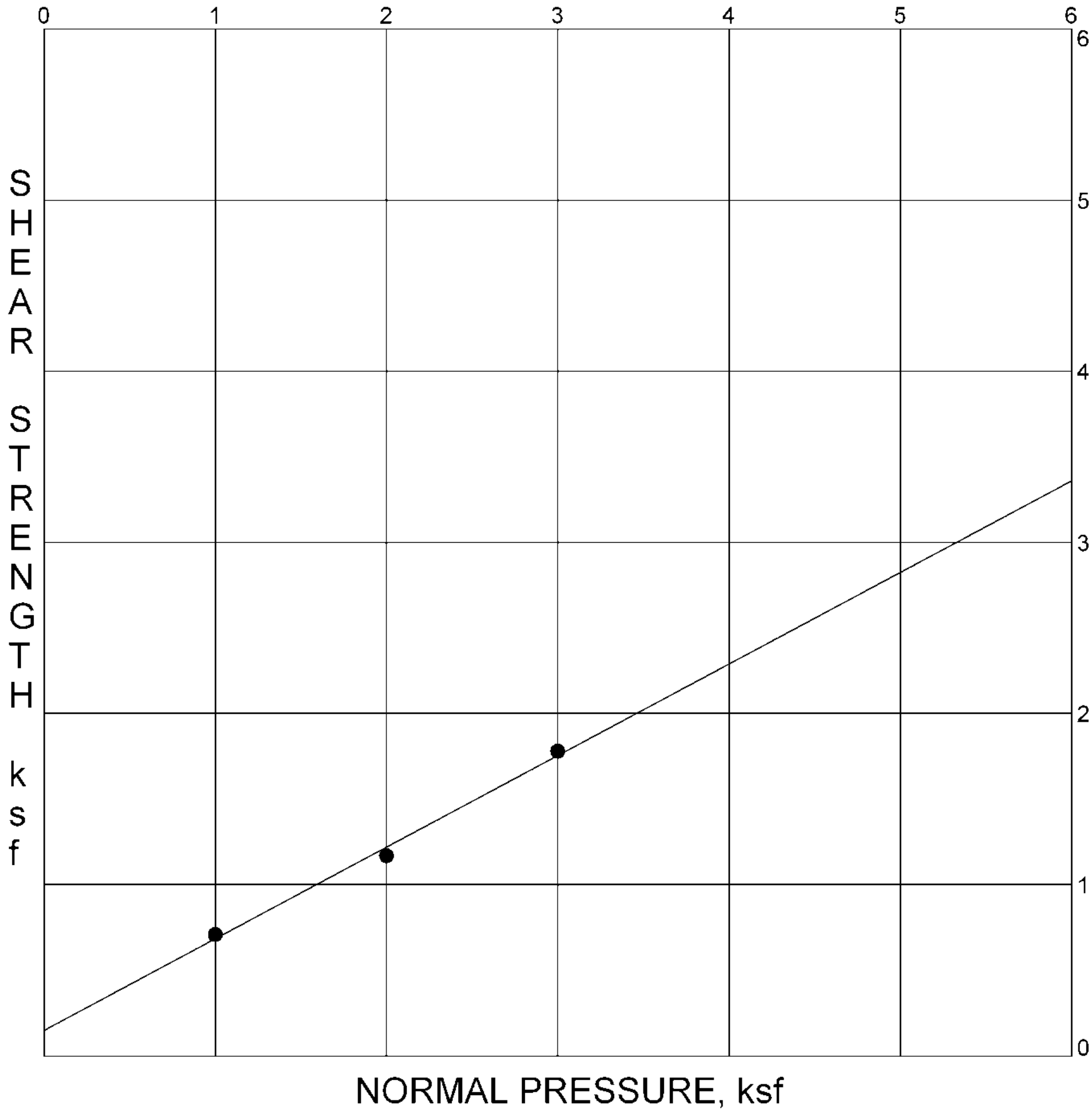
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 9.5	Dry Density: 107.6	Phi (deg): 29.0
Saturated: 21.0		Cohesion (ksf): 0.150

SHEAR TEST DIAGRAM

Figure S.2

PROJECT LOCATION: 1600 W. Commonwealth Ave	PROJECT NO.: 22-02182
SAMPLE LOCATION: B-3 @ 5.0	DESCRIPTION: Qal



Test Results

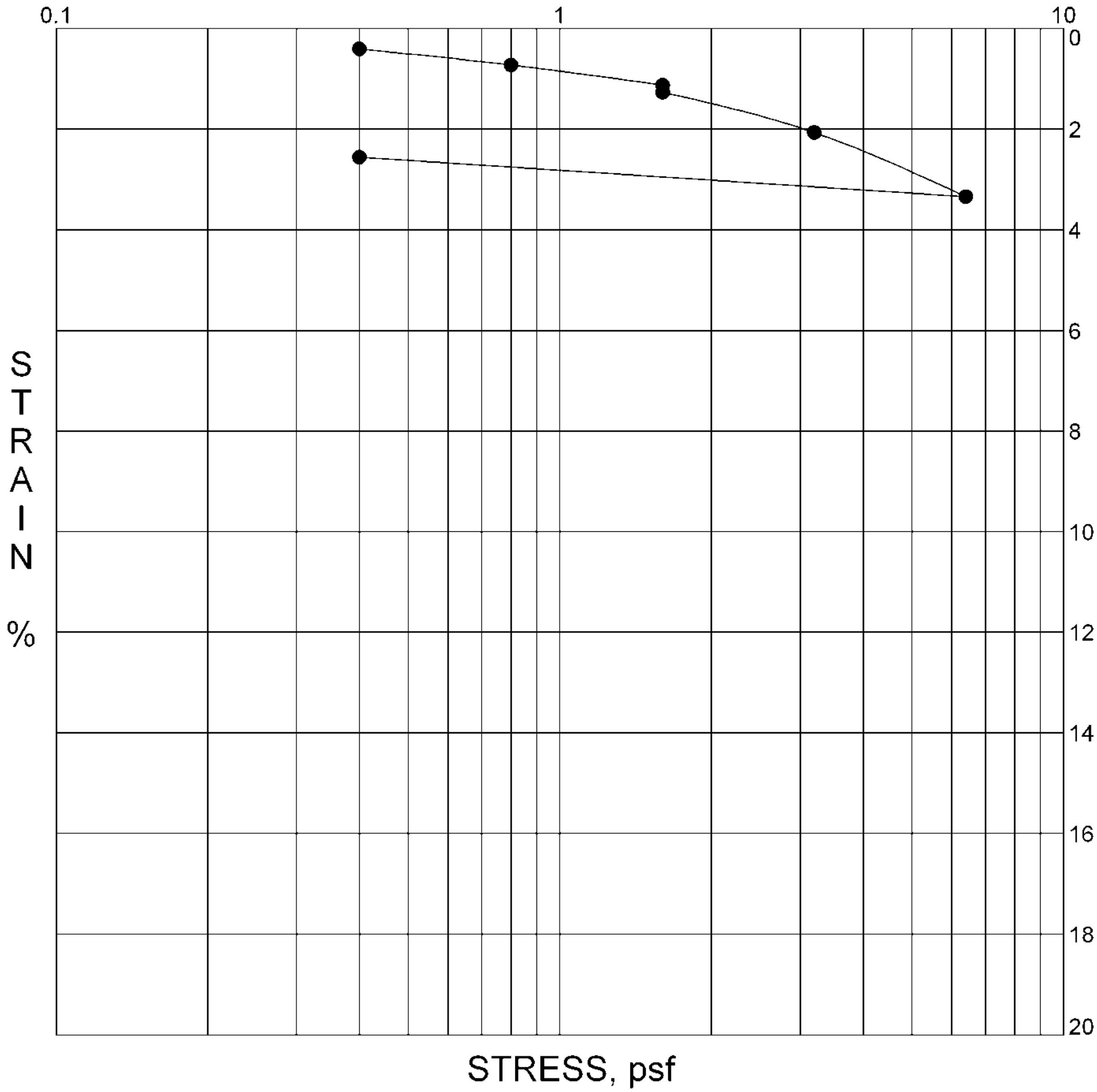
Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 16.9	Dry Density: 104.0	Phi (deg): 28.0
Saturated: 23.0		Cohesion (ksf): 0.150

SHEAR TEST DIAGRAM

Figure S.3

PROJECT LOCATION: 1600 W. Commonwealth Ave | PROJECT NO.: 22-02182

SAMPLE LOCATION: B-1 @ 7.5 | DESCRIPTION: Qal



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 16.4	Dry Density: 104.6	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

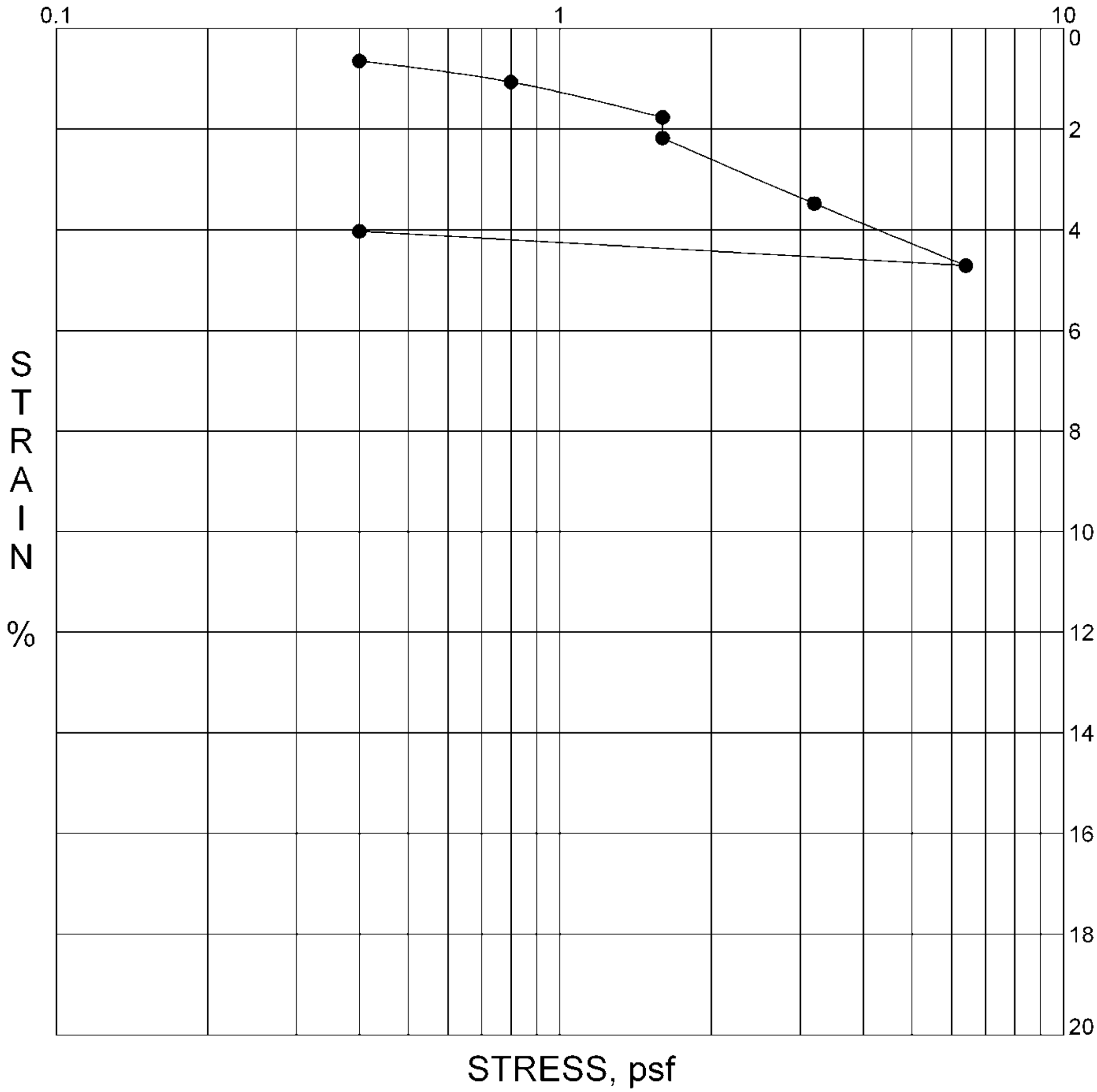
Figure C.1

PROJECT LOCATION: 1600 W. Commonwealth Ave

PROJECT NO.: 22-02182

SAMPLE LOCATION: B-1 @ 12.5

DESCRIPTION: Qal



Test Results

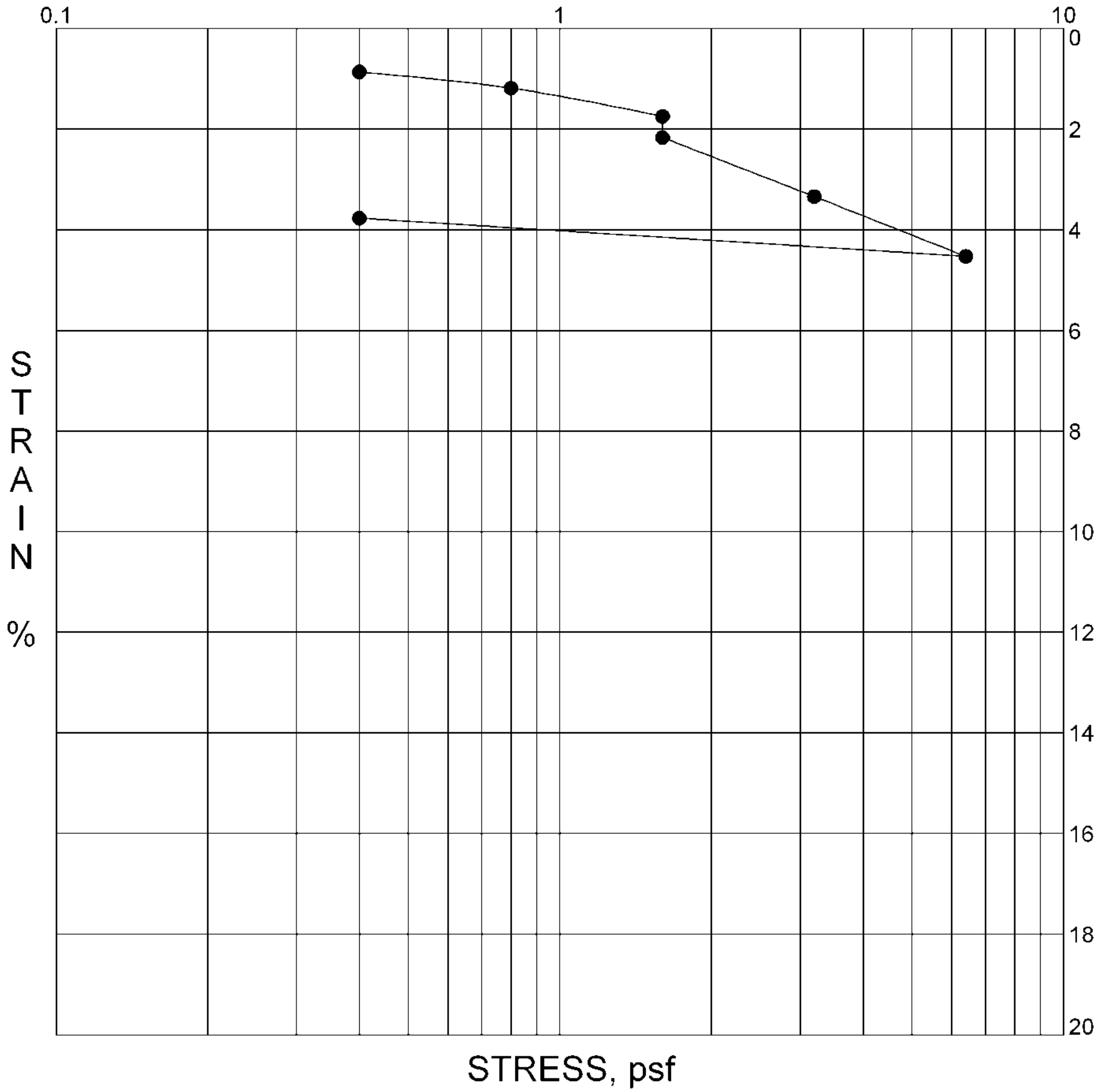
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 18.4	Dry Density: 107.9	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.2

