APPENDIX B

Preliminary Geotechnical Engineering Investigation

GeoConcepts INC
Geology - Geotechnical Engineering

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed 13-Story Hotel Expansion
Over Partial One Level Subgrade Parking
&
22-Story Residential Building
Over 3-4 Levels Subgrade Parking

1246 S. Hope Street
&
427 W. Pico Boulevard
Los Angeles, California

for

Five Chairs

Attn: Richard Heyman
1605 Cahuenga Blvd.
Los Angeles, CA 90028

Project 5076

March 31, 2017

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

TABLE OF CONTENTS

APPENDICES

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 Steinberg.
- Review of 4 borings. Explorations were backfilled with the excavated materials but not compacted.
- Preparation of the enclosed Plot Map and Cross Sections, (see Appendix I).
- Sampling of representative earth materials, laboratory testing, and engineering analyses (see Appendix II).
- Review of referenced materials and available public reports at the City of Los Angeles (see Appendix V).
- Presentation of findings, conclusions, and recommendations for the proposed project.

A general plot map was prepared from data collected from NavigateLA and utilized as a base map for this investigation. Preliminary design concept by Steinberg was used a basis for the preliminary recommendations.

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 13-story hotel expansion and basement expansion. A portion of the 13-story hotel expansion will be over the basement expansion and remaining portion at grade. In addition a 22-story residential building surrounded by 2 two-story podium all underlain by 3-4 levels of subgrade parking. The proposed development is depicted on the enclosed Plot Map and Cross-Sections.

The proposed hotel expansion will be supported on conventional foundations with anticipated foundations will range from 10 to 15 kips per lineal foot and 600-800 kips for column foundations

The proposed 22-story residential building will be supported on mat foundation. It is anticipated that the mat will impart pressures between 6,000 & 10,000 pounds per square foot (psf) on the underlying soil. The surrounding podium structure may be supported on conventional foundations with anticipated foundations will range from 10 to 15 kips per lineal foot and 500-700 kips for column foundations

Grading will consist of excavation for the subgrade parking and retaining wall backfilling. 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 Pico Boulevard and Hope Street (see Location Map in Appendix I). The property is bounded to the west by Hope Street, to the east by an alley, to the south by Pico Boulevard and north by commercial building. The southern portion of the property is developed with a 4-story hotel with a partial basement and surface parking lot. The central and northern portions of the site are development with one to two commercial buildings.

Drainage

Surface water at the site consists of direct precipitation onto the property.

Groundwater

No active surface groundwater seeps or springs were observed on the subject site. The subsurface exploration did encounter groundwater seepage at a depth of 157.5 & 187.5 feet. The depth to seeps/perched water 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 greater than 100 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 design concepts by Steinberg available at the time of the exploration and was limited due to the existing portion of the site. The locations of the explorations are depicted on the Plot Map and Cross Sections.

The field exploration of the site was conducted on November 6, December 30 and 31, 2015 and February 9, 2017. The geotechnical conditions were mapped by a representative of this office (refer to Exploration Logs). Subsurface exploration was performed by a hollow stem drill rig excavating into the underlying earth materials. Explorations were excavated to a maximum depth of 201 feet. Casing was placed within Boring 1 and the annular space around the casing was backfilled with bentonite slurry. The remaining explorations were backfilled and tamped upon completion of drilling. 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 an auto-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

Previous Work

No geology and/or geotechnical reports were found on file at the City of Los Angeles covering the sites.

Stratigraphy

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 materials were presumably placed during past grading. Fill was encountered in all of the borings. Fill generally consists of coarse grained silty sand.

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 brown to yellowish brown to gray, dense to very dense, silty sand to gravelly sands. These deposits were encountered within all three of the exploratory borings ranging to the depth of the exploration.

Excavation Characteristics

Subsurface exploration was performed through the use of a hollow-stem drill rig excavating into the fill and alluvium. 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

Earthquake Faults

The Alquist-Priolo Earthquake Fault Zoning (AP) Act was passed into law following the destructive February 9, 1971 San Fernando earthquake. The intent of the Act is to increase public safety by reducing the siting of most structures for human occupancy across an active fault. The Act only addresses the hazard of surface fault rupture and is not directed toward other earthquake hazards. The property is not located within an Alquist-Priolo Earthquake Fault Zone. The general locations of major faults within Southern California are depicted on a fault map provided by the USGS in Appendix I.

Active Faults

The following active faults are capable of producing seismic waves (ground shaking) on the subject property. A summary description of the closest active faults and potentially active faults to the site are described herein and labeled by number on the map below. An active fault, as defined by the State Mining and Geology Board, is one, which has “had surface displacement within Holocene time (about the last 11,000 years)”.

The San Andreas Fault zone (42) is the dominant active fault in California. Geologic studies show that over the past 1,400 to 1,500 years large earthquakes have occurred at about 150-year intervals on the southern San Andreas Fault. It consists of numerous subparallel faults of varied lengths in a zone generally 0.3 to 1.5 km wide in Southern California. The dip of the fault is near vertical and the sense of motion is right lateral. Historically, the 1857 Fort Tejon earthquake with an estimated magnitude of 7.9 ruptured the ground surface from the vicinity of Cholame (near Paso Robles) to somewhere between the Cajon Pass and San Gorgonio Pass (Wrightwood), approximately 200 miles. Studies of offset stream channels indicate that as much as (29) feet of movement occurred in 1857. The fault extends from the Gulf of California northward to the Cape Mendocino area where it continues along the ocean floor, approximately 750 miles in length.

The Northridge earthquake occurred on January 17, 1994, in the San Fernando Valley. The epicenter was about 1 mile south-southwest of Northridge at a focal depth of 12 miles. The surface wave magnitude was issued by the National Earthquake Information Center at Mw=6.7. This event occurred on a previously unrecognized south-dipping blind reverse fault without surface rupture. This earthquake produced the strongest ground motions ever instrumentally recorded in an urban setting in North America. Damage was wide-spread with sections of major freeways collapsed include some parking structures and office buildings. Common surface disruptions included buckled curbs and sidewalks, fissured concrete and asphalt, and rupture of utility lines which are generally aligned in northwest and east-west directions. Shattered ridges were reported along Mulholland Drive in the Sherman Oaks area, consisting of intense ground disturbances associated with strong vibratory ground motions within the north trending ridges underlain by shale of the Lower Modelo formation.

The Whittier-Elsinore fault zone (60) consists of several subparallel, overlapping and en echelon fault strands in a zone up to 1.2 km wide. It extends nearly 125 miles from the Mexican border to the northern edge of the San Fernando Valley. Seismicity includes the Whittier Narrows earthquake of October 1, 1987 with a magnitude of 5.9 and an epicenter in the city of Rosemead. This earthquake occurred on a previously unknown and concealed thrust fault. There was no reported surface rupture from the earthquake. Also, numerous close and scattered small earthquakes have occurred in historic time near and along the fault.

The San Fernando fault (45) consists of five major en echelon strands at least 9.5 miles in length. The "San Fernando" earthquake of February 9, 1971 produced a magnitude of Mw 6.5 at a depth of 8.4 km along an east west trending reverse fault with a northerly dip. The length of the surface rupture was about 9.5 miles and ground shaking lasted for approximately 60 seconds. The earthquake ruptured the northwestern end of the Sierra Madre Fault zone forming the San Fernando Fault. Major damage included the Olive View and Veterans Administration Hospitals and collapse of freeway overpasses. Landslides occurred in the Upper Lake area of Van Norman Lakes. Additionally the Van Norman Dam and the Pacoima Dam were severely damaged.

The eastern portion of the Santa Susana fault (52) ruptured during the 1971 San Fernando Earthquake. The Santa Susana fault consists of several strands in a zone as wide as 1 km. It generally strikes from north 75 degrees west to north 50 degrees east and dips to the north. The fault is a high angle reverse fault. The fault appears to have been generated by northeast-southwest oriented compressional stress.

The Newport-Inglewood fault zone (31) consists of several strands that extend from offshore by Laguna Beach to either merge with or be truncated by the Malibu-Santa Monica fault zone near Beverly Hills. The fault has a length of about 45 miles. It was the source of the "Long Beach" earthquake, which occurred on March 10, 1933 with a magnitude of 6.3. Numerous small earthquakes have occurred in historic time along and near the fault zone. The fault zone is easily observed by an alignment of hills and mesas including Cheviot Hills, Baldwin Hills, Rosecrans Hills, Dominguez Hills, Signal Hill, Reservoir Hill, Alamitos Heights, Landing Hill, Bolsa Chica Mesa, and Newport Mesa.

In June 1995, two portions of the Malibu Coast fault zone (27) were reclassified as active fault zones by the State of California. On August 16, 2007, the fault zone near the east side of Malibu Bluff Park was removed from the State of California Earthquake Fault Zone map by the State of California. The east west trending Malibu Coast fault consists of several subparallel strands in a zone as wide as 0.5 km, with a length of at least 17 miles. It strikes east west and dips (45) to (80) degrees to the north. The Malibu Coast fault has the potential to produce a large Maximum Credible Peak and Repeatable Acceleration on the subject property. The duration of the Malibu Coast fault is estimated at (11) seconds assuming fault end nucleation and unidirectional rupture propagation, (Bolt, 1981). The Malibu Coast fault is thought to be part of other faults such as the Santa Monica fault and Hollywood fault that separate the Transverse Ranges on the north from the Peninsula Range on the south. Two Malibu Earthquakes occurred with Magnitudes of M_L 5.2 and M_L 5.0 on January 1, 1979 and January 18, 1989, respectively. It was reported that only minor damage occurred in the areas closest to the epicenter.

The Hollywood fault zone (22) extends along the base of the Santa Monica Mountains. This fault was added to the list of active fault by the State of California in 2014. Generally, the Hollywood fault extends eastward for a distance of 15 km through Beverly Hills, West Hollywood, and Hollywood to the Los Angeles River. The fault is primarily expressed at the ground surface by scarp-like features. This is a left–reverse fault with an estimated slip rate between 0.33 mm/yr and 0.75 mm/yr, (Petersen and Wesnousky 1994).

The Raymond fault (39) is a combination fault with reverse and left slip movement that acts as a groundwater barrier within the densely populated San Gabriel Valley. The activity of the fault is attested to by the numerous geomorphic features found along its entire length of approximately 14 miles. Scattered small earthquakes have occurred north of the fault trace. It may be the source of the 1855 Los Angeles earthquake. The Raymond fault is an east-trending fault made up of other faults such as the Hollywood and Santa Monica faults that separate the Transverse Ranges on the north form the Peninsula Range on the south.

The Sierra Madre fault zone (53) is often divided into five main segments; Vasquez Creek fault, Clamshell fault (10), Sawpit Canyon fault (10), Duarte fault and the Cucamonga fault (14). The Sierra Madre earthquake of June 28, 1991 (Mw5.8) was in the San Gabriel Mountains. An estimated 33.5 million dollars of damage has been reported. The Sierra Madre fault zone is about 75 km long. It’s a thrust fault system along the south edge of the San Gabriel Mountains. The east end of the Sierra Madre fault zone intersects the San Jacinto fault and the San Andreas Fault. The 1971 San Fernando earthquake occurred on the San Fernando-Sunland segment of the Sierra Madre fault zone.

The San Gabriel fault (46) consists of several en echelon fault strands in a zone approximately 0.5 km wide, with a length of about 90 miles. The fault trends northwestward and subparallel to the San Andreas Fault. As of March 1, 1988, a portion of the Newhall segment of the fault zone was reclassified as an active fault. Fault activity has been dated between 1550 and 3500 years before present within the Newhall segment. The youngest ground rupture event has broken alluvial beds to within five feet of the ground surface. Geologic evidence suggests 38 miles of right lateral offset has occurred between 14 million and 3 million years ago and may have functioned as an ancestral branch of the San Andreas Fault. Recent studies suggest that major strike slip movement has become inactive and dip slip movement is active at the present time.

Potentially Active Faults

A potentially active fault, as defined by the State Mining and Geology Board, is one, which has had surface displacement during Quaternary time (last 1.6 million years). "These faults are those based on available data along which no known historical ground surface ruptures or earthquakes have occurred. These faults, however, show strong indications of geologically recent activity". The following list provides potentially active faults that are capable of producing seismic waves (ground shaking) on the property.

The Santa Monica fault (50) extends east from the coastline in Pacific Palisades through Santa Monica and West Los Angeles and merges with the Hollywood fault. Several local geologists believe portions of the Santa Monica fault zone are active. Currently, it is listed by the State of California as a potentially active fault. Portions of the fault zone may change to "active" and be placed within the Alquist-Priolo Earthquake Fault Zone as additional geologic reports are submitted to the State containing evidence of Holocene activity. The Santa Monica fault consists of one or more fault strands, with a poorly known geometry. Generally, the fault strikes northeast 60 to 80 degrees and dips 45 to 65 degrees northwest at depth with a few near vertical surface traces. The length of the fault is at least 25 miles. The composite local mechanism of fault displacement is a reverse left lateral along the Santa Monica-Hollywood-Raymond fault zone. The Santa Monica and Hollywood faults may be part of a larger fault system that includes Malibu Coast, Raymond and Cucamonga fault system. This fault zone forms the central portion of a major tectonic boundary separating the east west trending Transverse Ranges province to the north from the northwest trending Peninsular Ranges province to the south.

The Benedict Canyon fault zone trends eastward through the Santa Monica Mountains. The fault may be part of the Hollywood-Santa Monica-Raymond fault system. The activity of the fault is based on offsets in groundwater bearing sediments that correlate with steep dipping gravity gradients. The fault extends through Universal City and along the north side of the eastern part of the Santa Monica Mountains.

The Simi fault (54) consists of a single strand that bifurcates at the western end. Generally, it strikes north 70-80 degrees east and dips 60 to 75 degrees north with a length of about 31-km.

The Mission Hills fault (30) is an east west trending fault with a length of about 9 km. The fault is presumed to be a single strand that strikes north 80 degrees east to east west and dips about 80 degrees to the north.

The Chatsworth fault (8) is a reverse fault which juxtaposes Cretaceous Chatsworth formation and Paleocene Martinez formation over Miocene Modelo formation within the San Fernando Valley.

The Palos Verdes Hills fault (35) consists of several en echelon strands locally in a zone as wide as 2 km with a length of 50 miles. It strikes north between 20 and 60 degrees west with dips of 70 degrees to the southwest.

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.

There are several active and/or potentially active faults that could possibly affect the site within Los Angeles County. 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 an 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 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 active and potentially active 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. The site is not located along the beach; therefore, there is very little potential for inundation of the site from a tsunami event.

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 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 not 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.

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, dense nature of alluvium, 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, dense nature of the alluvium, 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.

The soils encountered at the subject site consist of dense silty sand and sand with clay binder. Although the magnitude of the seismically induced settlement is not readily predicted, based upon the depth to groundwater, dense nature of alluvium, seismically induced settlement should not pose any significant hazard to the proposed development provided the recommendations contained within this report followed and maintained.

Seismic Velocity Measurements

Downhole seismic velocity measurements were performed by GeoPentech in Boring No. 1, which was drilled to a depth of 201 feet below the existing ground surface. The results are included within the Downhole Seismic Survey Results report by GeoPentech dated March 22, 2016. The soils from 0-100 feet were determined to have a V_s30 (ft/sec) of 1450 ft/sec.

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 not located within a liquefaction or earthquake-induced landslide hazard zone.

3. Based on the seismic velocity measurement the soils would be are considered very dense and should be classified as Site Class C.

4. Based upon field observations, laboratory testing and analysis, the alluvium found in the explorations at the proposed basement elevations should possess sufficient strength to support the development.

RECOMMENDATIONS

Specific

1. The proposed 13-story hotel expansion over partial one level subgrade parking should be supported on foundations embedded into dense alluvium.
2. The proposed 22-story residential building over 3-4 levels of subgrade parking should be supported on foundations embedded into the dense alluvium encountered at the basement elevation.
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 excavation for the proposed subgrade parking and retaining wall backfilling and foundation excavations.

Following the completion of the excavation, the subgrade soils should be evaluated by the project geotechnical engineer to verify their suitability 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 excavation.

Foundations

It is recommended that the proposed structure be founded into alluvium encountered at the proposed basement elevation.

Conventional

The minimum continuous footing size is (24) inches wide and (24) inches deep into the alluvium found at the basement elevation, measured from the lowest adjacent grade. Continuous footings may be proportioned, using a bearing value of (4200) pounds per square foot. Column footings placed into the alluvium at basement elevation may be proportioned, using a bearing value of (6000) pounds per square foot, and should be a minimum of (2) feet in width and (24) inches deep, below the lowest adjacent grade. The allowable bearing capacity presented above may be increased 20% for each additional foot of width or depth up to (10,000) pounds per square foot.

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.

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 into the recommended bearing materials will not be acceptable to this office.

Lateral loads may be resisted by friction at the base of the conventional foundations with a maximum embedment of (5) feet and by passive resistance within the alluvium. A coefficient of friction of (0.35) may be used between the foundations and the alluvium. The passive resistance may be assumed to act as a fluid with a density of (300) pounds per cubic foot. A maximum passive earth pressure of (3000) pounds per square foot may be assumed.

Mat Foundation Recommendations

The proposed structure may be supported on mat foundation system embedded into the alluvium. Rigid and flexible mat foundation design values are presented below:

Although foundation loads were not available at the time of this investigation, it is anticipated that the mat foundation load will range from 8,000 to 10,000 psf. It is anticipated that a mat foundation would be on the order of 5 feet thick.

Conventional rigid method:

The mat foundation may be proportioned using an average bearing value of (10,000) pounds per square foot. The mat foundation structural design should be done by the project structural engineer.

Approximate flexible method:

The coefficient of subgrade reaction of foundations measuring (1x1) square foot, k_1, may be taken as (300) lb/in^3. The mat foundation structural design should be done by the project 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.

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.

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 the appropriate standards and expert recommendations pertaining to vapor barriers/retarders, engineers (especially those aware of the issues surrounding below-slab moisture protection and its effects on the success of their projects) should consider recommending and citing specific performance characteristics. The following paragraph includes criteria from the latest standards and expert recommendations and should be considered for use in your firm's own recommendations:

Vapor barrier shall consist of a minimum 15 mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditions (ASTM E 1745 Section 7.1 and Sub-Paragraph 7.1.1-7.1.5): less than 0.01 perms [grains/(ft2-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 building supported on conventional foundations will occur. Settlement of (1/2) to (1) 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 (1.5) inches should be anticipated.

Settlement of proposed mat foundation is anticipated. Based on the current loading condition, settlements are estimated to range from (2.5) to (3.0) inches under the heavily-loaded center of the proposed mat foundation, and settlements are estimated to range from (1.0) to (1.5) inch under the edge of the proposed mat foundation.

Expansive Soils

Expansive soil was not encountered on the subject property that is anticipated affect the proposed development. 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 45 feet will be required for the subgrade parking. Conventional excavation equipment may be used to make these excavations. Excavations should expose alluvium. Shoring is anticipated to be required for all the excavations due to the adjacent structures, street and alley. This should be verified by the project geotechnical engineer during construction so that modifications can be made if variations in the soil occur.

Temporary Shoring

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

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

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation, may be assumed to be 300 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as0.4 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 450 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation.

The exploration was performed using a hollow stem drill rig and although no caving was detected it is difficult to detect caving in hollow stem boring. Casing may be required should caving be experienced in the saturated earth materials. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

Groundwater was not encountered during exploration, although seepage was encountered at depths of 157.5 & 187.5 feet below grade. Therefore, it is not anticipated that the proposed shoring piles will encounter water. If groundwater is encountered, piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 10 inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

Lagging

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

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of cantilevered shoring system. A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. Equivalent fluid pressures for the design of cantilevered and restrained shoring are presented in the following table:

Height of Shoring (feet)	Active Pressure Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (pcf) (At-Rest Pressure) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* (At-Rest Pressure) Trapezoidal Distribution of Pressure
15 feet	36	52	35H
35 feet	40	52	35H
45 feet	40	52	35H

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

Additional active pressures should be applied where the shoring will be surcharged by adjacent traffic or structures.

Tied-Back Anchors

Tie-back anchors may be used to resist lateral loads. Friction anchors consisting of high stress thread bars are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge and to greater lengths if necessary to develop the desired capacities.

Drilled friction anchors may be designed for a skin friction of 300 pounds per square foot. Pressure grouted anchor may be designed for a skin friction of 2,000 pounds per square foot. Where belled anchors are utilized, the capacity of belled anchors may be designed by assuming the diameter of the bonded zone is equivalent to the diameter of the bell. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated.

It is recommended that at least 3 of the initial anchors have their capacities tested to 200 percent of their design capacities for a 24-hour period to verify their design capacity. The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.

All anchors should be tested to at least 150 percent of design load. The total deflection during this test should not exceed 12 inches. The rate of creep under the 150 percent test load should not exceed 0.1 inch over a 15 minute period in order for the anchor to be approved for the design loading.

After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors be installed until satisfactory test results are obtained. The installation and testing of the anchors should be observed by a representative of this firm. Minor caving during drilling of the anchors should be anticipated.

Raker Braces

The proposed soldier piles may be laterally supported by raker braces supported by temporary footings, or dead-men. Temporary footings inclined at an angle of 45 degrees to the horizontal may be designed for an allowable bearing value of 1500 psf. To utilize this allowable bearing pressure, the inclined footings should be a minimum of 24 inches in width, and should be embedded a minimum of 24 inches below the lowest adjacent grade. An increase of 300 pounds per square foot may be utilized for each additional foot of width.

Deflection

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

Monitoring

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

Shoring Observations

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

<u>Excavations Maintenance – Erosion Control</u>

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.

Open Pile/Caisson Excavations:

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

Grading In Progress:

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

Additionally, it is suggested that all stock-piled fill materials be covered with plastic sheeting. This action will reduce the potential for the moisture content of the fill from becoming too high for compaction. If the fill stockpile is not covered during inclement weather, then aerating the fill to reduce the moisture content would be required. This action is generally very time consuming and may result in construction delays. Work may recommence, after the rain event, once the site has been reviewed by the project geotechnical engineer.

Retaining Walls

Cantilever retaining walls should be designed to resist an active earth pressure such as that exerted by compacted backfill. Retaining walls up to (45) feet in height may be designed per the following table. The ‘active’ pressure assumes that the wall will be allowed to deflect 0.01H to 0.02H. Basement walls and other walls where horizontal movement is restricted at the top or not allowed to deflect shall be designed for at-rest pressure.

Height of Retained Material (ft)	Active Equivalent Fluid Weight (p.c.f.)	At-Rest Pressure Fluid Weight (p.c.f.)
15	50	60
35	53	65
45	53	65

In addition to lateral earth pressure, these retaining walls should be designed to resist the surcharge imposed by the proposed structures, footings, any adjacent buildings, or by adjacent traffic surcharge.

The wall pressure stated assumes that the wall has been backfilled as outlined below with a permanent drainage system. Proper compaction of the backfill is recommended to provide lateral support to adjacent properties. Even with proper compaction of required backfill, settlement of the backfill may occur. Accordingly, utility lines, footings, slabs, or falsework should be planned and designed to accommodate potential settlement.

Walls to be backfilled must be reviewed by the project Geotechnical Engineer prior to commencement of the backfilling operation.

1. Adequate permanent drainage is required behind the wall to minimize the buildup of hydrostatic pressures. A perforated pipe, with perforations placed down, shall be installed at the base of the wall footing. The pipe shall be encased in at least one foot (1') of three-quarter inch (3/4") gravel. The pipe shall exit from behind the retaining wall and drain to a location approved by the architect or civil engineer.

   When space does not permit the installation of standard pipe and gravel drainage system, i.e. walls adjacent the property line, a flat drainage product is acceptable subject to approval of the governing agency. It is recommended that a drainage composite geotextile (such as MiraDrain / QuickDrain) be placed at the base of the proposed retaining wall. The drainage composite geotextile will provide comparable drainage to the conventional four inch perforated pipe encased in gravel per Code Sections 1805.4.2 and 1805.4.3.

   If a drainage system is not provided the walls should be designed to resist an external hydrostatic pressure due to water in addition to the lateral earth pressure in Retaining Wall section. The entire wall should be design for full hydrostatic pressure based on a water level at the ground surface. In addition, floors would need to be designed for hydrostatic uplift and waterproofed.

2. A continuous vertical drain, consisting of a gravel blanket six inches (6") thick or geotextile vertical drainage system, shall be placed along the back side of the wall to within 2 feet of the ground surface.

3. Water and moisture affecting retaining walls is a common post-construction complaint. Poorly applied or omitted waterproofing can lead to standing water inside the building or efflorescence on the wall.

   It is recommended that the retaining walls be waterproofed. Waterproofing design and inspection of installation is not the responsibility of the geotechnical engineer. 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.

4. After the wall backdrain system has been placed and the waterproofing installed, fill may be placed, if sufficient room allows, in layers not exceeding four inches (4") in thickness and compacted to 90 percent of the maximum density, as determined by ASTM D 1557. 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.
5. Where space does not permit compaction of material behind the wall (<24 inches wide), a granular backfill shall be used. This granular backfill shall consist of one-half inch (1/2") to three-quarter inch (3/4") crushed rock and should be densified by tamping into place. The crushed rock backfill should not exceed a depth of ten feet.
6. All granular free-draining wall backfills shall be capped with a clayey compacted soil within the upper two feet (2') of the wall backfill. This compacted material should start below the required wall freeboard.

Lateral Earth Pressure Due to Earth Motion

Cantilever retaining walls should be designed to resist an active earth pressure due to earth motion, if required by the building official, distributed as a triangle pressure. Retaining walls up to (45) feet in height may be designed per the following table. The seismic equivalent fluid pressure is in addition to static earth pressures. The seismic loading is based on a horizontal acceleration coefficient of 0.29 (one-half of two-thirds of PGA_m).

Surface Slope of Retained Material Horizontal to Vertical	Seismically Induced Earth Pressure - Equivalent Fluid Weight p.c.f.
Level	6
Level	8
Level	10

Surcharge from Adjacent Structures:

In addition to lateral earth pressure, the proposed shoring and retaining walls should be designed to resist the surcharge imposed by the proposed structures, footings, any adjacent buildings, or by adjacent traffic surcharge.

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 alluvium. Provisions for cracks should be incorporated into the design and construction of the foundation system, slabs, and proposed floor coverings. Concrete slabs should have sufficient control joints spaced at a maximum of approximately 8 feet. These recommendations are considered minimums unless superseded by the project structural engineer.

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 the appropriate standards and expert recommendations pertaining to vapor barriers/retarders, engineers (especially those aware of the issues surrounding below-slab moisture protection and its effects on the success of their projects) should consider recommending and citing specific performance characteristics. The following paragraph includes criteria from the latest standards and expert recommendations and should be considered for use in your firm's own recommendations:

Vapor barrier shall consist of a minimum 15 mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditions (ASTM E 1745 Section 7.1 and Sub-Paragraph 7.1.1-7.1.5): less than 0.01 perms [grains/(ft2-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.

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.

REGISTERED PROFESSIONAL ENGINEER
SCOTT JOHN WALTER
No.2476
GEOTECHNICAL
STATE OF CALIFORNIA

Scott J. Walter
Project Engineer
GE 2476
SJW/KNC/RMH/RD: 5076-4
Distribution: (3) Addressee

APPENDIX I

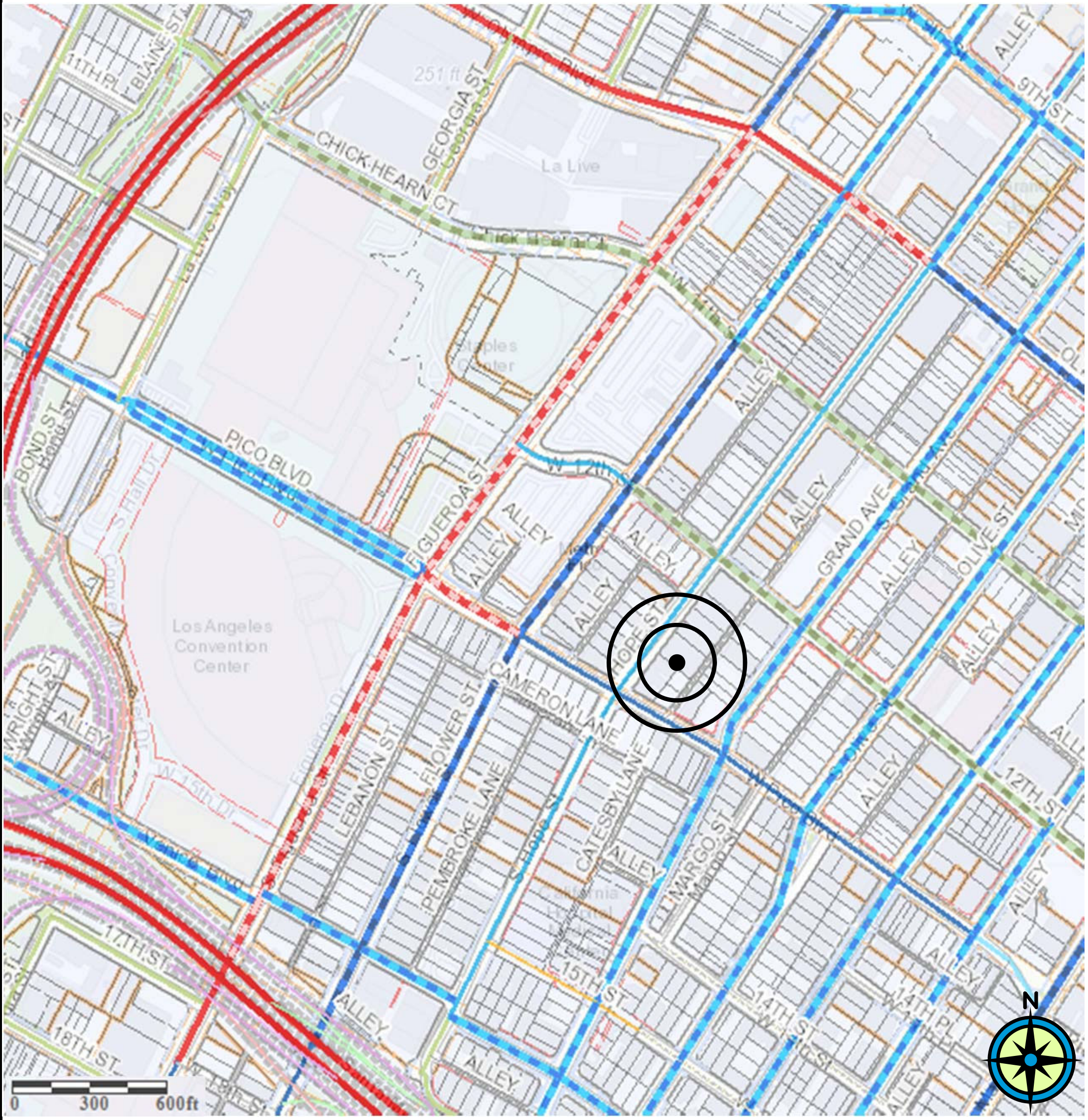
SITE INFORMATION

Location Map
Groundwater Map
Regional Geologic Map
USGS Fault Map
Seismic Hazard Map