PROJECT LOCATION: 1600 W. Commonwealth Ave   PROJECT NO.: 22-02182

SAMPLE LOCATION: B-1 @ 17.5   DESCRIPTION: Qal



Test Results

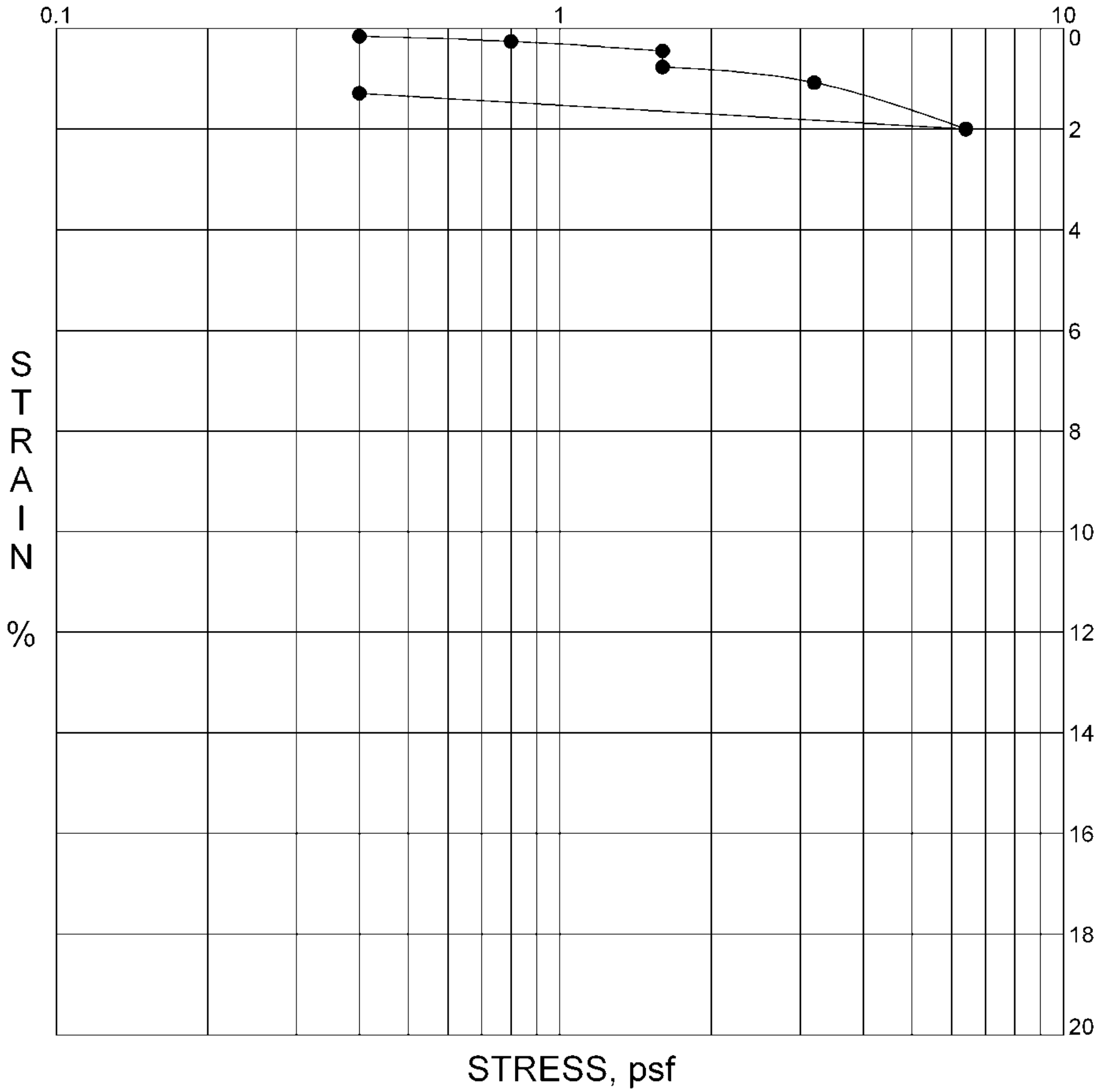
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 15.9	Dry Density: 113.8	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.3

PROJECT LOCATION: 1600 W. Commonwealth Ave PROJECT NO.: 22-02182

SAMPLE LOCATION: B-2 @ 5.0 DESCRIPTION: Qal



Test Results

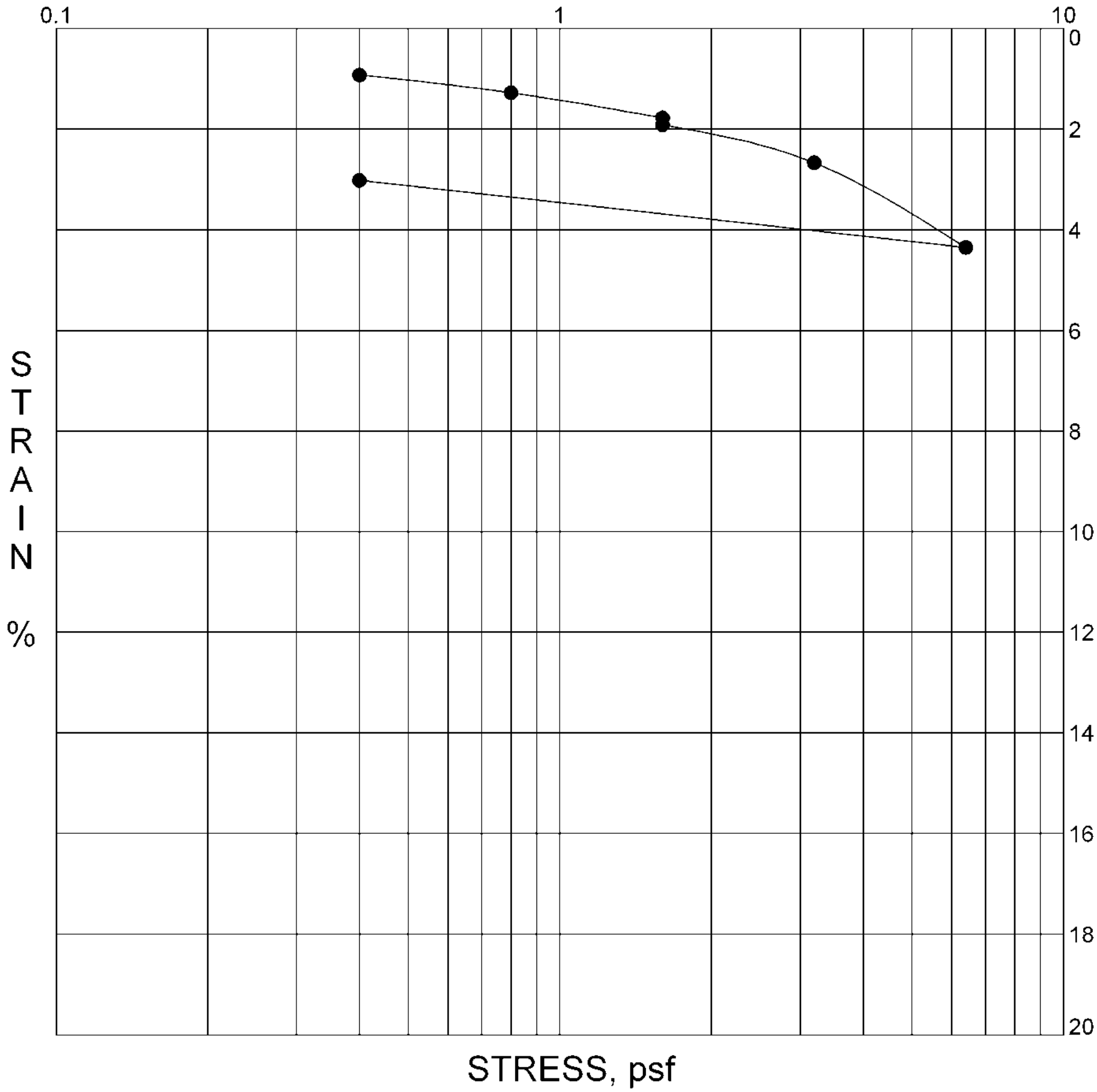
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 9.5	Dry Density: 107.6	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.4

PROJECT LOCATION: 1600 W. Commonwealth Ave PROJECT NO.: 22-02182

SAMPLE LOCATION: B-2 @ 10.0 DESCRIPTION: Qal



Test Results

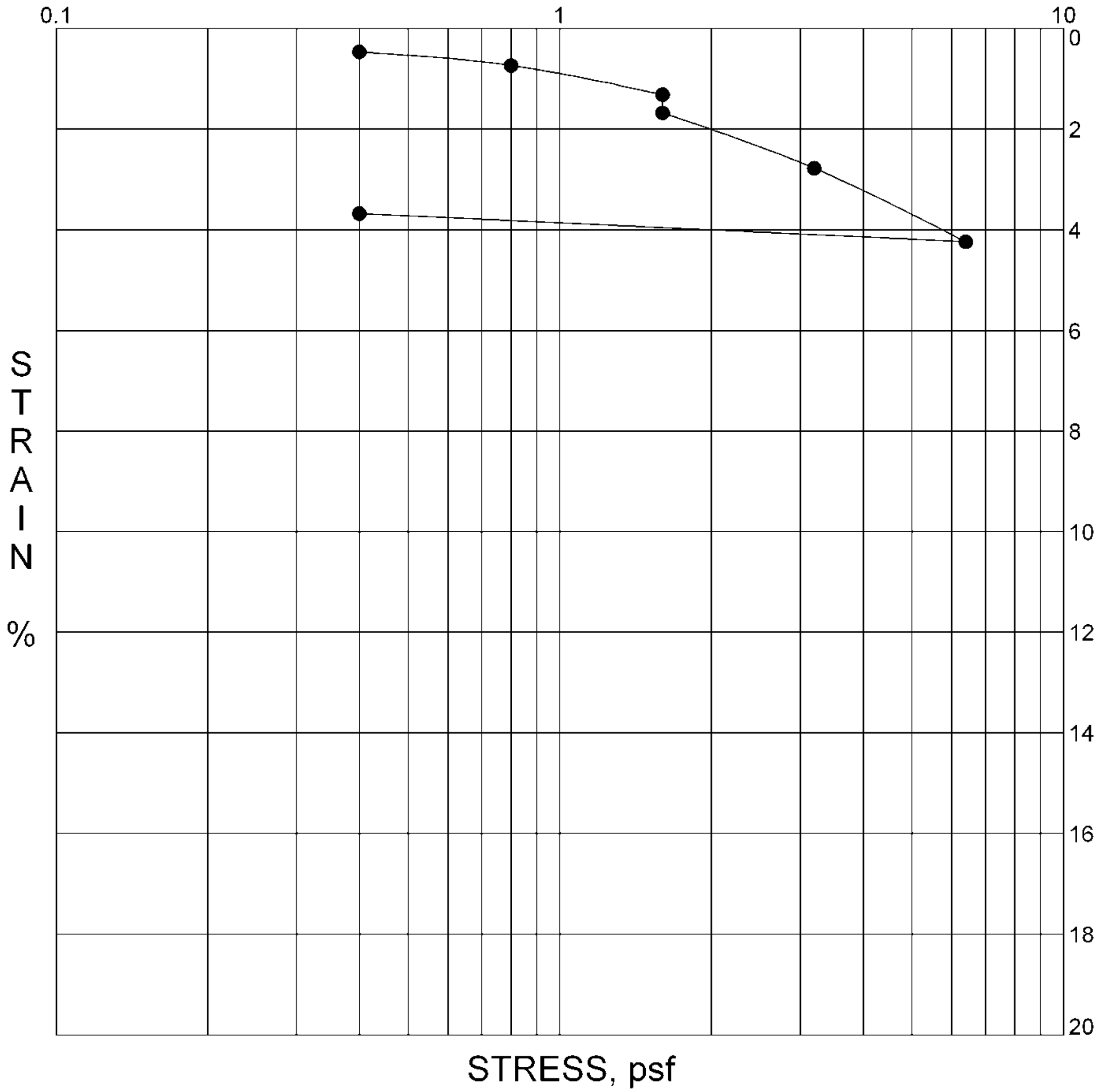
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 20.7	Dry Density: 99.6	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.5

PROJECT LOCATION: 1600 W. Commonwealth Ave   PROJECT NO.: 22-02182

SAMPLE LOCATION: B-2 @ 15.0   DESCRIPTION: Qal



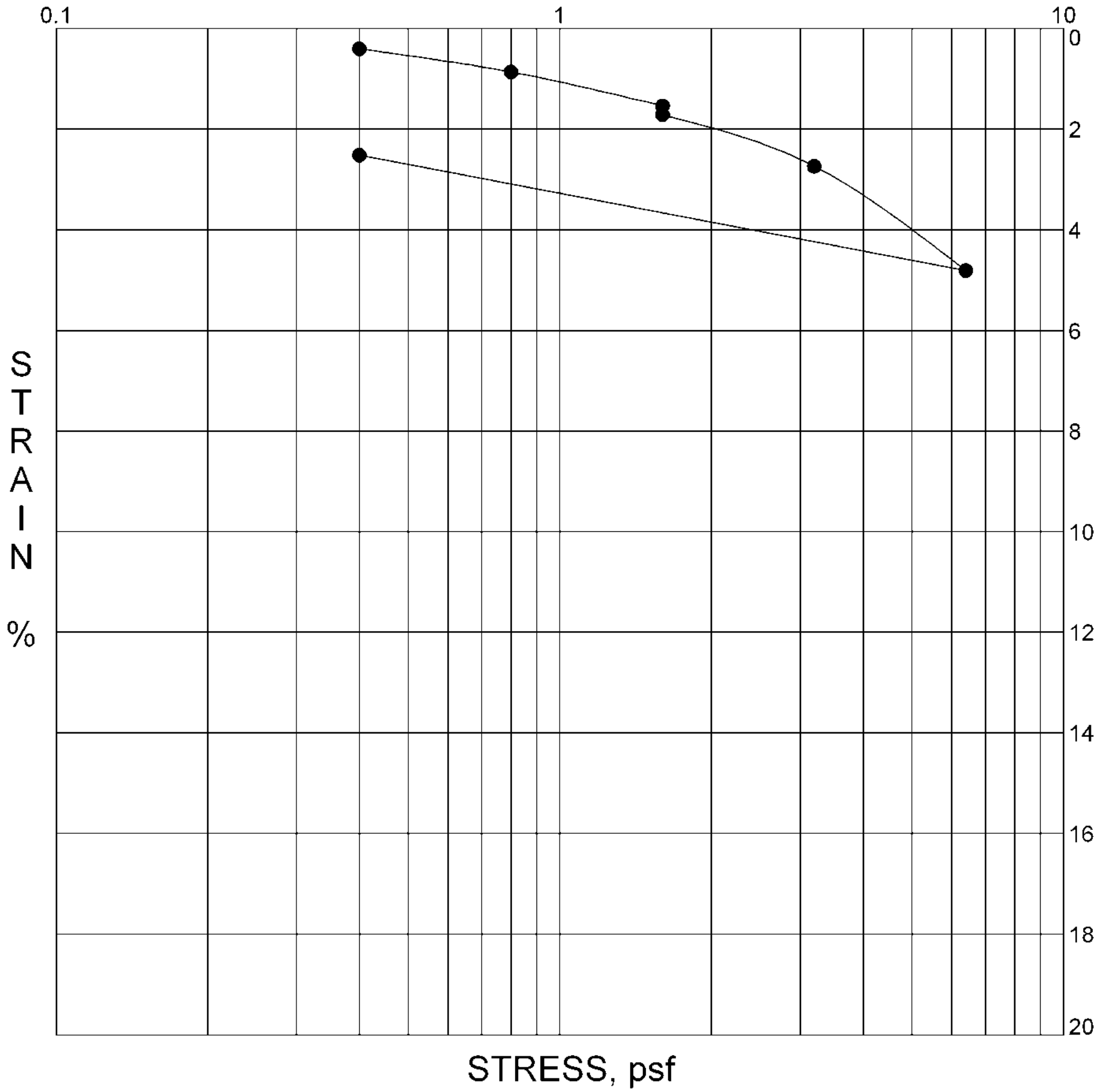
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 15.2	Dry Density: 117.6	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.6

PROJECT LOCATION: 1600 W. Commonwealth Ave	PROJECT NO.: 22-02182
SAMPLE LOCATION: B-3 @ 10.0	DESCRIPTION: Qal



Test Results

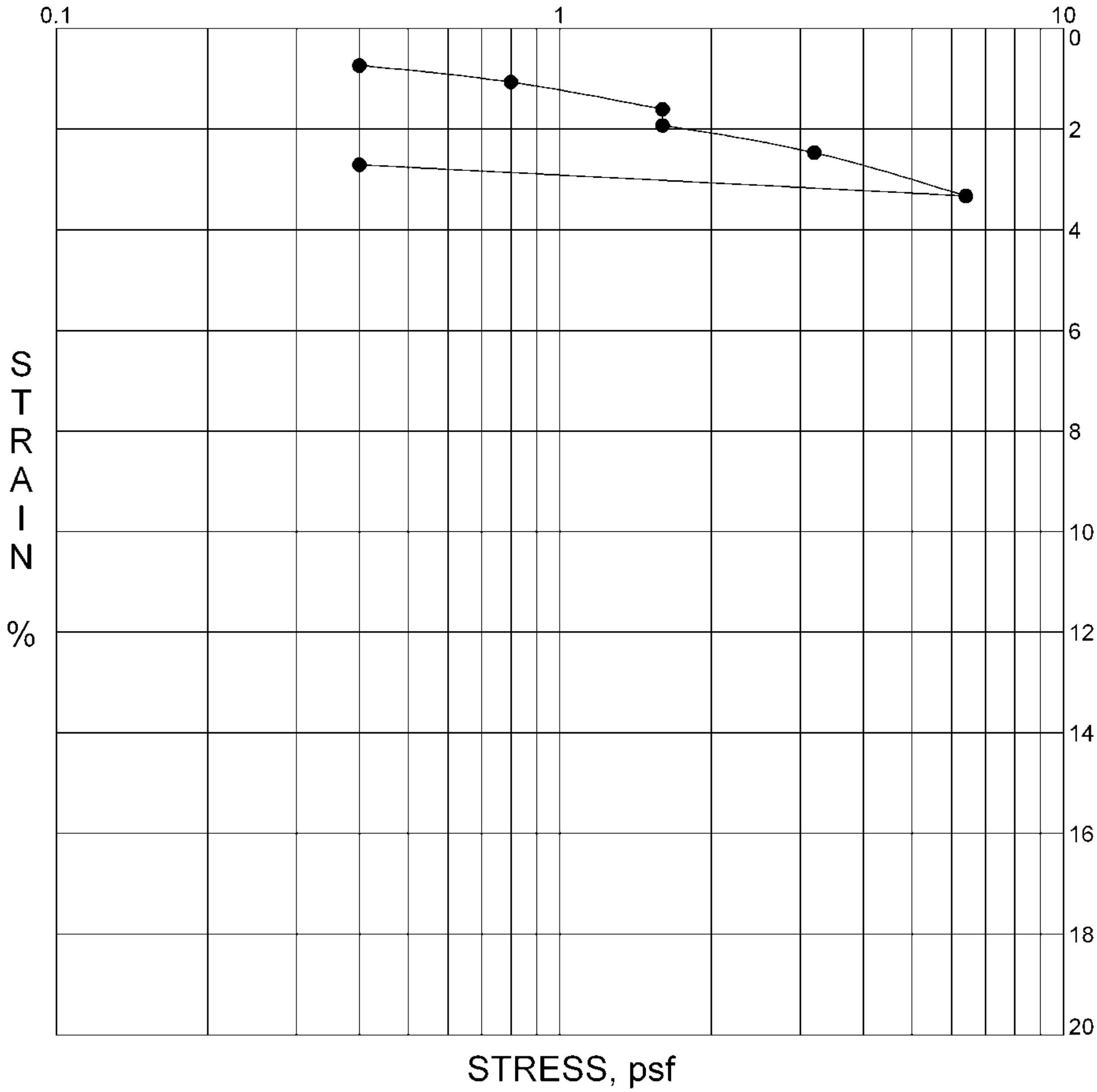
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 14.1	Dry Density: 90.2	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.7

PROJECT LOCATION: 1600 W. Commonwealth Ave PROJECT NO.: 22-02182

SAMPLE LOCATION: B-3 @ 15.0 DESCRIPTION: Qal



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 14.6	Dry Density: 109.4	1600 lbs.

CONSOLIDATION TEST DIAGRAM

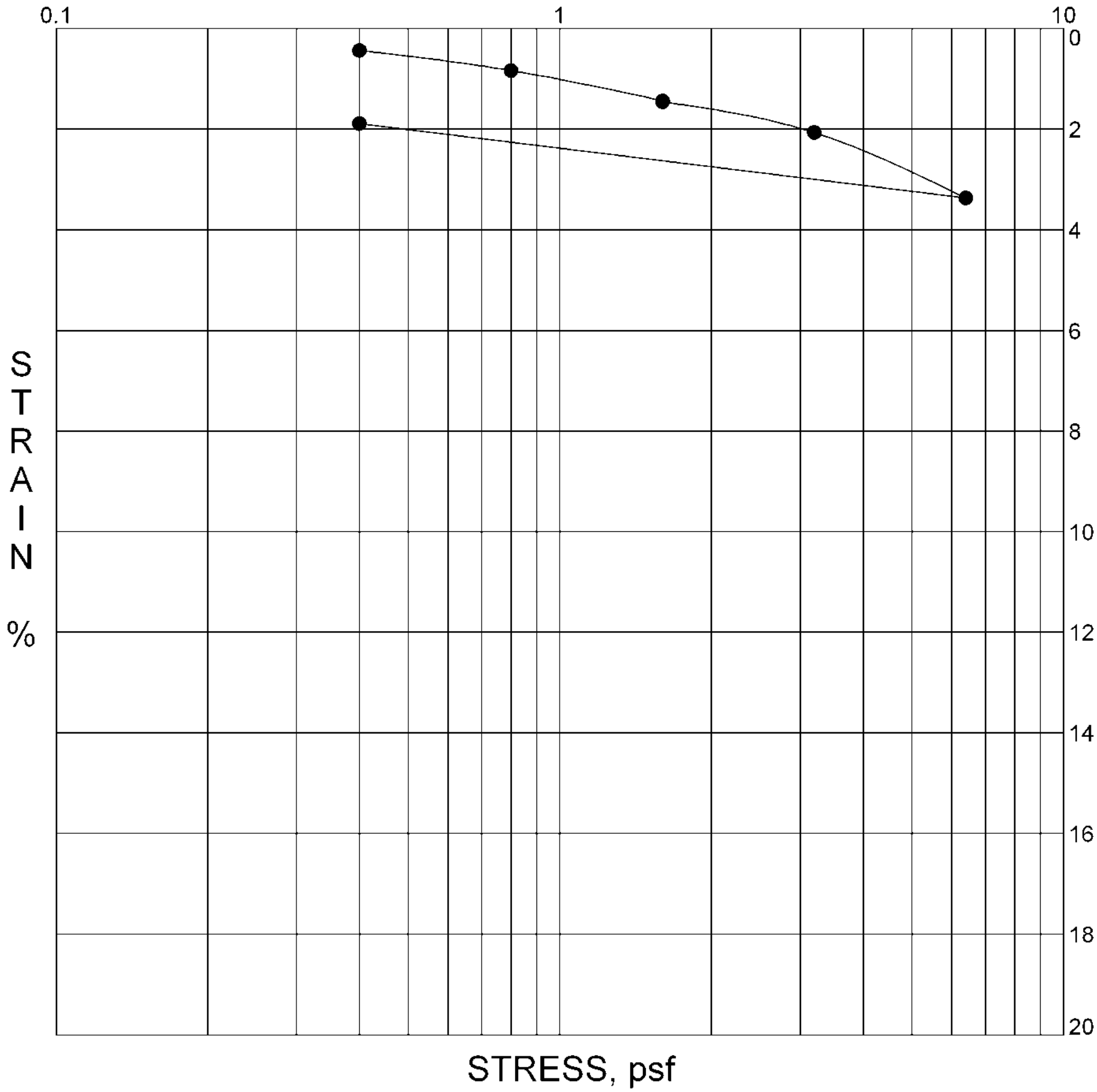
Figure C.8

PROJECT LOCATION: 1600 W. Commonwealth Ave

PROJECT NO.: 22-02182

SAMPLE LOCATION: B-4 @ 5.0

DESCRIPTION: Qal



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 23.5	Dry Density: 101.3	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

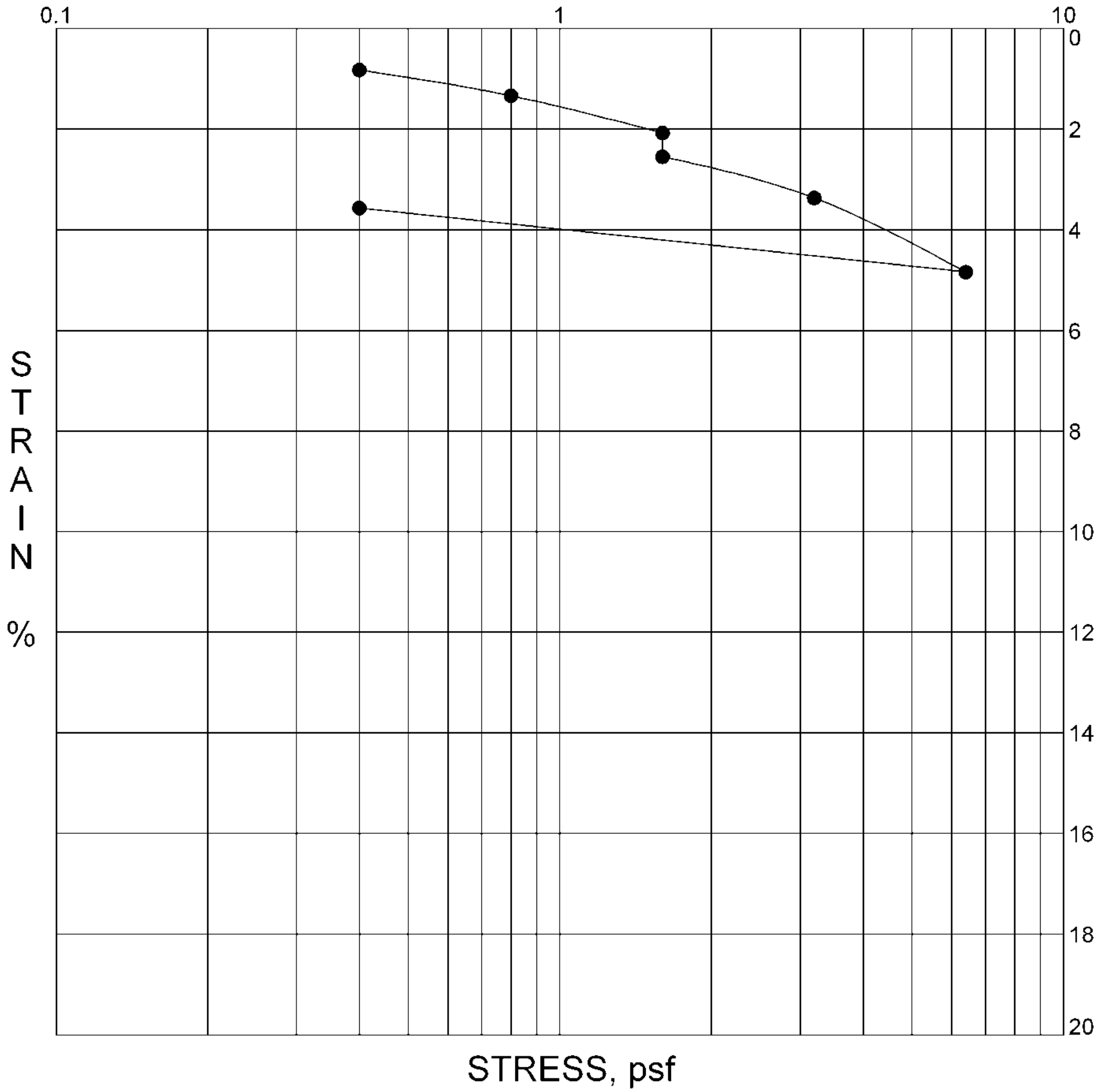
Figure C.9

PROJECT LOCATION: 1600 W. Commonwealth Ave

PROJECT NO.: 22-02182

SAMPLE LOCATION: B-4 @ 10.0

DESCRIPTION: Qal



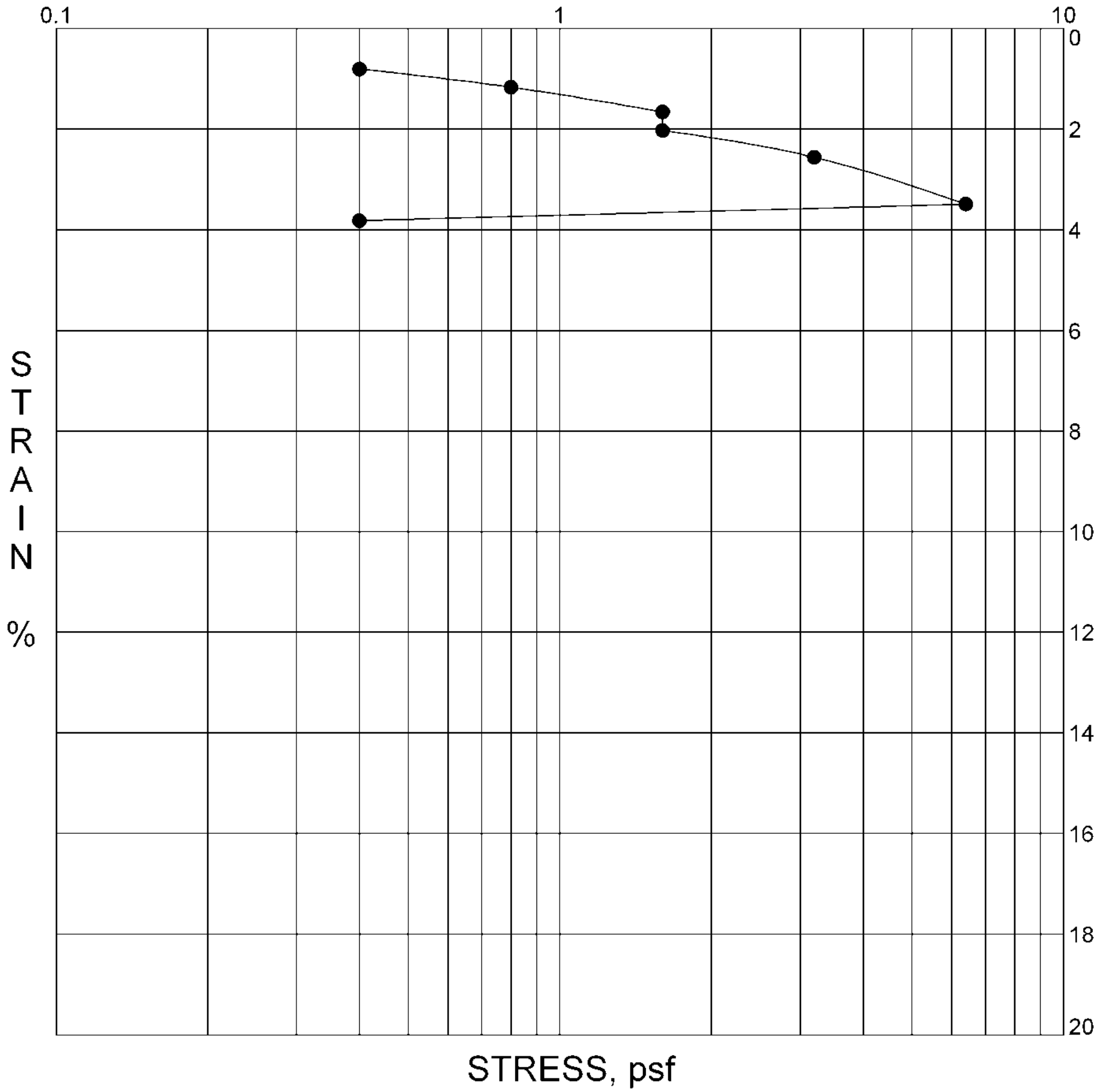
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 18.9	Dry Density: 102.2	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.10