Plot Map
Cross Sections

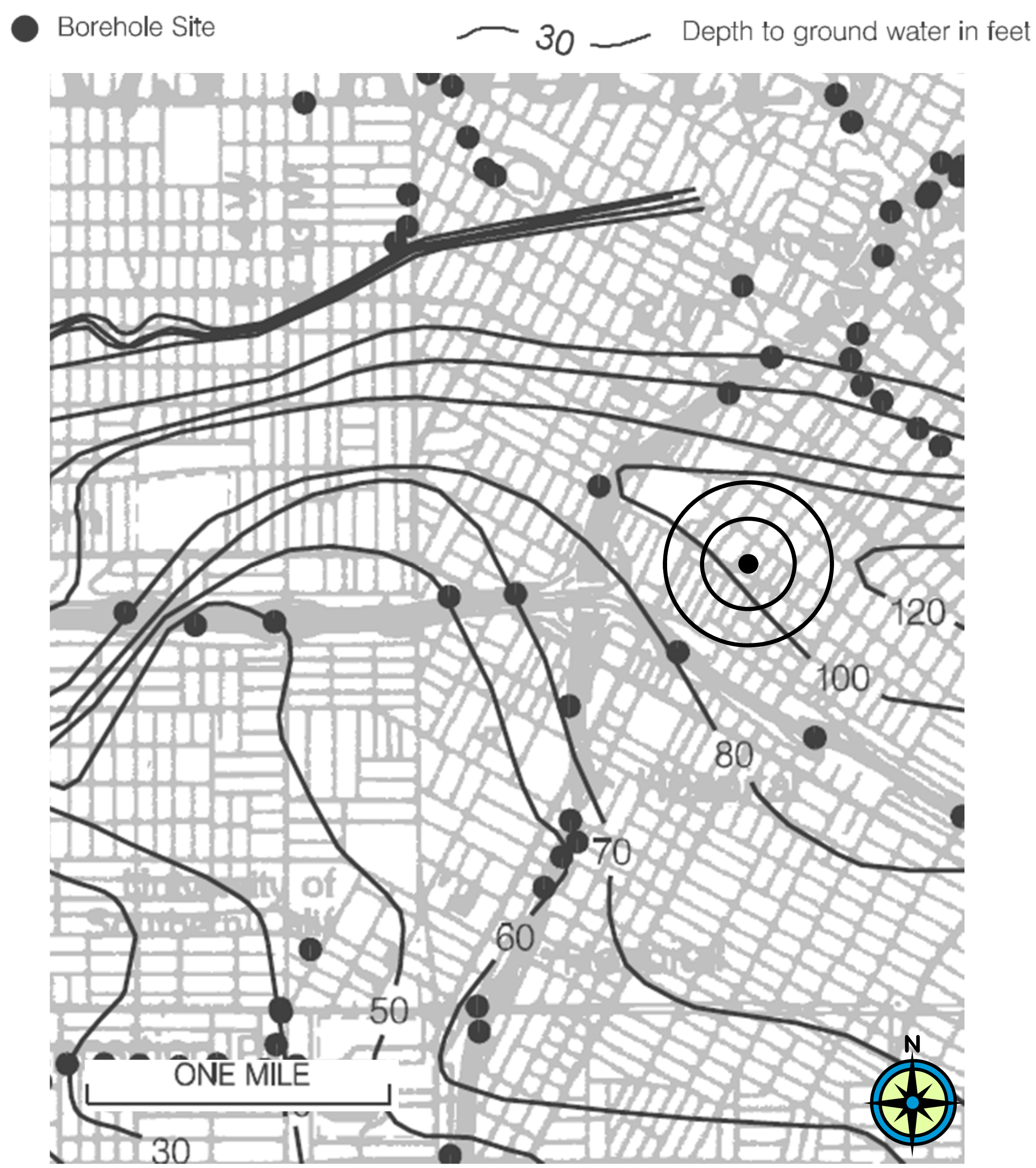
Field Exploration
Borings 1 through 4

LOCATION MAP



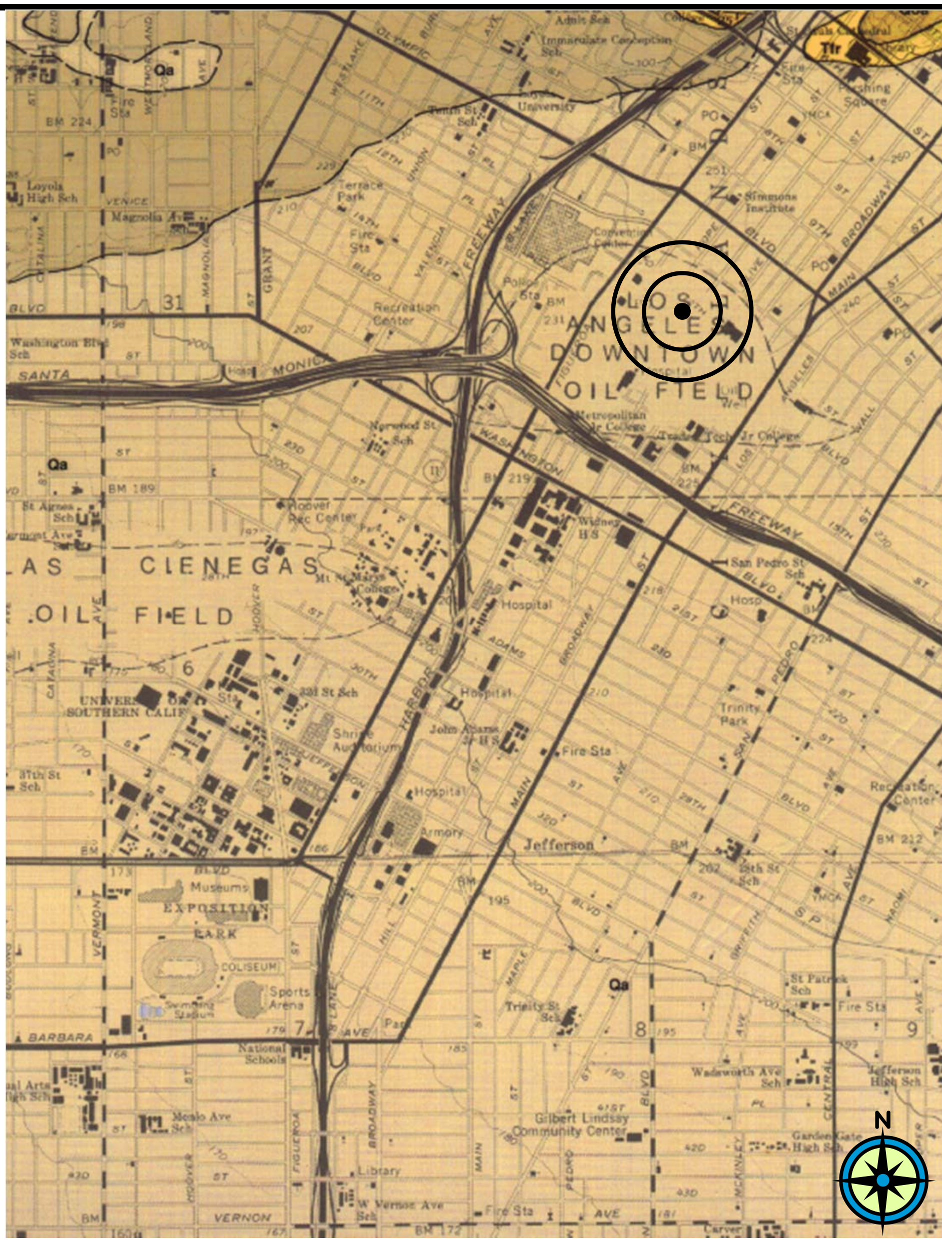
Reference:	City of Los Angeles, Bureau of Engineering, NavigateLA	Scale: As Shown

GROUNDWATER MAP



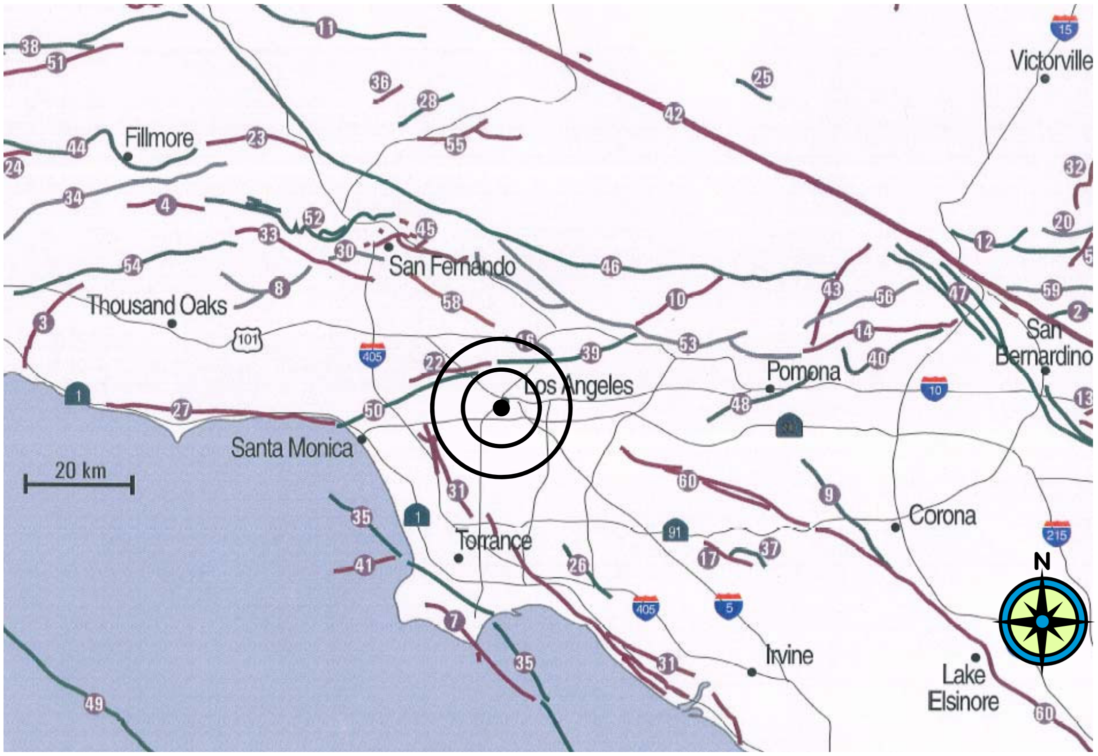
Reference:	State of California Seismic Hazard Report, Hollywood Quadrangle	Scale: As Shown

REGIONAL GEOLOGIC MAP



Reference:	Dibble Geologic Map, Hollywood Quadrangle	Scale: 1" = 5280'

USGS FAULT MAP

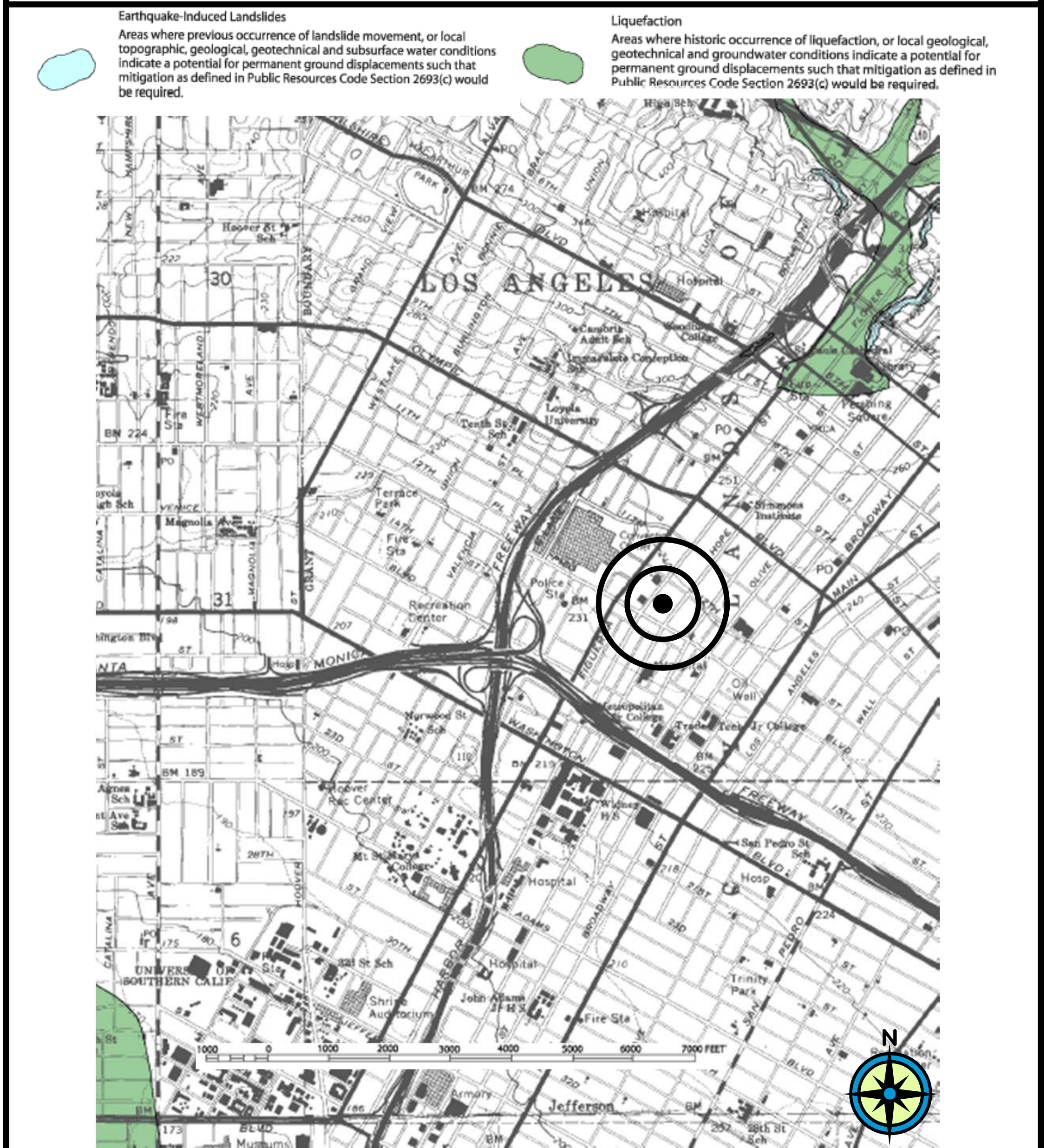


Lisa Wald, U.S. Geologic Survey (modified from SCEC).

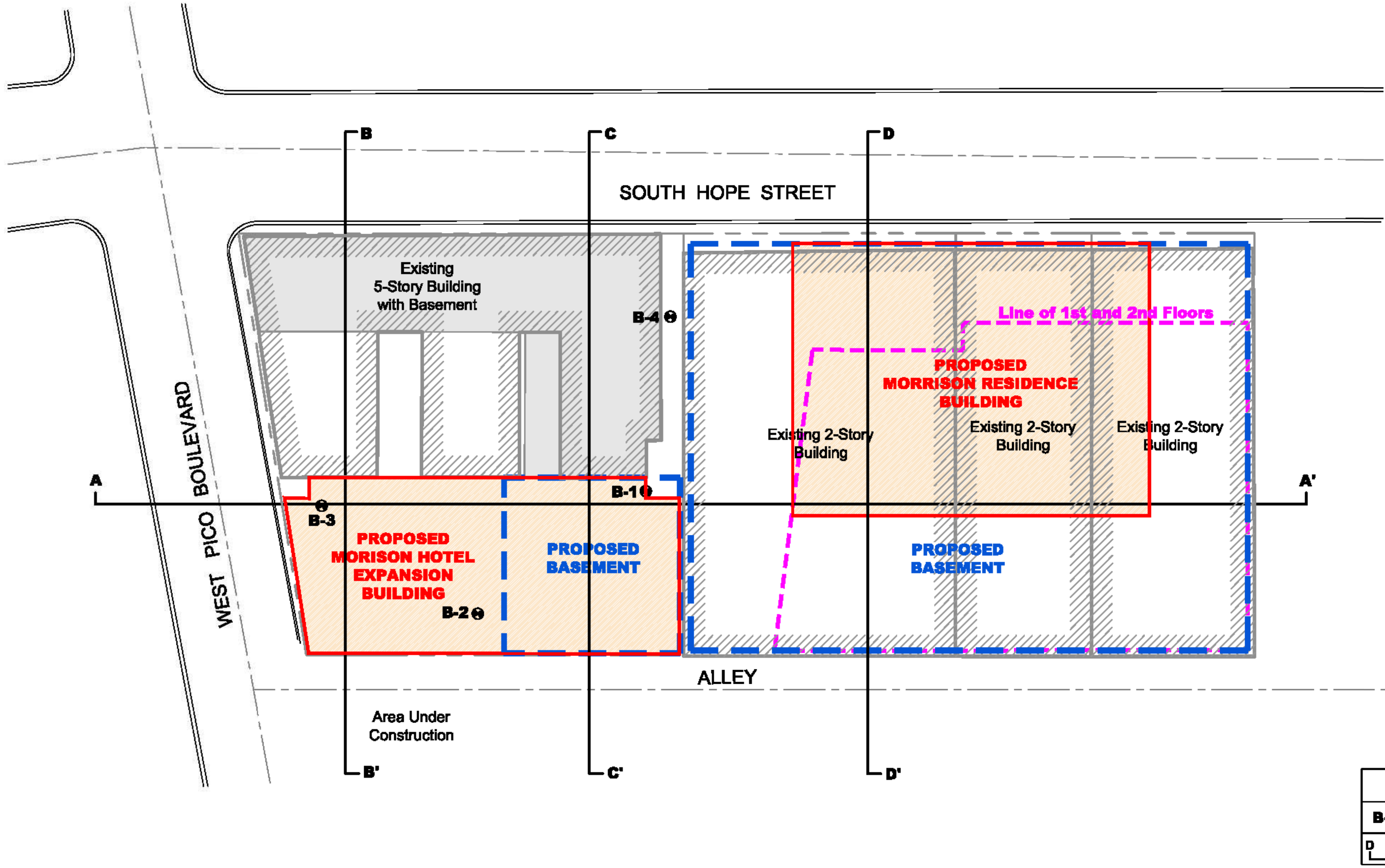
1 Alamo thrust
2 Arrowhead fault
3 Bailey fault
4 Big Mountain fault
5 Big Pine fault
6 Blake Ranch fault
7 Cabrillo fault
8 Chatsworth fault
9 Chino fault
10 Clamshell-Sawpit fault
11 Clearwater fault
12 Cleghorn fault
13 Crafton Hills fault zone
14 Cucamonga fault zone
15 Dry Creek
16 Eagle Rock fault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

Reference:	U. S. G. S: active fault (red) and potentially active fault (green)	

SEISMIC HAZARD MAP



Reference:	State of California, Seismic Hazard Map of the Hollywood Quadrangle	Scale: As Shown



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

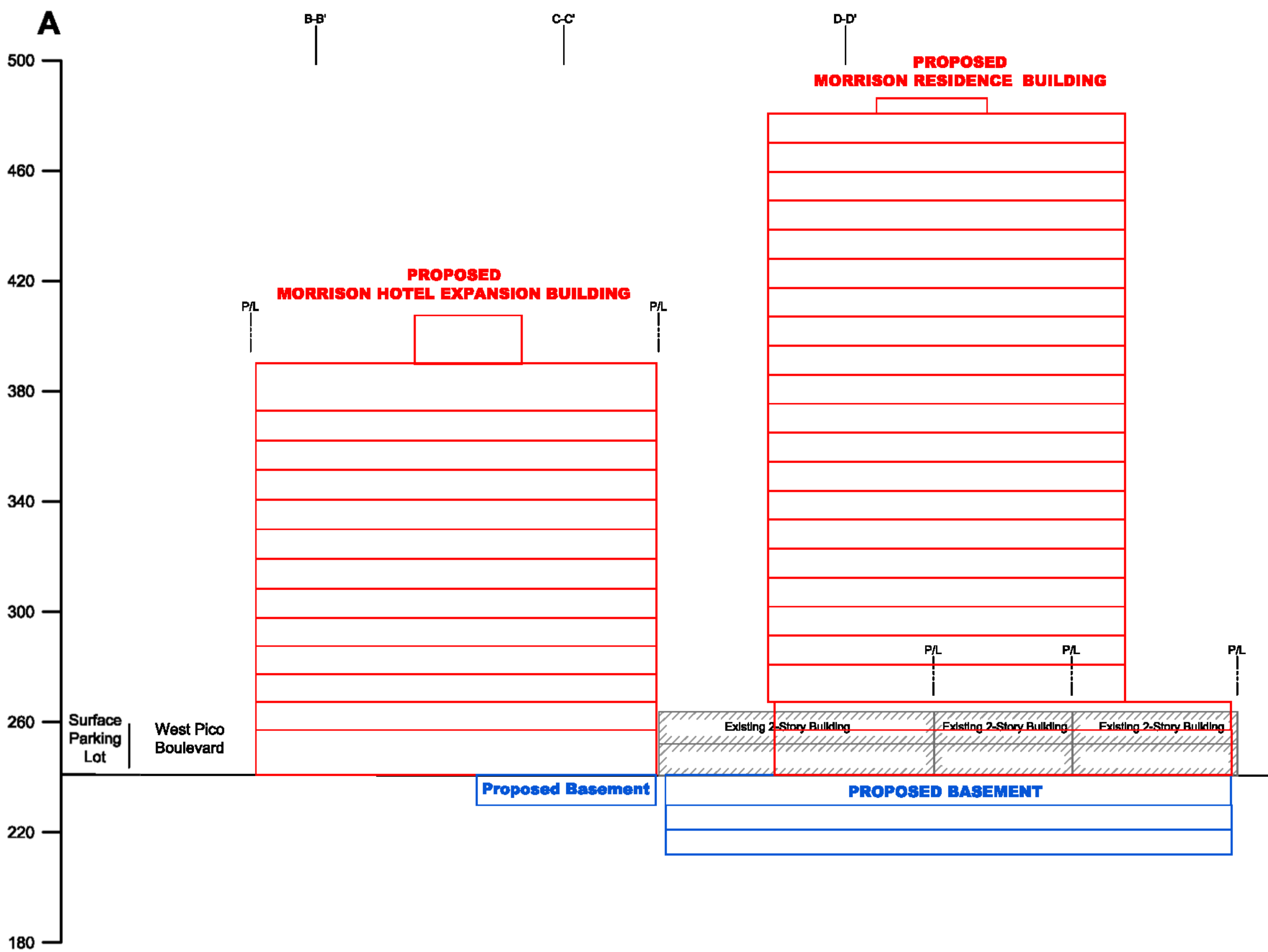
Base Map Provided By:

STINBERG FIVE CHAIRS

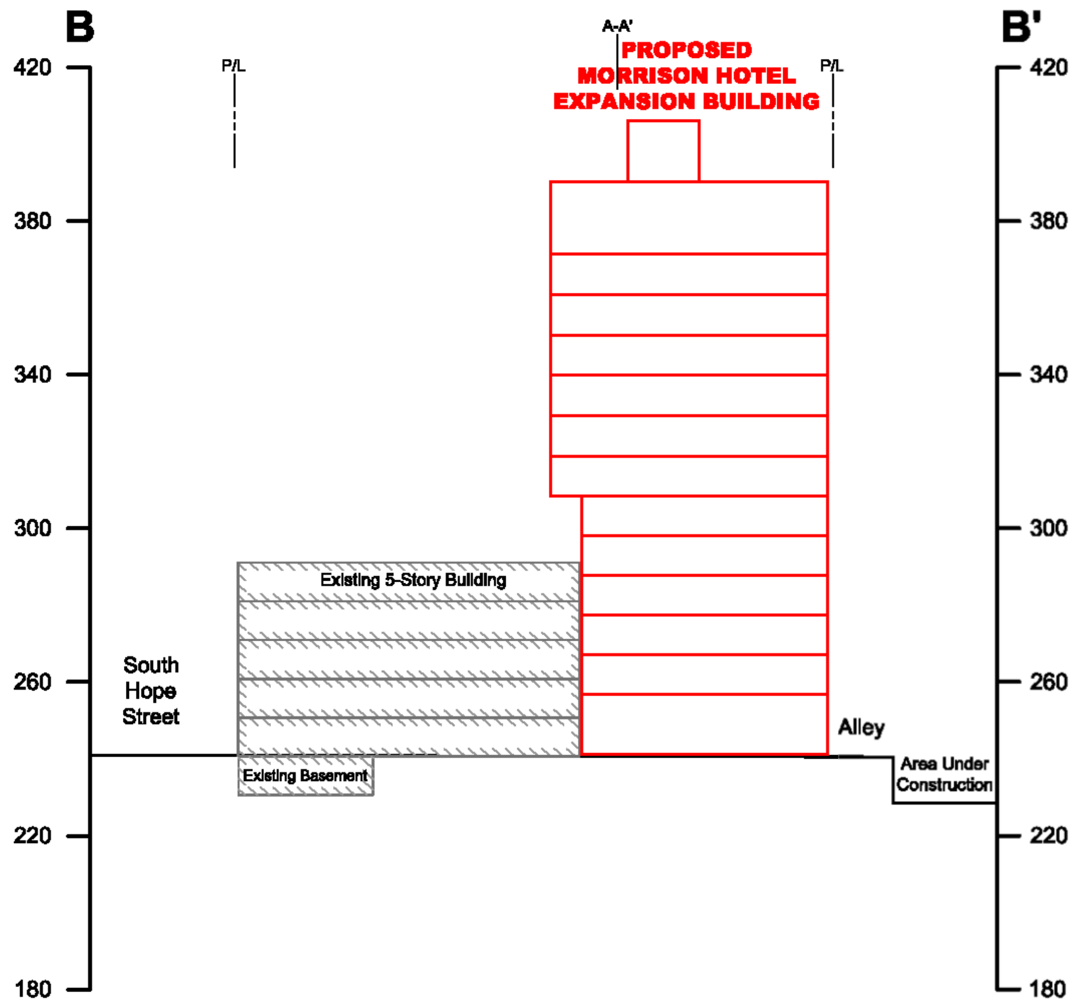
Project Address:

1222 South Hope Street
Los Angeles, California

Page Intentionally Left Blank

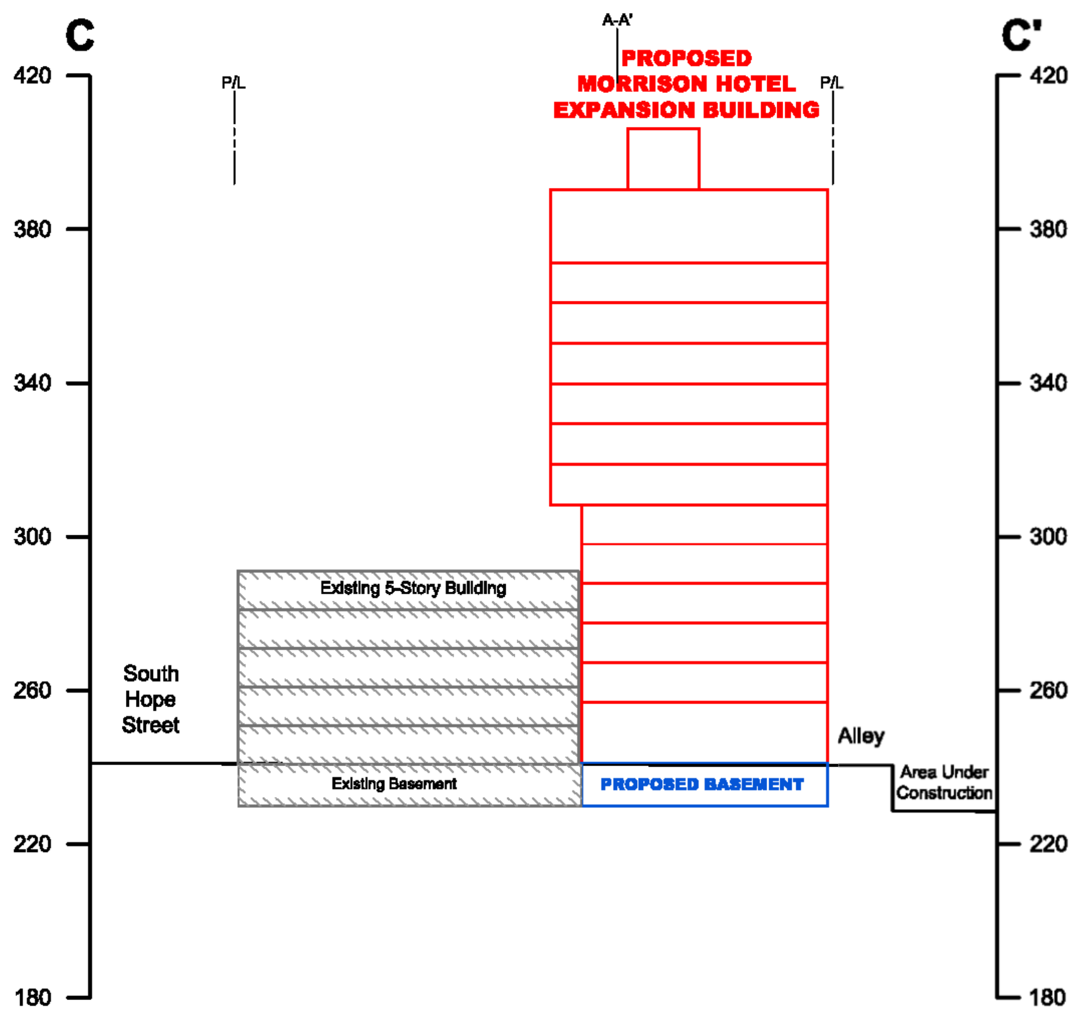

A
B-B'
C-C'
D-D'
500
460
420
380
340
300
260
220
180
PROPOSED
MORRISON RESIDENCE BUILDING
PROPOSED
MORRISON HOTEL EXPANSION BUILDING
P/L
P/L
P/L
P/L
P/L
Surface Parking Lot
West Pico Boulevard
Existing 2-Story Building
Existing 2-Story Building
Existing 2-Story Building
Proposed Basement
PROPOSED BASEMENT

Page Intentionally Left Blank



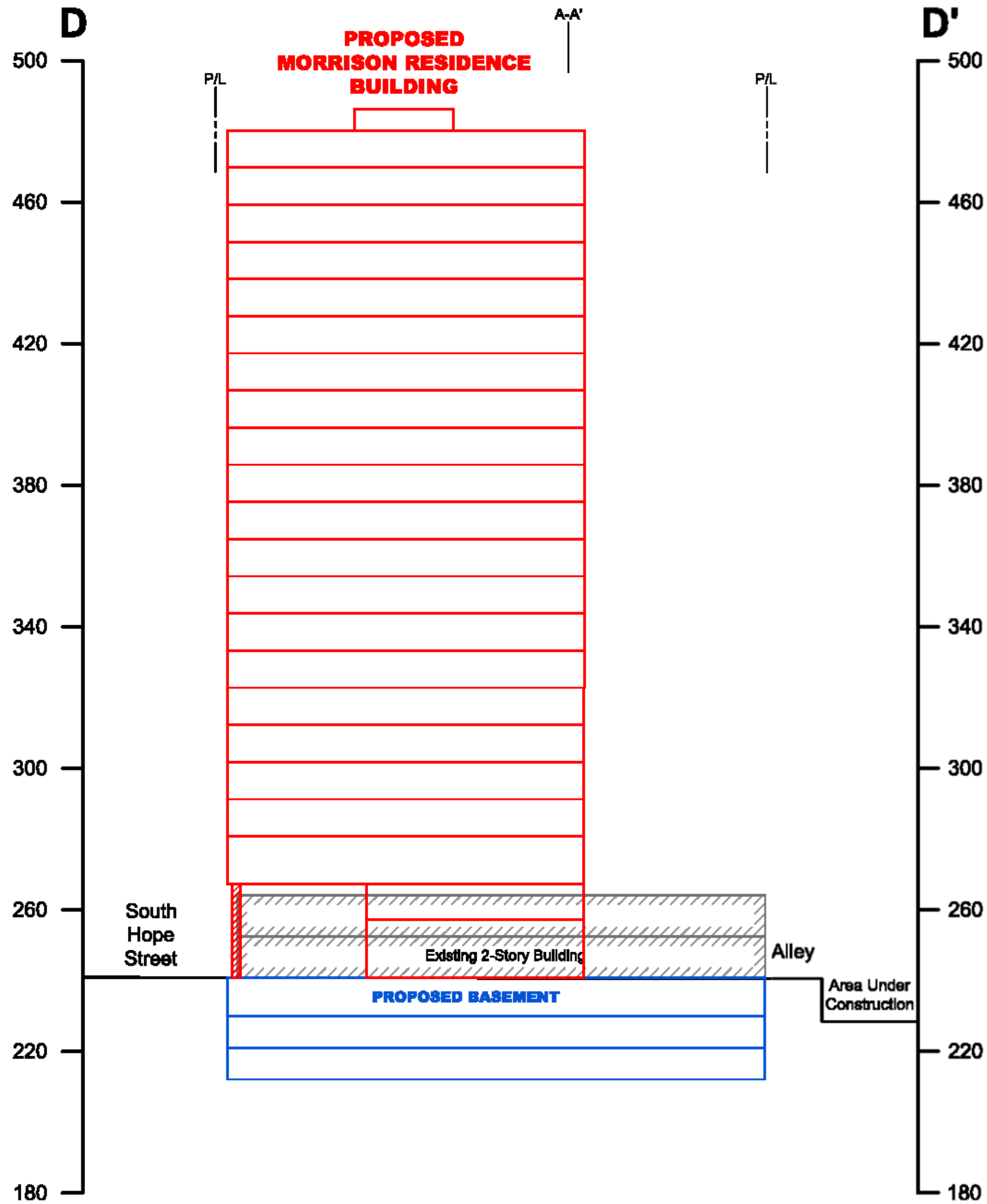
B
B'
420
380
340
300
260
220
180
P/L
A-A'
PROPOSED
MORRISON HOTEL
EXPANSION BUILDING
P/L
South
Hope
Street
Existing 5-Story Building
Existing Basement
Alley
Area Under
Construction

Page Intentionally Left Blank



C
C'
420
380
340
300
260
220
180
P/L
A-A'
P/L
PROPOSED
MORRISON HOTEL
EXPANSION BUILDING
South
Hope
Street
Existing 5-Story Building
Existing Basement
PROPOSED BASEMENT
Alley
Area Under
Construction

Page Intentionally Left Blank


D
D'
500
460
420
380
340
300
260
220
180
PROPOSED
MORRISON RESIDENCE
BUILDING
A-A'
P/L
P/L
South
Hope
Street
Existing 2-Story Building
Alley
Area Under
Construction
PROPOSED BASEMENT