PROJECT LOCATION: 1600 W. Commonwealth Ave	PROJECT NO.: 22-02182
SAMPLE LOCATION: B-4 @ 15.0	DESCRIPTION: Qal

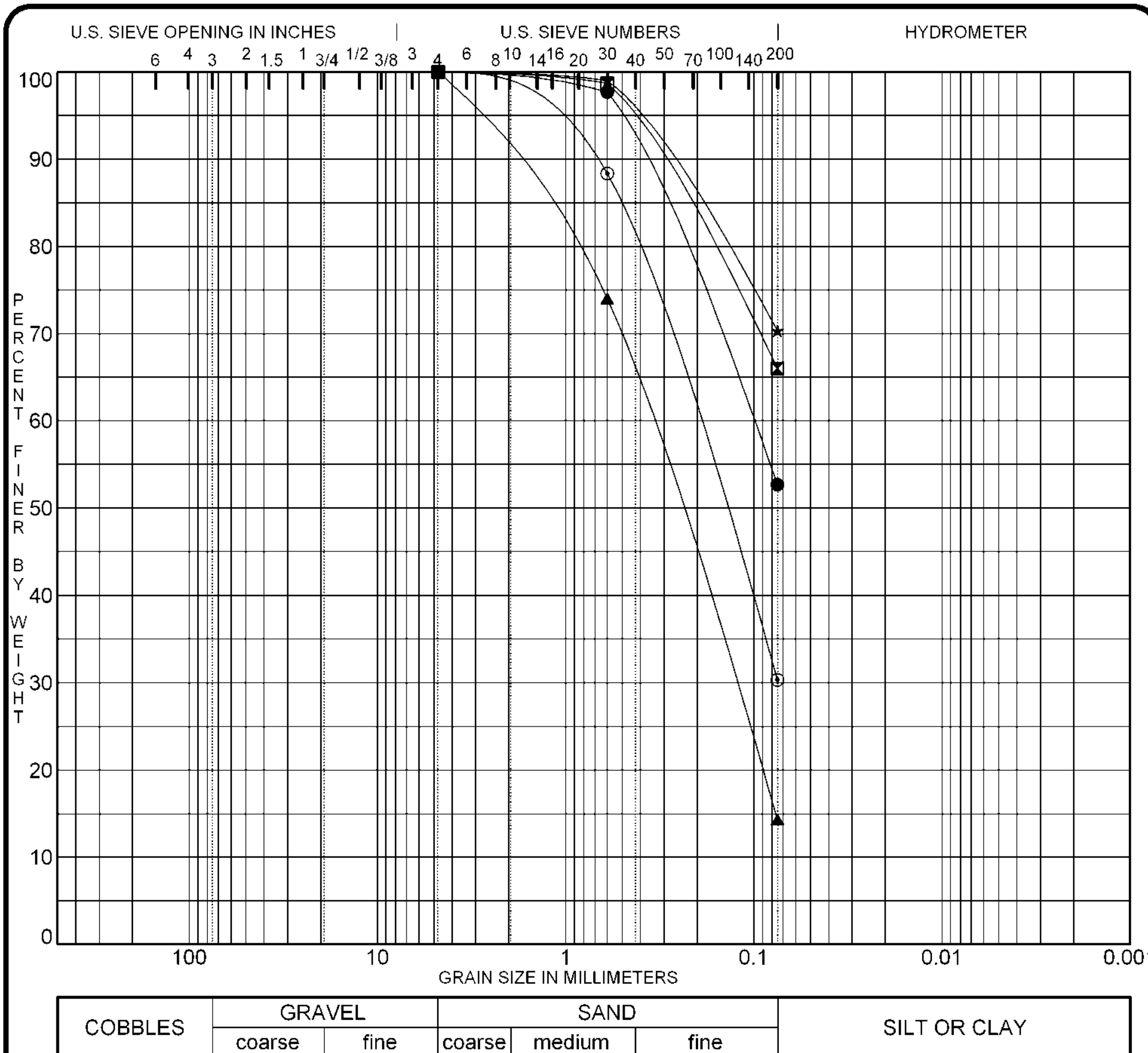


Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 6.2	Dry Density: 107.2	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.11



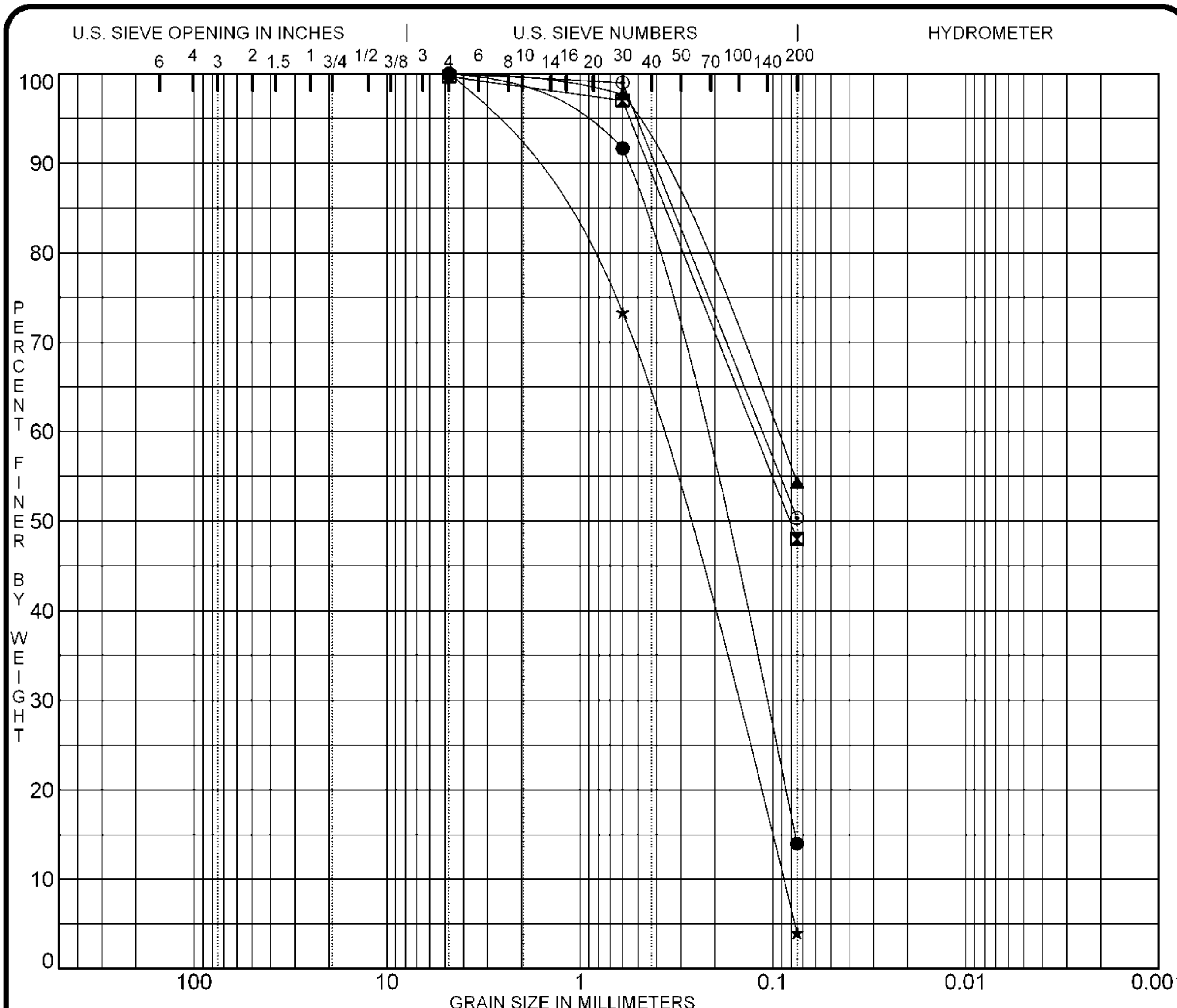
	Specimen Identification		USCS Classification	MC%	LL	PL	PI	Cc	Cu
●	**B-1**	**5.0**	**Qal**		**24**	**19**	**5**		
⧗	**B-1**	**10.0**	**Qal**		**38**	**22**	**15**		
▲	**B-1**	**15.0**	**Qal**						
★	**B-1**	**20.0**	**Qal**		**25**	**22**	**3**		
⊙	**B-1**	**25.0**	**Qal**						

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**5.0**	**4.75**	**0.11**			**0.0**	**47.3**	**52.7**	
⧗	**B-1**	**10.0**	**4.75**				**0.0**	**34.0**	**66.0**	
▲	**B-1**	**15.0**	**4.75**	**0.37**	**0.129**		**0.0**	**85.7**	**14.3**	
★	**B-1**	**20.0**	**4.75**				**0.0**	**29.7**	**70.3**	
⊙	**B-1**	**25.0**	**4.75**	**0.22**			**0.0**	**69.7**	**30.3**	

PROJECT **- 1600 W. Commonwealth Ave**

GRADATION CURVES

GeoConcepts



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

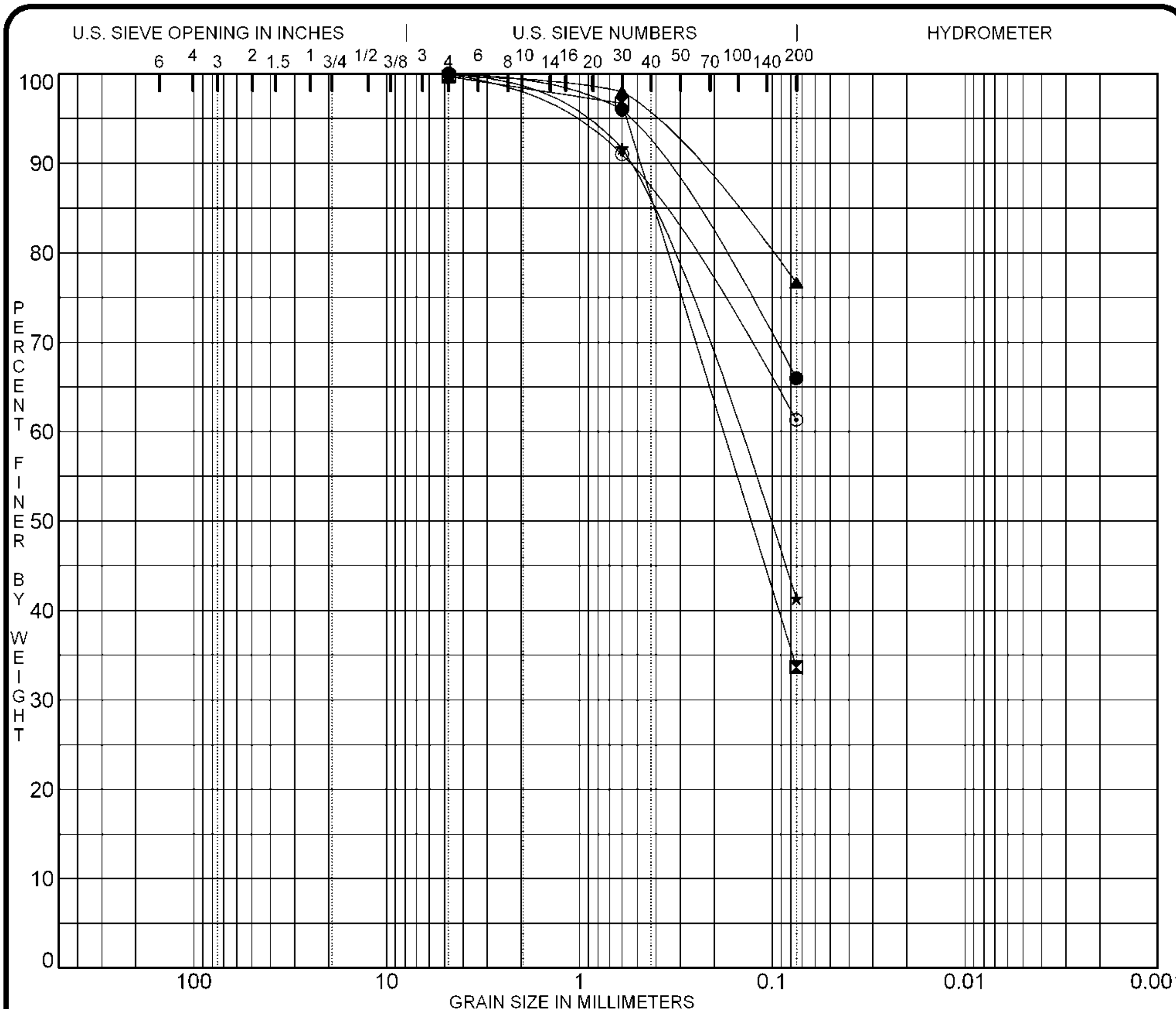
	Specimen Identification		USCS Classification	MC%	LL	PL	PI	Cc	Cu
●	**B-1**	**30.0**	**Qal**						
⧗	**B-1**	**35.0**	**Qal**						
▲	**B-1**	**40.0**	**Qal**						
★	**B-1**	**50.0**	**Qal**					**0.74**	**4.5**
⊙	**B-2**	**2.5**	**Qal**						

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**30.0**	**4.75**	**0.26**	**0.115**		**0.0**	**86.0**	**14.0**	
⧗	**B-1**	**35.0**	**4.75**	**0.12**			**0.0**	**51.7**	**48.0**	
▲	**B-1**	**40.0**	**4.75**	**0.10**			**0.0**	**45.7**	**54.3**	
★	**B-1**	**50.0**	**4.75**	**0.40**	**0.164**	**0.0898**	**0.0**	**96.0**	**4.0**	
⊙	**B-2**	**2.5**	**4.75**	**0.11**			**0.0**	**49.7**	**50.3**	