Page Intentionally Left Blank

BORING: B-1

ADDRESS: 1222 S. Hope Street — PROJECT NO.: 5076

DATE LOGGED: December 31, 2015 — LOGGED BY: KNC

ATTITUDES b - bedding j - joint s - shear f - fault	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
							0.0' - 3.0" **ASPHALT**
	5	110	45				3.0" - 2.0' **ARTIFICIAL FILL; Af,** silty sand, dark brown, slightly moist, fine to coarse grained
					5		2.0' - 201.0' **ALLUVIUM; Qal**
	6	113	57				@2.5' silty sand, medium brown, slightly moist, fine to medium grained, slightly porous
	2	113	85				@5.0' silty sand, medium brown, slightly moist, fine to medium grained
	1	120	67		10		@7.5' silty sand to sand, medium brown, fine to coarse grained, slightly moist, gravels up to 1" in length
							@10.0' silty sand to sand, medium brown, slightly moist, fine to coarse grained, gravels up to 2" in length
	6	122	97		15		@15.0' sand, yellowish greenish gray, slightly moist to moist, fine to coarse grained, gravels up to 1/2" in length
	7	112	85		20		@20.0' silty sand to sand, dark yellowish brown, slightly moist to moist, fine to coarse grained, gravels up to 2" in length
	8	112	80		25		@25.0' silty sand, dark yellowish brown, moist, fine to coarse grained, strong hydrocarbon odor
	8	113	82		30		@30.0' silty sand, grayish brown, moist, fine to coarse grained, strong hydrocarbon odor, gravels up to 1/2" in length to silty sand, grayish brown, moist, fine to medium grained
	7	115	84		35		@35.0' silty sand to sand, light brown, moist, fine to coarse grained, hydrocarbon odor, gravels up to 1/2" in length
	9	111	89		40		@40.0' silty sand to sand, light brown, moist, fine to coarse grained, gravels up to 1/2" in length
			64				@42.5' silty sand to sand, light brown, moist, fine to coarse grained, gravels up to 1/2" in length
	11	107	83		45		@45.0' sand, olive brown, moist, fine to medium grained, gravels up to 1/4" in length
			41				@47.5' sandy silt with minor clay, olive brown, fine to medium grained, very dense

BORING: B-1

ADDRESS: 1222 S. Hope Street PROJECT NO.: 5076

DATE LOGGED: December 31, 2015 LOGGED BY: KNC

ATTITUDES b - bedding j - joint s - shear f - fault	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
	7	104	51				@50.0' silty sand with minor clay, medium brown, slightly moist, fine to medium grained
			36				@52.5' clayey sand, medium brown to brown, moist, fine to medium grained
	9	108	51		55		@55.0' silty sand with clay, medium brown to brown, moist, fine to medium grained
			38				@57.5' silty sand with clay, medium brown to brown, moist, fine to medium grained
	10	105	45		60		@60.0' silty sand with minor clay, light brown to medium brown, moist, fine to medium grained
			24				@62.5' clayey sand, medium brown to yellowish brown, moist, fine to medium grained
	13	112	41		65		@65.0' silty sand, medium brown to yellowish brown, moist, fine to medium grained
			75				@67.5' silty sand, medium brown to yellowish brown, moist, fine to medium grained
	7	120	86		70		@70.0' sand, light yellowish brown, moist, fine to medium grained
			85				@72.5' sand, light yellowish brown, moist, fine to medium grained
	8	111	81		75		@75.0' silty sand with minor clay, medium brown, moist, fine to coarse grained, hydrocarbon odor
			89				@77.5' silty sand to sand, light yellowish brown, slightly moist, fine to coarse grained, gravels up to 1/4" in length
	1	117	90		80		@80.0' silty sand to sand, light yellowish brown, slightly moist, fine to coarse grained, gravels up to 1/4" in length
			47				@82.5' silty sand to sand, light yellowish brown, slightly moist, fine to coarse grained, gravels up to 1/4" in length
	3	117	80		85		@85.0' silty sand to sand, light yellowish brown, slightly moist, fine to coarse grained, gravels up to 1/4" in length
			86				@87.5' silty sand to sand, light yellowish brown, slightly moist, fine to coarse grained, gravels up to 1/4" in length
	5	111	87		90		@90.0' silty sand to sand, light yellowish brown, slightly moist, fine to coarse grained, gravels up to 1/4" in length
			70				@92.5' sand, light yellowish brown, moist, fine to medium grained
	8	120	84		95		@95.0' sand, light yellowish brown, moist, fine to medium grained
			24				@97.5' sandy silt, olive brown and grayish brown, moist, fine grained to medium grained

BORING: B-1

ADDRESS: 1222 S. Hope Street PROJECT NO.: 5076

DATE LOGGED: December 31, 2015 LOGGED BY: KNC

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
	15	96	51				@100.0' sandy silt, olive brown and grayish brown, moist, fine grained to medium grained
			89				@102.5' sandy silt, olive brown and grayish brown, moist, fine grained to medium grained
	2	115	71		105		@105.0' sand with silt, light yellowish brown, slightly moist, fine to medium grained, gravels up to 1" in length
			97				@107.5' sand, light yellowish brown, slightly moist, fine to medium grained, gravels up to 1" in length
	3	114	85		110		@110.0' silty sand, medium brown, slightly moist, fine to medium grained
			89				@112.5' gravelly sand, grayish brown, moist, fine to coarse grained, gravels up to 1" in length
	7	118	89		115		@115.0' gravelly sand, grayish brown, moist, fine to coarse grained, gravels up to 1" in length
			98				@117.5' gravelly sand, grayish brown, moist, fine to coarse grained, gravels up to 1" in length
	8	112	87		120		@120.0' silty sand, yellowish brown to medium brown, moist, fine to medium grained
			80				@122.5' silty sand, medium brown, slightly moist, fine to medium grained, gravels up to 1/2" in length
	2	114	77		125		@125.0' sand, light yellowish brown, slightly moist, fine to medium grained
			82				@127.5' silty sand, medium brown, slightly moist, fine to medium grained, gravels up to 1/2" in length
	3	110	89		130		@130.0' silty sand, medium brown, slightly moist, fine to medium grained, gravels up to 1/2" in length
			73				@132.5' sand, medium brown, slightly moist, fine to medium grained, gravels up to 1/2" in length
	4	111	50		135		@135.0' sand, yellowish brown, slightly moist, fine to medium grained
			78				@137.5' sand, yellowish brown, slightly moist, fine to medium grained
	3	111	50		140		@140.0' silty sand, yellowish brown, slightly moist, fine to medium grained
			97				@142.5' silty sand, yellowish brown, slightly moist, fine to medium grained
	5	115	83		145		@145.0' silty sand, light grayish brown, slightly moist, fine to medium grained
			70				@145.0' sandy silt, light grayish brown, slightly moist, fine to medium grained

BORING: B-1

ADDRESS: 1222 S. Hope Street

PROJECT NO.: 5076

DATE LOGGED: December 31, 2015

LOGGED BY: KNC

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

ATTITUDES	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
	5	100	81				@150.0' sandy silt, grayish brown, slightly moist, fine to medium grained
			99				@152.5' sandy silt, grayish brown, slightly moist, fine to medium grained
	6	106	88		155		@155.0' silty sand, yellowish brown, moist, fine to medium grained
			64				@157.5' silty sand, yellowish brown, moist, fine to medium grained, perched water
	15	103	96		160		@160.0' sandy silt to silty sand, dark brown, moist, fine to medium grained
			60				@162.5' sandy silt to silty sand, dark brown, moist, fine to medium grained
	20	102	80		165		@165.0' sandy silt to silty sand, grayish brown, wet, fine to medium graine
			69				@167.5' clayey sand, dark gray, moist, fine to medium grained
	10	111	67		170		@170.0' clayey sand, dark gray, moist, fine to medium grained
			50				@172.5' sandy silt with minor clay, dark grayish brown, slightly moist to moist, fine to medium grained
	7	116	83		175		@175.0' sandy silt with minor clay, dark grayish brown, slightly moist to moist, fine to medium grained
			99				@177.5' sandy silt with minor clay, dark grayish brown, slightly moist to moist, fine to medium grained
	9	115	83		180		@180.0' sandy silt, bluish gray, moist, fine to medium grained
			67				@182.5' sandy silt, bluish gray, moist, fine to medium grained
	15	104	81		185		@185.0' silty sand, dark bluish gray, moist, fine to medium grained
			76				@187.5' sandy silt, dark bluish gray, moist, fine to medium grained, perched water
	7	117	50		190		@190.0' sand, bluish gray, moist, fine to coarse grained
			84				@192.5' sand, bluish gray, wet, fine to coarse grained, gravels up to 3/4" in length
	13	120	83		195		@195.0' sand, bluish gray, wet, fine to coarse grained, gravels up to 3/4" in length
			73				@197.5' sand, bluish gray, wet, fine to coarse grained, gravels up to 3/4" in length

BORING: B-1

ADDRESS: 1222 S. Hope Street

PROJECT NO.: 5076

DATE LOGGED: December 31, 2015

LOGGED BY: KNC

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
	14	119	79				@200.0' sand, bluish gray, wet, fine to coarse grained, gravels up to 3/4" in length
							Total Depth 201 Feet Perched Groundwater @157.5 and @187.5 8" Hollow Stem Auger with Autohammer
					205		
					210		
					215		
					220		
					225		
					230		
					235		
					240		
					245		

BORING: B-2

ADDRESS: 1222 S. Hope Street | PROJECT NO.: 5076

DATE LOGGED: January 5, 2016 | LOGGED BY: KNC

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
							0.0' - 3.0" **ASPHALT**
							3.0" - 2.0' **ARTIFICIAL FILL; Af**
							2.0' - 81.0' **ALLUVIUM; Qal,**
Shear	6	113	78		5		@5.0' silty sand, medium brown, sllightly moist, fine to medium grained, slightly porous
	5	109	50		10		@10.0' silty sand, light brown to yellowish brown, slightly moist, fine to coarse grained, gravels up to 1" in length
			50		15		@15.0' no recovery
	5	115	57		20		@20.0' silty sand, medium brown to grayish brown, slightly moist, fine to coarse grained, gravels up to 2" in length
Shear	9	113	99		25		@25.0' silty sand to sand, yellowish brown, moist, fine to coarse grained, gravels up to 1.5" in length
Shear	11	114	40		30		@30.0' silty clayey sand, grayish brown to greenish gray, moist, fine to coarse grained, gravels up to 3/4" in length, rock fragments
Shear	11	115	66		35		@35.0' silty sand with minor clay, greenish gray, moist, fine to coarse grained, gravels up to 3/4" in length
	11	115	63		40		@40.0' silty sand with minor clay, moist, hydrocarbon odor, greenish gray, moist, fine to medium grained
Consolidation	12	112	84		45		@45.0' clayey silt, greenish gray, moist, fine grained

BORING: B-2

ADDRESS: 1222 S. Hope Street

PROJECT NO.: 5076

DATE LOGGED: January 5, 2016

LOGGED BY: KNC

ATTITUDES b - bedding j - joint s - shear f - fault	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
	10	113	87				@50.0' silty sand with minor clay, yellowish bown, moist, fine to medium grained
	9	121	79		55		@55.0' silty sand with clay, grayish brown, moist, fine to medium grained
Consolidation	10	116	78		60		@60.0' silty sand with clay, grayish brown, moist, fine to medium grained
Consolidation	10	119	76		65		@65.0' sandy silt, greenish brown, moist, fine to medium grained
	2	118	50		70		@70.0' sand, light brown, slightly moist, fine to coarse grained, gravels up to 1.5" in length
Consolidation	3	119	50		75		@75.0' sand with silt, light brown, slightly moist, fine to coarse grained, gravels up to 1.5" in length
Shear	3	119	50		80		@80.0' sand, light brown, slightly moist, fine to coarse grained, gravels up to 1.5" in length
							Total Depth 81.0 Feet No Groundwater 8" Hollow Stem Auger with Autohammer
					85		
					90		
					95		

BORING: B-3

ADDRESS: 1222 S. Hope Street

PROJECT NO.: 5076

DATE LOGGED: January 5, 2016

LOGGED BY: KNC

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
							0.0' - 3.0" **ASPHALT**
							3.0" - 2.0' **ARTIFICIAL FILL; Af**
							2.0' - 81.0' **ALLUVIUM; Qal,**
Shear	4	117	75		5		@5.0' silty sand, medium brown, slightly moist, fine to medium grained, gravels up to 1/2" in length
	5	119	79		10		@10.0' silty sand, light brown, slightly moist, fine to medium grained, gravel up to 1/2" in length
Shear	2	110	50		15		@15.0' silty sand, medium brown, slightly moist, fine to coarse grained, gravels up to 1" in length
	3	125	82		20		@20.0' silty sand, medium brown, slightly moist, fine to coarse grained, gravels up to 2" in length
Shear	5	118	84		25		@25.0' silty sand, medium brown, slightly moist, fine to coarse grained, gravels up to 2" in length
	5	128	67		30		@30.0' silty sand with minor clay, grayish brown, moist, fine to coarse grained, gravels up to 2" in length
	9	115	90		35		@35.0' sand, brown, moist, fine to coarse grained
	8	122			40		@40.0' sand with minor silt, orangish brown, moist, fine to coarse grained, few gravels up to 1/2" in length
Consolidation	15	104	47		45		@45.0' silt, yellowish brown, moist, fine grained

BORING: B-3

ADDRESS: 1222 S. Hope Street | PROJECT NO.: 5076

DATE LOGGED: January 5, 2016 | LOGGED BY: KNC

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
Shear	13	100	56		50		@50.0' silt, yellowish brown, moist, fine grained
Consolidation	12	113	46		55		@55.0' clayey sand, medium brown, moist, fine to medium grained
	10	115	40		60		@60.0' sandy silt with clay binder, medium brown, moist, fine to medium grained
	14	104	36		65		@65.0' sandy silt, medium brown, moist, fine to medium grained
Consolidation	5	123	50		70		@70.0' sandy silt to silty sand, yellowish brown, slightly moist, fine to coarse grained, gravels up to 1/4" in length
Consolidation	2	124	82		75		@75.0' silty sand, yellowish brown, slightly moist, fine to coarse grained, gravels up to 1/2" in length
	4	126	90		80		@80.0' silty sand, yellowish brown, slightly moist, fine to coarse grained, gravels up to 1/2" in length
					85		Total Depth 81.0 Feet No Groundwater 8" Hollow Stem Auger with Autohammer
					90		
					95		

BORING: B-4

ADDRESS: 1222 S. Hope Street | PROJECT NO.: 5076

DATE LOGGED: February 9, 2017 | LOGGED BY: KNC

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' - 1.0" **CONCRETE**
							1.0" - 3.0' **ARTIFICIAL FILL; Af,** sandy clay, dark brown, moist, fine to medium grained, concrete fragments
							3.0' - 71.0' **ALLUVIUM; Qal,**
	6	117	35		5		@5.0' clayey sand, medium brown, moist, fine to coarse grained, gravels up to 1" in length
	4	117	40		10		@10.0' gravelly sand, brown, moist, fine to coarse grained, gravels up to 1.5" in length
	5	114	50		15		@15.0' silty sand, brown to grayish brown, slightly moist, fine to coarse grained, gravels up to 1" in length
	9	115	51		20		@20.0' no recovery
	5	115	50		25		@25.0' silty sand, grayish brown, slightly moist, fine to coarse grained, occasional to frequent gravels up to 1.5" in length, cobbles up top 3.5" in length, strong hydrocarbon odor
	6	113	50		30		@30.0' clayey silt, brownish gray, slightly moist, fine to medium grained, hydrocarbon odor
	4	116	50		35		@35.0' sand, medium brown, slightly moist, fine to coarse grained
	7	117	50		40		@40.0' clayey sand, dark gray, moist, fine to coarse grained, hydrocarbon odor
	6	117	51		45		@45.0' sand, brownish gray, slightly moist, fine to medium grained, hydrocarbon odor

BORING: B-4

ADDRESS: 1222 S. Hope Street | PROJECT NO.: 5076

DATE LOGGED: February 9, 2017 | LOGGED BY: KNC

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
	12	110	61				@50.0' silty clay, reddish brown, slightly moist, fine to medium grained
	12	107	60		55		@55.0' silty clay, reddish brown, moist, fine grained
	13	109	50		60		@60.0' sandy clay to clayey sand, medium brown, slightly moist, fine to coarse grained
	14	114	41		65		@65.0' silty clay, yellowish brown, slightly moist, fine grained
	3	112	50		70		@70.0' silty sand, light brown, slightly moist, fine to medium grained, few gravels up to 1" in length
							Total Depth 71.0 Feet No Groundwater 8" Hollow Stem Auger
					75		
					80		
					85		
					90		
					95		

APPENDIX II

LABORATORY TESTING

Laboratory Procedures

Laboratory Recapitulation 1
Laboratory Recapitulation 2

Figures S.1 through S.15
Figures C.1 through C.22

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.

pH (CTM 532)

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 532)

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: 1222 S. Hope Street PROJECT NO.: 5076						
Exploration	**Depth (ft)**	**Material**	**Dry Density In Situ (P.C.F.)**	**Moisture Content (%)**	**Cohesion (K.S.F)**	**Friction Angle (degree)**
B-1	2.5	Qa	110.1	4.8		
B-1	5	Qa	112.8	5.9		
B-1	7.5	Qa	113.1	1.9		
B-1	10	Qa	119.9	1	0.1	33
B-1	15	Qa	122	6.2		
B-1	20	Qa	112.1	7.3	0.15	32
B-1	25	Qa	112.1	8		
B-1	30		113.4	7.6		
B-1	35	Qa	114.6	7.1		
B-1	40	Qa	110.9	9.2	0.05	35
B-1	45	Qa	107.3	11		
B-1	47.5	Qa (BAG)				
B-1	50	Qa	103.5	7.3	0.175	33
B-1	55	Qa	107.5	8.7		
B-1	60	Qa	105.3	10.2		
B-1	65	Qa	111.8	12.8	0.15	33
B-1	70	Qa	119.6	6.9		
B-1	75	Qa	111.4	8		
B-1	77.5	Qa (BAG)				
B-1	80	Qa	117.5	1.4	0.1	37
B-1	85	Qa	117.3	3.2		
B-1	90	Qa	111.2	5.2		
B-1	95	Qa	120.2	7.7		
B-1	100	Qa	95.7	15.2		
B-1	105	Qa	115.2	1.5		
B-1	110	Qa	113.8	3.3		
B-1	115	Qa	117.9	7		
B-1	120	Qa	111.8	8.3		
B-1	125	Qa	113.5	2		
B-1	130	Qa	110	2.8		
B-1	135	Qa	111.1	3.7		
B-1	140	Qa	111.4	3.4		
B-1	145	Qa	115.3	5.1		
B-1	150	Qa	100.3	5.1		
B-1	155	Qa	106.2	6.2		
B-1	160	Qa	103.2	14.9		
B-1	165	Qa	102.3	20		

B-1	170	Qa	111.4	10.2		
B-1	175	Qa	116.5	6.9		
B-1	180	Qa	114.9	9		
B-1	185	Qa	104	14.9		
B-1	190	Qa	116.8	6.6		
B-1	195	Qa	120.1	12.8		
B-1	200	Qa	118.6	14.1		
B-2	5	Qa	113.1	5.6	0.1	31
B-2	10	Qa	109.4	4.9		
B-2	15	Qa				
B-2	20	Qa	115.4	4.9		
B-2	25	Qa	112.9	8.7	0.1	36
B-2	30	Qa	113.9	11.5	0.3	35
B-2	35	Qa	115.1	10.7	0.2	36
B-2	40	Qa	115.1	11.3		
B-2	45	Qa	111.8	12.1		
B-2	50	Qa	113.3	9.9		
B-2	55	Qa	120.8	8.6		
B-2	60	Qa	116.2	10		
B-2	65	Qa	119.5	10.1		
B-2	70	Qa	118.3	2.4		
B-2	75	Qa	118.5	3		
B-2	80	Qa	118.9	2.5	0.15	36
B-3	5	Qa	117.4	3.8	0.15	36
B-3	10	Qa	118.7	4.8		
B-3	15	Qa	110.2	2.4	0.1	35
B-3	20	Qa	124.8	3		
B-3	25	Qa	118	4.8	0.1	36
B-3	30	Qa	128	5.1		
B-3	35	Qa	115.4	9.3		
B-3	40	Qa	122.2	8		
B-3	45	Qa	103.9	15		
B-3	50	Qa	100.3	13.3	0.35	28
B-3	55	Qa	113	11.6		
B-3	60	Qa	114.7	9.9		
B-3	65	Qa	104.3	14.2		
B-3	70	Qa	122.6	4.9		
B-3	75	Qa	124	2.4		
B-3	80	Qa	126.1	4		
B-4	5	Qal	116.9	5.7		
B-4	10	Qal	116.8	4.2	0.15	33
B-4	15	Qal	114	4.8		

B-4	20		115.2	8.7	0.1	33
B-4	25	Qal	114.9	4.7		
B-4	30		112.8	5.9		
B-4	35		116.5	4.1	0.075	36
B-4	40		117.4	7.4		
B-4	45		117.3	5.7	0.2	34
B-4	50		109.6	12		
B-4	55		107.1	12.3	0.1	36
B-4	60		109.1	13.3		
B-4	65		113.7	14.4	0.15	37
B-4	70		112.2	2.7		

LABORATORY RECAPITULATION 2
PROJECT: 1222 S. Hope Street
PROJECT NO.: 5076

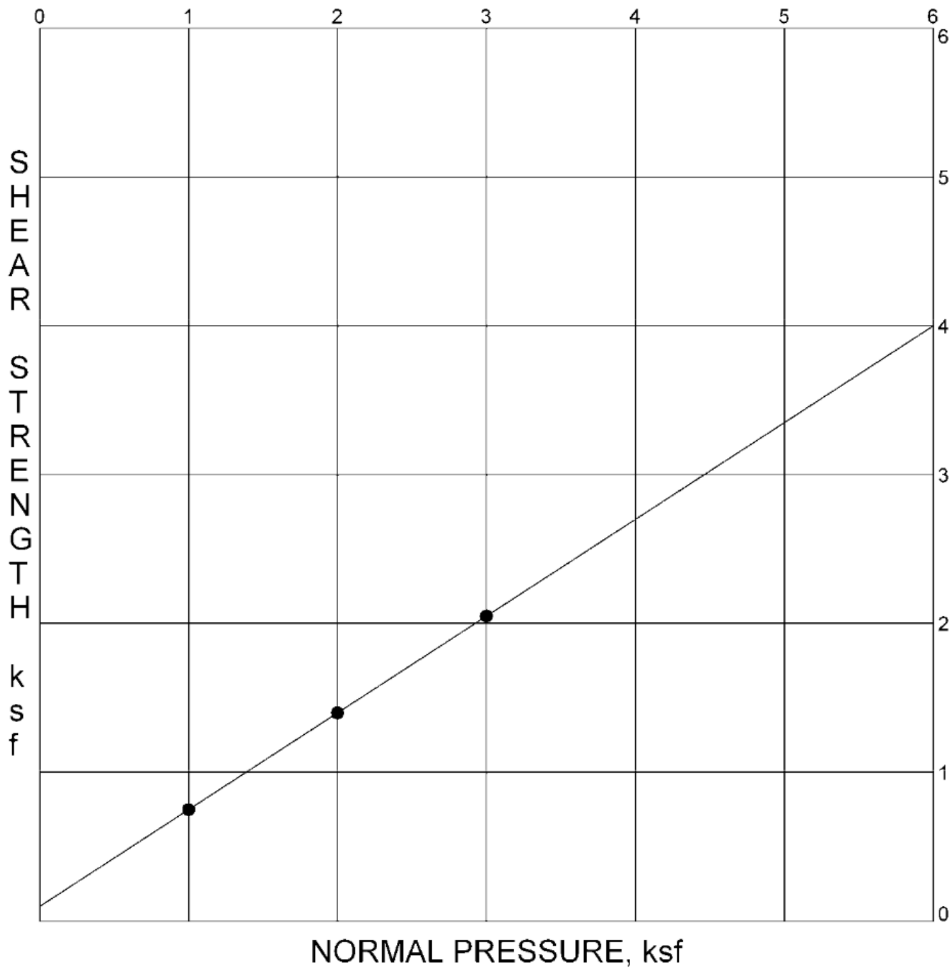
Exploration	Depth (ft)	pH	As-Is Soil Resistivity (ohm-cm)	Minimum Soil Resistivity (ohm-cm)	Chloride (%)	Sulphate (%)
B-1	25	7.67	20000	5500	0.003	0.00312
B-1	47.5	7.94	2400	1900	0.002	0.00291
B-1	77.5	6.61	120000	11000	0.004	0.00033

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-1 @ 10.0

DESCRIPTION: Qa



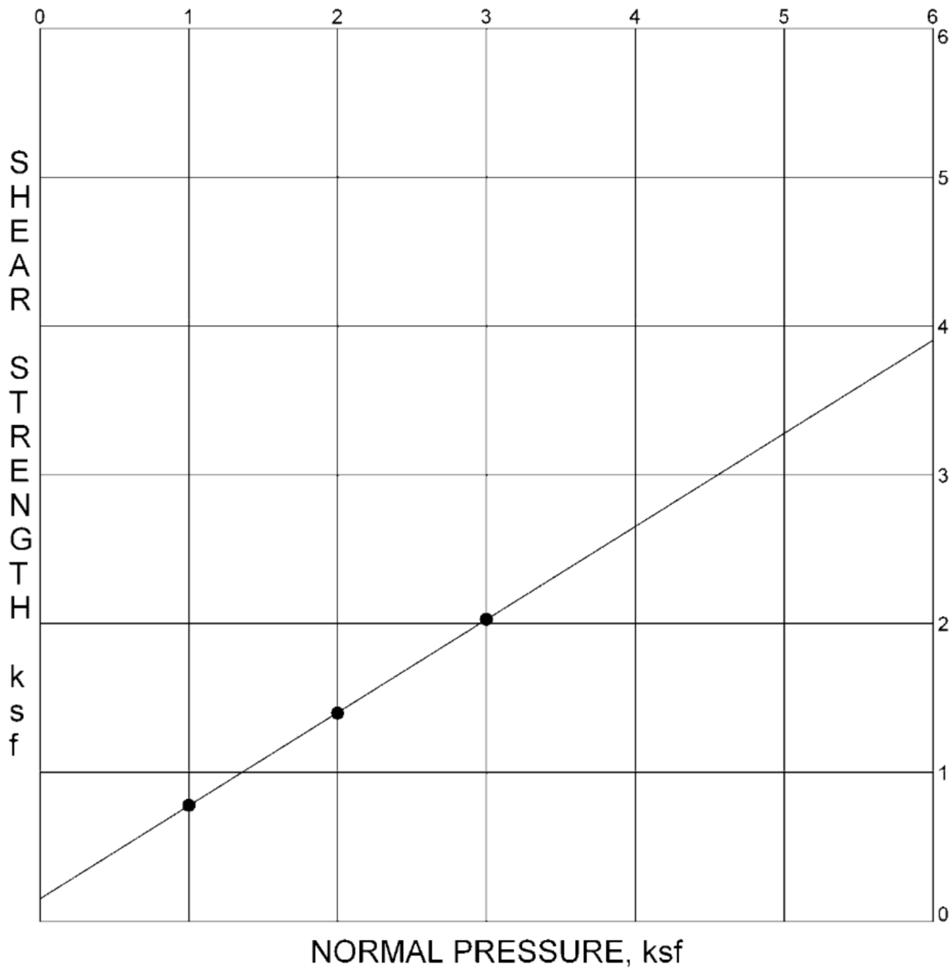
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 1.0	Dry Density: 119.9	Phi (deg): 33.0
Saturated: 15.0		Cohesion (ksf): 0.100

SHEAR TEST DIAGRAM

Figure S.1

PROJECT LOCATION: 1222 S. Hope Street	PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 20.0	DESCRIPTION: Qa



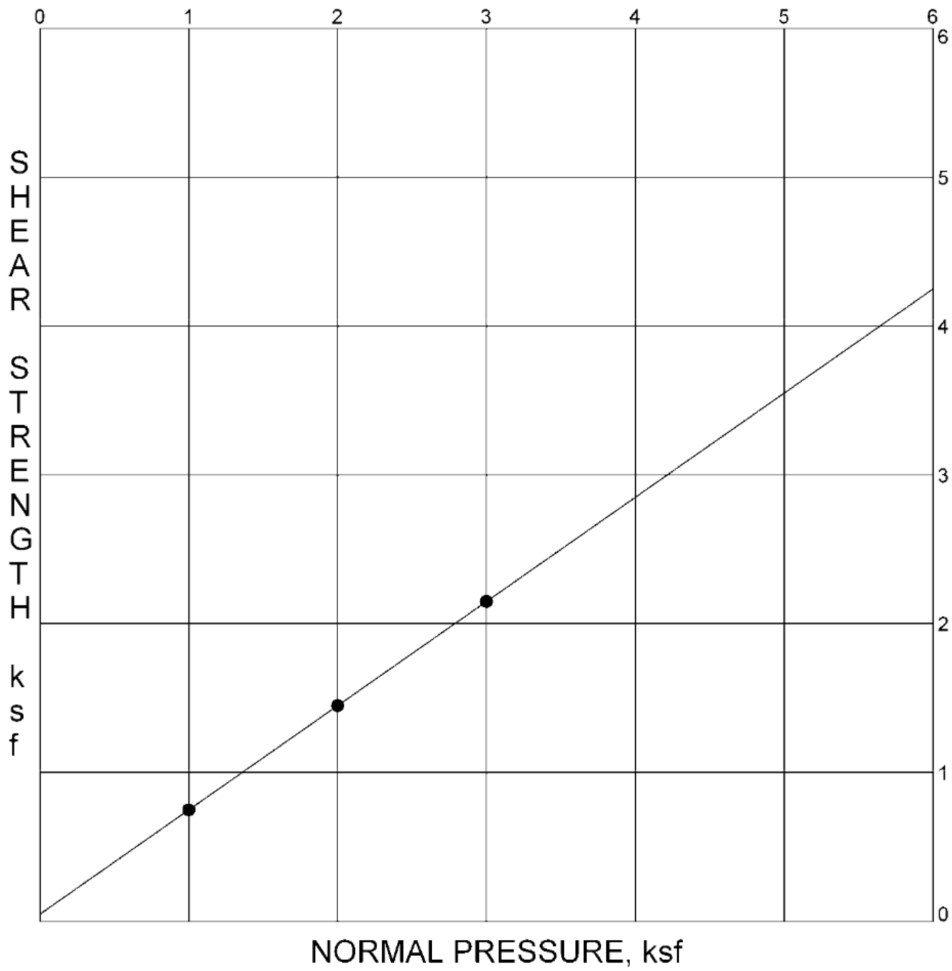
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 7.3	Dry Density: 112.1	Phi (deg): 32.0
Saturated: 18.6		Cohesion (ksf): 0.150

SHEAR TEST DIAGRAM

Figure S.2

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 40.0 | DESCRIPTION: Qa



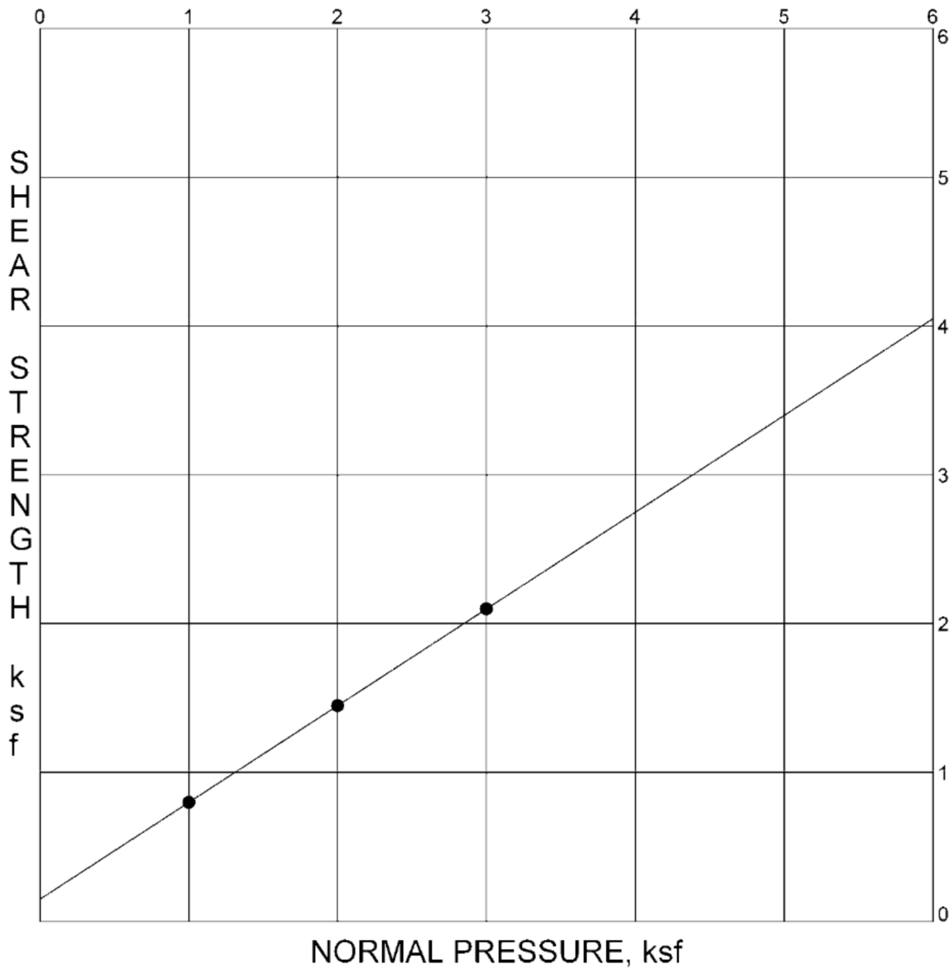
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 9.2	Dry Density: 110.9	Phi (deg): 35.0
Saturated: 19.2		Cohesion (ksf): 0.050

SHEAR TEST DIAGRAM

Figure S.3

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 50.0 | DESCRIPTION: Qa



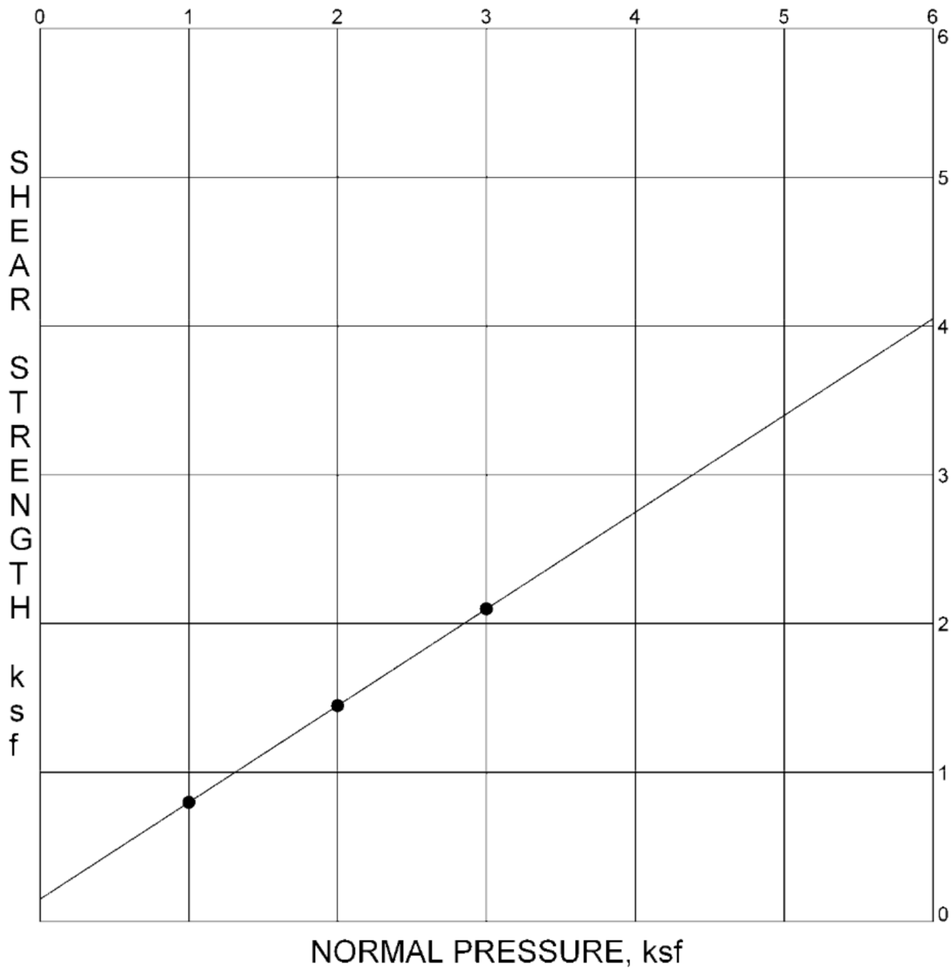
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 7.3	Dry Density: 103.5	Phi (deg): 33.0
Saturated: 23.2		Cohesion (ksf): 0.175

SHEAR TEST DIAGRAM

Figure S.4

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 65.0 | DESCRIPTION: Qa



Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 12.8	Dry Density: 111.8	Phi (deg): 33.0
Saturated: 18.7		Cohesion (ksf): 0.150

SHEAR TEST DIAGRAM

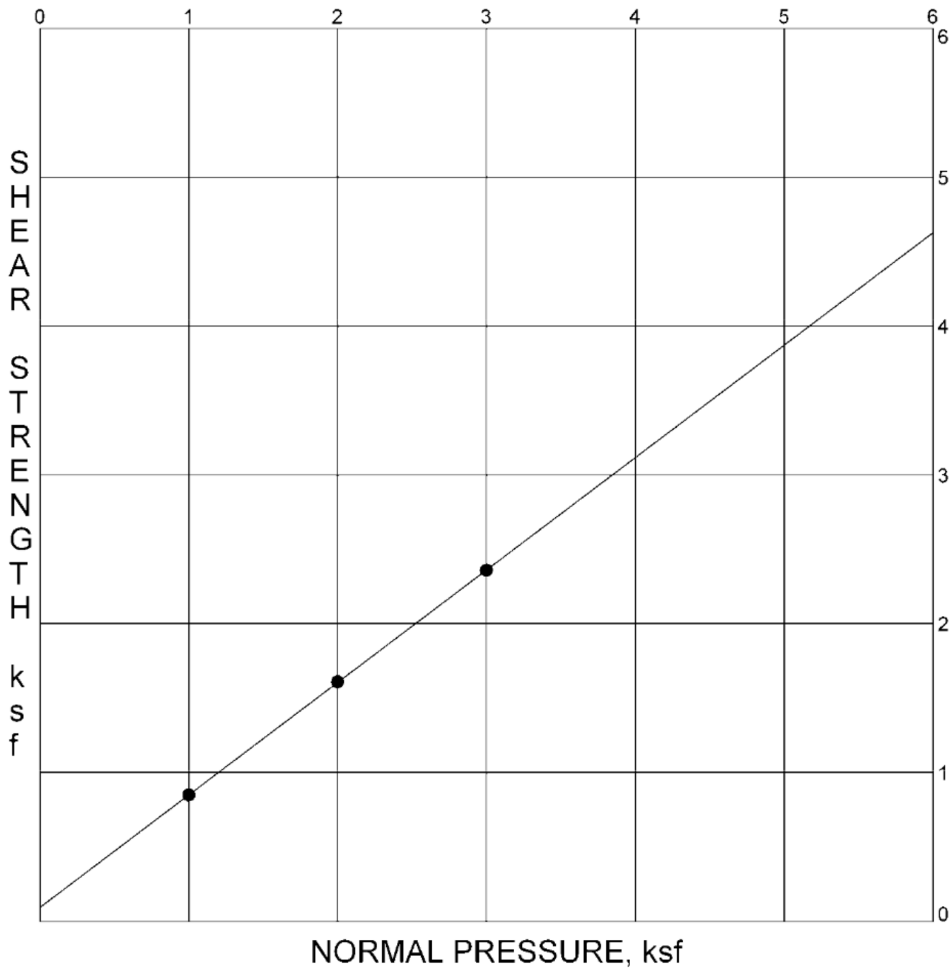
Figure S.5

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-1 @ 80.0

DESCRIPTION: Qa



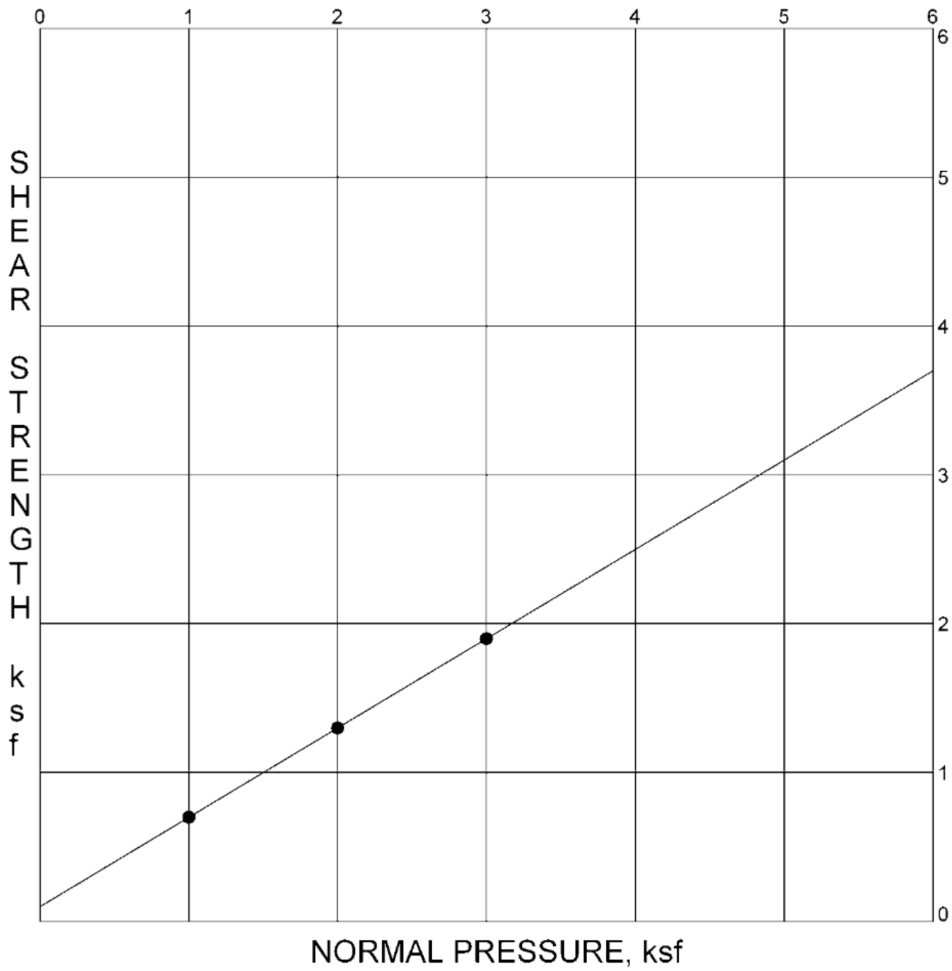
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 1.4	Dry Density: 117.5	Phi (deg): 37.0
Saturated: 16.0		Cohesion (ksf): 0.100

SHEAR TEST DIAGRAM

Figure S.6

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076
SAMPLE LOCATION: B-2 @ 5.0 | DESCRIPTION: Qa



Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 5.6	Dry Density: 113.1	Phi (deg): 31.0
Saturated: 18.1		Cohesion (ksf): 0.100

SHEAR TEST DIAGRAM

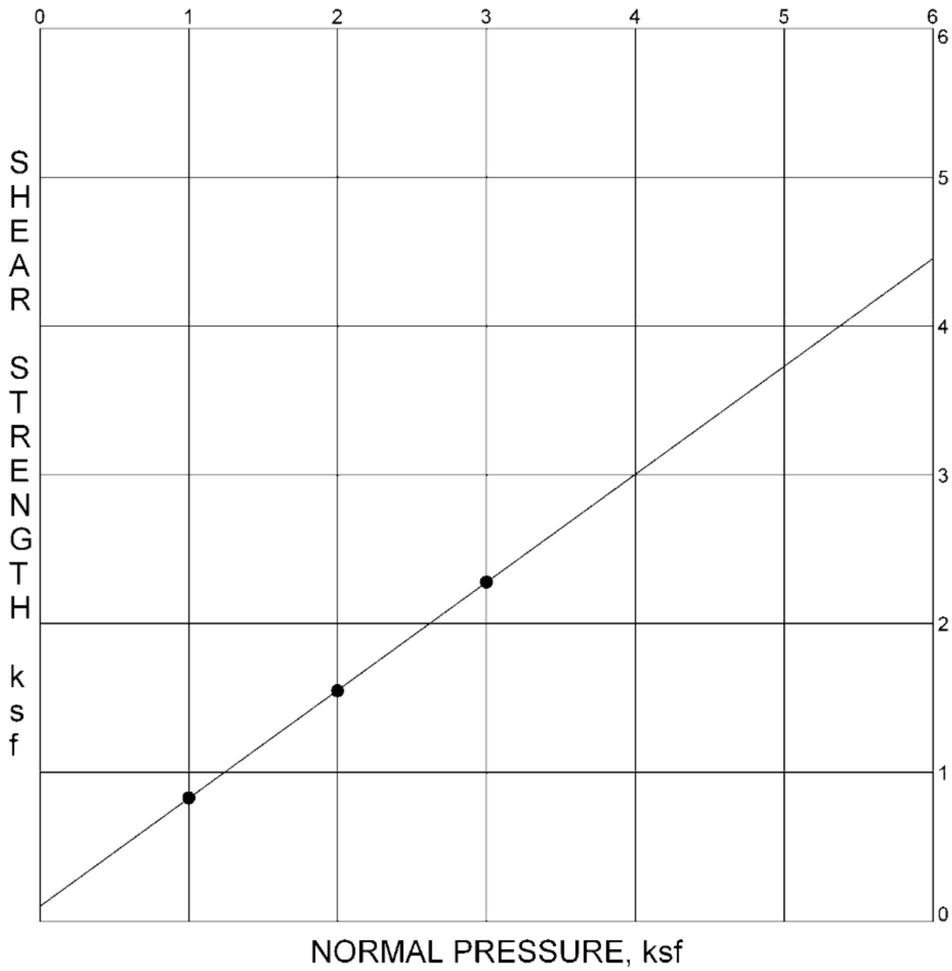
Figure S.7

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-2 @ 25.0

DESCRIPTION: Qa



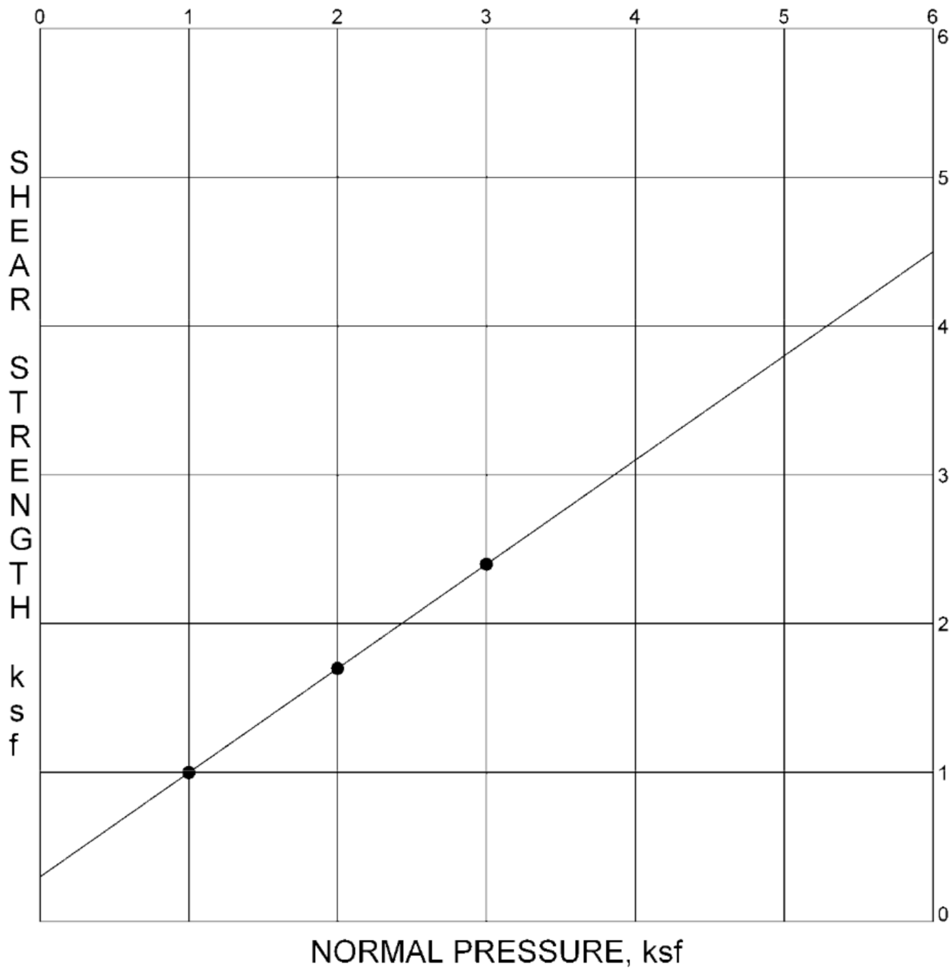
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 8.7	Dry Density: 112.9	Phi (deg): 36.0
Saturated: 18.2		Cohesion (ksf): 0.100

SHEAR TEST DIAGRAM

Figure S.8

PROJECT LOCATION: 1222 S. Hope Street PROJECT NO.: 5076
SAMPLE LOCATION: B-2 @ 30.0 DESCRIPTION: Qa



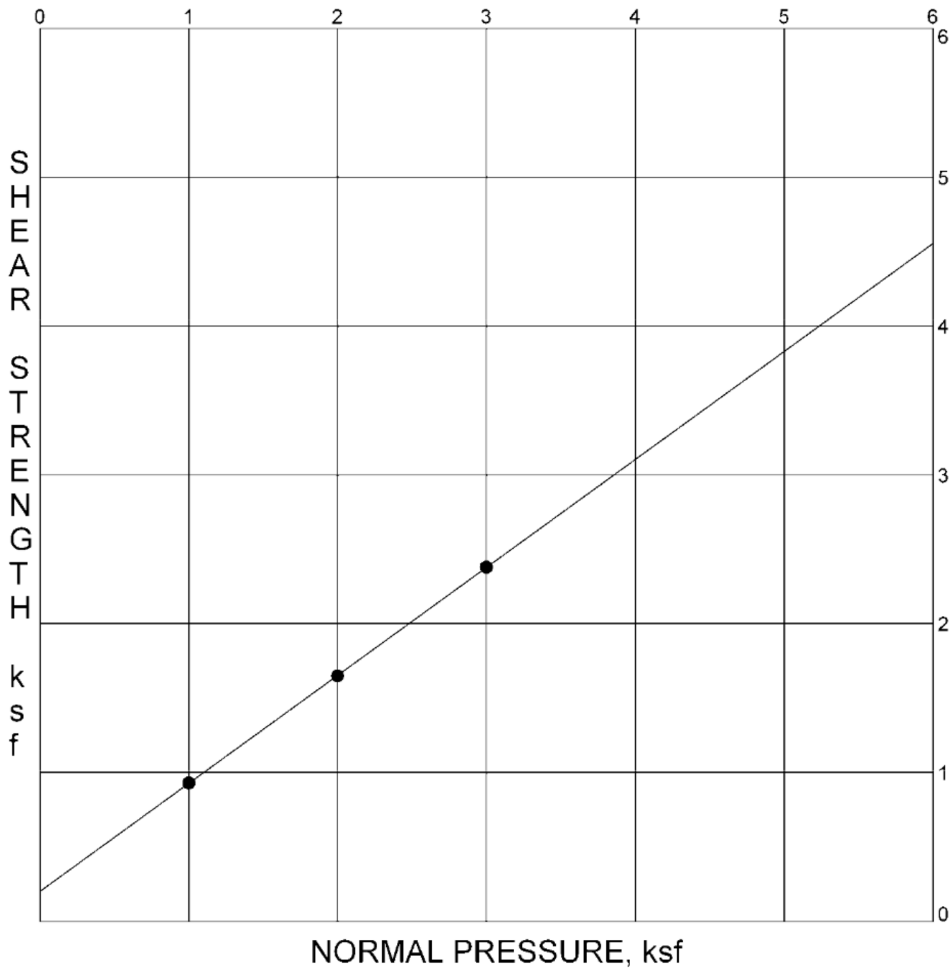
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 11.5	Dry Density: 113.9	Phi (deg): 35.0
Saturated: 17.7		Cohesion (ksf): 0.300

SHEAR TEST DIAGRAM

Figure S.9

PROJECT LOCATION: 1222 S. Hope Street
PROJECT NO.: 5076
SAMPLE LOCATION: B-2 @ 35.0
DESCRIPTION: Qa



Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 10.7	Dry Density: 115.1	Phi (deg): 36.0
Saturated: 17.1		Cohesion (ksf): 0.200

SHEAR TEST DIAGRAM

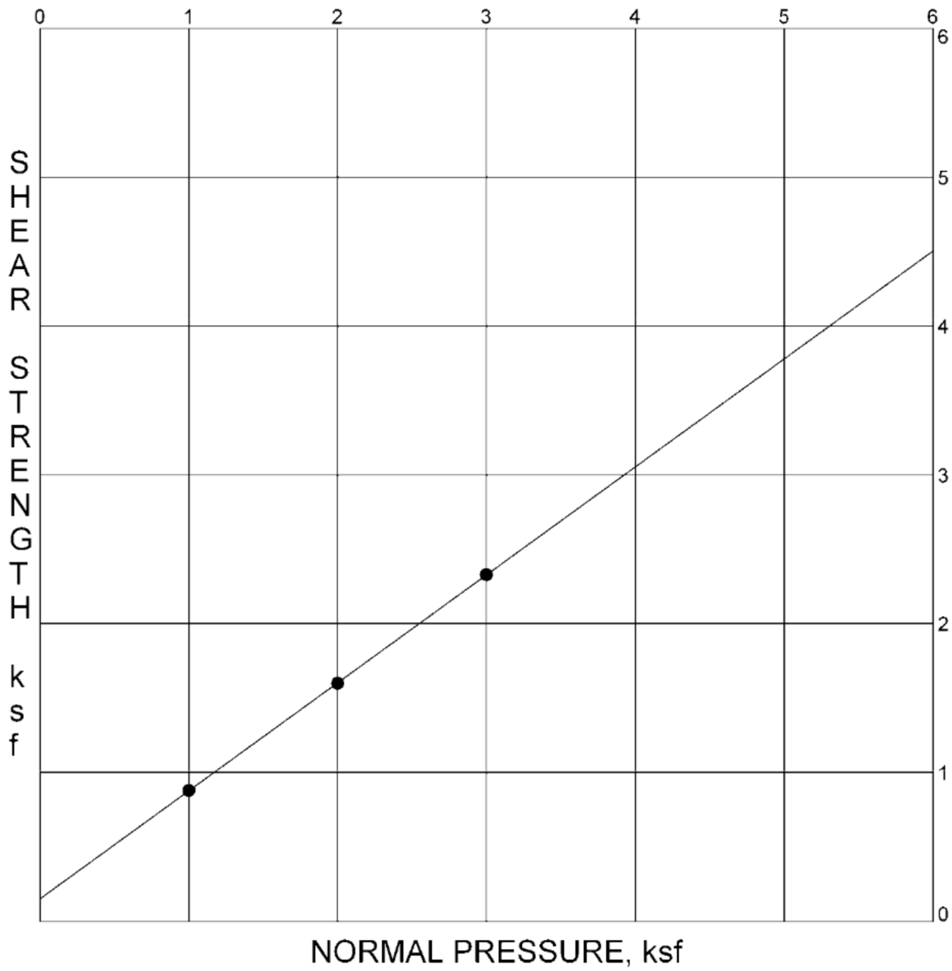
Figure S.10

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-2 @ 80.0

DESCRIPTION: Qa



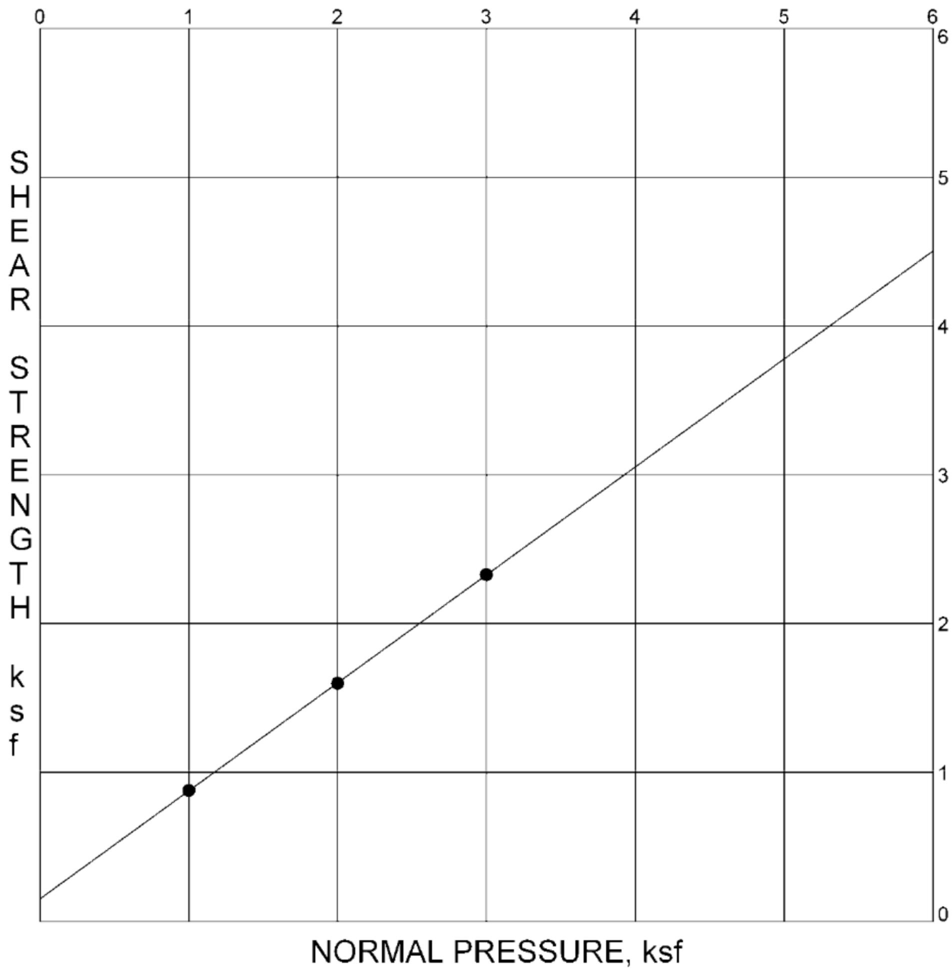
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 2.5	Dry Density: 118.9	Phi (deg): 36.0
Saturated: 15.4		Cohesion (ksf): 0.150

SHEAR TEST DIAGRAM

Figure S.11

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076
SAMPLE LOCATION: B-3 @ 5.0 | DESCRIPTION: Qa



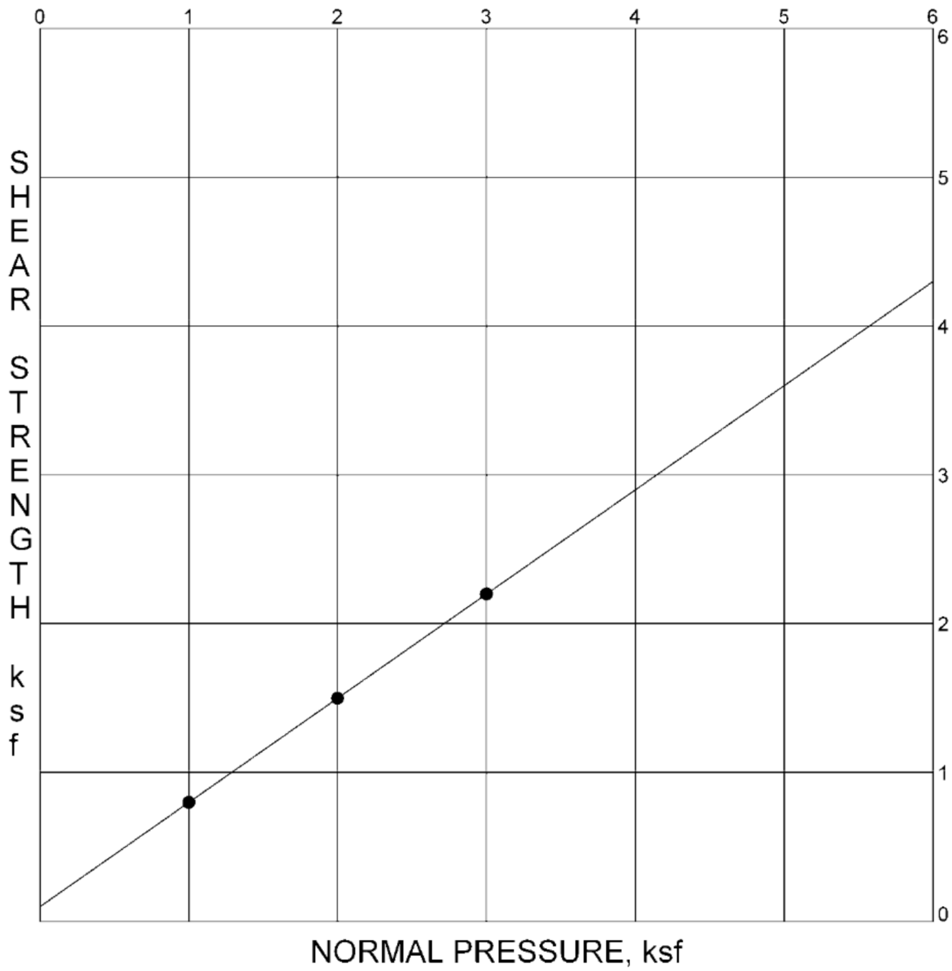
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 3.8	Dry Density: 117.4	Phi (deg): 36.0
Saturated: 16.1		Cohesion (ksf): 0.150

SHEAR TEST DIAGRAM

Figure S.12

PROJECT LOCATION: 1222 S. Hope Street   PROJECT NO.: 5076
SAMPLE LOCATION: B-3 @ 15.0   DESCRIPTION: Qa



Test Results

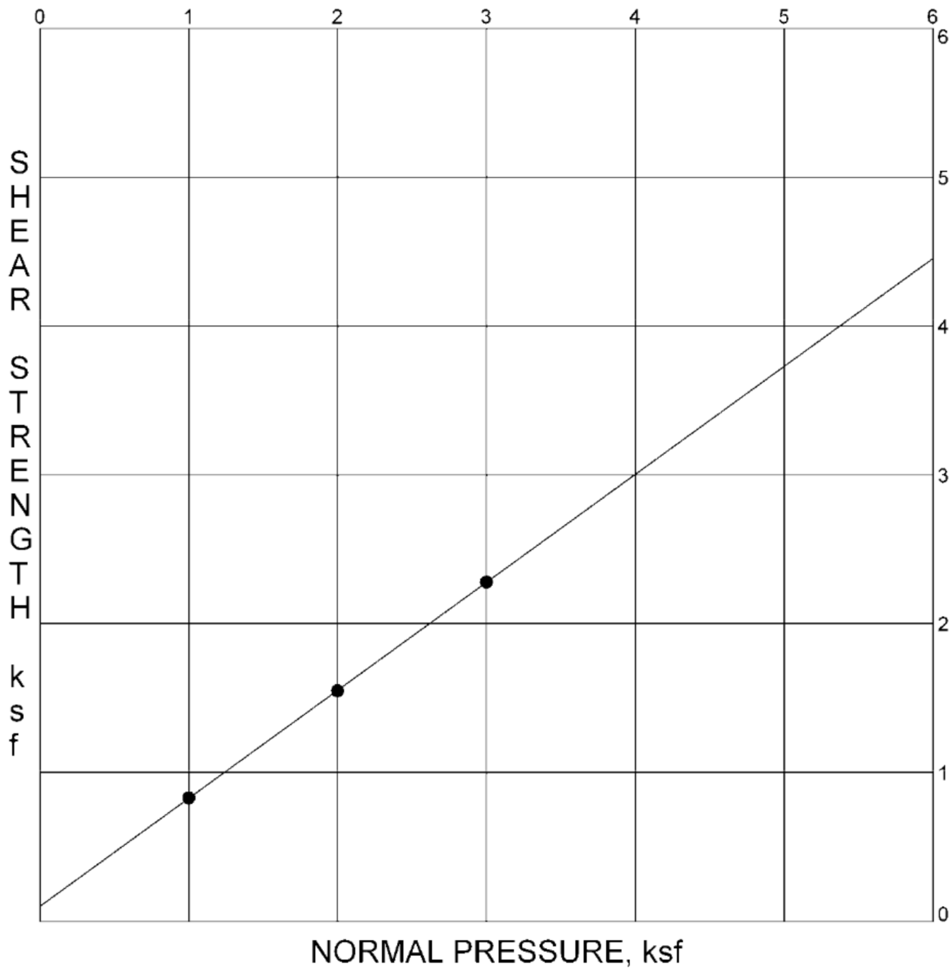
Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 2.4	Dry Density: 110.2	Phi (deg): 35.0
Saturated: 19.5		Cohesion (ksf): 0.100