PROJECT **- 1600 W. Commonwealth Ave**

GRADATION CURVES

GeoConcepts



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

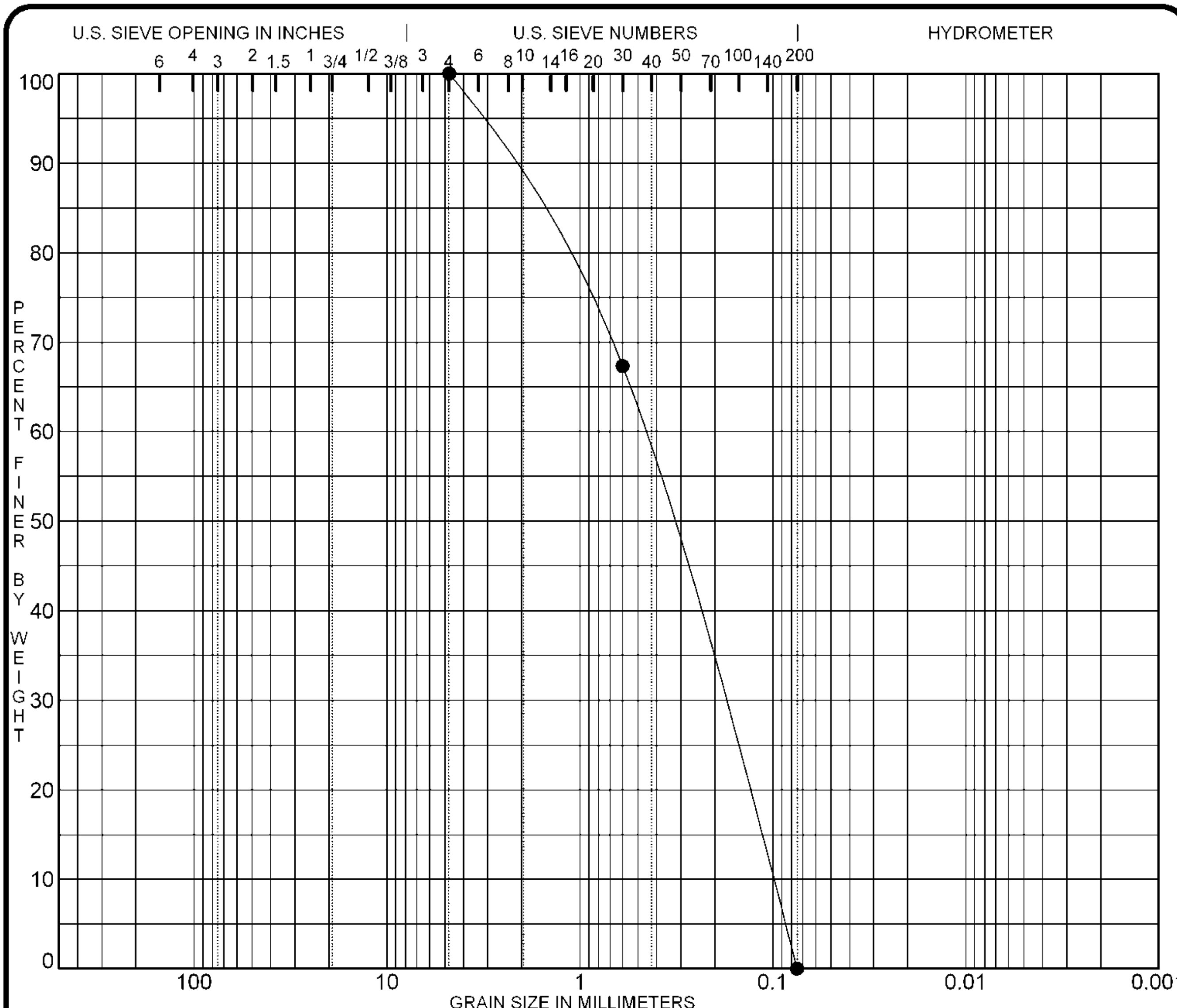
	Specimen Identification		USCS Classification	MC%	LL	PL	PI	Cc	Cu
●	B-2	12.5	Qal		35	25	10		
⧗	B-2	17.5	Qal						
▲	B-2	22.5	Qal		29	25	5		
★	B-2	32.5	Qal						
⊙	B-2	40.0	Qal		24	18	6		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-2	12.5	4.75				0.0	34.0	66.0	
⧗	B-2	17.5	4.75	0.18			0.0	66.0	33.7	
▲	B-2	22.5	4.75				0.0	23.0	76.7	
★	B-2	32.5	4.75	0.16			0.0	58.7	41.3	
⊙	B-2	40.0	4.75				0.0	38.7	61.3	

PROJECT - 1600 W. Commonwealth Ave

GRADATION CURVES

GeoConcepts



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		USCS Classification	MC%	LL	PL	PI	Cc	Cu
●	**B-2**	**50.0**	**Qal**					**0.73**	**4.7**

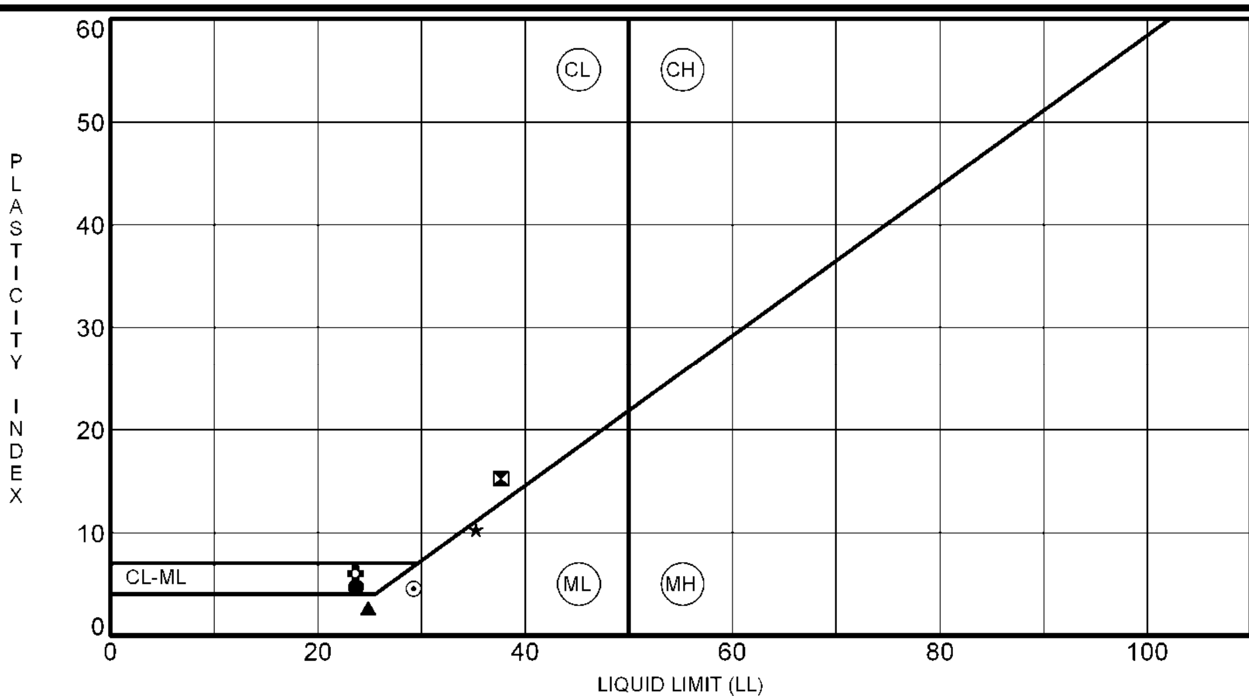
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-2**	**50.0**	**4.75**	**0.48**	**0.189**	**0.1021**	**0.0**	**100.0**	**0.0**	

PROJECT **- 1600 W. Commonwealth Ave**

GRADATION CURVES

GeoConcepts

PROJECT LOCATION: 1600 W. Commonwealth Ave PROJECT NO.: 22-02182



Specimen Identification		LL	PL	PI	Fines	Classification
● B-1	5.0	24	19	5	52.7	SANDY SILTY CLAY CL-ML
⊠ B-1	10.0	38	22	16	66.0	SANDY LEAN CLAY CL
▲ B-1	20.0	25	22	3	70.3	SILT with SAND ML
★ B-2	12.5	35	25	10	66.0	SANDY SILT ML
⊙ B-2	22.5	29	25	4	76.7	SILT with SAND ML
✿ B-2	40.0	24	18	6	61.3	SANDY SILTY CLAY CL-ML

ATTERBERG LIMITS' RESULTS

Figure A.1

APPENDIX III

ANALYSES

Lateral Design

Liquefaction

Seismic Evaluation

Maximum Vertical Cut Height

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:	Qal	WALL HEIGHT:	6 feet
SHEAR DIAGRAM:	B-3@5	BACKSLOPE ANGLE:	0 degrees
COHESION:	150 psf	SURCHARGE:	0 pounds
PHI ANGLE:	28 degrees	SURCHARGE TYPE:	U Uniform
DENSITY:	120 pcf	INITIAL FAILURE ANGLE:	20 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	4 feet
CD (C/FS):	120.0 psf	FINAL TENSION CRACK:	30 feet
PHID = ATAN(TAN(PHI)/FS) =	23.0 degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	45 degrees
AREA OF TRIAL FAILURE WEDGE	12.0 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	1440.0 pounds
NUMBER OF TRIAL WEDGES ANALYZED	2754 trials
LENGTH OF FAILURE PLANE	5.7 feet
DEPTH OF TENSION CRACK	1.0 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	4.0 feet
CALCULATED HORIZONTAL THRUST	**-93.0 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-7.4 pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**5.0 feet**

Slot Cuts (Six Feet High with Level Backslope)

TEMPORARY EXCAVATION HEIGHT

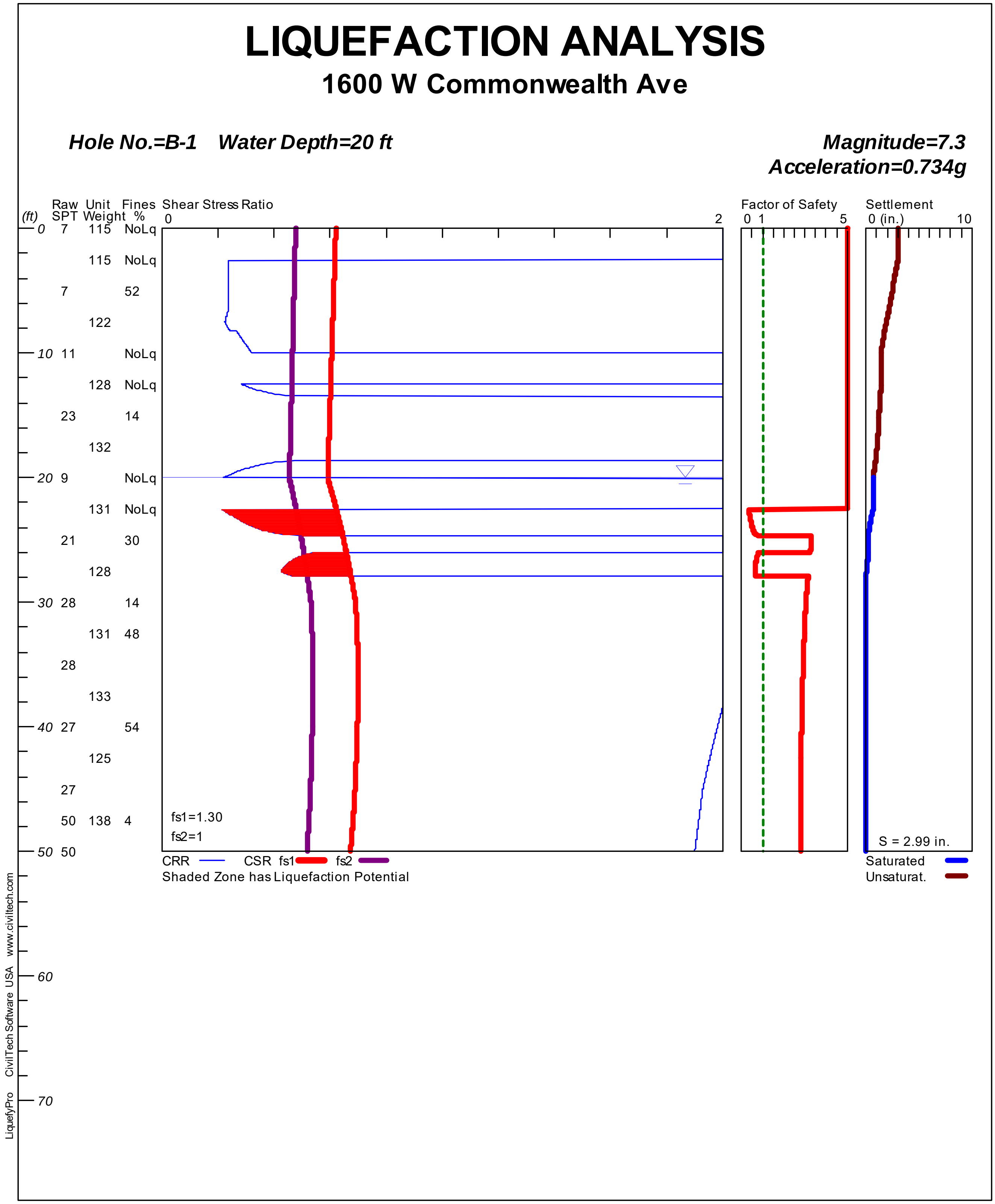
CALCULATE THE HEIGHT TO WHICH TEMPORARY EXCAVATIONS ARE STABLE (NEGATIVE THRUST).
THE EXCAVATION HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW.
ASSUME THE EARTH MATERIAL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

CALCULATION PARAMETERS

EARTH MATERIAL:	Qal	WALL HEIGHT:	6 feet
SHEAR DIAGRAM:	B-3@5	BACKSLOPE ANGLE:	0 degrees
COHESION:	150 psf	SURCHARGE:	0 pounds
PHI ANGLE:	28 degrees	SURCHARGE TYPE:	U Uniform
DENSITY:	120 pcf	INITIAL FAILURE ANGLE:	20 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	4 feet
CD (C/FS):	120.0 psf	FINAL TENSION CRACK:	30 feet
PHID = ATAN(TAN(PHI)/FS) =	23.0 degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	45 degrees
AREA OF TRIAL FAILURE WEDGE	12.0 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	1440.0 pounds
NUMBER OF TRIAL WEDGES ANALYZED	2754 trials
LENGTH OF FAILURE PLANE	5.7 feet
DEPTH OF TENSION CRACK	1.0 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	4.0 feet
CALCULATED HORIZONTAL THRUST	**-93.0 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-7.4 pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**5.0 feet**


LIQUEFACTION ANALYSIS
1600 W Commonwealth Ave
Hole No.=B-1 Water Depth=20 ft
Magnitude=7.3
Acceleration=0.734g
(ft) Raw SPT Unit Weight Fines %
Shear Stress Ratio
0
2
Factor of Safety
0 1 5
Settlement
0 (in.) 10
0 7 115 NoLq
115 NoLq
7 52
122
10 11 NoLq
128 NoLq
23 14
132
20 9 NoLq
131 NoLq
21 30
128
30 28 14
131 48
28
133
40 27 54
125
27
50 138 4
50 50
60
70
fs1=1.30
fs2=1
S = 2.99 in.
CRR CSR fs1 fs2
Shaded Zone has Liquefaction Potential
Saturated
Unsaturat.
LiquefyPro CivilTech Software USA www.civiltech.com

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

*****************************************************************************************************
       Font: Courier New, Regular, Size 8 is recommended for this report.
       Licensed to ,  8/18/2022      3:31:29 PM

       Input File Name: Z:\OUR DOCUMENTS\Liquefaction Analysis\22-02182-1 B-1.liq
       Title:  1600 W Commonwealth Ave
       Subtitle:  22-02182

 Input Data:
       Surface Elev.=
       Hole No.=B-1
       Depth of Hole=50.00 ft
       Water Table during Earthquake= 20.00 ft
       Water Table during In-Situ Testing= 45.00 ft
       Max. Acceleration=0.73 g
       Earthquake Magnitude=7.30
       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
       8. Sampling Method,                                         Cs= 1.2
       9. User request factor of safety (apply to CSR) ,   User= 1.3
          Plot two CSR (fs1=User, fs2=1)
       10. Average two input data between two Depths: Yes*
       * Recommended Options

       In-Situ Test Data:
```

Depth ft	SPT	Gamma pcf	Fines %
0.00	7.00	115.00	NoLiq
2.50	7.00	115.00	NoLiq
5.00	7.00	115.00	52.00
7.50	7.00	122.00	52.00
10.00	11.00	122.00	NoLiq
12.50	11.00	128.00	NoLiq
15.00	23.00	128.00	14.00
17.50	23.00	132.00	14.00
20.00	9.00	132.00	NoLiq
22.50	9.00	131.00	NoLiq
25.00	21.00	131.00	30.00
27.50	21.00	128.00	30.00
30.00	28.00	128.00	14.00
32.50	28.00	131.00	48.00
35.00	28.00	131.00	48.00
37.50	28.00	133.00	48.00
40.00	27.00	133.00	54.00
42.50	27.00	125.00	54.00
45.00	27.00	125.00	54.00
47.50	50.00	138.00	4.00
50.00	50.00	138.00	4.00

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

       Peak Ground Acceleration (PGA), a_max = 0.73g

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

0.00	115.00	0.000	115.00	0.000	1.00	0.000	0.734	0.48	1.30	0.62
2.00	115.00	0.109	115.00	0.109	1.00	0.000	0.734	0.47	1.30	0.62
4.00	115.00	0.217	115.00	0.217	0.99	0.000	0.734	0.47	1.30	0.61
6.00	117.80	0.327	117.80	0.327	0.99	0.000	0.734	0.47	1.30	0.61
8.00	122.00	0.440	122.00	0.440	0.98	0.000	0.734	0.47	1.30	0.61
10.00	122.00	0.556	122.00	0.556	0.98	0.000	0.734	0.47	1.30	0.61
12.00	126.80	0.673	126.80	0.673	0.97	0.000	0.734	0.46	1.30	0.60
14.00	128.00	0.794	128.00	0.794	0.97	0.000	0.734	0.46	1.30	0.60
16.00	129.60	0.915	129.60	0.915	0.96	0.000	0.734	0.46	1.30	0.60
18.00	132.00	1.039	132.00	1.039	0.96	0.000	0.734	0.46	1.30	0.59
20.00	132.00	1.164	132.00	1.164	0.95	0.000	0.734	0.45	1.30	0.59
22.00	131.20	1.288	68.80	1.231	0.95	0.000	0.734	0.47	1.30	0.62
24.00	131.00	1.412	68.60	1.296	0.94	0.000	0.734	0.49	1.30	0.64
26.00	129.80	1.536	67.40	1.360	0.94	0.000	0.734	0.51	1.30	0.66
28.00	128.00	1.657	65.60	1.423	0.93	0.000	0.734	0.52	1.30	0.68
30.00	128.00	1.778	65.60	1.485	0.93	0.000	0.734	0.53	1.30	0.69
32.00	130.40	1.900	68.00	1.548	0.91	0.000	0.734	0.54	1.30	0.70
34.00	131.00	2.024	68.60	1.613	0.90	0.000	0.734	0.54	1.30	0.70
36.00	131.80	2.148	69.40	1.678	0.88	0.000	0.734	0.54	1.30	0.70
38.00	133.00	2.273	70.60	1.744	0.86	0.000	0.734	0.54	1.30	0.70
40.00	133.00	2.399	70.60	1.811	0.85	0.000	0.734	0.54	1.30	0.70
42.00	126.60	2.522	64.20	1.875	0.83	0.000	0.734	0.53	1.30	0.69
44.00	125.00	2.640	62.60	1.934	0.82	0.000	0.734	0.53	1.30	0.69
46.00	130.20	2.760	67.80	1.994	0.80	0.000	0.734	0.53	1.30	0.69
48.00	138.00	2.887	75.60	2.063	0.78	0.000	0.734	0.52	1.30	0.68
50.00	138.00	3.018	75.60	2.134	0.77	0.000	0.734	0.52	1.30	0.67

CSR is based on water table at 20.00 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	7.00	1.50	0.75	0.000	1.70	13.39	NoLiq	7.20	20.59	0.22
2.00	7.00	1.50	0.75	0.109	1.70	13.39	NoLiq	7.20	20.59	0.22
4.00	7.00	1.50	0.75	0.217	1.70	13.39	71.60	7.20	20.59	0.22
6.00	7.00	1.50	0.75	0.327	1.70	13.39	52.00	7.20	20.59	0.22
8.00	7.80	1.50	0.75	0.440	1.51	13.22	61.80	7.20	20.42	0.22
10.00	11.00	1.50	0.85	0.556	1.34	18.81	NoLiq	7.20	26.01	0.30
12.00	11.00	1.50	0.85	0.673	1.22	17.09	NoLiq	7.20	24.29	0.27
14.00	18.20	1.50	0.85	0.794	1.12	26.04	48.80	7.20	33.24	2.00
16.00	23.00	1.50	0.95	0.915	1.05	34.26	14.00	2.16	36.42	2.00
18.00	20.20	1.50	0.95	1.039	0.98	28.24	31.40	6.34	34.57	2.00
20.00	9.00	1.50	0.95	1.164	0.93	11.89	NoLiq	7.20	19.09	0.21
22.00	9.00	1.50	0.95	1.288	0.88	11.30	NoLiq	7.20	18.50	0.20
24.00	16.20	1.50	0.95	1.412	0.84	19.43	58.40	7.20	26.63	0.31
26.00	21.00	1.50	0.95	1.536	0.81	24.15	30.00	6.00	30.15	2.00
28.00	22.40	1.50	1.00	1.657	0.78	26.10	26.80	5.23	31.33	2.00
30.00	28.00	1.50	1.00	1.778	0.75	31.49	14.00	2.16	33.65	2.00
32.00	28.00	1.50	1.00	1.900	0.73	30.47	41.20	7.20	37.67	2.00
34.00	28.00	1.50	1.00	2.024	0.70	29.52	48.00	7.20	36.72	2.00
36.00	28.00	1.50	1.00	2.148	0.68	28.66	48.00	7.20	35.86	2.00
38.00	27.80	1.50	1.00	2.273	0.66	27.66	49.20	7.20	34.86	2.00
40.00	27.00	1.50	1.00	2.399	0.65	26.15	54.00	7.20	33.35	2.00
42.00	27.00	1.50	1.00	2.522	0.63	25.50	54.00	7.20	32.70	2.00
44.00	27.00	1.50	1.00	2.640	0.62	24.92	54.00	7.20	32.12	2.00
46.00	36.20	1.50	1.00	2.732	0.61	32.85	34.01	6.96	39.81	2.00
48.00	50.00	1.50	1.00	2.800	0.60	44.82	4.00	0.00	44.82	2.00
50.00	50.00	1.50	1.00	2.872	0.59	44.26	4.00	0.00	44.26	2.00

CRR is based on water table at 45.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 7.30:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	0.22	1.00	0.22	1.07	2.00	0.62	5.00 ^
2.00	0.07	0.22	1.00	0.22	1.07	2.00	0.62	5.00 ^
4.00	0.14	0.22	1.00	0.22	1.07	0.24	0.61	5.00
6.00	0.21	0.22	1.00	0.22	1.07	0.24	0.61	5.00
8.00	0.29	0.22	1.00	0.22	1.07	0.24	0.61	5.00
10.00	0.36	0.30	1.00	0.30	1.07	2.00	0.61	5.00 ^

12.00	0.44	0.27	1.00	0.27	1.07	2.00	0.60	5.00	^
14.00	0.52	2.00	1.00	2.00	1.07	2.14	0.60	5.00	
16.00	0.60	2.00	1.00	2.00	1.07	2.14	0.60	5.00	
18.00	0.68	2.00	1.00	2.00	1.07	2.14	0.59	5.00	
20.00	0.76	0.21	1.00	0.21	1.07	0.22	0.59	5.00	
22.00	0.84	0.20	1.00	0.20	1.07	2.00	0.62	5.00	^
24.00	0.92	0.31	1.00	0.31	1.07	0.33	0.64	0.52	*
26.00	1.00	2.00	1.00	2.00	1.07	2.14	0.66	3.26	
28.00	1.08	2.00	0.99	1.99	1.07	2.13	0.68	3.15	
30.00	1.16	2.00	0.98	1.96	1.07	2.10	0.69	3.05	
32.00	1.24	2.00	0.97	1.94	1.07	2.08	0.70	2.99	
34.00	1.32	2.00	0.96	1.92	1.07	2.05	0.70	2.94	
36.00	1.40	2.00	0.95	1.89	1.07	2.03	0.70	2.90	
38.00	1.48	2.00	0.94	1.87	1.07	2.00	0.70	2.87	
40.00	1.56	2.00	0.92	1.85	1.07	1.98	0.70	2.84	
42.00	1.64	2.00	0.91	1.83	1.07	1.96	0.69	2.82	
44.00	1.72	2.00	0.90	1.81	1.07	1.94	0.69	2.81	
46.00	1.78	2.00	0.90	1.80	1.07	1.92	0.69	2.80	
48.00	1.82	2.00	0.89	1.78	1.07	1.91	0.68	2.81	
50.00	1.87	2.00	0.89	1.77	1.07	1.90	0.67	2.83	

* 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	-	-	-	20.59	NoLiq	0.00	20.59
2.00	-	-	-	20.59	NoLiq	0.00	20.59
4.00	-	-	-	20.59	71.60	0.00	20.59
6.00	-	-	-	20.59	52.00	0.00	20.59
8.00	-	-	-	20.42	61.80	0.00	20.42
10.00	-	-	-	26.01	NoLiq	0.00	26.01
12.00	-	-	-	24.29	NoLiq	0.00	24.29
14.00	-	-	-	33.24	48.80	0.00	33.24
16.00	-	-	-	36.42	14.00	0.00	36.42
18.00	-	-	-	34.57	31.40	0.00	34.57
20.00	-	-	-	19.09	NoLiq	0.00	19.09
22.00	-	-	-	18.50	NoLiq	0.00	18.50
24.00	-	-	-	26.63	58.40	0.00	26.63
26.00	-	-	-	30.15	30.00	0.00	30.15
28.00	-	-	-	31.33	26.80	0.00	31.33
30.00	-	-	-	33.65	14.00	0.00	33.65
32.00	-	-	-	37.67	41.20	0.00	37.67
34.00	-	-	-	36.72	48.00	0.00	36.72
36.00	-	-	-	35.86	48.00	0.00	35.86
38.00	-	-	-	34.86	49.20	0.00	34.86
40.00	-	-	-	33.35	54.00	0.00	33.35
42.00	-	-	-	32.70	54.00	0.00	32.70
44.00	-	-	-	32.12	54.00	0.00	32.12
46.00	-	-	-	39.81	34.01	0.00	39.81
48.00	-	-	-	44.82	4.00	0.00	44.82
50.00	-	-	-	44.26	4.00	0.00	44.26

(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 ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.
49.95	0.67	1.00	0.67	2.82	4.00	44.27	100.00	0.000	0.0E0	0.000	0.000
48.00	0.68	1.00	0.68	2.81	4.00	44.82	100.00	0.000	0.0E0	0.000	0.000
46.00	0.69	1.00	0.69	2.80	34.01	39.81	100.00	0.000	0.0E0	0.000	0.000
44.00	0.69	1.00	0.69	2.81	54.00	32.12	95.07	0.000	0.0E0	0.000	0.000
42.00	0.69	1.00	0.69	2.82	54.00	32.70	96.51	0.000	0.0E0	0.000	0.000
40.00	0.70	1.00	0.70	2.84	54.00	33.35	98.17	0.000	0.0E0	0.000	0.000

38.00	0.70	1.00	0.70	2.87	49.20	34.86	100.00	0.000	0.0E0	0.000	0.000
36.00	0.70	1.00	0.70	2.90	48.00	35.86	100.00	0.000	0.0E0	0.000	0.000
34.00	0.70	1.00	0.70	2.94	48.00	36.72	100.00	0.000	0.0E0	0.000	0.000
32.00	0.70	1.00	0.70	2.99	41.20	37.67	100.00	0.000	0.0E0	0.000	0.000
30.00	0.69	1.00	0.69	3.05	14.00	33.65	98.97	0.000	0.0E0	0.000	0.000
28.00	0.68	1.00	0.68	3.15	26.80	31.33	93.14	0.000	0.0E0	0.000	0.000
26.00	0.66	1.00	0.66	3.26	30.00	30.15	90.39	0.000	0.0E0	0.219	0.219
24.00	0.64	1.00	0.64	0.52	58.40	26.63	82.90	1.590	9.5E-3	0.108	0.327
22.00	0.62	1.00	0.62	5.00	NoLiq	18.50	67.83	0.000	0.0E0	0.342	0.669
20.05	0.59	1.00	0.59	5.00	NoLiq	19.07	68.86	0.000	0.0E0	0.000	0.669

Settlement of Saturated Sands=0.669 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 ft	sigma' atm	sigC' atm	(N1)60s	CSRsf	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
20.00	1.16	0.76	19.09	0.59	1038.59	6.6E-4	0.7486	0.7833	1.01	0.7900	0.00E0	0.000	0.000
18.00	1.04	0.68	34.57	0.59	1196.01	5.2E-4	1.0000	0.4465	1.01	0.4503	5.40E-3	0.317	0.317
16.00	0.92	0.60	36.42	0.60	1142.06	4.8E-4	0.7778	0.3116	1.01	0.3143	3.77E-3	0.195	0.512
14.00	0.79	0.52	33.24	0.60	1031.86	4.6E-4	0.6217	0.2988	1.01	0.3013	3.62E-3	0.127	0.639
12.00	0.67	0.44	24.29	0.60	855.91	4.7E-4	0.7338	0.5606	1.01	0.5654	0.00E0	0.157	0.796
10.00	0.56	0.36	26.01	0.61	795.56	4.2E-4	0.3770	0.2629	1.01	0.2651	0.00E0	0.000	0.796
8.00	0.44	0.29	20.42	0.61	653.41	4.1E-4	1.0000	0.9584	1.01	0.9665	1.16E-2	0.345	1.141
6.00	0.33	0.21	20.59	0.61	564.25	3.5E-4	1.0000	0.9484	1.01	0.9564	1.15E-2	0.474	1.615
4.00	0.22	0.14	20.59	0.61	460.27	2.9E-4	1.0000	0.9484	1.01	0.9564	1.15E-2	0.391	2.006
2.00	0.11	0.07	20.59	0.62	325.47	2.1E-4	0.1056	0.1002	1.01	0.1010	0.00E0	0.319	2.325
0.00	0.00	0.00	20.59	0.62	3.12	2.0E-6	0.0010	0.0010	1.01	0.0010	0.00E0	0.000	2.325

Settlement of Unsaturated Sands=2.325 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=2.994 in.
Differential Settlement=1.497 to 1.976 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.

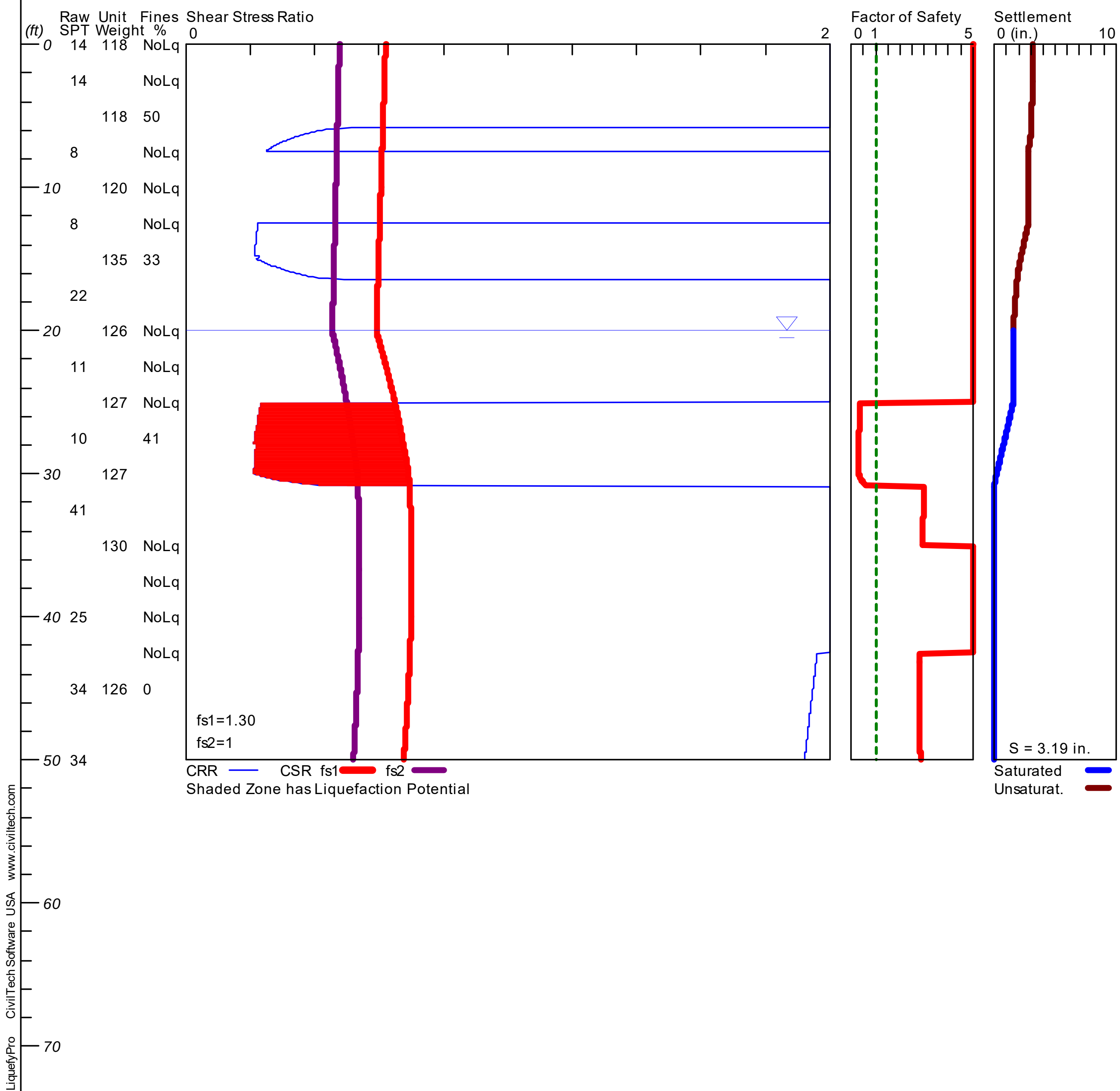
Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

LIQUEFACTION ANALYSIS

1600 W Commonwealth Ave

Hole No.=B-2 Water Depth=20 ft

Magnitude=7.3
Acceleration=0.734g



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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 ,  8/18/2022      3:32:32 PM

       Input File Name: Z:\OUR DOCUMENTS\Liquefaction Analysis\22-02182-1 B-2.liq
       Title:  1600 W Commonwealth Ave
       Subtitle:  22-02182

 Input Data:
       Surface Elev.=
       Hole No.=B-2
       Depth of Hole=50.00 ft
       Water Table during Earthquake= 20.00 ft
       Water Table during In-Situ Testing= 42.00 ft
       Max. Acceleration=0.73 g
       Earthquake Magnitude=7.30
       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
       8. Sampling Method,                                         Cs= 1.2
       9. User request factor of safety (apply to CSR) ,   User= 1.3