SHEAR TEST DIAGRAM

Figure S.13

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076

SAMPLE LOCATION: B-3 @ 25.0 | DESCRIPTION: Qa



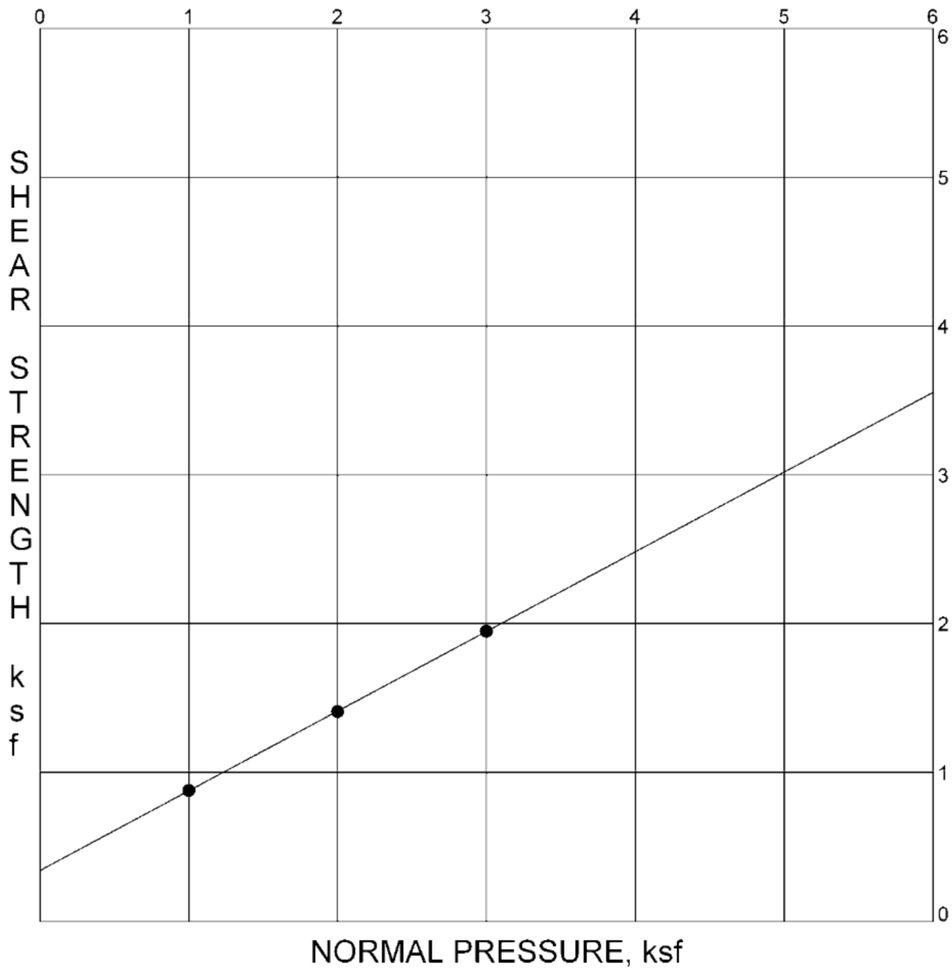
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 4.8	Dry Density: 118.0	Phi (deg): 36.0
Saturated: 15.8		Cohesion (ksf): 0.100

SHEAR TEST DIAGRAM

Figure S.14

PROJECT LOCATION: 1222 S. Hope Street
PROJECT NO.: 5076
SAMPLE LOCATION: B-3 @ 50.0
DESCRIPTION: Qa



Test Results

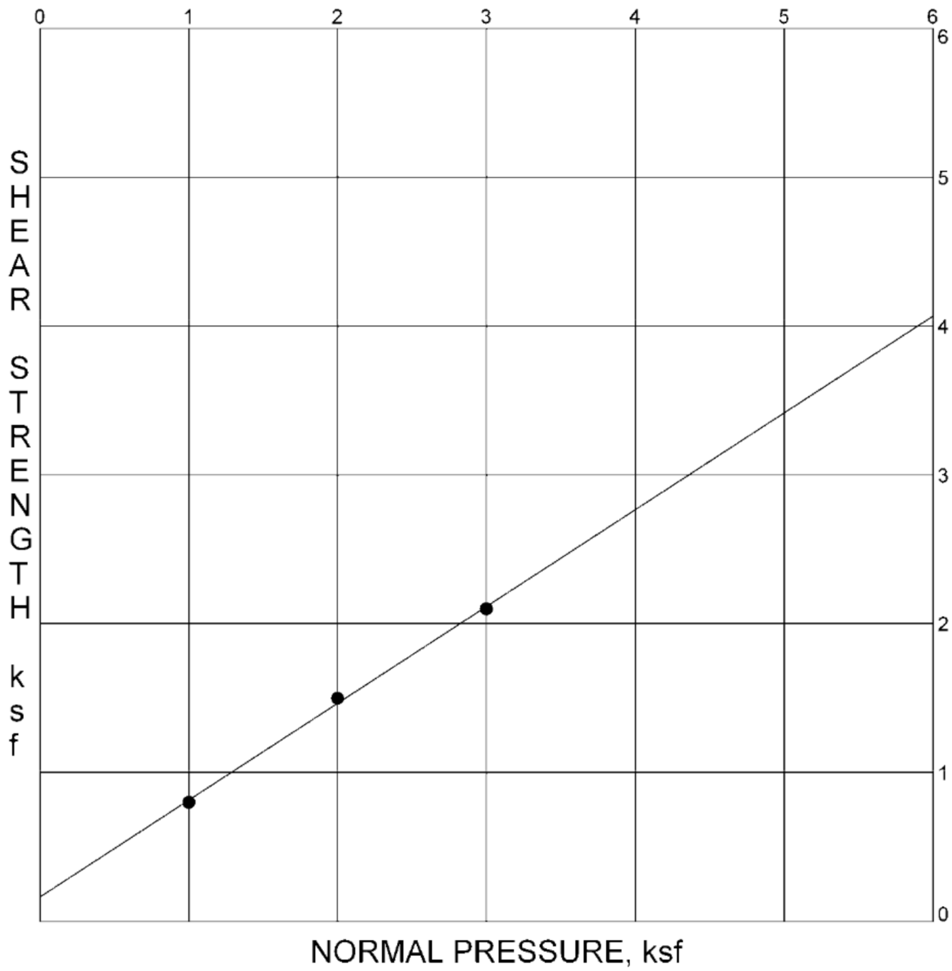
Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 13.3	Dry Density: 100.3	Phi (deg): 28.0
Saturated: 25.1		Cohesion (ksf): 0.350

SHEAR TEST DIAGRAM

Figure S.15

PROJECT LOCATION: 1222 S. Hope Street — PROJECT NO.: 5076

SAMPLE LOCATION: B-4 @ 10.0 — DESCRIPTION: Qal



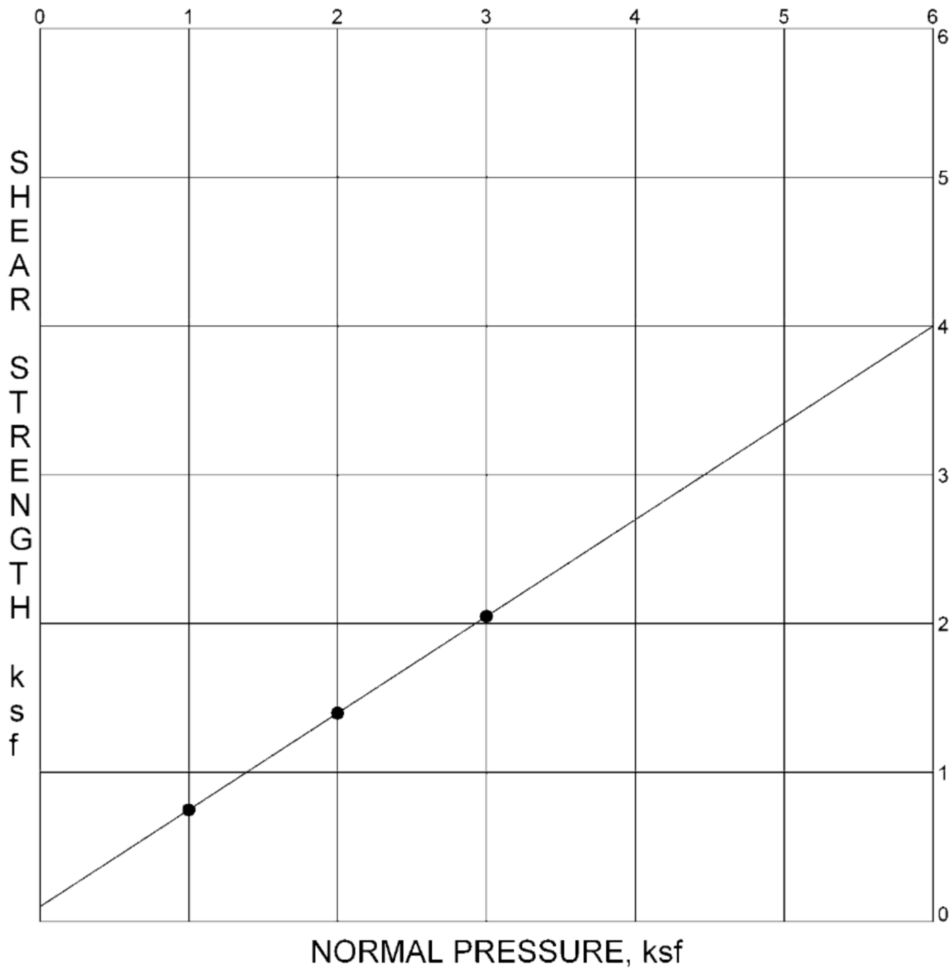
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 4.2	Dry Density: 116.8	Phi (deg): 33.0
Saturated: 16.4		Cohesion (ksf): 0.150

SHEAR TEST DIAGRAM

Figure S.16

PROJECT LOCATION: 1222 S. Hope Street	PROJECT NO.: 5076
SAMPLE LOCATION: B-4 @ 20.0	DESCRIPTION:



Test Results

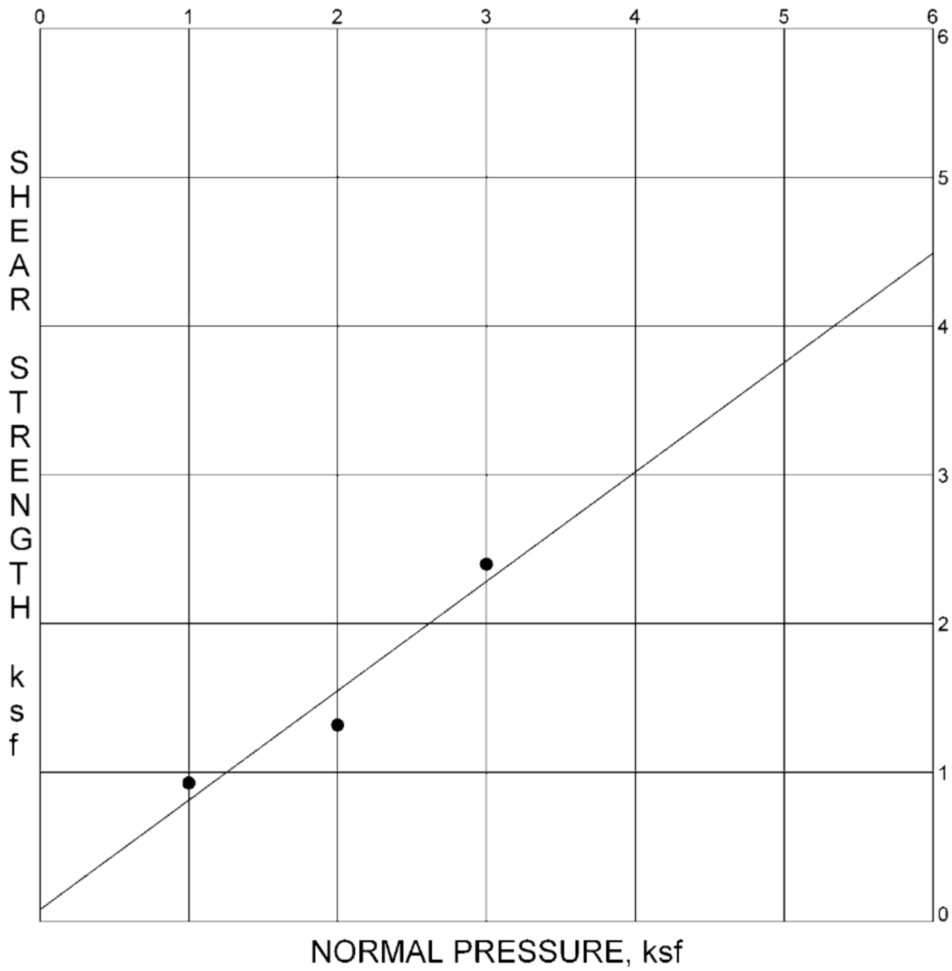
Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 8.7	Dry Density: 115.2	Phi (deg): 33.0
Saturated: 17.1		Cohesion (ksf): 0.100

SHEAR TEST DIAGRAM

Figure S.17

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076

SAMPLE LOCATION: B-4 @ 35.0 | DESCRIPTION:



Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 4.1	Dry Density: 116.5	Phi (deg): 36.0
Saturated: 16.5		Cohesion (ksf): 0.075

SHEAR TEST DIAGRAM

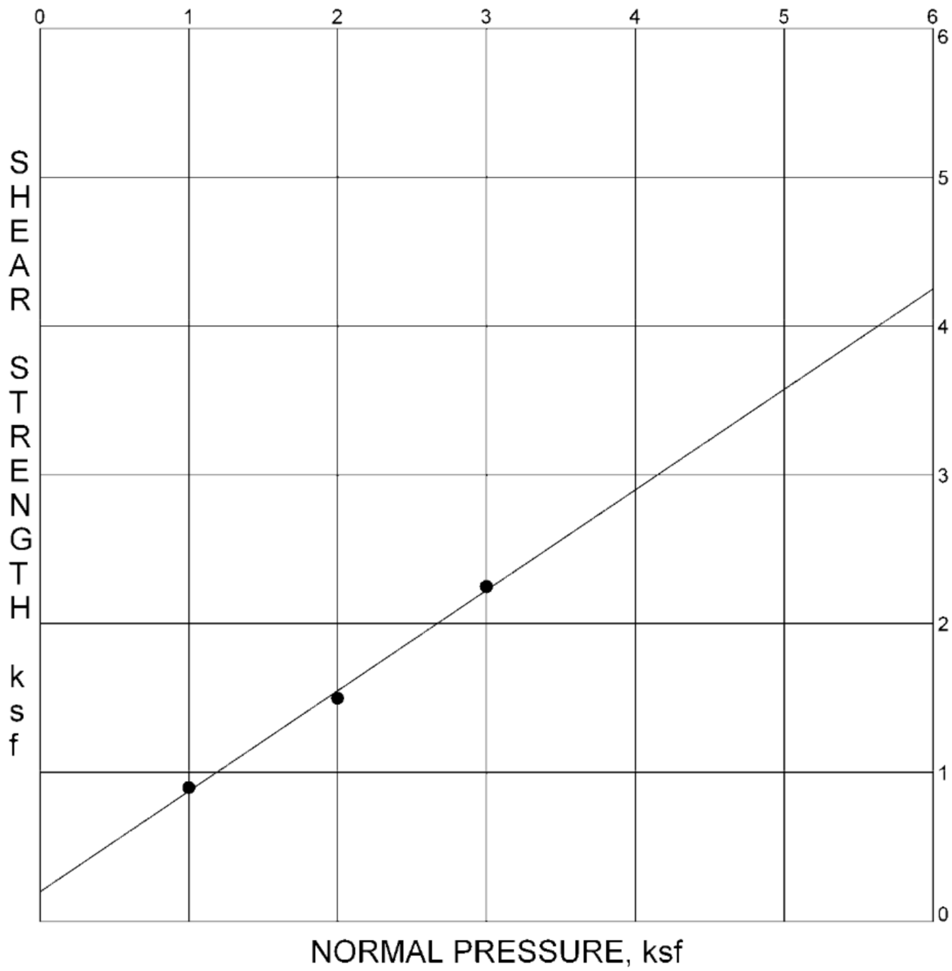
Figure S.18

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-4 @ 45.0

DESCRIPTION:



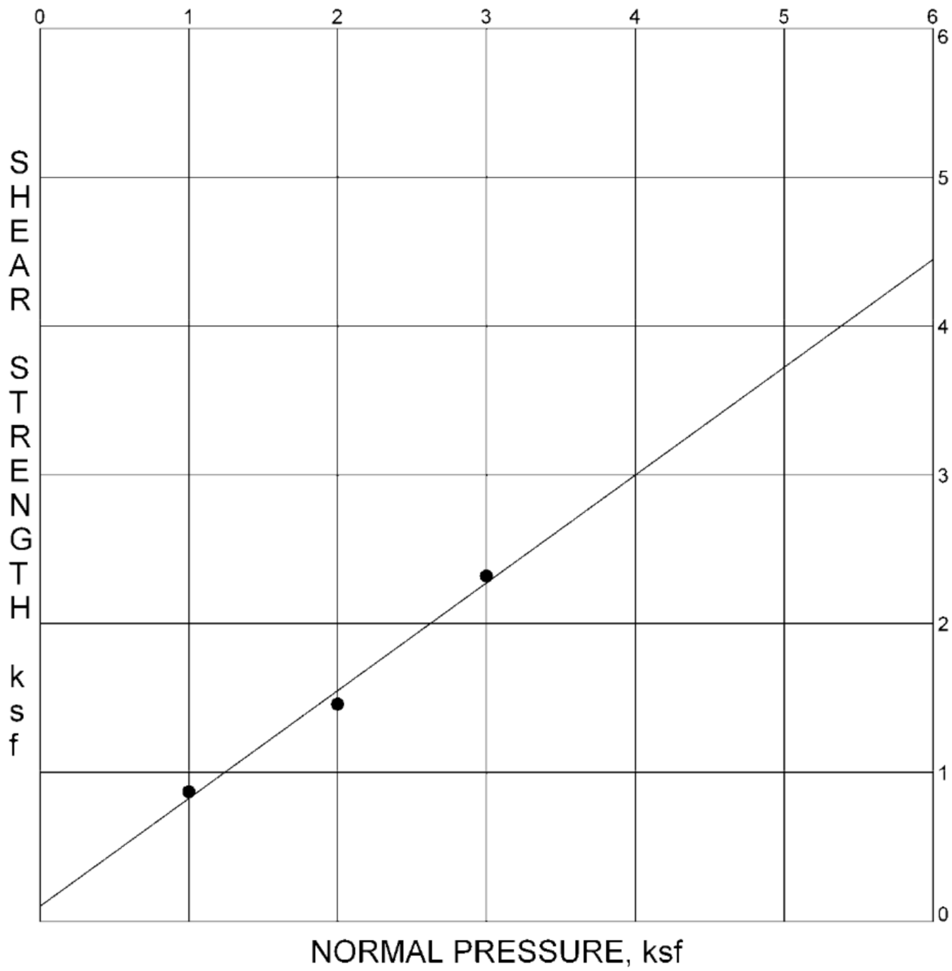
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 5.7	Dry Density: 117.3	Phi (deg): 34.0
Saturated: 16.1		Cohesion (ksf): 0.200

SHEAR TEST DIAGRAM

Figure S.19

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076
SAMPLE LOCATION: B-4 @ 55.0 | DESCRIPTION:



Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 12.3	Dry Density: 107.1	Phi (deg): 36.0
Saturated: 21.2		Cohesion (ksf): 0.100

SHEAR TEST DIAGRAM

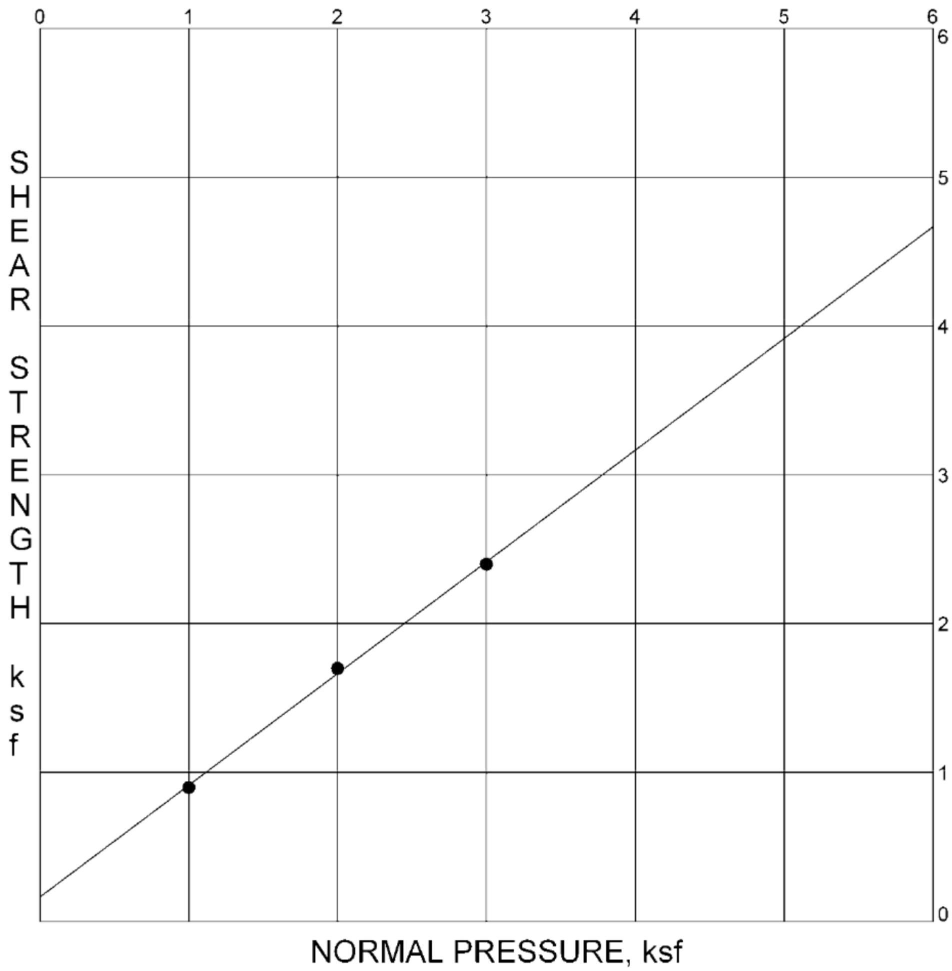
Figure S.20

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-4 @ 65.0

DESCRIPTION:



Test Results

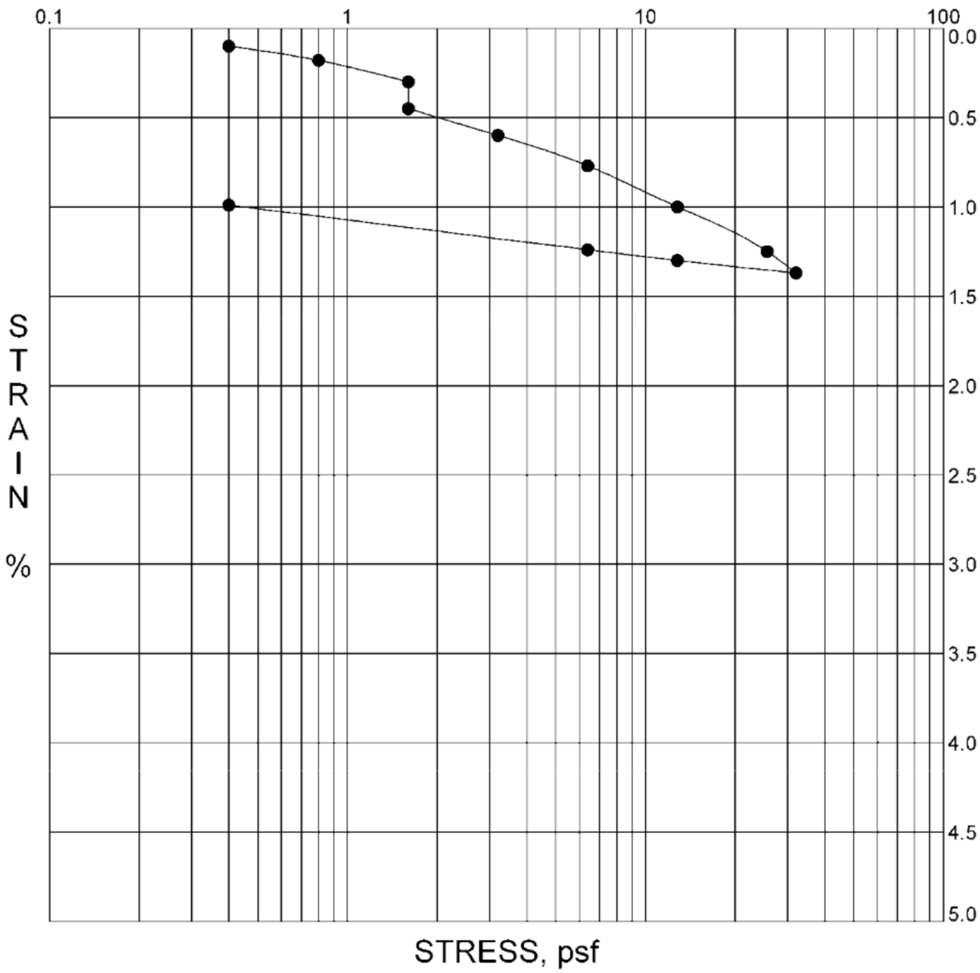
Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 14.4	Dry Density: 113.7	Phi (deg): 37.0
Saturated: 17.8		Cohesion (ksf): 0.150

SHEAR TEST DIAGRAM

Figure S.21

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076

SAMPLE LOCATION: B-1 @ 40.0 | DESCRIPTION: Qa



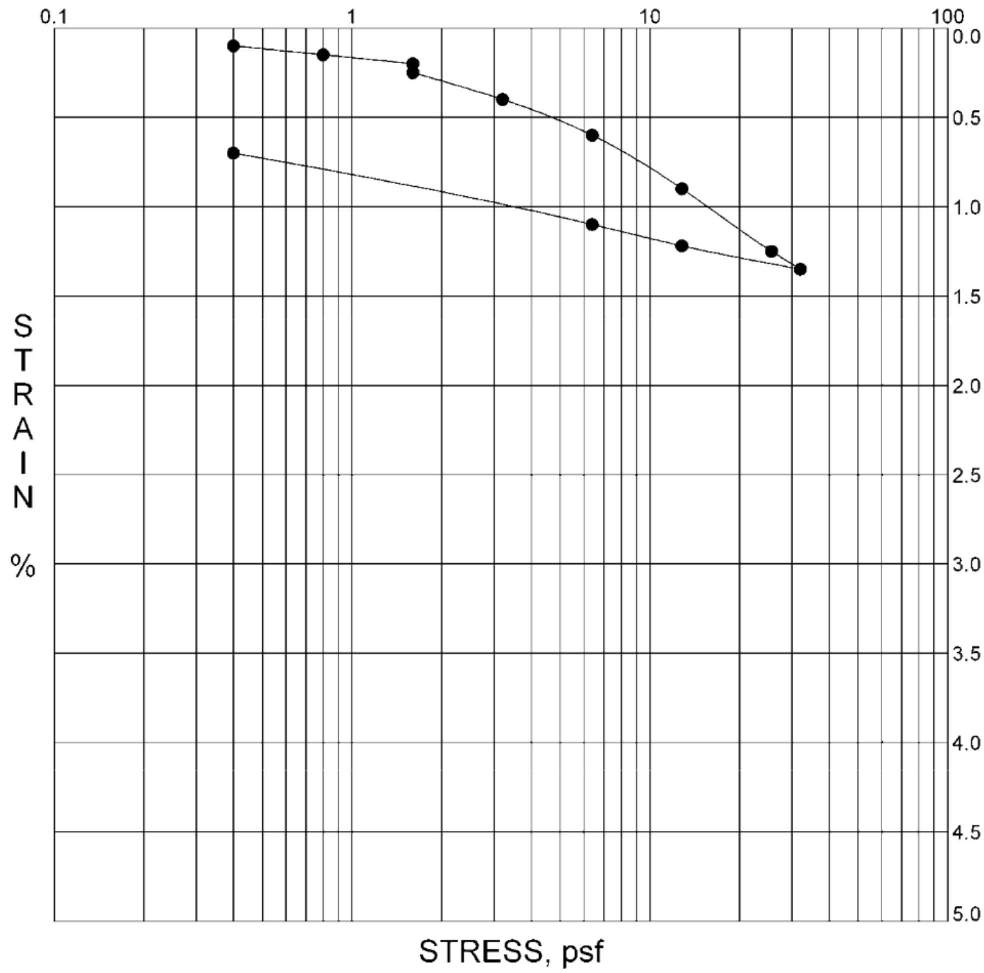
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 9.2	Dry Density: 110.9	1600 lbs.

CONSOLIDATION TEST DIAGRAM

Figure C.1

PROJECT LOCATION: 1222 S. Hope Street	PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 45.0	DESCRIPTION: Qa



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 11.0	Dry Density: 107.3	1600 lbs.

CONSOLIDATION TEST DIAGRAM

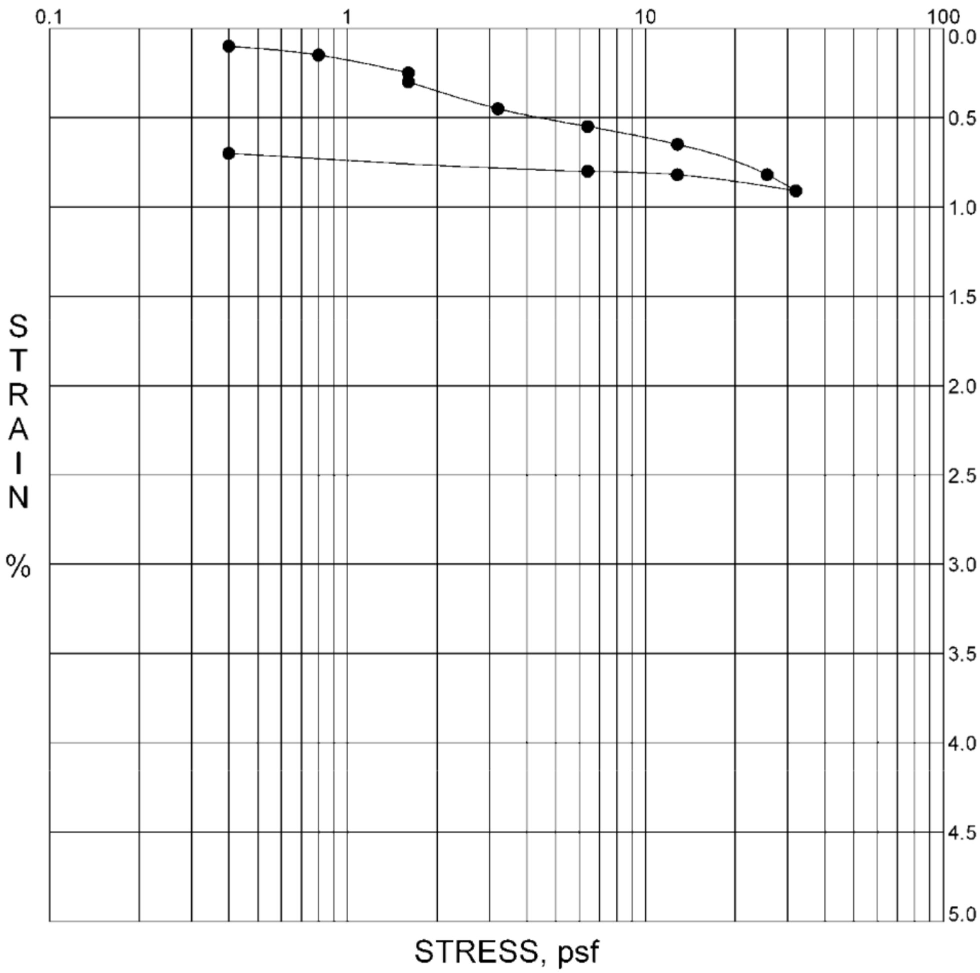
Figure C 2

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-1 @ 60.0

DESCRIPTION: Qa



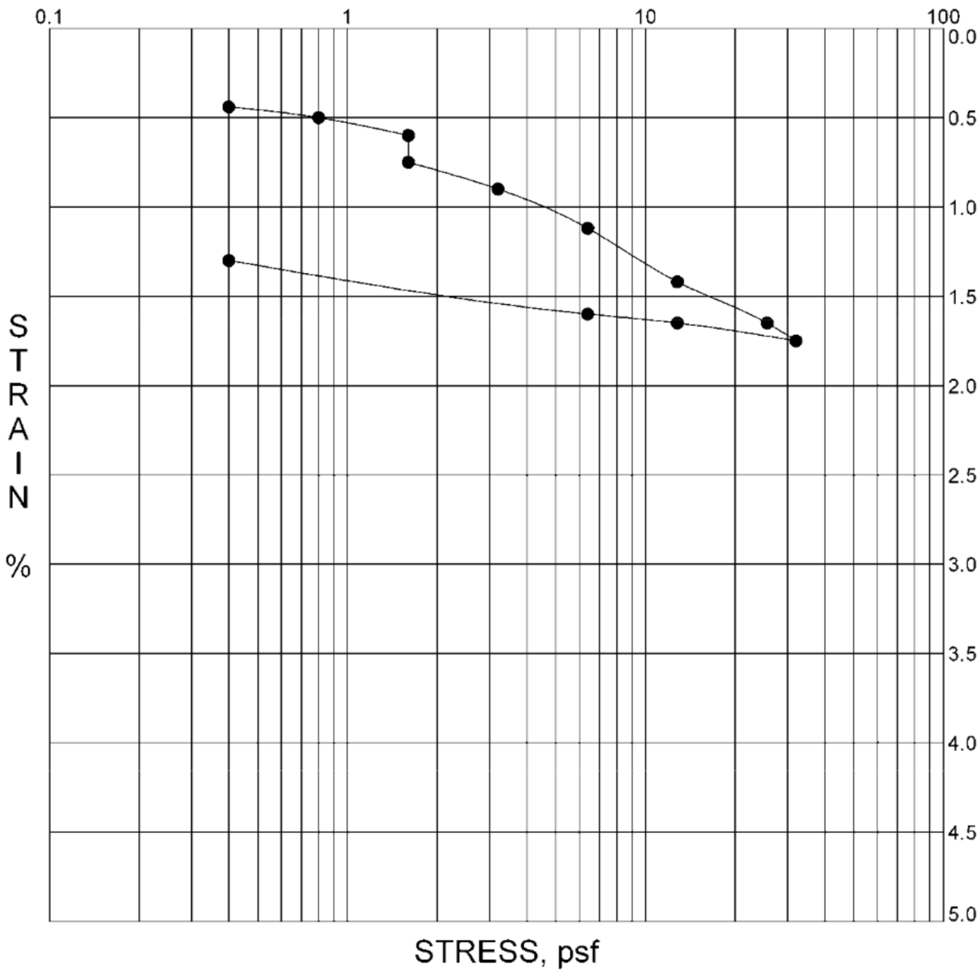
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 10.2	Dry Density: 105.3	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C-3

PROJECT LOCATION: 1222 S. Hope Street	PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 75.0	DESCRIPTION: Qa



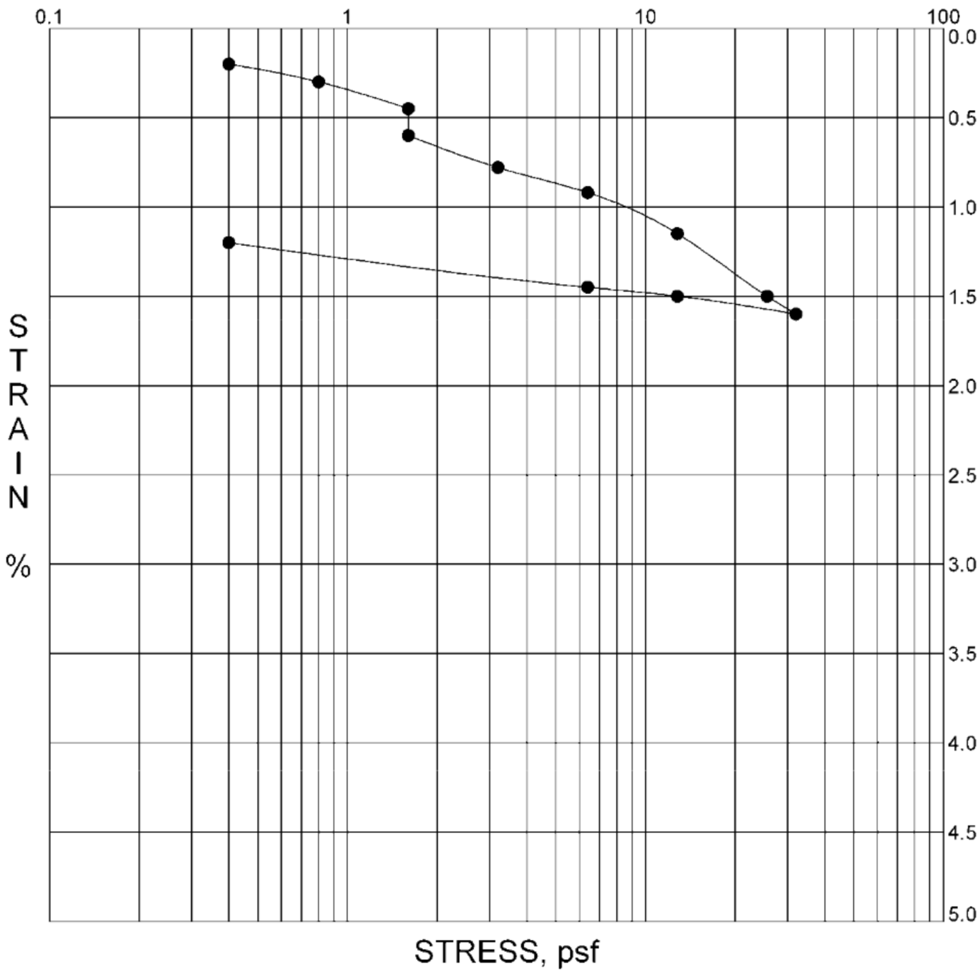
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 8.0	Dry Density: 111.4	1600 lbs.