          Plot two CSR (fs1=User, fs2=1)
       10. Average two input data between two Depths: Yes*
       * Recommended Options

       In-Situ Test Data:
       Depth   SPT    Gamma  Fines
       ft             pcf    %
       __________________________________
       0.00    14.00  118.00 NoLiq
       2.50    14.00  118.00 NoLiq
       5.00    14.00  118.00 50.00
       7.50    8.00   118.00 NoLiq
       10.00   8.00   120.00 NoLiq
       12.50   8.00   120.00 NoLiq
       15.00   8.00   135.00 33.00
       17.50   22.00  135.00 33.00
       20.00   22.00  126.00 NoLiq
       22.50   11.00  126.00 NoLiq
       25.00   11.00  127.00 NoLiq
       27.50   10.00  127.00 41.00
       30.00   10.00  127.00 41.00
       32.50   41.00  127.00 41.00
       35.00   41.00  130.00 NoLiq
       37.50   41.00  130.00 NoLiq
       40.00   25.00  130.00 NoLiq
       42.50   25.00  130.00 NoLiq
       45.00   34.00  126.00 0.00
       47.50   34.00  126.00 0.00
       50.00   34.00  126.00 0.00
       __________________________________


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

       Peak Ground Acceleration (PGA), a_max = 0.73g

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

0.00	118.00	0.000	118.00	0.000	1.00	0.000	0.734	0.48	1.30	0.62
2.00	118.00	0.112	118.00	0.112	1.00	0.000	0.734	0.47	1.30	0.62
4.00	118.00	0.223	118.00	0.223	0.99	0.000	0.734	0.47	1.30	0.61
6.00	118.00	0.335	118.00	0.335	0.99	0.000	0.734	0.47	1.30	0.61
8.00	118.40	0.446	118.40	0.446	0.98	0.000	0.734	0.47	1.30	0.61
10.00	120.00	0.559	120.00	0.559	0.98	0.000	0.734	0.47	1.30	0.61
12.00	120.00	0.672	120.00	0.672	0.97	0.000	0.734	0.46	1.30	0.60
14.00	129.00	0.789	129.00	0.789	0.97	0.000	0.734	0.46	1.30	0.60
16.00	135.00	0.915	135.00	0.915	0.96	0.000	0.734	0.46	1.30	0.60
18.00	133.20	1.042	133.20	1.042	0.96	0.000	0.734	0.46	1.30	0.59
20.00	126.00	1.165	126.00	1.165	0.95	0.000	0.734	0.45	1.30	0.59
22.00	126.00	1.284	63.60	1.226	0.95	0.000	0.734	0.47	1.30	0.62
24.00	126.60	1.403	64.20	1.287	0.94	0.000	0.734	0.49	1.30	0.64
26.00	127.00	1.523	64.60	1.348	0.94	0.000	0.734	0.51	1.30	0.66
28.00	127.00	1.643	64.60	1.409	0.93	0.000	0.734	0.52	1.30	0.68
30.00	127.00	1.763	64.60	1.470	0.93	0.000	0.734	0.53	1.30	0.69
32.00	127.00	1.883	64.60	1.531	0.91	0.000	0.734	0.54	1.30	0.70
34.00	128.80	2.004	66.40	1.592	0.90	0.000	0.734	0.54	1.30	0.70
36.00	130.00	2.126	67.60	1.656	0.88	0.000	0.734	0.54	1.30	0.70
38.00	130.00	2.249	67.60	1.720	0.86	0.000	0.734	0.54	1.30	0.70
40.00	130.00	2.372	67.60	1.784	0.85	0.000	0.734	0.54	1.30	0.70
42.00	130.00	2.495	67.60	1.848	0.83	0.000	0.734	0.54	1.30	0.70
44.00	127.60	2.617	65.20	1.911	0.82	0.000	0.734	0.53	1.30	0.69
46.00	126.00	2.736	63.60	1.971	0.80	0.000	0.734	0.53	1.30	0.69
48.00	126.00	2.856	63.60	2.031	0.78	0.000	0.734	0.53	1.30	0.68
50.00	126.00	2.975	63.60	2.091	0.77	0.000	0.734	0.52	1.30	0.68

CSR is based on water table at 20.00 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	14.00	1.50	0.75	0.000	1.70	26.78	NoLiq	7.20	33.98	2.00
2.00	14.00	1.50	0.75	0.112	1.70	26.78	NoLiq	7.20	33.98	2.00
4.00	14.00	1.50	0.75	0.223	1.70	26.78	70.40	7.20	33.98	2.00
6.00	11.60	1.50	0.75	0.335	1.70	22.18	70.40	7.20	29.38	0.39
8.00	8.00	1.50	0.75	0.446	1.50	13.47	NoLiq	7.20	20.67	0.22
10.00	8.00	1.50	0.85	0.559	1.34	13.65	NoLiq	7.20	20.85	0.23
12.00	8.00	1.50	0.85	0.672	1.22	12.44	NoLiq	7.20	19.64	0.21
14.00	8.00	1.50	0.85	0.789	1.13	11.49	60.20	7.20	18.69	0.20
16.00	13.60	1.50	0.95	0.915	1.05	20.26	33.00	6.72	26.98	0.32
18.00	22.00	1.50	0.95	1.042	0.98	30.71	46.60	7.20	37.91	2.00
20.00	22.00	1.50	0.95	1.165	0.93	29.05	NoLiq	7.20	36.25	2.00
22.00	13.20	1.50	0.95	1.284	0.88	16.60	NoLiq	7.20	23.80	0.26
24.00	11.00	1.50	0.95	1.403	0.84	13.23	NoLiq	7.20	20.43	0.22
26.00	10.60	1.50	0.95	1.523	0.81	12.24	77.00	7.20	19.44	0.21
28.00	10.00	1.50	1.00	1.643	0.78	11.70	41.00	7.20	18.90	0.20
30.00	10.00	1.50	1.00	1.763	0.75	11.30	41.00	7.20	18.50	0.20
32.00	34.80	1.50	1.00	1.883	0.73	38.04	41.00	7.20	45.24	2.00
34.00	41.00	1.50	1.00	2.004	0.71	43.45	76.99	7.20	50.65	2.00
36.00	41.00	1.50	1.00	2.126	0.69	42.18	NoLiq	7.20	49.38	2.00
38.00	37.80	1.50	1.00	2.249	0.67	37.81	NoLiq	7.20	45.01	2.00
40.00	25.00	1.50	1.00	2.372	0.65	24.35	NoLiq	7.20	31.55	2.00
42.00	25.00	1.50	1.00	2.495	0.63	23.74	NoLiq	7.20	30.94	2.00
44.00	30.40	1.50	1.00	2.559	0.63	28.50	40.42	7.20	35.70	2.00
46.00	34.00	1.50	1.00	2.620	0.62	31.51	0.00	0.00	31.51	2.00
48.00	34.00	1.50	1.00	2.680	0.61	31.15	0.00	0.00	31.15	2.00
50.00	34.00	1.50	1.00	2.740	0.60	30.81	0.00	0.00	30.81	2.00

CRR is based on water table at 42.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 7.30:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	2.00	1.00	2.00	1.07	2.00	0.62	5.00 ^
2.00	0.07	2.00	1.00	2.00	1.07	2.00	0.62	5.00 ^
4.00	0.14	2.00	1.00	2.00	1.07	2.14	0.61	5.00
6.00	0.22	0.39	1.00	0.39	1.07	0.42	0.61	5.00
8.00	0.29	0.22	1.00	0.22	1.07	2.00	0.61	5.00 ^
10.00	0.36	0.23	1.00	0.23	1.07	2.00	0.61	5.00 ^

12.00	0.44	0.21	1.00	0.21	1.07	2.00	0.60	5.00 ^
14.00	0.51	0.20	1.00	0.20	1.07	0.22	0.60	5.00
16.00	0.59	0.32	1.00	0.32	1.07	0.34	0.60	5.00
18.00	0.68	2.00	1.00	2.00	1.07	2.14	0.59	5.00
20.00	0.76	2.00	1.00	2.00	1.07	2.14	0.59	5.00
22.00	0.83	0.26	1.00	0.26	1.07	2.00	0.62	5.00 ^
24.00	0.91	0.22	1.00	0.22	1.07	2.00	0.64	5.00 ^
26.00	0.99	0.21	1.00	0.21	1.07	0.22	0.66	0.34 *
28.00	1.07	0.20	1.00	0.20	1.07	0.22	0.68	0.32 *
30.00	1.15	0.20	0.98	0.20	1.07	0.21	0.69	0.30 *
32.00	1.22	2.00	0.97	1.94	1.07	2.08	0.70	2.99
34.00	1.30	2.00	0.96	1.92	1.07	2.06	0.70	2.94
36.00	1.38	2.00	0.95	1.90	1.07	2.00	0.70	5.00 ^
38.00	1.46	2.00	0.94	1.88	1.07	2.00	0.70	5.00 ^
40.00	1.54	2.00	0.93	1.85	1.07	2.00	0.70	5.00 ^
42.00	1.62	2.00	0.92	1.83	1.07	2.00	0.70	5.00 ^
44.00	1.66	2.00	0.91	1.82	1.07	1.95	0.69	2.82
46.00	1.70	2.00	0.91	1.81	1.07	1.94	0.69	2.82
48.00	1.74	2.00	0.90	1.80	1.07	1.93	0.68	2.83
50.00	1.78	2.00	0.90	1.79	1.07	1.92	0.68	2.84

* 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	-	-	-	33.98	NoLiq	0.00	33.98
2.00	-	-	-	33.98	NoLiq	0.00	33.98
4.00	-	-	-	33.98	70.40	0.00	33.98
6.00	-	-	-	29.38	70.40	0.00	29.38
8.00	-	-	-	20.67	NoLiq	0.00	20.67
10.00	-	-	-	20.85	NoLiq	0.00	20.85
12.00	-	-	-	19.64	NoLiq	0.00	19.64
14.00	-	-	-	18.69	60.20	0.00	18.69
16.00	-	-	-	26.98	33.00	0.00	26.98
18.00	-	-	-	37.91	46.60	0.00	37.91
20.00	-	-	-	36.25	NoLiq	0.00	36.25
22.00	-	-	-	23.80	NoLiq	0.00	23.80
24.00	-	-	-	20.43	NoLiq	0.00	20.43
26.00	-	-	-	19.44	77.00	0.00	19.44
28.00	-	-	-	18.90	41.00	0.00	18.90
30.00	-	-	-	18.50	41.00	0.00	18.50
32.00	-	-	-	45.24	41.00	0.00	45.24
34.00	-	-	-	50.65	76.99	0.00	50.65
36.00	-	-	-	49.38	NoLiq	0.00	49.38
38.00	-	-	-	45.01	NoLiq	0.00	45.01
40.00	-	-	-	31.55	NoLiq	0.00	31.55
42.00	-	-	-	30.94	NoLiq	0.00	30.94
44.00	-	-	-	35.70	40.42	0.00	35.70
46.00	-	-	-	31.51	0.00	0.00	31.51
48.00	-	-	-	31.15	0.00	0.00	31.15
50.00	-	-	-	30.81	0.00	0.00	30.81

(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 ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.
49.95	0.68	1.00	0.68	2.84	0.00	30.82	91.93	0.000	0.0E0	0.000	0.000
48.00	0.68	1.00	0.68	2.83	0.00	31.15	92.72	0.000	0.0E0	0.000	0.000
46.00	0.69	1.00	0.69	2.82	0.00	31.51	93.57	0.000	0.0E0	0.000	0.000
44.00	0.69	1.00	0.69	2.82	40.42	35.70	100.00	0.000	0.0E0	0.000	0.000
42.00	0.70	1.00	0.70	5.00	NoLiq	30.94	92.22	0.000	0.0E0	0.000	0.000
40.00	0.70	1.00	0.70	5.00	NoLiq	31.55	93.67	0.000	0.0E0	0.000	0.000

38.00	0.70	1.00	0.70	5.00	NoLiq	45.01	100.00	0.000	0.0E0	0.000	0.000
36.00	0.70	1.00	0.70	5.00	NoLiq	49.38	100.00	0.000	0.0E0	0.000	0.000
34.00	0.70	1.00	0.70	2.94	76.99	50.65	100.00	0.000	0.0E0	0.000	0.000
32.00	0.70	1.00	0.70	2.99	41.00	45.24	100.00	0.000	0.0E0	0.000	0.000
30.00	0.69	1.00	0.69	0.30	41.00	18.50	67.82	2.328	1.4E-2	0.191	0.191
28.00	0.68	1.00	0.68	0.32	41.00	18.90	68.56	2.284	1.4E-2	0.553	0.745
26.00	0.66	1.00	0.66	0.34	77.00	19.44	69.53	2.224	1.3E-2	0.550	1.295
24.00	0.64	1.00	0.64	5.00	NoLiq	20.43	71.32	0.000	0.0E0	0.250	1.544
22.00	0.62	1.00	0.62	5.00	NoLiq	23.80	77.46	0.000	0.0E0	0.000	1.544
20.05	0.59	1.00	0.59	5.00	NoLiq	35.92	100.00	0.000	0.0E0	0.000	1.544

Settlement of Saturated Sands=1.544 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 ft	sigma' atm	sigC' atm	(N1)60s	CSRsf	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
20.00	1.16	0.76	36.25	0.59	1286.26	5.4E-4	0.2811	0.1138	1.01	0.1147	0.00E0	0.000	0.000
18.00	1.04	0.68	37.91	0.59	1234.99	5.0E-4	1.0000	0.3645	1.01	0.3676	4.41E-3	0.176	0.176
16.00	0.91	0.59	26.98	0.60	1033.19	5.3E-4	1.0000	0.6632	1.01	0.6689	8.03E-3	0.227	0.403
14.00	0.79	0.51	18.69	0.60	848.85	5.6E-4	1.0000	1.0756	1.01	1.0848	1.30E-2	0.463	0.865
12.00	0.67	0.44	19.64	0.60	796.78	5.1E-4	1.0000	1.0082	1.01	1.0167	0.00E0	0.369	1.234
10.00	0.56	0.36	20.85	0.61	741.00	4.6E-4	0.5826	0.5436	1.01	0.5482	0.00E0	0.000	1.234
8.00	0.45	0.29	20.67	0.61	660.30	4.1E-4	1.0000	0.9432	1.01	0.9512	0.00E0	0.000	1.234
6.00	0.33	0.22	29.38	0.61	642.84	3.2E-4	0.8200	0.4815	1.01	0.4856	5.83E-3	0.263	1.497
4.00	0.22	0.14	33.98	0.61	550.87	2.5E-4	1.0000	0.4617	1.01	0.4656	5.59E-3	0.073	1.571
2.00	0.11	0.07	33.98	0.62	389.53	1.8E-4	0.0408	0.0188	1.01	0.0190	0.00E0	0.078	1.649
0.00	0.00	0.00	33.98	0.62	3.69	1.7E-6	0.0010	0.0005	1.01	0.0005	0.00E0	0.000	1.649

Settlement of Unsaturated Sands=1.649 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=3.193 in.
Differential Settlement=1.597 to 2.108 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).

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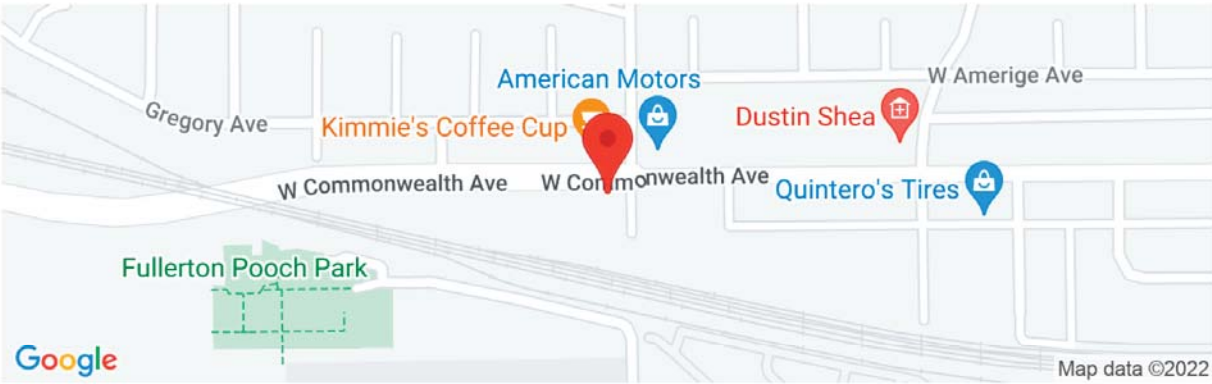
1600 West Commonwealth Avenue

1600 W Commonwealth Ave, Fullerton, CA 92833, USA

Latitude, Longitude: 33.8699893, -117.9509524



Date	8/12/2022, 3:47:55 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.565	MCE_R ground motion. (for 0.2 second period)
S_1	0.552	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.565	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.043	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.668	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.734	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.565	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.719	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.37	Factored deterministic acceleration value. (0.2 second)
S1RT	0.552	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.606	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.793	Factored deterministic acceleration value. (1.0 second)
PGAd	0.956	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.668	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration

Type	Value	Description
C_{RS}	0.91	Mapped value of the risk coefficient at short periods
C_{R1}	0.91	Mapped value of the risk coefficient at a period of 1 s
C_V	1.413	Vertical coefficient

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 (u...

Latitude

Decimal degrees

33.8699893

Longitude

Decimal degrees, negative values for western longitudes

-117.9509524

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

Time Horizon

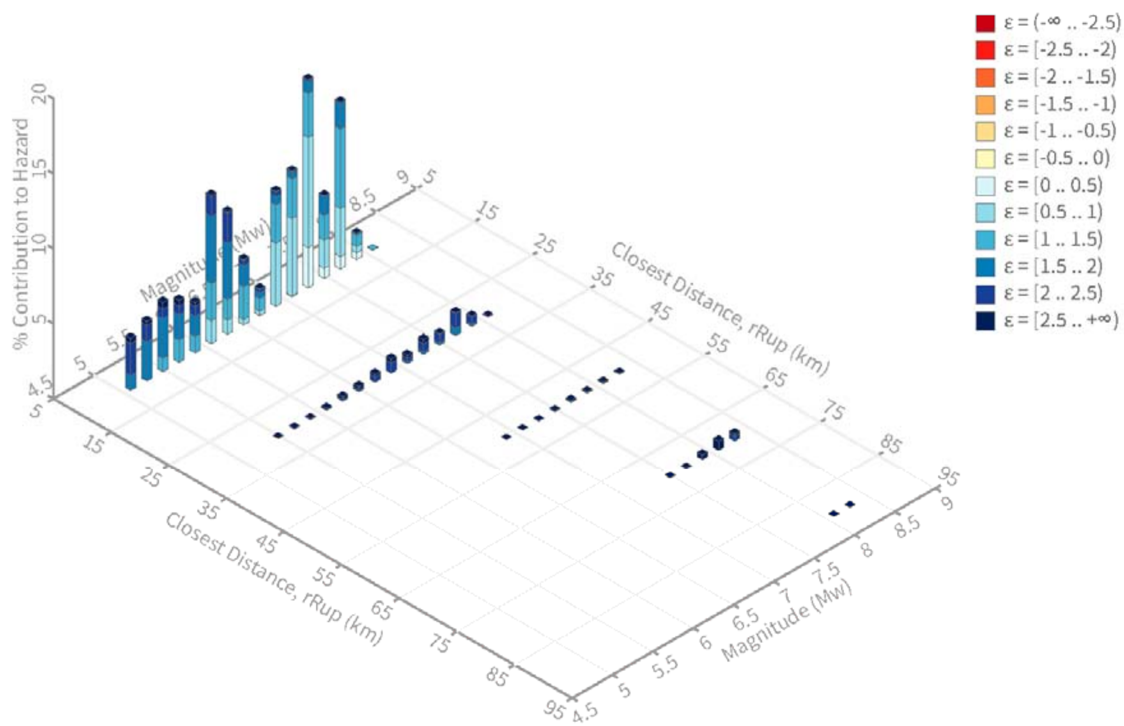
Return period in years

2475

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.73276523 g

Recovered targets

Return period: 2941.8482 yrs
Exceedance rate: 0.00033992237 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.06 %

Mean (over all sources)

m: 6.73
r: 11.55 km
ε_0: 1.4 σ

Mode (largest m-r bin)

m: 7.3
r: 10.69 km
ε_0: 0.8 σ
Contribution: 13.83 %

Mode (largest m-r-ε_0 bin)

m: 7.29
r: 9.73 km
ε_0: 0.67 σ
Contribution: 7.32 %

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							37.78
Puente Hills (Coyote Hills) [1]		4.86	7.24	0.80	117.954°W	33.895°N	355.11	9.72
Compton [0]		12.71	7.25	0.64	118.078°W	33.724°N	215.84	6.28
Whittier alt 2 [4]		9.81	7.39	1.33	117.915°W	33.952°N	20.20	4.74
Richfield [1]		8.37	6.26	1.60	117.870°W	33.882°N	79.87	3.33
Anaheim [1]		8.22	6.93	0.84	118.004°W	33.829°N	227.23	3.09
Newport-Inglewood alt 2 [3]		18.68	7.52	1.75	118.094°W	33.752°N	225.39	1.29
UC33brAvg_FM31	System							33.03
Whittier alt 1 [5]		9.86	7.22	1.40	117.911°W	33.952°N	21.84	6.98
Compton [0]		12.71	7.21	0.65	118.078°W	33.724°N	215.84	6.01
Anaheim [1]		8.22	6.88	0.87	118.004°W	33.829°N	227.23	3.00
Peralta Hills [1]		6.55	6.91	1.22	117.885°W	33.854°N	106.45	2.71
Puente Hills [0]		10.04	7.17	1.25	117.946°W	33.944°N	3.27	1.75
Newport-Inglewood alt 1 [2]		18.81	7.54	1.73	118.089°W	33.746°N	222.95	1.74
Whittier alt 1 [6]		9.98	6.21	1.91	117.931°W	33.957°N	10.62	1.53
Yorba Linda [0]		6.05	7.40	0.72	117.889°W	33.858°N	102.53	1.20
Elysian Park (Lower CFM) [0]		10.47	6.29	1.06	117.951°W	33.868°N	178.02	1.00
UC33brAvg_FM32 (opt)	Grid							15.42
PointSourceFinite: -117.951, 33.919		7.39	5.67	1.62	117.951°W	33.919°N	0.00	5.09
PointSourceFinite: -117.951, 33.919		7.39	5.67	1.62	117.951°W	33.919°N	0.00	5.09
PointSourceFinite: -117.951, 33.955		9.88	5.87	1.88	117.951°W	33.955°N	0.00	1.47
PointSourceFinite: -117.951, 33.955		9.88	5.87	1.88	117.951°W	33.955°N	0.00	1.47
UC33brAvg_FM31 (opt)	Grid							13.77
PointSourceFinite: -117.951, 33.919		7.42	5.65	1.64	117.951°W	33.919°N	0.00	4.27
PointSourceFinite: -117.951, 33.919		7.42	5.65	1.64	117.951°W	33.919°N	0.00	4.27
PointSourceFinite: -117.951, 33.955		9.86	5.88	1.87	117.951°W	33.955°N	0.00	1.50
PointSourceFinite: -117.951, 33.955		9.86	5.88	1.87	117.951°W	33.955°N	0.00	1.50

APPENDIX IV

REFERENCES

1. Bowles, Joseph, E., Foundation Analysis and Design (McGraw-Hill, New York: 1988).
2. 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.
3. California Department of Conservation, Division of Mines and Geology, April 15, 1998, State of California Seismic Hazard Zones Map of the Anaheim Quadrangle.
4. California Department of Conservation, Division of Mines and Geology, 1997, Seismic Hazard Zone Report for the Anaheim 7.5 Minute Quadrangle, Los Angeles County, California. Seismic Hazard Zone Report 03.
5. Monahan, Edward J., PE, Construction of and on Compacted Fills (Wiley & Sons, New York: 1986).
6. Naval Facilities Engineering Command Foundations and Earth Structures - Design Manual 7.02 (Naval Publications and Forms Center, Philadelphia: 1986).
7. Taylor, Donald W., Fundamentals of Soil Mechanics (Wiley & Sons, New York: 1948).
8. Terzaghi, Karl, Peck, Ralph B., Mesri, Gholamreza, Soil Mechanics in Engineering Practice (Wiley & Sons, New York: 1996).

GeoConcepts INC
Geology - Geotechnical Engineering

December 7, 2022 Project 22-02182

Meta Housing Corporation
11150 West Olympic Boulevard
Los Angeles, CA 90064

Subject: **SUPPLEMENTAL REPORT No. 1**
1600 W Commonwealth Ave
Fullerton, California

References:

1) Geotechnical Review Sheet by LOR Geotechnical Group, Inc. for the City of Fullerton, dated October 27, 2022.
2) Preliminary Geology and Geotechnical Engineering Investigation report by GeoConcepts, Inc. covering the subject site dated August 23, 2022.

Dear Meta Housing Corporation:

Pursuant to your request, presented herein is a response to Reference 1. A copy of the review sheet is attached. To facilitate the review, the following responses are provided per the review letter:

Review Comment Responses:

Item #1: It is our understanding that the structural engineer of record will design the structure to meet the requirements of the exception noted on Section 11.4.8 of ASCE 7-16.

Item #2: Based on the distance of the subject site to the ocean, the potential for tsunami inundation is nil.

Item #3: A site specific ground motion hazard analysis is not anticipated to be conducted; therefore, the liquefaction analysis remains applicable.

Item #4: It is generally accepted that structures may be designed for settlement limits of 4 inches total settlement and 2 inches differential settlement, which includes static and seismic settlements. Based on the calculated seismic settlements and the recommended static settlements, it is anticipated that the total will be less than the limits mentioned previously. Typically, mat foundations are used when settlements increase past 1.5 to 2 inches of total settlement and 0.75 to 1 inch of differential settlement. Foundation designs for conventional and mat foundations were provided previously and may be used by the project structural engineer.

Item #5: The mention of suitability in the Grading and Earthwork section is for suitability of the fill after grading, not for the onsite soils to support the recommended structural fill and foundations. Maximum density testing and corresponding density tests during grading are recommended with a compilation of testing in a compaction report to properly assess the suitability of the future compacted fill blanket.

Item #6: Based on lab testing and geologic observation of materials by the onsite engineer, the soils are anticipated to be primarily silty/sandy soils. A hydrometer test can be conducted by this office prior to grading in order to better understand the compaction effort required or the entire site can be graded to a 95% maximum density.

Item #7: Based on lab testing and geologic observation of materials by the onsite engineer, the soils are anticipated to be primarily silty/sandy soils. An expansion index test can be conducted prior to grading to confirm expansion potential of the soils and ensure proper moisture control of the fill.

Item #8: The Cal-OSHA type for the onsite soils is C.

Should you have any questions regarding this report, please do not hesitate to contact the undersigned at your convenience.

Respectfully submitted,
GeoConcepts, Inc.

LICENSED PROFESSIONAL ENGINEER
RAFFI HRAG DERMENDJIAN
C 88261
CIVIL
STATE OF CALIFORNIA

Raffi Dermendjian
Project Engineer
PE C. 88261
RD: 22-02182-3

Enclosures: Geotechnical Review Sheet by the City of Fullerton

Distribution: (1) Addressee

City of Fullerton Entitlement PC#1	Proposed Apartment Building 1600 West Commonwealth Avenue Fullerton, California
Submittal:	GeoConcepts, Inc. Preliminary Geotechnical Engineering Investigation Proposed Apartment Building 1600 West Commonwealth Avenue Fullerton, California Project 22-02182 dated August 23, 2022
Reviewed By:	John P. Leuer, LOR Geotechnical Group, Inc.
Review Discipline:	Geotechnical
Date:	October 27, 2022
LOR Project No:	63628BW.1

Comment No.	Submittal Section	Comments
1	Ground Shaking	The seismic design should be conducted in accordance with ASCE 7-16 and the 2019 CBC. As noted in Section 11.4.8 of ASCE 7-16, a site-specific ground motion hazard analysis is required unless an exception applies. Justification for use of an exception should be provided. If an exception is being used in lieu of conducting a site-specific ground motion hazard analysis, the structural engineer of record must agree to such.
2	Tsunami & Seiches	Please provide a definitive statement on the potential for tsunami inundation to occur at the site.
3	Liquefaction	If a site-specific ground motion hazard analysis is conducted, the liquefaction analysis should be re-evaluated based on the resultant PGAm.
4	Seismically Induced Settlement	Provide accepted limits of seismically induced settlement. Discuss if settlement can be reduced by means of geotechnical means and/or structural design, if required.
5	Grading and Earthwork	Please provide a quantitative definition of suitable/satisfactory soils to support the recommended structural fill and foundations (i.e. unit weight, in-place relative compaction, etc.).
6	Flatland Grading, 5.	Please clarify the compaction standard for the materials on this project and how it can be implemented. No analysis was run to determine the percent finer than 0.005 millimeters (i.e. soils with less than 15 percent clay).

7	Expansive Soils	Please clarify the expansion potential of the soils. No expansion testing was conducted.
8	Excavations	Please provide the Cal-OSHA soil type.