CONSOLIDATION TEST DIAGRAM

Figure C 4

PROJECT LOCATION: 1222 S. Hope Street	PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 90.0	DESCRIPTION: Qa



Test Results

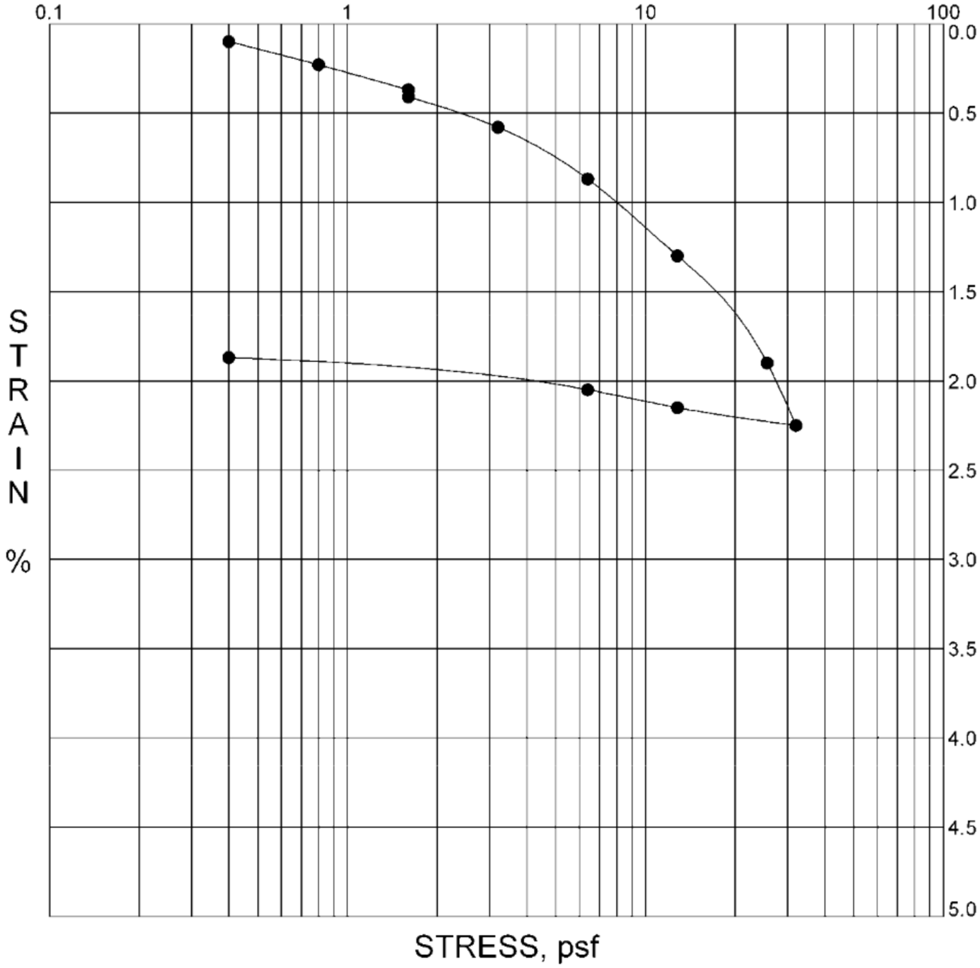
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 5.2	Dry Density: 111.2	1600 lbs.

CONSOLIDATION TEST DIAGRAM

Figure C.5

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076

SAMPLE LOCATION: B-1 @ 100.0 | DESCRIPTION: Qa



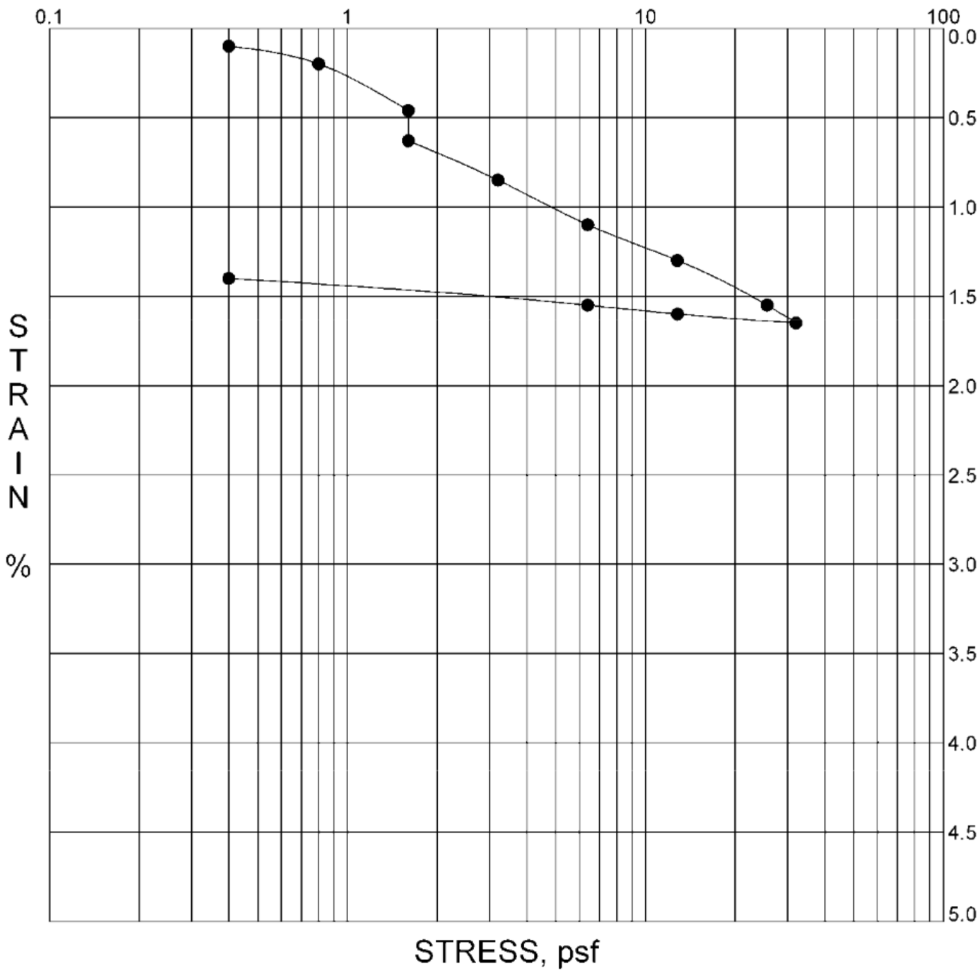
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 15.2	Dry Density: 95.7	1600 lbs.

CONSOLIDATION TEST DIAGRAM

Figure C.6

PROJECT LOCATION: 1222 S. Hope Street	PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 105.0	DESCRIPTION: Qa



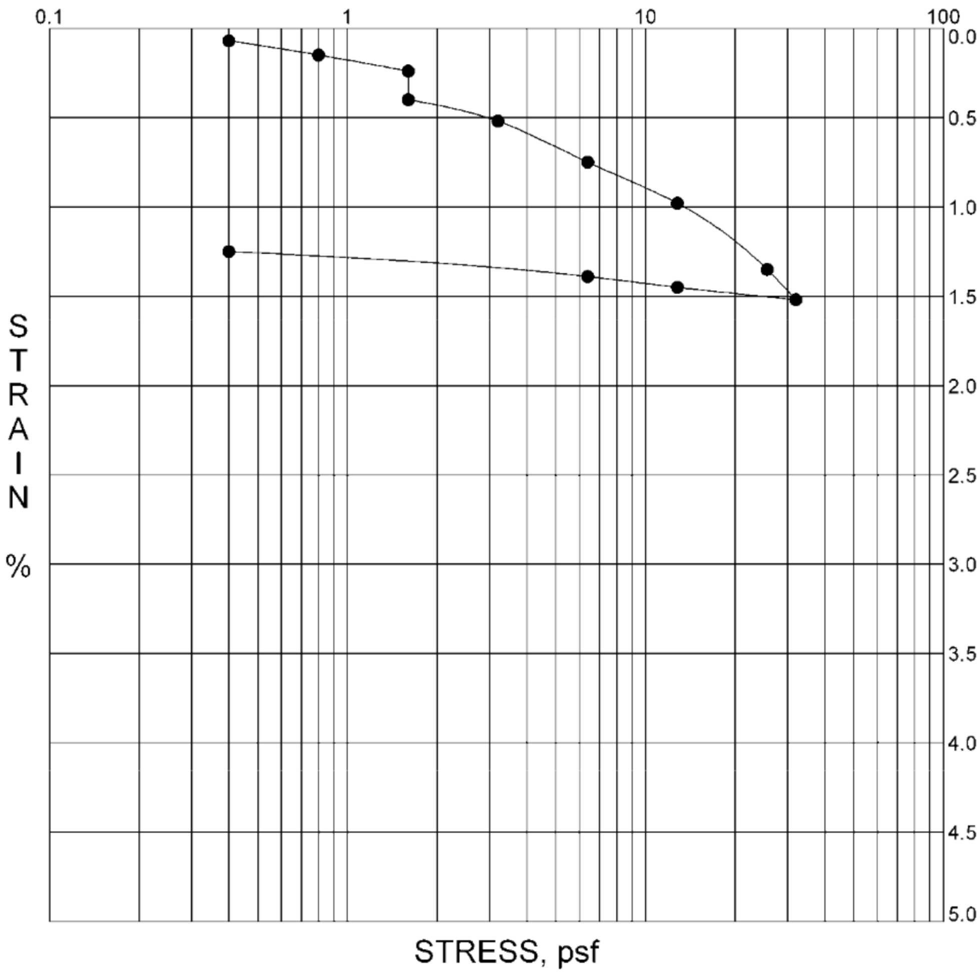
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 1.5	Dry Density: 115.2	1600 lbs.

CONSOLIDATION TEST DIAGRAM

Figure C.7

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 115.0 | DESCRIPTION: Qa



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 7.0	Dry Density: 117.9	1600 lbs.

CONSOLIDATION TEST DIAGRAM

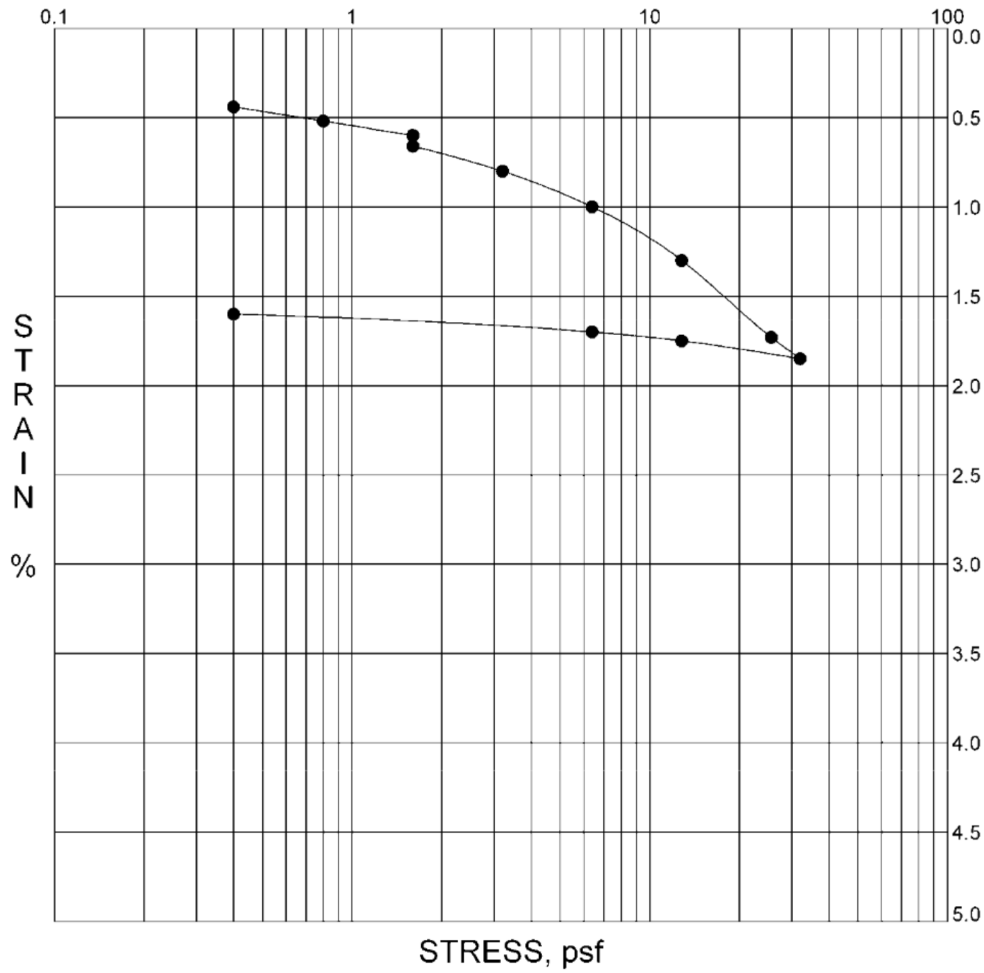
Figure C 8

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-1 @ 135.0

DESCRIPTION: Qa



Test Results

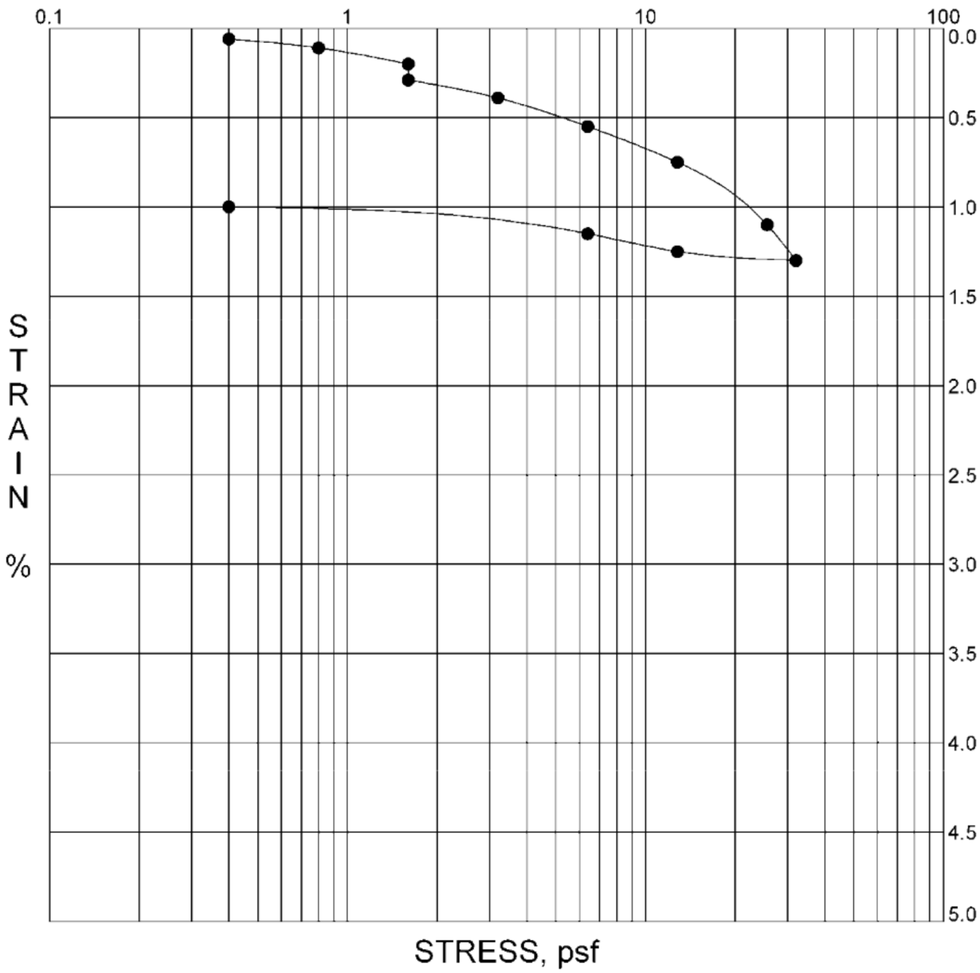
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 3.7	Dry Density: 111.1	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C 9

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076

SAMPLE LOCATION: B-1 @ 145.0 | DESCRIPTION: Qa



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 5.1	Dry Density: 115.3	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

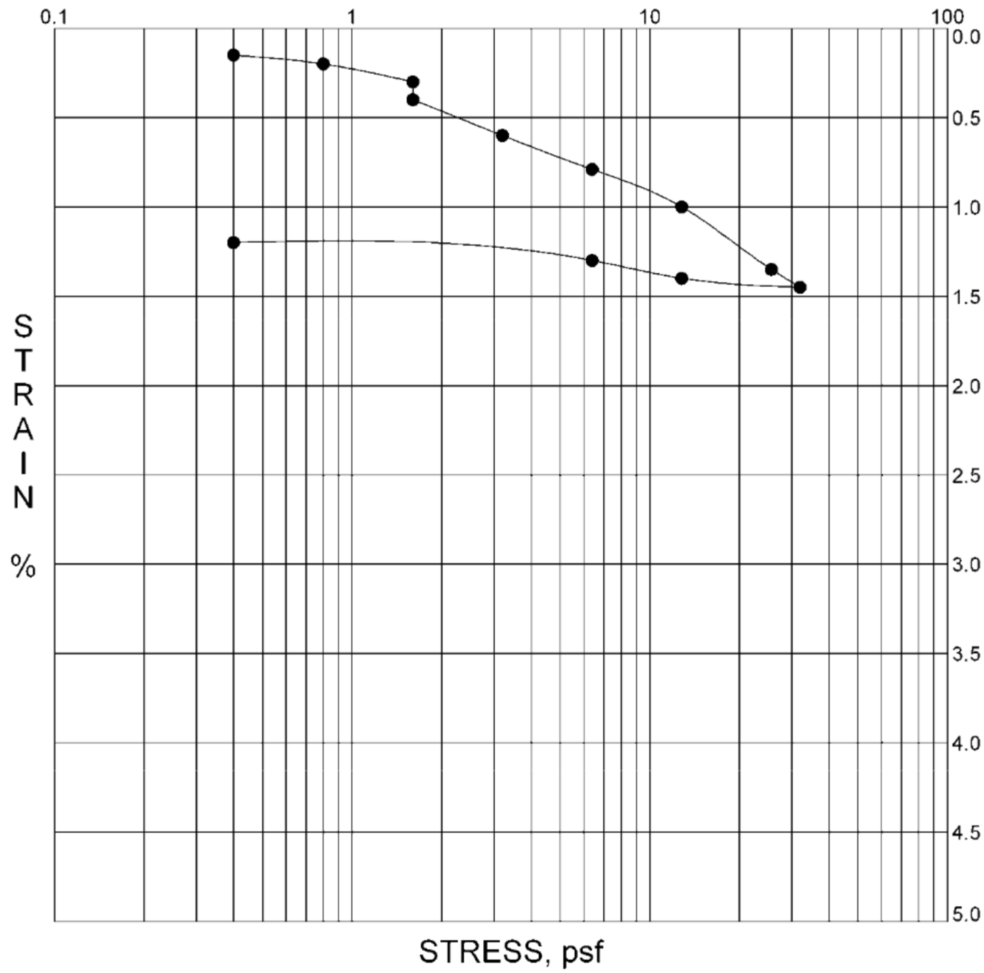
Figure C 10

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-1 @ 160.0

DESCRIPTION: Qa



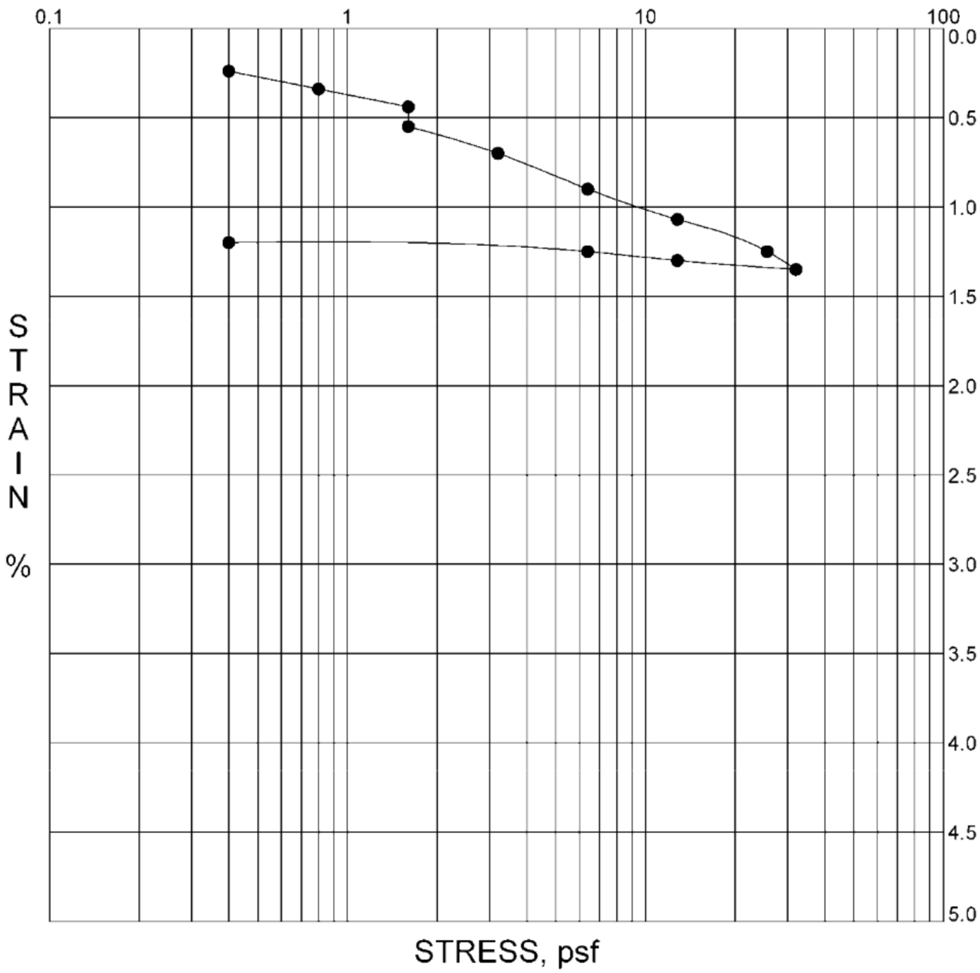
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 14.9	Dry Density: 103.2	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.11

PROJECT LOCATION: 1222 S. Hope Street PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 170.0 DESCRIPTION: Qa



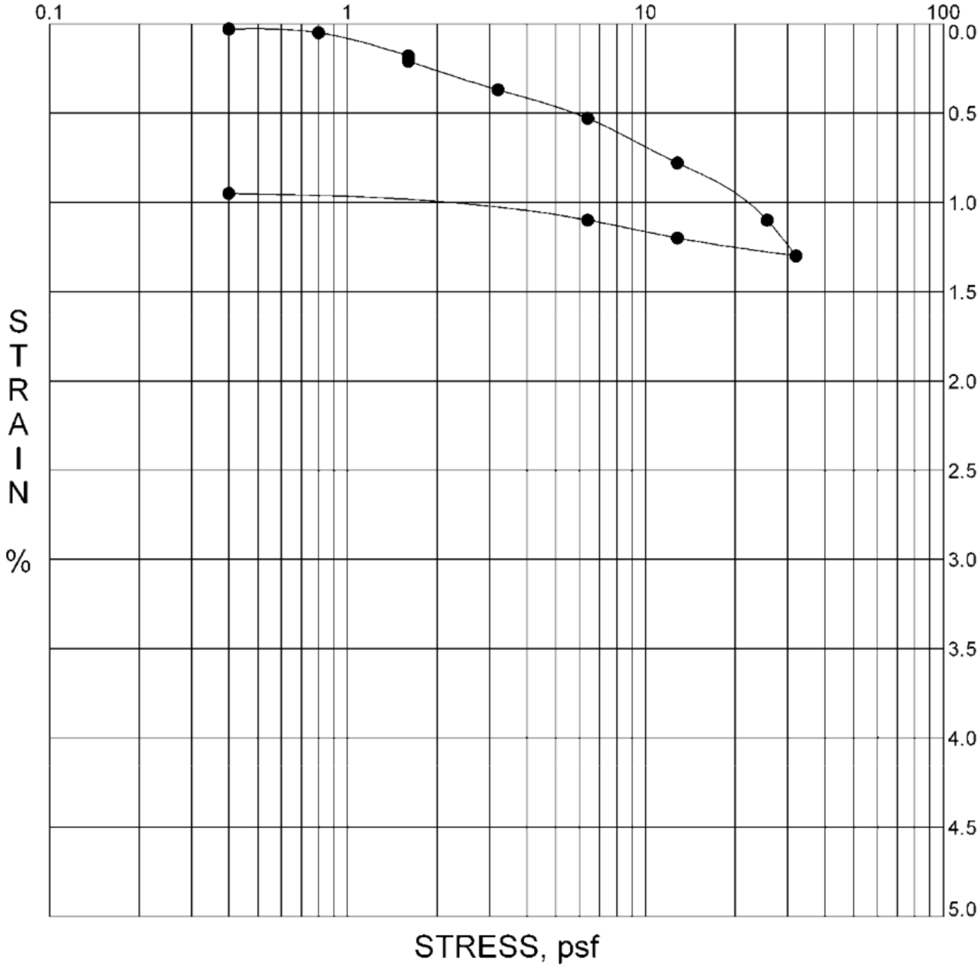
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 10.2	Dry Density: 111.4	1600 lbs.

CONSOLIDATION TEST DIAGRAM

Figure C 12

PROJECT LOCATION: 1222 S. Hope Street PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 180.0 DESCRIPTION: Qa

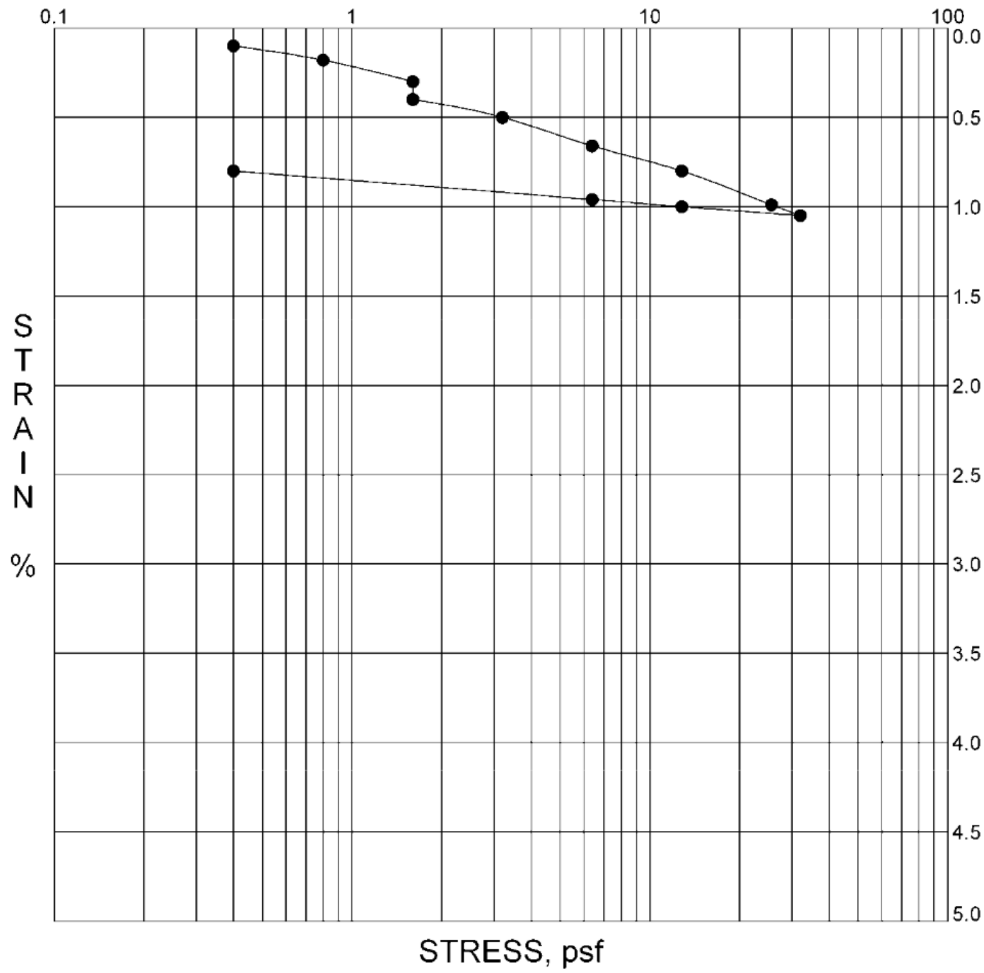


Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 9.0	Dry Density: 114.9	1600 lbs.

CONSOLIDATION TEST DIAGRAM Figure C 13

PROJECT LOCATION: 1222 S. Hope Street	PROJECT NO.: 5076
SAMPLE LOCATION: B-1 @ 190.0	DESCRIPTION: Qa



Test Results

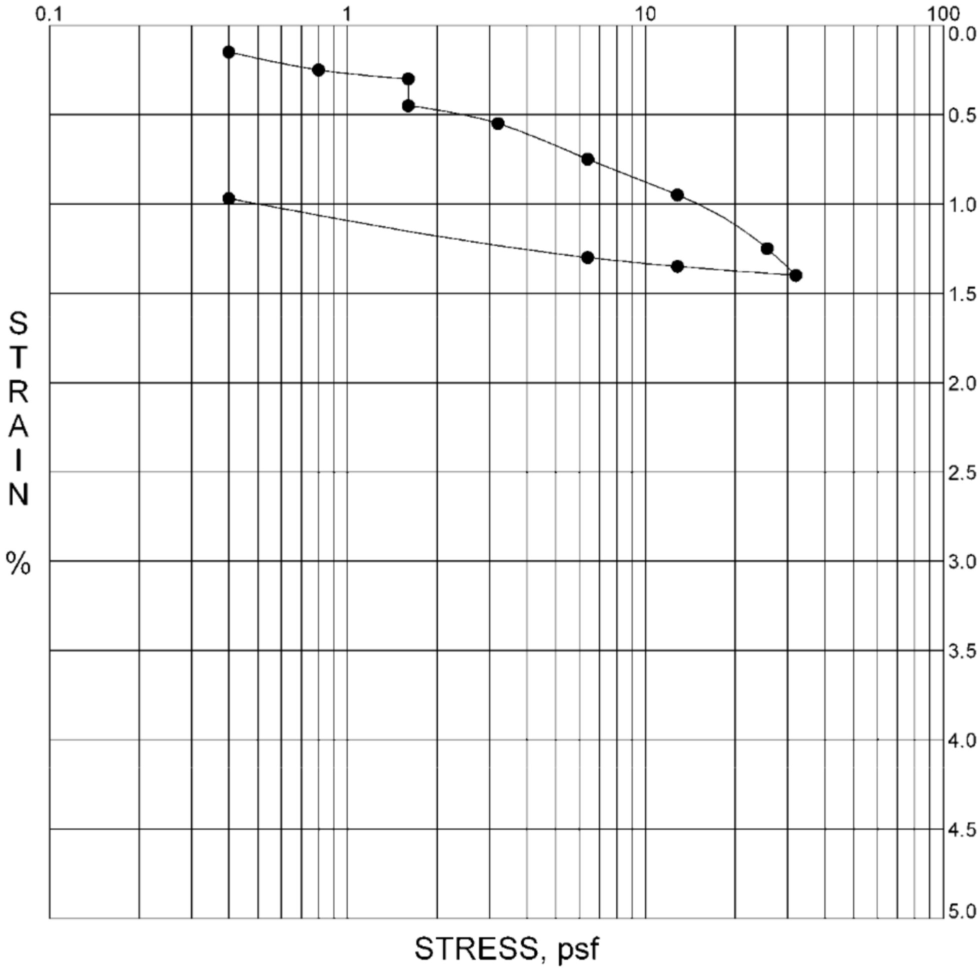
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 6.6	Dry Density: 116.8	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C 14

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076

SAMPLE LOCATION: B-2 @ 45.0 | DESCRIPTION: Qa



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 12.1	Dry Density: 111.8	1600 lbs.

CONSOLIDATION TEST DIAGRAM

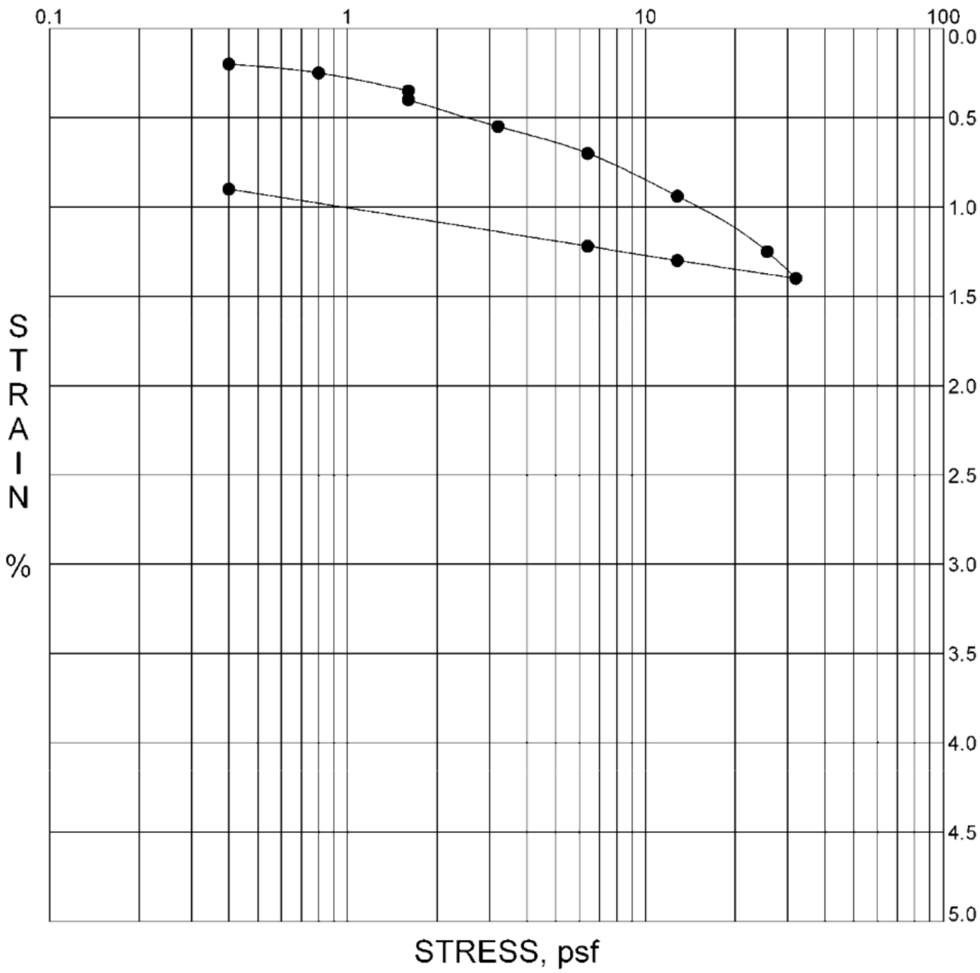
Figure C.15

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-2 @ 60.0

DESCRIPTION: Qa



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 10.0	Dry Density: 116.2	1600 lbs.

CONSOLIDATION TEST DIAGRAM

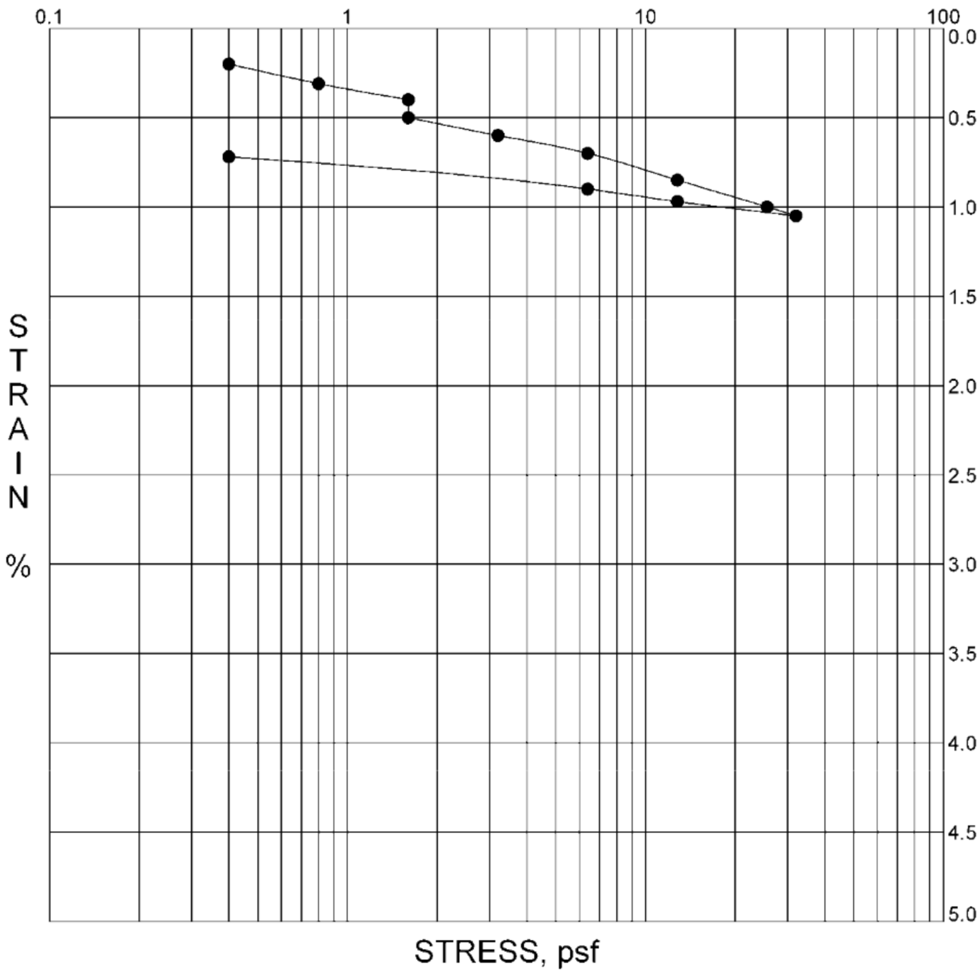
Figure C 16

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-2 @ 65.0

DESCRIPTION: Qa



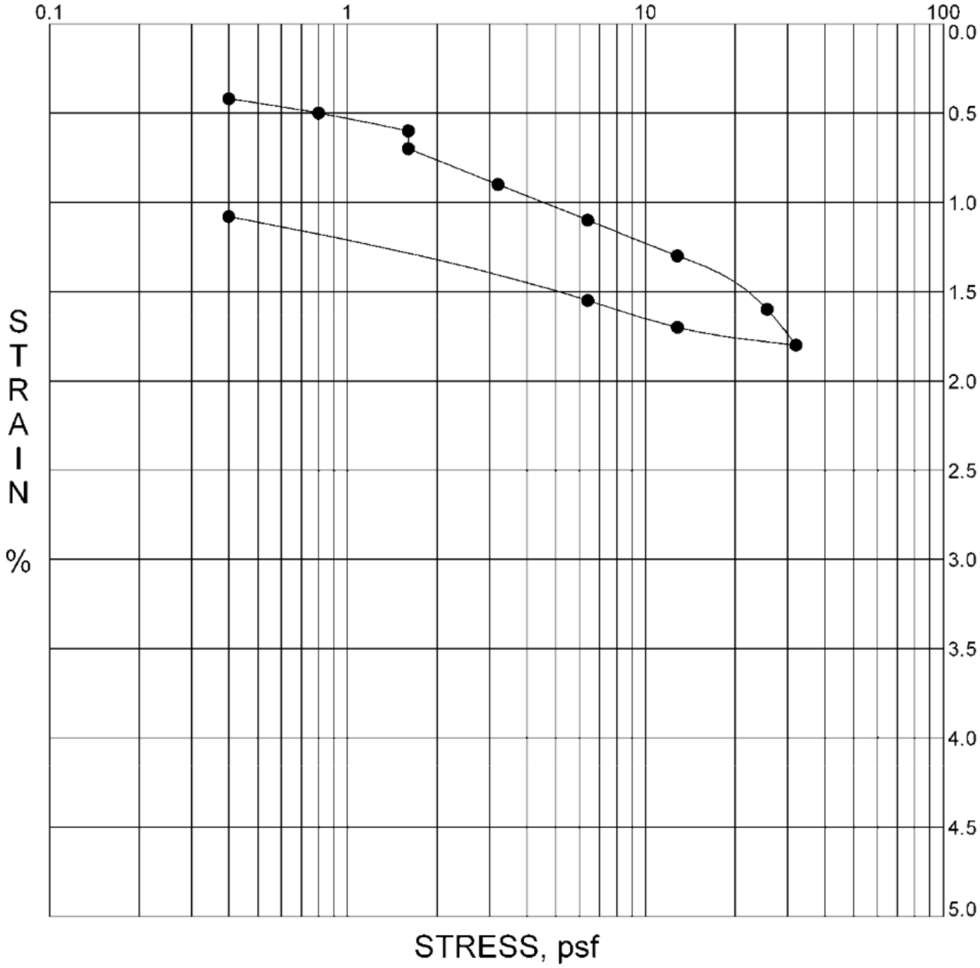
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 10.1	Dry Density: 119.5	1600 lbs.

CONSOLIDATION TEST DIAGRAM

Figure C 17

PROJECT LOCATION: 1222 S. Hope Street	PROJECT NO.: 5076
SAMPLE LOCATION: B-2 @ 75.0	DESCRIPTION: Qa



Test Results

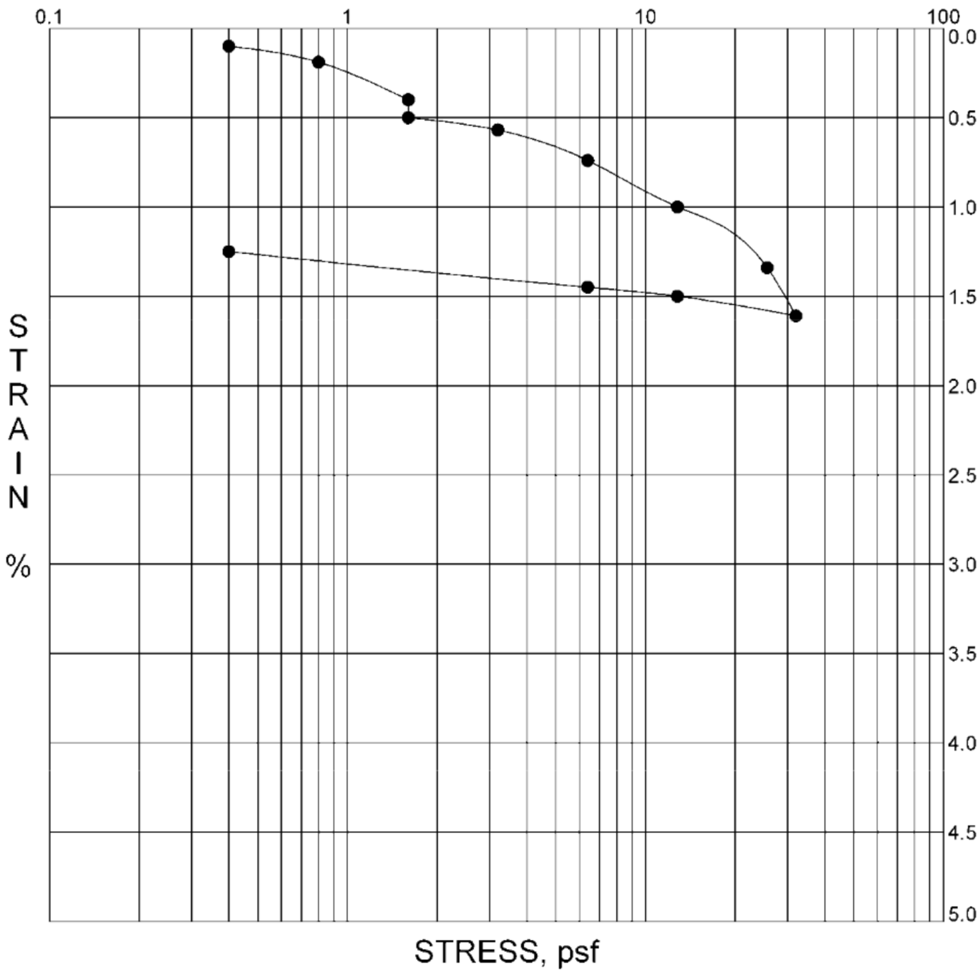
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 3.0	Dry Density: 118.5	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C 18

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076

SAMPLE LOCATION: B-3 @ 45.0 | DESCRIPTION: Qa



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 15.0	Dry Density: 103.9	1600 lbs.

CONSOLIDATION TEST DIAGRAM

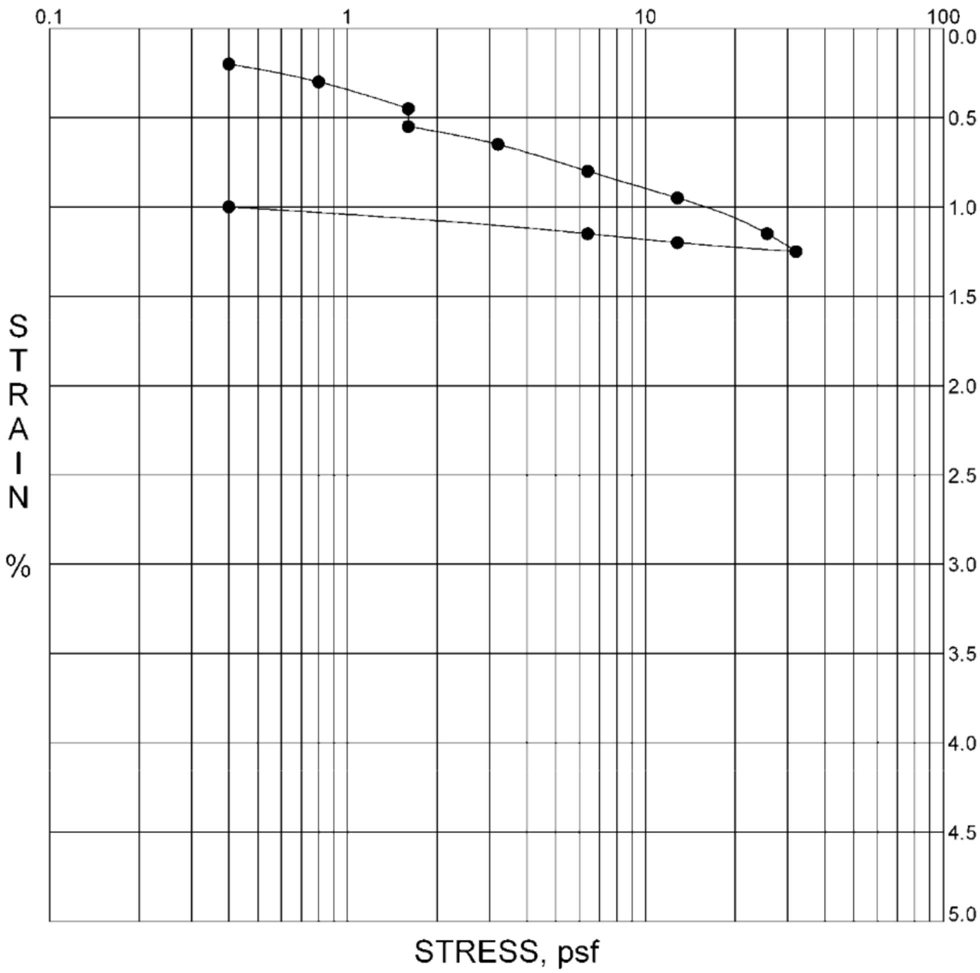
Figure C 19

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-3 @ 55.0

DESCRIPTION: Qa



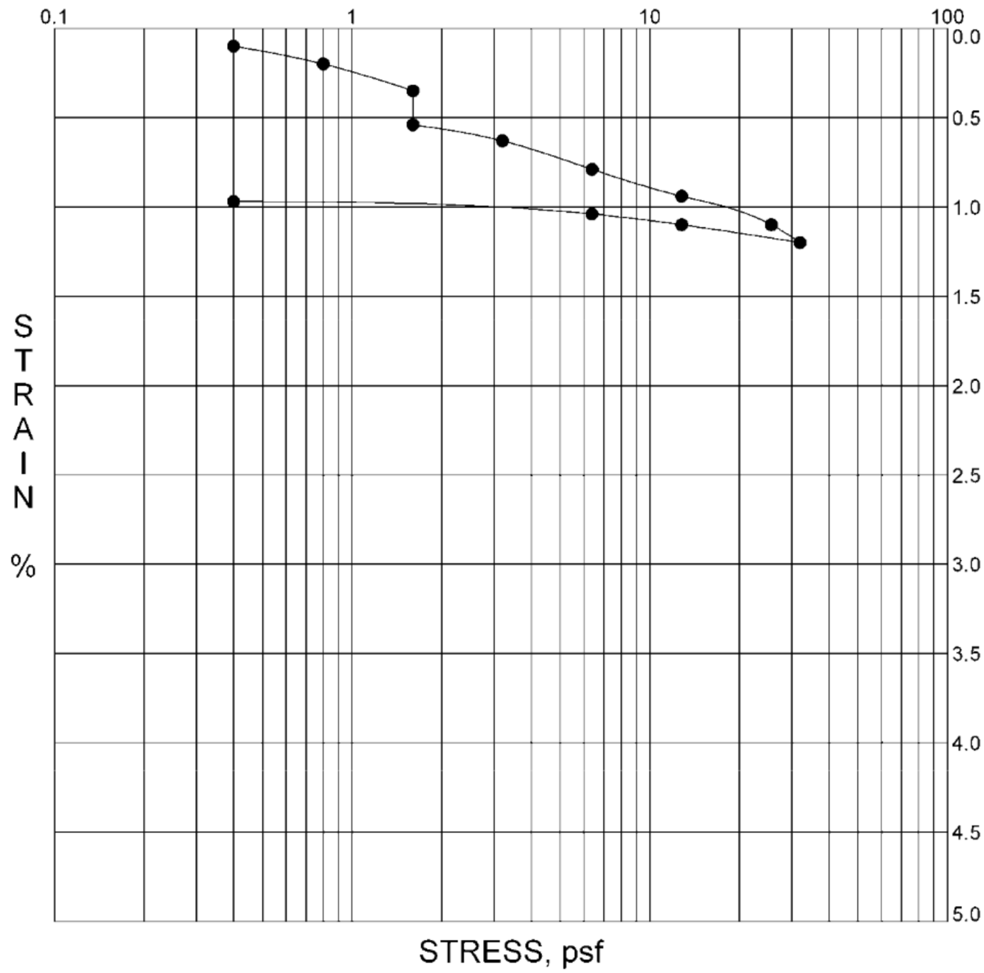
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 11.6	Dry Density: 113.0	1600 lbs.

CONSOLIDATION TEST DIAGRAM

Figure C 20

PROJECT LOCATION: 1222 S. Hope Street	PROJECT NO.: 5076
SAMPLE LOCATION: B-3 @ 70.0	DESCRIPTION: Qa



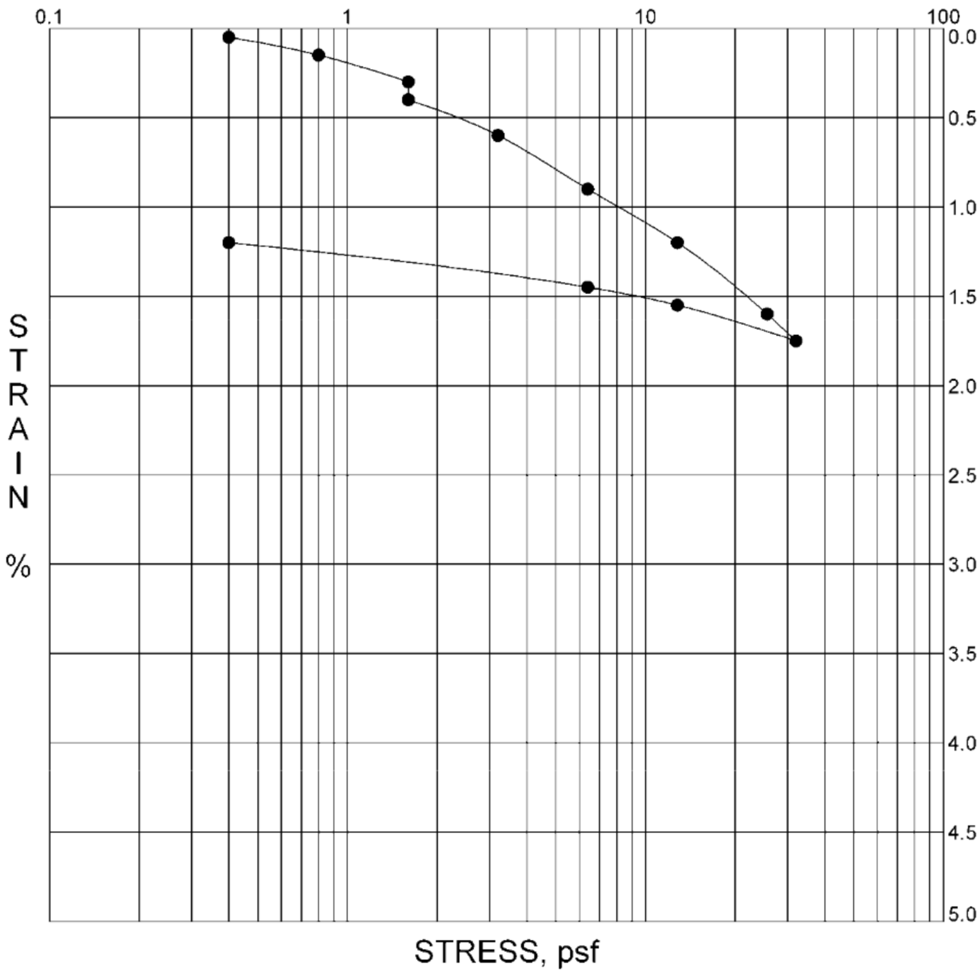
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 4.9	Dry Density: 122.6	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C 21

PROJECT LOCATION: 1222 S. Hope Street	PROJECT NO.: 5076
SAMPLE LOCATION: B-3 @ 75.0	DESCRIPTION: Qa



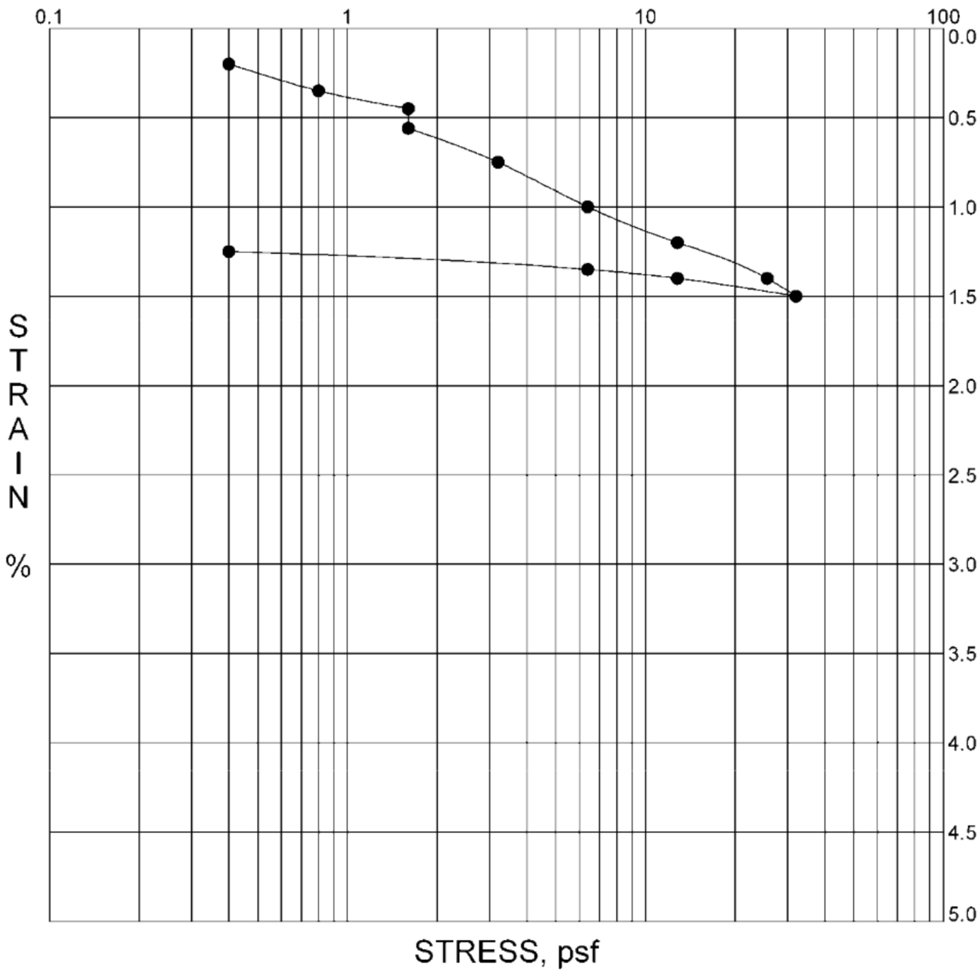
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 2.4	Dry Density: 124.0	1600 lbs.

CONSOLIDATION TEST DIAGRAM

Figure C 22

PROJECT LOCATION: 1222 S. Hope Street
PROJECT NO.: 5076
SAMPLE LOCATION: B-4 @ 10.0
DESCRIPTION: Qal



Test Results

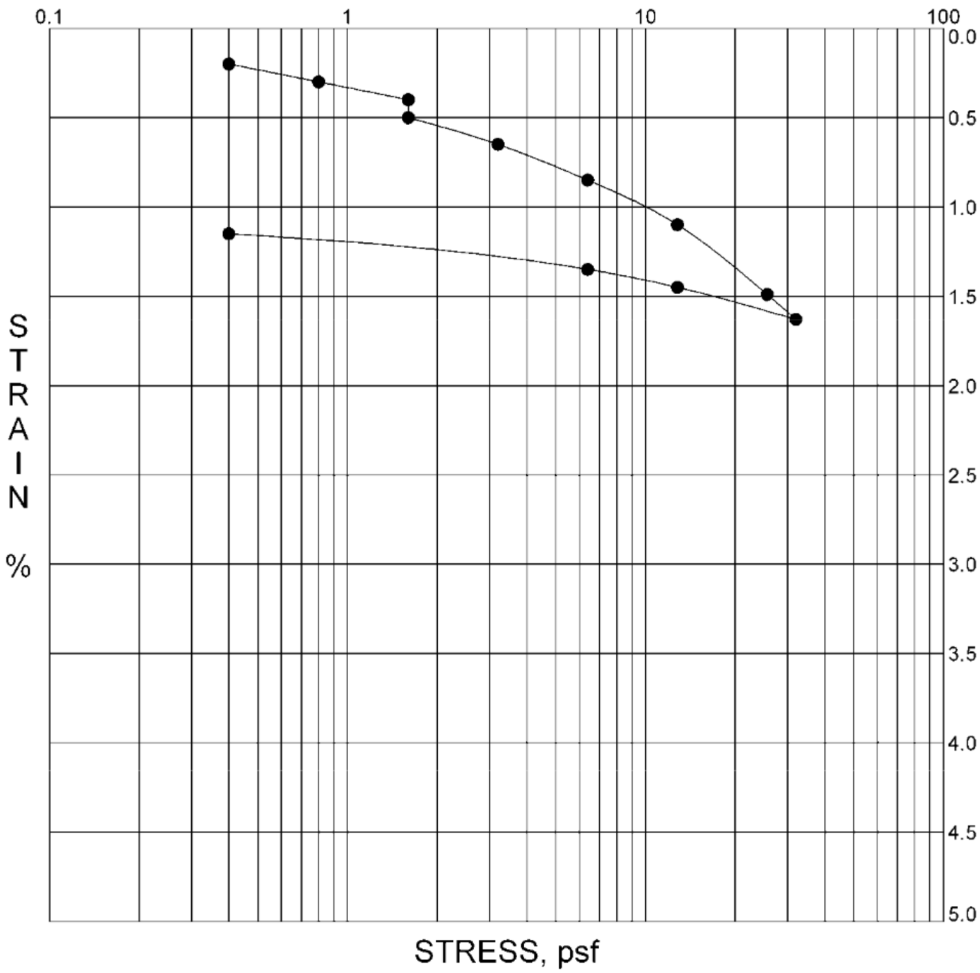
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 4.2	Dry Density: 116.8	1600 lbs.

CONSOLIDATION TEST DIAGRAM

Figure C 23

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076

SAMPLE LOCATION: B-4 @ 20.0 | DESCRIPTION:



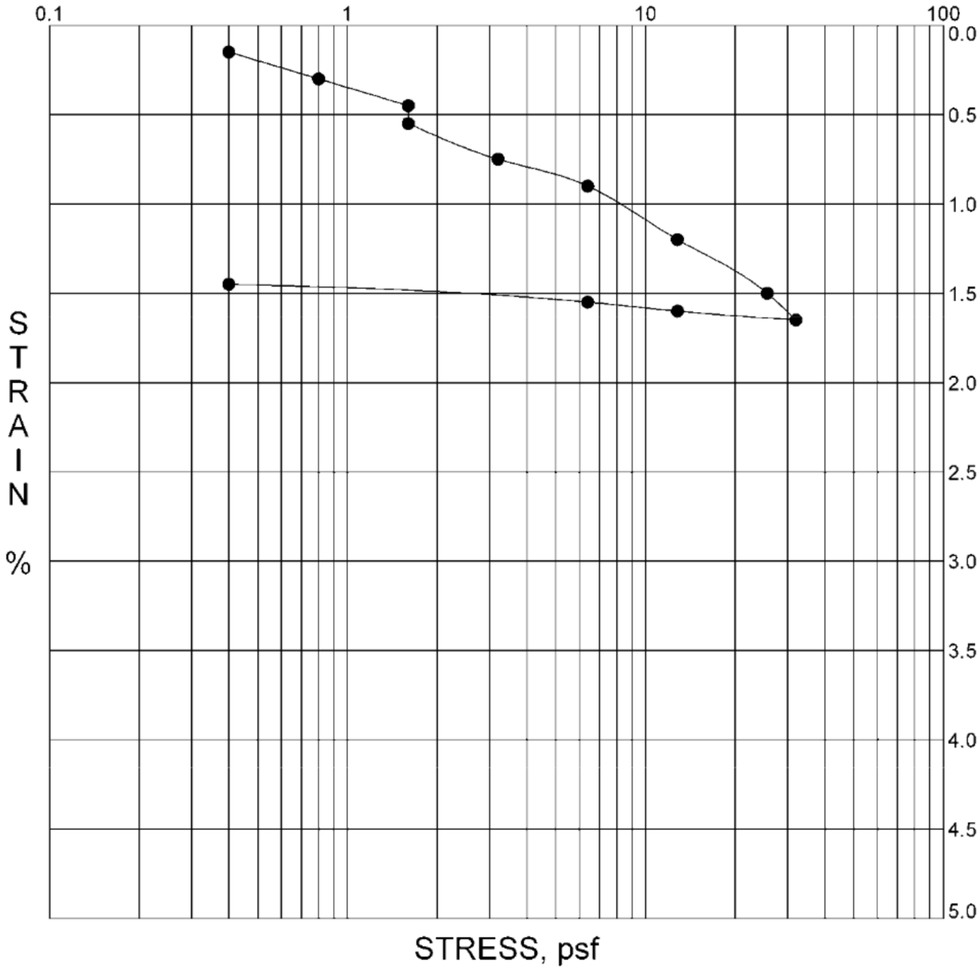
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 8.7	Dry Density: 115.2	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C 24

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076
SAMPLE LOCATION: B-4 @ 25.0 | DESCRIPTION: Qal



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 4.7	Dry Density: 114.9	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

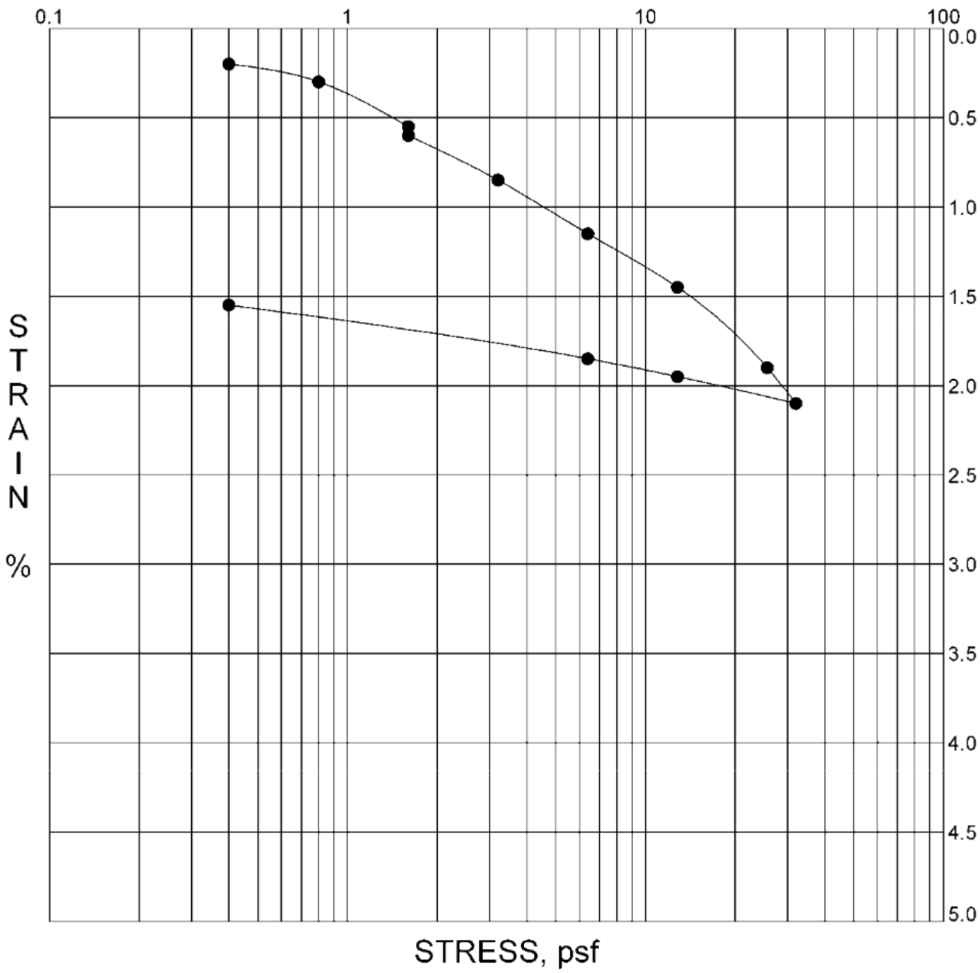
Figure C 25

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-4 @ 30.0

DESCRIPTION:



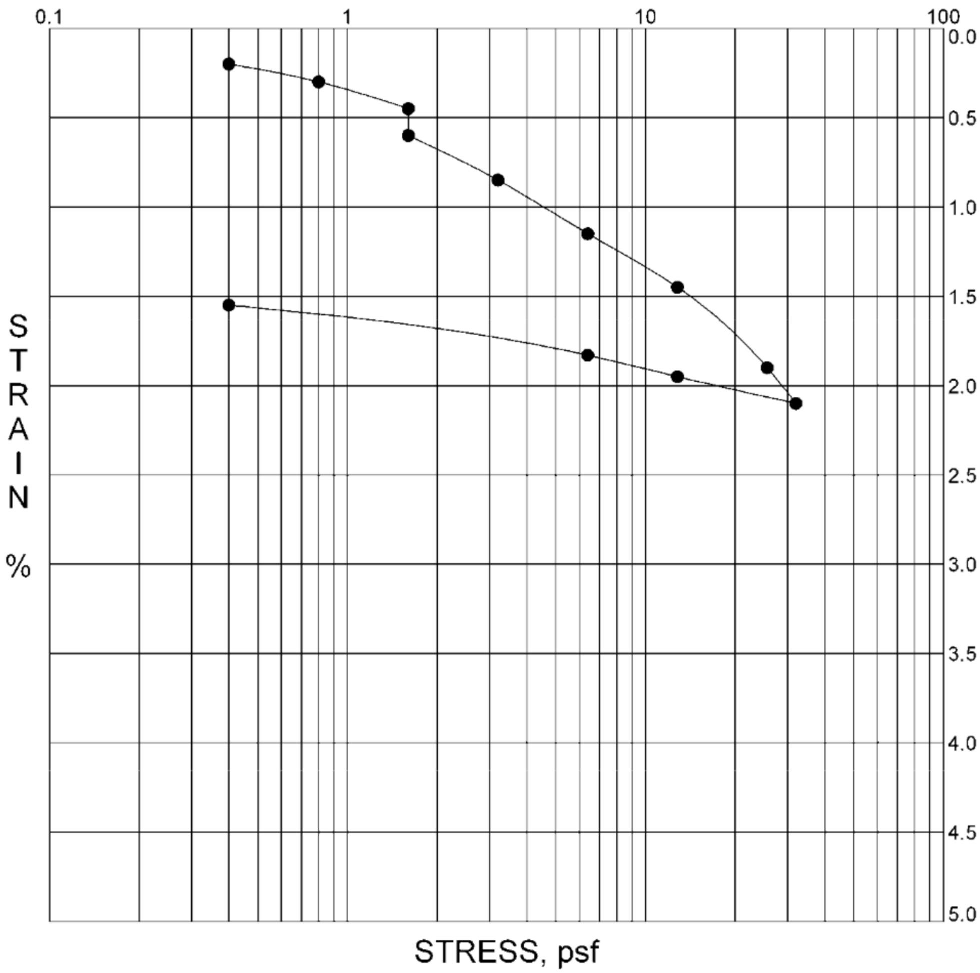
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 5.9	Dry Density: 112.8	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C 26

PROJECT LOCATION: 1222 S. Hope Street
PROJECT NO.: 5076
SAMPLE LOCATION: B-4 @ 35.0
DESCRIPTION:



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 4.1	Dry Density: 116.5	1600 lbs.

CONSOLIDATION TEST DIAGRAM

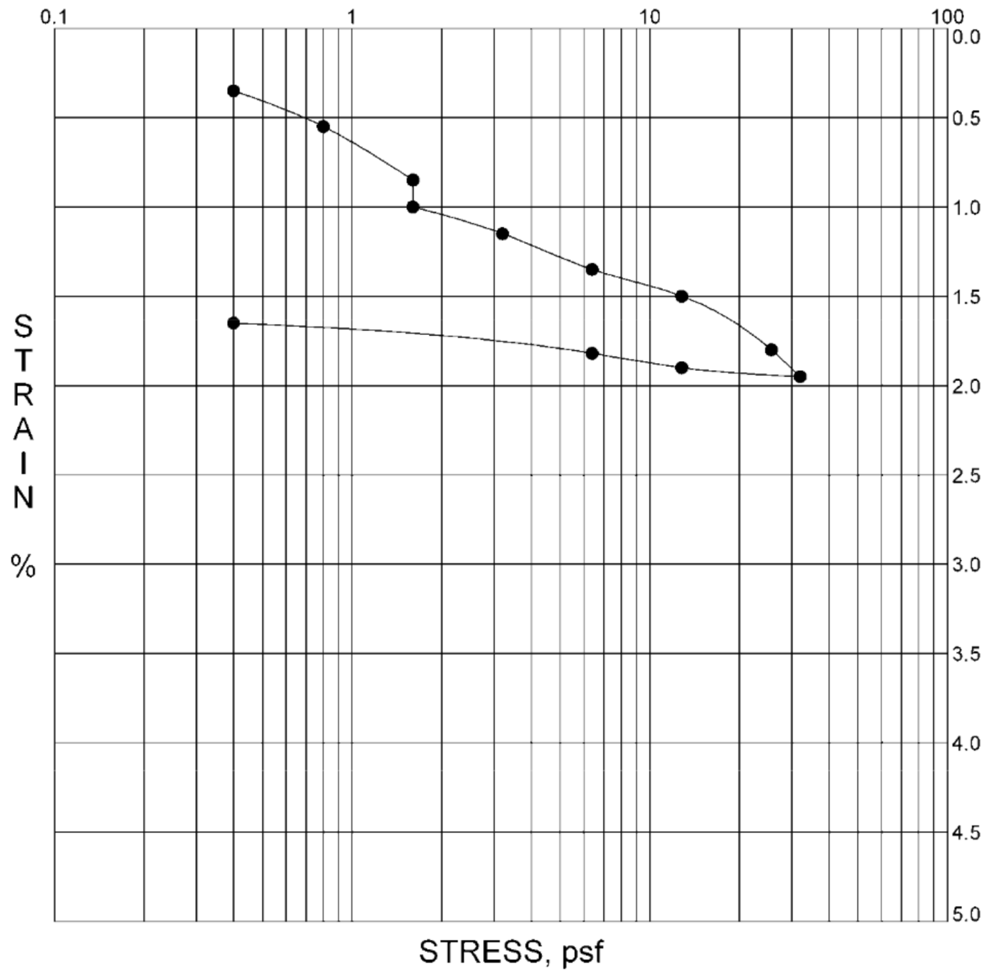
Figure C 27

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-4 @ 45.0

DESCRIPTION:



Test Results

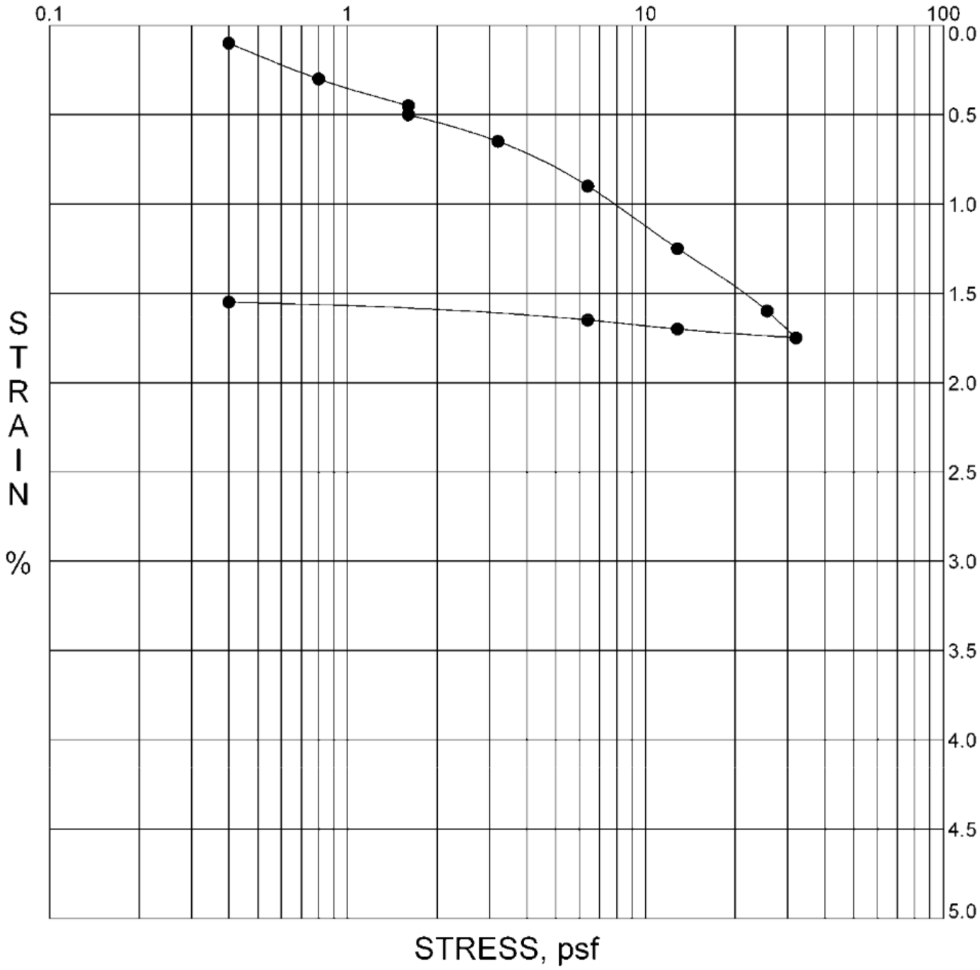
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 5.7	Dry Density: 117.3	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C 28

PROJECT LOCATION: 1222 S. Hope Street | PROJECT NO.: 5076

SAMPLE LOCATION: B-4 @ 55.0 | DESCRIPTION:



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 12.3	Dry Density: 107.1	1600 lbs.

CONSOLIDATION TEST DIAGRAM

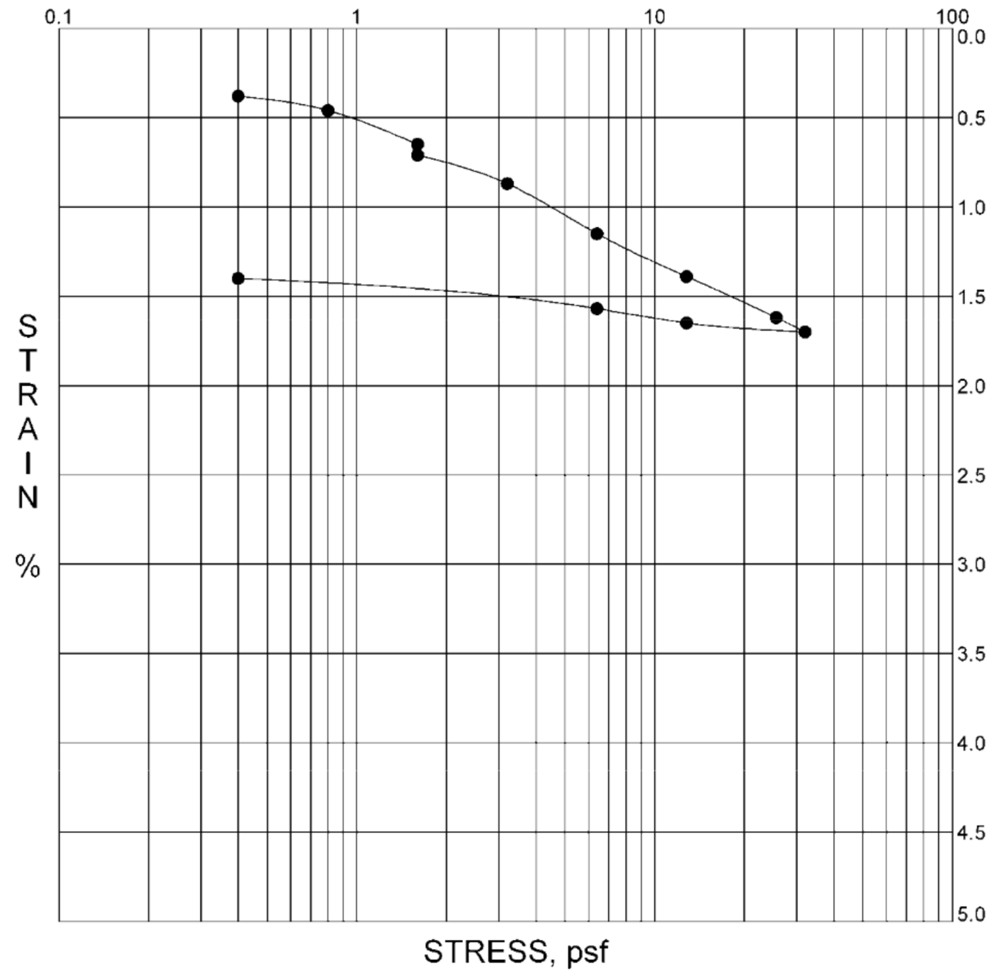
Figure C 29

PROJECT LOCATION: 1222 S. Hope Street

PROJECT NO.: 5076

SAMPLE LOCATION: B-4 @ 65.0

DESCRIPTION:



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 14.4	Dry Density: 113.7	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C 30

APPENDIX III

ANALYSES

Bearing Capacity

Lateral Design

Slope Stability

Seismic Evaluation

BEARING CAPACITY ANALYSIS

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	EMBEDMENT DEPTH:	2 feet
SHEAR DIAGRAM:	0	PAD LENGTH:	2 feet
COHESION:	50 psf	PAD WIDTH:	2 feet
PHI ANGLE:	35 degrees	SLOPE ANGLE:	0 degrees
DENSITY:	120 pcf	PAD INCLINATION:	0 degrees
SAFETY FACTOR:	4		
FOOTING TYPE:	P Pad		

CALCULATED RESULTS

HANSEN'S SHAPE, DEPTH, AND INCLINATION FACTORS

Nq =	33.30	Dq =	1.25	Sy =	0.60
Nc =	46.12	Gc =	1.00	Dy =	1.00
Ny =	33.92	Bc =	1.00	Iy =	1.00
Sc =	1.72	Iq =	1.00	Gy =	1.00
Sq =	1.70	Ic =	1.00	Gq =	1.00
Dc =	1.40	Bq =	1.00	By =	1.00

CALCULATED ULTIMATE BEARING CAPACITY (Qult)	25,047.9 pounds
ALLOWABLE BEARING CAPACITY (Qa = Qult / fs)	6,262.0 pounds
PERCENT INCREASE FOR EMBEDMENT DEPTH	33.6%

BEARING CAPACITY ANALYSIS

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	EMBEDMENT DEPTH:	2 feet
SHEAR DIAGRAM:	0	FOOTING LENGTH:	100 feet
COHESION:	50 psf	FOOTING WIDTH:	2 feet
PHI ANGLE:	35 degrees	SLOPE ANGLE:	0 degrees
DENSITY:	120 pcf	FOOTING INCLINATION:	0 degrees
SAFETY FACTOR:	4		
FOOTING TYPE:	S Strip		

CALCULATED RESULTS

HANSEN'S SHAPE, DEPTH, AND INCLINATION FACTORS

Nq =	33.30	Dq =	1.25	Sy =	0.99
Nc =	46.12	Gc =	1.00	Dy =	1.00
Ny =	33.92	Bc =	1.00	Iy =	1.00
Sc =	1.01	Iq =	1.00	Gy =	1.00
Sq =	1.01	Ic =	1.00	Gq =	1.00
Dc =	1.40	Bq =	1.00	By =	1.00

CALCULATED ULTIMATE BEARING CAPACITY (Qult)	17,479.6 pounds
ALLOWABLE BEARING CAPACITY (Qa = Qult / fs)	4,369.9 pounds
PERCENT INCREASE FOR EMBEDMENT DEPTH	28.7%

PASSIVE EARTH PRESSURE

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	SAFETY FACTOR (fs):	1.5
SHEAR DIAGRAM:	0	INITIAL SEARCH DEPTH:	1
COHESION:	50 psf	FINAL SEARCH DEPTH:	10
PHI ANGLE:	35 degrees	LIMIT PASSIVE (Y OR N):	Y
DENSITY:	120 pcf	MAXIMUM PASSIVE:	100,000.0 pounds
		Cd (C/fs):	33.3 psf
		PhiD = atan(tan(phi)/fs) =	25.0 degrees

FOOTING DEPTH (feet)	TOTAL PASSIVE FORCE Pp (pounds)	PASSIVE EARTH PRESSURE AT DEPTH - SigmaP (psf)	INCREASE IN PASSIVE EARTH PRESSURE WITH EMBEDMENT DEPTH (psf/f)
1	252.7	400.6	400.6
2	801.3	696.6	295.9
3	1,645.8	992.5	295.9
4	2,786.3	1,288.4	295.9
5	4,222.7	1,584.4	295.9
6	5,955.0	1,880.3	295.9
7	7,983.3	2,176.2	295.9
8	10,307.5	2,472.2	295.9
9	12,927.7	2,768.1	295.9
10	15,843.7	3,064.1	295.9

RETAINING WALL

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	WALL HEIGHT	15 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	50 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	135 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	33.3 psf	FINAL TENSION CRACK:	40 feet
PHID = ATAN(TAN(PHI)/FS) =	25.0 degrees		
HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h)		0 %g	
VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v)		0 %g	

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	58 degrees
AREA OF TRIAL FAILURE WEDGE	70.2 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	9475.2 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	17.0 feet
DEPTH OF TENSION CRACK	0.6 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	9.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**5536.2 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**49.2 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**50.0 pcf**

RETAINING WALL

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	WALL HEIGHT	15 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	50 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	135 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	50.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 35.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.29 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	50 degrees
AREA OF TRIAL FAILURE WEDGE	94.2 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	12716.2 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	18.7 feet
DEPTH OF TENSION CRACK	0.7 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	12.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**6303.4 pounds**

RETAINING WALL

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	WALL HEIGHT	35 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	50 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	135 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	33.3 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 25.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	57 degrees
AREA OF TRIAL FAILURE WEDGE	397.4 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	53642.6 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	40.4 feet
DEPTH OF TENSION CRACK	1.1 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	22.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**32050.8 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**52.3 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**53.0 pcf**

RETAINING WALL

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	WALL HEIGHT	35 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	50 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	135 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	50.0 psf	FINAL TENSION CRACK:	40 feet
PHID = ATAN(TAN(PHI)/FS) =	35.0 degrees		
HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h)	0.29 %g		
VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v)	0 %g		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	49 degrees
AREA OF TRIAL FAILURE WEDGE	532.3 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	71865.1 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	45.7 feet
DEPTH OF TENSION CRACK	0.5 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	30.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**36828.6 pounds**

RETAINING WALL

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	WALL HEIGHT	45 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	50 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	135 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	33.3 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 25.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	57 degrees
AREA OF TRIAL FAILURE WEDGE	657.5 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	88760.7 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	53.2 feet
DEPTH OF TENSION CRACK	0.3 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	29.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**53517.5 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**52.9 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**53.0 pcf**

RETAINING WALL

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	WALL HEIGHT	45 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	50 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	135 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	50.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 35.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.29 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	49 degrees
AREA OF TRIAL FAILURE WEDGE	879.4 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	118723.6 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	57.9 feet
DEPTH OF TENSION CRACK	1.3 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	38.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**61586.0 pounds**

SHORING PILE

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	RETAINED LENGTH	15 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	50 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	120 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	40.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 29.3 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	60 degrees
AREA OF TRIAL FAILURE WEDGE	64.6 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	7748.9 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	16.0 feet
DEPTH OF TENSION CRACK	1.1 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	8.0 feet
CALCULATED THRUST ON PILE	**3959.3 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**35.2 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**36.0 pcf**

SHORING PILE

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	RETAINED LENGTH	35 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	50 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	120 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	40.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 29.3 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	59 degrees
AREA OF TRIAL FAILURE WEDGE	367.1 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	44057.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	38.8 feet
DEPTH OF TENSION CRACK	1.7 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	20.0 feet
CALCULATED THRUST ON PILE	**23613.8 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**38.6 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**39.0 pcf**

SHORING PILE

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qa	RETAINED LENGTH	45 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	50 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	120 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	40.0 psf	FINAL TENSION CRACK:	40 feet

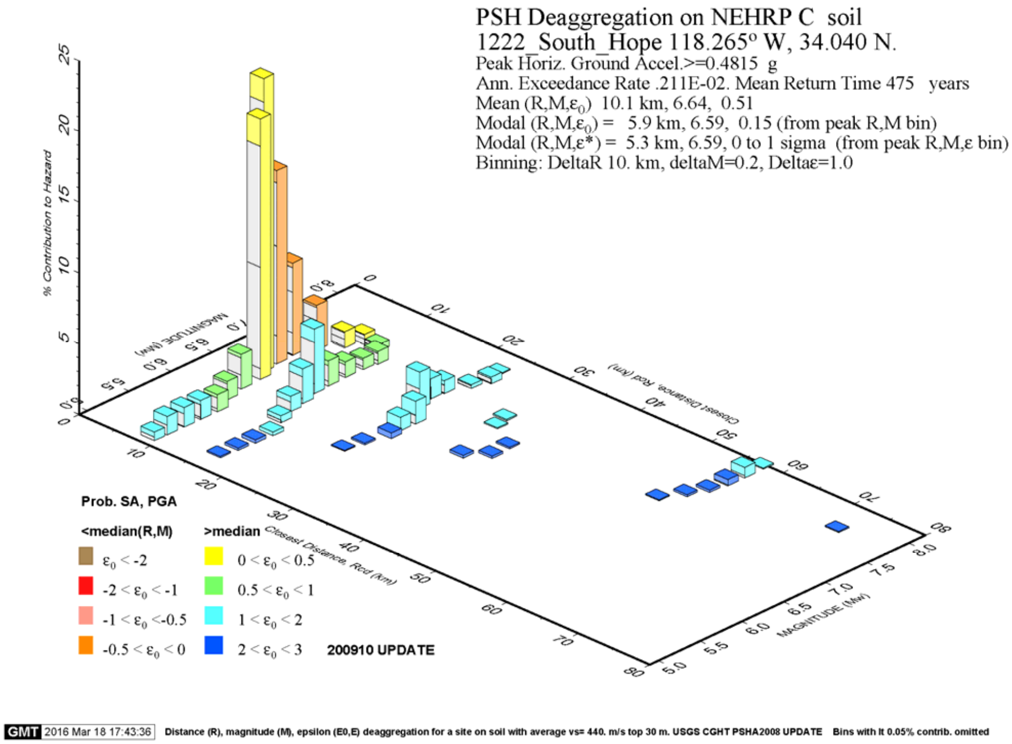
PHID = ATAN(TAN(PHI)/FS) = 29.3 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	60 degrees
AREA OF TRIAL FAILURE WEDGE	583.7 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	70048.1 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	50.0 feet
DEPTH OF TENSION CRACK	1.7 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	25.0 feet
CALCULATED THRUST ON PILE	**39634.0 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**39.1 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**40.0 pcf**


PSH Deaggregation on NEHRP C soil
1222_South_Hope 118.265° W, 34.040 N.
Peak Horiz. Ground Accel.>=0.4815 g
Ann. Exceedance Rate .211E-02. Mean Return Time 475 years
Mean (R,M,ε0) 10.1 km, 6.64, 0.51
Modal (R,M,ε0) = 5.9 km, 6.59, 0.15 (from peak R,M bin)
Modal (R,M,ε*) = 5.3 km, 6.59, 0 to 1 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
Closest Distance, Rcd (km)
MAGNITUDE (Mw)
Prob. SA, PGA
<median(R,M)
>median
ε0 < -2
-2 < ε0 < -1
-1 < ε0 <-0.5
-0.5 < ε0 < 0
0 < ε0 < 0.5
0.5 < ε0 < 1
1 < ε0 < 2
2 < ε0 < 3
200910 UPDATE
GMT 2016 Mar 18 17:43:36
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 440. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.04035°N, 118.26482°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

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

From Figure 22-1 [1]	S_S = 2.285 g
From Figure 22-2 [2]	S_1 = 0.804 g

Section 11.4.2 — Site Class

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

Table 20.3–1 Site Classification

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

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

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

Table 11.4–1: Site Coefficient F_a

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

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

For Site Class = C and S_s = 2.285 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

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

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

For Site Class = C and S_1 = 0.804 g, F_v = 1.300

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 2.285 = 2.285$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.300 \times 0.804 = 1.045$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

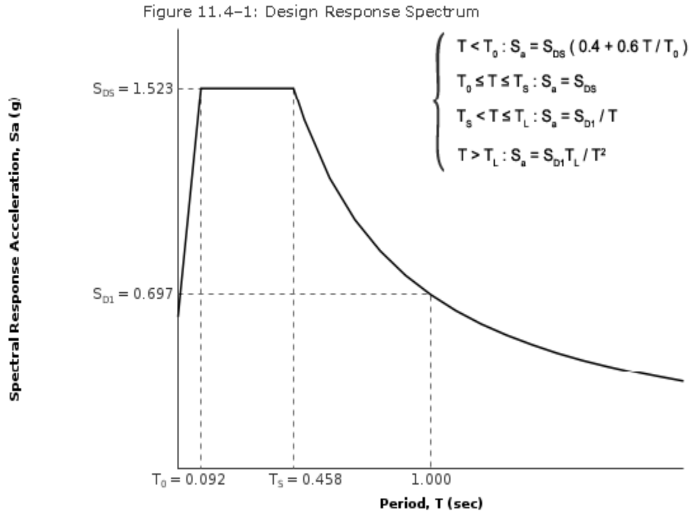
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.285 = 1.523$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.045 = 0.697$ g

Section 11.4.5 — Design Response Spectrum

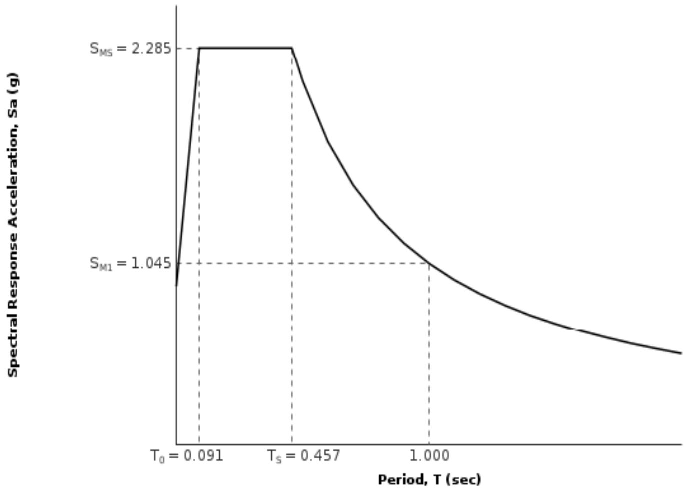
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



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

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



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

From Figure 22-7 [4] PGA = 0.852

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.852 = 0.852$ g

Table 11.8–1: Site Coefficient F_{PGA}

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

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

For Site Class = C and PGA = 0.852 g, F_{PGA} = 1.000

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

From Figure 22-17 [5] C_{RS} = 0.957

From Figure 22-18 [6] C_{R1} = 0.973

Section 11.6 — Seismic Design Category

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

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

For Risk Category = I and S_{DS} = 1.523 g, Seismic Design Category = D

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

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

For Risk Category = I and S_{D1} = 0.697 g, Seismic Design Category = D

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

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

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

References

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

APPENDIX IV

REFERENCES

1. Abramson, Lee W., et. al., Slope Stability and Stabilization Methods (Wiley & Sons, New York: 1996).
2. Bowles, Joseph, E., Foundation Analysis and Design (McGraw-Hill, New York: 1988).
3. 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.
4. Lamar, D. L., 1991, Geology of the Elysian Park-Repetto Hills Area, Los Angeles County California: California Division of mines and geology, Special Report 101.
5. Hoots, H. W., 1930, Geology of the eastern part of the Santa Monica Mountains, Los Angeles County, California: U. S. Geological Survey, Professional Paper 165-C.
6. Monahan, Edward J., PE, Construction of and on Compacted Fills (Wiley & Sons, New York: 1986).
7. Naval Facilities Engineering Command Foundations and Earth Structures - Design Manual 7.02 (Naval Publications and Forms Center, Philadelphia: 1986).
8. Northridge Earthquake January 17, 1994, preliminary reconnaissance report: Earthquake Engineering Research Institute, March 1994.
9. Poulos, H. G., and Davis, E. H., Pile Foundation Analysis and Design (Wiley & Sons, New York: 1980).
10. Taylor, Donald W., Fundamentals of Soil Mechanics (Wiley & Sons, New York: 1948).
11. Terzaghi, Karl, Peck, Ralph B., Mesri, Gholamreza, Soil Mechanics in Engineering Practice (Wiley & Sons, New York: 1996).